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Spacing of connections in compression elements for cold-formed steel members (beams)

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Civil Engineering Study 97-6
Cold-Formed Steel Series

Summary Report

SPACING OF CONNECTIONS IN COMPRESSION ELEMENTS FOR COLD-FORMED STEEL MEMBERS (BEAMS)

by

Michael L. Jones
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December 1997

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PREFACE

The spacing of connectors in compression elements of built-up flexural members was evaluated both experimentally and analytically at the University of Missouri-Rolla (UMR) under the sponsorship of the American Iron and Steel Institute. A computational model was developed for determining the bending capacity of single-fluted sections with cover plates that do not have edge stiffeners. The results of the investigation are presented and recommendations are made for design of single-flute cross sections having cover plates without edge stiffeners.

Previous research developments by Yener, at Purdue University, and Luttrell, at the University of West Virginia, were compared to the UMR experimental data, which consisted of 83 single flute built-up hat sections with and without edge stiffened cover plates. All hat sections were tested as simple span beams with cover plates in compression. The buckling behavior of edge stiffened cover plates and cover plates without edge stiffeners was investigated and discussed herein. The effective length factor and plate buckling coefficient were determined from the test results. A discussion of the correlation between the UMR experimental findings and the work of both Luttrell and Yener is presented. The UMR research validates the AISI Design Specification spacing criteria (Section D1.2) which restricts spacing to a value that will prevent any separation of the cover plate from the hat section between the connectors. This provision, however, is very conservative when applied to a section in bending because it yields very small connector spacings. The spacing of the connectors may be increased beyond that required by Section D1.2 which results in a reduction of strength. Tests have indicated that the capacity of the hat section is not diminished due to the onset of plate buckling in the cover plate. The hat section will continue to carry additional load because of the post-buckling strength provided by the cover plate.

This report is based on a thesis submitted to the Faculty of the Graduate School of the University of Missouri-Rolla in partial fulfillment of the requirements for the degree of Masters of Science in Civil Engineering.

Technical guidance for this investigation was provided by the American Iron and Steel Institute's Subcommittee on Flexural Members (J.N. Nunnery, Chairman) and Connections (M.Golovin, Chairman). The Subcommittees' guidance is gratefully acknowledged. Thanks are also extended to H. H. Chen, D. F. Boring and S.P. Bridgewater, AISI staff, for their assistance.

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LIST OF SYMBOLS

D	Total depth of section including cover plate (Figure 7)
d	Width of flange on hat section (Figure 7)
d_c	Width of stiffener on edge stiffened cover sheet (Figure 7)
E	Modulus of elasticity of steel, 29,500 ksi
f_c	Stress at service load in cover plate
f_{cr}	Local buckling stress of a plate
F_y	Yield point used for design
F_u	Tensile strength of steel
k	Plate buckling coefficient
k_c	Effective length factor
M_{cr}	Bending moment at cover plate buckling stress level
R	Inside bend radius
r	Radius of gyration of cover plate
s_m	Maximum spacing allowed by AISI Design Specification Section D1.2 Criterion #2[2]
s_t	Tested connector spacing
t	Base steel thickness of any element of section
w	Width between adjacent lines of connectors (Figures 6 and 7)
w_c	Total width of edge stiffened cover plate (Figure 7)
w_h	Total width of hat section (Figures 6 and 7)
w_u	width of unstiffened or partially stiffened edge of cover plate outside the connection line

λ_t	Transitions stress slenderness factor
ρ	reduction factor
ρ_m	reduction factor
ρ_t	Transition stress reduction factor
σ_{cr}	Euler elastic column buckling stress

I. INTRODUCTION

A. GENERAL

The use of cold-formed steel in building construction began in the 1930's[1], but it was not until the development of the Specification for Design of Cold-Formed Steel Structural Members[2] by the American Iron and Steel Institute in 1946 that cold-formed steel became more widely accepted. Today many structures, from residential to commercial buildings, employ a wide variety of cold-formed steel members. With the development of built-up steel sections, greater economy can be achieved in building construction. For example, the use of the composite floor system has led to the use of closed cellular decks, consisting of a fluted or "hat-shaped" deck with a flat bottom sheet attached together most commonly by welds. This floor system has advantages over the non-cellular floor systems by providing open channels or raceways for the distribution of electrical conduit, or even as heating and air conditioning duct[1].

The behavior of built-up sections with connectors in compression elements is quite complicated. Because of the compressive forces the flat areas between and outside the connectors are susceptible to plate and column-like buckling. Current provisions in the 1996 AISI Design Specification[2] use a conservative approach which limits the spacing of connections in compression elements to a value that does not allow column-like buckling of the flat sheet between the connectors, or buckling of the unstiffened outside edge of the flat sheet. However, it is well known that buckling of the sheet does not immediately cause failure of the deck section. This increased strength occurs because of a redistribution of stress (post-buckling strength) allowing the deck to carry a much greater load[1].

B. PURPOSE OF INVESTIGATION

The purpose of this investigation was to study the required spacing of connections in compression elements of built-up members and to develop an improved design criteria for cold-formed steel design. The study focused on cellular deck type cross-sections with the cover plate in compression. The current AISI Design Specification[2] criteria are also applicable to other types of built-up compression members.

C. SCOPE OF INVESTIGATION

This study involved three phases: a literature survey, experimental and analytical investigations, and development of design criteria. The literature survey involved collecting and evaluating all available publications and test data on built-up sections with intermittent connections in compression elements. The second phase of the study involved an analytical and experimental investigation where consideration was given to factors such as column-like buckling and plate-like buckling of the stiffened element between the connectors as affected by the spacing of the connectors, and buckling of the free or partially stiffened edge outside the connection line. The final phase of the study involved developing appropriate design criteria based on the literature survey and the results of the analytical and experimental investigation.

II. LITERATURE REVIEW

A. GENERAL

A literature review of all available publications on built-up sections with intermittent connectors in compression elements began at the University of Missouri-Rolla in May 1996. All available publications studied the spacing of connectors on cellular deck sections. Two researchers, Muzaffer Yener[3] and Larry Luttrell[4], studied the effects of various parameters on the spacing of connectors. Yener[3] studied the spacing of connectors on cellular deck sections, manufactured by Walcon Corporation, in positive and negative bending. Yener developed a modified spacing criteriion that is less conservative when compared to the AISI Design Specification[2]. Luttrell[4] studied the spacing of connectors on cellular deck sections, produced by Epic Metals Corporation, with the cover plates in compression only. Luttrell's work focused on the effective width of the cover plate between the connectors. His model allows the use of any spacing, and requires a further reduction in the effective width between connectors for larger spacings. The work of both researchers as well as the AISI Design Specification[2] will be discussed in detail.

B. AISI DESIGN SPECIFICATION

Current provisions in Section D1.2 of the AISI Design Specification[2] provide spacing requirements that attempt to make the flat plate act monolithically with the fluted sheet. Spacing is limited to that which is needed to develop the required shear strength, to limit column-like buckling behavior between fasteners, and to eliminate buckling of the unstiffened edge of the cover plate. When these provisions are met the cover plate between fasteners can be assumed to be a fully stiffened element of width, w, between connection lines (Figure 1).[1,5,6]

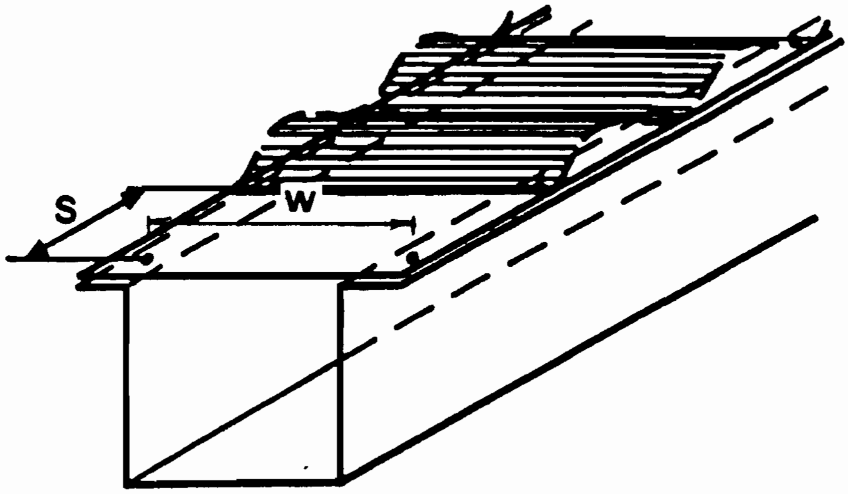


Figure 1. Spacing of Connectors in Composite Sections for Column-like Buckling

The following provisions, as stated in the 1996 AISI Design Specification[2], are given in Section D1.2 of the Specification titled Spacing of Connections in Compression Elements:

The spacing, s, in the line of stress, of welds, rivets, or bolts connecting a cover plate, sheet, or a non-integral stiffener in compression to another element shall not exceed

(a) that which is required to transmit the shear between the connected parts based on the design strength per connection

(b) 1.16t $\sqrt{(E/F_c)}$, where t is the thickness of the cover plate or sheet, and f_c is the stress at service load in the cover plate or sheet

(c) three times the flat width, w, of the narrowest unstiffened compression element tributary to the connections, but need not be less than $1.11t \sqrt{(E/F_y)}$ if $w/t<0.50 \sqrt{(E/F_y)}$,or $1.33t \sqrt{(E/F_y)}$ if $w/t \geq 0.50 \sqrt{(E/F_y)}$, unless closer spacing is required by (a) or (b) above.

Item (b) of the above criteria was developed based on the assumed failure pattern shown in Figure 1. If the spacing is close enough to prevent column-like buckling of the cover plate between the connectors and local buckling of the unstiffened or partially stiffened edge of the cover sheet, the portion of the cover plate between connector lines can be analyzed as a fully stiffened compression element of width "w". The model assumes that a strip of the compressed plate between adjacent connectors acts as a column of length s[3] (Figure 1). This strip of length, s, is analyzed as a fixed ended column using a conservative $k_c = 0.6$. The limiting spacing can be obtained by substituting $\sigma_{cr} = 1.67f_c$, $k_c = 0.6$, L = s, and $r^2 = t^2/12$ into the Euler column buckling formula and solving for s. Additional considerations must be given to the local buckling of the unstiffened or partially stiffened compression elements. Requirement (c) limits the spacing to a value such that the unstiffened edge will not buckle. This provision does not account for the post-buckling strength of the sheet.[1, 5, 6]

C. YENER'S STUDY

In 1983, Yener studied the AISI requirements of connection spacing on cellular panels under vertical loading conditions[3]. Testing involved single lap joint tests, and a series of one, two, and three-span beam tests. Thirteen simple span beam tests were performed (four with the cover sheet in compression). Four panels were tested in a three- span uniform load situation and four panels were tested in a uniform load two-span situation.[3] Yener developed a modified spacing criterion that is less conservative than the current AISI design specification criteria. Drawings of the cross-sections tested are shown in Figures 2 and 3. A summary of Yener's Test data is reproduced in Appendix A.

1. Simple Span Beam Tests. The testing program involved 13 specimens consistinof combinations of 18 gage flat and fluted sheets and 22 gage flat and fluted and fluted sheets. The specimens ranged in actual thickness between 0.0329 in. and 0.0507 in. with yield stresses varying from 36.3 ksi to 46.6 ksi. Specimen spacing was based on ultimate shear load and load at first slip from a series of 190 single-lap joint shear tests using Milford-type 530 and 541 steel rivets.

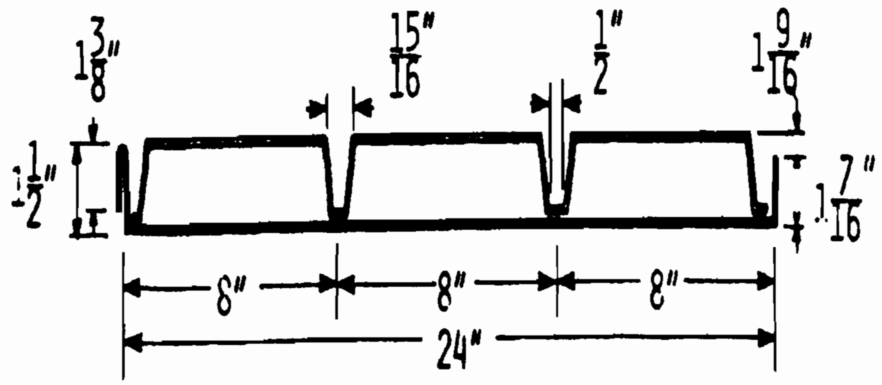


Figure 2. 24-inch NDU Panel without Stiffeners[3].

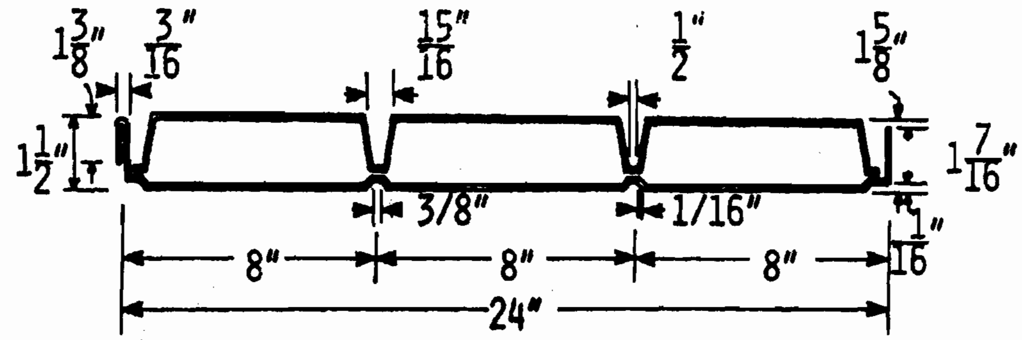


Figure 3. 24-inch NDU Panel with Stiffeners[3]

Panels were loaded in a four-point loading pattern creating a constant moment at the center of the span.[3]

2. Three-Span Beam Tests. Four panels were tested under a uniform load condition using the vacuum box method of applying the loads. Panels consisted of 18 gage flat and fluted sections combined, and 22 gage flat and fluted sections combined. Sheet thickness ranged from 0.0336 to 0.0489 inches, and the yield stress varied from 43.9 to 46.6 ksi. Two specimens were tested with the cover plate up. Connection spacing was based on the ultimate strength of the connection and varied from 4.5 to 7.0 inches. AISI requirements for the spacing varied from 1.3 to 1.5 inches for the four tests.[3]

3. Two-Span Beam Tests. Four panels were tested with the cover sheet down. All four panels were 20 gage (0.041 inches) flat sheet and 18 gage (0.0515 inches) fluted sheet with a yield stress of 52.4 ksi. Connection spacing for the test panels varied from 6 to 12 inches. AISI requirements required a 1.5 in. spacing for the four tests.

4. Spacing Recommendation. Based on Yener's findings the spacing of the connections shall be limited to the smallest value of the following three requirements:[3]

(a) Spacing shall not exceed that required to transmit the force induced by the applied loads and based on the allowable design strength of the connectors.

(b) Spacing shall not exceed that required to prevent buckling of the cover plate between the connection lines such that s = 0.6w, but not less than $133t/(F_y)^{1/2}$, where w is the width of the flat plate between the connection lines.

(c) Spacing shall not exceed that as to prevent the separation of the unstiffened compression plate element such that s = $8w_u$, but not less than $507t/(F_y)^{1/2}$, where w_u is the width of the smallest unstiffened edge of the flat plate.

5. Criteria Basis. Yener developed the above spacing criteria using the same basic approach as the AISI Design Specification[2]. Using the approach that built-up sections should be designed for maximum capacity (monolithic action) and the connectors detailed to obtain this capacity. Therefore, spacing is limited to that which is needed to develop the required shear strength (requirement a), to limit column-like buckling behavior between fasteners (requirement b), and to eliminate buckling of the unstiffened edge of the cover plate (requirement c). The plate buckling pattern shown in Figure 4 was used to develop the spacing criteria that prevents separation of the cover plate from the fluted section. Requirement b can be found based on Figure 4, using the elastic critical plate buckling stress, f_{cr}, given by Eq 1. [3] The complete derivation of this criteria can be reviewed in Reference 3.

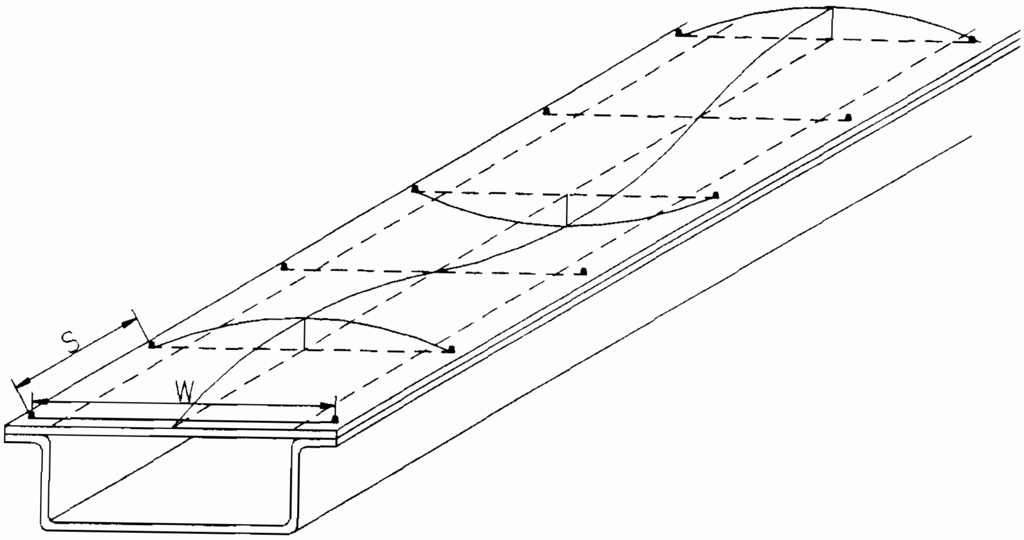


Figure 4. Plate Buckling Configuration of Built-up Sections.

$$f_{cr} = \frac{k\pi^2 E}{12(1-\nu)(\frac{w}{t})^2} \qquad (1)$$

Where:

E = Modulus of elasticity of steel, 29,500 ksi

f_{cr} = Local buckling stress of a plate

k = Plate buckling coefficient

t = Base steel thickness of cover plate

w = Width between adjacent lines of connectors (Figure 4)

6. Summary. Yener's simple span test results validate the conservative nature of the current AISI specification. Four panels were tested with the cover sheet in compression. The connection spacing on these four panels ranged from 12 to 23 inches. The AISI Design Specification[2] required a spacing of 4 inches. Each of these panels developed their full flexural capacity. It should be noted that buckling of the flat cover sheet occurred prior to failure. This buckling, however, did not hinder the development of the computed flexural capacity, $M_n = S_eF_y$.[3]

Yener's spacing criteria at onset looks very conservative as compared to his actual tested spacing based on the connection strength. The results of the simple span beam tests revealed that the AISI specification as well as Yener's own criteria, are very conservative. This conservatism is shown by specimen S11(Appendix A). A spacing of 13 in. was used in the test and full flexural capacity was developed. AISI requirements for specimen S11 was 1.5 in. spacing and Yener's spacing was 4 in. Yener does, however, note that the cover plate did buckle at ultimate load, and the resulting deflection was larger than that predicted by using only the effective section at yield stress level. Yener concludes by stating that connection spacing should be evaluated on the importance of deflection and its limitations in design.[3]

D. LUTTRELL AND BALAJI'S STUDY

The research efforts of Luttrell and Balaji[4] focused on cellular decks with cover plates in compression. The basis for this research is founded on the premise that the AISI effective width equations are not valid when column-like buckling of the flat cover plate occurs. If connections are spaced close enough, column-like buckling between connectors is prevented, i.e. $f_c < \sigma_{cr}$, where f_c = compressive stress at extreme fiber and σ_{cr} = Euler column buckling stress, allowing the use of the AISI effective width equations. When spacing increases between welds the possibility of column-like buckling between welds is increased, i.e. f_c approaches σ_{cr}. If column-like buckling between connectors occurs, the AISI effective width equations are invalidated because the connection lines can not create edge supports for the stiffened plate.[4]

The experimental study involved eighty-two panel assemblies in six different configurations. All decks were supplied by Epic Metal Corporation (Figure 5). Connector spacing on the decks was standardized at 4, 6, and 8 inches. The thickness of flat and fluted sheets varied from 0.034 to 0.0582 inches. The main parameters studied in this test program were weld spacing, sheet thickness combinations, and profile depths.[6] Specimens were tested on simple spans with flat sheets in compression and line loads applied typically at third points. End bearing was four inches[4]. The test data are included in Appendix B. Luttrell and Balaji developed a modifier to decrease the effective width of the flat plate when $f_c > \sigma_{cr}$ The tested moment capacity showed acceptable correlation with the computed moment capacity when the effective width modifier was used. However, Luttrell and Balaji failed to show a comparison between tested and computed results with respect to the AISI specification. A summary of the modified effective width equations that were developed and the associated limits are given as follows:[4]

> When $f_c < \sigma_{cr}$ the AISI effective width equations are valid for the flat sheet between the connection lines. When $f_c = \sigma_{cr}$ the flat sheet between the connection lines is at a transition stress and the transition effective width factor ρ_t is found as follows:

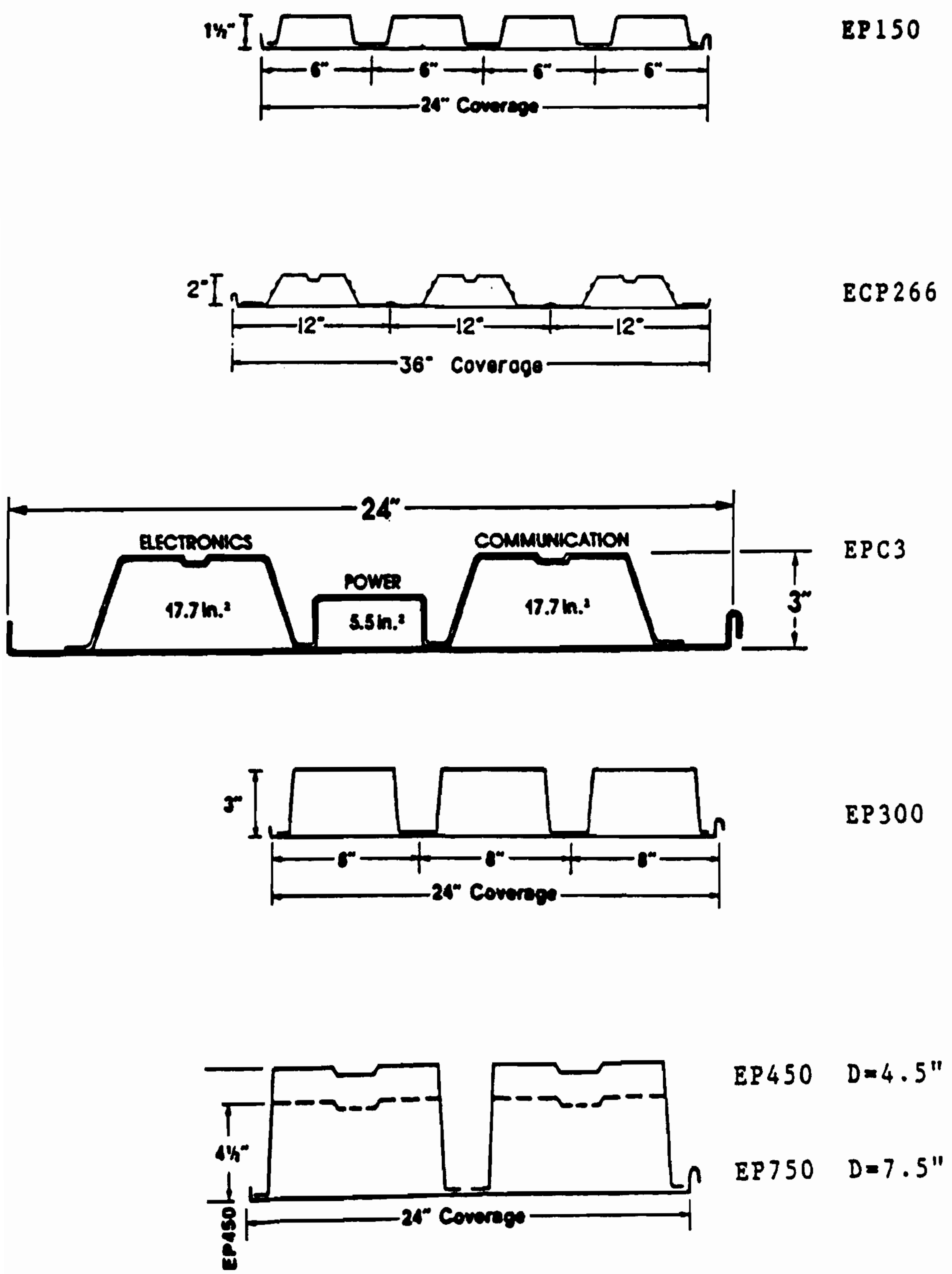


Figure 5. Deck Sections Tested by Luttrell[4]

$$\lambda_t = (\frac{1.052}{\sqrt{k}})(\frac{w}{t})\sqrt{\frac{\sigma_{cr}}{E}} \tag{2}$$

Where

$$\sigma_{cr} = \frac{\pi^2 E}{(\frac{k_c * s}{r})^2} \quad where,\ r^2 = \frac{t^2}{12}\ ,\ k_c = 0.5 \tag{3}$$

$$\rho_t = 1.0 \text{ when } \lambda_t < 0.673$$

$$\rho_t = \frac{(1.0 - \frac{0.22}{\lambda_t})}{\lambda_t} \quad when\ \lambda_t \geq 0.673 \tag{4}$$

When the value of f_c increases above the critical stress, (σ_{cr}), in the flat sheet the effective width will decrease and the final value of ρ is found as follows:

When $f_c > \sigma_{cr}$

$$\rho_m = (\frac{F_y}{f_c})\sqrt{\frac{\sigma_{cr}}{D\ f_c}} \tag{5}$$

$$\rho = \rho_m \rho_t$$

Where:

λ_t = transition stress slenderness factor

σ_{cr} = Euler elastic column buckling stress

w = flat width between connection lines (Figure 1)

t = thickness of flat sheet

k = plate buckling coefficient

k_c = column buckling effective length factor

s = fastener spacing

r = radius of gyration of cover plate

ρ_t = transition stress reduction factor

ρ_m = reduction factor

f_c = stress at service load in the cover plat or sheet

D = Overall depth of section including the cover plate

E = Modulus of elasticity of steel, 29,500 ksi

III. UMR EXPERIMENTAL STUDY

A. GENERAL

A study of built-up sections with the connectors in compression elements began at the University of Missouri-Rolla in May 1996. The purpose of this study was to gain a better understanding of the structural behavior and the parameters that affect the spacing of connectors in built-up sections. The study began with the design of hat shaped beam sections with cover plates. These specimens were tested to determine the column buckling effective length factor for the flat sheet between connectors, as well as the ultimate load capacity of the built-up section. A total of 83 full-scale beam tests were conducted.

B. TEST SPECIMENS

The sections used in this study were basic hat sections with flat cover sheets with and without edge stiffeners. The specimens were divided into four groups: h-type material without edge stiffened cover plate (Figure 6), gsh-type material without edge stiffened cover plate (Figure 6), h-type material with edge stiffened cover plate (Figure 7), and gsh-type material with edge stiffened cover plate (Figure 7). All connections were made with 3/4 inch, No.10, self-drilling screws. The initial study focused on flat cover sheets without stiffened edges and was later modified to include a limited study of sections with stiffened flat sheets. The mechanical properties of the materials were determined by performing tensile tests on coupons cut from the unformed flat sheets. The specimens were tested following the guidelines outlined in ASTM A370, Standard Methods and Definitions for Mechanical Testing of Steel Products.[9] Table I lists the mechanical properties of the steel sheet. Typical stress strain curves for the h-type and gsh-type material is provided in Appendix G.

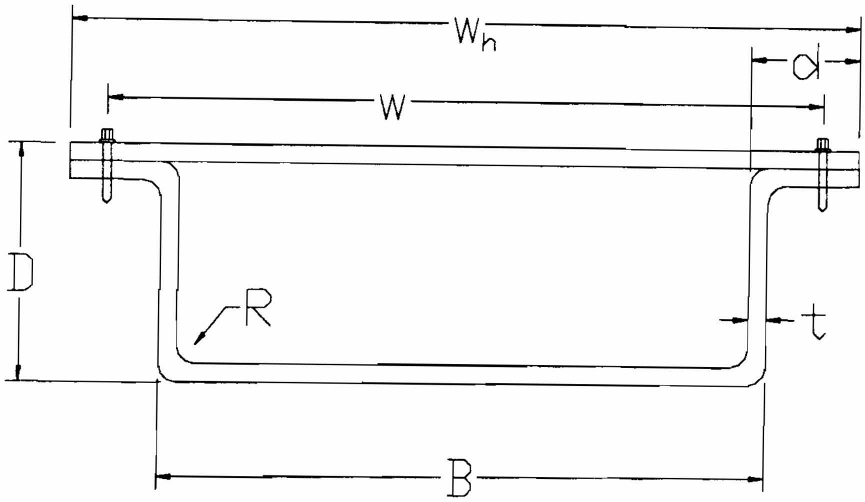


Figure 6. Section without Edge Stiffened Cover Plate.

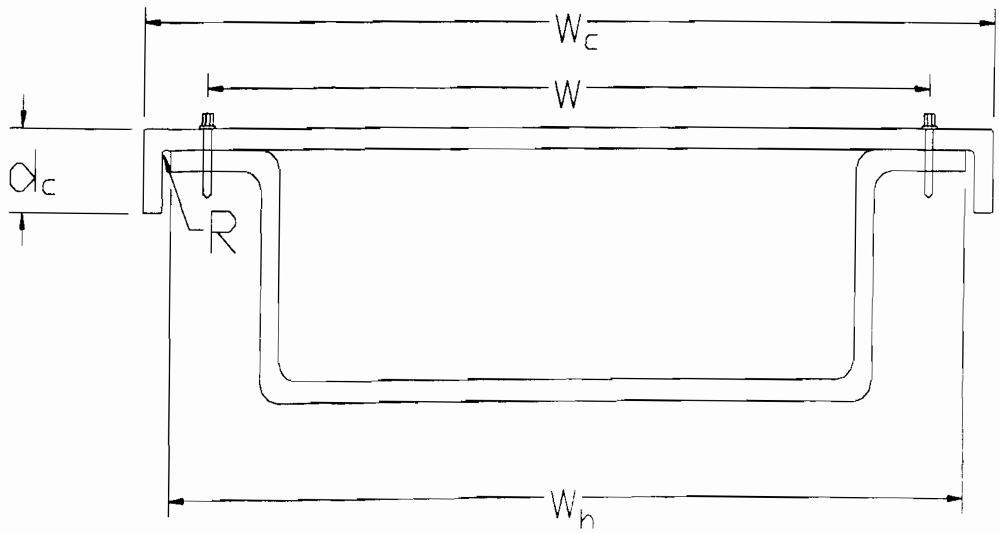


Figure 7. Section with Edge Stiffened Cover Plate.

Table I. Mechanical Properties of Tested Steel

Section Type	Gage #	t (in.)	F_y (ksi)	F_u (ksi)	% elongation in two inch gage length
hat-type	18	0.0452	33	52	45
galv sheet-type	26	0.0174	53	66	24

Specimens used in the study were designed to determine the effects of the following parameters: yield strength, F_y, thickness, t, spacing of connectors, s, w/t ratio of the flat sheet between the connection lines, depth of the section, D, width of the flange on the hat section, d, and the width of partially stiffened and unstiffened edges on the flat cover plates. Figures 6 and 7 show the sections with edged stiffened cover plates and sections without edge stiffened cover plates. Table II lists the dimensions of the sections used in the test program.

C. <u>SPECIMEN FABRICATION</u>

The fabrication process involved the placement of strain gages on selected specimens and the attachment of the flat sheet to the hat section using 3/4 inch, No. 10, self- drilling screws. The strain gages used on all test specimens were 120 Ω resistance gages, produced by Micro-Measurements Division, Measurements Group Inc. The gages were attached to the specimens using M-line accessories following the manufactures recommended procedures and guidelines. Figures 8 and 9 show the typical placement of the strain gages. Gages were placed on the top and bottom sides of the cover plate. C clamps were used in the fabrication process to hold the flat sheet and hat section together while attaching with self-drilling screws. The use of the clamps decreased the

Table II. Cross Section Dimensions.

Section Type	Sheet Gage #	F_y (ksi)	D (in.)	B (in.)	t (in.)	w_h (in.)	R (in.)	d (in.)	w_c (in.)	d_c (in.)
h1	18	33	2.0	3.1	0.0452	4.0	0.0625	0.5	-----	-----
h2	18	33	2.0	2.6	0.0452	4.6	0.0625	1.0	5.00	0.63
h3	18	33	3.0	5.8	0.0452	6.7	0.0625	0.5	6.50	0.63
h4	18	33	3.0	8.5	0.0452	9.4	0.0625	0.5	-----	-----
h5	18	33	3.0	8.0	0.0452	9.9	0.0625	1.0	-----	-----
gsh1	26	53	1.5	2.5	0.0174	3.5	0.0625	0.5	4.00	0.63
gsh2	26	53	1.5	2.0	0.0174	4.0	0.0625	1.0	4.50	0.63
gsh3	26	53	2.0	4.0	0.0174	5.0	0.0625	0.5	5.38	0.63
gsh4	26	53	2.0	3.5	0.0174	5.5	0.0625	1.0	7.63	0.63

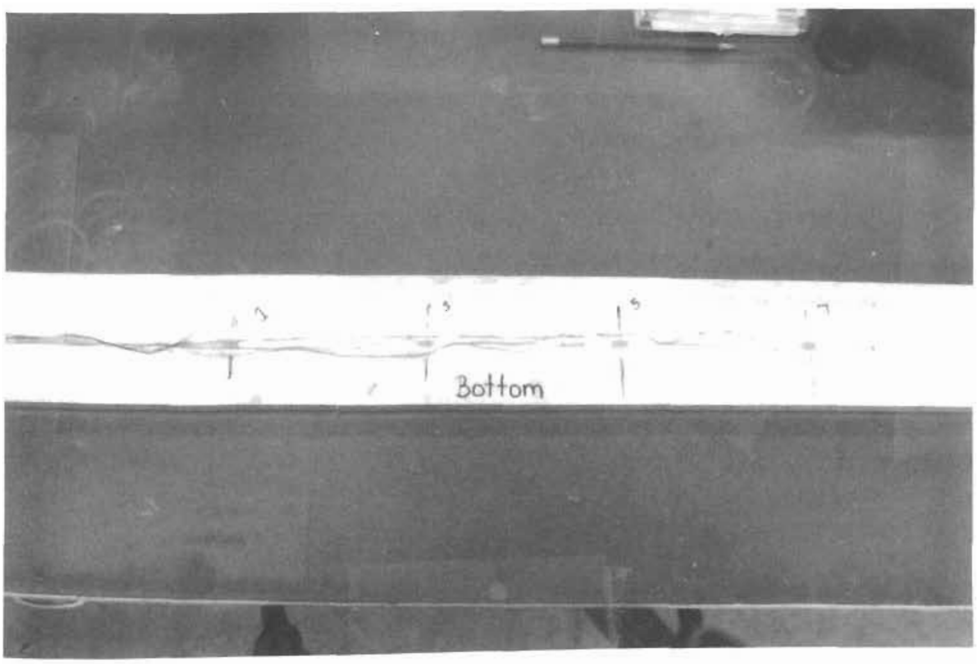


Figure 8. Section gsh3, Top and Bottom View of Flat Plate.

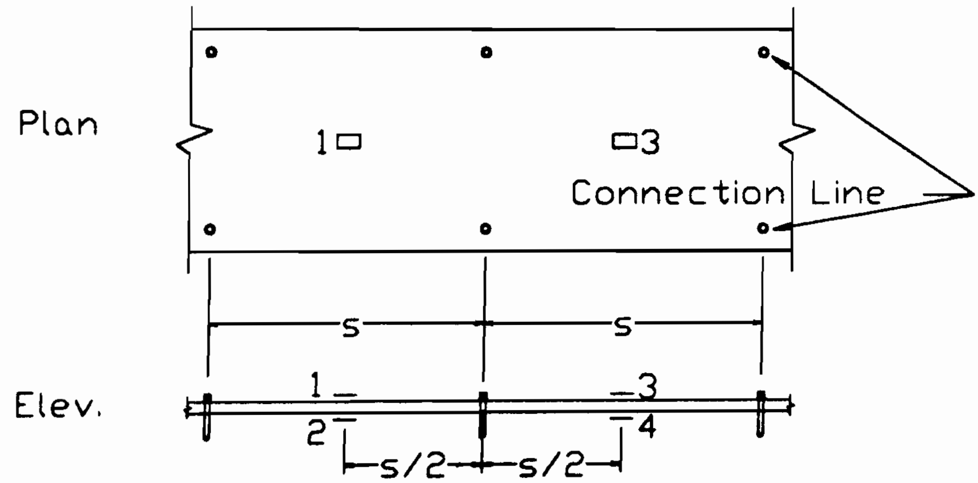


Figure 9. Location of Strain Gages on Flat Plate

possibility of an initial buckle between connectors during fabrication. However, the clamps did not eliminate all imperfections. The effects of this initial buckle will be discussed later in Section IV-C.

D. TEST SETUP

The test program involved two different test setups. Test setup #1, shown in Figure 10, was used on all but four specimens. The overall length of each specimen was 60 inches. Three inch wide bearing plates were used at all loading points. The actual distance between bearing plates for the two-point load varied depending on the spacing used on the specimen (Table III). Connector spacing was adjusted such that the bearing plate and screw connection would not coincide, as shown in Figure 11.

Test Setup #2, Figure 12, was used to determine the effects of a moment gradient on connector spacing. The spacing of connectors on the specimens were adjusted such that the

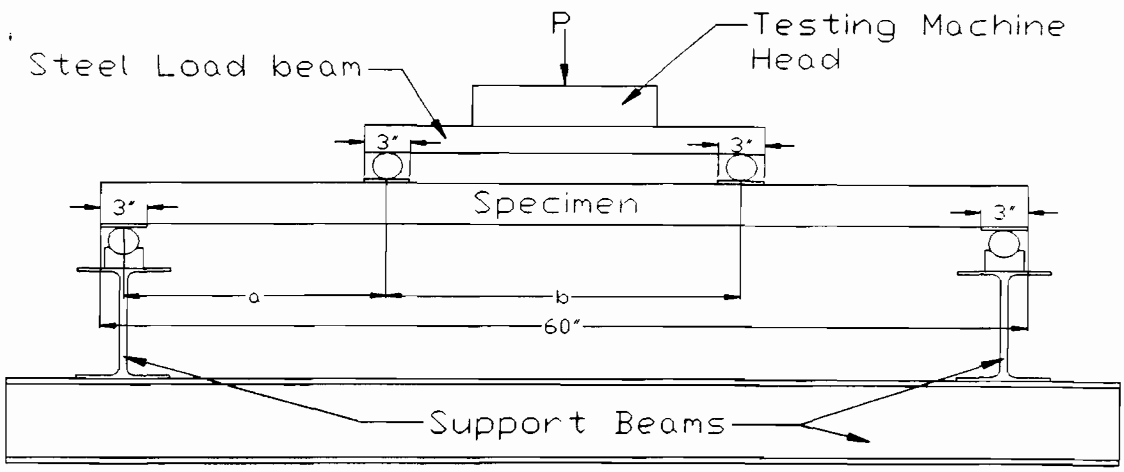


Figure 10. Test Setup #1, Two Point Loading.

Table III. Distances a & b on Figure 10.

S (inches)	a (inches)	b (inches)
1.5	17.0	23
3.0	17.0	23
3.5	16.0	25
4.0	16.5	24
6.0	14.5	28

bearing plates did not coincide with screw connectors. The screws were placed the same distance away from the bearing plates as in Test Setup #1, shown in Figure 11. The only variations used in test setup #1 was the employment of a load cell to record load reading on the gsh-type material and all tests in which strain gages were used. The load cell was fastened

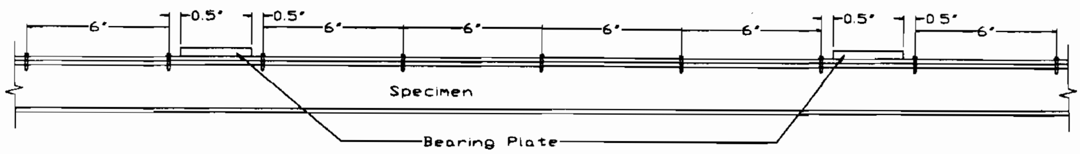


Figure 11. Spacing of Connections around Bearing Plate.

to the testing machine head. Figure 13 shows a picture of this arrangement. Use of the load cell was necessitated to increase the accuracy of the measured applied loads because of the small failure load of the gsh-type material.

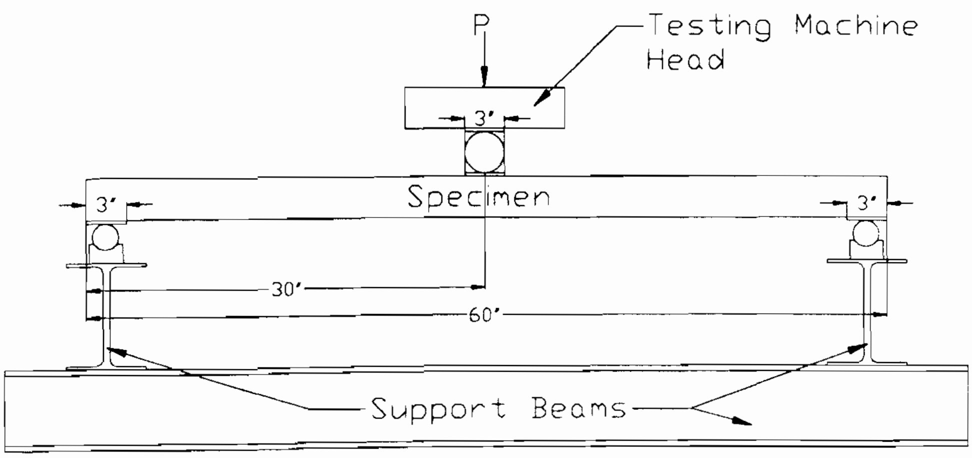


Figure 12. Test Setup #2, One Point Loading.

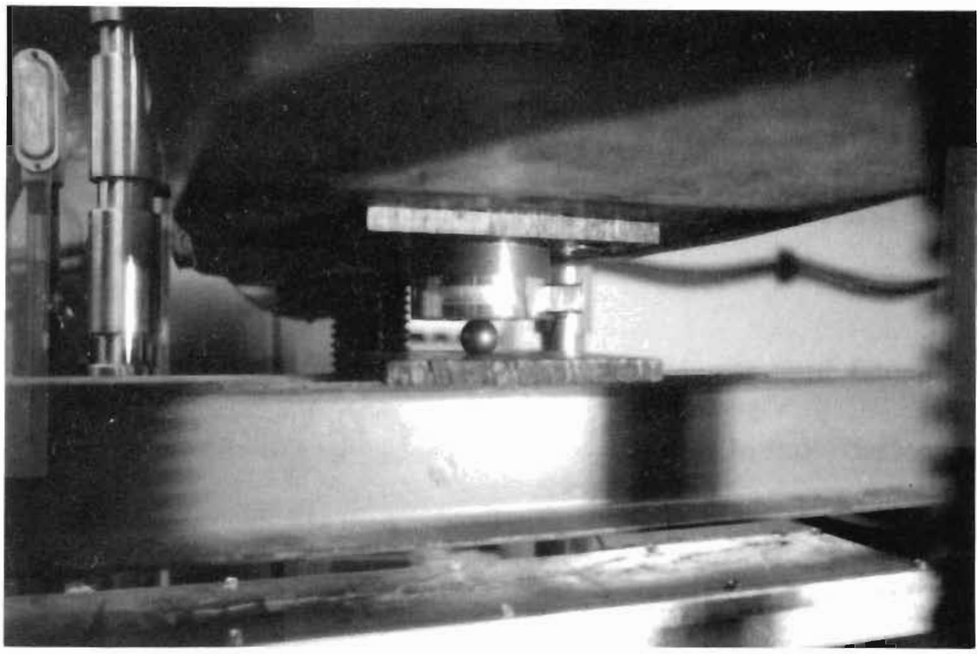


Figure 13. Load Cell used in Test Setup #1.

