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Behavior of cold-formed steel roof trusses

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First Summary Report

BEHAVIOR OF COLD-FORMED STEEL ROOF TRUSSES

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PREFACE

To aid in adapting cold-formed steel to the residential market, a research project was initiated in 1993 at the University of Missouri-Rolla. Design issues relating to the use of cold-formed steel members and connections in residential roof truss systems were the focus of the project. The purpose of this research was to study the behavior of cold-formed steel roof truss systems and to establish appropriate design recommendations. Overall, the research findings were intended to aid the promotion of cold-formed steel as a safe, serviceable, and cost effective alternative in residential construction.

The project consisted of a review of available literature, followed by a comparative analysis of experimental truss behavior to a computer generated model. An in-depth review of research reports and publications yielded minimal information, however design issues were discussed with interested design engineers and truss manufacturers regarding the state-of-the-art. The experimental investigation involved an evaluation of the overall truss behavior using full-scale truss assemblies. Based on this information, a computer generated model was created to simulate the truss assembly. An evaluation of deflection and stress data was used to correlate the computer model to the full-scale truss. The computer model and AISI Specification formed the basis used to establish the predicted failure load, which was then compared to the tested failure of the full-scale truss assembly.

The conclusions obtained from the experimental investigation were used to create design recommendations. The recommendations prescribe minimum strength and serviceability requirements for trusses fabricated using cold-formed C-sections and self-drilling screws. The design recommendations are intended to compliment the AISI Specification.

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

This investigation was sponsored by the National Science Foundation and the American Iron and Steel Institute. The technical guidance provided by the Technological Research Subcommittee of the AISI Residential Advisory Committee and Steve Walker (chairman) is gratefully acknowledged. Thanks are also extended to A. Ziolkowski and R. B. Haws, AISI staff, and J. B. Scalzi of the National Science Foundation.

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I. INTRODUCTION

A. GENERAL

Conservation is becoming more prevalent in our society as it is a necessity to protect our environment and ensure our future. Recently, this growing environmental awareness has created concerns regarding the use of wood as an appropriate construction material. In addition, economic and safety concerns are pressuring the competitiveness of the wood industry. Timber prices have risen sharply as the result of a supply and demand crisis. Also, the recent devastation to wood structures by storms have led to the adoption of building codes which require engineered residential construction to minimize safety concerns.

To improve the feasibility of residential construction, alternative building materials are being explored. One such alternate material is cold-formed steel. Due to its recyclability it is an environmentally attractive solution. In addition to satisfying environmental concerns, cold-formed steel members have many other positive physical characteristics. They are mass produced with consistent dimensional properties, as well as being non-combustible, and insect and rodent resistant. Cold-formed steel has long been the preferred construction material for commercial and light-industrial construction because it is cost competitive, possesses a high strength-to-weight ratio and is simple and fast to erect.

To aid in the adaptation of cold-formed steel to the residential construction market a research project was initiated at the University of Missouri-Rolla in 1993. The research is jointly funded by the National Science Foundation and the American Iron and Steel Institute. Design issues relating to the use of cold-formed steel members in residential roof truss systems are the focus of the project.

Residential roof truss systems using cold-formed steel typically entail C-shaped channel sections connected using self-drilling screws. The top chords are continuous from ridge to heel and the bottom chord is continuous from heel to heel. The diagonal members are connected between the top and bottom chords and a conventional wood sheathing creates the roof surface.

B. PURPOSE OF INVESTIGATION

The purpose of this research project was to discover necessary knowledge regarding the behavior of cold-formed steel roof truss systems and to establish design recommendations. Overall, the research findings are intended to aid in the promotion of cold-formed steel as a safe, serviceable, and cost effective alternative in residential construction.

C. SCOPE OF INVESTIGATION

This project consisted of a review of available research followed by a comparative analysis of experimental truss behavior to a computer generated model. An in-depth review of research reports and publications yielded minimal information. Design issues were discussed with various manufacturers regarding recommended geometries and testing histories. Section II contains a summary of the literature review.

The experimental investigation involved an evaluation of the overall truss behavior using full-scale truss assemblies. Based on this information, a computer generated model was created to simulate the truss assembly. An evaluation of deflection and stress data was used to correlate the computer model to the full-scale truss. The computer model and AISI Specification[1] formed the basis used to establish the predicted failure load, which was then compared to the tested failure of the full-scale truss assembly. Sections III-V contain a summary of the experimental investigation.

The conclusions obtained from the experimental investigation were used to create the design recommendations contained in Section VI. The recommendations prescribe minimum strength and serviceability requirements and are intended to compliment the AISI Specification[1].

II. LITERATURE REVIEW

A. GENERAL

A roof truss is a system composed of individual members and connections each contributing to the integrity of the system. The individual members, commonly C-shaped, when subjected to a superimposed load may resist the load in tension, compression, bending, or combinations thereof. However, truss connectors, usually screws, are typically used to provide load transfer from member to member as shear transfer mechanisms. The following review will present some of the pertinent research findings regarding member behavior, connection behavior, and system behavior.

B. COMPRESSION MEMBERS

Depending on the cross-section geometry, steel sheet thickness, and member length, a cold-formed steel compression member may be susceptible to one or more of the following limit states:

1. Yielding of the cross-section
2. Overall column buckling
3. Local buckling of elements in the section

Classical buckling theory for flexural buckling, torsional buckling, and torsional-flexural buckling forms the basis for establishing the overall stability of a compression member. Elastic and inelastic column buckling are both discussed by Yu[2]. In all three limit states the degree of end restraint may be reflected in the computations by use of the effective length factor, K.

Elements within the cross section of a cold-formed steel member generally have large width/thickness ratios, therefore, local buckling is a major design consideration. The AISI Specification[1] recognizes the potentially detrimental influence of local buckling by using the "effective width" concept as first proposed by Von Karman[3]. The effective width concept has been extensively researched by Winter[4] and Pekoz[5].

Design Requirements for a cold-formed steel compression member fall under the authority of the AISI Specification[1]. These design provisions are founded on an extensive research effort conducted at several universities. A summary of the research programs and findings is presented by Pekoz[5] and Yu[2].

C. TENSION MEMBERS

The limiting strength consideration for a tension member, excluding the region of the connection, is based on yielding of the net cross section. All research pertaining to tension member design has focused on net section yielding at the connection and the capacity of the connection[2]. The relatively thin nature of cold-formed steel tension members generally creates a connection failure mode. Design provisions for yielding of the net section and fracture in the net section are contained in the AISI Specification[1].

D. FLEXURAL MEMBERS

A flexural member is a composite of a compression member and a tension member because of the variable stress distribution through the depth of the member. Therefore, all of the above limit states that pertain to either a compression member or a tension member may be present in a flexural member.

When bracing is adequate to prevent lateral buckling, the nominal moment capacity is the yield moment determined as follows:

$$M_y = S_e \times F_y \tag{1}$$

where

F_y = yield stress

S_e = effective section modulus

The effective section modulus, S_e, is used to reflect the decrease in moment capacity due to the presence of local buckling. The yield moment may occur in either the tension, compression, or both regions of the cross section.

The lateral buckling strength of cold-formed steel flexural members was studied by Pekoz and Winter[6] and Pekoz[5] which forms the background for the following AISI Specification[1] equation (C3.1.2-5) which considers minor axis lateral buckling:

$$(M_y)_e = C_b\, r_o\, A \sqrt{(\sigma_{ey}\sigma_t)} \qquad (2)$$

where

C_b = a modifier for unbraced segment end moments

r_o = polar radius of gyration

A = full cross-sectional area

σ_{ey}= Euler buckling stress about the minor axis

σ_t = torsional buckling stress

For both σ_{ey} and σ_t the end restraint of the member is reflected by the effective length factor, K.

In addition to the above limit states, a flexural member is also susceptible to web failure resulting from shear, local concentrated loads causing web crippling and combinations of flexure and shear, or flexure and web crippling. These limit states have been extensively researched at the University of Missouri-Rolla by LaBoube and Yu[7], Hetrakul and Yu[8], and Bhakta, LaBoube and Yu[9].

E. BENDING AND COMPRESSION

A beam-column may experience one of several limit states when subjected to combined bending and compression:

1. Yielding
2. Flexural buckling
3. Torsional-flexural buckling
4. Local buckling

The behavior of cold-formed steel beam-columns has been extensively studied by Pekoz and Winter[6], Rhodes and Harvey[10], Loughlan and Rhodes[11], and Loh and Pekoz[12].

The tendency of a member to adopt one of the previously mentioned limit states depends on the geometry of the cross section, the location of an applied eccentric load, the member length, and the frequency of bracing. To address the complexity of a beam-column, the Specification[1] presents design provisions based on the cross-section geometry and its possible buckling modes.

The current specification provisions addressing interaction of axial load and bending (AISI Specification, Section C5) account for end restraint by use of the effective length factor, K, and the end moment coefficient, C_m.

F. CONNECTIONS

Although screws are widely used to connect cold-formed steel members, the only design provisions which exist in the United States are published in a Center for Cold Formed Steel Structures Technical Bulletin[13]. These design provisions were recently adopted by the Specification[1] and are based on a study by Pekoz[14]. This study used over 3500 tests conducted in the United States, Canada, Sweden, and the Netherlands. It was difficult to develop equations that provided a high degree of accuracy because of the wide range of screw types (self-drilling, self-tapping, diameter, thread pitch, etc.) and connection dimensions used in the industry.

The strength of wood- or plywood-to-metal screw connections are not addressed by the design specification[1]. Proprietary strength data is used for design calculations in this instance.

G. TRUSS ASSEMBLY

Four proprietary roof trusses were examined and briefly summarized by Ife[15]. The first truss used cold-formed C-sections as the top and bottom chords as well as the diagonals. The connections were created using gusset plates and spot welding. The failure load was approximately one-half the desired load capacity. The premature failure was caused by failure of the spot welds in the connections. The second truss was fabricated using C-sections for all truss components but making connections using self-drilling screws rather than welds. Failure was, again, premature and caused by a connection failure. Secondary stresses created by the eccentric nature of the connections were attributed as the cause of the early collapse. The third truss assembly maintained self-drilling screw connectors, but, consisted of hat-sections used for the top and bottom chords while the diagonals were fabricated from tubing. This truss assembly eliminated the eccentric nature of the connections and exceeded the desired load capacity. The fourth truss was a geometric modification of the previous assembly resulting in satisfactory performance.

Design criteria with supporting test data for the use of cold-formed steel shapes in latticed transmission towers was presented by Zavelani and Faggiano[16]. Two of the test programs consisted of full-scale tower tests. Favorable test results were achieved, however, both tests used bolted connections. Connection details were not presented and the presence of connection eccentricities were not mentioned.

H. INDUSTRY REVIEW

Several industry representatives were contacted regarding the history of design issues and the associated testing programs. The consensus was that stability has been a concern in trusses using cold-formed steel sections. The C-section is singly-symmetrical and subject to lateral-torsional buckling behavior. Bracing that is perpendicular to the truss has been added to many roofs to transfer lateral loads. This is particularly evident in the high coastal wind areas such as Florida. In larger structures, some designers are using a perpendicular horizontal truss system for lateral load transfer.

Connections have been a source of concern within the industry. The nature of the connections result in eccentricities that have to be minimized by the selection of connection configurations. Various heel connections have been tested to determine capacity. The ridge connection has received the most attention because of its local buckling potential. A variety of ridge configurations have been investigated in an effort to reduce the local buckling potential. When the C-sections of the top chord both face the same direction, the ridge connection is a point of weakness. This is a result of the required coping of the flanges which creates a condition for local plate buckling of the web. Overall, while the suppliers are addressing these issues using a variety of proprietary configurations, no data supporting a design procedure is complete.

III. EXPERIMENTAL STUDY

A. GENERAL

An effort was made to establish the behavior of a typical residential roof truss constructed using cold-formed steel members. Current residential construction details prompted the use of 20 ft. span Fink trusses at a 4:12 pitch for the basic test assembly. A series of full-scale tests were conducted using various connection modifications of this basic test assembly. Test assemblies were gravity loaded at 5 psf intervals using common masonry bricks. Improvements were made in the connection details throughout the testing until an assembly was developed that created consistent member failures rather than truss connection failures. The connection modifications required during this test program provided a basis for general recommendations addressing connection considerations. The final truss test assembly served as the standard to which the computer model correlations and failure predictions were compared. All tests were performed in the Engineering Research Laboratory of the University of Missouri-Rolla from May to December of 1994.

