Behavior of crimped cold-formed steel C-section beams

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BEHAVIOR OF CRIMPED COLD-FORMED STEEL C-SECTION BEAMS

by

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ABSTRACT

This work covers the behavior of crimped cold-formed steel C-section beams under flexural loading. The specific beam shape that was tested was 600S162-54. The crimps were placed in the flange and web of the beam to curve the beam. Three crimp sizes were tested: 0.5 degree, 1.5 degree, and 3.0 degree. In construction practice, the company placed a reinforcing steel band that overlapped the crimped areas to restore strength. These various crimp degrees were tested to evaluate how much strength, if any, was lost. The reinforcing steel band was tested simultaneously with the beam to evaluate if its presence significantly increased the strength and ductility of the beam. The beams were tested with the crimps on both the compression flange and the tension flange to cover updraft and other upward vertical loads. Straight beams without crimps were used as the control case, and both crimped and control specimens were evaluated for their moment capacity, deflection limits, and the effective moment of inertia of the beam.

After testing, tables comparing the crimped beams' moment and deflection capacity and their effective moment of inertia compared to straight beams were created. Based on the series of flexural tests, the deeper the crimp geometry extended into the compression zone of the beam, the greater the loss of moment capacity, and the lower the effective moment of inertia. Tables 5.1 through 5.5 along with Equations 2 through 7 are meant to aid in the safe design of crimped cold-formed steel beams. Testing Standards such as ASTM A370 and the SSMA Cold-Formed Steel Flexural Members were followed, but with minor deviations because of the unique geometry of the crimped beams.

ACKNOWLEDGEMENTS

Thank you to everyone who made this research possible. I give my special thanks to Amanda Steele for all of her help with the beam specimen testing. The testing would never have finished so quickly without her help. Thank you to Dr. Baur, Dr. LaBoube, and Dr. Birman for giving me the opportunity to conduct this research, and for keeping me on track to writing this thesis and analyzing the data. I am also thankful to Dr. Wei-Wen Yu for the Wei-Wen Yu Graduate Fellowship award, which helped fund the research.

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NOMENCLATURE

SYMBOL	DESCRIPTION
A	Cross-sectional area of the curved beam
A_m	Integral of the change in area over the radius of the beam
B	Sum of primary and secondary crimp dimensions
C	Sum of primary, secondary, and Tertiary crimp dimensions
E	Modulus of Elasticity
I	Effective Moment of Inertia at service
L	Member Length
M_c	Moment capacity of beam with crimp in compression flange
M_m	Ultimate Moment Capacity of Crimped Beam
M_s	Straight Beam Ultimate Moment Capacity
M_t	Moment capacity of beam with crimp in tension flange
M_{ult}	Ultimate Applied Moment
a	Primary crimp Dimension
b	Secondary Crimp Dimension
b'_i	Effective width of the inner flange of the beam
b'_o	Effective width of the outer flange of the beam
c	Tertiary Crimp Dimension
d	Beam depth
d_c	Compression zone depth
r	Radius of curvature of the beam
r_i	Radius of curvature of the innermost fiber
r_o	Radius of curvature of the outermost fiber
t_w	Thickness of the web
y	Moment arm
δ	Member service deflection
σ_t	Tension stress in the beam
σ_c	Compression stress in the beam
θ_c	Degree of crimp in compression flange
θ_t	Degree of crimp in tension flange
%B	% of d_c that is occupied by B

1. INTRODUCTION

The subject of this thesis was the behavior of crimped cold-formed steel C-section beams. The beams were created by placing a series of crimps in the top or bottom flanges of the members. These crimped members are used for various curved shaped forms, due to their economic benefits and ease in fabrication. Such members have been used in archways, domes, and other architectural and decorative forms. The number of crimps and the degree of bend differ depending on the length of the span, the amount of loading required, and the desired architectural shape. The different degree bends that were tested include 0.5, 1.5, and 3.0. These bend degrees were chosen by the manufacturer for testing. A reinforcing band was also sometimes placed over the crimped section in an attempt to recover lost capacity from the crimping process. This research focused on testing the three degree bends previously mentioned as well as testing the crimped beams with and without the reinforcing band. Only the 600S162-54 beam cross-section was tested.

1.1. OVERVIEW OF EXPERIMENT

A series of flexural tests were performed using a four-point bending configuration. A diagram of this configuration can be seen in Figure 1.1. The test method provided a visual analysis of the stresses in the flanges of the member. Testing for both compression and tension in the crimped flange was accomplished by placing the crimp on the top of the beam for compression flange testing, and on the bottom of the beam for tension flange testing. The reason for the compression flange testing was to account for the presence of uplift forces that would place the crimped flange of the beams in compression. Besides uplift forces, the crimp could be placed in the compression flange using alternative designs, such as the column capital design in Figure 1.2. Testing the crimp in the tension flange simulated normal gravity loads that are experienced during the construction phase, and service life of a typical structure.

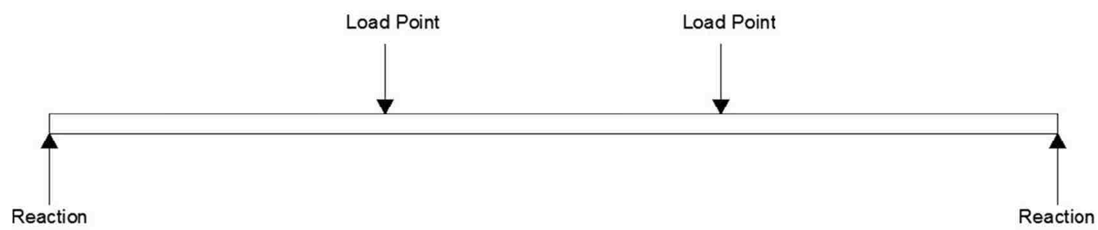


Figure 1.1 Four-Point Bending

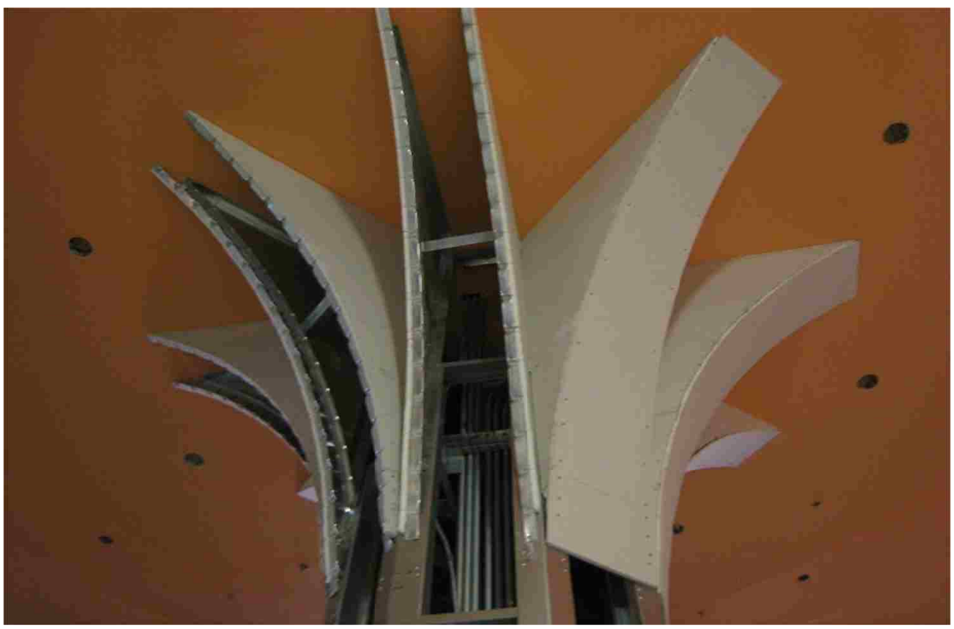


Figure 1.2 Beam Configuration that Places Crimps in Compression

The particular subject of study of these crimped cold-formed steel beams was the reduction in moment capacity and serviceability as a result of the crimping process. The variables in the study were: the degree of the bend, the presence of a reinforcing strap, and if the crimp is on the compression or tension flange. The reduction in moment capacity that was caused by these variables was tabulated, and strength reduction factors were tabulated for safe design. The serviceability of the crimped beams was also assessed, and whether further restrictions were necessary to limit service deflections. The serviceability of the beams was evaluated for design by calculating the effective moment of inertia of the crimped beams. Thus, the following C-section configurations were tested:

- Straight member
- Curved member with crimp in tension
- Curved member with crimp in compression
- Curved member with crimp in tension reinforced by a strap
- Curved member with crimp in compression reinforced by a strap

1.2. OVERVIEW OF ANALYSIS

The data that was gained from the experiments was analyzed to calculate the total moment that was applied to each C-section beam at failure. The maximum moment for the straight beams were considered the baseline for comparison. A ratio of the moment capacity for crimped beams to straight beams was calculated for each configuration. This ratio was used to create strength reduction factors for the capacity of the various crimp degrees while taking into account if the crimp was in compression or tension. The crimped beams' serviceability was also analyzed, using the applied loads and deflections to calculate the effective moment of inertia of the crimped beams. The effective moment of inertia was tabulated for each beam and was intended for use in the serviceability checks for beam design. A table was created which shows how the beam types perform against deflection requirements L/180, L/240, and L/360 at service level loads. Service level loads were defined as loading that resulted in 60% of the ultimate moment capacity. The deflections in the serviceability analysis were conservative results compared to in-place construction. The experiment was conducted using rollers, but in construction and design practice, semi-rigid connections are used. Pinned and fixed connections would have experienced smaller deflections than those seen in this experiment. Once a correlation was seen between the ultimate moment capacity and the values of certain dimensions of the crimp, equations were created to predict the capacity of intermediate crimp degrees. Lastly, in an attempt to compare the moment capacity of the crimped beams to that of curved beams, two different methods for evaluating the moment capacity of curved beams were demonstrated. The results yielded unreasonable values when compared to the experimental ultimate moment capacities of crimped beams with similar radii.

2. LITERATURE REVIEW

The literature review for this thesis yielded some related works. There have been previously related studies done on the behavior of curved beams, and this information can be found in Advanced Mechanics of Materials [Boresi 2003]. The challenge is that the theories address continuous curved beams. The beams in this experiment were made to curve by a crimping process, but the beam was straight between the crimped areas. Similar research has been done in the past on corrugated steel panels [Jorgenson 1973] [Jorgenson 1982] which are created by a similar process. These panels are used as structural siding for buildings such as barns and other low importance structures. These studies of corrugated panels; however, do not address the behavior of crimped beams. There has also been research done on cold-formed steel curved panels [Sivakumaran 2000] [Xu 2001], but these works, while helpful, are not applicable to the behavior of crimped beams. The Society for Automotive Engineers have also done research on curved beams, in which more specific, less general, curved geometries were analyzed. The Winkler-Bach equation was also created to predict the moment capacity of curved beams, but none of these theories are applicable to the special geometry of crimped beams. The loading and geometry of these structural members is unique to this particular research. Searching for prior studies of crimped cold-formed steel beams has yielded minimal related research.

3. METHODOLOGY

The bending of curved beams was not explicitly covered by any particular testing standard, but because the curvature of the beams was limited by the length of the specimens, testing standards for straight beams were easily adapted for use with crimped beams. ASTM A370 was used for testing definitions and methodology, as well as for the coupon testing which verified the strength of the steel. The testing setup used for the bending test of the beams was hydraulic and complies with the requirements in ASTM E4. The machine operated by specifying a deflection rate for the beams, which it then followed while measuring the applied load. The Steel Stud Manufacturers Association (SSMA) Cold-Formed Steel Flexural Members Test Procedure was used as the basic guideline for the entire setup of the test, although variations were made because of the crimps which are detailed below. The SSMA guideline specified the use of C-section beams and outlined the proper interval of the bracing angles, depending on the design parameter that was being tested. Using this procedure, two beams were tested per specimen, which allowed for a shorter testing phase.

3.1. TENSILE COUPON TEST SETUP

The tensile coupon tests were completed following the specifications in ASTM A370. The zinc coating was removed using a 10% solution of hydrochloric acid. The coupons were cut from one of the beams after the beam was tested. The portion of the beam that the coupons were cut from was not in the pure moment zone, and therefore had not failed during beam testing. All of the beams were produced from the same coil of steel, so each beam had approximately the same strength properties. The tension tests were performed on an MTS 880, which was displacement controlled, and the applied tension load was measured. The typical coupon tensile test setup can be seen in Figure 3.1. The first coupon was pulled at a rate of 0.063 inches/minute until yielding occurred. After yielding occurred, the rate of pull was increased to 0.125 inches/minute to decrease the amount of time needed for testing. For the second and third tests, the rate of deflection was increased to 0.3 inches/minute after yielding to decrease the time needed for the test. The data showed that this change in the displacement rate did not affect the amount of load needed to yield or rupture the material nor was the maximum deflection affected.

Figure 3.1 Typical Tensile Coupon Test Setup

3.2. BEAM TEST SETUP

The designated size of the beams was 600S162-54. This means that each member was a C-section with an edge stiffener, a nominal depth of 6 inches, a nominal flange width of 1.625 inches, and a minimum delivered material thickness of 0.054 inches. The thickness that was measured was 0.0566 inches. All other dimensions were within 1/32" of the nominal dimensions. A series of tensile coupon tests were performed to verify that the yield strength of the steel was a minimum of 50 ksi. Testing revealed the yield strength of the steel was 50.5 ksi. Each beam was 72 inches long, and each specimen consisted of 2 beams. The beams were connected face-to-face with 1" x 1" x 1/8" aluminum angles. The angles were 6 inches long, which corresponds to the side-to-side distance of the assembled specimen. An example of a longitudinal view of beam specimen can be seen in Figure 3.2. A section view of the beam specimen is shown in Figure 3.3.

