STRENGTH OF SINGLY SYMMETRIC STEEL BEAMS
WITH NON-COMPACT AND SLENDER COMPRESSION ELEMENTS

by

Kevin Spencer Moore, B.S.

THESIS

Presented to the Faculty of the Graduate School

of The University of Texas at Austin

in Partial Fulfillment

of the Requirements

for the Degree of

MASTER OF SCIENCE IN ENGINEERING

The University of Texas at Austin

May 1995

STRENGTH OF SINGLY SYMMETRIC STEEL BEAMS

WITH NON-COMPACT AND SLENDER COMPRESSION ELEMENTS

APPROVED BY

SUPERVISING COMMITTEE:

Joseph A. Yura

Michael D. Engelhardt

ACKNOWLEDGMENTS

This research program was conducted at the Phil M. Ferguson Structural Engineering Laboratory, located at the J.J. Pickle's research center of The University of Texas at Austin. Funding for the program was provided by Morris Engineering, Inc., Dallas, Texas. Deep gratitude is extended to those who enabled this research to be conducted.

The author would like to express his deep appreciation and gratitude to Dr. Joseph A. Yura for his undying patience, assistance and guidance provided throughout his study at The University of Texas at Austin. The author would also like to thank Dr. Michael D. Engelhardt and Andrew Haines for their unselfish help throughout various stages of the research. Gratitude is also extended to all the faculty and staff of FSEL who contributed in every way to the research contained in this thesis.

I would like to thank my parents for the tremendous support, guidance and unselfish love they gave me throughout my studies at The University of Texas at Austin, and throughout my life. This thesis is a product much larger than the sum of its parts. Through my personal advancement and knowledge, words do not aptly describe the gratitude I feel towards all who have helped me achieve this point in my life. This said, I would like to extend a humble thank you.

Kevin Spencer Moore

January 3, 1995

ABSTRACT

STRENGTH OF SINGLY SYMMETRIC STEEL BEAMS WITH NON-COMPACT AND SLENDER COMPRESSION ELEMENTS

by

Kevin Spencer Moore, M.S.E.

The University of Texas at Austin, 1995

Supervisor: Joseph A. Yura

Many pre-engineered buildings utilize very efficient steel plate girders with different size flanges, referred to as singly symmetric beams. An unusual amount of structures with these types of members collapsed in a 1992 snowstorm in the eastern United States of America. This research program was conducted to determine the capacity of undamaged typical singly symmetric sections taken from a collapsed structure. An experimentation program of five tests was conducted on four specimens. Load-displacement behavior was recorded, experimental behavior and capacity was compared to predictions according to the American Institute of Steel Construction Specification as well as theoretical predictions. Local buckling behavior of compression elements for singly symmetric sections and a critique of the design specification were the major focus of the research program.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Table 3.1
Section Dimensions

		#9	#10	#11 (Test 1 & 2)	#11 (Test 5)
Top Flange	w (in)	6.037	6.050	5.977	5.968
	t (in)	0.628	0.450	0.215	0.235
Bottom Flange	w (in)	5.962	5.945	6.005	5.979
	t (in)	0.378	0.374	0.625	0.627
web	h (in)	23.807	23.801	23.910	23.888
	t (in)	0.135	0.155	0.198	0.196
	S_{xc} (in^3)	96.272	76.897	57.664	59.762
	S_{xt} (in^3)	70.294	68.921	93.293	94.023
	y_t (in)	9.844	11.198	15.081	14.682
	y_b (in)	14.341	12.986	9.454	9.618

Table 3.2 Summary of Averages for Tension Tests

	Area	Gage Length	Percent Elongation	Static Yield	Ultimate Stress	Strain at Strain Hardening
	in^2	in	%	ksi	ksi	in/in
Section #11, Untested						
Top Flange	0.112	2.00	27.9	**55.8**	82.8	0.010
Bottom Flange	0.315	2.00	31.9	**58.3**	79.3	0.006
Web	0.097	2.00	28.8	**47.8**	67.2	0.012
Section #11, Tests 1 & 2						
Top Flange	0.233	4.99	22.4	**55.1**	78.8	0.015
Bottom Flange	0.956	8.00	21.1	**55.5**	65.5	0.013
Web	0.305	7.99	24.4	**40.7**	59.2	0.014
Section #10, Test 3						
Top Flange	0.221	2.00	28.4	**58.1**	88.4	0.011
Bottom Flange	0.193	2.00	26.6	**51.3**	80.5	0.016
Web	0.099	2.00	21.9	**45.8**	66.4	0.021
Section #9, Test 4						
Top Flange	0.314	2.00	34.4	**50.9**	83.9	0.011
Bottom Flange	0.186	2.00	33.6	**52.5**	82.6	0.012
Web	0.063	2.00	21.1	**50.2**	64.9	0.025
Section #11, Test 5						
Top Flange	0.112	2.00	28.2	**58.0**	87.7	0.016
Bottom Flange	0.313	2.00	34.4	**50.4**	84.2	0.009
Web	0.068	2.00	27.5	**57.7**	67.8	0.000

1. Introduction

Economy is a dominant factor in the design of pre-engineered steel buildings. To provide adequate capacity at the least cost, use of highly efficient steel sections that vary throughout the frame is common. To match economy to capacity, plate girders rather than rolled sections are used. Two categories of plate girders are available for use by the designer, doubly symmetric and singly symmetric sections. A doubly symmetric section has identical flange sizes, while a singly symmetric section has two different size flanges.

Many pre-engineering companies utilize a steel moment frame which is comprised of singly symmetric members. The construction of these members depends on the capacity requirement of the frame. Most singly symmetric sections are fabricated by welding three thin plates in an I shape, using a single fillet weld on one side of the beam, creating the two different size flanges, and a web. This type of construction is usually automated and extremely economical. The frame in which a singly symmetric member is placed may have many different cross sections, throughout the frame, usually butt-spliced together using full penetration butt-welds between two different sections and bolted splices between major lengths of frame. This construction allows the engineer to match the moment to the design requirements at various locations throughout the frame.

Design of singly symmetric beams is based on American Institute of Steel Construction (AISC) specifications. There are two available specifications, Allowable Stress Design, Ninth Edition (ASD) and Load and Resistance Factor Design, Second Edition (LRFD). ASD is based on the philosophy that the stress present in a section subjected to a load is less than the available stress capacity in the section being considered by a certain factor of safety. LRFD is based on the philosophy that the section has a specific capacity, and this capacity, reduced by a factor accounting for material uncertainties, should be greater than the load condition applied to the section under consideration. The ASD provides guidance for design and analysis of sections subjected to flexure in chapters B, F, G and Appendix B, but does not expressly address singly symmetric sections. The LRFD provides guidance in chapters B, F, G, and Appendices B, F and G and does provide consideration for singly symmetric members. It is assumed that the reader is familiar with the use and terminology of both ASD and LRFD.

1.1 Specifications

There are minor philosophical differences between the methods utilized by each of the Specifications, but the underlying theory and eventual design values are closely correlated and produce nearly identical designs. Each Specification is slightly different and both depend on the definition of the section in terms of slenderness parameters. Slenderness parameters are defined as width to thickness ratios for each compression element, flange and web. The formulae and basis for these Specifications will be examined in Chapter 2 of this thesis.

1.2 Scope and Objective

An unusual amount of pre-engineered buildings collapsed during a recent snowstorm (1992) in the eastern United States of America. While failures might be expected in areas where significant overloads were present, most of these buildings were located where this was not the case. Sections of frames from one of these collapsed buildings were examined at The University of Texas at Austin to assess the capacity of three different singly symmetric sections. Some sections were subjected to shear and flexure while others were subjected to pure flexure. In addition to finding the ultimate moment capacity of each section, behavior was also examined. Chapter 3 of this thesis describes the experimentation program and the techniques used to gather information about the sections tested. Chapter 4 describes each experiment in detail, and discusses the results of the individual experiments.

Further investigation of the influence of shear on moment capacity for a singly symmetric section was also conducted. A software program developed at The University of Texas at Austin, was used to examine buckling capacity of singly symmetric sections subjected to different loading schemes and moment gradients using linear elastic theory. These investigations were performed to give a reasonable assessment of shear/moment interaction, without performing numerous costly experiments. The results from these analyses are examined in Chapter 5 of this thesis, and help address the influence of shear on the capacity of singly symmetric members, as well as the applicability of the Specifications and possible weaknesses therein. Questions concerning the singly symmetric section, and the lack of Specification guidance regarding these types of sections are described and discussed in both Chapter 5 and Chapter 6 of this thesis. To find the weaknesses in the Specification, a critique and examination of ASD and LRFD will be presented in the next chapter.

2. Specifications

For design of a member subjected to flexure in a steel building, two different AISC specifications are permitted, the Allowable Stress Design, (ASD) and the Load and Resistance Factor Design, (LRFD). For most doubly symmetric shapes where yielding of the section governs capacity, the Specifications provide almost identical answers. For shapes where buckling of compression elements controls behavior , the two methods differ. This chapter will discuss the differences between the two Specifications.

2.1 Section Limit States

The Specifications recognize four limit states governing the resistance of a member subjected to flexure. The limit states are flange local buckling (FLB), web local buckling (WLB), lateral torsional buckling (LTB), and yielding of the section. The limit state which occurs at the lowest stress, then controls the nominal capacity of the section. Adequate lateral bracing is provided in many pre-engineered buildings essentially eliminating LTB behavior controlling capacity, so the focus of this chapter will be FLB and yielding of the section with some consideration of WLB. Most Specification formulae are based on compression element behavior of doubly symmetric sections; a focus of this chapter will be on the compression elements of singly symmetric sections. While the Specifications accurately predict the strength of doubly symmetric sections, the accuracy of the Specifications as applied to singly symmetric sections has not been documented.

Within each buckling limit state, the behavior is classified as compact, non-compact and slender and also elastic and inelastic. The width-thickness ratios (slenderness parameters) of the compression elements (flange and web) are used to classify the section.

2.2 Slenderness Parameters

Figure 2.1 shows the classification of a cross section according to the width-thickness ratio of the compression flange used by the ASD and LRFD specifications. The width-thickness ratio for the compression flange of a cross section is half the flange width divided by the flange thickness. The width-thickness ratio for the web is the clear height of the web divided by the thickness of the web. The web width-thickness parameter varies depending upon which Specification is used, and the dimensions of the section. The differences are slight and of no major consequence to this examination of the Specification.