E. EXPERIMENTAL PROCEDURE

The data collected in this experimental study consisted of the ultimate load capacity of the beam section and incremental strain versus load readings when strain gages were employed. The ultimate load capacity was defined as the maximum load the cross section was able to support. For simplicity, the discussion will be presented in three sections: h-type without strain gages, gsh-type without strain gages, and all tests with strain gages. A summary of all the test data collected during this study is provided in Appendix C.

1. H-type without Strain Gages. This set of tests employed test setup #1 for all but four tests. Test setup #2 was used for four tests. The test beam was placed on the Tinius Olson test machine and properly aligned with the load beam and supports. The test machine load was zeroed and the load beam was placed on the specimen. The load was applied at a constant rate until failure of the specimen. Once the specimen was unable to carry additional

load the specimen was unloaded. The failure load of the sections was recorded from the test machine dial gage. The dial gage on the machine can accurately be read in 12.5 lb increments. This large increment amounts to about 1% of the failure load of the specimens tested.

2. Gsh-type without Strain Gages. Test setup #1 was used for all gsh-type test specimens in this series. The Tinius Olson machine was used to apply the load at a constant rate and a 2000 lb load cell was used to obtain the failure load of the specimen. The load cell was adjusted at the beginning of each test to include the weight of the load beam and bearing plates. The load was then applied at a constant rate until failure of the specimen. The output of the load cell was recorded using the computer data acquisition system employing Labtech Data acquisition software. The load was recorded once per second until failure of the specimen was achieved.

3. All Tests with Strain Gages. This series of tests used test setup #1 and the Tinius Olson Machine to apply the loads. Load readings were recorded using the Labtech data acquisition system and the load cell. After placement of the specimen on the support beams the strain indicators were zeroed. The load cell was adjusted to include the weight of the load beams and bearing plates. The load was then applied at a constant rate until failure of the specimen. Strain and load readings were recorded by the data acquisition system once per second.

IV. EVALUATION OF TEST RESULTS

A. GENERAL

A total of 83 full-scale beam tests were completed at the University of Missouri-Rolla for evaluation of built-up sections with the cover plate in compression. The tests included 60 sections without edge stiffened cover plates (16 with strain gages) and 23 sections with edge stiffened cover plates (6 with strain gages). Also used in the evaluation was Luttrell's data[4], which consisted of 82 deck panels with edge stiffened cover plates. All of the data used in this evaluation has been reproduced for convenience in the Appendix of this document.

Evaluation of the test results consisted of a comparison of the predicted moment capacity using the AISI Design Specification[2], Yener's Spacing Criteria[3], Luttrell's modified effective width equation[4], and a UMR model. Strain gage results will be presented as well as a comparison of one point and two point load cases.

B. BEHAVIOR OF TESTED SPECIMENS

A summary of the behavior of the test sections is provided here to show how the buckling behavior varied based on spacing, material thickness, and cross-section changes. The two buckling behavior categories are column-like buckling of the cover plate and plate like bucking behavior of the cover plate. The buckling behavior was column-like for all sections in which the tested spacing, s_t, exceeded that required by the AISI Design Specification[2], s_m, and plate-like buckling for tested spacings, s_t, less than, s_m. The main differences in behavior can be attributed to the edge conditions of the cover plate and the thickness of the cover plate. When the spacing of the connectors was less than, s_m, the behavior of the cover plate was that of plate-buckling. Plate buckling behavior of the h-type and gsh-type material was basically identical except for buckling along the unstiffened edge of the cover plate. Because of the thinness of the gsh-type material, severe buckling of the outside edge of the cover plate and the hat section flange occurred. This plate buckling behavior is shown in Figure 14 for the gsh-type material and Figure 15 for the h-type material.

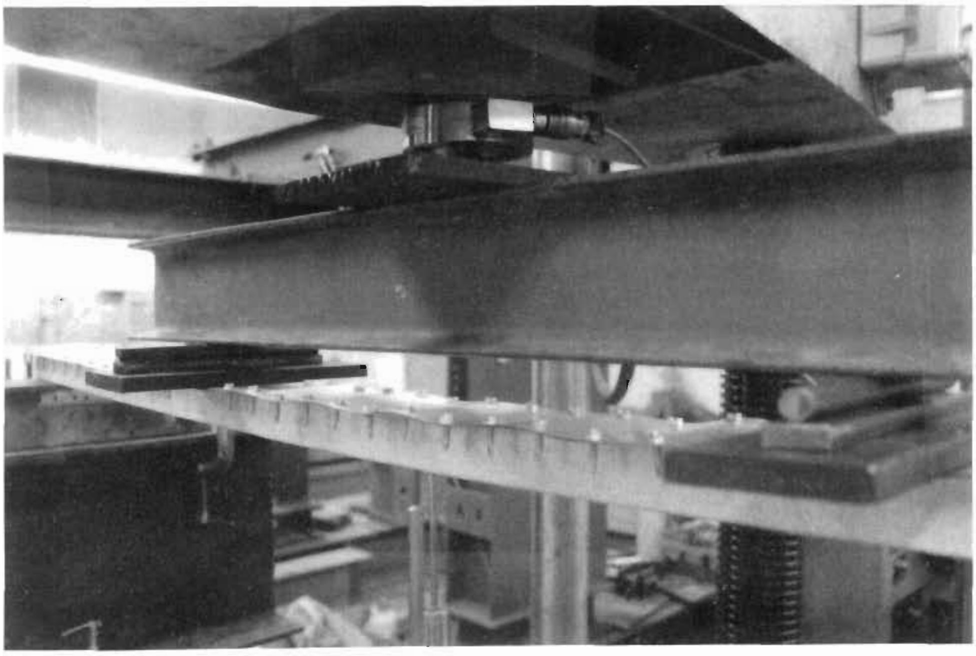


Figure 14. Plate-buckling Behavior of gsh-type Material

Sections with stiffened cover plates with spacings less than, s_m, buckled in a similar manner as the sections in Figures 14 and 15. The edge stiffener provided additional strength increasing the moment capacity with less deformation of the cover plate. This can be seen in Figure 16 which shows a gsh-type section with the same spacing as the section in Figure 14. The failure of the sections with edge stiffened cover plates with spacing less than, s_m, occurred when the web of the hat shape buckled inward toward the center of the section and the cover plate and hat flange buckle upward usually in between two adjacent connectors. This failure can be seen in Figure 16 and is common to all test specimens with edge stiffened cover plates with spacings less than s_m,

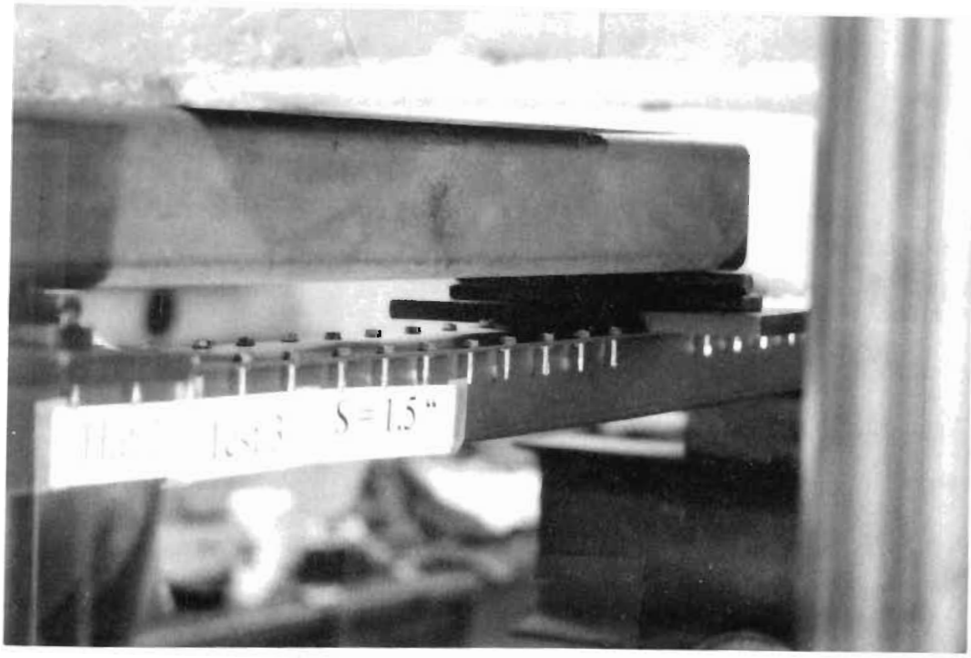


Figure 15. Plate-buckling Behavior of h-type Material

When the tested spacing of the section was increased beyond that required by the AISI Design Specification[2] the buckling behavior of the cover plate was more like a column of length, s, as depicted in Figure 1. This buckling pattern can be seen on the gsh-type material with spacings larger than s_m (Figure 17). This pattern did not hold true as the thickness of the cover plate increased. As the thickness of the cover plate increased the restraint or resistance to this buckling pattern was increased. The buckling pattern was column like buckling, but between adjacent sets of connectors. Because of the additional

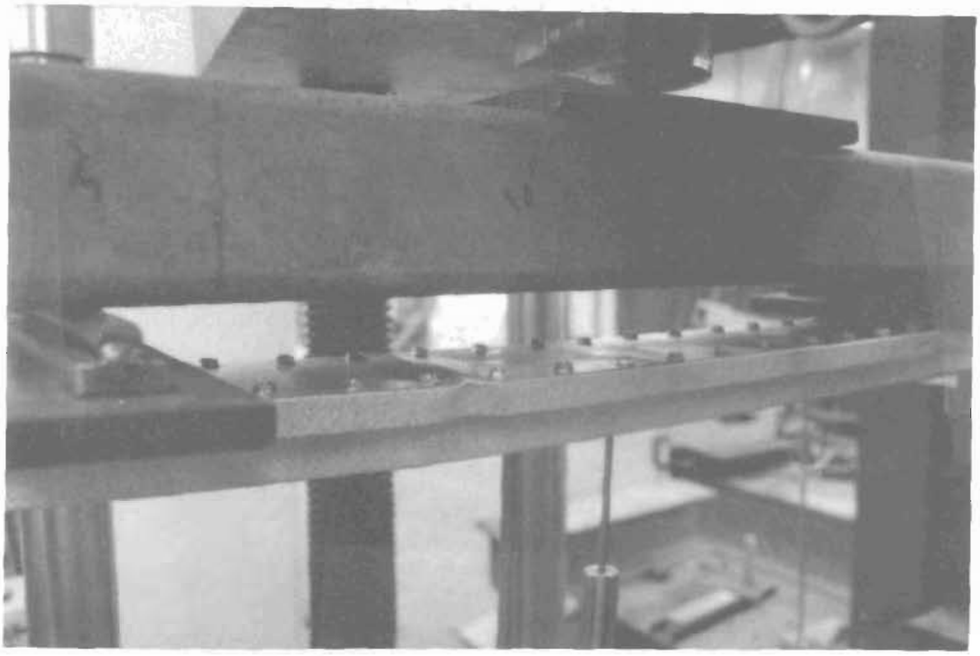


Figure 16. Buckling Behavior of ghs-type Material with Edge Stiffened Cover Plate and a Spacing Equal to s_m.

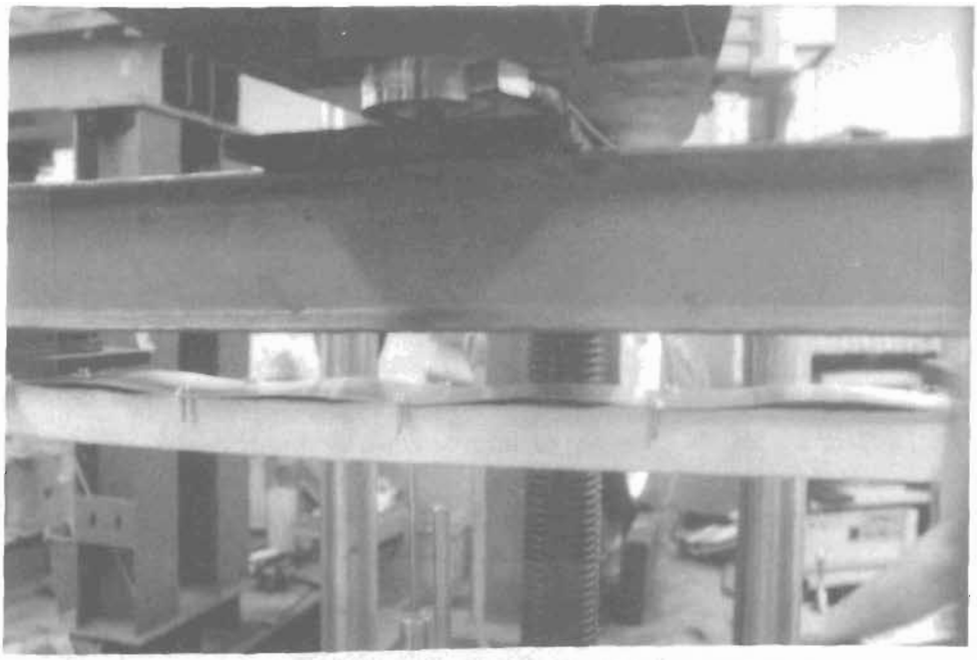


Figure 17. Column-like Buckling Behavior of gsh-type Material.

restraint provided by the adjacent portion of the cover plate, the cover plate buckles in patterns as shown in figures 18 and 19. As the width, w_h, of the section increased the possibility of the cover plate buckling inward into the hat section was increased. This only occurred on the widest of the sections tested, specimens h4 and h5. This inward buckling behavior did not occur on the gsh-type material.

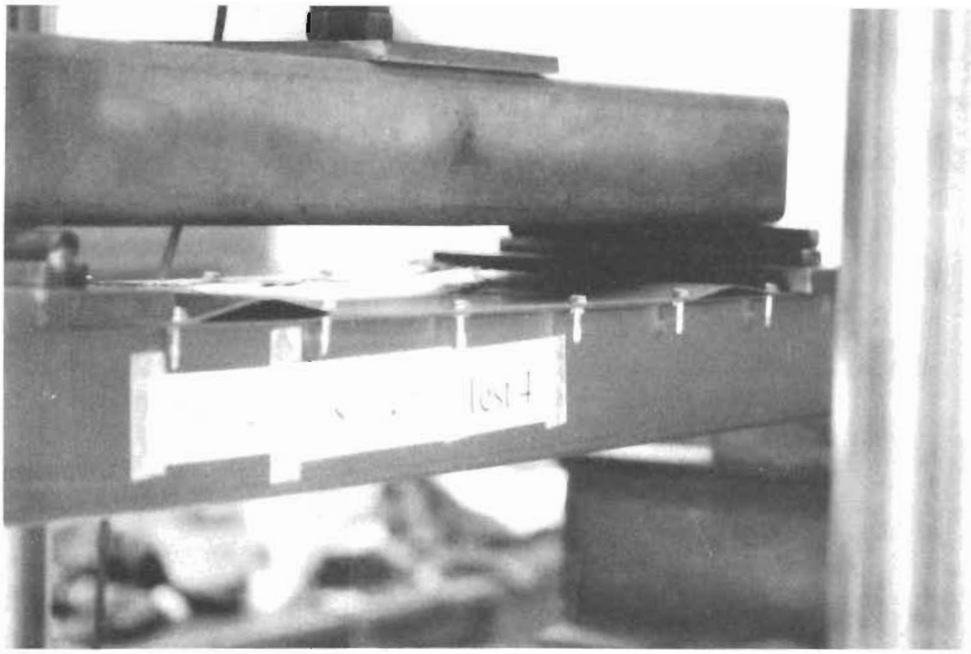


Figure 18. Column-like Buckling Behavior of h-type Material.

Sections with edge stiffened cover plates and spacing larger than, s_m, show column like buckling between connections similar to sections of the same spacings without edge stiffened cover plates. However, two factors change in the buckling behavior. First, the edge of the cover plate is not allowed to buckle with the center of the cover plate because of the edge stiffner. Second, because of the additional restraint provided by the edge stiffener, the cover plate was often forced into a buckling pattern in which each adjacent space between connectors buckled upward (Figure 20). This differed from that shown in Figures 18 and 19.

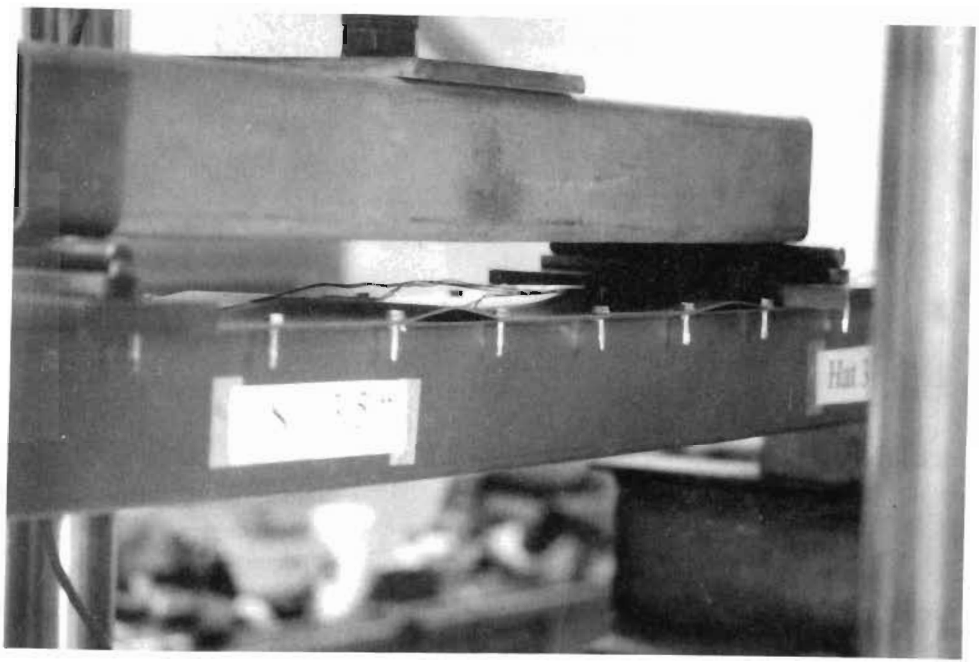


Figure 19. Column-like Buckling Behavior of h-type Material.

C. STRAIN GAGE RESULTS

Strain gages were used to determine the effective length factor and plate buckling coefficient for buckling of the flat plate between the connectors. Two gages were used at each location as shown in Figures 8 and 9. The data obtained from the gages was then plotted on an applied load vs strain graph. A typical graph of this data is shown in Figure 21. The stress reversal method[8] was used to determine the buckling load of the flat plate. Because the buckles occurred gradually, the buckling load was defined as the load at maximum compressive strain before decreasing and ultimately reversing to tension, i.e. point A on

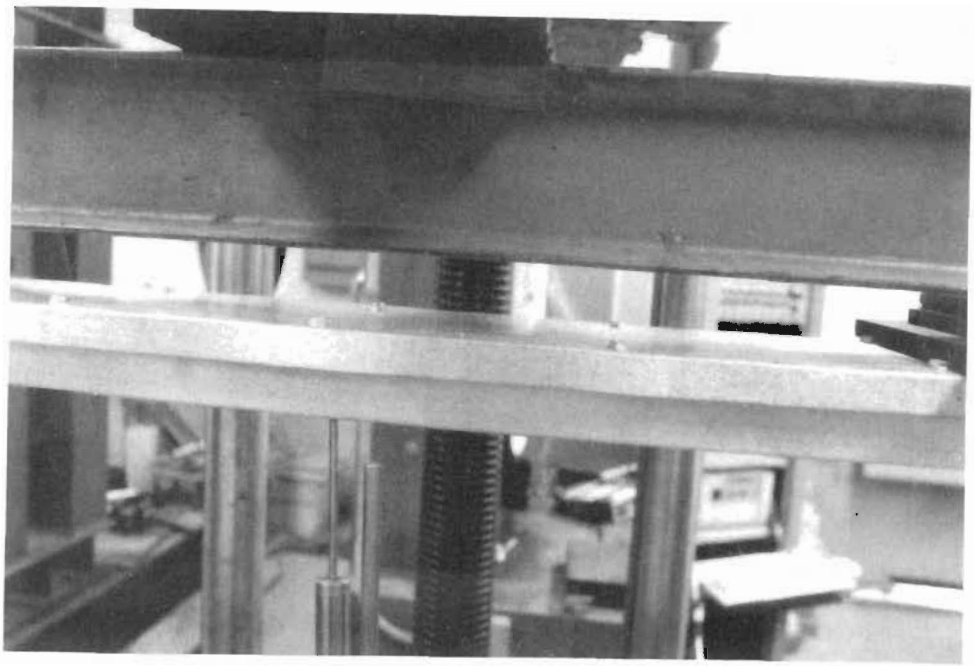


Figure 20. Typical Buckling Pattern for h-type and gsh-type Material with Edge Stiffened Cover Plates.

Figure 21. Once the buckle initiated it was restrained by the hat section, which prevented rapid overall buckling behavior.

Several assumptions were made in the evaluation of the strain gage data. The placement of the gage was assumed to be at the location of the first initiation of the buckle. A constant moment was assumed between the two load points. While this is correct in theory, the test results show there may have been imperfect curvature in this section reducing the moment at the center span. During the tests the first buckle typically occurred at points closer to the bearing plates (Figure 18). This occurred due to the interaction of bending stress and bearing stress near the bearing plates at the load points. A complete set of test data is provided in Appendix F.