B. MECHANICAL PROPERTIES

Tensile coupon tests were conducted to obtain the mechanical properties of the C-sections used in fabricating the trusses. The tensile tests were conducted per standard procedures detailed in ASTM A370 and the results are summarized in Table I.

Table I. MATERIAL PROPERTIES

	AVERAGE	VALUES		
SECTION	THICKNESS (IN)	YIELD (KSI)	ULTIMATE (KSI)	ELONGATION (%) 2IN GA
TYPE I	0.045	44.4	54.8	41.4
TYPE I	0.040	44	52.5	43.8
TYPE II	0.062	37.6	49.5	46.9

C. BASIC TEST ASSEMBLY

The basic test assembly consisted of two 20 ft. span Fink trusses. Each truss was fabricated using cold-formed steel C-sections of two sizes presented in Figure 1. The top and bottom chords consisted of C 3.65 x 1.635 inch sections with stiffened flanges (Type I) and the diagonal, or web, members consisted of C 2.55 x 1.66 inch sections with stiffened flanges (Type II), both types are assumed to have a radius equal to their thickness.

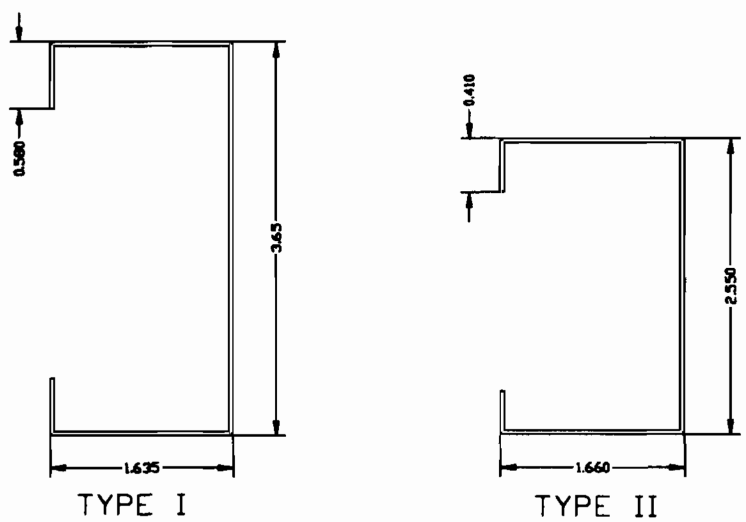


Figure 1. Cross-Section Types

The top chords were continuous from ridge to heel and the bottom chord continuous from heel to heel. Four web members spanned between the chords. The heel connection was created by coping the top flange of the bottom chord, thereby, the top chord rests inside the bottom chord with both channels facing the same direction. The ridge connection was made by coping the bottom flange of one chord, thereby, one chord rests inside the other with webs overlapping. Web, or diagonal, member connections were back-to-back channel connections. Connections were made using 3/4 inch No.10 self-drilling screws through the webs of the sections. Figure 2 presents a sketch of the basic truss configuration. Detail drawings are provided in Appendix A.

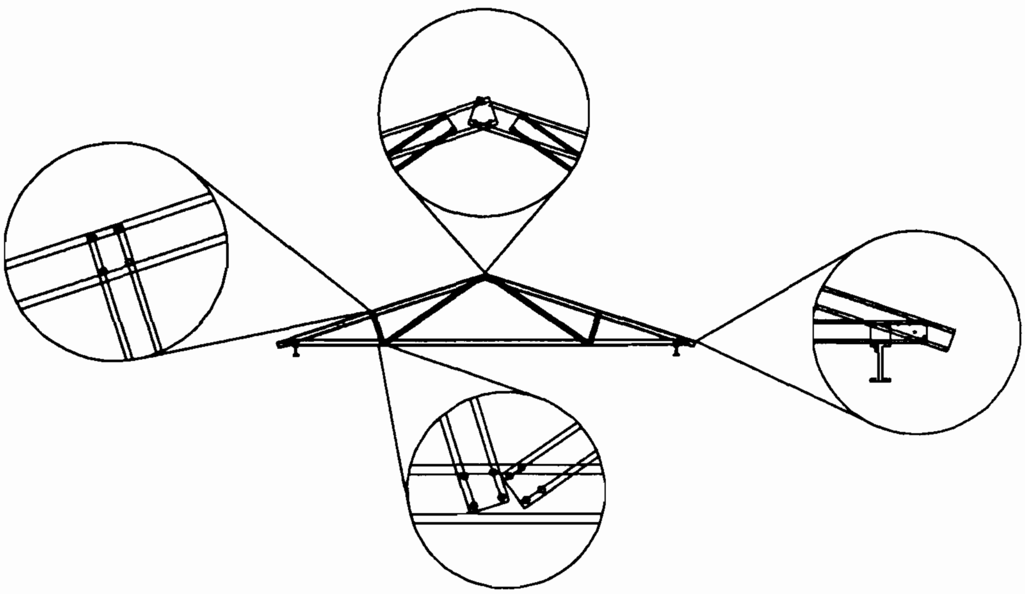
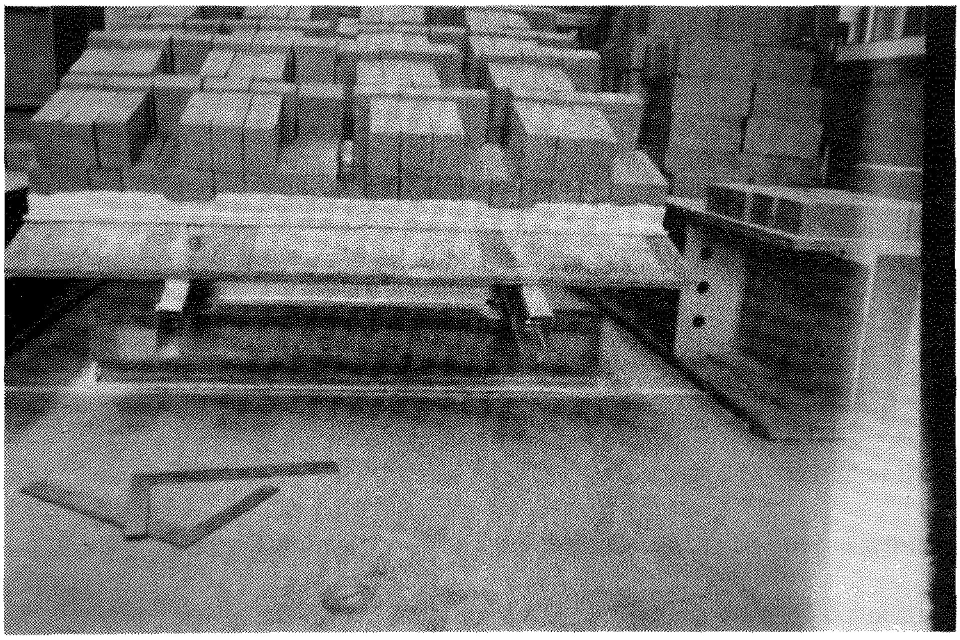


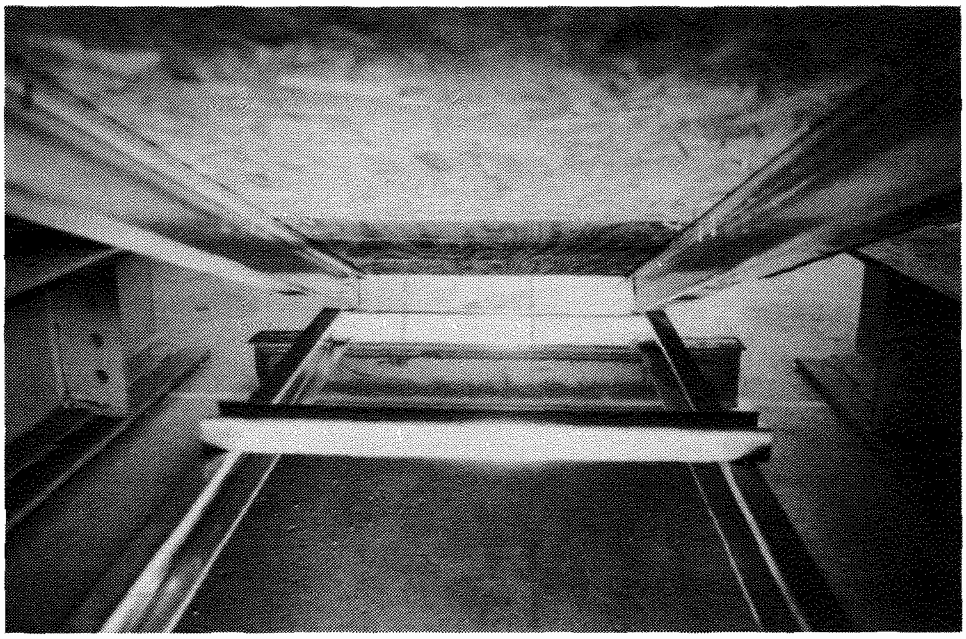
Figure 2. Basic Truss Configuration

The assembly simulated a cold-formed steel residential roof system with a 4:12 roof pitch and provided 2 square feet of tributary area per linear foot for each truss in the assembly. The wide standard 1/2 inch flakeboard sheathing. The sheathing was attached using 1 1/2 inch No. 12 self-drilling screws, penetrating the top flange, every 24 inches along the length of the top chords as seen in Figure 3. In addition, a segment of Type II channel was attached across the top flange of the bottom chords approximately 8 feet from either end of the test assembly to provide stability during construction and to simulate any attachments such as flooring or ceiling board.



a) Sheathing

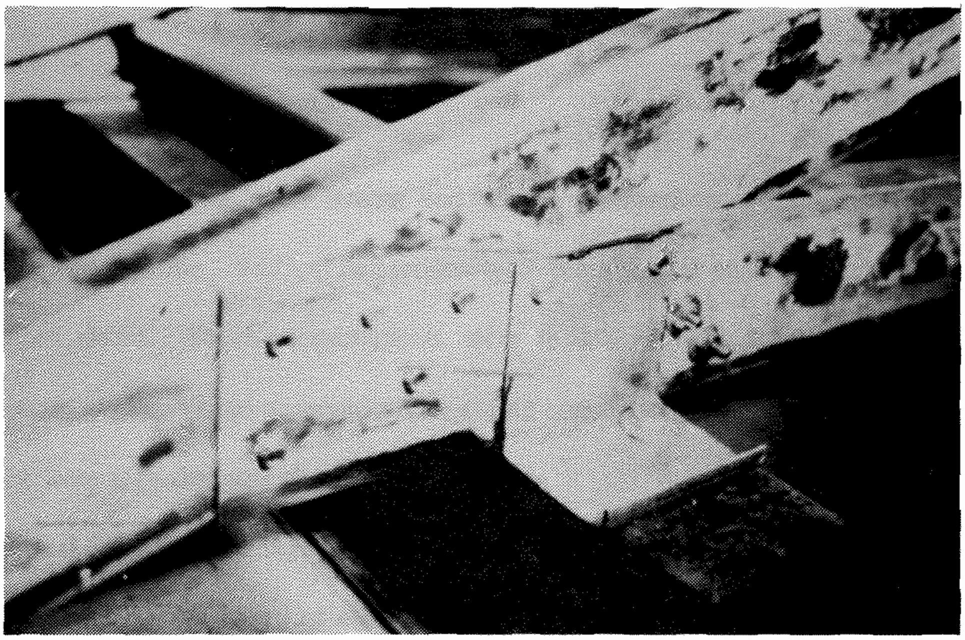
Figure 3. Sheathing and Bottom Chord Brace