Figure 2.1 graphically represents the Specification equations for various flange slenderness ratios. The capacity shown on the y-axis is non-dimensionalized with respect to the yield moment of the section, assuming a shape factor of 1.10. There are three basic regions shown on the curve: compact, non-compact and slender. The definition of these sections is different for the ASD and LRFD, but

both essentially deal with behavior of the cross section as controlled by stability of compression elements.

The compact region denotes a section that has compression elements which are extremely stable, i.e. buckling will not occur in the cross section before the full plastic moment of the cross section is reached. Therefore, the capacity of a compact section, both in ASD and LRFD, is the plastic moment capacity.

The non-compact region shown on the curve in Figure 2.1, is different for each Specification. The ASD treats the non-compact section as one which will have a capacity controlled by yielding of the flange, but full plastic moment can not be reached. The LRFD defines the non-compact section in a comparable manner to the slender inelastic section in the ASD. This non-compact/slender inelastic zone denotes a section where buckling and yielding both occur. In pre-engineered buildings, most sections fall within the LRFD non-compact region of the curve in Figure 2.1. The slender (LRFD) and slender elastic (ASD) sections are identical, and denote a section which has a moment resistance limited entirely by elastic buckling of the compression elements of the section. This region of the curve is based on the plate buckling equation:

$F_{cr} = \dfrac{\pi^2 Ek}{12(1-\nu^2)\lambda^2} = \dfrac{(26{,}200)(0.763)}{(b/t)^2}$ for both ASD and LRFD. The 0.763 is the k factor assumed for flange local buckling which relies on some web restraint (Johnson, 1985). The k factor for no web restraint is 0.425 whereas a fully rigid web gives k = 1.28. In Figure 2.1, the two Specifications do not plot identically because of a small difference in the k_c value calculated. This difference will be addressed later in this chapter.

The boundaries between regions of classification, i.e. compact, non-compact, etc. are derived from experimental and theoretical examination of doubly symmetric sections. The limiting width-thickness ratios for compression flange dimensions are shown in Table 2.1 and in Figure 2.1. All limits, except the compact limit, are functions of k_c. This parameter is introduced to account for flange-web interaction and will be addressed later in this chapter. The limiting width-thickness ratio for compact/non-compact sections in LRFD is identical to that in the ASD, but the non-compact/slender limit has a 16.5 ksi value as well as the k_c parameter. The 16.5 ksi represents a residual stress of 16.5 ksi assumed for welded I-shapes. The Specification recommends using 10 ksi for rolled sections.

There are also limiting width-thickness ratios for the web of a section. These values are shown in Table 2.2. The treatment of a section is handled similarly in both Specifications, referring to a web as compact, non-compact and slender. The limits in each Specification are virtually identical. From these limiting width-thickness ratios, specific sections of the Specification for design are defined.

Table 2.1 Limiting Flange Width-Thickness Ratios

ASD	LRFD
$b/t \le \dfrac{65}{\sqrt{F_y}}$ Compact	$b/t \le \dfrac{65}{\sqrt{F_y}}$ Compact
$\dfrac{65}{\sqrt{F_y}} < b/t \le \dfrac{95}{\sqrt{F_y/k_c}}$ Non-Compact	$\dfrac{65}{\sqrt{F_y}} < b/t \le \dfrac{162}{\sqrt{(F_y - 16.5)/k_c}}$ Non-Compact
$\dfrac{95}{\sqrt{F_y}} < b/t \le \dfrac{195}{\sqrt{F_y/k_c}}$ Slender Inelastic	$\dfrac{162}{\sqrt{(F_y - 16.5)/k_c}} < b/t$ Slender
$\dfrac{195}{\sqrt{F_y/k_c}} < b/t$ Slender Elastic	
$k_c = \dfrac{4.05}{(h/t)^{0.46}}$ For h/t > 70 If h/t < 70, $k_c = 10$ **(A-1)**	$k_c = \dfrac{4}{\sqrt{h/t}}$ $0.35 < k_c < 0.763$ **(L-1)**

2.3 Specification Sections

Chapter B of ASD and LRFD gives the limiting flange and web slenderness ratios defining the different zones of behavior: compact, non-compact and slender. The formulae for strength within the various zones are given in Chapter F, G or Appendix B in the ASD, and Appendix B, F or G in the LRFD. In ASD, Chapter F

Table 2.2 Limiting Web Width-Thickness Ratios

ASD	LRFD
$d/t \le \dfrac{640}{\sqrt{F_y}}$ Compact	$h/t_w \le \dfrac{640}{\sqrt{F_y}}$ Compact
$d/t > \dfrac{640}{\sqrt{F_y}}$ *and* $h/t_w \le \dfrac{760}{\sqrt{F_b}}$ Non-Compact	$\dfrac{640}{\sqrt{F_y}} < h/t_w \le \dfrac{970}{\sqrt{F_y}}$ Non-Compact
$h/t_w > \dfrac{760}{\sqrt{F_b}}$ Slender	$\dfrac{970}{\sqrt{F_y}} < h/t_w$ Slender
F_b is calculated in Chapter F or Appendix B.	For members with unequal flanges (singly symmetric members) $\dfrac{970}{\sqrt{F_y}}$ is replaced with Eq. (L-2)
	$\dfrac{253}{\sqrt{F_y}}\left[1+2.83\left(\dfrac{h}{h_c}\right)\right]$ *with* $\dfrac{3}{4} \le \dfrac{h}{h_c} \le \dfrac{3}{2}$ As found in Appendix B **(L-2)**

defines the strength of members with compact and non-compact flanges and webs whereas Appendix F in LRFD is used for such sections. For a slender compression flange, Appendix B is used in ASD and either Appendix F or B for LRFD, depending on the compression flange dimensions.

The equations used in these sections of ASD and LRFD are shown in Table 2.3. These equations are applicable to welded sections, and differ slightly from those used for rolled sections. The λ shown in the LRFD equations,

corresponds

Table 2.3 Chapter F (ASD) Appendix F (LRFD)

ASD	LRFD
$F_b = 0.66F_y$ Compact Web and Compact Flange **(AF-1)**	$M_n = M_p$ Compact Web and Compact Flange **(LF-1)**
$F_b = F_y\left[0.79 - 0.002\frac{b_f}{2_{tf}}\sqrt{\frac{F_y}{k_c}}\right]$ Non-Compact Flange Compact or Non-Compact Web **(AF-2)**	$M_n = M_p - \left(M_p - M_r\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)$ $M_r = (F_y\text{-}16.5)(S_x)$ Non-Compact or Compact Flange Non-Compact Web **(LF-2)**
$F_b = 0.60F_y$ Non-Compact Flange Not Included Above **(AF-3)**	$M_n = M_{cr} = S_xF_{cr} \leq M_p$ $F_{cr} = \frac{26200k_c}{\lambda^2}$ Slender Flange Non-Compact Web **(LF-3)**
$k_c = \frac{4.05}{\left(h/t\right)^{0.46}}$ For $h/t > 70$ If $h/t < 70$, $k_c = 10$ **(A-1)**	$k_c = \frac{4}{\sqrt{h/t}}$ $0.35 < k_c < 0.763$ **(L-1)**

to $b_f/2t_w$, with λ_p and λ_r corresponding to the compact and non-compact limiting flange width-thickness ratios shown in Table 2.2.

Each formula is used according to the type of section under investigation. A compact flange and web would require the use of Eq. AF-1 or LF-1 in Table

2.3. A non-compact flange with a non-compact or compact web would require the use of Eq. AF-2 or LF-2. A slender flange with a non-compact or compact web would require the use of Appendix B for ASD and Eq. LF-3 for LRFD. Either method gives similar results for most sections. Members with non-compact and slender flanges require the calculation of k_c to consider web-flange interaction which is discussed next.

2.4 Web Flange Interaction Parameter

The k_c formula in ASD, Eq. A-1 in Table 2.1, was developed by Johnson (1985), based on experiments on beams subjected to uniform moment. The beams failed by both flange and web buckling. The k_c equation used in ASD was developed through back calculation of k_c to provide a correlation between the theoretical plate buckling formula and experiments in both the elastic and inelastic ranges. The limit of $h/t > 70$ is applied to the k_c equation to limit the interaction effect to slender welded shapes as used in the Johnson tests. Rolled sections have $h/t < 70$. Note that there is a different formula for k_c in LRFD (Eq. L-1, Table 2.1) which is similar to the ASD formula. A comparison of the LRFD equation and the Johnson tests is shown in Figure 2.2 (Yura, 1991).

The effect of the difference between the two k_c values is evident in Figure 2.1. Here the section is identical for both Specifications, but k_c changes the capacity of the section by a few percent, depending on the Specification used. The derivation and definition of k_c is beyond the scope of this thesis, but the effect of k_c on the capacity of a given section will be examined.

Figure 2.3 Capacity of Sections with Varied h/t Ratios (ASD)

The curves in Figure 2.3 show the capacity of a section as limited by flange local buckling in ASD for three different sections. These sections have identical tension flanges, but have different web width-thickness ratios. Compression flange sizes were varied, by varying the flange thickness only, the flange width was held at a constant value of 6 inches. Thus the only variable affecting the k_c value is the web width-thickness ratio. The three k_c values shown correspond to specific parameters within the Specification. The 0.350 value corresponds to the lower limit set by LRFD for k_c, which corresponds to a section with h/t = 205. This h/t value classifies the web as slender, and requires the use of Chapter G for calculating the capacity of the section. Other factors contribute to reduce the capacity of the section using that chapter, so the capacity reduction is due to more than just k_c. The 0.574 value corresponds to the k_c limit of h/t > 70 shown in Table 2.3. The 1.0 value corresponds to a section with a compact web, i.e., no influence from k_c. There is a significant difference between the moment capacity for sections with no influence of k_c and the maximum allowable ASD k_c value of 0.574. The capacity difference is due to k_c and the different section modulus due to the web dimension. Consideration of the moment capacity of a section with a high value of k_c will be addressed in the discussion of Chapter G, where the influence is more applicable.