The k_c value for all tests without edge stiffened cover plates was determined using

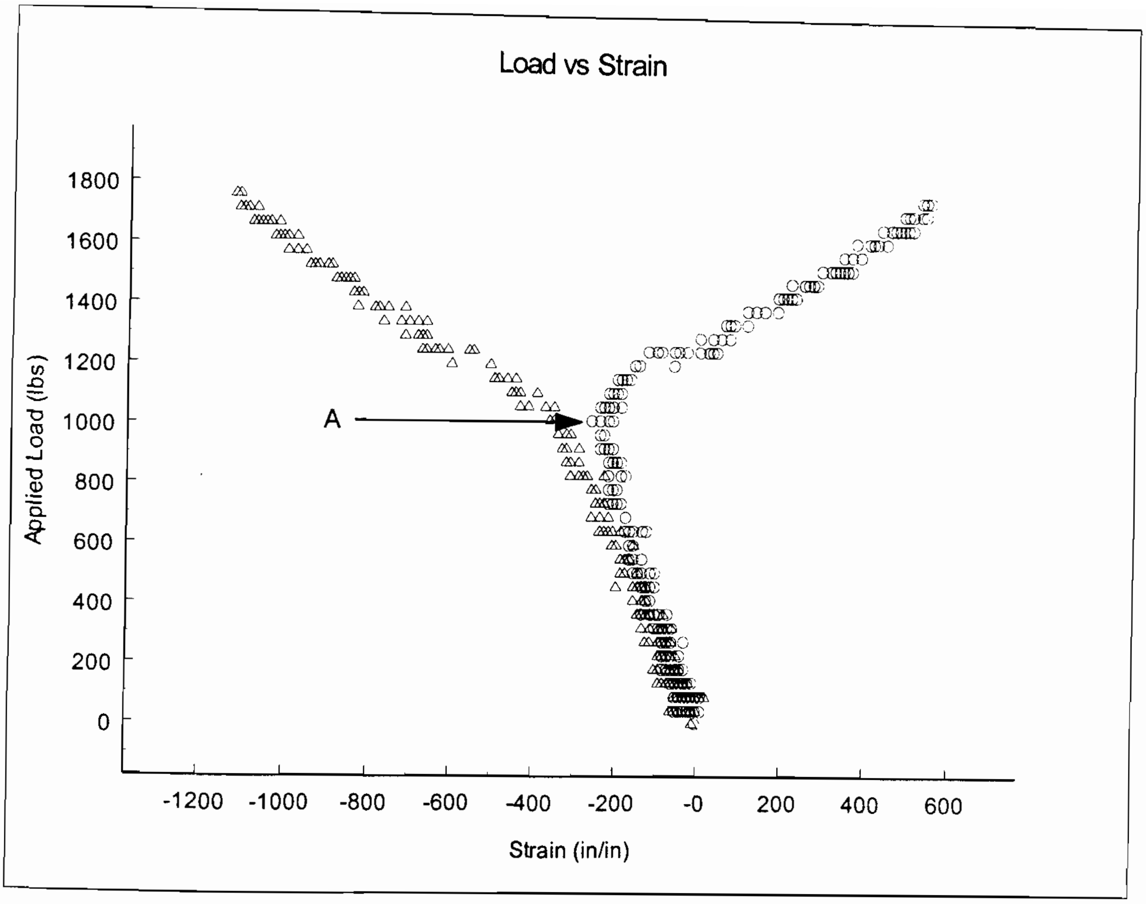


Figure 21. Typical Applied Load vs Strain Plot.

Euler's column buckling formula, Equation 6. The buckling coefficient for the edge stiffened flat plate was determined using Equation 7, which determines the critical elastic buckling stress for a rectangular plate. This equation more closely models the actual behavior of the edge stiffened cover plate cross sections. The buckles that occurred on the specimen occurred across the full width of the specimen, w_c (Figures 7 and 20).

$$\sigma_{cr} = \frac{\pi^2 E}{\frac{(k_c\ s_t)^2}{r^2}} \qquad (6)$$

$$f_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)(\frac{w_c}{t})} \qquad (7)$$

Where s_t = tested spacing of connectors

E = 29,500 ksi

σ_{cr} = Euler elastic column buckling stress

= M_{cr}/S_x

f_{cr} = local buckling stress of a plate

= M_{cr}/S_x

μ = 0.3

M_{cr} = moment at initiation of buckling

w_h = full width of flat plate (Figure 6 and 7)

t = thickness of flat plate

w_c = full width of edge stiffened flat plate (Figure 1)

The results obtained from the h-type material were very consistent and reasonable. Table IV shows that the k_c ranged from 0.74 to 0.85 with a mean value of 0.81. These values are between the theoretical fixed-fixed case (k_c = 0.5) and a pin-pin boundary condition (k_c = 1.0).

The results of the tests on the gsh-type material shown in Tables V & VI are not reasonable. The average value obtained using the same model as the h-type material yields unreasonably low k values. The k_c < 0.5 for column buckling would mean there is more edge

Table IV. h-type Material, k_c Determination, Cover Plate without Edge Stiffeners.

Test #	s_t (in.)	w_c (in.)	mean k_c, individual test	mean k_c, all tests
h3t3	3.5	6.7	0.80	0.81
h3t4	3.5	6.7	0.78	
h3t5	3.5	6.7	0.80	
h4t4	4.0	9.5	0.83	
h4t5	4.0	9.5	0.85	
h4t6	6.0	9.5	0.84	
h4t7	6.0	9.5	0.74	

Table V. gsh-type Material, k_c Determination, Cover Plates without Edge Stiffeners.

Test #	s_t (in.)	w_c (in.)	mean k_c, individual test	mean k_c, all tests
gsh1t3	3.0	3.5	0.49	0.43
gsh1t4	3.0	3.5	0.47	
gsh3t1	6.0	5.0	0.30	
gsh3t2	6.0	5.0	0.33	
gsh4t1	6.0	5.5	0.30	
gsh4t2	6.0	5.5	0.37	
gsh4t3	3.0	5.5	0.47	
ghs4t4	3.0	5.5	0.72	

restraint than a fixed-fixed end condition. These unreasonable results may be attributed to several factors. As previously mentioned, the gage was assumed to be at the exact location

Table VI. gsh-type Material k & k_c Determination, Edge Stiffened Cover Plate.

			Column Buckling		Plate Buckling	
test #	s_t (in.)	w_c (in.)	mean k_c, individual test	mean k_c, all tests	mean k, individual test	mean k, all tests
gsh1t9	6.0	4.0	0.36	0.43	0.80	0.83
gsh1t10	6.0	4.0	0.31		1.08	
gsh2t9	6.0	4.5	0.29		1.36	
gsh2t10	6.0	4.5	0.29		1.38	
gsh3t7	6.0	5.6	0.58		0.46	
gsh3t8	6.0	5.6	0.60		0.41	

the buckle initiated. All gages were placed at the centerline of the connection spacings as shown in Figures 8 and 9. However, the buckle would often occur off center. If the first initiation of the buckle did not occur at the gage location, the measured buckling load obtained would be larger than the actual buckling load. This phenomena was very common on the gsh-type material. Imperfections from fabrication were also noted during testing. These imperfections included dents in the flat sheets and initial buckles left after the C-clamps were removed. Many of the sections buckled under the initial load from the bearing plates and load beam. It was not always possible to determine if the buckle was initially present as a result of fabrication or if the buckle occurred from the initial load. Errors may also have occurred because any initial buckle recorded by the strain gage was zeroed out before testing began.

The edge stiffened sections in Table VI were also evaluated as plates of width, w_c, (Figure 7) and length s_t. The behavior of the cover plate as shown in Figure 20 is consistent with the behavior of rectangular plate supported on all four sides under uniform compression.

The results of the testing showed a mean k = 0.83. This value is questionable because a plate simply supported on all four edges would be evaluated using a k = 4.0. Possible reasons for the low k value may be attributed to the thickness of the material and the size and configuration of the edge stiffener. Because the material is very thin, t = 0.0174 inches, the rotational restraint provided by the material between connection lines is much less than the rotational restraint provided the thicker h-type material. This can be seen in the difference in buckling behavior of the h-type and gsh-type materials. The cover plate of the gsh-type material buckled upward at each adjacent space between connectors (Figure 17) as compared to the h-type material that does not buckle in this pattern(Figures 18 and 19). If the same test was run using a thicker material the k value would have been larger due to the additional rotational restraint provided by the material between connection lines. The edge stiffner on the gsh-type material buckled during the tests. These buckles reduced the effectiveness of the stiffner and provided less edge restraint. This buckling pattern occurred on the h-type material but not until failure. Therefore the edge stiffner provided increased stiffness to the edge of the plate.

D. HAT SECTION BENDING CAPACITY

Ideally, in the design of built-up sections, each component contributes to carrying the applied load. To ensure that built-up cross section behavior was being achieved, the experimentally determined moment capacity was compared to the fully braced moment capacity for the hat shape alone. If the experimentally obtained moment capacity is less than the fully braced hat shape capacity alone, built up action between the flat sheet and hat shape

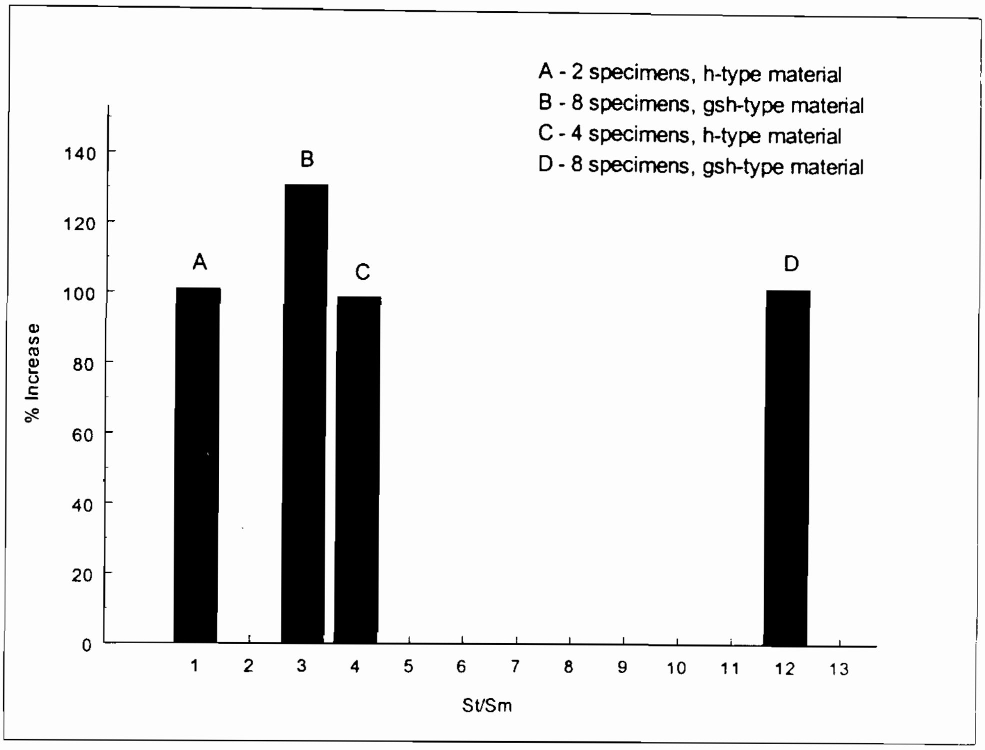


Figure 22. Percent Increase in Capacity above Fully Braced Hat (Edge Stiffened Cover Plate).

was not obtained. Figures 22 and 23 show the average percent increase in capacity over the fully braced hat capacity alone for the sections with edge stiffened cover plates and without edge stiffened cover plates. The ratio s_t/s_m is a ratio of the tested spacing, s_t, divided by AISI Specification[2] maximum spacing as required by Section D1.2.

As shown by Figures 22 and 23, there was an increase in capacity due to the presence of a cover plate. The cover plate served two purposes: first, the plate braced the compression flange and webs of the hat section not allowing them to buckle laterally, and second, the cover plate adds additional capacity as a small compression element between connectors. The increase in capacity as shown in Figure 23 varied from 20 - 120 percent. As the ratio of s_t/s_m increased there was an exponential decrease in the contribution made by the cover plate. The graph also shows that for similar ratios of s_t/s_m between the h-type and gsh-type material that similar capacity increases were obtained as shown by bar C and D of Figure 23.

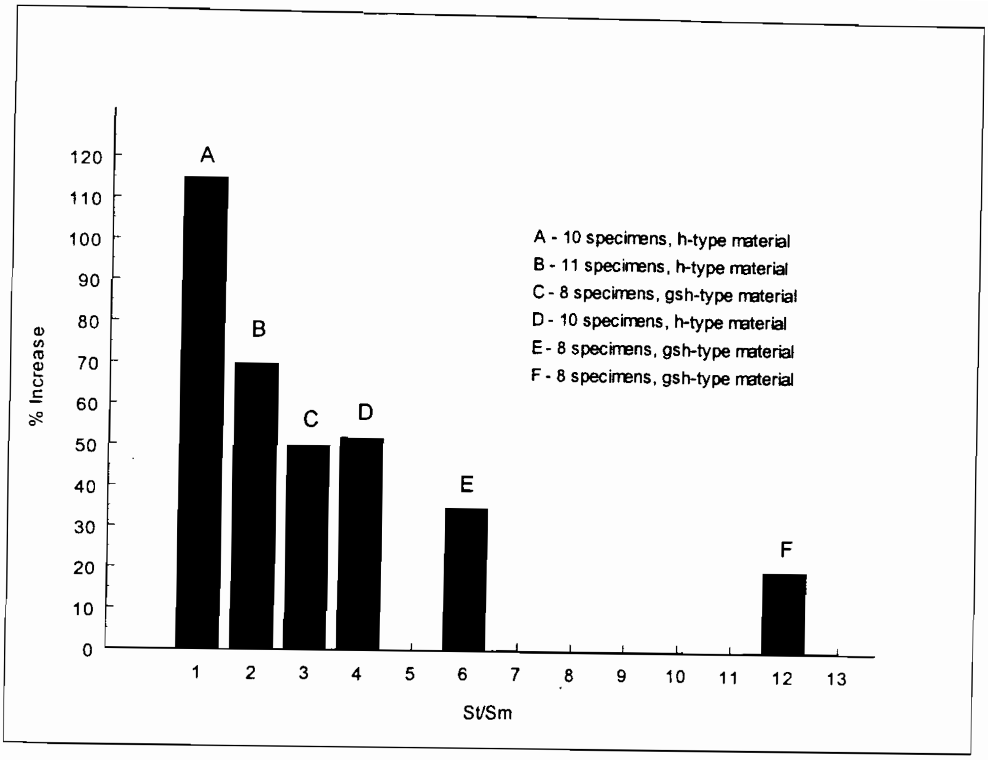


Figure 23. Percent Increase in Capacity above Fully Braced Hat (Cover Plate without Edge Stiffeners).

E. BUILT-UP SECTION BENDING CAPACITY

The AISI Design Specification[2] requires that spacing of connectors meet requirements in Section D1.2, Spacing of Connections in Compression Elements. When this spacing requirement is met, Procedure I - Based on Initiation of Yielding, $M_n = S_e F_y$, given in Section C3.1.1 Nominal Section Strength can be used to calculate the cross sectional moment capacity. The tested beam sections were similar to a closed box beam, thus lateral buckling strength need not be checked. In the calculation of the effective section modulus, the portion of the plate between the connection lines, w (Figures 6 and 7), was considered to be a uniformly compressed stiffened element with k = 4.0. The area outside the connection line was considered to be unstiffened or partially stiffened depending on the edge condition of the cover plate.

Comparisons with the AISI Design Specification[2] included two parts. First, a comparison was made with the sections in the test program that met the requirements of Section D1.2 on Spacing of Connections in Compression (Figure 24). Second, a comparison was made with all test specimens which included spacings, s_t, that exceeded the maximum spacing required, s_m (Figures 25 and 26). For the test data of this study, the ratio s_t/s_m ranged from 1 to 12. Figure 24, 25, and 26 show the comparison of the tested moment capacity, M_t, to the computed moment capacity M_c, where $M_c = S_eF_y$.

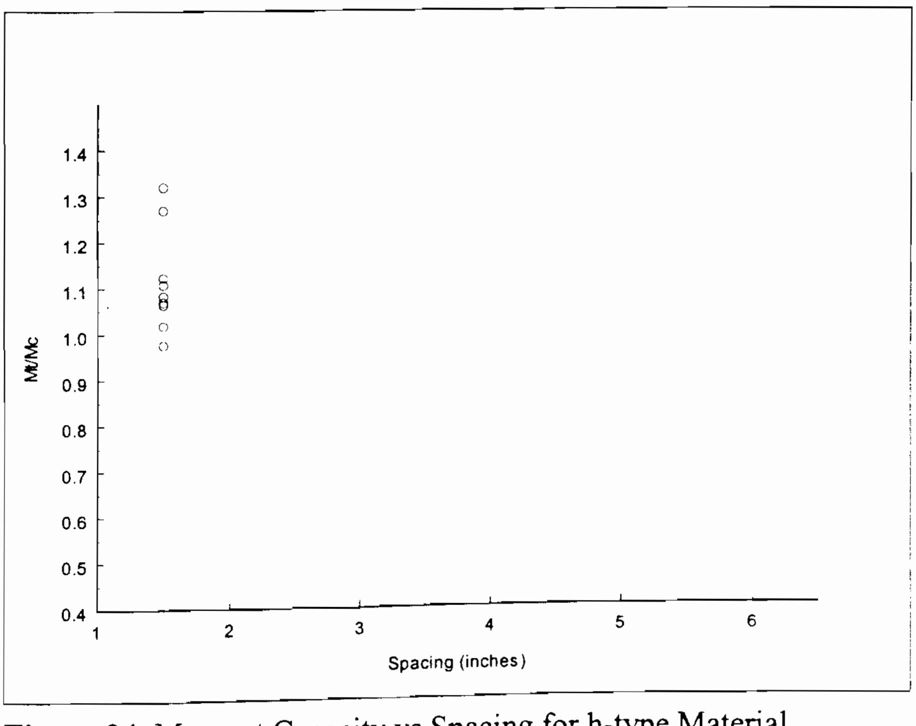


Figure 24. Moment Capacity vs Spacing for h-type Material.

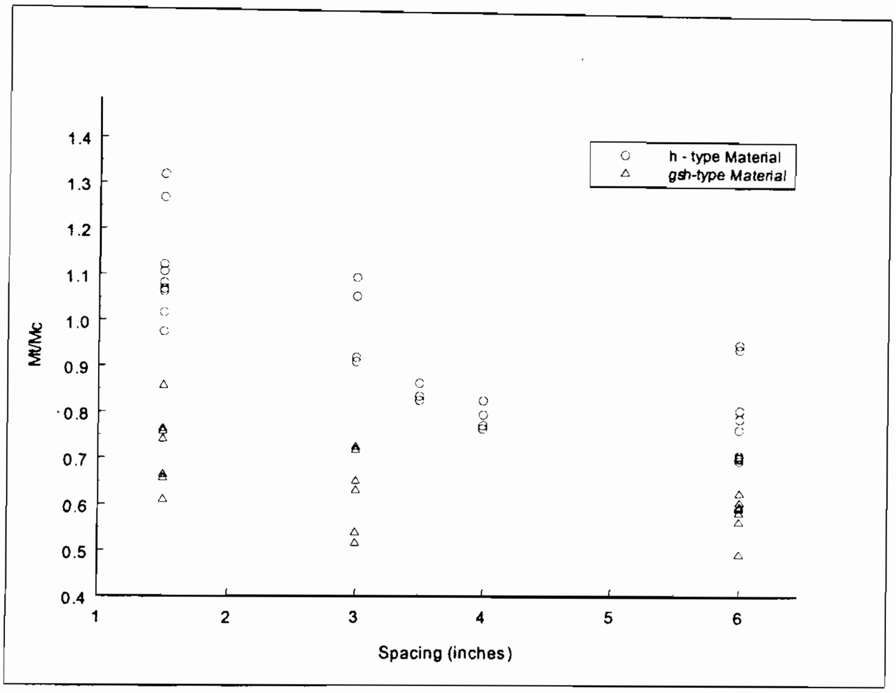


Figure 25. Moment Capacity vs Spacing for Sections without Edge Stiffened Cover Plates, Neglecting Section D1.2[2].

As shown in Figure 24, ten tests met the requirements of Section D1.2 of the AISI Design Specification[2]. The results of these tests showed that the AISI Design Specification can accurately predict the moment capacity of the tested sections. The ratio of the tested moment capacity to the computed moment capacity, M_t/M_c varied from 0.98 to 1.32 with the mean = 1.11 and Coefficient of Variation (C.O.V.) = 9.01%. A summary is provided in Appendix C.

Figure 25 shows the results of the comparison between the AISI Design Specification[1] and the sections without edge stiffened cover plates. Spacings greater than 1.5 inches did not satisfy the spacing requirements in Section D1.2 for the h-type material. The graph shows that the AISI Design Specification[1] can accurately predict the moment

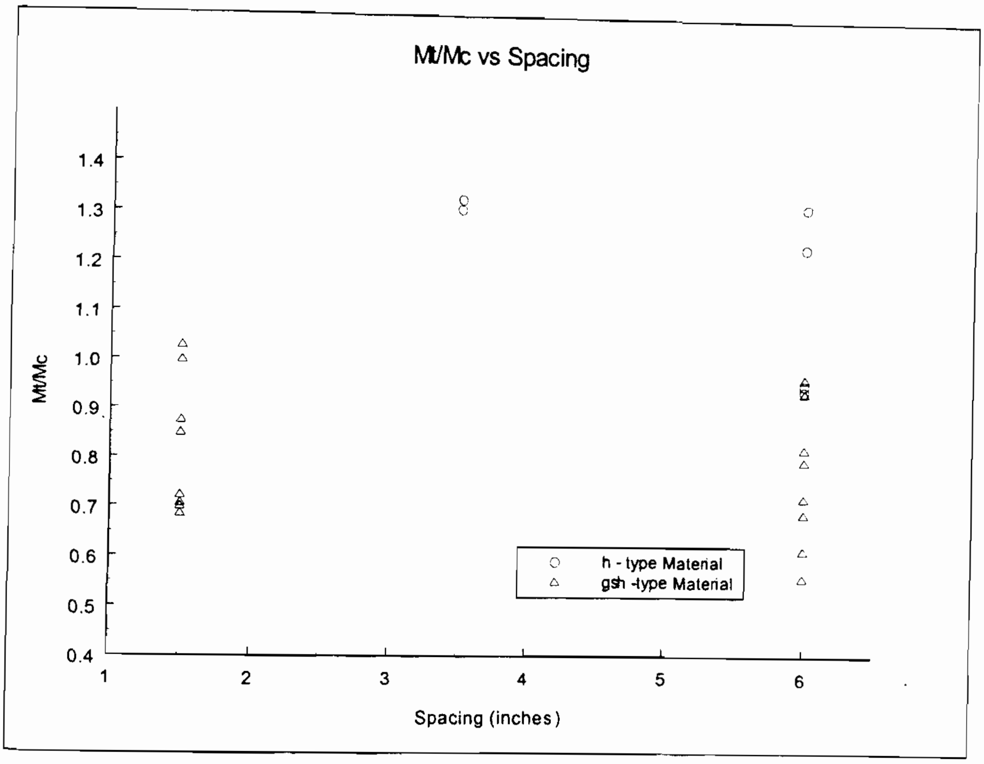


Figure 26. Moment Capacity vs Spacing for Sections with Edge Stiffened Cover Plates, Neglecting Section D1.2 of the AISI Design Specification[2].

capacity of the h-type material when the required spacing of Section D1.2 is not exceeded. The AISI Design Specification[2] did not accurately predict the capacity of the sections when the spacing was increased beyond that required by Section D1.2 for the h-type or gsh-type material. The test results show that increasing the spacing of connectors beyond that required by Section D1.2 does not significantly diminish the capacity of the section.