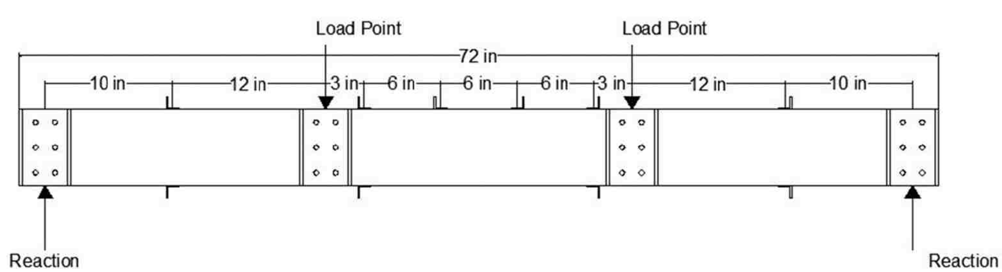


Figure 3.2 Longitudinal View of Beam Test Specimen

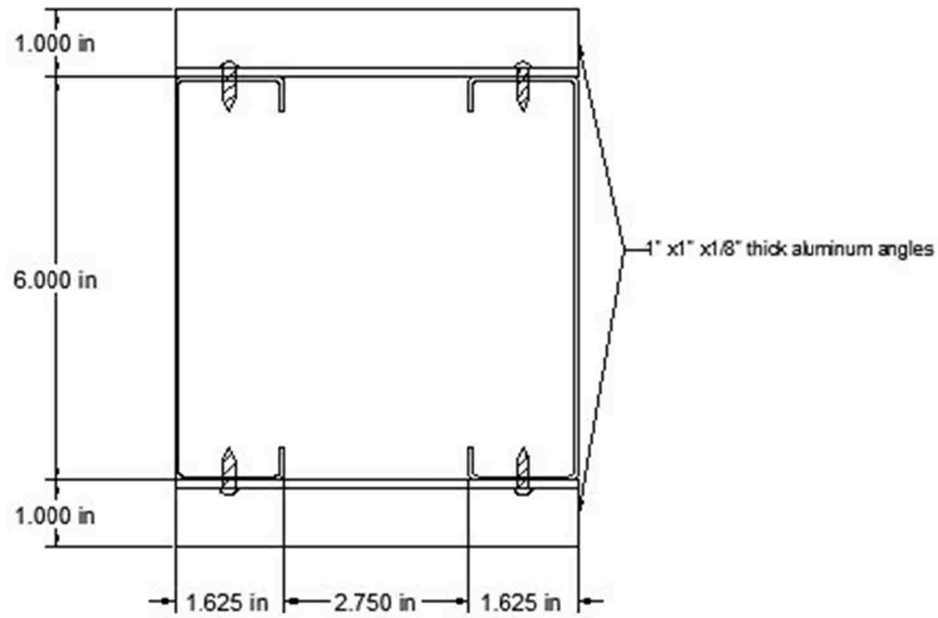


Figure 3.3 Section View of Beam Specimen

Variations made to the SSMA guidelines are as follows. On the compression flanges of the specimen within the pure bending zone, the aluminum angles were placed at 6 inch intervals. The pure bending zone was located between the two load points that were placed on the beam. Due to the spacing of the crimps, the 3 inches spacing requirement as specified by the SSMA guideline was modified to 6 inches. The angles were spaced 18 inches apart on the tension side of the beam, at either end of the pure bending zone. The 12 inch SSMA guideline was not followed because of the geometry of the beams. The length of the pure bending zone did not allow for a 12 inch spacing along with symmetry around the pure bending zone. The importance of symmetry was considered, so that each half-length of the beam behaves the same as the other half-length. Another set of angles were placed 12 inches from the end of the beam to resist torsional buckling on the tension and compression flanges. Web stiffener brackets were screw attached to the exterior sides of each beam under each of the load points to restrict web crippling. Wooden blocks were attached with screws to the interior of the beams under the load points to reduce local buckling. The beams were tested using the simple span condition. This was accomplished by placing rollers at both ends of the member, and using clamps to restrain the beam to the rollers. Because of this connection configuration, the connection was idealized as a roller. The beams had a four-point loading configuration. The reason for the 4-point loading configuration was to create the pure moment zone in the interior of the beam's span. This pure moment zone, ideally, had no shear stress present, so the effects of shear were neglected. An example of the typical test set-up for straight and curved beams can be seen in Figures 3.4. This four-point configuration creates a constant moment region in the mid-section of the beam between the two point loads.

Figure 3.4 Typical Beam Test Setup

The first tests were done using a load rate of 0.01 inch/minute, but these tests took an unreasonable amount of time, and produced an excessive amount of data. For this reason the loading rate was changed to 0.02 inch/minute. After analyzing the data, it was decided that the increased loading rate had no effect on the validity of the test. A total of 31 specimens were tested in bending, where each specimen consisted of two C-section beams, for a total of 62 beams tested.

Figure 3.5 shows the layout for crimps placed in the tension flange of the beam in the pure moment zone of the beam. In the picture can be seen the placement of the aluminum angles to prevent lateral movement in the tension flange and compression flange, as well as the reinforcement band that was placed over the crimped area. Figure 3.6 shows the corresponding view of the pure moment region with crimps in the compression flange. It can be seen in these pictures that the aluminum angle spacing was independent of the location of the crimps, and was dependent upon which flange of the beam was experiencing compression stress.

Figure 3.5 Tension Flange Crimp Layout

Figure 3.6 Compression Flange Crimp Layout

4. RESULTS

The results of the experimentation showed that the degree of crimp greatly affected the ultimate moment capacity and the effective moment of inertia of the beam. In general, a larger degree of crimp meant a greater reduction in capacity, with the exact behavior differing when the crimp was placed in the compression or tension flange. When the crimp was in the compression flange, the decrease in the ultimate moment capacity of the beam began at more rapid rate than if the crimp was in the tension flange. The presence of the reinforcing band increased the moment capacity of the beams between approximately 2%-7%, with the single exception being the 1.5 degree crimp in compression which saw a 33% increase in its moment capacity. Another exception was the 0.5 degree crimp in compression with the reinforcing band which experienced a 2% decrease in its moment capacity. The accompanying graphs in the Appendix are "Moment vs. Deflection" instead of "Load vs. Deflection" because the moment arm varied slightly between 24 inches and 22.5 inches due to changes in the setup between experiments. To account for this variance of the moment arm, the applied moment was calculated to more accurately represent the moment capacity of the beams.

4.1. TENSILE COUPON TESTS

The coupons yielded above 2,000 lb. of force, and ruptured at roughly 2,800 lb. of force. After yielding, the coupons had a large amount of deflection that continued until strain hardening began, and then the strength increased once again until rupture occurred in the coupon. The yield strength of the tensile coupons was approximately 50.5 ksi. The behavior of the coupons in tension was uniform, and for this reason only a typical Stress-Strain Diagram is included in Figure 4.1, instead of all three coupons. The data from these coupon tests verified that the minimum yield strength of the beams in the flexural tests was 50 ksi, and that their rupture strength was 65 ksi.

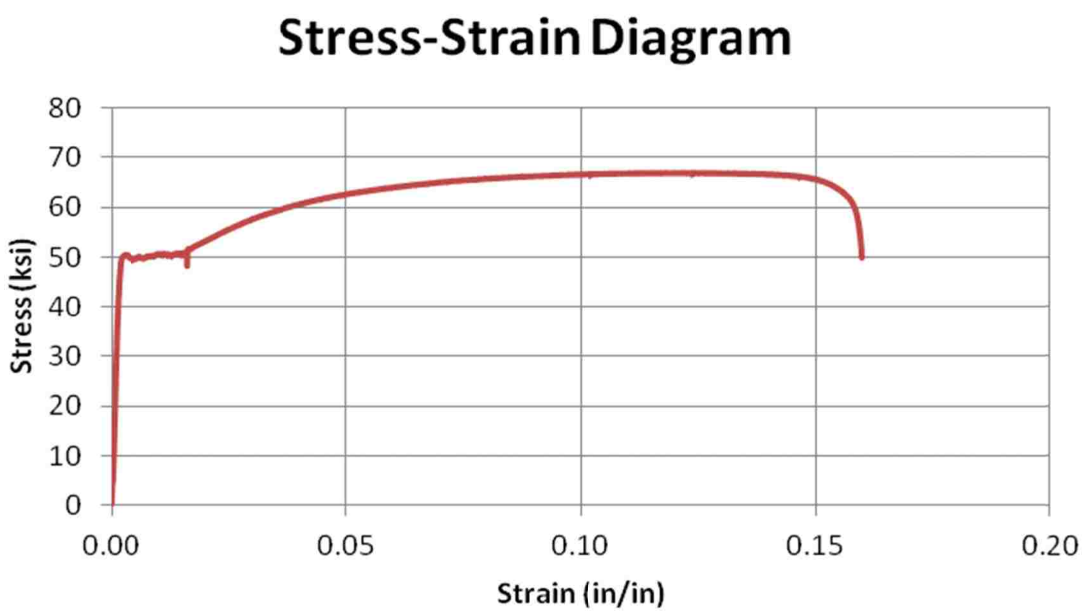


Figure 4.1 Typical Stress-Strain Diagram of a Tensile Coupon

4.2. TYPICAL FAILURE MODES

The mode of failure for the beams was local buckling. If the crimp was in the tension flange, the typical location for the local buckling was directly under the load point. Under the load point the flange buckled downward, and next to the load point, just inside the pure moment region, the flange buckled upward. A picture of this failure can be seen in Figure 4.2. The presence of the band did not affect the failure mode for the beams. Whether the reinforcing band was present or not, the beam still failed in local buckling directly under the load point.

Figure 4.2 Typical Failure of Crimp in Tension Flange

When the crimp was in the compression flange, the local buckling always took place at one of the crimp locations. This failure also took place very gradually, instead of the sudden failure that the beam experienced when the crimp was in the tension flange. The crimp acted as a predetermined failure point for the loading, and as load increased, the crimp deepened. After the crimp reached a maximum allowable load the beam only deflected further without taking any more load. A picture of this failure can be seen in Figure 4.3. The presence of the band over the crimps did not change the mode or location of the failure; it still occurred at the crimp. The additional failure that took place when the reinforcing band was placed over the compression crimps was that the reinforcing band also buckled in compression. This failure took place simultaneously with the local buckling at the crimp. This buckling of the reinforcing band took place between the rivets which held the band to the beam. This buckling failure of the reinforcing band can be seen in Figure 4.4.

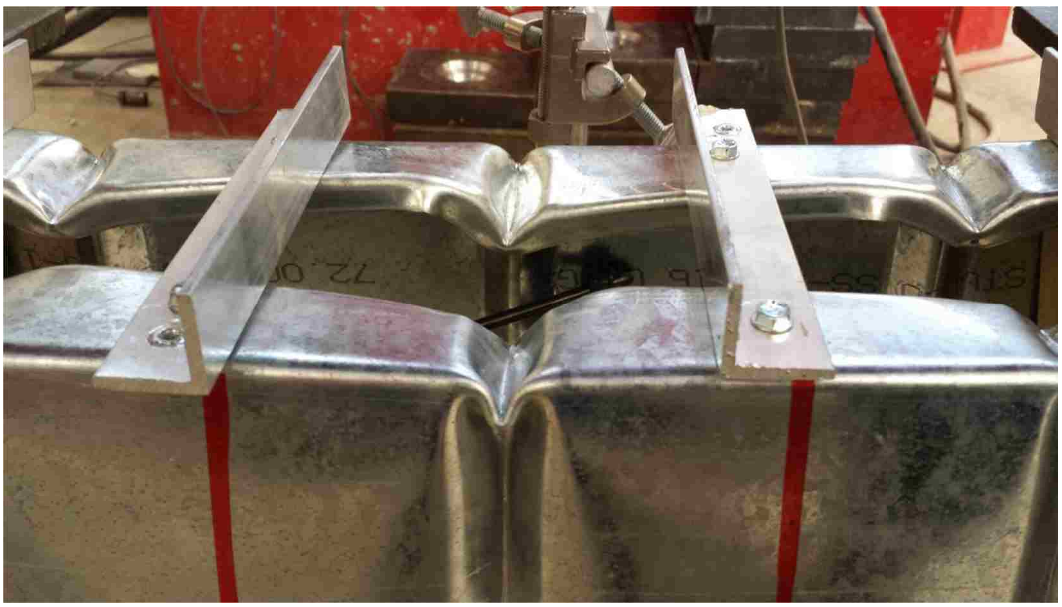


Figure 4.3 Typical Failure of Crimp in Compression Flange

Figure 4.4 Typical Failure of Crimp in Compression with Reinforcing Band

4.3. 0.5 DEGREE CRIMP

When placed in tension, the crimp of the 0.5 degree bend beam caused little to no reduction in ultimate moment capacity. This is shown in Figure 4.5. The ultimate applied moment on the 0.5 degree crimped beam, both with and without the reinforcing band, was approximately 45 kip-in. This was similar to the ultimate moment capacity of the straight beam. The 0.5 degree crimped beam's deflection was increased relative to the straight beam, and the beam deflected more than twice as far as the straight beam. The crimped beam reached its ultimate moment capacity of 45 kip-in after 0.65 inches of deflection, as opposed to 0.3 inches for the straight beam. During loading, the beam deflected downward and the crimp was gradually pulled out, which reduced the crimp size until the crimp became non-existent. When the beam deflected to the point of being practically straight, then the beam began to buckle under the loading in much the same way a straight beam would in local buckling under the load points in the pure bending zone of the beam.

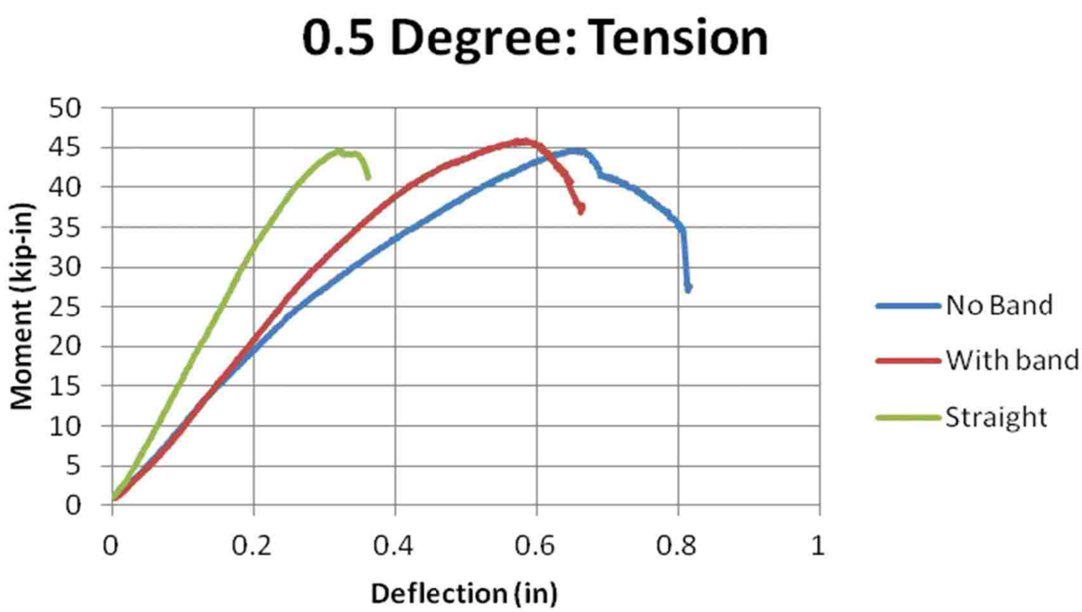


Figure 4.5 0.5 Degree Crimp in Tension Fiber

The presence of the band over the crimp when the crimp was in tension did not significantly increase the ultimate moment capacity of the beam, but the reinforcing band did decrease the amount of deflection at the ultimate load. The beam still failed at 45 kip-in, but it reached the ultimate moment at 0.6 inches instead of 0.65 inches. Also, instead of straightening out entirely before taking on its ultimate load, the beam resisted the load immediately and failed in local buckling before the beam was completely straight.