Figure 2.4 Capacity of Section with Varied h/t Ratios (LRFD)

The effect of k_c on section capacity according to LRFD can be seen in Figure 2.4. The same comments for Figure 2.3, apply to Figure 2.4 also. The reduction of section capacity shown in both Figures 2.3 and 2.4, is due to both the effect of k_c and the change in section modulus caused by the variation in the h/t ratio.

The k_c calculated according to the equation in the LRFD varies slightly from that in the ASD. A comparison of the two k_c values and their influence on a given section with a slender web (h/t = 130, use Appendix G, LRFD), is shown in Figure 2.5. There is less than a ten percent difference between the capacities according to the two Specifications for both compact webs and slender webs, so the specific k_c formulation is acceptable for each Specification. Note that the h/t ratio is the same for each curve, but the k_c value is different. Figure 2.6 shows the moment capacity of two sections with different. h/t ratios, but identical k_c values. The h/t ratio was back calculated from the k_c value using the applicable equation from each Specification. Again, there is less than 10 percent difference between the two curves.

From this comparison, it may be concluded that k_c in LRFD is comparable to k_c in ASD. The above arguments and conclusions, as well as Figures 2.4 and 2.6, were based on Chapter G of ASD and Appendix G of LRFD. The same arguments and conclusions are also true for Chapter F of ASD and Appendix F of LRFD.

Figure 2.5 Specification Comparison of Section with h/t = 130

2.5 Slender Webs

If the section under examination has a non-compact or compact flange and a slender web, the bending strength will be based on Chapter G (ASD) and Appendix G (LRFD). The appropriate equations used in these sections of ASD and LRFD are shown in Table 2.4. The equations in Table 2.4 have two parameters, R_{PG} and R_e, that should be addressed. R_{PG} is defined as the plate girder reduction factor, and is calculated using equation AG-2 for ASD and LG-2 for LRFD. R_e is defined as the hybrid girder reduction factor, and is calculated using equation AG-3 for ASD and LG-3 for LRFD. Both these terms are then applied to the appropriate formula (AG-1 and LG-1) in Table 2.4, to calculate the beam capacity for a member subjected to flexure. These factors and their effect on the capacity of a flexural member are examined below.

Figure 2.6 Specification Comparison for Sections with $k_c = 0.35$

2.5.1 R_{PG}, Plate Girder Reduction Factor

The effect of R_{PG} on any section capacity is limited to a maximum of 1.0, which has no effect on the section capacity. All other values less than 1.0 reduce the capacity of the section. The R_{PG} equation for both Specifications depends on a_r, the web area to flange area ratio, (A_w/A_f) and the web slenderness. The different equation in LRFD is presented to alleviate a problem with the equation in the ASD.

The curves shown in Figure 2.7 represent the first term in the R_{PG} equation for each Specification. The difference between the two curves denotes the effect that the first term of the formula has on R_{PG}. Although the difference between the two curves is not drastic, the effect on R_{PG} is substantial, and creates a problem in ASD for sections with large a_r values. The LRFD alleviates any possible problems with the influence of a_r by using the more stable term shown in Figure 2.7. The LRFD equation may be applied to all a_r values, while the ASD equation is only applicable for a_r less than 2. Both equations are taken from work published by Basler (1963).

Figure 2.7 Sensitivity of A_w/A_f Term in R_{PG} Calculation

There are two other terms in equations AG-1, AG-2 and LG-1, LG-2 which might produce different values for R_{PG}. These are the F_b and h in ASD and F_{cr} and h_c in LRFD. The term which contains F_b and F_{cr} in each equation respectively, is virtually identical for most sections. This theory was investigated, and the term was found to vary by less than 5 percent for all cases examined. This leaves h and h_c as final differences to be examined in the two different R_{PG} formulae.

The clear height of the section between the flanges is referred to as h, while h_c is defined as twice the height of the compression section of the web. The difference between the two is usually not great, but it can have an effect on R_{PG}. As the ratio between the area of the tension flange and the compression flange varies while the area of the web is held constant, the location of h_c varies along the height of the web. Theoretically, h_c could vary from t_f to $h + t_f$. Figure 2.8 graphically shows an example of how this might occur.

Figure 2.8 Varied Cross Sections

The tension flange is exactly the same size for both sections in Figure 2.8, but the thickness of the compression flange varies for each. This moves the centroid higher on the web of the right hand section, lower on the left hand section. The curve in Figure 2.9 was plotted based on the condition shown in Figure 2.8. By keeping the area of the tension flange constant and varying the compression flange area from approximately 0.6 square inches to nearly 18 square inches, a representation of h_c and its effect on R_{PG} is represented in Figure 2.9. The web was kept at an arbitrary constant depth and thickness for the curve.

Figure 2.9 Variance of R_{PG} with A_{fc}/A_{ft} (h/t = 200)

Figure 2.9 shows the variation between R_{PG} and A_{fc}/A_{ft} for both LRFD and ASD. Underlying that description, is the effect that h and h_c have in the calculation of R_{PG}. While h_c varies, so does R_{PG}. This variance of R_{PG} is not applicable for extremely low compression flange area to tension flange area ratios. The high R_{PG} values in this zone are not applicable due to the low F_{cr} calculated for a section with such a small compression flange. This is evident for the extremely high R_{PG} values shown in Figure 2.9 for A_{fc}/A_{ft} less than 0.5. R_{PG} is also not significant for extremely large compression flange area to tension flange area ratios. Although the values for R_{PG} are still less than one, sections are rarely designed with an

A_{fc}/A_{ft} of greater than 4. Therefore, the usefulness of the R_{PG} factor would be limited to a range of compression flange area to tension flange area ratio of 0.5 to 4.0.

The curve in Figure 2.9 is plotted for a section with h/t = 200. This value is arbitrary, but must be greater than approximately 130, to ensure that R_{PG} is applicable. If h/t were less than 130, the section would not have a slender web, for either Specification, thus Chapter G and Appendix G would not be applicable and R_{PG} would not be used in the capacity calculation.

Figure 2.10 shows the percentage difference of R_{PG} values for ASD and LRFD with varying A_{fc}/A_{ft} values. The curve shows that for sections within the limiting 0.5 to 4 values for compression flange area to tension flange area ratios, the percent difference is between 10 to 2.2 percent respectively. The difference can be attributed to the h_c term as well as the web area to flange area ratio discussed earlier.

2.5.2 R_e, Hybrid Girder Reduction Factor

Hybrid girders have flanges with a higher flange steel yield stress than the web steel. The R_e factor is developed to account for hybrid girders. This parameter is important when a doubly symmetric hybrid section is designed to approach and reach full plastic moment capacity. Equations AG-3 and LG-3 in Table 2.4, are identical and each have only one critical term, the ratio of web yield stress to critical compression flange stress. This term is referred to as α in the

ASD version, and m in the LRFD version. This critical compression flange stress may either be calculated according to buckling behavior, or may be the yield stress of the flange if compression yielding controls the capacity of the section.

Figure 2.10 Percent Difference Between ASD and LRFD R_{PG}
(h/t = 200)

There is a range of possible values for α and m depending on the steel used in the web and the flanges. A realistic range would be from 1.0 to 0.6. These R_e values would correspond to a web and flange stress of equal value (1.0) and a web stress value of 36 ksi and a flange stress value of 60 ksi (0.6). With these values, the R_e calculated was 1.0 and 0.93 respectively. These values were calculated for the case where yielding controls section behavior. For the cases where buckling controls the behavior of the section, the critical flange stress approaches the web yield stress, thus the m and α values approach 1.0. For this type of behavior, the

R_e factor is closer to 1.0 than 0.93. Therefore, the most reduction that R_e can have on any given section is approximately 7 percent, which is of relatively little consequence.

Considering all the reduction factors present in the "G" sections of the Specifications, the resistance of a given section will vary to a rare maximum of approximately 20 percent, while most will be reduced by less than 10 percent. The final Specification section to examine is Appendix B for both Specifications.

Table 2.5 Appendix B (ASD) and (LRFD)

ASD	**LRFD**
$\frac{95.0}{\sqrt{F_y / k_c}} < b/t < \frac{195}{\sqrt{F_y / k_c}}$ $Q_s = 1.293 - 0.00309(b/t)\sqrt{F_y/k_c}$ **(AB-1)**	$\lambda_r = \frac{253}{\sqrt{F_y}}\left[1 + 2.83\left(\frac{h}{h_c}\right)\right]$ $\frac{3}{4} \le \frac{h}{h_c} \le \frac{3}{2}$ **(ABL-1)**
$b/t > \frac{195}{\sqrt{F_y/k_c}}$ $Q_s = \frac{26200\ k_c}{F_y (b/t)^2}$ **(AB-2)**	$\frac{109}{\sqrt{F_y/k_c}} < b/t < \frac{200}{\sqrt{F_y/k_c}}$ $Q_s = 1.415 - 0.00318(b/t)\sqrt{F_y/k_c}$ **(ABL-2)**
$F_b = 0.60F_yQ_s$	$F_b = 0.90F_yQ_s$

2.6 Appendix B ASD and LRFD

Appendix B in both Specifications is used when the section under examination is classified with slender elements in compression. A new expanded definition for the non-compact/slender limiting width-thickness ratio for webs is presented in the LRFD Appendix B and shown as Eq. ABL-1 in Table 2.5. This equation is developed for singly symmetric sections.

The capacity of a section calculated using Appendix B is related to the reduction factor Q_s shown in equations AB-1, AB-2, ABL-1 and ABL-2, and the flange yield stresses. These equations are applicable to a section which has a capacity controlled by FLB. Both Specifications provide nearly identical formulae for the reduction factor $Q_{s.}$

The above explanation of each Specification section will become valuable for comparison to test results for a given section. The next chapter will describe associated procedures and setups of the experimentation program.

3. Beam Experiments

3.1 Introduction

Test specimens came from a frame with only slight damage near the end of a collapsed building. The general geometry of the frame is shown in Figure 3.1. The frame is a symmetrical gable frame with a support column at the centerline of the frame. The clear span of the frame is approximately 100 feet with a height of approximately 30 feet at the peak of the gable. A typical moment diagram from uniform gravity loading for the frame is also shown in Figure 3.1.