Figure 26 shows a comparison of the tested moment capacity to the computed moment capacity for sections with edge stiffened cover plates using limited test data. Screw spacings greater than 1.5 inches do not satisfy the spacing requirements of Section D1.2 of the AISI Design Specification[2].

F. YENER SPACING CRITERIA

The spacing criteria developed by Yener[3] was outlined in Chapter II, Section C. The calculation of the nominal bending capacity, $M_n = S_eF_y$, is the same as outlined in Section E of this chapter. Figure 27 shows a comparison of tested moment capacity to the computed capacity, M_t/M_c. Only the test specimens having spacings close to the value required by Yener's criteria are given. Figure 27 also shows test specimens with spacings, s_t, that were smaller than the maximum value allowed by Yener's[3] spacing criteria.

Figure 27 shows that the spacing recommendations proposed by Yener are too liberal yielding computed values as much as 40% higher than the tested moment capacity. The tested spacings that are less than that required are shown here to indicate that at even smaller spacings, the computed capacity was in excess of the tested value. These results clearly show that Yener's criteria are inadequate for sections without edge stiffened cover plates. This inadequacy can be attributed to two factors. First, Yener's[3] spacing criteria was developed from tests on standard deck sections which consist of multiple flutes and stiffened cover plates. Second, Yener's criteria[3] were developed on a very small number of tested sections with cover plates in compression.

G. LUTTRELL AND BALAJI MODIFIED EFFECTIVE WIDTH MODEL

Luttrell's[4] modified effective width model was developed from 82 deck panel tests. The decks used were standard deck sections with multiple flutes and edge stiffened cover plates (Figure 5). Luttrell's model allows the use of any spacing desired and requires a reduction in bending capacity when the connector spacing is greater than s_m, which is equal

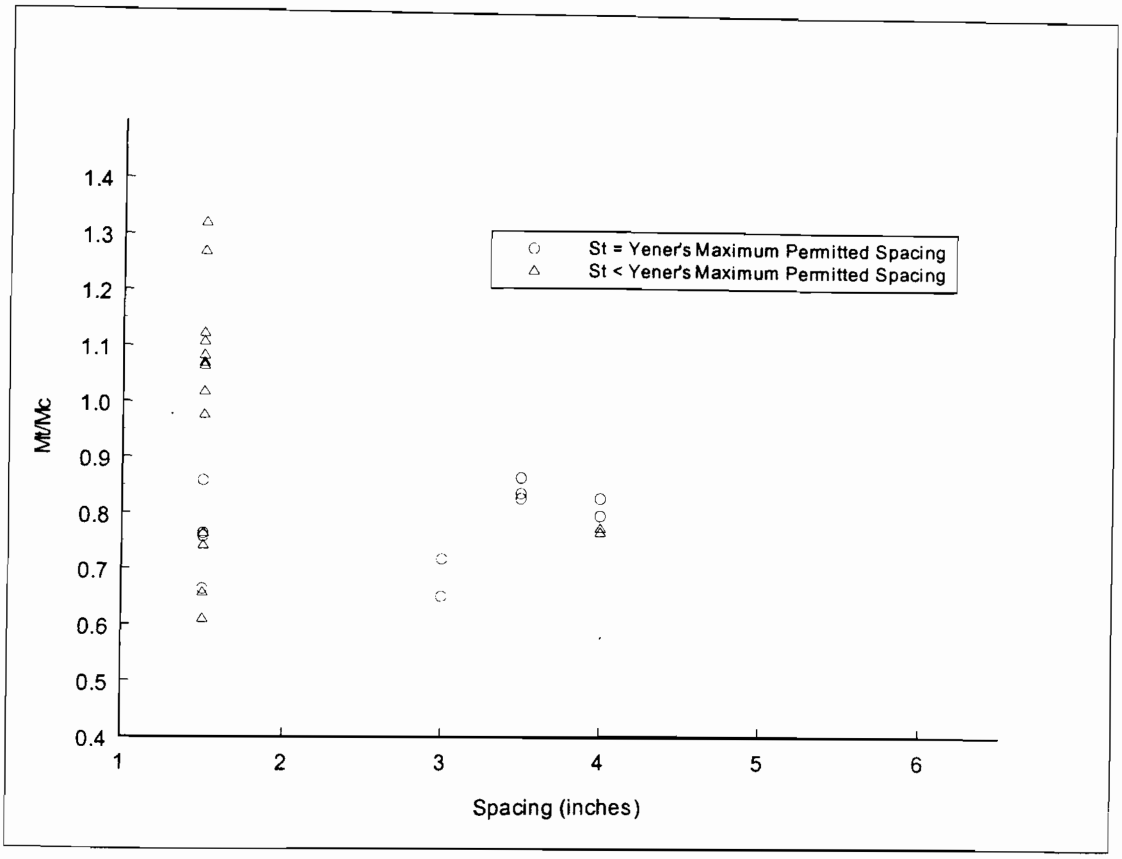


Figure 27. Comparison of Yener's[3] Spacing Criteria to Experimental Data.

to the maximum permitted spacing allowed by the AISI Design Specification[2]. For example, when s_t, the tested spacing, is equal to or less than s_m, Luttrell's model would not require a reduction in the current AISI Design Specification[2] moment capacity.

A comparison of Luttrell's model to the experimentally determined capacity is shown in Figures 28 and 29. Figure 28 shows all the sections with edge stiffened cover plates, for which Luttrell's model was developed, as well as Luttrell's original data. Using Luttrell's model the ratio of the tested moment to the computed moment varies by as much as plus or minus 35%. Analysis of the data shows that the h-type material across the full width of the

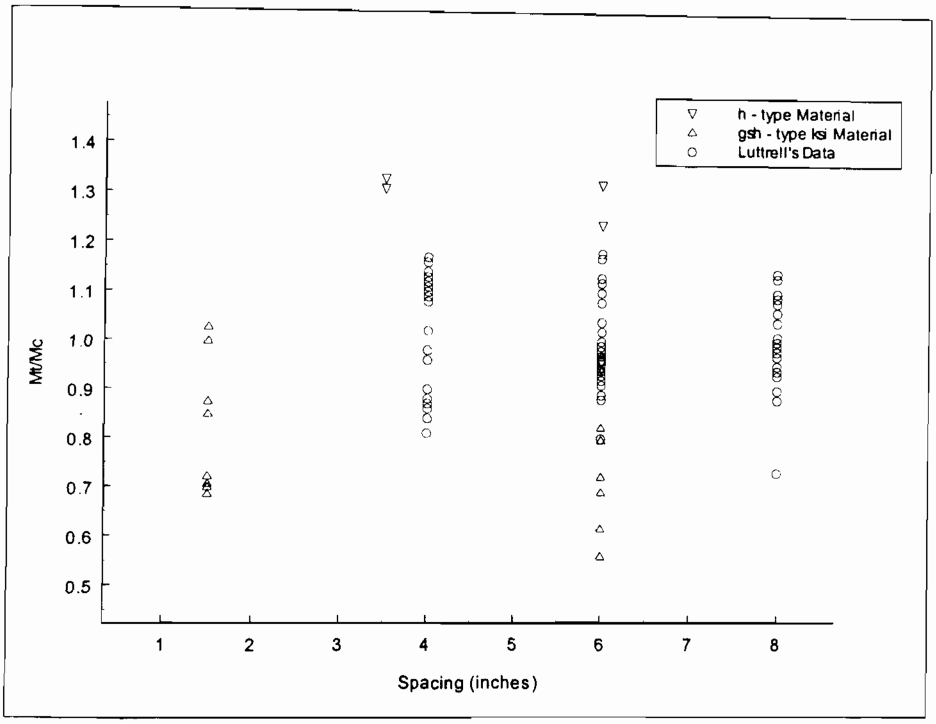


Figure 28. Comparison of Computed and Tested Moment Capacities using Luttrell's Model[4] for Sections with Edge Stiffened Cover Plates.

computed moment value is conservative while the gsh-type material computed moment value is very unconservative.

No single parameter explains the variations of the test data. It should be remembered that Luttrell's model was developed from standard deck sections with multiple flutes. Luttrell's modifications to the AISI effective width equations are based on column-like buckling behavior of the cover plate between the connectors. The effective length factor, k_c, used in Luttrell's model was 0.5, which is for a fixed-fixed column. The buckling behavior of the tests performed at the University of Missouri-Rolla exhibited plate-like buckling because of the edge restraint provided by the simple lips on the cover plate. Figure 20 shows a typical buckling pattern of a section with an edge stiffened cover plate. The buckles occured

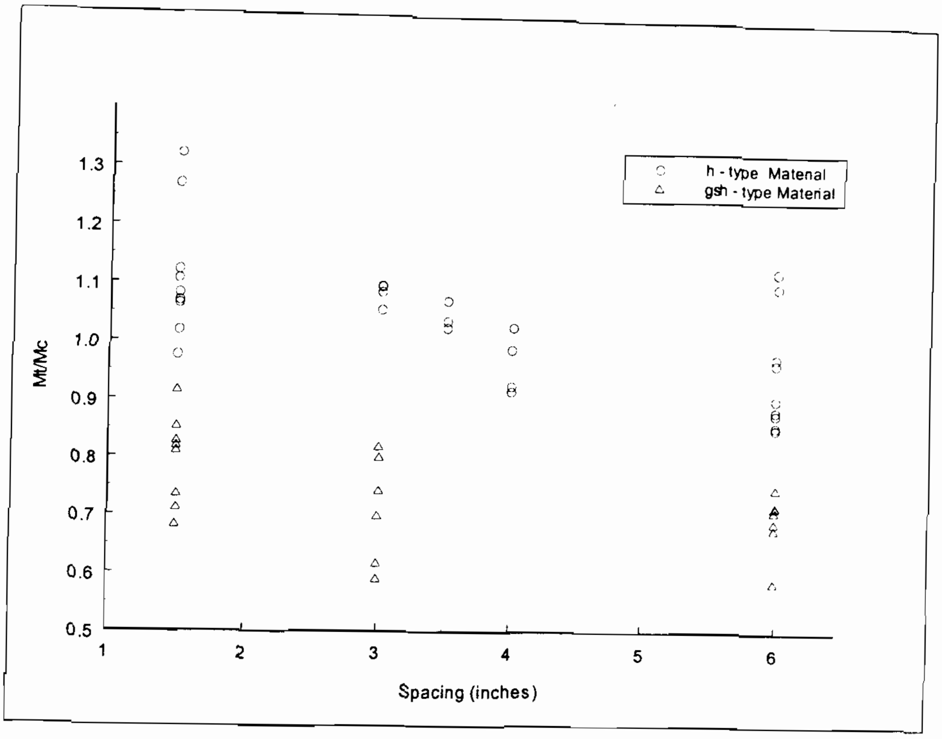


Figure 29. Comparison of Computed and Tested Moment Capacity using Luttrell's Model[4] for Sections without Edge Stiffened Cover Plates.

across the full width of the section. Luttrell did not indicate if the buckles occurred all the way to the outside edge of the cover plate. The specimens Luttrell tested were standard decks which are typically 24 inches in width or wider. As the width of a cross section increases the edge restraint provided by the edge stiffener on the cover plate decreases. Therefore, Luttrell's use of the Euler column buckling formula may be more accurate for the larger width sections. The test specimens tested at University of Missouri-Rolla ranged from 3.5 to 9.5 inches wide. For the UMR sections which had only a third the width of a standard deck section (Figure 5), the behavior of the edge stiffened cover plate behaved similar to a plate with edge restraint on all four sides and not like a column. Therefore, Luttrell's model does not accurately model the actual behavior of the edge stiffened sections which may account for the inability to predict the capacity for single flute sections.

Although Luttrell's model was developed for deck sections which had edge stiffened cover plates (Figure 5), a comparison was made using Luttrell's model[4] for the test sections without edge stiffened cover plates. Figure 29 shows a comparison of the computed moment capacity to the tested moment capacity. The graph shows that Luttrell's Model could be used to compute the capacity of the h-type material with a mean value of 1.03 and a Coefficient of Variation of 10%. The model, however, overestimates the capacity of the gsh-type material by as much as 30%. An attempt was made in this research study to modify Luttrell's model. It was found that in order to achieve the tested moment capacity of the gsh-type material the effective width of the cover plate between the connections, w, would have to decrease to zero. This is not reasonable, because the cover plate is providing some strength to the section.

H. UMR MODEL

Both Yener and Luttrell models failed to reasonably predict the capacity of the sections tested in this study. Because the test program involved only a small number of tests with edge stiffened cover plates, including only a small number of different variables, inadequate information was available and no new model was developed for the sections with edge stiffened cover plates. Adequate test data was obtained for the development of a new model for sections without edge stiffened cover plates. This section will discuss the parameters involved in the new model and show comparisons with the experimental data.

During the test program it became necessary to look at the modeling assumptions that had been made up to this point. All methods used before looked at the cover plate between the connections as a stiffened element and the section outside the connection line as an unstiffened element. This assumption is incorrect for cover plates without edge stiffeners because the flat plate buckles from edge to edge across the width in a column-like buckling pattern (Figure 17, 18, and 19). There is no edge restraint provided at the connection line along the length of the specimen. Various parameters were also found to have definite effects on the capacity of the section. For example, as the spacing of the connectors increased beyond s_m, the buckling behavior changed from plate buckling to column buckling. The tested capacity decreased as the spacing increased (Figure 30). As the width of the section increased the likelihood increased that sinusoidal plate buckling waves occurred. When the plate buckled, the specimen did not fail, meaning that there was some post-buckling strength provided by the cover plate. As the connector spacing increased the capacity of the built-up section decreased to that of a fully braced hat section alone (Section IV-D). Based on Figure 23, at $s_t/s_m = 12$ there is only a 20% increase in capacity above that of a fully braced hat section showing that the cover plate did provide additional capacity to the section.

The new model was developed based on the fact that spacings in excess of s_m, as defined by AISI Section D1.2, may be used. Because the larger spacings produced a column-like buckling behavior, the following model was developed. The bending capacity of the section was calculated based on the full section modulus times the elastic critical buckling stress, as calculated by the Euler column buckling formula. This capacity, however, does not consider any post-buckling strength of the cover plate or additional capacity provided by the portion of the cross section below the neutral axis. This additional capacity above $M_c = \sigma_{cr}S_x$ is shown in Figure 31 for section h5. The portion of the cross section below the neutral axis

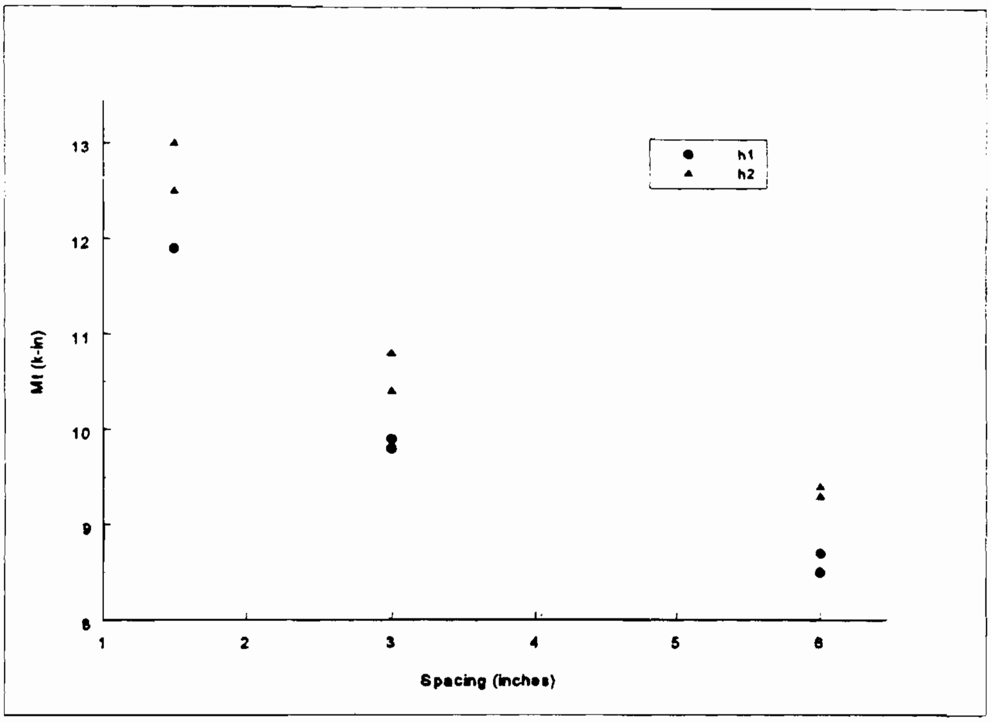


Figure 30. Tested Moment Capacity vs Tested Spacing

helps to stabilize the section not allowing quick overall failure of the section when the cover plate buckles. The post buckling strength of the cover plate was provided by the rotational restraint of the material between connection lines at the connectors. This rotational strength was a function of the material thickness and the width of the section. As the width of the section increased, the possibility of the adjacent space between connectors buckling inward into the cross section increased. When the adjacent space between connectors buckled inward the rotational restraint provided at the connectors decreased. As the width of the section increased, the possibility of the adjacent space between connectors buckling inward into the cross section increased. When the adjacent space between connectors buckled inward the rotational restraint provided at the connectors decreased. As the material thickness increased the rotational restraint at the connectors increased. This is shown in Figures 17, 18, and 19.

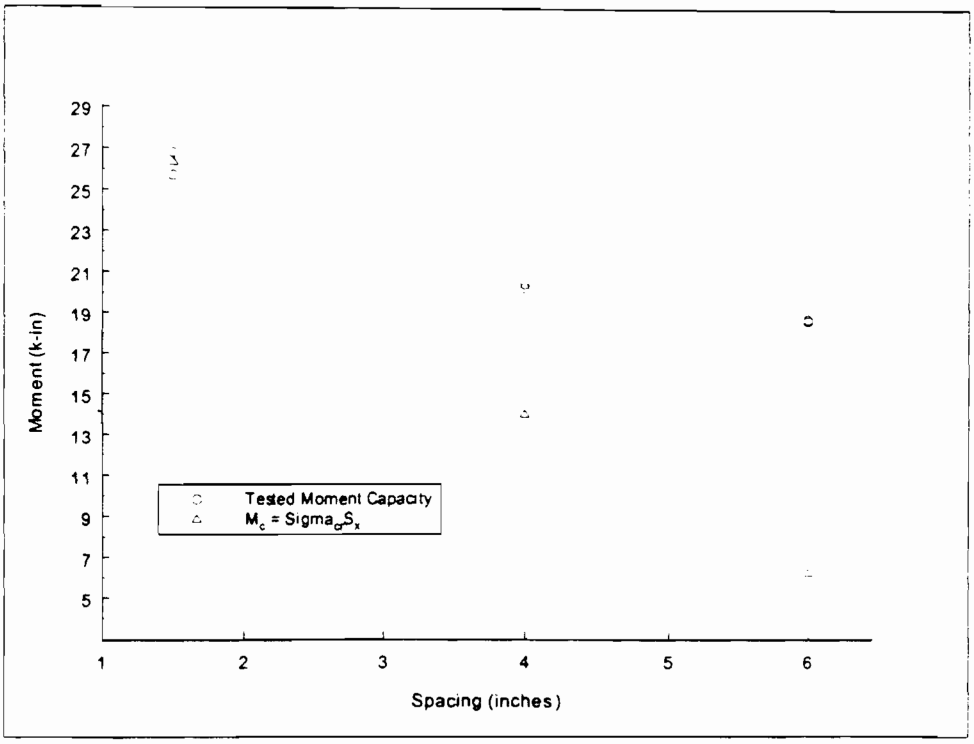


Figure 31. Tested Moment Capacity vs $M_c = \sigma_{cr}S_x$ for Section H5.

The bending capacity calculation was then modified to include this additional capacity. The parameter that was found to affect the increased capacity was the ratio of the actual spacing, s_t, divided by the minimum spacing, s_m. As the ratio of s_t/s_m increased the post-buckling strength of the section decreased. At ratios of s_t/s_m above three it was found that additional modification was necessary. Beyond $s_t/s_m = 3.0$ the width, w_h, of the specimen affected the post-buckling strength of the section. The ratio of the width w_h (Figures 6 and 7), to the desired spacing, s_t is used for further modification. The following is a summary of the UMR model:

When $s_t/s_m \leq 3.0$, and $k_c\ s_t / r < 328$

$$M_n = S_x \sigma_{cr} (\alpha_1) \tag{8}$$

$3.0 < s_t/s_m < 6.0$, and $k_c\, s_t / r < 350$

$$M_n = S_x \sigma_{cr}(\alpha_2)(\alpha_3) \tag{9}$$

Where:

$\alpha_1 = 0.849 + 0.253(s_t/s_m)$

$\alpha_2 = -9.11 + 4.683(s_t/s_m) - 0.363(s_t/s_m)^2$

$\alpha_3 = 1.634 - 0.464(w/s_t)$

$k_c = 0.6$

s_t = Desired Spacing

$s_m = 1.16t\sqrt{(E/f_c)}$

= Criterion # 2 Section D1.2 AISI Design Specification[1]

t = Thickness of cover plate

f_c = Stress at service load in the cover plate

w = Flat width of cover plate (Figure 1)

S_x = Full section modulus of section about x-axis with respect to the extreme compression fiber.

r = Radius of gyration of cover plate

σ_{cr} = Euler column buckling stress of the cover plate with $k = 0.6$

Figure 32 shows a comparison of the tested to computed value of the UMR model.

The graph of the data shows a reasonable correlation to the tested value with a mean of 1.00 with a coefficient of variation of 11%. The range of the parameters used to develop this model are given as follows:

$F_y \leq 53$ ksi

$88 \leq w/t \leq 287$

$t \geq 0.017$ in.