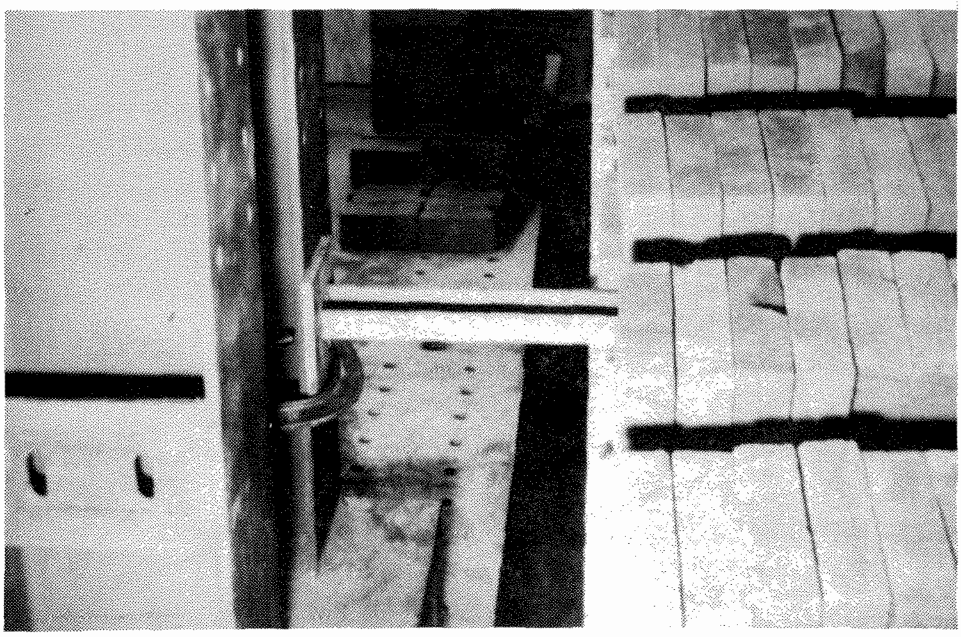
b) Bottom Chord Bracing

Figure 3 continued. Sheathing and Bottom Chord Brace

The basic assembly was tested within the structural test frame located at the Engineering Research Laboratory on the University of Missouri-Rolla campus. The test assembly was simply supported, 4 inches from each end of the bottom chord to avoid the heel connection, on a W 6 x 15 . Clips cut from a Type I section were attached with a single 1/2 inch diameter A307 bolt through the flange of the W-section and connected to the bottom chord at each heel location as shown in Figure 4. A lateral brace was attached across the sheathing and was free to slide vertically against the structural test frame columns. This not only attempted to recognize the lateral stability present in a complete roof system, but was installed as a safety precaution in the event the test assembly became unstable.



a) Truss Support



b) Lateral Bracing

Figure 4. Support and Lateral Bracing

D. DATA COLLECTION

Vertical deflections, axial strains and visual observations were recorded during the experimental investigation. Vertical deflections were measured at panel points and at mid-span of the bottom chord. Strain measurements were read at various locations using electrical resistance strain gages. Visual observations regarding connection performance, member behavior and stability were recorded and photographed.

Vertical deflection readings were measured using dial gages positioned at panel points and at mid-span of the bottom chord of each truss in a test assembly. Readings were taken at 90 seconds after each load increment was reached. Figure 5 depicts the identification nomenclature for the position of vertical deflection readings.

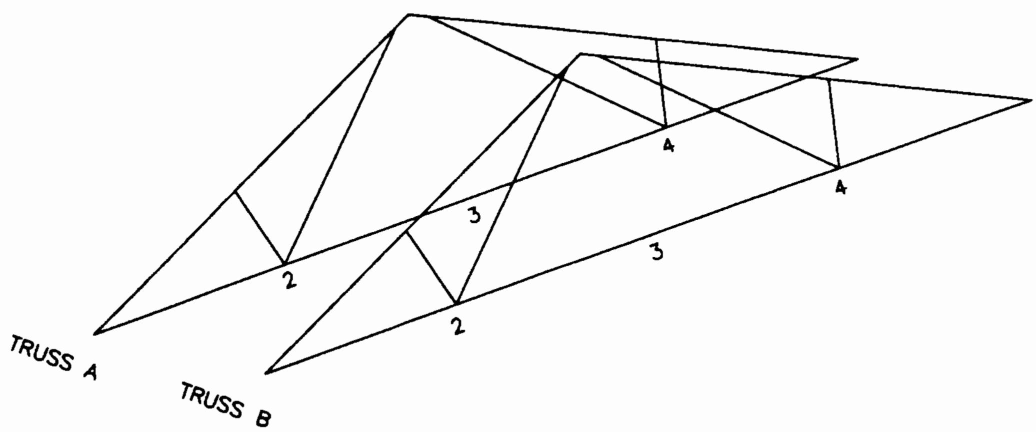


Figure 5. Vertical Deflection Nomenclature

Electrical resistance strain gages positioned parallel to the length of the members were used to record strain measurements at each load increment. Values recorded are the average of two readings taken at 90 second intervals. Instrumentation limited data collection to 10 gages during any given test. Strain measurements were recorded at various points on the trusses as well as various positions within the cross-section at any given point. Figure 6 depicts the identification nomenclature for the position of strain gage readings.

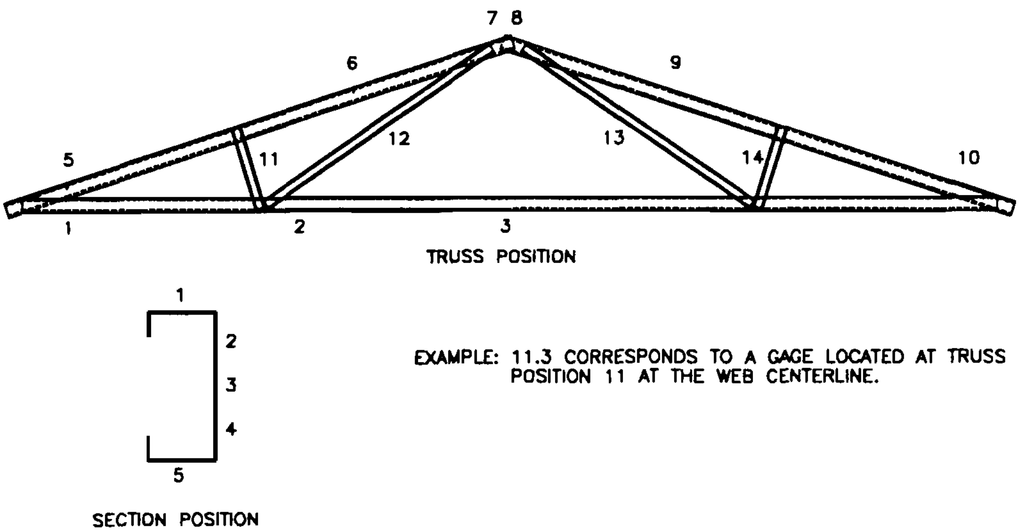


Figure 6. Strain Gage Nomenclature

Visual inspections between each load increment were performed by at least two individuals. Notes were made of any signs of connection distress, local buckling, excessive deformations or stability concerns. Photographs were taken of any notable items such as chord rotation, local buckling, or connector movements.

E. TEST PROCEDURE

Truss test assemblies were gravity loaded using common masonry bricks. To accommodate the placement of the bricks small segments of angle were attached to the sheathing, creating load shelves every 1 foot along the roof surface. Bulk quantities of bricks were weighed and averaged to establish the 5 lb per brick standard used in this investigation. Given that each truss maintains 2 feet of tributary width on a 4 foot width of sheathing, placement of one 5 lb brick on every square foot of sheathing corresponds to 10 lbs per linear foot on each truss. This 10 lb per linear foot gravity load was used as the load increment.

Two types of tests were performed throughout the test program. Non-destructive tests were conducted to establish a data base verified by repeated test cycles. Destructive tests were performed to determine test assembly capacity and failure mode.

Non-destructive tests were initiated by recording a data cycle at the zero load increment for a baseline reading. Loading progressed in 10 lb per linear foot increments. Dial gages were read until excessive deflections created a safety concern beneath the truss or the 100 lb/lft level (approx. 75% of failure load), whichever occurred first. Strain gage readings were always recorded to the 100 lb/lft load. Non-destructive tests were terminated at 100 lb/lft and unloaded followed by readings again taken at the zero load level.

Destructive tests were performed in the same manner. Measured data readings were terminated at 100 lb/lft to eliminate potential danger to personnel or damage to the instrumentation. Incremental loading continued with visual observations until failure.

F. EVOLUTION OF TEST ASSEMBLIES

The basic truss test assembly varied little, but, localized modifications were made throughout the experimental investigation. For example, modifications were made in connection details in an effort to improve overall truss performance. Connections that failed were strengthened in successive tests in order to create general member failures in future test assemblies.

Data collection was also a variable throughout the test program. Vertical deflection data was primarily collected in initial tests for comparison with early computer models. Once consistent deflection behavior was established, the dial gages were eliminated from subsequent tests. Strain measurements were of greater interest in latter tests when specific member behavior was being investigated. Conditions limited each test to 10 data channels, therefore, repetitive tests were performed using the same truss assembly and alternate strain gage locations.

The experimental test sequence began using the basic truss test assembly previously described. Modifications were made to successive truss test assemblies based on the performance of prior assemblies. The parameters outlined in Table II include the addition of ridge web stiffeners, lateral bracing of truss elements, movement of support locations, and the addition of connectors.

Table II. TEST ASSEMBLY PARAMETERS

Test Assembly	Chords (Type I)	Diagonals (Type II)	Ridge Web Stiffener	Lateral Bracing		Additional Connectors	Extended Supports	Number of Tests	
	Thickness (in)			Ridge	Heel			Non-Destructive	Destructive
I	.045	.062							1
II	.045	.062	●					3	1
III	.04	.062	●	●					1
IV	.04	.062	●	●	●				1
V	.04	.062	●	●	●	●			1
VI	.04	.062	●	●	●	●	●		1
VII	.04	.062	●	●	●	●	●	7	1
VIII	.04	.062	●	●	●	●	●		1

Destructive testing of Test Assembly I resulted in local buckling of the web plate element within the ridge connection created by coping of the top chord flanges . As a result, Test Assembly II included a ridge connection web stiffener shown in Figure 7.

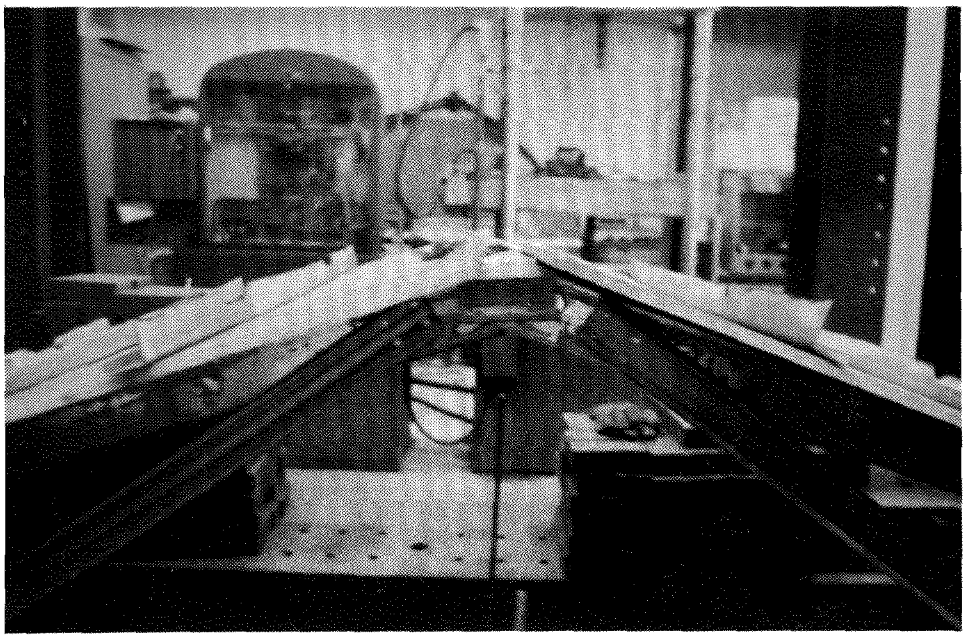


Figure 7. Assembly II Modification

Destructive testing of Test Assembly II resulted in a ridge connection stability failure. The connection rotated and folded about the newly added stiffener. Therefore, Test Assembly III employed lateral bracing on either side of the ridge connection to eliminate the connection instability as depicted in Figure 8. In addition, the use of thinner chord members was implemented to expedite member failures.