Figure 3.1 Frame Geometry and Loading

The frame consisted of seven bolted sections, two columns and five beams. Each bolted section had variable cross sections. Overall there were 10 distinct spliced sections in each symmetric side of the frame, some of which were tapered. The cross section consisted of a top flange and bottom flange welded on one side to the unstiffened web plate. Within each bolted section, the various different cross sections were splice welded together, forming the large bolted section.

In the general collapse of the structure, most failures occurred within one specific bolted section which was comprised of three different cross sections, #9, #10 and #11, shown in Figure 3.1. Sections #9 and #10 were near the peak of the moment diagram, with the top flange in compression. Section #11 was near an inflection point in the moment diagram, so moment reversal could occur in the section depending on the loading condition of the frame.

The experimental program focused on sections #9 #10 and #11 because the actual failures occurred within this bolted section. This section was 36 feet, 7 inches long with a uniform depth. Each of the different cross sections was approximately 12 feet in length.

Five separate tests were conducted, one on section #9, one on section #10 and three on section #11. Three of the test specimens came from cross section #9, #10 and #11 from the same bolted section, (south inside rafter (311)), from frame #11. One of the test specimens came from cross section #11 from the bolted section (north inside rafter (313)), taken from frame #11. Two tests were conducted on this #11 cross section. The preparation of the specimens varied for each test, but all specimens were flame cut from the original bolted sections. The ends of the specimens were then modified for attachment to end fixtures.

3.2 Test Setup

Two test setups were used in the experimental program conducted at the Ferguson Structural Engineering Laboratory at The University of Texas at Austin. A cantilever test setup, shown in Figure 3.2, was used to simulate the condition near the inflection point of the moment diagram shown in Figure 3.1. In this case the cross section has a moment gradient, and is subjected to both shear and moment.

Figure 3.2 Cantilever Setup

The uniform moment test setup shown schematically in Figure 3.3 represents conditions associated with the maximum moment location in the frame, i.e. high moment and no shear. This test setup is also similar to that used by Johnson (1985) for his study of flange and web buckling interaction. All sections;

#9, #10, and #11, were tested in the uniform moment test setup. Only section #11 was tested in the cantilever test setup.

Figure 3.3 Pure Moment Setup

Five static ultimate load tests were performed on these four specimens. Three separate ultimate capacity tests were performed on section #11, two cantilever tests and one pure moment test. One static ultimate pure moment test was performed on section #9 and one static ultimate pure moment test was performed on section #10.

3.2.1 Cantilever Test Setup

The cantilever setup utilized a frame attached to the strongwall at FSEL. The fixed end of the cantilever was attached to the wall while the free end of the cantilever was connected to the loading ram. For connection of the specimen to the strongwall, a lab fabricated end plate was welded to the specimen, which was then bolted to a column, which was bolted to the strongwall. The free end of the cantilever specimen, was left with the original end plate from the frame bolted section intact. A frame attached to the floor consisting of a W12x96 and vertical angles, provided lateral bracing by attaching contact plates to the vertical frames. Lubricant was applied at contact points between the braces and specimen to reduce friction between the specimen and the guides. The vertical frames were spaced at the five feet interval for lateral bracing, which is exactly how the top flange of the frame member was laterally braced in the original structure, the bottom (larger) flange was braced at 10 foot intervals. This overall setup is shown in Figure 3.4 with the specimen in place. A schematic showing the dimensions of the braces and typical brace detail is shown in Figure 3.5.

An extension beam was attached to the cantilever end of the specimen to produce a moment gradient that was consistent with that in the building frame. The test load was applied in an upward direction at the free end of the cantilever using a 25,000 pound capacity hydraulic ram. The difference between Test 1 and Test 2, was the orientation of the specimen. In Test 1, the specimen was tested as oriented in the original structure, with the thin flange in compression. In Test 2, the specimen was tested for capacity in the opposite direction where the thin flange was in tension. The section dimensions for each specimen will be presented in Section 3.3 of this chapter.

Figure 3.4 Test Setup for Tests 1 and 2

Figure 3.5 Bracing Details

3.2.2 Uniform Moment Test Setup

Tests 3, 4 and 5 were conducted in a pure moment loading setup. This setup used a 600,000 pound universal loading machine to load the specimen shown in Figure 3.3. A spreader beam was used to load the specimen at two separate points and the test beam was supported on a roller-roller configuration to ensure symmetric behavior, and constant moment in the region of interest. Figure 3.6 shows this setup with the specimen and lateral bracing in place.

Figure 3.6 Test Setup for Tests 3, 4 and 5

In order to force the failure to occur in the uniform moment region, the test specimen had to be reinforced immediately adjacent to the load points since both high shear and high moment occur at these locations. Outside the uniform moment region, the web was stiffened to increase its shear capacity. Two extension pieces were also fabricated and attached to the test specimen to reduce the load and shear required to reach the desired moment capacity in the area of interest. A typical extension piece and splice is shown in Figure 3.7.

Figure 3.7 Extension and Splice Detail for Tests 3, 4 and 5

Lateral bracing for the entire test specimen was supplied at the load points and at the supports, using angles clamped to the test machine support columns and to the end supports. The unsupported length of the uniform moment region was five feet long for Test 3 and Test 4, which was the unbraced length in the original frame. The unsupported length of the uniform moment region in Test 5 was three feet long, which is less than the original and critical unbraced length (5 feet) for this section. Thus, the sole purpose of the bracing was to provide global stability of the overall specimen, and to eliminate any possibility of unstable lateral behavior during testing. A detail of a typical brace and attachment is shown in Figure 3.8.

Figure 3.8 Lateral Bracing Detail for Tests 3, 4 and 5

3.3 Material and Section Properties

Each section tested had variable material and section properties. The section dimensions for all test specimens (Sections #9, #10, #11) are shown in Table 3.1. The elements of the section are named in a manner representative of the original orientation of the section in the frame. The measurements of flange and web thickness were taken with a micrometer, while all other measurements were taken with sixteenth-inch divided scales. All values in Table 3.1 are averages of at least three measurements taken for each element. The measurements were taken at various locations on the section to give a representative value for the element dimensions. The variation of the thickness dimensions and their averages are accurate to 0.001 inch, all other measurements are accurate to 0.01 inch.

Table 3.1 Section Dimensions

The material properties of each section were taken after the completion of all tests conducted. Standard ASTM, A370 tension tests were performed on steel taken from each flange and web of each test specimen. Pertinent average values for each specimen are reported in Table 3.2. The reported values are averages of at least two tests per specimen and vary from each tested value by a maximum of 20 percent, with most values within 10 percent of single test values. The static yield from the yield plateau region of the stress-strain curve of the tension test was used for all calculations which required a yield stress for a given section.

Table 3.2 Summary of Averages for Tension Tests

3.4 Data Acquisition and Testing Techniques

Data recorded was nearly identical for all tests and consisted of load displacement for the beam, lateral displacement of the top flange, rotation of the top flange, web deflections and rotation of the fixed end of the cantilever beam.

One-half inch long strain gauges, designed for use with steel, were applied to section #11 near the predicted failure area in two lines. The approximate location is shown in the schematic drawing in Figure 3.9, and on the actual specimen in Figure 3.10.

Figure 3.9 Strain Gauge Location for Test 1

Load deflection data was taken using a 6 inch dial gauge as well as electronically, using the data acquisition system and software supplied by the Ferguson Structural Engineering Lab. This data acquisition system was also used to record strain gauge data in Test 1, and to convert all data from voltages to displacements and stresses using CPROF7, also supplied by FSEL. Lateral displacement data for the top flange was taken using a theodolite and ruler for Tests 1 and 2 and a string and ruler for Tests 3, 4 and 5. Flange local buckling behavior was observed using an electronic inclinometer in Test 1, visual inspection was used for all other tests.

3.10 Strain Gauge Location on Test 1 Specimen

Web deflections were taken at points of interest for all tests. The data was taken using the hand held web deflection gauge shown in Figure 3.11. This gauge was placed on a perfectly flat surface of milled granite prior to each test to achieve a set of zero readings. The zero readings provide a benchmark to gauge initial web deformation by placing the instrument at designated points of the web. Readings taken during the test are then compared to the initial deformation to describe a changing web surface.

Rotation of the fixed end of the cantilever beam in Tests 1 and 2, was monitored using the dial gauge setup shown schematically in Figure 3.12. This data was used only to ensure that there was no major displacement due to connection rotation, and in the calculation for elastic load-deflection behavior shown in Chapter 4 for Tests 1 and 2. In-plane inclinometers were attached to the stiffeners under the load points for Tests 3, 4 and 5 to record the net change in rotation of the specimen at the load points. These inclinometers gave a value of change in

Figure 3.11 Hand Held Web Deflection Gauge rotation of the section during the test. These data were used for plotting moment-rotation information for the section.

In each test, the load was applied in equal load increments until the yield capacity of the section was reached, where displacement was then used to control the data increment for the test. The test then continued until satisfactory deformation was reached, or unsafe conditions were approached at which point the test was terminated. There were two test setups and procedures used for these tests.

Figure 3.12 Rotation Gauge System for Tests 1 and 2

3.5 Test Techniques

The following sections will describe the specific data acquisition and testing techniques used in each test. For Test 1, a beam which contained section #11 was loaded in the cantilever setup in approximately 2.5 kip increments with electronic data readings recorded at every increment. Static readings were taken in the inelastic region for this test. Web deflection readings were taken at 5 kip increments. The specimen was loaded to a maximum, then deflected further to provide adequate deformation for recognition of the failure area in photographs.

Figure 3.13 Test 2 Specimen Preparation

The specimen used for Test 2 was the same specimen used in Test 1, rotated 180 degrees about its longitudinal axis and reset into the test setup. The deformation of the section produced by Test 1 was still evident near the splice region of the specimen. The flange buckle was straightened to within one inch of straight before starting Test 2. Since this flange would now be subjected to tension, this imperfection was considered acceptable. The section was strengthened by welding a cover plate to each flange to force the weakest section of the specimen to a point approximately 2 feet from the splice region. The added capacity is shown schematically in Figure 3.13, in the form of cover plates. This addition of capacity to the specimen would force the failure into a previously unyielded section of the flange and web. The test proceeded in a similar fashion to Test 1, loading the specimen in 2.5 kip increments and taking data after each load increment.