$1.0 \leq s_t/s_m \leq 12.0$

$69.0 \leq k_c\, s / r \leq 328.0$

$1.0 \leq \alpha_1 \leq 1.7$

$3.8 \leq \alpha_2 \leq 6.0$

$0.75 \leq \alpha_3 \leq 1.3$

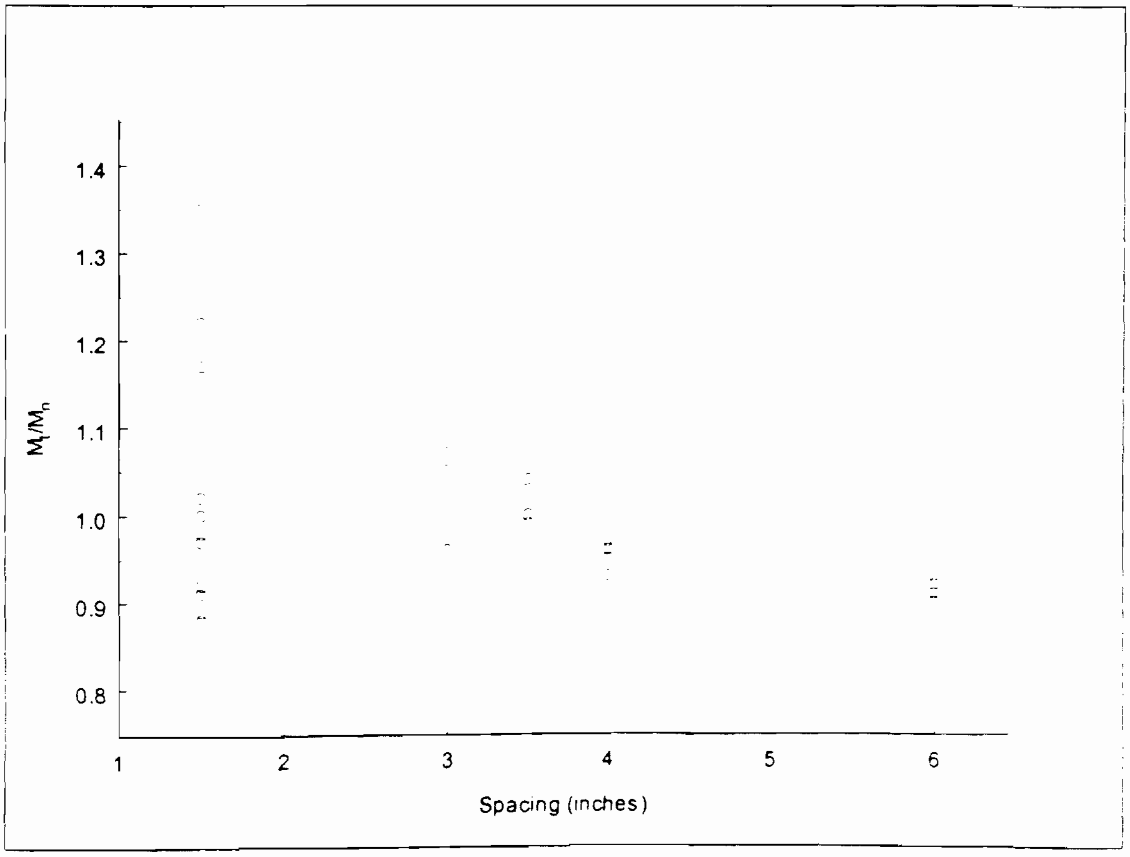


Figure 32. Comparison of Tested and Computed Capacities for UMR Model.

V. CONCLUSION

A. SUMMARY

This research project was initiated in May 1996, in order to determine if the current spacing criteria outlined in Section D1.2 of the AISI Design Specification is adequate in accurately predicting the capacity of built-up sections with the cover plate in compression. This study showed that criteria # 2 of the AISI Design Specification[1] spacing criteria is very conservative when applied to built-up cross sections in bending. All sections when tested continued to carry additional load after the cover plate buckled. The results of the tests showed that the buckling behavior of the cover plate as a simple column can be prevented when adhering to the 2nd criterion of the AISI Design Specification, Section D1.2[1]. Therefore the criterion is adequate for prevention of column-like buckling of the cover plate between connectors. The testing program found that the following parameters effect the capacity in bending of the section: the thickness of the cover plate, the width of the section, the number of flutes, the edge restraint provided by edge stiffeners, and the spacing of the connectors. The tests showed that as the spacing of connectors increased the moment capacity of the section decreased. As the width of the section increased the possibility of inward buckling of the cover plate into the hat section increased. As the cover plate thickness increased the rotational restraint provided at the connector increased. If the edge stiffener is inadequate the stiffener does not provide the same stiffness along the edge of the section as an adequate stiffener.

A review of available literature was conducted and compared with 83 simple-span beam tests. It was found that the spacing criteria developed by Yener[3] and Luttrell[4] could not accurately predict the bending capacity of the tested sections used in this program. A new model for computing the moment capacity was developed using sixty test specimens that had cover plates without edge stiffeners. The criteria developed along with design recommendations is discussed in the next section.

B. DESIGN RECOMMENDATION

The following recommendations were developed from the review of available literature and a test program involving 60 simple span beam test. The model developed from this test program can be applied to a single flute built up section with an unstiffened cover plate that is within the limits of this test program. The following equations can be used to determine the bending capacity of the section:

When $s_t/s_m \leq 3.0$, and $k\ s_t / r < 328$

$$M_n = S_x \sigma_{cr} (\alpha_1) \tag{10}$$

$3.0 < s_t/s_m < 6.0$, and $k\ s_t / r < 328$

$$M_n = S_x \sigma_{cr} (\alpha_2)(\alpha_3) \tag{11}$$

Where:

$\alpha_1 \quad = 0.849 + 0.253(s_t/s_m)$

$\alpha_2 \quad = -9.11 + 4.683(s_t/s_m) - 0.363(s_t/s_m)^2$

$\alpha_3 \quad = 1.634 - 0.464(w/s_t)$

$k_c \quad = 0.6$

$s_t \quad$ = Desired Spacing

$s_m \quad = 1.16t\sqrt{(E/f_c)}$

= Criterion # 2 Section D1.2 AISI Design Specification[2]

$t \quad$ = Thickness of cover plate

$f_c \quad$ = Stress at service load in the cover plate

w = Flat width of cover plate (Figure 1)

S_x = Full section modulus of section about x-axis with respect to the extreme compression fiber.

r = Radius of gyration of cover plate

σ_{cr} = Euler column buckling stress of the cover plate with k = 0.6

The limits of the parameters used to develop this model are given as follows:

$F_y \leq 53$ ksi

$88 \leq w/t \leq 287$

$t \geq 0.017$ in.

$1.0 \leq s_f/s_m \leq 12.0$

$69.0 \leq k_c\, s\, /\, r \leq 328.0$

$1.0 \leq \alpha_1 \leq 1.7$

$3.8 \leq \alpha_2 \leq 6.0$

$0.75 \leq \alpha_3 \leq 1.3$-

VI. FUTURE WORK

Development of one model that covers built-up sections having simple single cell deck sections and multiple cell deck sections will require additional testing. This project gathered information on 83 total tests. These 83 tests included 60 tests without edge stiffened cover plates and 23 sections with edge stiffened cover plates. To develop a model that can accurately predict the bending capacity of all built up sections additional testing needs to be performed on multiple flute sections with and without edge stiffened cover plates and additional tests on single flute sections with edge stiffened cover plates as well as Structural Grade 80 of A653 material. Additional attention should be given to the edge stiffener and unstiffened element of the fluted section. The additional testing can verify the model developed here, or it may verify that one model cannot accurately predict the capacity of all built-up sections in bending with the cover plates in compression.

This test program did not include any test on compression members (columns), but the results of the testing can be used to justify the current method of design outlined in the AISI Design Specification[2]. The results of the test program show that the current spacing requirements in Section D1.2 accurately predict the spacing of connectors to limit column like buckling of the cover plate. Therefore for the design of built up column sections it is recommended that the spacing criteria in the AISI Design Specification[2] be used and the section capacity evaluated following the procedures outlined in Section C4 of the AISI Design Specification[2]. Future testing of built-up column sections might allow the capacity of the section to be increased beyond the stress level at which local buckling of the material between the connectors occurs.

APPENDIX A

YENER'S TEST DATA

Simple Span Beam Tests[3]

Spec. #	sheet gage flat-fluted	thickness (inches)	F_y (ksi)	wire gage	percent perforation	position of cover plate	type of connection	s, inches (strength[a])	s, inches (AISI)	s, inches (proposed)	M_c (k-in)	M_t (k-in)	M_t / M_c
S1	18-18	0.0507	36.3			Down	riveted	5.5	5.5	5.5	29.6	31.0	1.05
S2	18-18	0.0507	36.3			Down	riveted	10.0	10.0	10.0	29.6	36.9	1.24
S3	18-18	0.0507	36.3			Down	riveted	5.5	5.5	5.5	29.6	39.3	1.33
S4	18-18	0.0507	36.3	17		Down	riveted	9.5	9.5	9.5	29.6	34.3	1.16
S5	18-18	0.0507	36.3		25	Down	riveted	6.5	6.5	6.5	28.4	35.9	1.27
S6	18-18	0.0507	36.3		25	Down	riveted	12.5	12.5	12.5	28.4	31.7	1.12
S7	22-22	0.0329	46.6			Down	riveted	5.5[b]	5.5	5.5	16.7	17.7	1.06
S8	22-22	0.0329	46.6			Down	riveted	14.0	14.0	14.0	16.7	18.6	1.11
S9	18-18	0.0489	46.6			Up	riveted	12.0	1.5	4.0	36.7	37.5	1.02
S10	18-18	0.0489	46.6	17		Up	riveted	11.0	1.5	4.0	36.9	39.2	1.06
S11	18-18	0.0489	46.6		25	Up	riveted	13.0	1.5	4.0	35.2	37.2	1.06
S12	22-22	0.0336	43.9			Up	riveted	23.0	1.3	4.0	20.0	19.3	0.97
												Mean	1.12
												C.O.V	0.094

Multiple Span Beam Tests[3]

Spec. #	no. spans	sheet gage flat-fluted	thickness (inches)	F_y (ksi)	wire gage	percent perforation	position of cover plate	type of connection	s, inches (strength) (a)	s, inches (AISI)	s, inches (proposed)	M_c (k-in)	M_t (k-in)	M_t / M_c
C1	3	18-18	0.0489	46.6	17	25	Up	riveted	4.5	1.5	4.0	33.62	42.26	1.26
C2	3	18-18	0.0489	46.6			Up	riveted	5.0	1.5	4.0	32.13	36.32	1.13
C3	3	18-18	0.0489	46.6			Down	riveted	7.0	1.5	4.0	35.78	36.05	1.01
C4	3	22-22	0.0336	43.9			Down	riveted	13.0	1.3	4.0	19.44	19.58	1.01
C5	2	20-18	0.041/0.0515	52.4		25	Down	riveted	12.0	1.5	4.0	38.14	48.77	1.28
C6	2	20-18	0.041/.0515	52.4		25	Down	welded	12.0	1.5	4.0	38.14	45.06	1.18
C7	2	20-18	0.041/.0515	52.4		25	Down	riveted	6.0	1.5	4.0	38.14	45.56	1.19
C8	2	20-18	0.041/.0515	52.4		25	Down	welded	6.0	1.5	4.0	38.14	48.43	1.27
													Mean	1.17
													C.O.V	0.088

[a]Spacing of the test specimens based on ultimated connection strength.

APPENDIX B

LUTTRELL AND BALAJI'S TEST DATA

EP150 Decks in Negative Bending[4]

Test No.	t (inches)	t_b (inches)	s (inches)	Bending M_c (k-in)	M_t/M_c	Deflection I_c (in^4)	I_t/I_c	Note*
1	0.0456	0.0452	8.0	23.39	0.99	0.436	0.96	--
2	0.0454	0.0457	8.0	23.33	1.00	0.437	1.00	W
75	0.0356	0.0459	4.0	21.04	1.08	0.383	1.19	W
76	0.0357	0.0459	6.0	20.41	1.04	0.373	1.07	--
77	0.0351	0.0456	8.0	19.53	1.10	0.357	1.10	--
78	0.0486	0.0578	4.0	26.31	1.17	0.543	1.21	--
79	0.4600	0.0574	6.0	25.33	1.18	0.512	1.17	--
80	0.4630	0.0582	8.0	24.89	1.14	0.498	1.11	BHF
81	0.0459	0.0462	4.0	24.64	1.10	0.468	1.12	BHF
82	0.4550	0.0461	6.0	24.02	1.12	0.452	1.15	BHF
				Average	1.09		1.11	
				C.O.V.	0.057		0.069	

* W: Web crippling noted at ultimate.
BHF: Buckling in hat flange at ultimate.

t_b - thickness of cover plate
t - thickness of deck section

EP300 Decks In Negative Bending.[4]

Test No.	t (inches)	t_b (inches)	s (inches)	Bending M_c (k-in)	M_t/M_c	Deflection I_c (in^4)	I_t/I_c	Note*
37	0.0462	0.0462	8.0	42.86	0.95	1.669	0.81	W
38	0.0457	0.0456	8.0	41.86	1.00	1.647	0.90	W
39	0.0452	0.0447	6.0	44.74	0.93	1.645	0.90	W
40	0.0450	0.0447	6.0	44.52	0.97	1.640	1.06	--
41	0.0454	0.0466	4.0	53.03	0.88	1.715	1.01	--
42	0.0458	0.0466	4.0	53.63	0.87	1.729	0.98	--
43	0.0353	0.0452	8.0	35.14	0.97	1.374	0.96	W
44	0.0355	0.0455	8.0	35.51	0.95	1.383	1.06	W
45	0.0348	0.0460	6.0	38.02	0.98	1.374	0.98	--
46	0.0352	0.0456	6.0	38.15	0.96	1.378	0.97	--
47	0.0351	0.0453	4.0	43.37	0.88	1.387	1.04	--
48	0.0357	0.0455	4.0	44.47	0.84	1.408	1.03	--
49	0.0456	0.0570	8.0	52.96	0.93	1.826	1.00	--
50	0.0453	0.0570	8.0	52.55	0.94	1.817	1.04	W
51	0.0453	0.0572	6.0	55.83	0.88	1.866	0.98	--
52	0.0454	0.0571	6.0	55.99	0.91	1.871	0.95	--
				Average	0.93		0.98	
				C.O.V.	0.048		0.066	

* W: Web crippling noted at ultimate.

t_b - thickness of cover plate
t - thickness of deck section

ECP266 Decks in Negative Bending[4]

Test No.	t (inches)	t_b (inches)	s (inches)	Bending M_c (k-in)	M_t/M_c	Deflection I_c (in^4)	I_t/I_c	Note*
19	0.0464	0.0455	4.0	25.35	1.11	0.735	0.98	--
20	0.0464	0.0456	4.0	25.36	1.13	0.736	0.95	--
21	0.0465	0.0453	6.0	24.74	1.12	0.722	0.92	--
22	0.0466	0.0456	6.0	24.83	1.08	0.725	0.85	--
23	0.0467	0.0456	8.0	24.43	1.10	0.713	0.84	--
24	0:0467	0.0460	8.0	24.46	1.08	0.716	0.83	--
25	0.0478	0.0562	8.0	25.90	1.14	0.792	0.86	--
26	0.0476	0.0570	8.0	24.99	1.13	0.733	0.84	--
27	0.0474	0.0569	6.0	26.26	1.13	0.811	0.89	--
28	0.0476	0.0575	6.0	26.42	1.10	0.781	0.88	--
29	0.0472	0.0562	4.0	26.25	1.17	0.904	0.90	--
30	0.0476	0.0570	4.0	26.48	1.16	0.916	0.92	--
31	0.0344	0.0459	8.0	19.67	0.98	0.570	1.03	--
32	0.0357	0.0459	8.0	20.42	0.99	0.584	0.84	--
33	0.0351	0.0460	6.0	20.41	1.00	0.589	0.94	--
34	0.0353	0.0466	6.0	20.57	0.97	0.595	0.91	--
35	0.0356	0.0463	4.0	21.23	0.96	0.604	0.90	--
36	0.0352	0.0459	4.0	20.95	0.98	0.597	0.90	--
				Average	1.07		0.90	
				C.O.V.	0.066		0.058	

* No web crippling noted.

t_b - thickness of cover plate
t - thickness of deck section

EP450 Decks in Negative Bending.[4]

Test No.	t (inches)	t_b (inches)	s (inches)	Bending M_c (k-in)	M_t/M_c	Deflection I_c (in^4)	I_t/I_c	Note*
53	0.0347	0.0569	4.0	74.24	0.59	3.631	0.99	W
54	0.0342	0.0460	4.0	58.56	0.71	3.217	0.95	W
55	0.0348	0.0460	6.0	52.97	0.74	3.247	0.96	W
56	0.0347	0.0459	6.0	52.73	0.76	3.238	0.85	W
57	0.0349	0.0459	8.0	49.70	0.75	3.265	0.84	W
58	0.0348	0.0459	8.0	49.49	0.74	3.262	0.86	W
59	0.0483	0.0570	8.0	76.99	0.81	4.466	0.97	W
60	0.0481	0.0570	8.0	76.63	0.84	4.453	0.98	W
61	0.0480	0.0573	6.0	87.03	0.89	4.560	1.05	--
62	0.0480	0.0574	4.0	94.89	0.86	4.755	1.00	--
63	0.0478	0.0460	8.0	62.77	1.04	4.057	1.08	--
64	0.0480	0.0458	6.0	68.05	0.97	4.099	1.16	--
65	0.0480	0.0458	4.0	79.60	0.88	4.231	1.16	--
				Average	0.81		0.99	
				C.O.V.	0.139		0.102	

W: Strength limited by web crippling.

t_b - thickness of cover plate
t - thickness of deck section

EPC3 Decks in Negative Bending[4]

Test No.	t (inches)	t_b (inches)	s (inches)	Bending M_c (k-in)	M_t/M_c	Deflection I_c (in^4)	I_t/I_c	Note*
3	0.0497	0.0566	8.0	38.33	1.09	1.492	1.09	--
4	0.0490	0.0564	8.0	37.74	1.06	1.475	1.05	--
5	0.0491	0.0533	6.0	38.40	1.17	1.471	1.10	--
6	0.0486	0.0566	6.0	38.65	1.13	1.510	1.09	--
7	0.0483	0.0569	4.0	39.19	1.14	1.731	1.03	--
8	0.0496	0.0569	4.0	40.21	1.09	1.764	0.98	--
9	0.0352	0.0467	4.0	29.78	1.12	1.103	1.02	--
10	0.0351	0.0454	4.0	29.45	1.02	1.083	1.07	--
11	0.0343	0.0455	6.0	27.91	0.99	1.056	1.05	--
12	0.0345	0.0457	6.0	28.10	1.02	1.063	1.04	--
13	0.0348	0.0457	8.0	27.22	0.98	1.047	0.99	--
14	0.0345	0.0458	8.0	27.56	1.01	1.043	0.98	--
15	0.0494	0.0456	8.0	35.89	1.01	1.331	0.96	--
16	0.0487	0.0456	8.0	35.38	1.00	1.318	0.94	--
17	0.5180	0.0462	6.0	38.91	0.94	1.413	0.91	--
18	0.4950	0.0458	6.0	37.03	0.97	1.363	0.97	--
				Average	1.05		1.02	
				C.O.V.	0.064		0.055	

* No web crippling noted.

t_b - thickness of cover plate

t - thickness of deck section

EP750 Decks in Negative Bending[4]

Test No.	t (inches)	t_b (inches)	s (inches)	Bending M_c (k-in)	M_t/M_c	Deflection I_c (in^4)	I_t/I_c	Note*
66	0.0459	0.0577	8.0	135.83	0.73	13.542	1.15	WS
67	0.0451	0.0579	6.0	144.06	0.80	13.487	1.11	WS
68	0.0458	0.0573	4.0	166.58	0.81	13.935	1.29	BCB
69	0.0340	0.0460	6.0	86.74	0.91	10.594	1.16	BCB
70	0.0341	0.0460	4.0	96.60	0.90	10.246	1.25	BCB
71	0.0342	0.0457	8.0	82.55	0.88	10.773	0.94	BCB
72	0.0464	0.0458	6.0	117.80	0.92	12.770	1.16	BCB
73	0.0458	0.0460	8.0	108.82	0.95	12.812	1.02	BCB
74	0.0455	0.0458	4.0	130.83	0.87	12.575	1.19	BCB
				Average	0.86		1.14	
				C.O.V.	0.076		0.089	

* WS: Web cripling at support.
BCB: Cell ends blocked up eliminating end crippling control.

t_b - thickness of cover plate
t - thickness of deck section

APPENDIX C

COMPARISON OF TESTED CAPACITIES TO THE AISI DESIGN SPECIFICATION AND USING YENER'S SPACING CRITERIA

Test Summary Sections without Edge Stiffened Cover Plates
Comparison to AISI Design Specification[2].

Test No.	Tested Spacing (in)	AISI Spacing (in)	Yener Spacing (in)	w/t	d (in)	w (in)	M_t (k-in)	M_c AISI (k-in)	M_t/M_c AISI
h1t1	1.5	1.5	2.1	78	0.5	3.5	11.90	10.75	1.11
h1t3	1.5	1.5	2.1	78	0.5	3.5	11.90	10.75	1.11
h2t1	1.5	1.5	2.1	78	1.0	3.6	12.50	9.85	1.27
h2t3	1.5	1.5	2.1	78	1.0	3.6	13.00	9.85	1.32
h3t1	1.5	1.5	3.7	137	0.5	9.2	23.00	21.25	1.08
h3t3	1.5	1.5	3.7	137	0.5	9.2	22.60	21.25	1.06
h4t1	1.5	1.5	4.0	198	0.5	9.0	24.00	22.45	1.07
h4t3	1.5	1.5	4.0	198	0.5	9.0	25.20	22.45	1.12
h5t1	1.5	1.5	5.4	198	1.0	8.9	25.70	26.34	0.98
h5t3	1.5	1.5	5.4	198	1.0	8.9	26.80	26.34	1.02
						all test	Mean		1.11
						combined	Coef of Variation		9.01

**See Table II and Figures 6 and 7 for definitions of cross section dimensions.

Test Summary Sections without Edge Stiffened Cover Plates
Comparison to AISI Design Specification[2] Neglecting Section D1.2.