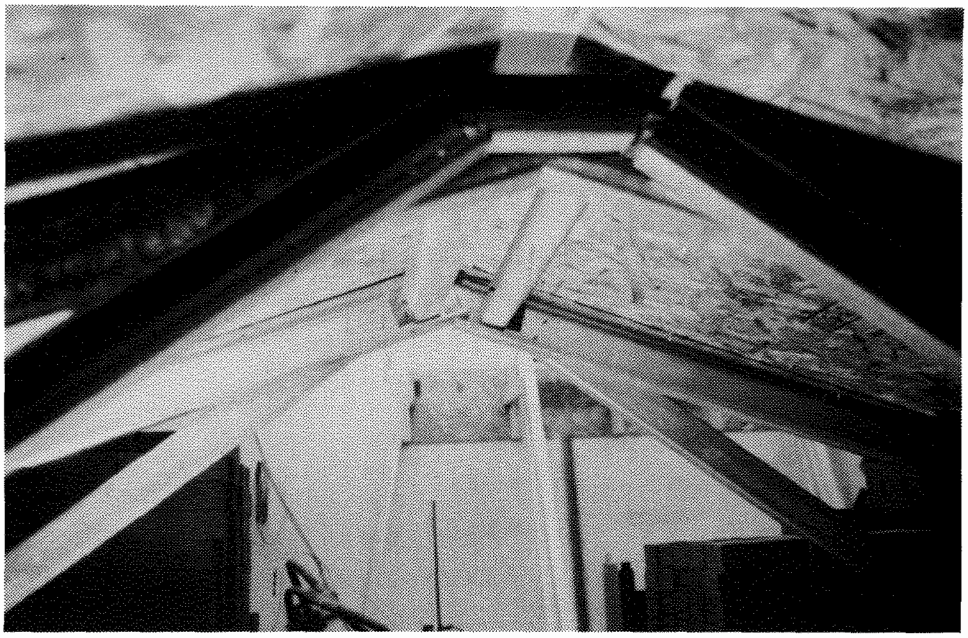
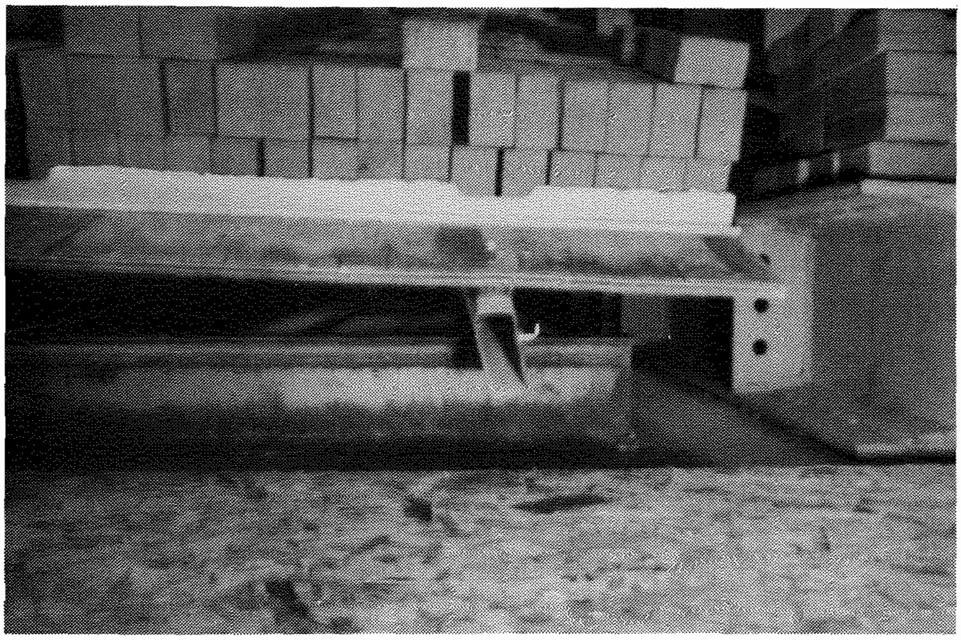
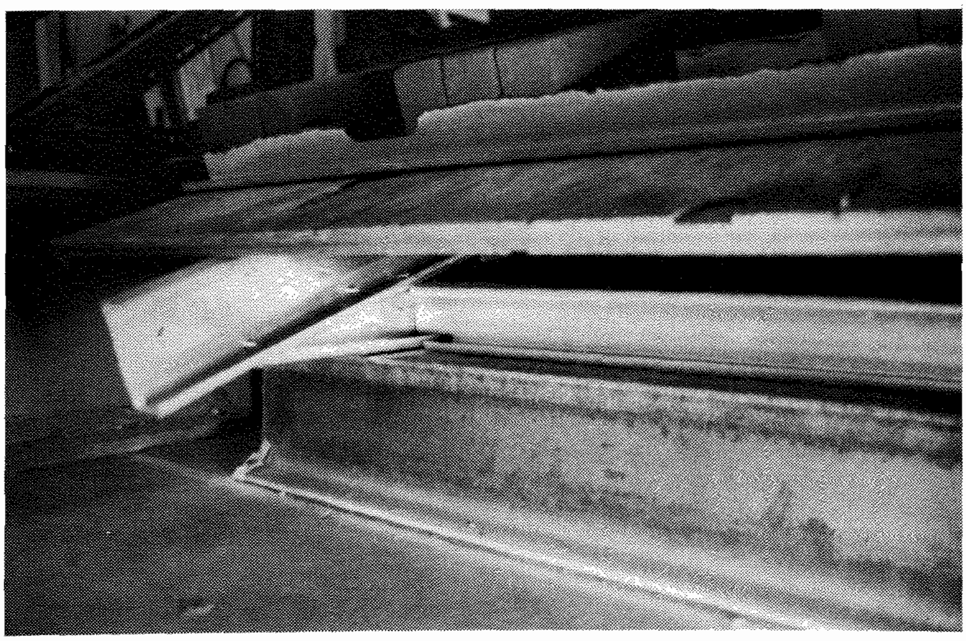


Figure 8. Assembly III Modification

The failure of Test Assembly III was caused by a combination of local buckling of the top chord web above the heel connection and excessive rotation of the top chord in the heel connection which can be seen in Figure 9. An additional lateral brace was placed in Test Assembly IV to eliminate the top chord rotation of the heel connection. The lateral brace was positioned between heel connections and bolted through the top flange of the W 6 x 15 support.



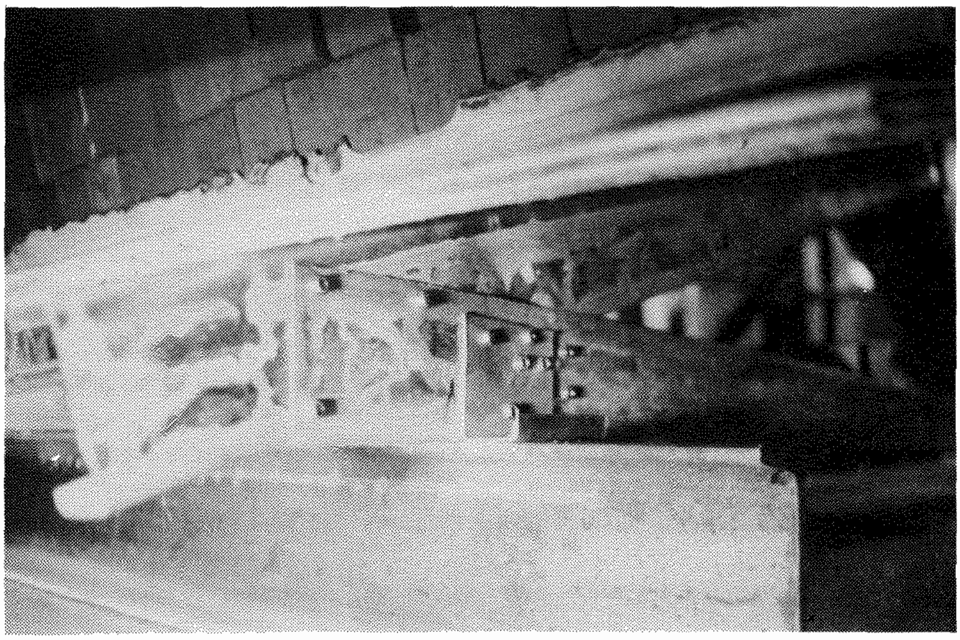
a) Assembly III Failure



b) Assembly IV Modification

Figure 9. Assembly III Failure and Assembly IV Modification

Destructive testing of Test Assembly IV resulted in local buckling of the top chord web at the heel connection and eventual collapse. The top chord web buckle can be seen in Figure 10. Prior to failure, screw connectors were observed to have rotated and pulled out in the heel connection. Calculations based on the Specification Provisions for Screw Connections[13], using estimated member loads at the 170 lb/lft load level, indicated approximately 9 connectors would be required to sustain the higher truss assembly capacity being developed. Accordingly, Test Assembly V included additional screw connectors placed in all heel connections (Fig. 10).



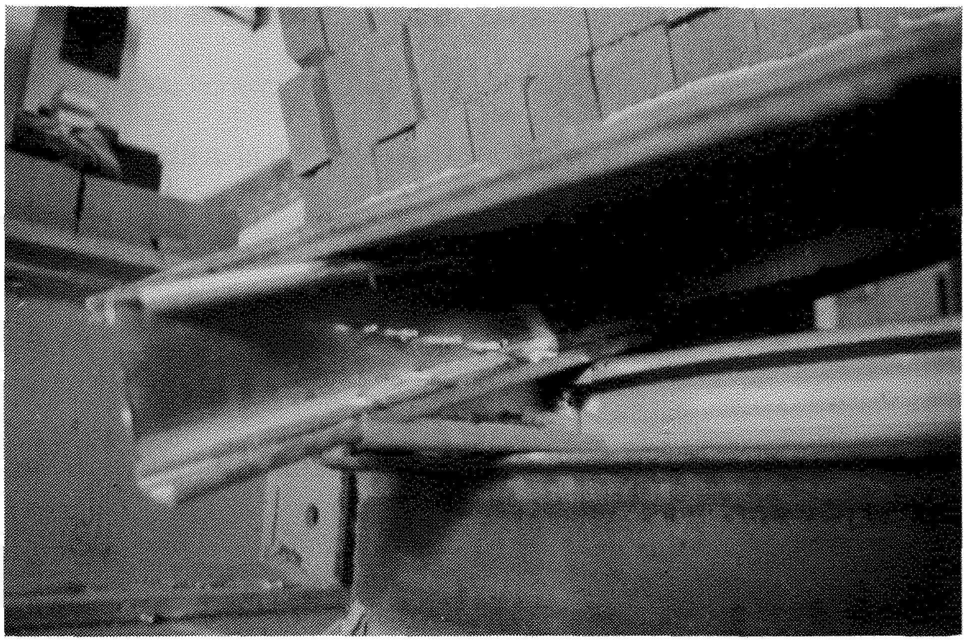
a) Assembly IV Failure

Figure 10. Assembly IV Failure and Assembly V Modification

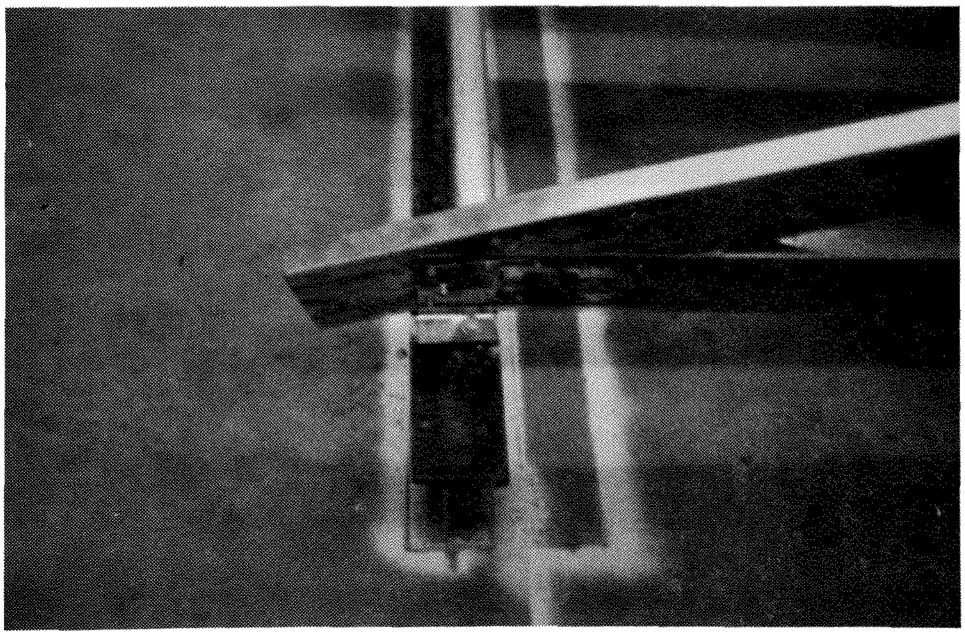
b) Assembly V Modification

Figure 10 continued. Assembly IV Failure and Assembly V Modification

The Test Assembly V failure was caused by local buckling of all top chord web elements just above the heel connections, see Figure 11. A support modification was made to Test Assembly VI in an effort to reduce any major axis eccentricity created by the heel connection overhanging the support member. The support was extended 4 inches at each end of the truss, placing it directly beneath the top chord where it intersects the heel connection.



a) Assembly V Failure



b) Assembly VI Modification

Figure 11. Assembly V Failure and Assembly VI Modification

A member failure was achieved in the destructive testing of Assembly VI. The top chord buckled in the region between the ridge and the diagonal. Attempted rotation of the top chord was noted prior to failure in the same segment. The rotation of the top chord between the ridge and tensile diagonal was restrained every 24 inches by the sheathing connectors penetrating the top chord flange which is shown in Figure 12.