Section #10 was tested in Test 3. The section was loaded in 5 kip increments. Web deflection readings were taken at the centerline of the section at 10 kip load increments starting with the 15 kip load. Electronic data was recorded for applied load and displacements at the load points and centerline of the specimen. Local rotation values were read on the inclinometers attached to the stiffeners under the load points of the specimen, and recorded by hand. A schematic of this setup is shown in Figure 3.14.

Figure 3.14 Beam Rotation Setup for Tests 3, 4 and 5

The setup for Test 3 was not completely symmetric, which allowed for a difference of about 0.4 inches between the two moment arms for the section. The support setup was also not completely symmetric as a roller and pin support system was used rather than a roller and roller system. It was decided that these flaws were minor in the scope of the test. The difference in desired and actual moment application to the section is minimal and the performance of the section under test load was satisfactory and showed no signs of moment gradient problems.

Test 4 was used to examine section #9, and was performed in a similar manner with Test 3. Web readings were taken at the centerline of the section, and between the holes shown in the schematic representation shown in Figure 3.15, at 10 kip increments. Load and displacement values were recorded electronically, while rotation values were read from inclinometers attached to the stiffeners under the load points. Maximum load for the specimen was approximately 110 kips, while maximum deformation was much greater than for other sections. At high deformation levels, the specimen began to exhibit lateral torsional buckling behavior, between the load points, at which point the test was terminated due to safety concerns.

Figure 3.15 Holes in Test 4 Specimen

Test 5 examined a different section #11 than the specimen tested in Tests 1 and 2. The specimen was tested in the same orientation as Test 1, with the small flange in compression. A local deformation in the top flange was removed prior to testing. The flange was straightened to within one-sixteenth of an inch of straight. The location of this imperfection was approximately seven inches from the southern load point. Identical procedures were followed for this test, as they were for Test 3 and Test 4. The specific results of these tests will be discussed in Chapter 4.

4. Beam Experiment Results

The purpose of the beam experiments conducted on the three different sections, was to achieve an understanding of the adequacy of the Specifications to predict the capacity for singly symmetric sections. The main focus of the Specification is capacity, but the prediction of failure mode is also important. These tests gave an indication of how closely the Specification predicts the capacity and failure mode of any given section. The capacities calculated using the Specification (LRFD) used the actual yield stress values of the steel from the test sections (shown in Table 3.2), which were taken after the testing program was completed. Each experiment will be discussed in the following sections with special attention paid to the Specification and the accuracy of section capacity predictions.

4.1 Test 1

Section #11 was tested as a cantilever in an effort to duplicate the moment gradient present in the original structure under gravity loads. The section under examination was oriented in the same manner as it was in the original structure. This test subjected the small flange of the section to compression forces. The beam was instrumented with strain gauges to give stress readings on two lines near the suspected region of failure, while all other instrumentation, delineated in Chapter 3, was used to plot a force displacement curve for the section, and to aid in understanding the behavior of the section. Using the data from the instrumentation, a moment displacement curve is plotted and presented in Figure 4.1.

Figure 4.1 Moment-Deflection Test 1

The load displacement curve shown in Figure 4.1 uses the moment present at the section where the failure was observed, versus the displacement at the end of the test beam. The moment-deflection performance is reasonably linear up to failure, as shown with the elastic moment deflection line. The initiation of web buckling in the test section was noticed at approximately 1870 in-kips and major flange buckling was observed at the maximum moment for the test (2480 in-kips).

The straight horizontal dashed lines denote the three calculated capacities for the section, based on section dimensions and material properties of the test specimen. The nominal moment capacity calculated using Appendix G of the LRFD, second addition, is 2414 in-kips, with a FLB mode of failure. The yield capacity of the section according to Appendix G is 3190 in-kips, with the compression section modulus governing the capacity. The plastic moment capacity of the section is 4035 in-kips. The section reached a capacity which exceeded the predicted nominal capacity by 2.7 percent. Thus the Specification predicted value is within acceptable limits for the behavior and capacity of this section.

The web of the test section had initial shape imperfections and local waves prior to testing. A majority of the initial imperfections may be attributed to heat caused by splice and fabrication welds. The initial shape of the web is shown in the upper right hand corner of Figure 4.2. The different shades denote deflection variations of one tenth of an inch. The contour shown is from web deflection data taken over a section which is approximately 36 inches long, by the height of the web (approximately 24 inches). The top and bottom flange locations are noted on each surface plot. The web deformations changed from a relatively wavy surface, to a straighter surface at low loads, and finally a buckled surface at the maximum load, shown in the lower right hand sector of Figure 4.2. The greatest deflection was approximately 0.6 inches.

The compression flange was observed to be rotating slightly throughout the test. A noticeable rotation was evident at a load of approximately 1870 in-kips, which corresponds to the first observations of web buckling. Both the compression flange, and the web showed significant buckling behavior at the maximum load of 2480 in-kips. After the maximum load was reached, the specimen was deflected further to provide clear photographs of the failure and its location. This failure area is shown in Figure 4.3 and Figure 4.4.

Near the failure area, two lines of strain gauges were attached to the web and flanges of the test beam. The attachment of the strain gauges followed manufacturer

Figure 4.3 Test 1 Failure

recommendations utilizing a clean steel surface and a recommended adhesive. All gauges and data channels read satisfactorily during testing. A total of 32 strain gauges were attached to the specimen, and supplied strain information about the specimen during testing.

Figure 4.4 Test 1 Failure

The examination of two strain locations, on the top flange, and in the compression section of the web, shows the load at which buckling occurred. When the strain in the gauge becomes drastically non-linear, buckling stress is reached, and further information is of little use. This is shown in the stress plots of the top

Figure 4.5 Strain/Load Plot for Top Flange Strain Gauge

Figure 4.6 Strain/Load Plot for Top Web Strain Gauge flange strain gauge in Figure 4.5 and the top web strain gauge in Figure 4.6. The load at which this event occurs is identical for both gauges, so it may be concluded that both elements buckled simultaneously.

The curves in Figures 4.7 and 4.8 represent the stress gradient in the cross section of section #11 at two different locations as described in Chapter 3. The figures show typical stress distributions according to strain gauge data, as well as linear elastic theory for the cross section. The curves shown in Figure 4.7 show the distribution at line 1, which is closer to the fixed end of the specimen, resulting in higher stresses throughout the cross section. Figure 4.8 shows the stress distribution for line 2, which corresponds to a lower stress distribution throughout the cross section. Both figures are based on data taken at the 10 kip load increment.

The distributions according to strain gauge data shown in Figures 4.7 and 4.8 are quite different than the straight line representation shown according to linear elastic theory. Strain gauge data taken for both locations were examined for validity by checking that tension forces were equal to compression forces. The data for line 2 was accurate (within 1.3 percent) and is used for further discussion, comparisons and conclusions. The moment calculated using the strain/stress data provided by these strain gauges was within 1.5 percent of linear elastic theory and the measured test moment, validating the strain gauge data for line 2. The summation of forces at line 1 based on the strain gauge data produced a compressive force which was 10 percent greater than the tension force. The inaccuracy of the stress distribution in line 1 may be due to the low measured stress at web gauge location t caused by the holes immediately adjacent to the gauge. The data for line 1, except for location t, are similar to those for line 2.

Figure 4.7 Stress at Line 1

Figure 4.8 Stress at Line 2

The stress distribution for the cross section is non-linear in the compression region of the section, and shows that the prediction of stress in the compression flange for this section is underestimated by approximately 20 percent according to linear elastic theory. The tension stresses in the section are slightly more consistent with linear elastic theory, and differ for each line of strain gauges by the ratio of moment arm for each location, which is consistent with theory. Discussion regarding the Specification and its treatment of non-linear stress distributions in thin element plate girders will be addressed in Section 4.6 of this chapter.

4.2 Test 2

Test 2 was performed on the same specimen that was used in Test 1. The identical loading scheme was used, but the failure area was moved, or "forced", away from the previous failure location. Stiffeners were added to the web near the fixed end of the beam (in section #10) to add shear capacity in this region. The stiffeners are shown in Figure 4.9. The new failure area was approximately two feet closer to the load from the splice between section #10 and #11. This test was conducted to find the moment capacity of the section with the small flange subjected to tension force. The curve in Figure 4.10 shows the final moment-deflection behavior of the specimen in this test.

Figure 4.9 Stiffeners for Increased Shear Capacity, Test 2

The section reached a maximum load of approximately 23 kips and failed through yielding of the thin bottom flange. No local buckling was observed, but a very slight lateral buckle was observed at the final stages of the experiment.

The yield moment capacity of the section is shown on the graph as well as the nominal moment capacity and the plastic moment capacity, as calculated according to the LRFD second edition. The yield moment is calculated as 3190 in-kips, the nominal moment is calculated as 3395 in-kips and the plastic moment capacity is calculated as 4035 in-kips. The actual section yielded at approximately

3750 in-kips and the maximum moment capacity was 3850 in-kips. The maximum was within 5 percent of the plastic moment and was 12 percent greater than M_n from LRFD, Appendix F, which was governed by web buckling. Although yielding behavior controlled the capacity of the section, web deflection was again evident.

Figure 4.10 Moment-Deflection Test 2

The web surface plot shown in Figure 4.11 is a combination of small web section surface plots, using data taken at the noted load points. The web section is approximately 10 inches wide by the height of the web (24 inches). The first section shows the initial web surface, while the last section, at the 24 kips mark, shows the web surface at the end of the test. The shading gradations represent one tenth inch deformations. While the web behavior had little influence on the capacity of the section, the presence of such deformations may be of importance when considering the influence of moment gradient for a singly symmetric section. The surface plot shows a web buckle more closely related to flexural behavior than shear behavior. A comparison of web behavior will be examined for Test 1, Test 2 and Test 5, all of which will be discussed later in this chapter.

Figure 4.11 Web Surface Plot Test 2

4.3 Test 3

Section #10 was placed in the test setup in the same manner as it was originally oriented in the structure. The compression flange was the larger of the two flanges, and behavior was predicted to be limited by yielding of the tension flange. The curve shown in Figure 4.12 shows the moment rotation curve for the section. Rotation was chosen for the displacement value for this section because centerline deflection was affected by the deflection of the beam extensions and slip in the web splices. The rotation data only relates to the behavior between the load points, i.e. the area of the test specimen of interest.