Test No.	Tested Spacing (in)	AISI Spacing (in)	Yener Spacing (in)	w/t	d (in)	w (in)	M_t (k-in)	M_c AISI (k-in)	M_t/M_c AISI
h1t1	1.5	1.5	2.1	78	0.5	3.5	11.90	10.75	1.11
h1t3	1.5	1.5	2.1	78	0.5	3.5	11.90	10.75	1.11
h1t2	3.0	1.5	2.1	78	0.5	3.5	9.80	10.75	0.91
h1t4	3.0	1.5	2.1	78	0.5	3.5	9.90	10.75	0.92
h1t5	6.0	1.5	2.1	78	0.5	3.5	8.70	10.75	0.81
h1t6	6.0	1.5	2.1	78	0.5	3.5	8.50	10.75	0.79
h2t1	1.5	1.5	2.1	78	1.0	3.6	12.50	9.85	1.27
h2t3	1.5	1.5	2.1	78	1.0	3.6	13.00	9.85	1.32
h2t2	3.0	1.5	2.1	78	1.0	3.6	10.80	9.85	1.10
h2t4	3.0	1.5	2.1	78	1.0	3.6	10.40	9.85	1.06
h2t5	6.0	1.5	2.1	78	1.0	3.6	9.40	9.85	0.95
h2t6	6.0	1.5	2.1	78	1.0	3.6	9.30	9.85	0.94
h3t1	1.5	1.5	3.7	137	0.5	9.2	23.00	21.25	1.08
h3t3	1.5	1.5	3.7	137	0.5	9.2	22.60	21.25	1.06
h3t2	3.5	1.5	3.7	137	0.5	9.2	18.40	21.25	0.87
h3t4	3.5	1.5	3.7	137	0.5	9.2	17.60	21.25	0.83
h3t5	3.5	1.5	3.7	137	0.5	9.2	17.80	21.25	0.84
h3t6	6.0	1.5	3.7	137	0.5	9.2	16.30	21.25	0.77
h3t7	6.0	1.5	3.7	137	0.5	9.2	15.10	21.25	0.71
h4t1	1.5	1.5	4.0	198	0.5	9.0	24.00	22.45	1.07
h4t3	1.5	1.5	4.0	198	0.5	9.0	25.20	22.45	1.12
h4t2	4.0	1.5	4.0	198	0.5	9.0	17.90	22.45	0.80
h4t4	4.0	1.5	4.0	198	0.5	9.0	17.90	22.45	0.80
h4t5	4.0	1.5	4.0	198	0.5	9.0	18.60	22.45	0.83
h4t6	6.0	1.5	4.0	198	0.5	9.0	15.70	22.45	0.70
h4t7	6.0	1.5	4.0	198	0.5	9.0	15.80	22.45	0.70
h5t1	1.5	1.5	5.4	198	1.0	8.9	25.70	26.34	0.98
h5t3	1.5	1.5	5.4	198	1.0	8.9	26.80	26.34	1.02
h5t2	4.0	1.5	5.4	198	1.0	8.9	20.40	26.34	0.77
h5t4	4.0	1.5	5.4	198	1.0	8.9	20.20	26.34	0.77
h5t5	6.0	1.5	5.4	198	1.0	8.9	18.70	26.34	0.71
h5t6	6.0	1.5	5.4	198	1.0	8.9	18.60	26.34	0.71
						h-type material	Mean		0.919
							Coef of Variation		0.183

**See Table II and Figures 6 and 7 for definitions of cross section dimensions.

Test Summary Sections without Edge Stiffened Cover Plates
Comparison to AISI Design Specification[2] Neglecting Section D1.2.

Test No.	Tested Spacing (in)	AISI Spacing (in)	Yener Spacing (in)	w/t	d (in)	w (in)	M_t (k-in)	M_c AISI (k-in)	M_t/M_c AISI
gsh1t5	1.5	0.5	1.8	156	0.5	2.962	2.00	3.01	0.66
gsh1t6	1.5	0.5	1.8	156	0.5	2.962	2.30	3.01	0.76
gsh1t3	3.0	0.5	1.8	156	0.5	2.962	1.90	3.01	0.63
gsh1t4	3.0	0.5	1.8	156	0.5	2.962	1.90	3.01	0.63
gsh1t1	6.0	0.5	1.8	156	0.5	2.962	1.70	3.01	0.56
gsh1t2	6.0	0.5	1.8	156	0.5	2.962	1.80	3.01	0.60
gsh2t5	1.5	0.5	1.8	156	1.0	2.962	2.60	3.03	0.86
gsh2t6	1.5	0.5	1.8	156	1.0	2.962	2.30	3.03	0.76
gsh2t3	3.0	0.5	1.8	156	1.0	2.962	2.20	3.03	0.73
gsh2t4	3.0	0.5	1.8	156	1.0	2.962	2.20	3.03	0.73
gsh2t1	6.0	0.5	1.8	156	1.0	2.962	1.80	3.03	0.59
gsh2t2	6.0	0.5	1.8	156	1.0	2.962	1.90	3.03	0.63
gsh3t5	1.5	0.5	2.0	235	0.5	4.462	2.80	4.26	0.66
gsh3t6	1.5	0.5	2.0	235	0.5	4.462	2.60	4.26	0.61
gsh3t3	3.0	0.5	2.0	235	0.5	4.462	2.30	4.26	0.54
gsh3t4	3.0	0.5	2.0	235	0.5	4.462	2.20	4.26	0.52
gsh3t1	6.0	0.5	2.0	235	0.5	4.462	2.10	4.26	0.49
gsh3t2	6.0	0.5	2.0	235	0.5	4.462	2.10	4.26	0.49
gsh4t5	1.5	0.5	2.7	235	1.0	4.462	3.40	4.45	0.76
gsh4t6	1.5	0.5	2.7	235	1.0	4.462	3.30	4.45	0.74
gsh4t3	3.0	0.5	2.7	235	1.0	4.462	3.20	4.45	0.72
gsh4t4	3.0	0.5	2.7	235	1.0	4.462	2.90	4.45	0.65
gsh4t1	6.0	0.5	2.7	235	1.0	4.462	2.60	4.45	0.58
gsh4t2	6.0	0.5	2.7	235	1.0	4.462	2.70	4.45	0.61
						gsh-type material	Mean		0.642
							Coef of Variation		0.166

**See Table II and Figures 6 and 7 for definitions of cross section dimensions.

Test Summary with Edge Stiffened Cover Sheets
Comparison To AISI Design Specification[2] Neglecting Section D1.2.

Test No.	Tested Spacing (in)	Required AISI Spacing (in)	Required Yener Spacing (in)	w/t	d (in)	w (in)	M_t (k-in)	M_c AISI (k-in)	M_t/M_c AISI
h2t7sl	3.5	1.5	2.1	78	1.0	3.6	13.00	9.79	1.33
h2t8sl	3.5	1.5	2.1	78	1.0	3.6	12.80	9.79	1.31
h2t9sl	6.0	1.5	2.1	78	1.0	3.6	12.10	9.79	1.24
h2t10sl	6.0	1.5	2.1	78	1.0	3.6	12.90	9.79	1.32
h3t8sl	6.0	1.5	3.7	137	0.5	9.2	21.5	22.48	0.96
h3t9sl	6.0	1.5	3.7	137	0.5	9.2	21.2	22.48	0.94
gsh1t7sl	1.5	0.5	1.8	156	0.5	2.962	3.40	3.89	0.87
gsh1t8sl	1.5	0.5	1.8	156	0.5	2.962	3.30	3.89	0.85
gsh1t9sl	6.0	0.5	1.8	156	0.5	2.962	3.10	3.89	0.80
gsh1t10sl	6.0	0.5	1.8	156	0.5	2.962	3.20	3.89	0.82
gsh2t7sl	1.5	0.5	1.8	156	1.0	2.962	3.50	3.41	1.03
gsh2t8sl	1.5	0.5	1.8	156	1.0	2.962	3.40	3.41	1.00
gsh2t9sl	6.0	0.5	1.8	156	1.0	2.962	3.20	3.41	0.94
gsh2t10sl	6.0	0.5	1.8	156	1.0	2.962	3.30	3.41	0.97
gsh3t9sl	1.5	0.5	2.0	235	0.5	4.462	4.40	6.24	0.71
gsh3t10sl	1.5	0.5	2.0	235	0.5	4.462	4.50	6.24	0.72
gsh3t7sl	6.0	0.5	2.0	235	0.5	4.462	4.50	6.24	0.72
gsh3t8sl	6.0	0.5	2.0	235	0.5	4.462	4.30	6.24	0.69
gsh4t7sl	1.5	0.5	2.7	235	1.0	4.462	5.00	7.16	0.70
gsh4t8sl	1.5	0.5	2.7	235	1.0	4.462	4.90	7.16	0.68
gsh4t9sl	6.0	0.5	2.7	235	1.0	4.462	4.40	7.16	0.61
gsh4t10sl	6.0	0.5	2.7	235	1.0	4.462	4.00	7.16	0.56

all test combined	Mean	0.898
	Coef of Variation	25.18
h-type material	Mean	1.181
	Coef of Variation	14.09
gsh-type material	Mean	0.791
	Coef of Variation	17.15

**See Table II and Figures 6 and 7 for definitions of cross section dimensions.

APPENDIX D

COMPARISONS OF TESTED MOMENT CAPACITIES TO CALCULATED CAPACITIES USING LUTTRELL'S MODIFIED EFFECTIVE WIDTH MODEL

Test Summary Section without Edge Stiffened Cover Plates, Luttrell's Model[4].

Test No.	Tested Spacing (in)	AISI Spacing (in)	w/t	d (in)	w (in)	M_t (k-in)	M_c Luttrell (k-in)	M_T/M_c
h1t1	1.5	1.5	78	0.5	3.5	11.90	10.75	1.11
h1t3	1.5	1.5	78	0.5	3.5	11.90	10.75	1.11
h1t2	3.0	1.5	78	0.5	3.5	9.80	9.02	1.09
h1t4	3.0	1.5	78	0.5	3.5	9.90	9.02	1.10
h1t5	6.0	1.5	78	0.5	3.5	8.70	7.70	1.13
h1t6	6.0	1.5	78	0.5	3.5	8.50	7.70	1.10
h2t1	1.5	1.5	78	1.0	3.6	12.50	9.85	1.27
h2t3	1.5	1.5	78	1.0	3.6	13.00	9.85	1.32
h2t2	3.0	1.5	78	1.0	3.6	10.80	9.85	1.10
h2t4	3.0	1.5	78	1.0	3.6	10.40	9.85	1.06
h2t5	6.0	1.5	78	1.0	3.6	9.40	9.58	0.98
h2t6	6.0	1.5	78	1.0	3.6	9.30	9.58	0.97
h3t1	1.5	1.5	137	0.5	9.2	23.00	21.24	1.08
h3t3	1.5	1.5	137	0.5	9.2	22.60	21.24	1.06
h3t2	3.5	1.5	137	0.5	9.2	18.40	17.14	1.07
h3t4	3.5	1.5	137	0.5	9.2	17.60	17.14	1.03
h3t5	3.5	1.5	137	0.5	9.2	17.80	17.14	1.04
h3t6	6.0	1.5	137	0.5	9.2	16.30	16.61	0.98
h3t7	6.0	1.5	137	0.5	9.2	15.10	16.61	0.91
h4t1	1.5	1.5	198	0.5	9.0	24.00	22.45	1.07
h4t3	1.5	1.5	198	0.5	9.0	25.20	22.45	1.12
h4t2	4.0	1.5	198	0.5	9.0	17.90	18.09	0.99
h4t4	4.0	1.5	198	0.5	9.0	17.90	18.09	0.99
h4t5	4.0	1.5	198	0.5	9.0	18.60	18.09	1.03
h4t6	6.0	1.5	198	0.5	9.0	15.70	17.78	0.88
h4t7	6.0	1.5	198	0.5	9.0	15.80	17.78	0.89
h5t1	1.5	1.5	198	1.0	8.9	25.70	26.33	0.98
h5t3	1.5	1.5	198	1.0	8.9	26.80	26.33	1.02
h5t2	4.0	1.5	198	1.0	8.9	20.40	22.00	0.93
h5t4	4.0	1.5	198	1.0	8.9	20.20	22.00	0.92
h5t5	6.0	1.5	198	1.0	8.9	18.70	21.70	0.86
h5t6	6.0	1.5	198	1.0	8.9	18.60	21.70	0.86

h-type material	Mean	1.032
	Coef of Variation	0.101
all test combined	Mean	0.903
	Coef of Variation	0.196

** See Table II and Figures 6 and 7 for definition of cross section dimensions.

Test Summary Section without Edge Stiffened Cover Plates, Luttrell's Model[4].

Test No.	Tested Spacing (in)	AISI Spacing (in)	w/t	d (in)	w (in)	M_t (k-in)	M_c Luttrell (k-in)	M_t/M_c
gsh1t5	1.5	0.5	156	0.5	2.962	2.00	2.81	0.71
gsh1t6	1.5	0.5	156	0.5	2.962	2.30	2.81	0.82
gsh1t3	3.0	0.5	156	0.5	2.962	1.90	2.72	0.70
gsh1t4	3.0	0.5	156	0.5	2.962	1.90	2.72	0.70
gsh1t1	6.0	0.5	156	0.5	2.962	1.70	2.50	0.68
gsh1t2	6.0	0.5	156	0.5	2.962	1.80	2.50	0.72
gsh2t5	1.5	0.5	156	1.0	2.962	2.60	2.84	0.92
gsh2t6	1.5	0.5	156	1.0	2.962	2.30	2.84	0.81
gsh2t3	3.0	0.5	156	1.0	2.962	2.20	2.75	0.80
gsh2t4	3.0	0.5	156	1.0	2.962	2.20	2.75	0.80
gsh2t1	6.0	0.5	156	1.0	2.962	1.80	2.53	0.71
gsh2t2	6.0	0.5	156	1.0	2.962	1.90	2.53	0.75
gsh3t5	1.5	0.5	235	0.5	4.462	2.80	3.81	0.73
gsh3t6	1.5	0.5	235	0.5	4.462	2.60	3.81	0.68
gsh3t3	3.0	0.5	235	0.5	4.462	2.30	3.73	0.62
gsh3t4	3.0	0.5	235	0.5	4.462	2.20	3.73	0.59
gsh3t1	6.0	0.5	235	0.5	4.462	2.10	3.58	0.59
gsh3t2	6.0	0.5	235	0.5	4.462	2.10	3.58	0.59
gsh4t5	1.5	0.5	235	1.0	4.462	3.40	3.99	0.85
gsh4t6	1.5	0.5	235	1.0	4.462	3.30	3.99	0.83
gsh4t3	3.0	0.5	235	1.0	4.462	3.20	3.91	0.82
gsh4t4	3.0	0.5	235	1.0	4.462	2.90	3.91	0.74
gsh4t1	6.0	0.5	235	1.0	4.462	2.60	3.76	0.69
gsh4t2	6.0	0.5	235	1.0	4.462	2.70	3.76	0.72
					gsh-type material	Mean		0.729
						Coef of Variation		0.135
					all test combined	Mean		0.903
						Coef of Variation		0.196

** See Table II and Figures 6 and 7 for definitions of cross section dimensions

Test Summary, Section with Edge Stiffened Cover Plates, Comparison with Luttrell's Model[4].

Test No.	Tested Spacing (in)	Required AISI Spacing (in)	w/t	d (in)	w (in)	M_t (k-in)	M_c Luttrell (k-in)	M_t/M_c
h2t7sl	3.5	1.5	78	1.0	3.6	13.00	10.09	1.29
h2t8sl	3.5	1.5	78	1.0	3.6	12.80	10.09	1.27
h2t9sl	6.0	1.5	78	1.0	3.6	12.10	9.79	1.24
h2t10sl	6.0	1.5	78	1.0	3.6	12.90	9.79	1.32
h3t8sl	6.0	1.5	137	0.5	9.2	21.5	17.87	1.20
h3t9sl	6.0	1.5	137	0.5	9.2	21.2	17.87	1.19
gsh1t7sl	1.5	0.5	156	0.5	2.962	3.40	3.85	0.88
gsh1t8sl	1.5	0.5	156	0.5	2.962	3.30	3.85	0.86
gsh1t9sl	6.0	0.5	156	0.5	2.962	3.10	3.62	0.86
gsh1t10sl	6.0	0.5	156	0.5	2.962	3.20	3.62	0.88
gsh2t7sl	1.5	0.5	156	1.0	2.962	3.50	3.41	1.03
gsh2t8sl	1.5	0.5	156	1.0	2.962	3.40	3.41	1.00
gsh2t9sl	6.0	0.5	156	1.0	2.962	3.20	3.33	0.96
gsh2t10sl	6.0	0.5	156	1.0	2.962	3.30	3.33	0.99
gsh3t9sl	1.5	0.5	235	0.5	4.462	4.40	6.24	0.71
gsh3t10sl	1.5	0.5	235	0.5	4.462	4.50	6.24	0.72
gsh3t7sl	6.0	0.5	235	0.5	4.462	4.50	5.49	0.82
gsh3t8sl	6.0	0.5	235	0.5	4.462	4.30	5.49	0.78
gsh4t7sl	1.5	0.5	235	1.0	4.462	5.00	6.79	0.74
gsh4t8sl	1.5	0.5	235	1.0	4.462	4.90	6.79	0.72
gsh4t9sl	6.0	0.5	235	1.0	4.462	4.40	6.59	0.67
gsh4t10sl	6.0	0.5	235	1.0	4.462	4.00	6.59	0.61

all test combined	Mean	0.942
	Coef of Variation	0.231
h-type material	Mean	1.250
	Coef of Variation	0.037
gsh-type material	Mean	0.826
	Coef of Variation	0.149

** See Table II and Figures 6 and 7 for definitions of cross section dimensions.

APPENDIX E

COMPARISONS OF TESTED CAPACITIES TO THE UMR MODEL.

UMR Model, based on $\sigma_{cr}S_x$. Sections without Edge Stiffened Cover Plates.

Test No	Spacing (in)	AISI[2] Req'd spacing	st/sm	M_t (k-in)	M_{c1} (k-in)	M_{c2} (k-in)	M_t/M_{c1}	M_t/M_{c2}
h5t1	1.5	1.5	1.00	25.70	26.33	29.02	0.98	0.89
h5t3	1.5	1.5	1.00	26.80	26.33	29.02	1.02	0.92
h3t3	1.5	1.5	1.00	22.60	21.24	23.41	1.06	0.97
h4t1	1.5	1.5	1.00	24.00	22.45	24.74	1.07	0.97
h3t1	1.5	1.5	1.00	23.00	21.24	23.41	1.08	0.98
h1t3	1.5	1.5	1.00	11.90	10.75	11.85	1.11	1.00
h1t1	1.5	1.5	1.00	11.90	10.75	11.85	1.11	1.00
h4t3	1.5	1.5	1.00	25.20	22.45	24.74	1.12	1.02
h2t1	1.5	1.5	1.00	12.50	9.85	10.85	1.27	1.15
h2t3	1.5	1.5	1.00	13.00	9.85	10.85	1.32	1.20
h2t4	3.0	1.5	2.00	10.40	8.32	11.27	1.25	0.92
h2t2	3.0	1.5	2.00	10.80	8.32	11.27	1.30	0.96
h1t2	3.0	1.5	2.00	9.80	6.85	9.29	1.43	1.05
h1t4	3.0	1.5	2.00	9.90	6.85	9.29	1.44	1.07
h3t4	3.5	1.5	2.33	17.60	12.33	17.75	1.43	0.99
h3t5	3.5	1.5	2.33	17.80	12.33	17.75	1.44	1.00
h3t2	3.5	1.5	2.33	18.40	12.33	17.75	1.49	1.04
h4t2	4.0	1.5	2.67	17.90	12.63	19.26	1.42	0.93
h4t4	4.0	1.5	2.67	17.90	12.63	19.26	1.42	0.93
h5t4	4.0	1.5	2.67	20.20	14.00	21.33	1.44	0.95
h5t2	4.0	1.5	2.67	20.40	14.00	21.33	1.46	0.96
h4t5	4.0	1.5	2.67	18.60	12.63	19.26	1.47	0.97
gsh3t6	1.5	0.5	3.00	2.60	1.96	3.15	1.33	0.82
gsh4t6	1.5	0.5	3.00	3.30	2.33	3.75	1.42	0.88
gsh3t5	1.5	0.5	3.00	2.80	1.96	3.15	1.43	0.89
gsh4t5	1.5	0.5	3.00	3.40	2.33	3.75	1.46	0.91
gsh2t6	1.5	0.5	3.00	2.30	1.32	2.12	1.74	1.08
gsh1t5	1.5	0.5	3.00	2.00	1.06	1.71	1.89	1.17
gsh2t5	1.5	0.5	3.00	2.60	1.32	2.12	1.97	1.22
gsh1t6	1.5	0.5	3.00	2.30	1.06	1.71	2.17	1.35
h4t6	6.0	1.5	4.00	15.70	5.61	19.24	2.80	0.82
h4t7	6.0	1.5	4.00	15.80	5.61	19.24	2.81	0.82
h5t6	6.0	1.5	4.00	18.60	6.22	20.58	2.99	0.90
h5t5	6.0	1.5	4.00	18.70	6.22	20.58	3.01	0.91
h3t7	6.0	1.5	4.00	15.10	4.20	14.13	3.60	1.07
h3t6	6.0	1.5	4.00	16.30	4.20	14.13	3.88	1.15
h2t6	6.0	1.5	4.00	9.30	2.08	10.12	4.47	0.92
h2t5	6.0	1.5	4.00	9.40	2.08	10.12	4.52	0.93
h1t6	6.0	1.5	4.00	8.50	1.71	8.65	4.96	0.98
h1t5	6.0	1.5	4.00	8.70	1.71	8.65	5.08	1.01
gsh3t4	3.0	0.5	6.00	2.20	0.49	2.51	4.49	0.88
gsh3t3	3.0	0.5	6.00	2.30	0.49	2.51	4.69	0.92
gsh4t4	3.0	0.5	6.00	2.90	0.58	2.70	5.00	1.07
gsh4t3	3.0	0.5	6.00	3.20	0.58	2.70	5.52	1.18
gsh2t3	3.0	0.5	6.00	2.20	0.33	1.99	6.67	1.10
gsh2t4	3.0	0.5	6.00	2.20	0.33	1.99	6.67	1.10
gsh1t3	3.0	0.5	6.00	1.90	0.27	1.73	7.14	1.10
gsh1t4	3.0	0.5	6.00	1.90	0.27	1.73	7.14	1.10
					Mean		2.450	1.003
					Coef of Variation		0.667	0.113

Column Explanation:

M_{c1} - capacity based on $\sigma_{cr}S_x$ only.