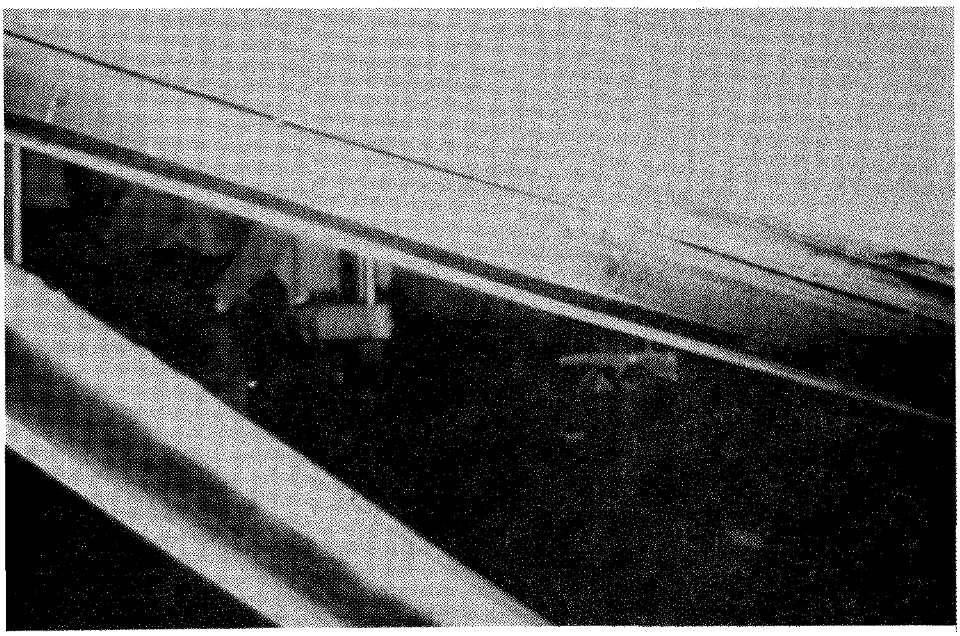


Figure 12. Assembly VI Failure

Test Assembly VII was built to verify the failure behavior of Assembly VI. The same behavior was observed in the destructive testing of Assembly VII as was observed for Assembly VI. Inadvertently, a connector used in placement of the lateral bracing penetrated the top chord at the point of failure, see Figure 13. In an effort to eliminate the possibility that this connector may have influenced the location of failure, another test assembly was constructed.

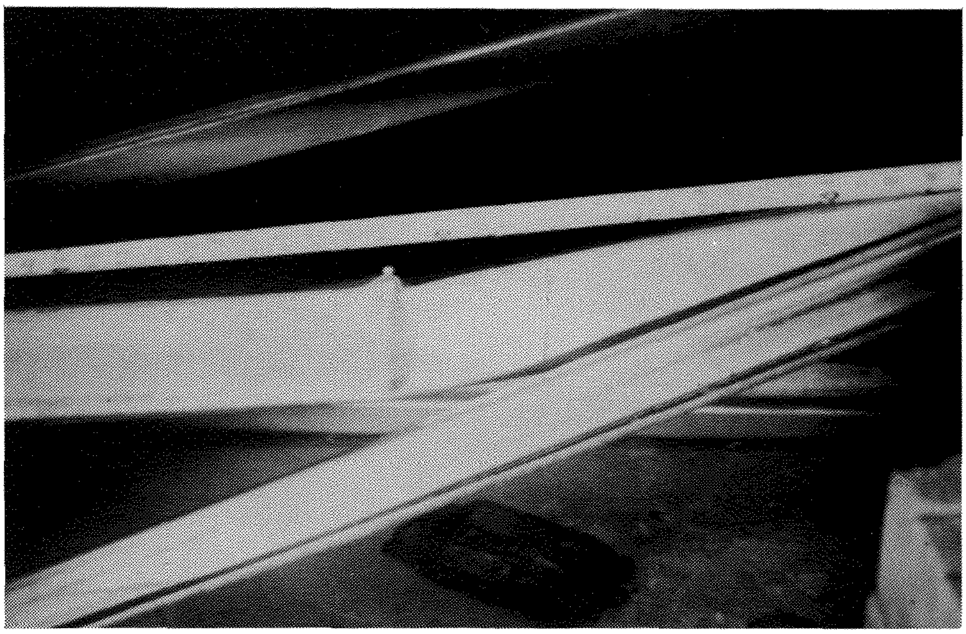


Figure 13. Assembly VII Failure

Test Assembly VIII was carefully constructed to avoid interfering with the truss top chords when placing the lateral bracing. Destructive testing of this assembly resulted in the buckling of the top chord in the region between the ridge and diagonal, as observed for Assemblies VI and VII and shown in Figure 14.

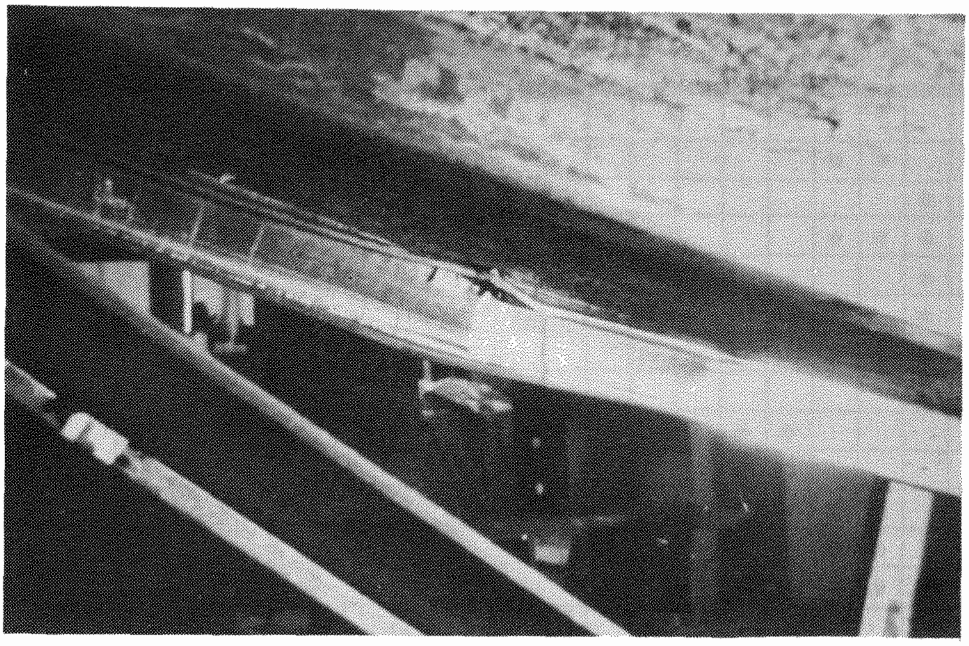


Figure 14. Assembly VIII Failure

Each of the truss test assemblies was subjected to several test cycles in order to create a consistent behavior pattern. Vertical deflection measurements were recorded during initial testing, while strain measurements were used in latter tests to help identify specific member behavior. The data recorded throughout the test program for each truss assembly is illustrated in Table III.

Table III. VERTICAL DEFLECTION AND STRAIN MEASUREMENTS

Test Assembly	Test	Load (lb/lft) Destructive (---)	Vertical Deflection Position						Strain Gage Position & Section Location																									
			Truss A			Truss B			1					2	3	4	5					6	7	8	9	10	11			12			13	14
			2	3	4	2	3	4	1	2	3	4	5	3	3	3	1	2	3	4	5	3	3	3	3	3	2	3	4	2	3	4	3	3
I	1	(100)																																
II	1	70	●	●	●	●	●	●																										
	2	70	●	●	●	●	●	●																										
	3	70	●	●	●	●	●	●																										
	4	(150)		●			●																											
III	1	(130)	●	●	●	●	●	●																										
IV	1	(170)	●	●	●	●	●	●																										
V	1	(140)	●	●	●	●	●	●																										
VI	1	(150)	●	●	●	●	●	●																										
VII	1	100																		●			●	●	●	●	●								
	2	100																		●			●	●	●	●	●								
	3	100												●	●				●			●	●	●				●			●		●	●
	4	100												●	●				●			●	●	●				●			●		●	●
	5	100																●	●	●		●					●	●	●	●	●	●		
	6	100							●	●	●	●	●				●	●	●	●	●													
	7	100							●	●	●	●	●				●	●	●	●	●													
	8	(160)																																
VIII	1	(180)																																

IV. COMPUTER MODELED TEST ASSEMBLY

A. GENERAL

A two-dimensional static analysis computer model was created using M-STRUDL, a microprocessor driven version of the popular linear, elastic, finite element based structural design language. The model was prepared as the basis for determining analysis recommendations such as support conditions and joint performance assumptions. Once a functional model was established, design provisions using this model were verified by comparison to the experimental full-scale truss performance.

B. MODEL ASSUMPTIONS

A two-dimensional static frame analysis was used in modeling an individual truss. Top chords were assumed continuous from ridge to heel and the bottom chord continuous from heel to heel. Chord members were pinned at the ridge and heel connections. All diagonal, or web, members were pin connected (end moments released). The model was simply supported and gravity loaded using a uniform vertical projection. All member lengths and joint indices for the model were determined using centerline dimensions and centerline intersections for Test Assemblies VI through VIII, which represented the final truss configuration tested. Cross-sectional properties were calculated, using averages of dimensional measurements to the nearest .005 inch and assuming radii equal to thickness, using CFS, a cold-formed steel design program.

The M-STRUDL analysis assumes the axial loads to act through the centroid of the section. The force transfer in the test assembly is predominately through the webs in the connections, therefore, additional eccentricity considerations had to be made when analyzing the model output. Also, the effects of coping, localized deformations at connectors, torsional eccentricities , and shear deformations of the webs of the members are neglected by M-STRUDL.

V. EVALUATION OF TEST RESULTS

A. GENERAL

The evaluation of the test results was a two part procedure. Phase I involved the development of a computer model and analysis technique to simulate the experimental truss. The second phase of the evaluation was to compare the experimental truss behavior to the AISI Specification[1] prediction of failure using the Phase I analysis results.

B. COMPUTER MODEL VS. EXPERIMENTAL ASSEMBLY

To evaluate the accuracy of the computer modeled truss, comparisons of member stresses and joint deflections between the model and experiment were performed. As previously stated, the nature of the STRUDL computer model required additional considerations regarding the location of axial forces when determining the member stresses. The following discussion contains the comparative determinations made on a joint and member basis.