When the 0.5 degree crimped beam was placed in compression flange, both the beam's moment capacity, and resistance to deflection were reduced. The behavior of these beams can be seen in Figure 4.6. The compression flange crimped beam experienced a greater amount of deflection before reaching its ultimate moment capacity, about 0.4 inches instead of 0.3 inches. Along with this increased deflection, the ultimate moment capacity was decreased to approximately 35 kip-in. With the crimp on the compression flange of the beam, the crimp was a predetermined point of failure. This failure point at the crimp allowed for a gradual failure in which the crimp deepened, which allowed for greater deflection as compared to the straight beam.

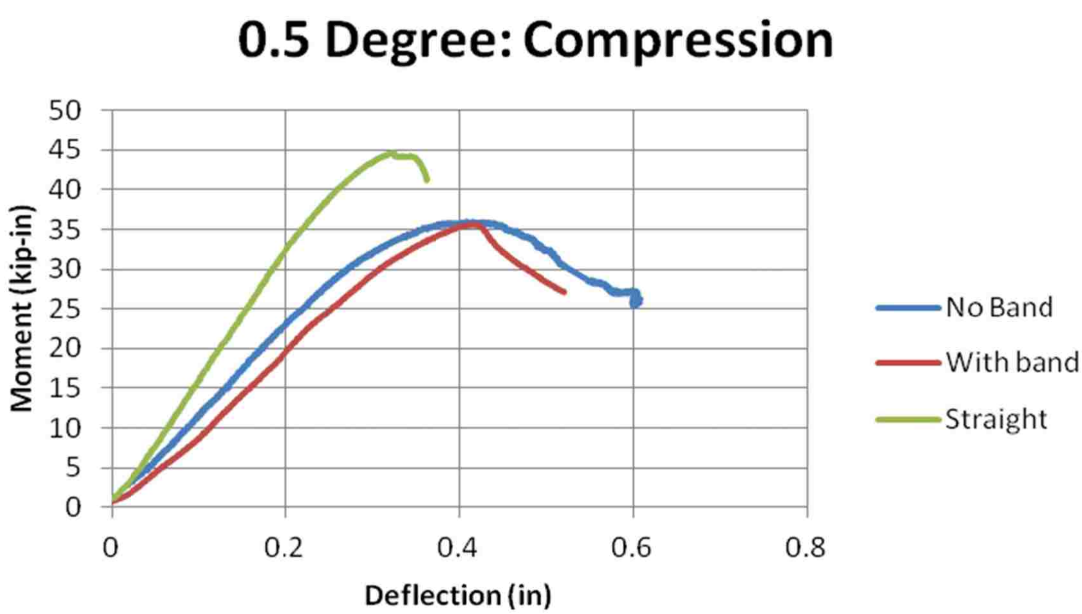


Figure 4.6 0.5 Degree Crimp in Compression Fiber

The presence of the reinforcing band over the crimp on the compression flange did not reveal any noticeable increase in the beam's moment capacity, and Figure 4.6 actually indicates a decline both in the beam's ultimate moment capacity, and resistance to deflection. The ultimate moment was still found to be 35 kip-in for the beam, and the beam achieved this failure at 0.4 inches of deflection. The beam failed by folding around the crimp, but there was an additional failure in the reinforcing band. When in compression, the reinforcing band buckled over the point of crimping in the beam simultaneously with the folding around the crimp.

4.4. 1.5 DEGREE CRIMP

The 1.5 degree bend crimp in the tension flange had roughly the same behavior as the 0.5 degree crimp in the tension flange, although the deflection at failure was larger. This similarity can be seen in Figure 4.7. The 1.5 degree crimped beams, both with and without the reinforcing band, had approximately the same ultimate moment capacity as the straight beam, 45 kip-in. The 1.5 degree crimp yielded at approximately the same amount of loading as the straight beam before it failed in local buckling in the pure bending zone. This behavior resulted in a higher deflection, approximately 38.5% more than the 0.5 degree crimp in tension. The deflection at ultimate loading was approximately 0.9 inches, compared to 0.3 inches for the straight beam.

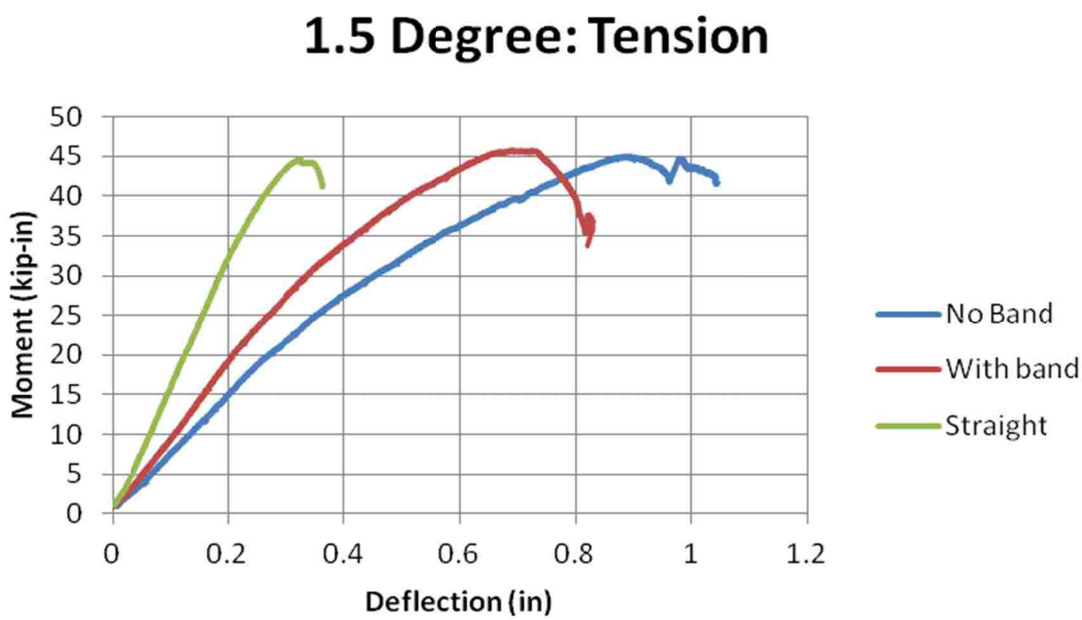


Figure 4.7 1.5 Degree Crimp in Tension Fiber

The inclusion of the reinforcing band over the 1.5 degree crimps in the tension flange for the beam slightly increased the moment capacity of the beam, by approximately 3.5% as can be seen in Figure 4.7. The amount of deflection at the ultimate applied moment was also reduced, similar to the deflection reduction in the 0.5 degree crimped beam with the reinforcing band. The deflection at the ultimate moment capacity with the reinforcing band was 0.7 inches instead of 0.9 inches for the crimped beam without the band.

When the 1.5 degree crimp was in the compression flange of the beam, the ultimate moment capacity was roughly half that of the straight beam. A graph of this beam's behavior can be seen in Figure 4.8. The beam failed at approximately 22.5 kip-in, and the associated deflection at failure was 0.4 inches. Like the 0.5 degree crimp in compression, it failed gradually by folding around the crimp. When the reinforcing band was present over the crimps in the compression flange, the ultimate moment capacity of the beam increased by approximately 33%, and the amount of deflection at failure was reduced. The beam failed at approximately 30 kip-in, and the deflection at failure was slightly less than 0.4 inches. The beam still failed at the points of crimping, but the band failed simultaneously in a buckling failure over the point of crimp.

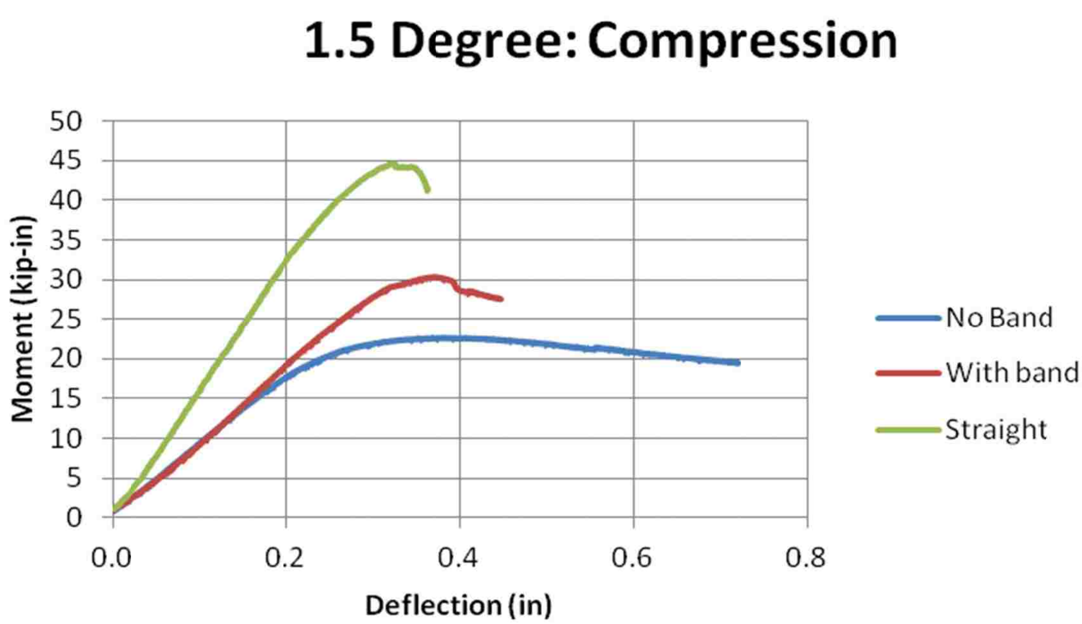


Figure 4.8 1.5 Degree Crimp in Compression Fiber

4.5. 3.0 DEGREE CRIMP

When the 3.0 degree crimp was in the tension flange, the beam gradually deflected downward until it was practically straight, then the beam began to fail in local buckling failure. At 37.5 kip-in, this beam was the only one with crimps in the tension flange that saw a significant decrease in ultimate moment capacity, when compared to the 0.5 and the 1.5 degree crimped beams. The behavior of the 3.0 degree crimped beam can be seen in Figure 4.9. The deflection was also large, because the beam straightened out before failure. The deflection at failure was 1.65 inches. The presence of the reinforcing band over the crimps did reduce the deflection at failure, but the band did not significantly increase the beam's ultimate moment capacity.

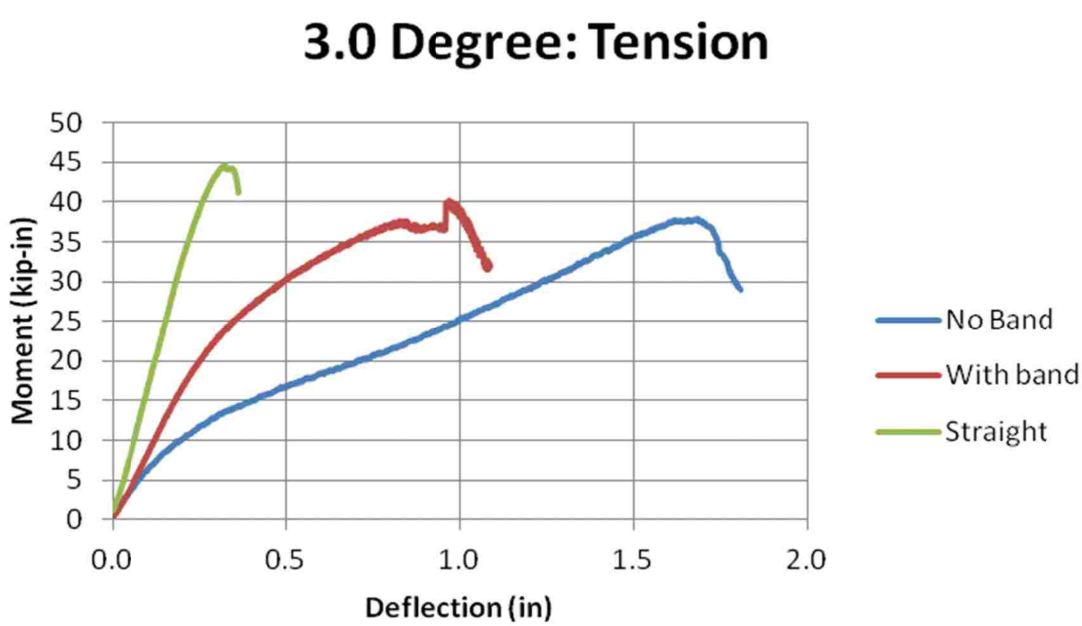


Figure 4.9 3.0 Degree Crimp in Tension Fiber

When the 3.0 degree crimp was in the compression flange, the beam failed gradually and there wasn't a sudden or peak moment for the point of failure. The beam's behavior can be seen in Figure 4.10. The applied load reached its plateau at approximately 15 kip-in and the crimps folded under the applied load. When the beam reached this plateau, its deflection was approximately 0.4 inches. The presence of the reinforcing band over the crimps in the compression flange did increase the moment capacity of the beam, but this increase was not significant.