Figure 4.12 Moment-Rotation Test 3

The curve shows calculated capacities according to LRFD Appendix G. The yield moment capacity of the section is approximately 3770 in-kips and the LRFD nominal moment capacity of the section is 3536 in-kips. The plastic moment capacity of the section is 4250 in-kips. The section showed a first yield capacity of approximately 4000 in-kips and a maximum capacity of 4043 in-kips. Both values are within fifteen percent of the conservative values predicted by the Specification. The failure mode was predicted as yielding, which occurred, as well as the section showing some local web and flange buckling.

Figure 4.13 Web Surface Plot Test 3

The web deflection pattern is shown in a similar manner as for Test 2 in Figure 4.13. The figure shows the surface plot as staying static until the higher test loads. This is consistent with the behavior of the section with respect to both the predicted behavior and actual behavior of the section during the test. The web

Figure 4.14 Local Buckle in Compression Flange, Test 3

Figure 4.15 Flange and Web Deformations, Test 3 showed little deflection until the yield moment of the section was reached. At the yield moment the web straightened from its initial wavy shape, and at maximum, the web then showed buckling behavior in the opposite direction as the initial wave prior to testing. Figure 4.14 shows the local buckle in the compression flange present at the final stages of the test. Figure 4.15 shows the slight lateral buckle of the compression flange and the shape of the web at the final stages of the test. These local buckles had little effect on the maximum load carrying capacity of the section. The local buckles simply limited the deformation capacity of the section.

4.4 Test 4

Section #9 was tested in a similar manner as section #10 in Test 3. Again the compression flange was the large of the two flanges, and the section capacity was predicted to be governed by yielding. Figure 4.16 shows the moment rotation curve for the test. The calculated yield moment in the section is 3636 in-kips and the nominal moment according to LRFD Appendix G is 3578 in-kips. The plastic moment capacity of the section is calculated as 4603 in-kips. The section reached a yield moment capacity of approximately 4250 in-kips and a plastic moment capacity of approximately 3600 in-kips under test load. The yield moment test value was slightly lower and within 1 percent of the predicted value, but the method used to arrive at the test value is an approximate visual curve fit. The plastic moment capacity of the section was seven percent under the predicted value. The web of section #9 was also observed to deflect at higher loads.

Figure 4.16 Moment-Rotation Test 4

The surface plot in Figure 4.17 shows the shape of the web after the test was completed. Web buckling was very slight until the latter stages of the test was reached. The surface plot shown, is representative of a classic web buckle, but in this case there was no limitation of the section moment capacity due to the web buckle, this buckle occurred well after the maximum moment capacity of the section was reached. Again, the shading differences represent a deflection variation of one tenth of an inch. Each line on the surface plot represents a five inch spacing on the section web. The 5-inch grid of points used to create the plot was taken from the upper section of the web shown in Figure 4.18.

Figure 4.17 Web Surface Plot After Test 4

Figure 4.18 Section for Web Deflection Readings, Test 4

In addition to the local web buckle present at the end of the test, there was also a slight lateral buckle of the compression flange at the maximum test load. The flange moved a total of one inch from the original alignment, at which point the test was terminated due to safety concerns. This behavior only limited deflection capacity, not moment capacity. The deflected compression flange is shown from the underside of the test specimen in Figure 4.19. Test 5 is the last test performed, and used a section similar to that used for Test 1.

Figure 4.19 Deflection of Compression Flange, Test 4

4.5 Test 5

Section #11 was tested in Test 5. A different specimen than that used for Tests 1 and 2 was used for Test 5, but the section dimensions were very similar. The test was conducted in an identical manner as Tests 3 and 4. The moment-rotation behavior of the specimen is shown in Figure 4.20. As in Test 1, the section is tested with the smaller of the two flanges in compression. The LRFD nominal moment capacity was calculated as 2772 in-kips, controlled by flange local buckling behavior. The yield moment capacity was calculated as 3240 in-kips and the plastic moment capacity was calculated as 4733 in-kips. The maximum moment capacity was 2210 in-kips. The predicted nominal moment capacity was approximately 20 percent greater than the moment reached in the section during the test. The difference between the test and the predicted strength may be due to a slight initial rotation in the compression flange, present in the section prior to testing. Every effort was made to straighten the flange to original geometry, but a slight rotation was still present at the start of testing. The location of the initial imperfection is nearly identical to where the local flange buckle occurred at failure of the section. This buckle is shown in Figure 4.21.

A web buckle was also observed to form as the maximum moment of the test section was reached. The surface plot in Figure 4.22 shows the variation of the web surface at the various loads during the test. Note how sudden the web surface

Figure 4.20 Moment-Rotation Test 5

Figure 4.21 Buckle of Compression Flange, Test 5 changes at the higher load level. This change in surface shape indicates the onset of web/flange buckling in the compression region. The comparison of web shape for Test 5, with Test 1 and 2 is helpful in understanding the influence of moment gradient in terms of web buckling.

Figure 4.22 Web Surface Plot, Test 5

4.6 Experimentation Results Compared with Specification Predictions

Comparing the surface plots on a very general basis, a particular trend is noticeable. In Tests 1 and 2, the web has an initial shape. As the test load progresses, this shape is slowly removed, similar to pulling a sheet tight on a bed. Once the buckling capacity of the compression elements in the section is reached, the web then exhibits buckling behavior, again wrinkling in a similar fashion to its initial shape. The progression of the web surface in Test 5 does not seem to match that description. The web has some initial shape, which is never "straightened", instead the shape is actually enhanced or increases into the final web buckle. This behavior, in addition to the loss of post buckling strength might be a reason for the moment capacity of the section to be lower than that predicted by the Specification.

While there might be a stress concentration near the holes in the web of section #11 in both Tests 1 and 5, the initial imperfections in the plate elements might actually have more to do with actual versus predicted moment capacity. In light of this observation, as well as the trend of web behavior in Tests 1 and 2 versus Test 5, a further investigation into the significance of imperfect plate elements in a section might be warranted. Since linear elastic theoretical and mathematical solutions to buckling equations and behavior assume perfectly straight members, true instability behavior must be examined through experimentation. Another discrepancy between actual behavior and predicted behavior, comes from the definition of the section according to the LRFD.

Section #11 is defined as having a slender web. The Specification states that sections having slender webs, will not have capacities limited by web local buckling behavior. Thus, the Specification states that the section capacity will be limited by the buckling capacity of the flange. The Specification then uses parameters for the flange behavior which consider flange-web interaction, and flange steel strength. The k_c discussed earlier is developed to address the flange-web interaction, according to experimentation and theoretical results, and the flange steel strength is addressed in R_{PG} and R_e. There is evidence from Test 1 which raise questions about the validity of the Specification and its treatment of singly symmetric sections and sections with non-compact or slender compression elements.

Experimental evidence shown in Figure 4.7 shows a non-linear stress distribution for the cross section in Test 1. R_{PG} accounts for the non-linear stress distribution caused by flexural web buckling based on work by Basler (1963), which assumes that sections have compact compression flanges which can reach yield stress. For the section used in Test 1, the calculated Specification R_{PG} value was 0.9997, which indicates no significant non-linear effect. The strain gauge data shows a need for nearly a 20 percent increase in compression flange stress. This translates into a 20 percent reduction of moment capacity for the section, which would require a calculated R_{PG} of 0.80. Thus, the Specification does not accurately consider the non-linear stress distribution in this section.

Although the Specification accurately predicted the capacity of the section, the experimental results described above show a weakness in the Specification and its treatment of sections with non-compact or slender compression elements. The strain gauge data from Test 1, as well as the reduced capacity found in Test 5 might warrant an examination into parameters regarding the buckling performance of sections with non-compact and slender compression elements. The two important parameters of a section found in these tests would be the influence of a non-linear stress distribution in a section (Test 1), and initially imperfect plate elements subjected to compressive forces (Test 5).

Section #11 is constructed in a typical manner for pre-engineered metal building frames. The thin plate elements are welded to each other using only one fillet weld on one side of the beam. Thus the interaction between the flange and the web may not be as predictable, and the Specification might not adequately address this phenomenon. There may also be a significant reduction in capacity of a section if the plate elements, especially the compression flange, are not initially straight. This initial out-of-straightness may be significant for singly symmetric sections subjected to pure moment. It seems as though the moment gradient used in Tests 1 and 2 naturally alleviated any significant imperfections in the web during loading. Another consequence of thin plate elements subjected to compression might be a non-linear stress distribution throughout the section, which would invalidate any analysis based on linear elastic theory. Both conditions, common in real applications of these types of sections, may have a significant effect on the capacity of a member, and should be investigated further.

The effect of natural occurrences due to construction and steel imperfections is difficult to model using theoretical principles. The understanding of beam behavior, especially for those singly symmetric sections with slender or non-compact sections in compression, is more useful when found through experimentation. The following section examines one aspect of Test 1 which might have effected the capacity of the test section.

The specimen used in Test 1 was subjected to a moment gradient, which applies both shear and moment at the failure area. To further understand the effect of shear-moment interaction on the moment capacity of a singly symmetric section the section was subjected to a moment gradient, and a pure moment loading using buckling behavior software developed at The University of Texas at Austin. The results of these computer experiments were compared to the results of other tests and the Specifications.

5. Discussion of Results

5.1 Introduction

The significance of the test results presented in Chapter 4 will be examined using Specifications, theory and the actual test results. A computational finite element program developed for elastic buckling analysis of stiffened plates (BASP), is a valuable tool used in the theoretical examination of singly symmetric beam behavior. Comparisons between BASP results for section #11 and laboratory test data will be made with consideration of previous work by Johnson (1985), Specification recommendations, and the laboratory test results presented earlier. A focal point of discussion will center on moment-shear interaction for a singly symmetric section, specifically section #11. Recommendations will be made using the various test and theory results regarding future research and weaknesses in the Specification.

5.2 BASP Computer Program

BASP is an acronym for Buckling Analysis of Stiffened Plates, which is a finite element computer program developed at The University of Texas at Austin by Akay (1977) and developed for the personal computer by Choo (1987). The BASP program allows for different types of support conditions, both in plane and out-of-plane, but the applied loads must be in the plane of the vertical plate. The program is limited to elastic modeling of initially straight plates in a vertical and horizontal orientation only. Cross sections may be varied, so virtually any I-shaped or T-shaped member may be analyzed. These limitations of initially straight plates and elastic modeling have a significant impact on the comparisons of the theoretical experimentation to the actual beam tests. Initial plate imperfections, and post buckling strength are not accounted for by the program which may yield conservative results.