1. Bottom Chord Panel Point: The measured vertical deflections of panel points A2, A4, B2, B4 (Fig. 5) of the bottom chord were compared to the corresponding computed deflections. The deflection data for Test Assembly VI, which represents the final configuration tested (Table III), is graphically presented in Figure 15. The measured deflections are within 27% of the computed panel point deflections at 80 lb/lft, which was approximately 1/2 the failure load. The 27% deviation is misleading because the computed deflection underestimated the measured deflection by only .06 inches. Relatively speaking the percentage is large, but the range is very small when considering the actual deflection and method of measurement. The computed deflections are based on assumed simple supports. The added stiffness of the end supports and proximity of the panel points to the end supports may have contributed to the underestimation of the computed deflections. In addition, the effects of shear deformation of the webs was not

accounted for and may be a contributing factor to the underestimation. These panel point deflections represent an L/1090 deflection limit.

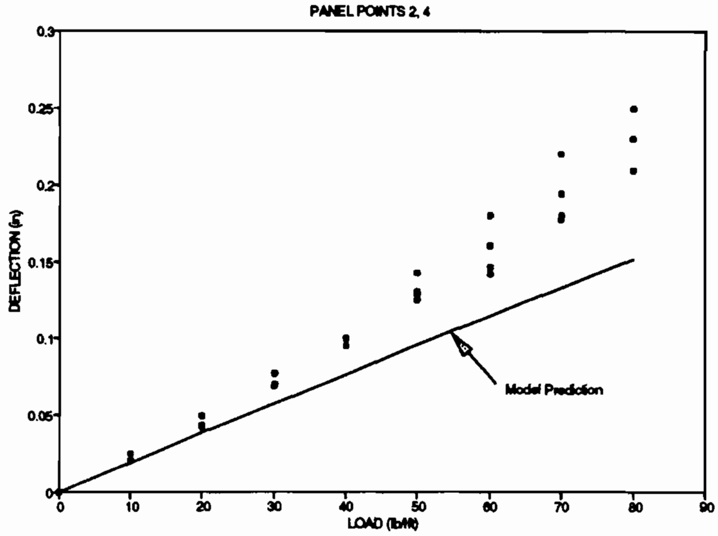


Figure 15. Typical Computed vs. Experimental Deflection

2. <u>Bottom Chord Mid-Span</u>: The measured vertical deflections of the mid-span of the bottom chord were compared to the corresponding computed deflections. The comparison is represented by Figure 16 and shows a better correlation of measured vs. tested deflections than that of the panel points. This tends to support the assumption that the end support conditions, lack of accounting for web shear deformation, and copping of sections at connections may have influenced the accuracy of the comparison and contributed to the deviation seen in Figure 15, regarding the panel point deflection prediction. The computed deflection at 80 lb/lft is within 17% of the measured deflection, a relative deflection difference of .04 inches. The mid-span deflections represent the ability to achieve a deflection limit of approximately L/1040.

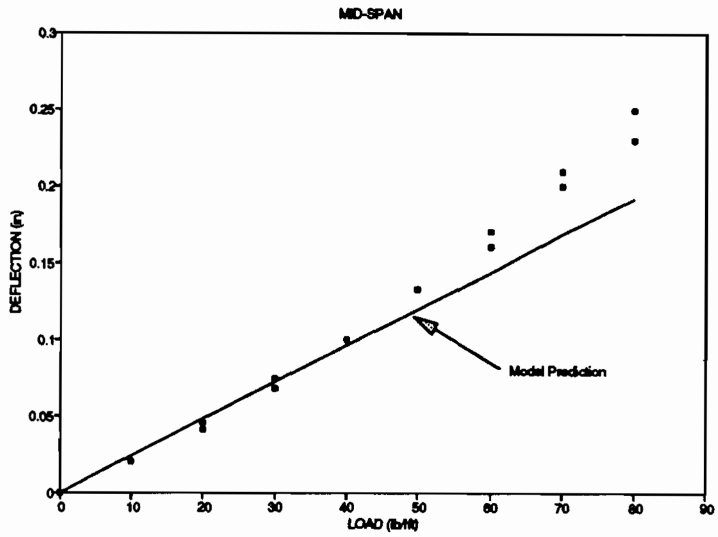


Figure 16. Typical Computed vs. Experimental Deflection

3. Compression Diagonal: (Member 11 or 14, Fig. 6) Axial strain measurements recorded on the member centerline at mid-span, section position 3, were used to perform stress comparisons between the truss assembly and computer model (Fig. 6). When determining the stress present in the web, an axial $\frac{P}{A}$ value was not sufficient, P being the axial compression load and A being the gross cross-sectional area. Considerations had to be made to account for stresses resulting from the eccentricity created by connecting through the web of the diagonal. Additionally, restraint against rotation of the top chord was provided by the diagonal which, in turn, applied a minor axis moment to the diagonal. To determine the web stress based on the computer generated model forces the following equations were used:

$$f = \frac{P}{A} + \frac{(P \times e)}{S_y} + \frac{(Q \times u)}{S_y} \tag{5}$$

Where:

P = Axial load in the diagonal from the computer model

A= Unreduced cross-sectional area of the diagonal member

S_y= Minor axis section modulus of the diagonal member

e = Web to c.g. distance for the diagonal member (Fig. 23)

u = Shear center to flange centerline distance for the top chord member (Fig. 24)

$$u = \frac{b_f}{2} - e_c + x_o \tag{6}$$

b_f = Top chord flange width (Fig. 24)

e_c = Web to c.g. distance for the top chord member (Fig. 23)

x_o = Shear center to c.g. distance for the top chord member

Q= Tributary load normal to the top chord member (Fig. 24)

$$Q = w \times COS(\Theta) \times (\frac{a_1}{2} + \frac{a_2}{2}) \tag{7}$$

w = Uniform vertical load

Θ = Angle of the top chord member to the horizontal plane

a_1 = The center to center distance between the diagonal member and the next sheating connector above the diagonal (Fig. 24)

a_2 = The center to center distance between the diagonal member and the next sheating connector below the diagonal (Fig. 24)

The variable Q is used in the determination of the magnitude of the rotational restraint which the diagonal member provides to the top chord member. It is assumed any point of rotational bracing is responsible for restraining the load on 1/2 the unbraced segment on either side of the brace. Thereby, $\frac{a}{2}$ on each side of the point of rotational restraint was used to determine the tributary load. The load, causing rotation of the top chord member, was assumed to act through the center of the top flange of the top chord member. The top chord rotation induced an applied moment to the diagonal member. To quantify the induced moment, the load, Q, was multiplied by u, the distance from the shear center to the top flange centerline of the top chord member.

The agreement of the predicted web stress, given by equation 5, compared to the tested value can be seen in Figure 17. The predicted behavior closely corresponds to that of the experimental stress results.

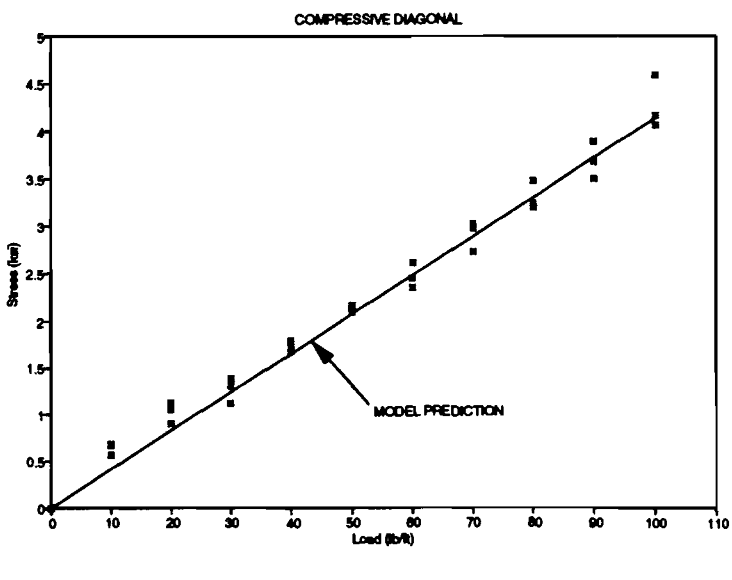


Figure 17. Experimental vs. Predicted Web Stress in the Compressive Diagonal

4. Tension Diagonal: (Member 12 or 13, Fig. 6) A comparison of the axial web stress at the member centerline, section position 3, was performed using axial strain measurements taken at the mid-length of the member (Fig. 6). As previously discussed, the eccentric nature of the loading had to be addressed when comparing computer model results to experimental results. The orientation of the diagonal to the top chord results primarily in torsion to the diagonal when the top chord attempts to rotate and, therefore, would not effect stress readings near the web centerline. In addition, the connection of the diagonal to the top chord is located very near the ridge connection where little rotation of the top chord was observed. Therefore, the rotational restraint provided to the top chord considered in the compressive diagonal (member 13) was not significant when determining web stress of the tensile diagonal. The following equation was used when determining the applied web stress:

$$f = \frac{P}{A} + \frac{(P \times e)}{S_y} \tag{8}$$

Where:

P = Axial load in the diagonal member from the computer model

A = Unreduced cross-sectional area of the diagonal member

S_y = Minor axis section modulus of the diagonal member

e = Web to c.g. distance for the diagonal member (Fig. 23)

The comparison between the computed web stress (Eq. 8) and the measured web stress is presented as Figure 18. A good correlation between the predicted stress and experimental stress was achieved.

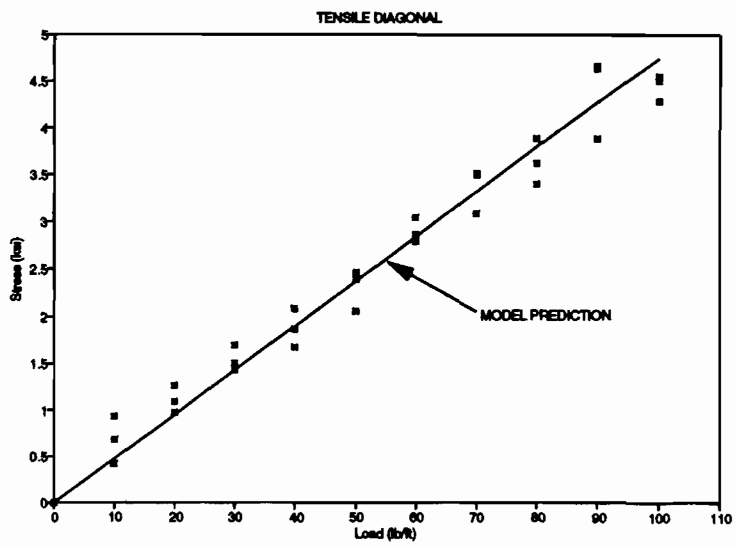


Figure 18. Experimental vs. Predicted Web Stress in the Tensile Diagonal

5. Top Chord: (Member 9, Fig. 6) The comparative evaluation performed on the top chord was conducted in the region between the compression diagonal connection and the ridge connection. The axial web stress measurements of the top chord at mid-depth were used for comparison with predicted stresses. The web stress was predicted using Equation 8 with the appropriate top chord section properties. The effect of composite action between the C-section and sheathing was investigated by calculating transformed steel section properties for use in the analysis. The composite action did not significantly alter the results and also involved many assumptions regarding the sheathing properties, connector performance, and composite action justification. The decision was made to conservatively limit the analysis to the steel member alone. Figure 19 relates the prediction made using Equation 8 to the experimental web stress.

The model consistently overestimated the measured stresses. The overestimation may be attributed to the observed twisting of the top chord, which altered (reduced) the cross-sectional properties (Fig. 21).