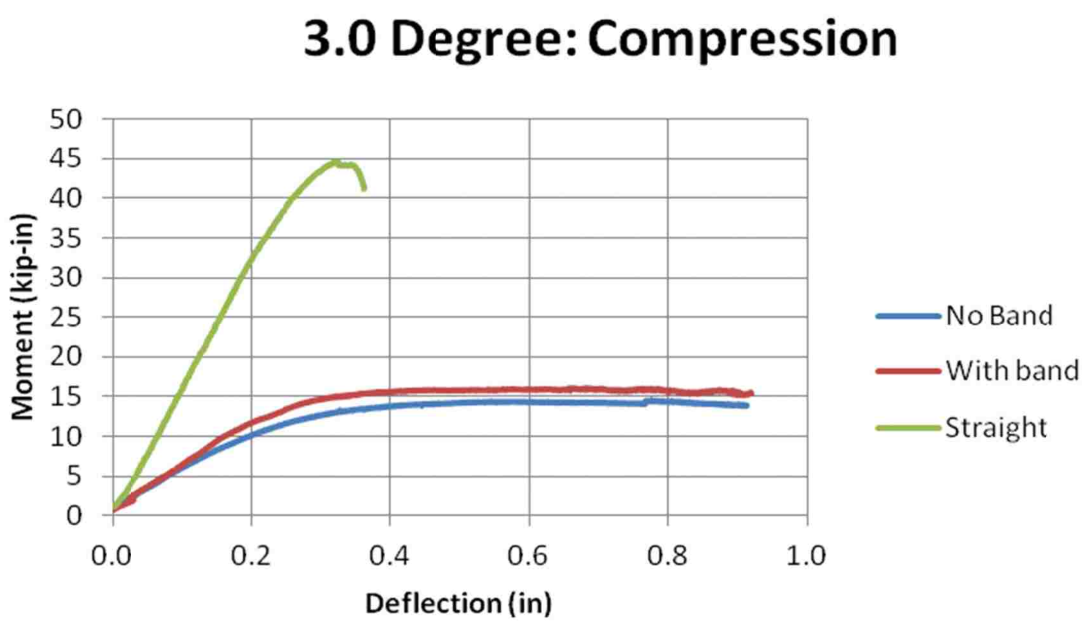


Figure 4.10 3.0 Degree Crimp in Compression Fiber

5. DISCUSSION

The effects of the crimps on the moment capacity of the beam, and the deflections of the beams under loading were used to calculate the effective moment of inertia for the crimped beams. The effects of the reinforcing band on the moment capacity and the beam's effective moment of inertia are also discussed. The results of the experimentation showed that as the indentation of the crimp increased the beam's ultimate moment capacity and effective moment of inertia decreased. As the effective moment of inertia decreased, the amount of deflection that occurred from the same amount of applied moment increased. The amounts by which these properties decrease are discussed further. Tables 5.1 through 5.4 tabulate the effects of the different crimps had on the beam's moment capacity and effective moment of inertia. Tables 5.5 through 5.7 and equations 2 through 7 give information that can be used to predict the ultimate moment capacity of crimps other than 0.5, 1.5, or 3.0. This collection of tables will help with the safe design with crimped cold-formed steel C-section beams in the future.

5.1 MOMENT CAPACITY

The goal of this research was to assess how the crimp affected the moment capacity, and deflection capacity of the crimped beams when compared to straight beams. The data showed that a small amount of crimping, such as 0.5, or 1.5 degree, on the tension face of the beam did not greatly reduce the moment capacity of the beam, although the deflection caused by the moment was increased. There was a general trend that the deeper the geometry of the crimp descends into the compression zone of the beam cross-section, the greater the loss of moment capacity. The 0.5 degree and 1.5 degree crimps in the tension flange did not significantly affect the moment capacity of the beam, but the 3.0 degree crimp in the tension flange did decrease the moment capacity. There was an observed reduction in moment capacity of 1% between the 0.5 and 1.5 degree crimped beams in the tension flange, but the 3.0 degree crimped beam in the tension flange saw a 16% reduction in its moment capacity when compared to the 0.5 degree crimped beam. This observation is further explored in Section 5.4. This trend also followed the reasonable result that the deeper the crimp in the compression zone, the greater the reduction in moment capacity. A tabulated list of beam types, their ultimate moment capacity, service moment capacity, and the ratio of the crimped beam's moment capacity to that of a straight beam are presented in Table 5.1 below.

Table 5.1 Moment Capacity of Beam Types

Beam Type	Reinforcing Band	Ultimate Moment Capacity (kip-in)	Service Moment Capacity (kip-in)	Ratio of Moment Capacity of Beam to Straight Beam
Straight	No	43.72	26.23	1.00
0.5/Compression	No	32.79	19.67	0.75
0.5/Compression	Yes	34.54	20.72	0.79
0.5/Tension	No	45.47	27.28	1.04
0.5/Tension	Yes	46.34	27.80	1.06
1.5/Compression	No	21.42	12.85	0.49
1.5/Compression	Yes	30.17	18.10	0.69
1.5/Tension	No	45.03	27.02	1.03
1.5/Tension	Yes	45.47	27.28	1.04
3.0/Compression	No	14.43	8.66	0.33
3.0/Compression	Yes	15.3	9.18	0.35
3.0/Tension	No	38.04	22.82	0.87
3.0/Tension	Yes	38.47	23.08	0.88

There was no significant increase in moment capacity from the presence of the reinforcing band of the 0.5 degree crimp or 1.5 degree crimped beams when the crimp was in the tension flange. This was demonstrated by the fact that the ultimate moment capacity for both of these beam types was 45 kip-in, regardless of whether the reinforcing band was present or not, as can be seen in Figure 4.5 and 4.7. This behavior probably occurred because the typical failure mode for these cases was local buckling in the compression flange and this failure mode is not restrained by the presence of a reinforcing band on the tension flange of the beam.

In the case where a 0.5 degree crimp was placed in the compression flange then there was significant loss of moment capacity, which decreased from 45 kip-in for the straight beam to 35 kip-in. This was nearly a 25% reduction in moment capacity for the 0.5 degree crimp in compression. When the band was present on the 0.5 degree compression crimp, then the data actually showed a decrease in moment capacity, although it was not significant. This probably occurs because the reinforcing band failed before the beam did, and its buckling preempted the failure of the beam.

When a 1.5 degree crimp was placed in the compression flange of the beam a more significant decrease in moment capacity took place, which dropped the moment capacity from 45 kip-in for a straight beam to 22.5 kip-in for the crimped beam. This amounted to a 50% reduction in moment capacity for a 1.5 degree crimp in the compression flange. This was a very significant drop in moment capacity. When the reinforcing band was present, then there was an increase in moment capacity up to 30 kip-in. This was the only beam type in which a significant increase in moment capacity occurred from the presence of the reinforcing band. The reason appeared to be that the reinforcing band's compression capacity was approximately the same as the compression capacity of the 1.5 degree crimp area, so that instead of the beam or the band failing before the other, they shared the stress and allowed for a higher moment capacity before failure occurred. This is merely a hypothesis, and further research should be done to explain why this beam type was the one that increased its ultimate moment capacity when the reinforcing band was present.

When a 3.0 degree crimp was placed in the tension flange of the beam, then there was a reduction in the moment capacity from 45 kip-in to approximately 37.5 kip-in. This was approximately a 15% reduction in moment capacity for the 3.0 degree crimp in tension. This reduction in capacity in the 3.0 degree crimp beam occurred, because the stress of the 3.0 degree crimp extended deep enough into the beam that the locations under the crimp became a point of localized failure. When the 3.0 degree crimp was in the tension flange the geometry of the crimp was deep enough to extend into the compression zone of the beam. This caused local buckling to occur more quickly The presence of the band did not significantly increase the moment capacity of the beam, although in Figure 4.9, after the first drop in moment capacity occurred, a spike in moment capacity was observed. After the beam had failed in local buckling and the reinforcing band had not yet yielded, the reinforcing band took the stress from the beam, and allowed a momentary increase in the beam's moment capacity before it buckled.

When the 3.0 degree crimp was placed in the compression flange of the beam, then its moment capacity was reduced to 15 kip-in from 45 kip-in. This was a 66% reduction in moment capacity for the 3.0 degree crimp in the compression flange compared to the straight beam. This reduction in moment capacity occurred because the deeper crimp geometry descended into the compression zone of the beam. The presence of the reinforcing band over the crimped area of the beam increased the moment capacity by 7%. This increase in moment capacity could be attributed to the reinforcing band, which had not buckled like the rest of the beam due to the crimping process.

The general trend for the crimped beams was that the deeper the crimp geometry extended into the compression zone of the beam, the greater the loss in moment capacity. When the crimp was in the compression flange, it acted as a predetermined failure point. It was observed that the area around the crimp was weakest, and failure often occurred there. When the crimp was in the tension flange, the resulting tensile stresses uncrimped the tension flange and returned the beam to a relatively straight shape. If the tension flange crimp extended into the compression zone of the beam, such as the 3.0 degree tension crimp, then a loss of moment capacity occurred. The presence of the steel reinforcing band was found to have a minimal effect on the restoration of the lost moment capacity of the beam. The exception was when the 1.5 degree crimp was in the compression flange. In this instance, the steel reinforcing band did significantly increase the moment capacity of the beam.

5.2 MOMENT OF INERTIA

For an accurate calculation of the expected deflection at service in design, the moment of inertia was necessary. The moment of inertia was calculated from the load and deflection data. The effective moment of inertia decreased as the crimp degree became larger. This trend was true for both the crimps in the compression flange and the tension flange. The moment of inertia was typically lower when the crimp was on the tension flange of the beam, compared to the compression flange. The presence of the reinforcing band increased the effective moment of inertia of the beam; however, the amount by which the band increased the moment of inertia of the beam was not consistent. The effective moment of Inertia was calculated using service level loads before yielding began taking place. As the steel yields and strain-hardening begins, the modulus of elasticity decreases, so service level loads were used to avoid this change in the modulus of elasticity. Equation 1 was used to calculate the effective moment of inertia at each beam, depending upon its ultimate moment, and the amount of deflection that was seen at service level loads. The moment of inertia values for crimped beams and straight beam are given in Table 5.2.

$$I = 0.6M_{ult}[3L^2 - 4y^2]/(24E\delta) \quad (1)$$

Table 5.2 Moment of Inertia by Beam Type

Beam Type	Reinforcing Band	Moment of Inertia (in^4)	Itest/Istraight
Straight	No	2.45	1
0.5/Compression	No	1.87	0.76
0.5/Compression	Yes	2.05	0.84
0.5/Tension	No	1.72	0.70
0.5/Tension	Yes	1.99	0.81
1.5/Compression	No	1.75	0.71
1.5/Compression	Yes	1.84	0.75
1.5/Tension	No	1.29	0.53
1.5/Tension	Yes	1.73	0.71
3.0/Compression	No	0.99	0.40
3.0/Compression	Yes	1.14	0.47
3.0/Tension	No	0.5	0.20
3.0/Tension	Yes	1.46	0.60

The strength reduction factors were determined by the relation of the crimped beams' moment capacity to the moment capacity of the straight beam, which can be seen in Table 5.1. Based on the experimental results the strength reduction factors can be seen in Table 5.3. These values should be applied to the allowable yield moment capacity of a straight beam to account for the effect of the crimp in safe design. The strength reduction factors are based solely on the research, and further study should be done to refine these factors and verify that they are accurate.

Table 5.3 Strength Reduction Factors

Curvature (degree)	Tension Flange Crimp	Compression Flange Crimp
0.5	1	0.75
1.5	1	0.5
3	0.86	0.33

5.3. DEFLECTION

For the deflections of the beams under service loads the general trend was that the deeper the crimp geometry descended into the beam the larger the amount of deflection that occurred for a given applied moment. Deflections were generally smaller at service loads if the crimp was in the compression flange of the beam. While the steel reinforcing band did not have a significant impact on the moment capacity of the beams, the band did generally decrease the amount of deflection that occurred at service loads. This was attributed to the increase in the effective moment of inertia that occurred because of the band. The tension flange crimp beams showed a higher deflection than the compression crimps because the tension stresses pulled the crimp out of the beam before significant amounts of load were carried. This allowed for large deflections early in the service life of the beam.

A minimum standard for the allowable deflections, L/180, was selected. For the experimentation length of 6 feet, this amounted to a maximum allowable deflection of 0.4 inches at service loads. The service load was defined as the moment that was applied to the beam at 60% of the beams maximum moment capacity before failure. For the L/180 criteria, the only beam type that did not pass was the 3.0 degree crimped beam in tension without the reinforcing band. This was not surprising because these beams had large displacements, and became nearly straight, before they took on a significant amount of load. At 0.87 inches, the 3.0 degree crimped beam's deflection was more than double the allowable deflection. Based on this observation, use of the 3.0 degree crimp in the tension flange without a reinforcing band is not recommended.

Another beam that nearly failed the L/180 deflection limit of 0.4 inches was the 1.5 degree crimp in tension without the reinforcing band. This beam type had service deflections of 0.397, barely passing the allowable limit. Because of this, the use of the 1.5 degree crimp in tension without the band is also not recommended.

An intermediate deflection limit of L/240, or 0.3 inches, was used to further assess the restrictions on the use of these beams. Several more beam types failed this requirement, and a common occurrence in the experimentation phase was that one test of the beam type would fail this requirement, but the other test passed the requirement. If the average of the two beams' deflections at service load was below the requirement, then the beam was stated to have passed the requirement. The beam types that failed the L/240 requirement were: 0.5 degree crimp in tension without the reinforcing band, the 1.5 degree crimp in tension both with and without the band, and the 3.0 degree crimp in tension both with and without the reinforcing band. The beams would become nearly straight before resisting a significant amount of load, and as a result revealed large deflections during experimentation. Based on the observation that most beams failed in deflection when compared to an allowable deflection of L/240, it is recommended not to use similar beams for applications requiring allowable deflections smaller then L/180.

For the instance of strict deflection requirements, a maximum deflection of L/360, 0.2 inches for the experiment, was selected. The 0.5 degree crimp in compression with the reinforcing band and the 1.5 and 3.0 degree crimp in compression, both with and without the reinforcing band, passed this requirement. The beams passed the L/360 service load deflection because of their low ultimate moment capacity, thus decreasing the beam's moment of inertia that occurred from the crimps. The 0.5 crimp in compression with the reinforcing passed due to its increased moment of inertia that the reinforcing band provided. The straight beam narrowly failed due to its high moment to and it exceeded its allowable deflection. A list of beam types and their deflection criteria results are shown in Table 5.4. These beam deflection criteria results will change for different beam lengths.

Table 5.4 Deflection Criteria Results

Beam Type	Reinforcing Band	Service Deflection (in)	L/180 (0.4 in.)	L/240 (0.3 in.)	L/360 (0.2in.)
Straight	No	0.2	Pass	Pass	Fail
0.5/Compression	No	0.194	Pass	Pass	Fail
0.5/Compression	Yes	0.19	Pass	Pass	Pass
0.5/Tension	No	0.302	Pass	Fail	Fail
0.5/Tension	Yes	0.264	Pass	Pass	Fail
1.5/Compression	No	0.14	Pass	Pass	Pass
1.5/Compression	Yes	0.19	Pass	Pass	Pass
1.5/Tension	No	0.397	Pass	Fail	Fail
1.5/Tension	Yes	0.302	Pass	Fail	Fail
3.0/Compression	No	0.16	Pass	Pass	Pass
3.0/Compression	Yes	0.154	Pass	Pass	Pass
3.0/Tension	No	0.86	Fail	Fail	Fail
3.0/Tension	Yes	0.301	Pass	Fail	Fail

5.4 MOMENT CAPACITY EQUATIONS FOR CRIMPED BEAMS

All of the failures that occurred in the crimped beams during testing occurred in the compression flange. Because of this, the compression stress was critical regardless of whether the crimps were placed in the tension or compression flange. If the crimps were in the tension flange, then the moment capacity was not significantly affected, unless the crimp was 3.0 degrees which resulted in the crimp’s geometry continuing deep into the web of the beam. This reoccurring mode of failure led to the hypothesis, the decreased moment capacity occurred specifically because the crimp’s geometry interfered with the stress distribution in the compression zone of the web and the compression flange of the beam. To test this hypothesis, measurements were taken of the different sections of the crimp. The crimp geometry was separated into its “a”, “b”, and “c” dimensions. Table 5.5 and Figure 5.1 illustrate different parts of the crimp geometry and its sizes depending upon the crimp degree.