The model used for Test 1 is a representation of a typical member which can be analyzed using BASP. The finite element mesh used for the Test 1 model is shown in Figure 5.1. The finite element mesh was broken into finer elements near the failure location. The boundary conditions were identical to those used in Test 1. A lateral brace was provided 182.5 inches from the load, which corresponded to the location of the brace in the actual test.

The finite element beam is loaded at the end of the cantilever span with a load of 1 kip in the positive y direction which corresponds to the loading for the actual

Figure 5.1 Finite Element Mesh for Model 1 test. By loading the computer model with 1 kip, the eigenvalue returned by the program for a buckling mode shape serves as a multiplier for the load applied. Thus the eigenvalue returned may be directly read as the critical buckling load for the model.

The output from the BASP program is in the form of a buckling stress, buckling mode, and a plot of the buckling shape of the beam at the buckling load. The buckled shape of section #11 is shown in Figure 5.2. The different lines in the buckling shape correspond to different points on the height of the cross section. Lateral displacement, vertical and horizontal rotation at each cross section in the finite element model, can all be established by careful observation of the output data and plots produced by BASP. All conclusions and references to BASP results occurring in this chapter utilize the typical information presented above.

Figure 5.2   Buckled Shape of Model 1

5.3 BASP Program

A series of analyses were performed using BASP. Each analysis was performed using a model with actual section properties of section #11 for both Test 1 and Test 5 of the beam experiment program. These analyses were performed to help validate and understand the behavior of singly symmetric beams under two different loading conditions. The loading conditions for the BASP tests mirrored the laboratory test conditions and utilized a moment gradient analysis (Test 1), and a pure moment analysis (Test 5).

5.3.1 BASP Modeling for Section #11 Test 1

Three separate models were developed in BASP for the section #11 specimen used in Test 1 of the beam experiments. The first model identically represented the specimen tested in the beam experiment. All dimensions and braces used in the model were identical to those present in Test 1. Thirteen separate runs were made to examine the influence of shear on the moment capacity of the section. The results of these tests will be discussed later in this chapter.

The second model was developed to find the capacity of section #11 used in Test 1 under pure moment, with no influence from shear. The finite element mesh used for this model is shown in Figure 5.3. This model used a panel that was square in dimension, i.e. as long as it is deep. The section was 24.330 inches deep and 24.330 inches long. Point loads were applied at the ends of the model to produce pure moment. Web thickness was increased from 0.198 inches to 0.300 inches to eliminate local deformations. The thicker web spread the stresses throughout the section, without restraining rotation as much as a vertical stiffener at the end boundary. The buckled shape and critical buckling moment of the model is shown in Figure 5.4. The solution for critical moment is consistent with values attained using the first model, and test results for Test 1. The moment and shear capacities calculated using BASP for these two models, is shown in Table 5.1.

Figure 5.3   Finite Element Mesh for Model 2

The third model used for section #11/Test 1, is a panel with minimal end stiffeners and restraints subjected to pure shear. The finite element mesh is shown in Figure 5.5. This model again had identical cross section dimensions as the other models, only restraints and stiffeners are different to allow shear buckling of the web to control capacity of the section. The shear capacity for section #11 for Test 1 was found to be 63.964 kips. The buckled shape produced by BASP for this test is shown in Figure 5.6.

Figure 5.7 shows the surface plot for the web of the second and third models for section #11. The plot with a high, symmetrically rounded wave shows the web surface for a buckled web due to pure moment (section #11, Test 1 and section #11, Test 5). The plot with the oval, flat buckled shape represents the buckled web due to pure shear (section #11, Test 1). The two buckled shapes are noticeably different.

Table 5.1 BASP Section Capacities
Test 1 Section #11

Model #1 (Test 1)	Moment Capacity (in-k)	Shear Capacity (kips)
Run 1	2320	10.17
Run 2	2326	10.95
Run 3	2332	11.85
Run 4	2336	12.90
Run 5	2337	14.12
Run 6	2332	15.01
Run 7	2323	15.98
Run 8	2307	17.05
Run 9	2280	18.21
Run 10	2235	19.42
Run 11	2157	20.54
Run 12	2016	20.64
Run 13	1754	19.42
Run 14	1457	17.56
Model #2 (Pure Moment)	2412	N/A
Model #3 (Pure Shear)	N/A	63.96

Figure 5.4   Buckled Shape of Model 2

Figure 5.5   Finite Element Mesh for Model 3

5.3.2 BASP Modeling for Section #11 Test 5

The models used to examine the behavior of section #11 tested in Test 5 are quite different, but yield similar results. The first model represented the actual test beam used in Test 5. Stiffened extension beams were attached to the three foot section of interest for the test. Two symmetric loads were placed at a distance from the supports, similar to the actual laboratory test. The second model is a duplicate of the pure moment model used for examining Test 1 capacity as described above. Both models yielded similar capacities.

Figure 5.6 Buckled Shape of Model 3

Four runs were performed using the first BASP model. All moment values calculated from the critical load values varied by at most three percent. Each run used a slightly different grid, with the same boundary conditions, which were identical to those used in the beam experiment. One run was performed on the second pure moment model described above. The moment capacities calculated in each run using BASP for these models is shown in Table 5.2.

Table 5.2 BASP Section Capacities Test 5 Section #11

Model #1 (Test 5)	Moment Capacity (in-k)
Run 1	2835
Run 2	2820
Run 3	2798
Run 4	2775
Model #2 (Test 5) Pure Moment	2792

5.4 Comparison of BASP, Experiment and Specification Capacities

Moment and shear capacities for singly symmetric beams have been calculated, analyzed and found through testing. Test 1 was designed to examine the

Buckled Shape of Web due to Pure Moment

Buckled Shape of Web due to Pure Shear

Figure 5.7 Surface Plots of Buckled Webs capacity of a section subjected to both moment and shear, a loading pattern not addressed by the AISC specifications. Test 5 was designed to examine the capacity of a section subjected only to moment. The test capacities, theoretical capacities and Specification capacities all differ for each of the tests. This section will address possible reasons for differences, and discuss the results of Test 1 and Test 5 with respect to each other, and to the results presented in BASP and the Specification. Test 1 will be examined first.

5.4.1 Test 1

The moment capacity found in Test 1, corresponds to a load at a distance of 186 inches from the failure location (2480 in-k, shown in Table 5.3). This failure load subjects the specimen, at the failure location, to both moment and shear, each of which is a percentage of the capacity of the section subjected to only moment, or only shear. The moment capacity for the section tested is greater than both the capacity calculated using BASP, and the capacity calculated according to Appendix G in the LRFD specification. While all three capacities differ, the difference is less than ten percent between all capacities, which signifies that the test is reasonable and that the capacity calculated by BASP is not too conservative compared to the test or Specification capacities. The three different capacities are listed in Table 5.3.

The capacity calculated according to the Specification does not consider shear. The calculation is made for a section subjected to pure moment. Test 1 and BASP produce moment capacities that is influenced by both shear and moment. There may be a weakness in the code since all equations are derived from doubly symmetric sections, and most of those sections were compact. There is no definite evidence from either the experiment or BASP capacities to validate or disprove the Specification. The examination of shear and moment interaction in section 5.5 of this chapter will address a possible reason for the accurate prediction by the Specification.

Table 5.3 Section Capacities
Section #11 (Test 1)

Derivation of Capacity	Moment Capacity (in-k)
Test 1 (experiment)	2480
BASP (average)	2320
Specification (LRFD)	2414

5.4.2 Test 5

The moment capacity found in Test 5, does not compare favorably with Specification capacity or BASP values for capacity. The different capacities according to each method are shown in Table 5.4. The test moment capacity for section #11 differs from the other capacities by more than twenty percent. The BASP and Specification values are within 2 percent. An investigation into previous work on beams subjected to pure moment was done with BASP. A test performed by Johnson (1985) with a similar h/t ratio to that of section #11 was examined using a pure moment model in BASP, similar to that used for Test 5.

The moment capacity found for the similar Johnson beam using BASP was approximately 10 percent lower than the capacity found by Johnson in a beam test. This difference between BASP and experiment capacities is consistent with information found in Test 1 for this thesis, and post buckling strength of real buckled web elements (Johnson, 1985). A plot showing the k_c interaction for Johnson's tests (1985), and Test 1 and Test 5 is shown in Figure 5.8. The k_c values, calculated using similar methods to those used by Johnson, calculated for Tests 1 and 5 are considerably lower, especially Test 5. The values shown are for h/t and h_c/t in each test. This information may contribute to the decreased moment capacity observed in Test 5.

Table 5.4 Section Capacities
Section #11 (Test 5)

Derivation of Capacity	Moment Capacity (in-k)
Test 5 (experiment)	2210
BASP (average)	2805
Specification (LRFD)	2772

With all aspects of the section considered, the low moment capacity of section #11 in Test 5 may possibly be attributed to the initial imperfection present in the flange prior to loading. Although the flange was straightened to within one-sixteenth of an inch, the failure occurred at almost the exact location of the initial imperfection. The capacity found using BASP will then be higher than the test capacity because BASP only considers perfectly straight

plate elements. Theory and experience show that initial imperfections in any straight element subjected to compression, significantly effects the stability of that element. Following from these observations, Test 5 may be considered to not truly represent actual behavior or moment capacity of a singly symmetric section subjected to pure moment.

5.5 Shear-Moment Interaction

The investigation of shear-moment interaction will be limited to section #11, specifically that section which was used in Test 1. A series of 14 BASP runs were performed on the cantilever model of section #11 described in section 5.2 and shown in Figure 5.1. These 14 runs examined different levels of shear subjected to the specimen by moving the point of load closer to the fixed end of the cantilever. The results of these tests are shown in Table 5.1, and are graphically illustrated in Figure 5.9.

The curve in Figure 5.9 examines the moment capacity of a section, in the presence of shear. The moment for the section, is non-dimensionalized by the moment found in the pure moment BASP run for this section. This is the point of highest moment and zero shear for the section (2412 in-k). The shear for the section is non-dimensionalized by the ultimate shear found in the pure shear BASP run for this section (64 kips). Values for pure moment and pure shear for section #11 are shown in Table 5.1.