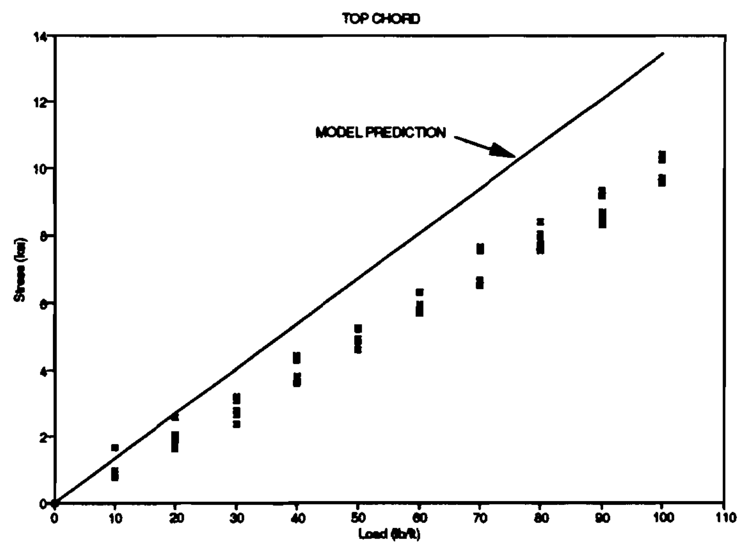


Figure 19. Experimental vs. Predicted Web Stress in the Top Chord

6. Bottom Chord: (Member 3, Fig. 6) The bottom chord was examined at mid-span using centerline web stress measurements. The predicted web stress was determined by a $\frac{P}{A}$ determination using the chord gross cross-sectional area and axial load P taken from the computer model. The additional consideration accounting for the eccentric application of the load, as used in previous member analysis, was neglected because of the extended distance to the nearest

connection. The relationship between the computed web stresses and the experimentally determined stresses can be observed in Figure 20. The predicted stress corresponds closely to the experimental values.

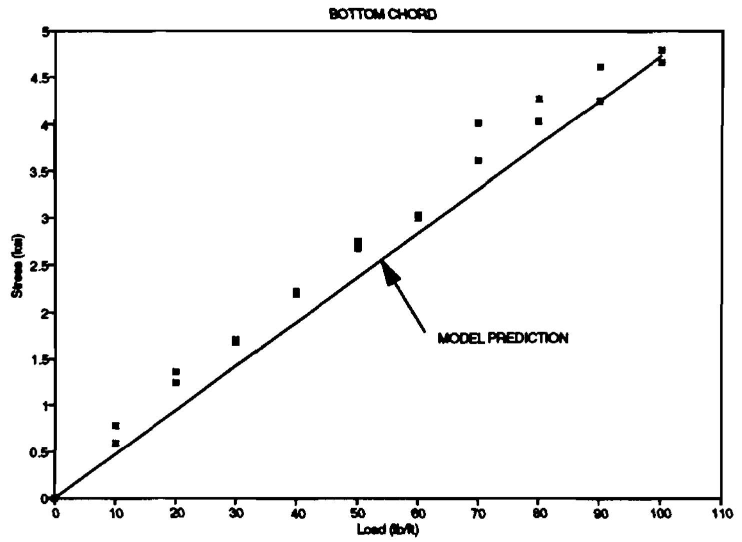


Figure 20. Experimental vs. Predicted Web Stress in the Bottom Chord

C. PERFORMANCE PREDICTION VS. EXPERIMENTAL BEHAVIOR

Based on the Phase I study, a functional analysis model was established. Thus, design provisions using this model were developed and verified by comparison to the experimental truss behavior. An effort was made to utilize the existing Specification[1] for member strength determination. Through the progression of tests, Section III-F.1-7, connection reinforcing methods and lateral bracing techniques were developed to provide adequate resistance to localized failures. The design provisions discussed in Section VI specify qualitative matters regarding the connection reinforcement and lateral restraint, yet, leave the final design determination to the design engineer.

The selection of members used in the truss test assembly was done in such a manner that the top chord was the critical member in the system. The test assembly utilized heavier sections than required for the diagonal members which were the least stressed. The bottom chord was a tension member of the same cross-section as the top chord. Therefore, the member of greatest concern in the truss test assembly was the top chord. This member was subjected to the distributed gravity loading which creates major axis moment in addition to axial compression. Based on this test assembly, the strength performance criteria for the truss system was established by the top chord member.

Specification[1] Section C5 was used to perform a failure analysis of the top chord as a beam-column using the following interaction equations for axial compression and bending:

$$\frac{P}{P_n} + \frac{C_{mx} \times M_x}{M_{nx} \times \alpha} \leq 1.0 \qquad (9)$$

$$\frac{P}{P_{no}} + \frac{M_x}{M_{nx}} \leq 1.0 \qquad (10)$$

The nomenclature and procedure for determining the required parameters used in the interaction equations is outlined in the Specification[1]. The failure analysis was performed using the subsequent assumptions regarding unbraced lengths, effective lengths, and amplification factors.

1. Unbraced Length: The unbraced length, L, was assumed to be the distance between panel points for the major and torsional axes. The sheathing connector spacing was used as the minor axis unbraced length.

2. Effective Length Factor: The effective length factor, K, was assumed to be .75 for all axes.

3. End Moment Coefficient: The end moment coefficient, C_m, was assumed to be .85 as per the Specification[1].

4. Bending Coefficient: The bending coefficient, C_b, was assumed to be 1.0 in accordance with AISI Specification[1] Section C3.1.2.

The top chord member failure typically occurred at the sheathing connector nearest to the location of maximum positive moment. This was likely caused by the localized rotational restraint of the top chord provided by the connector, which deforms the cross-section as shown in Figure 21.

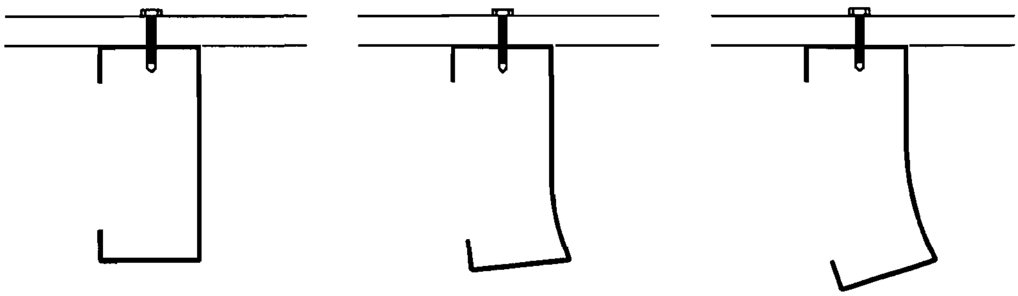


Figure 21. Cross-Section Deformation Resulting from Rotational Restraint

The strength evaluations for the top chord were conducted at points of maximum moment. Figure 22 shows the resultant predicted failure load in comparison to the experimental behavior of test assemblies VII and VIII. Failure was estimated to occur, using Equation 9, at a load of approximately 174 lb/lft. This corresponds favorably with the experimental failure loads of 160 lb/lft and 180 lb/lft for Test Assemblies VII and VIII, respectively.

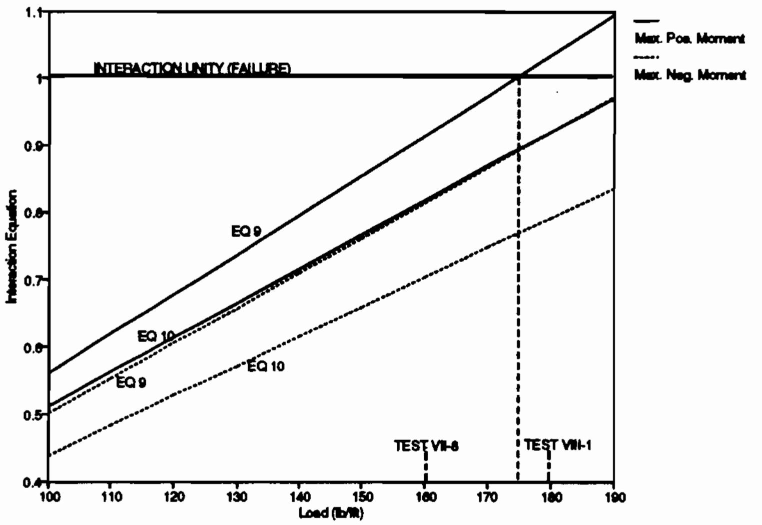


Figure 22. Experimental vs. Predicted Failure Load

VI. CONCLUSIONS

A. GENERAL

The purpose of this research project was to discover knowledge regarding the behavior of cold-formed steel roof truss systems in order to establish design recommendations. The conclusions of the research address analysis techniques and design requirements for trusses subjected to gravity loads applied to the top chord.

B. GENERAL ANALYSIS AND DESIGN ASSUMPTIONS

The following analysis model and design assumptions may be made to preclude a rigorous investigation to determine exact joint flexibilities, effective lengths and reduction or amplification factors:

1. Top Chord: The member is continuous from heel to ridge having pinned ends.

2. Bottom Chord: The member is continuous from heel to heel having pinned ends.

3. Diagonals: The member is pin connected at each end.

4. Unbraced Length: The unbraced length may be taken as the center-to-center distance between panel points, except for the top chord minor axis for which the unbraced length may be taken as the distance between sheathing connectors.

5. Effective Length Factor, K: A value of 0.75 may be used for the top chord member and a value of 1.0 may be used for the diagonal members.

6. End Moment Coefficient, C_m: A value of 0.85 may be used unless otherwise justified.

7. Bending Coefficient, C_b: To be determined according to Section C3.1.2 of the Specification[1].

C. MEMBER DESIGN

1. Top Chord: The interaction effects of axial compression and bending should be investigated at locations of peak moment in accordance with Section C5 of the Specification[1].

2. Bottom Chord: The member strength is to be investigated as an axially loaded tension member according to Specification[1] procedures.

3. Tension Diagonal: Evaluate the member strength for axial tension and minor axis bending resulting from eccentric loading in the plane of the member's web. The eccentricity, e, is defined as the distance from the outside of the web to the centroid of the member's cross-section as seen in Figure 23. For design, the member strength shall be determined in accordance with Section C5 of the Specification[1] with regards to the effects of axial load and bending.

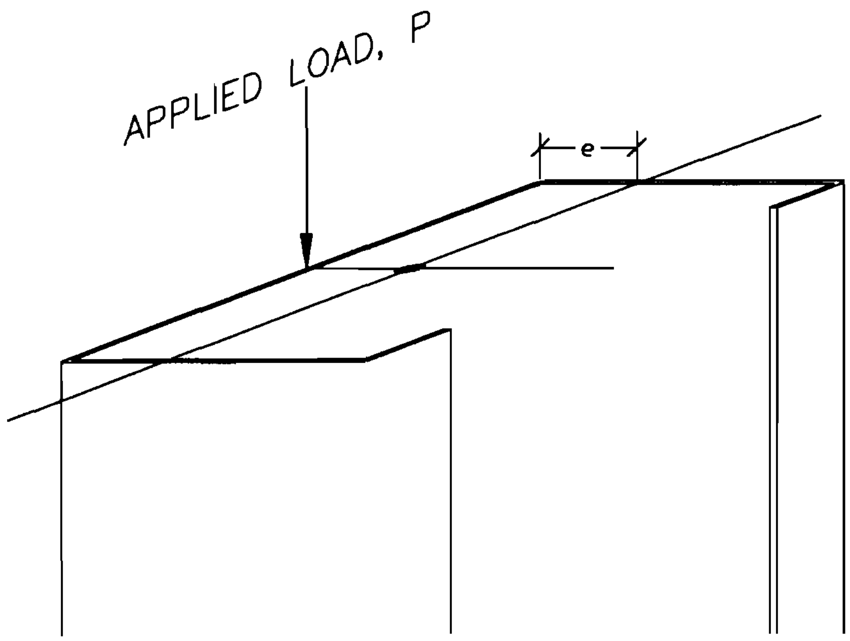


Figure 23. Eccentric Loading of the Diagonal Member

4. Compression Diagonal: Evaluate the member's strength for axial compression and minor axis bending resulting from eccentric loading through the web and minor axis bending induced by rotation of the top chord. The eccentric loading is as defined by the tensile diagonal design (Fig. 23). The minor axis moment created by rotation of the top chord is quantified by $Q \times u$. Where Q is the total distributed load applied normal to the top chord member for a length of .5a each side of the diagonal. The length a is the centerline to centerline distance between the diagonal member and the next sheathing connector. The coefficient u is the distance from the shear center to the flange centerline of the top chord as presented in Figure 24. For design, the member strength shall satisfy the interaction requirements of axial compression and bending as stipulated by Section C5 of the Specification[1].