Table 5.5 Crimp Measurements

Curvature/crimp (degrees)	Crimp Measurements		
	a (in)	b (in)	c (in)
0	0	0	0
0.5	0.132	0.878	2.064
1.5	0.178	0.806	2.181
3	0.358	0.663	2.544

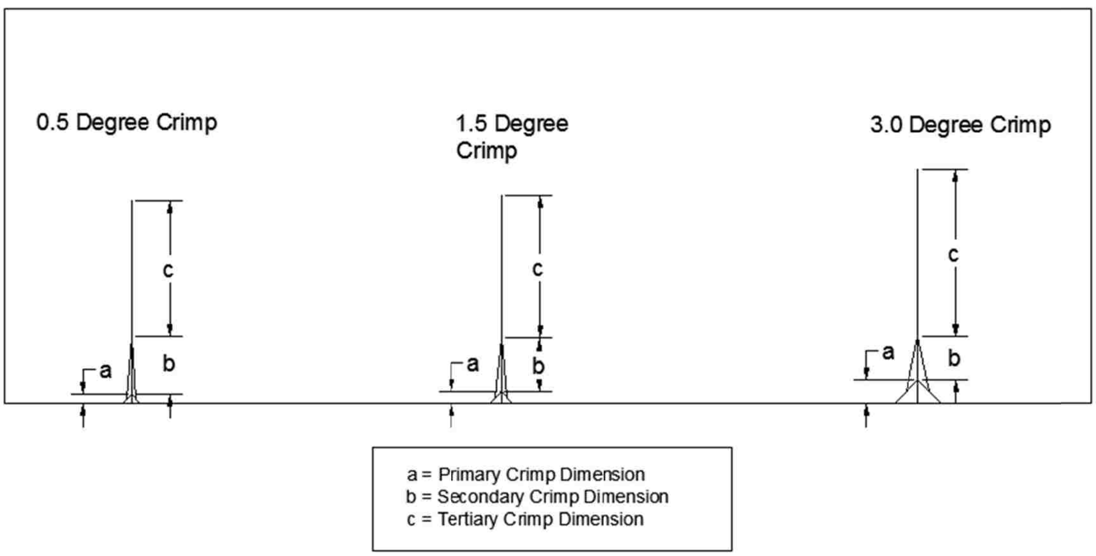


Figure 5.1 Crimp Dimension Diagram

The crimp geometry measurements were not affected by the presence of the reinforcing band, so it is not included as a factor in Table 5.5. Along with these crimp measurements, the compression stress at failure was calculated for each beam type, and these compressive stress values were used to calculate the location of the neutral axis of each crimped beam type. A summary of these calculations can be found in Table 5.6.

Table 5.6 Compression Zone Size

Beam	Reinforcing Band	Compression Zone size (in)	mpression zone that is affected by the di			Average Maximum Moment Moment (kip-in)
			%a	%B	%C	
0/NA		2.35	0	0	0	43.72
0.5/tension		2.93	0	0	0.02	45.56
0.5/tension	Yes	2.73	0	0	0.00	46.21
1.5/tension		3.34	0	0	15.07	45.02
1.5/tension	Yes	2.92	0	0	2.92	45.68
3/tension		4.39	0	0	44.61	38.08
3.0/tension	Yes	2.92	0	0	16.54	38.46
0.5/compression		2.27	5.80	44.38	100.00	32.85
0.5/compression	Yes	2.26	5.83	44.62	100.00	34.52
1.5/compression		1.84	9.70	53.58	100.00	21.41
1.5/compression	Yes	2.23	7.99	44.15	100.00	30.20
3.0/compression		2.06	17.39	49.57	100.00	14.39
3.0/compression	Yes	1.96	18.31	52.18	100.00	15.39

Where

%a = % of d_c that is occupied by a

%B = % of d_c that is occupied by B

%C = % of d_c that is occupied by C

d_c = depth of the compression zone

$$B = a + b$$

$$C = a + b + c$$

Different combinations of crimp geometry were paired to find a correlation, and the best correlation which illustrated the relationship was between the ultimate moment capacity and the percent amount that the B dimension penetrated in to the compression zone of the beam’s web. The larger the amount of the B dimension that was in the compression zone, the lesser the ultimate moment capacity of the beam. Figure 5.2 below shows this correlation between the B dimension in the compression zone and the corresponding ultimate moment capacity. Equation 2 was developed to predict the moment capacity of a beam depending on the amount of the B dimension. Equation 3 calculates the %B for the beam if the crimp is in the tension flange, and Equation 4 calculates the %B for the beam if the crimp is located on the compression flange. Equation 5 calculates d_c, the compression zone depth, which is implemented in Equations 3 and 4.

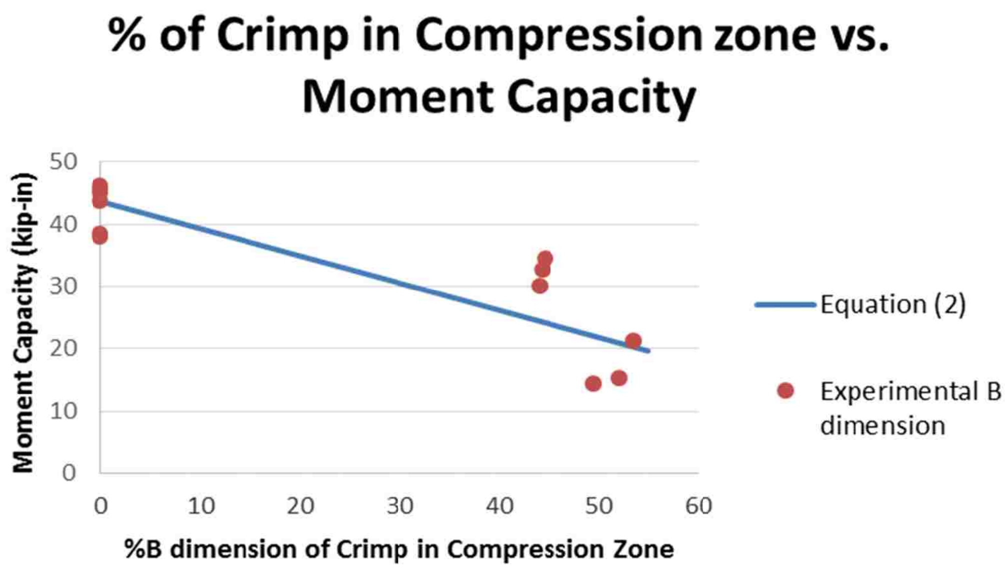


Figure 5.2 % of Crimp in Compression vs. Ultimate Moment Capacity

$$M_m = M_s - M_s\left(\frac{\%B}{100}\right) \tag{2}$$

$$\%B = 100\%\left(\frac{d_c+B-d}{d_c}\right) \tag{3}$$

$$\%B = 100\%\left(1 - \frac{d_c - B}{d_c}\right) \tag{4}$$

$$d_c = \frac{d\sigma_c}{(\sigma_y + \sigma_c)} \tag{5}$$

The amount of crimp geometry that is present in the compression zone of the beam depends on whether or not the crimp is in the tension or compression flange. If the crimp is in the tension flange, then a large crimp degree is necessary to reach the compression zone, but if the crimp is in compression flange, then the crimp immediately disrupts the stress in the compression zone and decreases the ultimate moment capacity of the beam. Figure 5.3 shows the correlation between the B dimension and moment capacity of the crimped beam if the crimp is located in the tension flange of the beam. Equation 6 was developed to predict the moment capacity of a beam if the crimp was placed in the tension flange depending on the degree of the crimp.

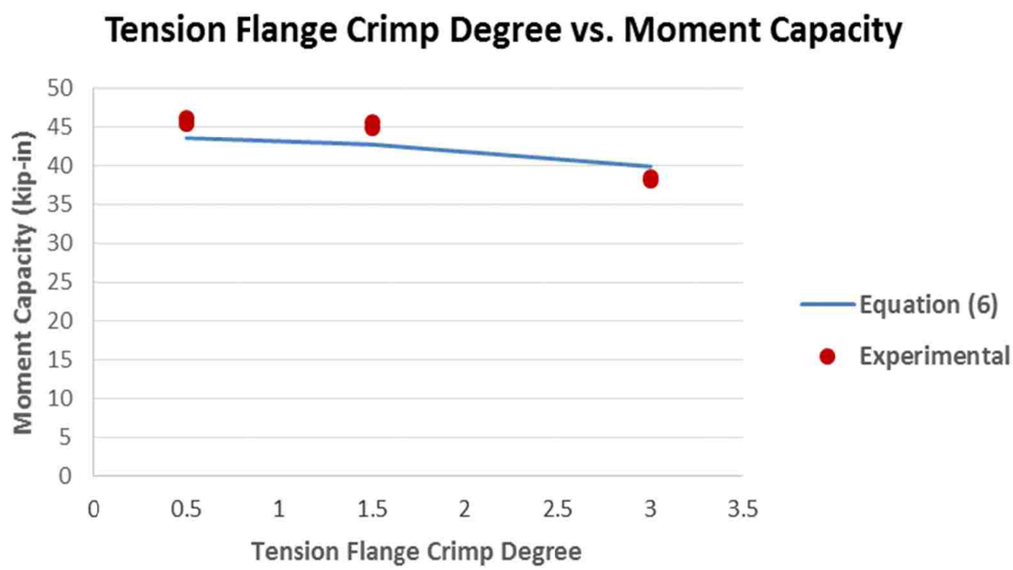


Figure 5.3 Tension Flange Crimp Moment Capacity

$$M_t = M_s \cos 8\theta_t \qquad (6)$$

Figure 5.4 shows the correlation between the crimp degree and the moment capacity if the crimp is in the compression flange, and Equation 7 was developed to predict the moment capacity of intermediate crimp degrees in the compression flange.

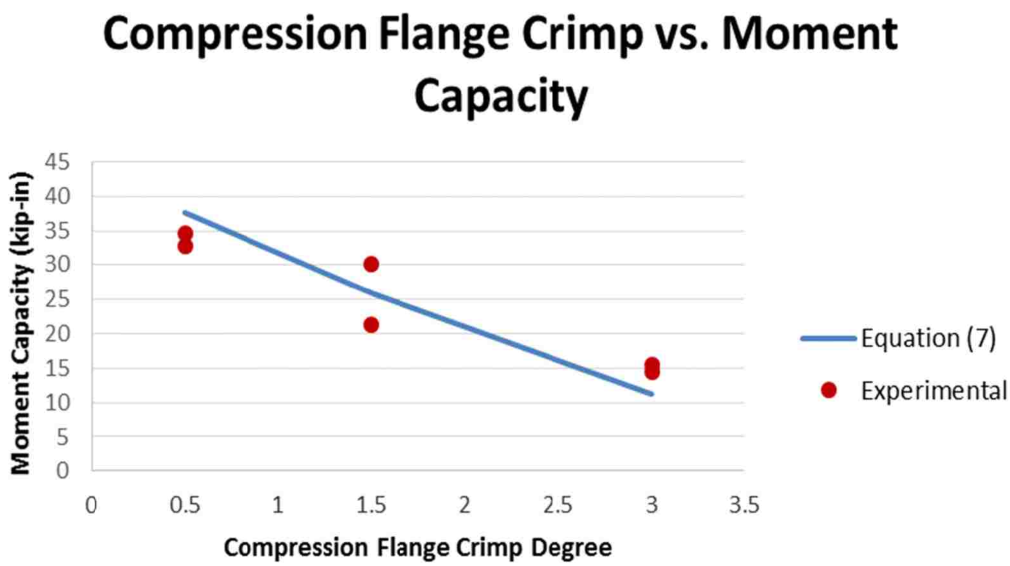


Figure 5.4 Compression Flange Crimp Moment Capacity

$$M_c = M_s[1 - \sin 16\theta_c] \quad (7)$$

5.5 CURVED BEAM ANALYSIS

To compare the performance of a crimped beam to that of a curved beam would be helpful in determining which method would be more efficient as a structural member. To make this comparison, the stress of a curved beam with a cross-section similar to that of the crimped beams was evaluated. There were two methods that were employed to attempt the comparison: The Winkler-Bach equation, and the curved beam method with circumferential stress [Boresi 2003]. A sample diagram showing the layout and the load that was applied in the analysis can be seen in Figure 5.5. Figure 5.5 shows a 3.0 degree crimped beam under compression loading, and the dimensional equivalent curved beam. The cross-sectional dimensions of the curved beam were assumed to be identical to those found in Figure 3.3 which match the nominal dimensional dimensions of an 600S162-54 Cold-formed steel beam.

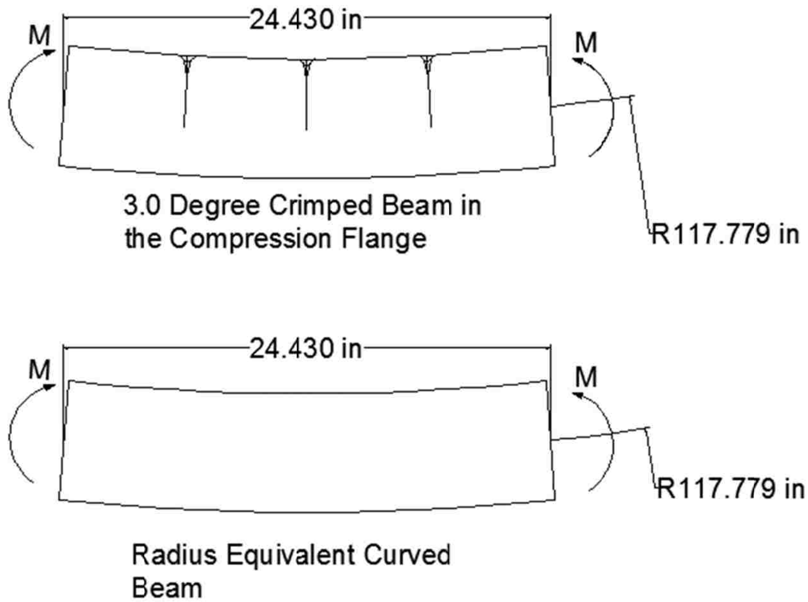


Figure 5.5 Sample Layout Comparison of Crimped and Curved Beams

5.5.1. Winkler-Bach Equation. An attempt was made to calculate what the capacity would be of 600S162-54 CFS beams would be if the beams were curved around a point rather than crimped. To achieve this analysis, two methods were employed. The first was the Winkler-Bach formula as shown in the 2004 Automotive Steel Design Manual. This formula uses a parameter known as Z, which differs depending on the geometry of the beam. For a C-section beam:

$$Z = -1 + (\frac{r}{A})[(b_o' + t_w)\ln(r_o) - (b_i' + t_w)\ln(r_i) - b_o'\ln(r_o - t_w) + b_i'\ln(r_i + t_w)] \quad (8)$$

This parameter Z is then used in the Winkler-Bach equation to calculate the stress in the curved beam. Table 5.7 shows the result of these calculations, where f_b is the stress in the curved beam.