Figure 5.9 Shear and Moment Capacity Interaction Model 1

The configuration of Test 1 was such that a moment capacity of 2320 in-kips was reached at a shear value of 10.2 kips. This location is noted on the curve in Figure 5.9. For this configuration, the shear applied was less than 16 percent of the shear capacity as calculated in BASP. In this region of the curve, there seems to be little effect of shear on the moment capacity of the section. At a value of approximately 30 percent of the shear capacity of the section, it seems that shear begins to have a significant effect on moment capacity of the section. The behavior of this singly symmetric section would seem to indicate that at higher levels of shear, not common to many pre-engineered structures, moment capacity may be seriously reduced. Test 1 was the only beam experiment in which this interaction was investigated. The loading scheme, and in turn the moment gradient present in the original structure, did not lend itself to a high enough moment gradient to accurately examine the effect of shear on moment capacity of singly symmetric sections. Further investigations may be warranted to examine this phenomenon.

5.6 Recommendations and Conclusions

The singly symmetric sections investigated in this research have yielded information worthy of further research regarding the specifications, shear-moment interaction and the behavior of singly symmetric sections with initial imperfections. The specifications do not directly address the design and analysis of singly symmetric sections, but they do seem to supply the engineer with a reasonable estimation of moment capacity of a straight singly symmetric section.

All sections tested, except for section #11 in Test 5, exceeded the Specification estimation of moment capacity. The Specification is also within 5 percent of theoretical capacities calculated using the BASP program. While the BASP program does not consider post buckling strength, the Specification is based on both theory and experimental results, which does consider post buckling strength. From this information it seems that the Specification is more aligned with theory than experimentation. A possible weakness within the Specification might be the lack of experimentation supporting the estimate of capacity for singly symmetric sections.

The curve presented in Figure 5.9 shows the possible concern with regards to the influence of shear on the moment capacity of a singly symmetric section. BASP considers all possible loads and conditions which might cause unstable behavior in a section. One of the key parameters for this singly symmetric section behavior is the web flange interaction. Further experimentation might be extremely helpful in clarifying the cause of moment capacity deterioration in the presence of high shear and also the low k_c values reported for Test 5. Further research may also yield valuable insight into the apparent lack of consideration for non-linear stress distributions found in sections with non-compact or slender compression elements.

While many structures may not be subjected to both high shear and high moment, the phenomenon is not impossible, and should be investigated further to ensure proper design and analysis of these sections. Further research in this area may be linked with a program designed to examine the effect of initial imperfections on singly symmetric sections with non-compact or slender elements. In addition, this research could target conditions like stress concentrations, section changes, and material properties which might cause non-linear stress distributions in sections and stiffness changes as seen in the strain gauge data for Test 1.

While Test 5 seems to be an invalid test for comparison of theoretical, experimental and Specification supported capacities, it does represent a possible problem with this type of highly efficient non-compact section. Most sections have compact compression flanges which would not be affected by small imperfections. Many pre-engineered sections have non-compact or slender compression flanges, which may be sensitive to small imperfections. The low moment capacity seen in Test 5 indicates that sections that do not have compact compression flanges, might be susceptible to imperfections in the compression elements. The section tested in Test 5 had appreciable imperfections in both the web and the compression flange. The imperfection in the compression flange was removed to a reasonable degree, but damage may have already taken place in the section geometry and material. Further tests on similar sections with imperfections may help validate this concern. A moment capacity twenty percent less than the Specification and theoretical value should be a cause for concern, and examined appropriately.

Many structures utilize the singly symmetric section, and all are designed using a specification based on doubly symmetric sections. The small changes that have been made for the second edition of the LRFD, like h_c, help lead the engineer to design conservatively, but the experimental results are not present to support these changes. Areas of importance seem to be the moment-shear interaction for these sections, initial imperfections in the compression elements of these sections, web-flange interaction for sections with non-compact and slender compression elements, and non-linear stress distributions present in some singly symmetric cross sections.

6. Conclusions

The Specification readily handles any member with compact compression elements, and accurately predicts behavior and capacity for sections with non-compact or slender compression elements. One of the primary reasons for the Specification predicting a reasonable value for capacity, is the post buckling strength of a section. Without the post buckling strength of a section, the Specification may possibly give an unconservative prediction for capacity. Parameters like k_c and R_{PG} do a good job of predicting behavior and capacity for doubly symmetric and compact members, but are questionable for singly symmetric and non-compact or slender members. Further investigation should be done to ensure that safe designs of singly symmetric beams follow from the Specifications. Certain limits and specific discussion of R_{PG} and k_c should be added to the Specification for all sections, especially singly symmetric members and those with non-compact and slender compression elements. This would also help alleviate some of the problems inherent in trying to follow the logic path presented in the Specification.

The experimentation performed in the research for this thesis will help further investigations into behavior and strength of singly symmetric sections. Two cantilever tests and three pure moment tests were performed on four different sections, with compact, non-compact and slender elements in the cross section.

The specimens tested were taken from actual field constructed frames, complete with initial imperfections present in most sections. Varied cross sections and material properties were tested in each specimen, with reasonable results for each of the five tests, except for Test 5, where capacity was 20 percent lower than the predicted Specification and theoretical values. Web buckling was present in all tests, and flange buckling occurred in three of the five tests. Lateral torsional buckling was eliminated by providing adequate lateral bracing of the section during all tests. Above predicted performance was observed in all tests that had compact compression flanges. The presence of non-compact compression flanges and slender webs reduced capacities of the sections to buckling behavior limits. A non-linear stress distribution was observed in one of the tests, indicating an unconservative estimate of compression flange stress according to linear elastic theory. Initial imperfections in plate elements, non-linear stress distributions and shear-moment interaction all may have affected the moment capacity of the singly symmetric sections examined in this research. Correlation of experimentation and Specification capacities was examined through the use of theoretical modeling.

The finite element program BASP was used to analyze typical singly symmetric sections with non-compact compression flanges and slender webs. The values attained from these analyses correlated reasonably well with Specification values and behavior observed in experiments. One pure moment test, Test 5, showed a capacity 20 percent lower than the Specification prediction (no phi factor), and provided a low k_c value when back calculated in accordance with previous work by Johnson (1985). This low flange-web interaction parameter may be attributed to a common occurrence: initial imperfections in plate elements. The importance of initial imperfections in compression elements becomes less of a concern as the compression flange becomes more compact. One aspect examined theoretically, that should be further investigated experimentally, is the influence of shear on the moment capacity of singly symmetric sections with non-compact and slender compression elements. Results from the BASP analysis showed that the moment capacity dramatically deteriorates when beams are subjected to approximately 30 percent of the ultimate shear capacity for that section. The finite element program does not consider post buckling strength, so this concern may not be as urgent as the data would suggest, but it is a very important unknown that must be investigated further. This phenomenon combined with the different k_c values found in these tests as compared to those found by Johnson (1985) warrants further investigation of singly symmetric sections.

Another significant recommendation for future research would be the determination of the cause of the non-linear stress distributions in singly symmetric cross sections with non-compact or slender compression elements. Data from Test 1 shows that linear elastic theory underestimates the compression flange stress by more than 20 percent for a section with a non-compact compression flange, and a slender web. While stress concentrations due to various section changes may change stiffness in the web or flange, the non-linear distribution is valid and indicates a glaring weakness in the Specification and the treatment of sections with non-compact or slender compression elements.

7. Bibliography

Akay, H.U., Johnson, C.P., and Will, K.M. "Lateral and Local Buckling of Beams and Frames." *Journal of the Structural Division*, ASCE, ST9, September, 1977, pp. 1821-1832.

AISC. *Manual of Steel Construction, Allowable Stress Design*, 9th ed. Chicago, IL: American Institute of Steel Construction, 1989.

AISC. *Manual of Steel Construction, Load and Resistance Factor Design, Volume 1*, 2nd ed. Chicago, IL: American Institute of Steel Construction, 1994.

Basler, Konrad. "Strength of Plate Girders in Bending." *Transactions*, ASCE, 128, Part II (1963), pp. 655-686.

Choo, K.M. Thesis presented to The University of Texas at Austin, May, "Buckling Program BASP for Use on a Microcomputer." 1987.

Galambos, T.V. AISC Committee Correspondence, 1994.

Johnson, D.L. (1985) An Investigation into the Interaction of Flanges and Webs in Wide-flange Shapes *1985 Proceedings SSRC Annual Technical Session.*

Salmon, Charles G., Johnson, John E. Steel Structures, Design and Behavior Emphasizing Load and Resistance Factor Design, 3rd ed. New York, NY: HarperCollins*Publishers*, Inc., 1990.

Yura, J.A. Committee Correspondence, 1991.

Yura, J.A. Personal Correspondence, 1994.

7. Bibliography

Akay, H.U., Johnson, C.P., and Will, K.M. "Lateral and Local Buckling of Beams and Frames." *Journal of the Structural Division*, ASCE, ST9, September, 1977, pp. 1821-1832.

AISC. *Manual of Steel Construction, Allowable Stress Design*, 9th ed. Chicago, IL: American Institute of Steel Construction, 1989.

AISC. *Manual of Steel Construction, Load and Resistance Factor Design, Volume 1*, 2nd ed. Chicago, IL: American Institute of Steel Construction, 1994.

Basler, Konrad. "Strength of Plate Girders in Bending." *Transactions*, ASCE, 128, Part II (1963), pp. 655-686.

Choo, K.M. Thesis presented to The University of Texas at Austin, May, "Buckling Program BASP for Use on a Microcomputer." 1987.

Galambos, T.V. AISC Committee Correspondence, 1994.

Johnson, D.L. (1985) An Investigation into the Interaction of Flanges and Webs in Wide-flange Shapes *1985 Proceedings SSRC Annual Technical Session.*

Salmon, Charles G., Johnson, John E. Steel Structures, Design and Behavior Emphasizing Load and Resistance Factor Design, 3rd ed. New York, NY: HarperCollins*Publishers*, Inc., 1990.

Yura, J.A. Committee Correspondence, 1991.

Yura, J.A. Personal Correspondence, 1994.