M_{c2} - Column 1 adjusted using α_1, α_2, α_3

APPENDIX F

TABLES OF ALL STRAIN GAGE DATA

H-type Material, k_c Determination, Cover Plates without Edge Stiffeners

test #	gage #	buckle Load	s	w_h	k_c	avg Individual test	avg all tests
h3t3	1, 2	1707	3.5	6.7	0.93	0.80	0.81
	3, 4	2293	3.5	6.7	0.80		
	5, 6	2187	3.5	6.7	0.82		
	7, 8	3387	3.5	6.7	0.66		
h3t4	1, 2	3970	3.5	6.7	0.61	0.78	
	3, 4	2110	3.5	6.7	0.84		
	5, 6	2360	3.5	6.7	0.79		
	7, 8	1990	3.5	6.7	0.86		
h3t5	1, 2	1710	3.5	6.7	0.93	0.80	
	3, 4	2290	3.5	6.7	0.80		
	5, 6	2190	3.5	6.7	0.82		
	7, 8	3390	3.5	6.7	0.66		
h4t4	1, 2	2130	4.0	9.5	0.84	0.83	
	3, 4	2130	4.0	9.5	0.84		
	5, 6	2630	4.0	9.5	0.76		
	7, 8	2010	4.0	9.5	0.87		
h4t5	1, 2	1880	4.0	9.5	0.90	0.85	
	3, 4	2500	4.0	9.5	0.78		
	5, 6	2010	4.0	9.5	0.87		
	7, 8	2130	4.0	9.5	0.84		
h4t6	1, 2	1138	6.0	9.5	0.77	0.84	
	3, 4	810	6.0	9.5	0.91		
	5, 6	1138	6.0	9.5	0.77		
	7, 8	810	6.0	9.5	0.91		
h4t7	1, 2	1247	6.0	9.5	0.74	0.74	
	3, 4	1247	6.0	9.5	0.74		
	5, 6	1247	6.0	9.5	0.74		
	7, 8	1247	6.0	9.5	0.74		

Gsh-type Material, k_c Determination, Cover Plates without Edge Stiffeners.

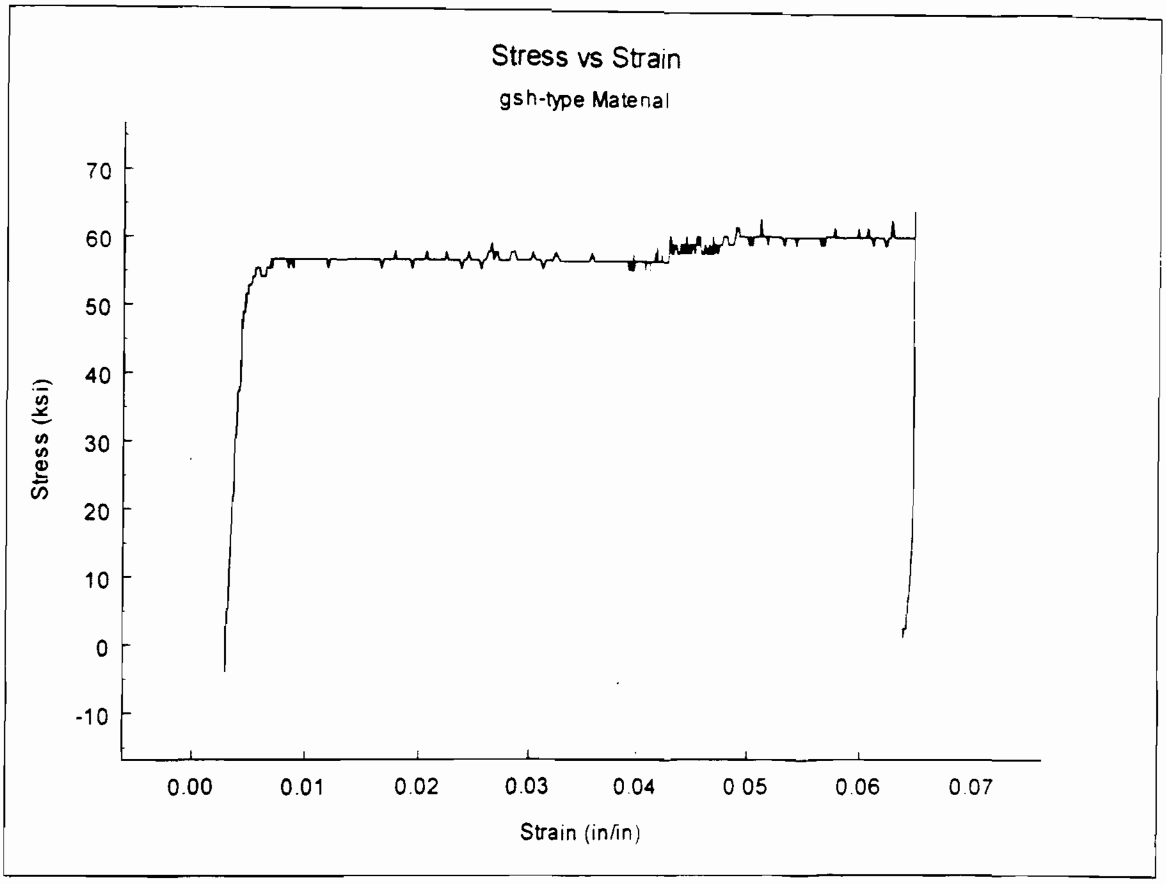
test #	gage #	buckle Load	s	w_h	k_c	avg Individual test	avg all tests
gsh1t3	1, 2	*	3	2.962	*	0.49	0.43
	3, 4	58	3	2.962	0.44		
	5, 6	49	3	2.962	0.48		
	7, 8	39	3	2.962	0.54		
gsh1t4	1, 2	48	3	2.962	0.48	0.47	
	3, 4	59	3	2.962	0.44		
	5, 6	78	3	2.962	0.38		
	7, 8	35	3	2.962	0.57		
gsh2t3	1, 2	I.B.	3	2.962	***		
	3, 4	I.B.	3	2.962	***		
	5, 6	*	3	2.962	*		
	7, 8	I.B.	3	2.962	***		
gsh3t1	1, 2	I.B.	6	4.462	***	0.30	
	3, 4	I.B.	6	4.462	***		
	5, 6	I.B.	6	4.462	***		
	7, 8	68	6	4.462	0.3		
gsh3t2	1, 2	I.B.	6	4.462	***	0.33	
	3, 4	63	6	4.462	0.31		
	5, 6	54	6	4.462	0.34		
	7, 8	I.B.	6	4.462	***		
gsh4t1	1, 2	I.B.	6	4.462	***	0.30	
	3, 4	*	6	4.462	*		
	5, 6	78	6	4.462	0.3		
	7, 8	I.B.	6	4.462	***		
gsh4t2	1, 2	54	6	4.462	0.37	0.37	
	3, 4	I.B.	6	4.462	***		
	5, 6	I.B.	6	4.462	***		
	7, 8	I.B.	6	4.462	***		
gsh4t1	1, 2	I.B.	3	4.462	***	0.47	
	3, 4	*	3	4.462	*		
	5, 6	I.B.	3	4.462	***		
	7, 8	112	3	4.462	0.47		
gsh4t2	1, 2	I.B.	3	4.462	***	0.72	
	3, 4	48	3	4.462	0.72		
	5, 6	*	3	4.462	***		
	7, 8	I.B.	3	4.462	***		

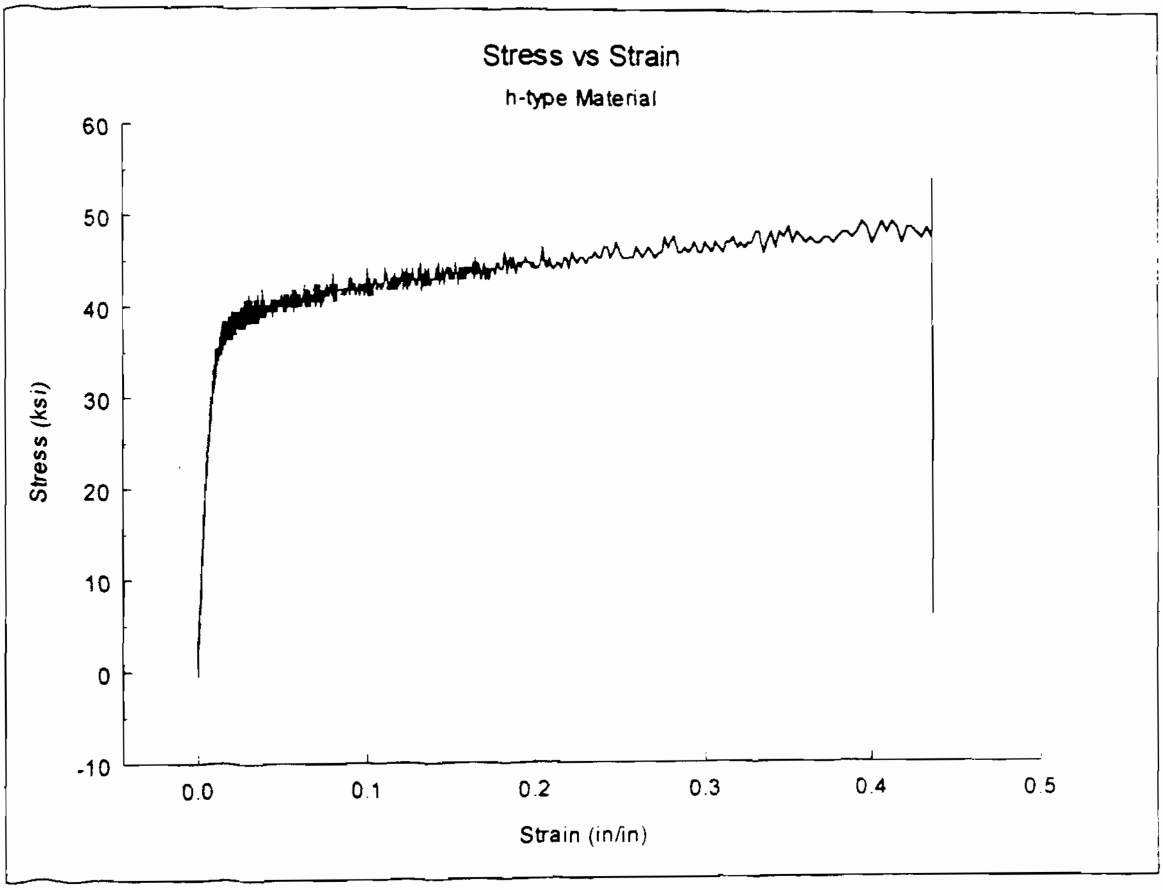
Gsh-type Material, k_c and k Determination, Cover Plates with Edge Stiffeners

					column buckling			Plate Buckling		
test #	gage #	buckle Load	s	w_c	k_c	avg Individual test	avg all tests	k	avg Individual test	avg all tests
gsh1t9	1, 2	103	6.0	4.0	0.3	0.36	0.43	1.14	0.80	0.83
	3, 4	73	6.0	4.0	0.35			0.81		
	5, 6	54	6.0	4.0	0.41			0.6		
	7, 8	59	6.0	4.0	0.39			0.65		
gsh1t10	1, 2	98	6.0	4.0	0.31	0.31		1.09	1.08	
	3, 4	83	6.0	4.0	0.33			0.92		
	5, 6	112	6.0	4.0	0.29			1.24		
	7, 8	*	6.0	4.0	*			*		
gsh2t9	1, 2	I.B.	6.0	4.5	***	0.29		***	1.36	
	3, 4	93	6.0	4.5	0.28			1.43		
	5, 6	83	6.0	4.5	0.3			1.28		
	7, 8	*	6.0	4.5	*			*		
gsh2t10	1, 2	*	6.0	4.5	*	0.29		*	1.38	
	3, 4	78	6.0	4.5	0.31			1.2		
	5, 6	73	6.0	4.5	0.32			1.13		
	7, 8	117	6.0	4.5	0.25			1.8		
gsh3t7	1, 2	*	6.0	5.6	*	0.58		*	0.46	
	3, 4	112	6.0	5.6	0.49			0.59		
	5, 6	I.B.	6.0	5.6	***			***		
	7, 8	63	6.0	5.6	0.66			0.33		
gsh3t8	1, 2	103	6.0	5.6	0.51	0.60		0.54	0.41	
	3, 4	59	6.0	5.6	0.68			0.31		
	5, 6	68	6.0	5.6	0.63			0.36		
	7, 8	78	6.0	5.6	0.59			0.41		

APPENDIX G

TESTED MATERIAL STRESS VS STRAIN CURVES


Stress vs Strain
gsh-type Matenal
Stress (ksi)
70
60
50
40
30
20
10
0
-10
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
Strain (in/in)



Stress vs Strain
h-type Material
Stress (ksi)
60
50
40
30
20
10
0
-10
0.0
0.1
0.2
0.3
0.4
0.5
Strain (in/in)

APPENDIX H

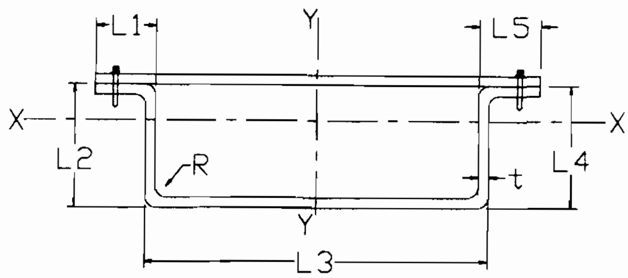
EXAMPLE PROBLEMS

Example Problem 1

Section: gsh4s3

Properties:

$L1 = 1.0$ in.
$L2 = 2.0$ in.
$L3 = 3.5$ in.
$L4 = 2.0$ in.
$L5 = 1.0$ in.
$t = 0.0174$ in.
$R = 0.0625$ in.
$s_t = 3.0$ in.
$F_y = 53.0$ ksi
Plate width = 5.465 in.



Calculation of Properties of the 90 degree corners:

$$r = R + t/2 = 0.0625 + 0.0174/2 = 0.0172 \text{ in.}$$

$$u = 1.57(0.0172) = 0.1118 \text{ in.}$$

$$c = 0.637(0.0172) = 0.0454 \text{ in.}$$

Full Section Properties calculated using the Linear Method:

Length of Elements:

Web 2 $= L2 - 2(R + t)$
$= 2.0 - 2(0.0625 + 0.0174) = 1.84$ in.

Web 4 $= L4 - 2(R + t)$
$= 2.0 - 2(0.0625 + 0.0174) = 1.84$ in.

Flange 1 $= L1 - (R + t)$
$= 1.0 - (0.0625 + 0.0174) = 0.92$ in.

Flange 5 $= L5 - (R + t)$
$= 1.0 - (0.0625 + 0.0174) = 0.92$ in.

Hat Base $= L3 - 2(R + t)$

$= 3.5 - 2(0.0625 + 0.0174) \quad = \quad 3.34$ in.

Element	**L Length**	**y Distance from Top fiber (in.)**	**Ly (in.^{2})**	**Ly^2 (in.^{3})**	**I'$_1$ About Own Axis (in.^{3})**
Flat plate	5.465	0.009	0.048	0.000	0.000
Web 2	1.840	1.017	1.872	1.905	0.519
Web 4	1.840	1.017	1.872	1.905	0.519
Flange 1	0.920	0.026	0.024	0.001	0.000
Flange 5	0.920	0.026	0.024	0.001	0.000
Hat base	3.340	2.009	6.709	13.477	0.000
90° corners	0.224	0.063	0.014	0.001	0.000
90° corners	0.224	1.972	0.441	0.869	0.000
Sum	14.77		11.004	18.159	1.038

Location of Center of Gravity of Section:

$$y_{cg} = Ly/L = 11.004/14.77 = 0.745 \text{ in.}$$

Calculation of Moment of Inertia and Section Modulus:

$$I_x' = Ly^2 + I'_1 - L(y_{cg})^2 = 18.159 + 1.038 - 14.77(0.745)^2 = 10.999 \text{ in.}$$

$$I_x = I_x'(t) = 10.999(0.0174) = 0.191 \text{ in.}^4$$

$S_x = I_x/c$ :: About extreme compression fiber, i.e. $c = y_{cg}$

$$= 0.191/(0.745) = 0.255 \text{ in.}^3$$

Calculation of Critical Buckling Stress of Flat Plate:

$$
\begin{aligned}
r^2 &= t^2/12 \\
&= 0.0174^2/12 \\
&= 0.005 \text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
\sigma_{cr} &= \pi^2\, E \,/\, (k_c\, s_t \,/\, r)^2 \\
&= \pi^2(29{,}500)(0.005) \,/\, (0.6(3))^2 \\
&= 2.267 \text{ ksi}
\end{aligned}
$$

Calculation of S_m:

$$
\begin{aligned}
s_m &= 1.16t\,\sqrt{(E/f_c)} \\
&= 1.16(0.0174)\,\sqrt{(29{,}000/53)} \\
&= 0.5 \text{ in.}
\end{aligned}
$$

Calculation of Moment Capacity:

$$
\begin{aligned}
s_t/s_m &= 3.0/0.5 \\
&= 6.0 \qquad \text{:: use Equation 11}
\end{aligned}
$$

$$
\begin{aligned}
\alpha_2 &= -9.11 + 4.683(s_t/s_m) - 0.363(s_t/s_m)^2 \\
&= -9.11 + 4.683(6.0) - 0.363(6)^2 \\
&= 5.92
\end{aligned}
$$

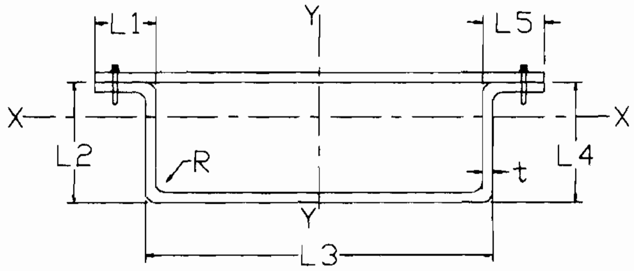
$$
\begin{aligned}
\alpha_3 &= 1.634 - 0.464(w/s_t) \\
&= 1.634 - 0.464(5.465/3) \\
&= 0.789
\end{aligned}
$$

$$
\begin{aligned}
Mn &= S_x\sigma_{cr}(\alpha_2)(\alpha_3) \\
&= 0.255(2.267)(5.92)(0.789) \\
&= 2.7 \text{ in. k}
\end{aligned}
$$

Example Problem 2

Section: h1s3

Properties:



$L1 = 0.469$ in.
$L2 = 2.0$ in.
$L3 = 3.1$ in.
$L4 = 2.0$ in.
$L5 = 0.484$ in.
$t = 0.0452$ in.
$R = 0.0625$ in.
$s_t = 3.0$ in.
$F_y = 53.0$ ksi
Plate width = 3.963 in.

Calculation of Properties of the 90 degree corners:

$$r = R + t/2 = 0.0625 + 0.0452/2 = 0.0851 \text{ in.}$$

$$u = 1.57(0.0851) = 0.1336 \text{ in.}$$

$$c = 0.637(0.0851) = 0.0542 \text{ in.}$$

Full Section Properties calculated using the Linear Method:

Length of Elements:

$$\text{Web 2} = L2 - 2(R + t) = 2.0 - 2(0.0625 + 0.0452) = 1.785 \text{ in.}$$

$$\text{Web 4} = L4 - 2(R + t) = 2.0 - 2(0.0625 + 0.0452) = 1.785 \text{ in.}$$

$$\text{Flange 1} = L1 - (R + t) = 0.469 - (0.0625 + 0.0452) = 0.361 \text{ in.}$$

$$\text{Flange 5} = L5 - (R + t)$$

$= 0.484 - (0.0625 + 0.0452) \quad = 0.376$ in.

Hat Base $= L3 - 2(R + t)$
$= 3.1 - 2(0.0625 + 0.0452) \quad = 2.885$ in.

Element	L Length	y Distance from Top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'$_1$ About Own Axis (in.^{3})
Flat plate	3.963	0.023	0.090	0.002	0.000
Web 2	1.785	1.045	1.865	1.950	0.474
Web 4	1.785	1.045	1.865	1.950	0.474
Flange 1	0.361	0.068	0.024	0.002	0.000
Flange 5	0.376	0.068	0.026	0.002	0.000
Hat base	2.885	2.023	5.834	11.801	0.000
90° corners	0.267	0.099	0.027	0.003	0.000
90° corners	0.267	1.991	0.532	1.059	0.000
Sum	11.69		10.263	16.769	0.948

Location of Center of Gravity of Section:

$y_{cg} = Ly/L$
$= 10.263/11.69$
$= 0.878$ in.

Calculation of Moment of Inertia and Section Modulus:

$I_x' = Ly^2 + I'_1 - L(y_{cg})^2$
$= 16.769 + 0.948 - 11.69(0.878)^2$
$= 8.705$ in.

$I_x = I_x'(t)$
$= 8.703(0.0452)$
$= 0.393$ in.^{4}

$$S_x = I_x/c \quad \text{:: About extreme compression fiber, i.e. } c = y_{cg}$$
$$= 0.393/(0.878)$$
$$= 0.448 \text{ in.}^3$$

Calculation of Critical Buckling Stress of Flat Plate:

$$r^2 = t^2/12$$
$$= 0.0452^2/12$$
$$= 0.00017 \text{ in.}$$

$$\sigma_{cr} = \pi^2 E / (k_c s_t / r)^2$$
$$= \pi^2(29{,}500)(0.00017) / (0.6(3))^2$$
$$= 15.30 \text{ ksi}$$

Calculation of S_m:

$$s_m = 1.16t\sqrt{(E/f_c)}$$

$$= 1.16(0.0452)\sqrt{(29{,}000/33)}$$

$$= 1.5 \text{ in.}$$

Calculation of Moment Capacity:

$$s_t/s_m = 3.0/1.5$$
$$= 2.0 \quad \text{:: use Equation 10}$$

$$\alpha_1 = 0.848 + 0.253(s_t/s_m)$$
$$= 0.848 + 0.253(2)$$
$$= 1.354$$

$$Mn = S_x\sigma_{cr}(\alpha_1)$$
$$= 0.448(15.3)(1.354)$$
$$= 9.29 \text{ in. k}$$

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