Table 5.7 Winkler-Bach Curved Beam Results

Beam	b_f (in)	a_{fi} (in)	a_{fo}	t_f	Z	M(kip-in)	$f_{b\ (ksi)}$
0.5C	1.5684	685	691	0.0566	-0.0699	33.685	0.135
1.5C	1.5684	227	233	0.0566	-0.0698	25.802	0.345
3.0C	1.5684	111	117	0.0566	-0.0695	14.889	0.470
0.5T	1.5684	685	691	0.0566	-0.0699	45.886	0.162
1.5T	1.5684	227	233	0.0566	-0.0698	45.352	0.416
3.0T	1.5684	111	117	0.0566	-0.0695	38.267	0.549

The M in the table is the moment that was applied to the curved beam's crimped counterpart. When compared to the stresses that were seen by the crimped beams in Table 5.4, the amount of stress that was expected in a similar curved beam was low. Upon further review of the Winkler-Bach equations, it was determined this equation was used for curved beams that possessed larger cross-sectional areas than the 600S162-54 beam, as well as a much smaller radius of curvature. The Winkler-Bach equation was also applied to cross-sections that were less slender and thus less prone to local buckling. For this reason, the Winkler-Bach equation was found to be a poor predictor of the moment capacity of a curved 600S162-54 CFS beam.

5.5.2. Curved Beam Circumferential Stresses. This method evaluates the circumferential stress of a curved beam by using a parameter, A_m, which is determined by the integral of the change in area over the radius of the beam.

$$A_m = \int dA/r \tag{9}$$

The A_m formula can be used on many different cross-sections to evaluate their stress at applied moments, and is therefore applicable to any cross-section that could be simplified to geometric shapes. With this general applicability, the circumferential stress equation was employed to evaluate the stress of curved beams that were similar in cross section and applied moments to their crimped counterparts. The results of the circumferential stress equation can be seen in Table 5.8.

Table 5.8. Circumferential Stress in Curved Beams

Beam	A (in^2)	a_{fi}	a_{fo}	A_m (in)	R (in)	M (kip-in)	$\sigma_{\theta\theta\ inner}$ (ksi)	$\sigma_{\theta\theta\ outer}$ (ksi)
0.5C	0.556	685	691	0.0030	688	33.685	-0.088	-0.088
1.5C	0.556	227	233	0.0089	230	25.802	-0.201	-0.203
3.0C	0.556	111	117	0.0180	114	14.889	-0.233	-0.237
0.5T	0.556	685	691	0.0030	688	45.886	-0.120	-0.120
1.5T	0.556	227	233	0.0089	230	45.352	-0.353	-0.356
3.0T	0.556	111	117	0.0180	114	38.267	-0.598	-0.609

Like the Winkler-Bach equation, this method also predicts low compression stress values. When compared to experimental results they were found to have a poor correlation to the predicted values. Because of these values and similar past examples, the conclusion was made that this method was also intended for beams with cross-sections with larger areas, and a much smaller radius of curvature. This method also doesn't take into account the effect of local buckling that occurs in slender members, so for this reason, the circumferential stress equation was found to also be a poor predictor of the stress in a curved 600S162-54 CFS beam.

6. CONCLUSION

A small amount of crimping such as the 0.5 or 1.5 degree crimp in the tension flange did not significantly affect the ultimate moment capacity of the beam. The 3.0 degree crimp in the tension fiber reduced the moment capacity somewhat, but the loss of capacity was smaller than if the crimp was placed in the compression fiber of the beam. If the crimp was placed in the compression flange of the beam, then losses accumulated quickly, with the 0.5 degree crimp failing at 75% of the straight beam's moment capacity, the 1.5 degree crimp failing at 50% of the straight beam's moment capacity, and the 3.0 degree crimp failing at 33% of the straight beam's moment capacity. The presence of the reinforcing band was found to have minimal effect on the ultimate moment capacity except for the 1.5 degree crimp in compression. This increase was neglected in the strength reduction factors in Table 5.3.

The crimping process resulted in larger deflection at service level loads for all beams, when compared to straight beams. The larger deflections were a result of the decrease in the beam's effective moment of inertia from the crimps in the beam. Larger crimping degrees resulted in greater losses in the effective moment of inertia. The presence of the reinforcing band over the crimp did increase the beam's effective moment of inertia, and therefore decreased the amount of deflection at service level loads. For this reason, it is advised to use the reinforcing band if excessive deflection is a concern. Crimped beams can be used in design, but their use should include the appropriate strength reduction factor from Table 5.3 to account for their decreased moment capacity compared to the straight beam, and should take into account the appropriate deflection requirement for the application.

7. FUTURE RESEARCH

This research served as the initial steps for a better understanding of the behavior of crimped cold-formed steel beams, with a focus specifically on the crimp size and the presence of the reinforcing band. Additional factors should be considered if and when additional tests are to be performed. The additional considerations include variable rivet spacing, axial forces, and other crimp sizes.

Future research should be a broader study of the geometry of the cross-section. Different material thicknesses should be used and other beam depths should be crimped and evaluated to study how the crimp changes the ultimate moment capacity, and the effective moment of inertia of the beam.

The rivets restraining the reinforcing strap were randomly placed, and this randomness created smaller and larger distances of the strap that were not restrained against the beam. The larger unrestrained sections could allow for premature buckling of the member. It is recommended a study should be done based on the distance between rivets that restrain the reinforcing strap to the member is limited to a certain distance, and then the extent to which the varying distance between rivets affects the moment capacity can be ascertained.

An unexplained phenomenon is the fact that the only beam that saw an increase in moment capacity is the 1.5 degree crimp in the compression fiber. A reasoning for this was presented showing that the increased moment capacity occurred because of the movement of the neutral axis and subsequent change in compressive stress. While this showed the reasoning for the increased moment capacity, it did not explain why this moment capacity increase only occurred in the 1.5 degree crimp in the compression flange. Further research should be conducted to ascertain the reason for the moment capacity increase.

The members only experienced bending forces during the testing for this research. In the field, during construction and during service, these members would also experience axial forces and shear stresses. Experimentation in the presence of axial forces along with bending stress and shear stress should be conducted to assess how these forces would interact under various combinations of axial and bending loads. Both tensile and compressive loads should be tested with the crimp in the compression fiber and tension fiber. The inclusion of axial loads on these members could change the strength and serviceability capacities of these members greatly.

Further testing should be done on intermediate crimp sizes between 0.5, 1.5 and 3.0. The equations that were presented in Section 5.4 should be compared with the tested values that are gained from further testing, and the equations should be refined to better represent the moment capacity as it changes with the size of the crimps.

The manufacturer indicates that the degree bends can be as high as 7.0 degrees, so beams with deeper indentations should be assessed for their strength and serviceability under structural loads. The manufacturer also constructs with crimped beams that have large holes punched in the web of the beam. This is done to allow bracing, wiring and other construction necessities to pass through the member, but it also reduces the strength and serviceability capacities of the beams further. The effect of these holes should be assessed as an additional variable to understand how they interact with the crimp geometry to affect the capacity of the beams.

APPENDIX A

MOMENT-DEFLECTION GRAPHS – ALL BEAM TYPES

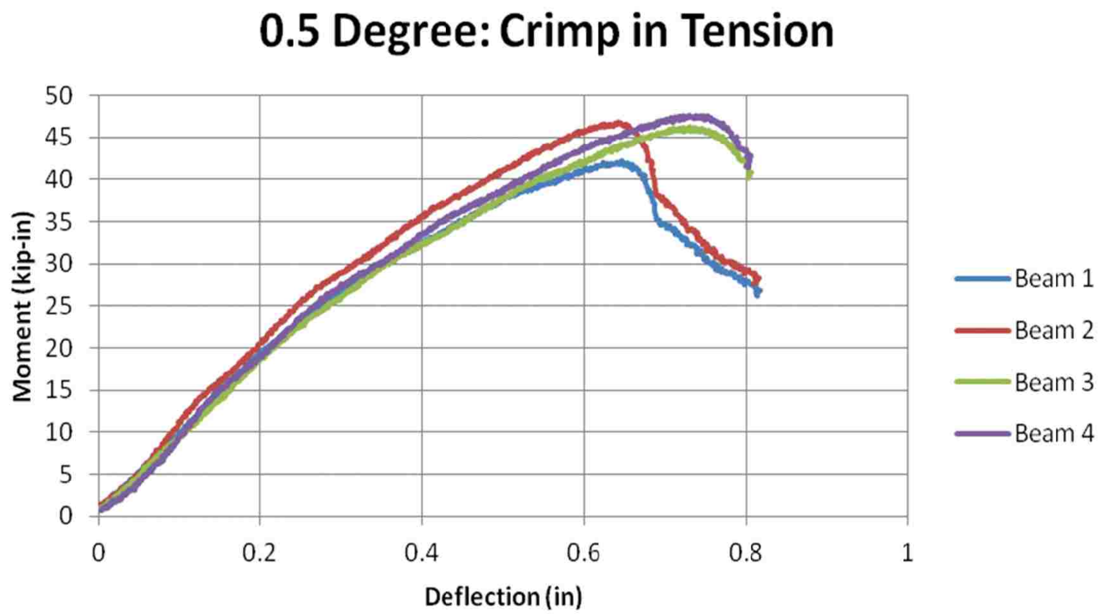


Figure A.1 0.5 Degree Crimp in the Tension Fiber

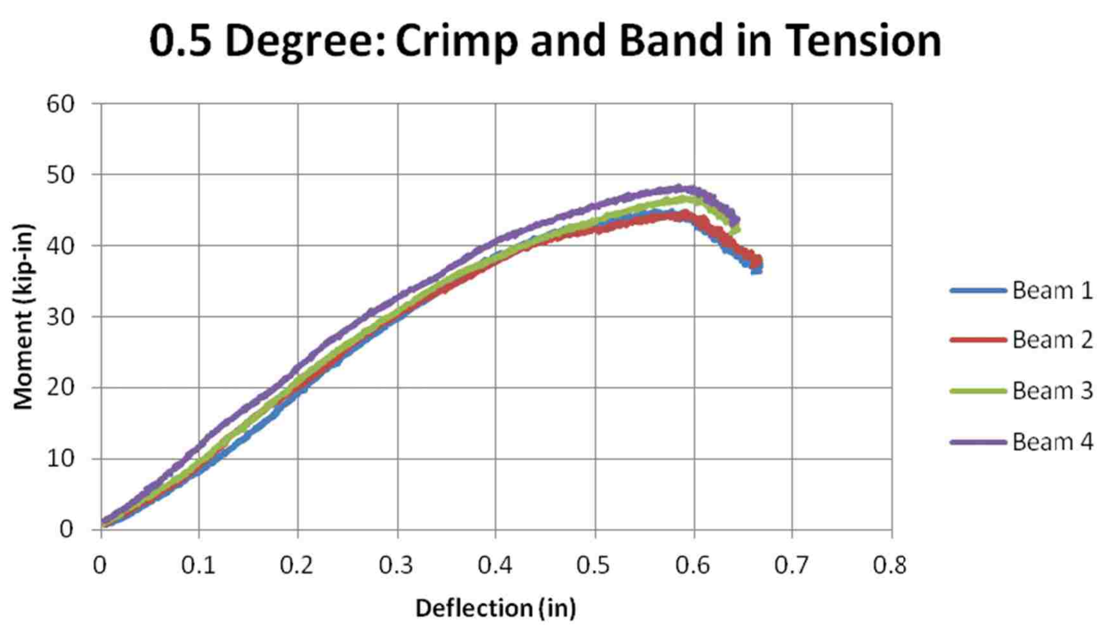


Figure A.2 0.5 Degree Crimp and Band in Tension Fiber

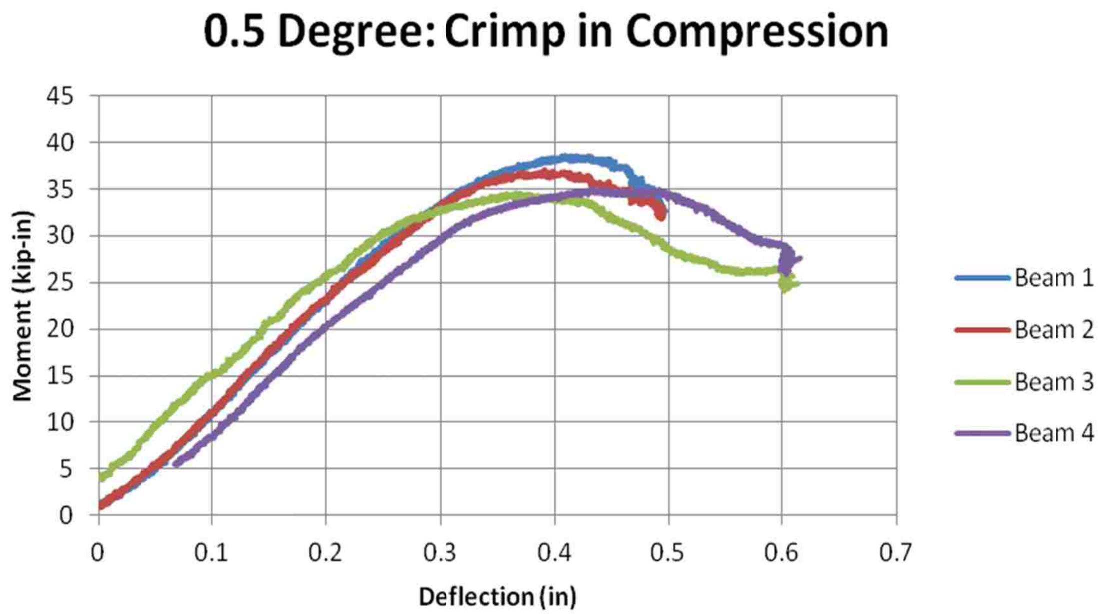


Figure A.3 0.5 Degree Crimp in Compression Fiber

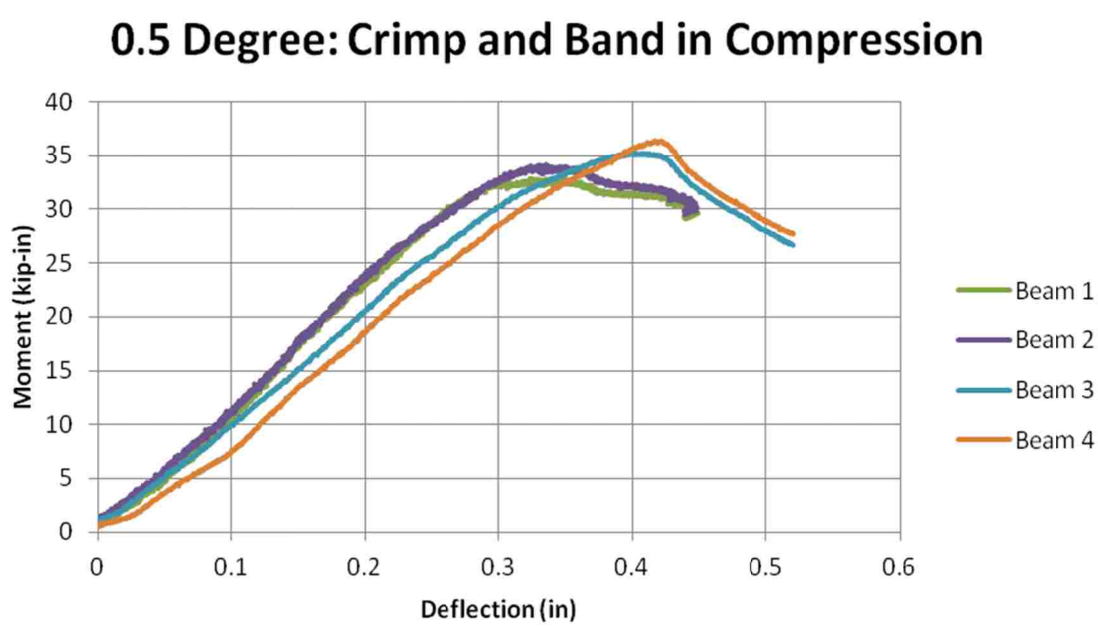


Figure A.4 0.5 Degree Crimp and Band in Compression Fiber

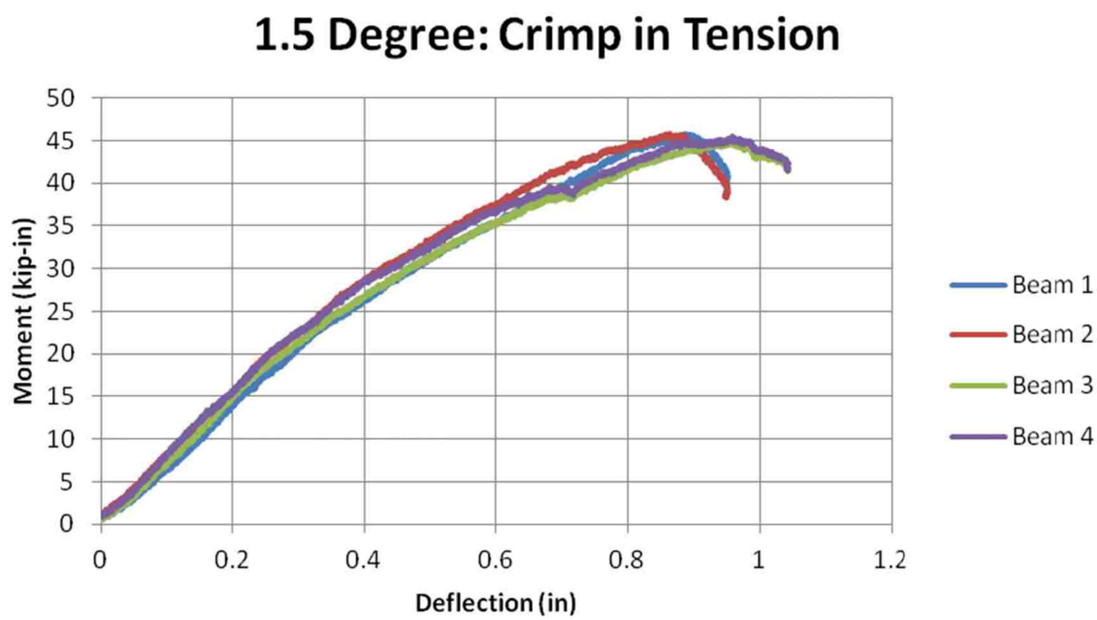


Figure A.5 1.5 Degree Crimp in Tension Fiber

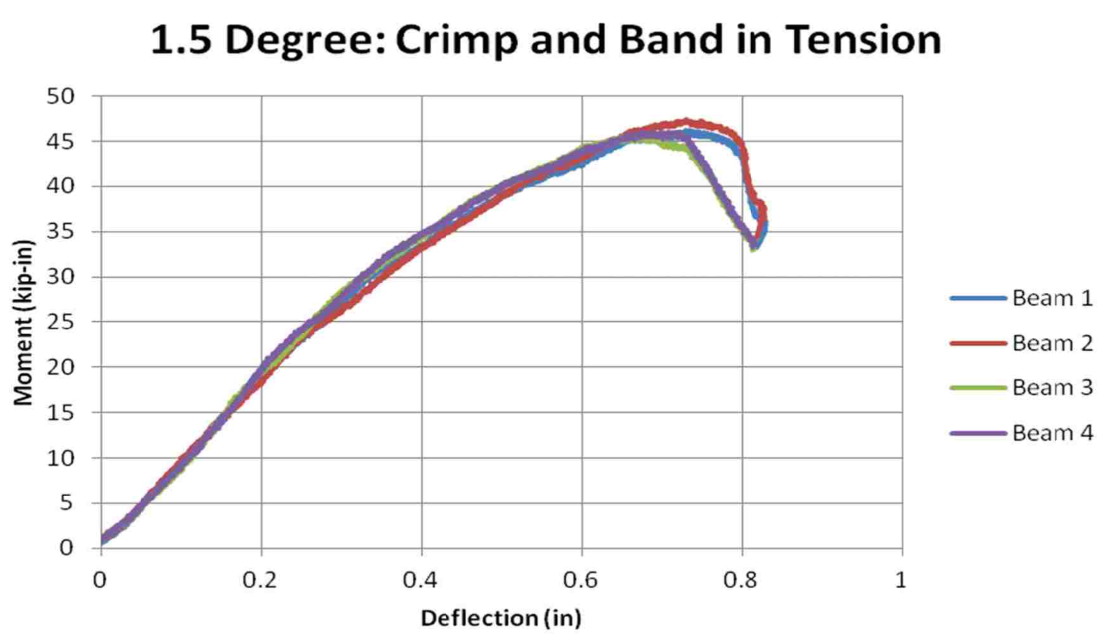


Figure A.6 1.5 Degree Crimp and Band in Tension Fiber

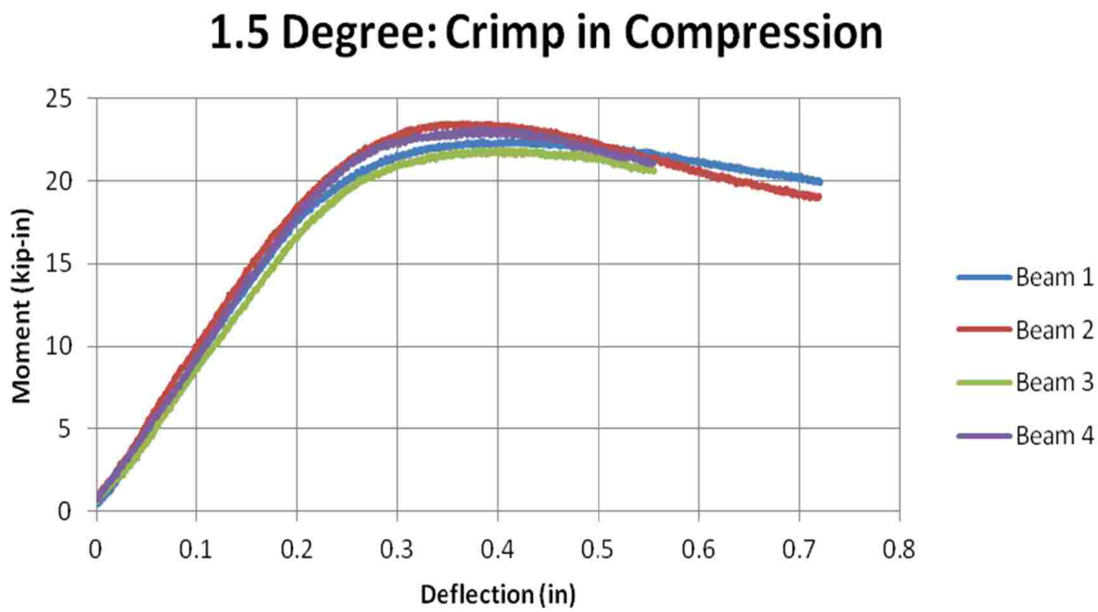


Figure A.7 1.5 Degree Crimp in Compression Fiber

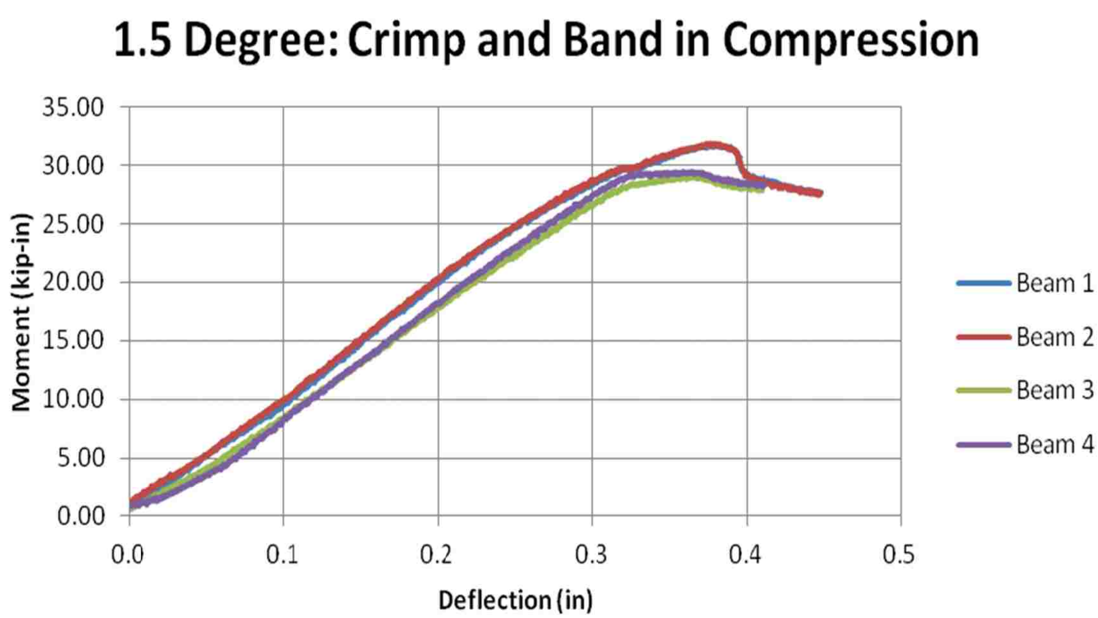


Figure A.8 1.5 Degree Crimp and Band in Compression Fiber

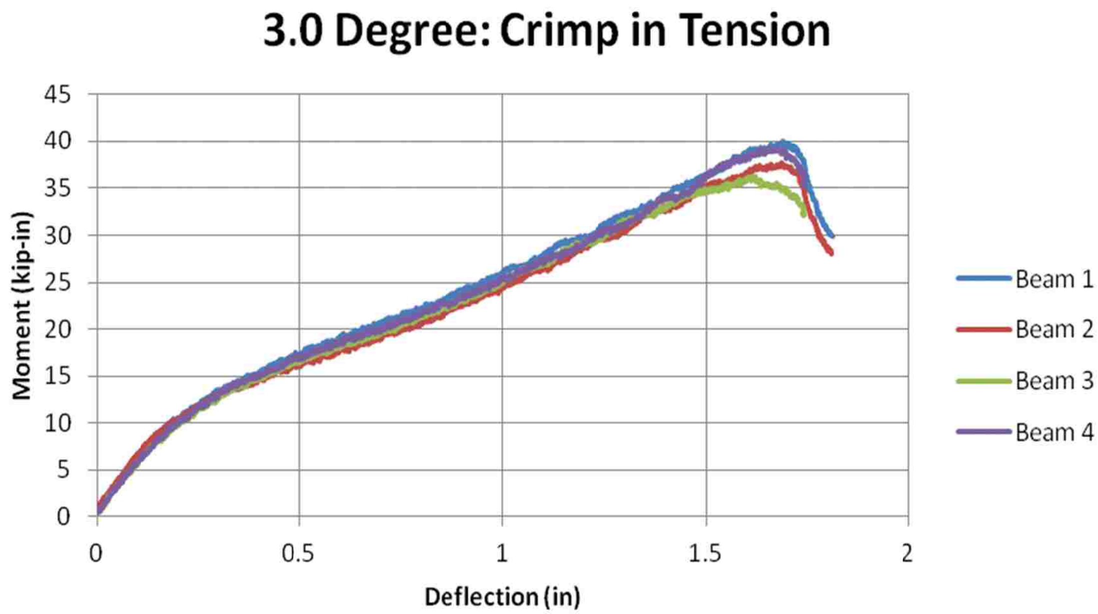


Figure A.9 3.0 Degree Crimp in Tension Fiber

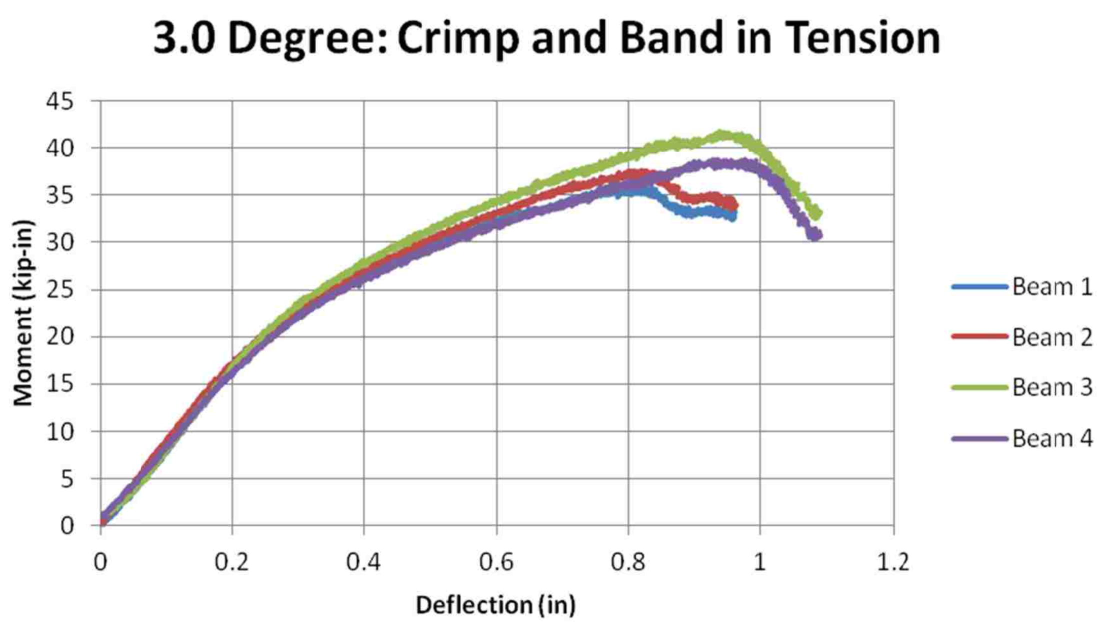


Figure A.10 3.0 Degree Crimp and Band in Tension Fiber

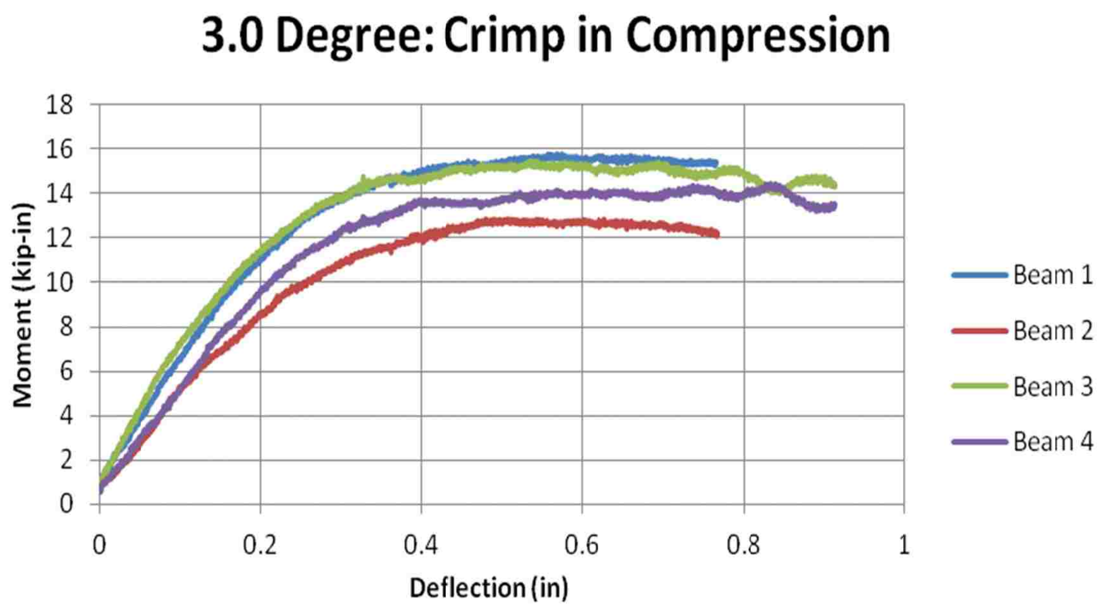


Figure A.11 3.0 Degree Crimp in Compression Fiber

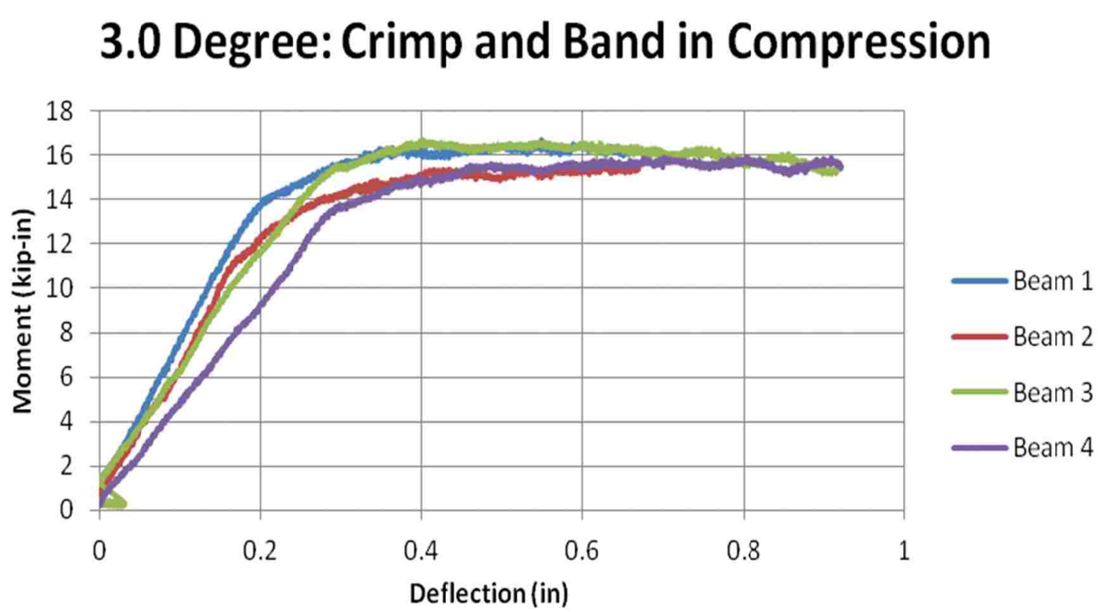


Figure A.12 3.0 Degree Crimp and Band in Compression Fiber

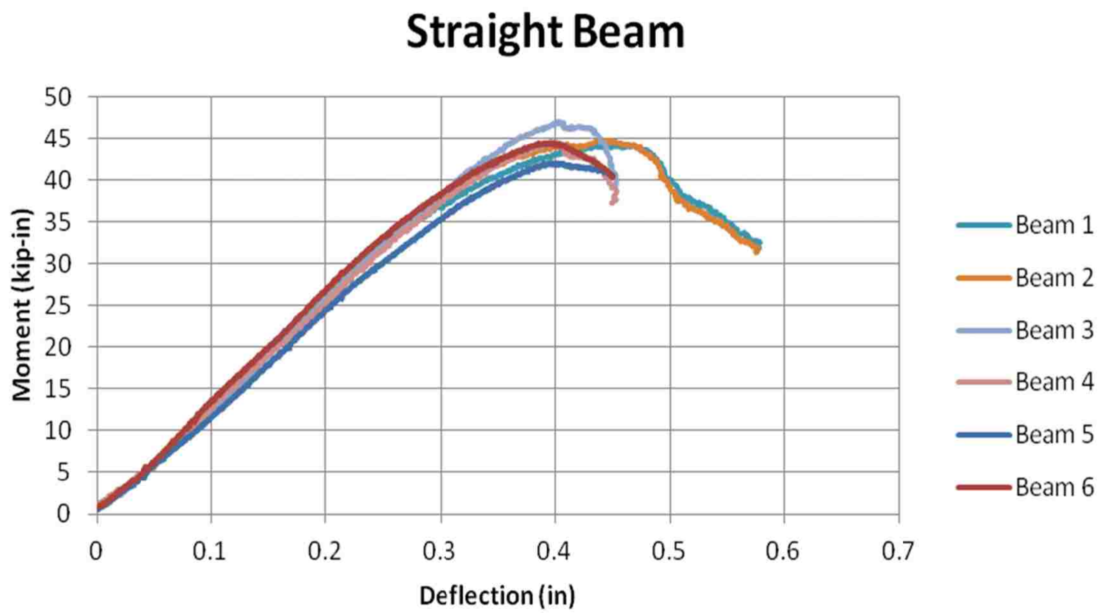


Figure A.13 Straight Beam

APPENDIX B

Derivation of Equations

Derivation of Equation (1)

Assumptions

- The two point loads in the experiment were perfectly equal
- The two point loads in the experiment were perfectly symmetrically spaced

Begin with Simple beam – Two Equal Concentrated Loads Symmetrically Placed Deflection Equation.

$$\delta = \left(\frac{Pa}{24EI}\right)(3L^2 - 4a^2)$$

Multiply both sides by I

$$\delta I = \left(\frac{Pa}{24E}\right)(3L^2 - 4a^2)$$

Divide both sides by Δ

$$I = \left(\frac{Pa}{24\delta E}\right)(3L^2 - 4a^2)$$

Assume that Moment of Inertia under service loading is constant, and does not cause yielding in the member. Moment at Service loading is 60% of ultimate Moment Capacity.

$$M_{ult} = Pa$$

$$M_{service} = 0.6M_{ult}$$

Combine these equations to get calculation for Moment of Inertia at Service loading.

$$I = 0.6M_{ult}\frac{[3L^2 - 4y^2]}{24E\delta}$$

Derivation of Equation (2)

Assumptions

- The amount by which the moment capacity decreased was directly proportional to the amount of B dimension in the compression zone.

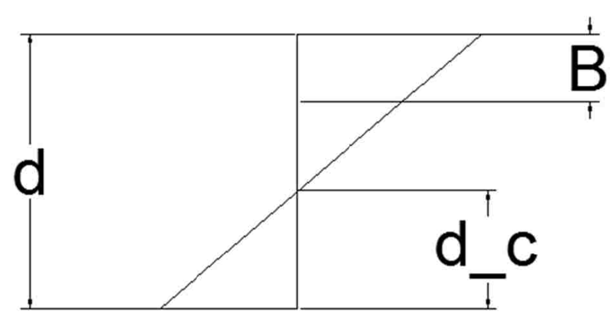
Take account for percentage of d_c that is occupied by B dimension

$$M_m = M_s\left(1 - \frac{\%B}{100\%}\right)$$

$$M_m = M_s - M_s\left(\frac{\%B}{100\%}\right)$$

Derivation of Equation (3)

Begin with the stress gradient diagram for a small B dimension of a crimp that is in the tension flange of the beam



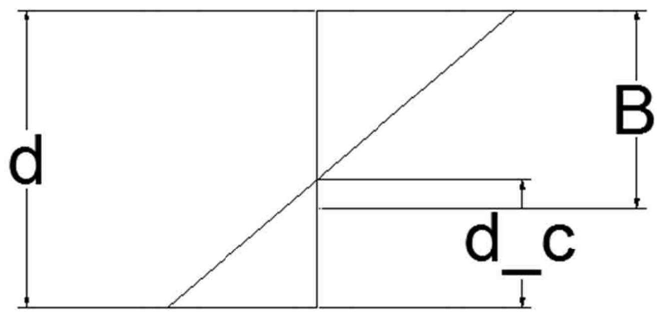
If the B dimension of the crimp is smaller than the depth of the tension zone

$$d - d_c > B$$

Then d_c is not affected by the B dimension

$$\%B = 0$$

If the B dimension of the crimp is larger than the depth of the tension zone



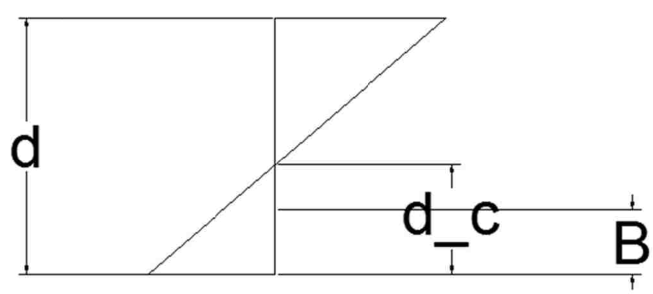
$$d - d_c < B$$

Then the compression zone of the beam is affected by the crimp's geometry, and the percentage of d_c that is affected by the B dimension (%B) is as follows

$$\%B = \left(\frac{d_c + B - d}{d_c}\right) x\ 100\%$$

Derivation of Equation (4)

Begin with the stress gradient diagram for the B dimension of a crimp that is in the compression flange of the beam.



Because the crimp is in the compression flange of the beam, the B dimension is automatically in the compression zone of the beam's stress gradient diagram.

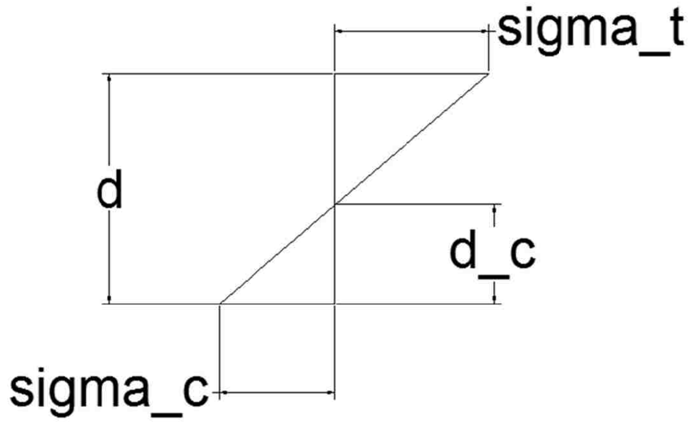
The percentage of the compression zone that is affected by the B dimension is

$$\%B = \left(1 - \frac{d_c - B}{d_c}\right) x\ 100\%$$

Derivation of Equation (5)

Assumptions

- Full yielding has not occurred in the beam's cross-section



Using the theorem of similar triangles, a relationship between the size of the beam and the tension and compression stresses can be created.

$$\frac{d_c}{\sigma_c} = \frac{d - d_c}{\sigma_y}$$

Multiply each side by σ_y and σ_c

$$d_c\sigma_y = (d - d_c)\sigma_c$$

Solve so that d_c is on one side of the equation

$$d_c\sigma_y + d_c\sigma_c = d\sigma_c$$

Factor out d_c

$$d_c(\sigma_y + \sigma_c) = d\sigma_c$$

Solve for d_c

$$d_c = \frac{d\sigma_c}{(\sigma_y + \sigma_c)}$$

Derivation of Equation (6) and (7)

These equations were created to be a fit to the experimental data points that would predict the capacity of intermediate crimp degrees. These equations should be verified with future testing and research.

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VITA

The author, Jeffrey Smith, was born on December 19, 1990 in Houston, Texas. He attended Cypress Woods High School and graduated in 2009. He began attending Missouri University of Science & Technology in 2009 to acquire his bachelor’s degrees in Civil Engineering and Architectural Engineering, and graduated in December of 2013 with a minor in History. He began research for his master’s degree in Civil Engineering in January of 2014. In May, 2015, he received his Master’s degree in Civil Engineering from Missouri University of Science and Technology.