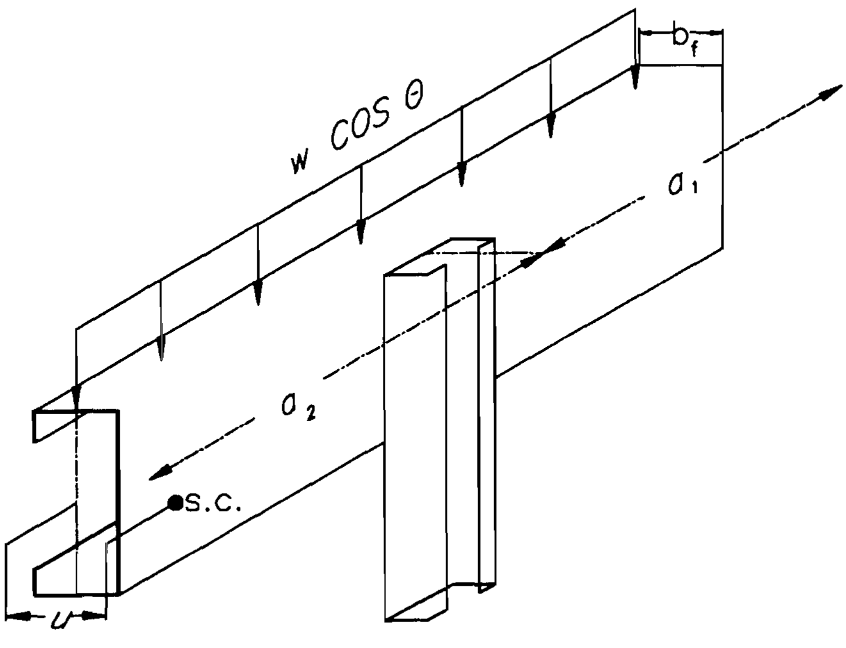
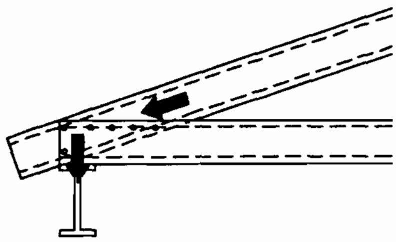


Figure 24. Moment Induced by the Top Chord Rotation

D. CONNECTION DESIGN

The predominant concern when designing truss systems is the creation of eccentric connections and the compromising of the structural integrity of the sections by the coping of flanges. The design engineer must reinforce webs of members in connections to prevent web plate buckling caused by coping of flanges. Web crippling, created by heavy localized loads must also be considered. The heel connection must provide a direct load path for top chord axial loads to transfer into the support, see Figure 25. Careful consideration must be given to the indirect load path created by eaves or overhanging of supports. Connector capacities shall be defined according to the Specification provisions for screw connections[13].



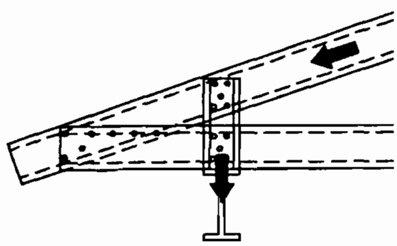


Figure 25. Examples of Direct Load Paths to the Support

E. BRACING

The function of bracing is to assure that the individual trusses act as elements of a system. Specifically, the ridge and heel connections need to be laterally braced between trusses to prevent stability failures. The top chord in a heel connection is prone to rotate. The ridge connections tend to fold or buckle laterally under load if not aligned and laterally restrained. The design engineer must appropriately address these issues.

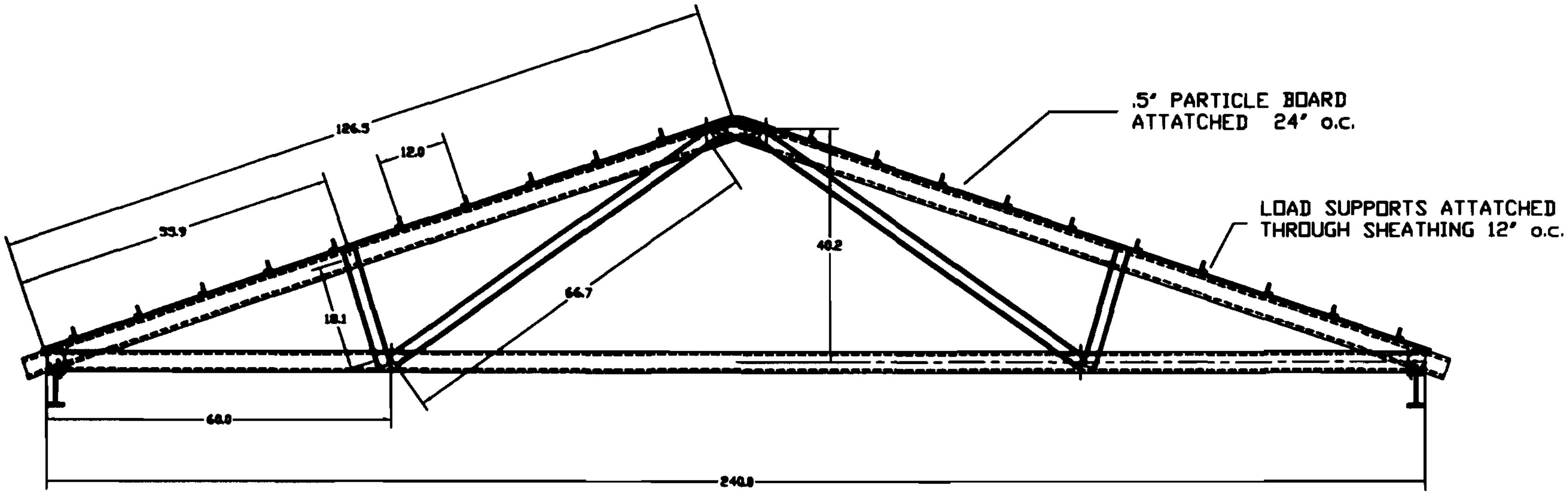
VII. FUTURE RESEARCH

The scale of the test assemblies and the variability of the failure location limited the scope of this initial investigation. Future studies should benefit from not only the theoretical and analytical conclusions of this research, but also from the logistical developments achieved throughout the investigation. The conclusions lack the development of diagonal member design requirements regarding the ultimate failure load. Additionally, the behavior of the truss connections needs to be more closely examined, including design provisions for the coping of members for the construction of connections. The current study was limited to one truss configuration, therefore, the conclusions of this study should be compared to alternate truss geometries.

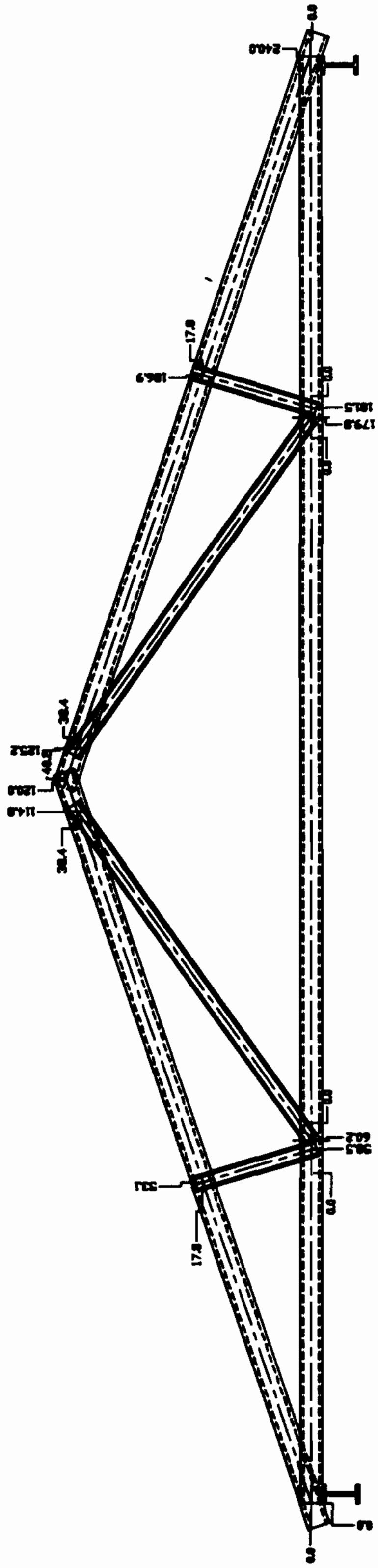
APPENDIX

DETAIL DRAWINGS OF THE TRUSS ASSEMBLY

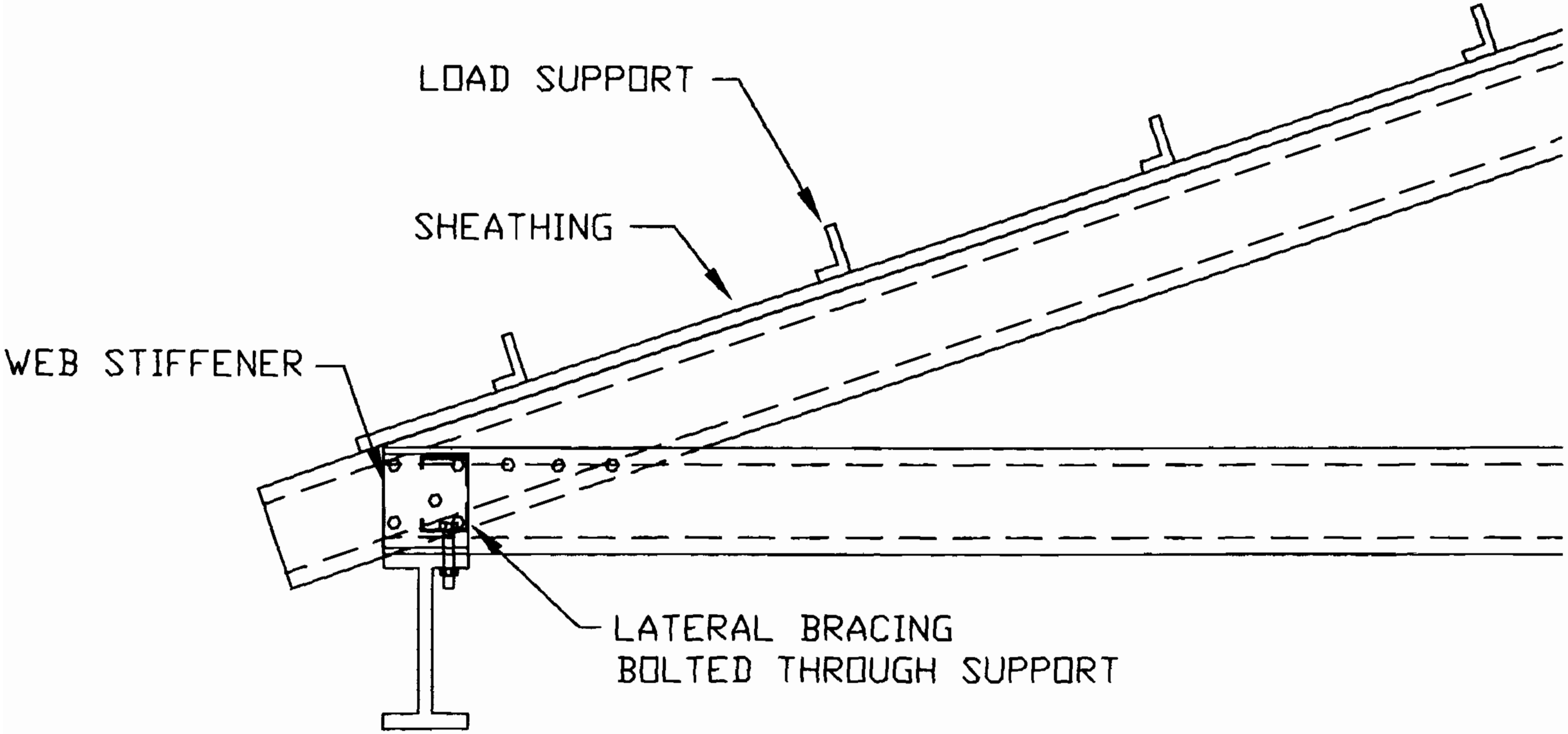
FINAL TRUSS GEOMETRY

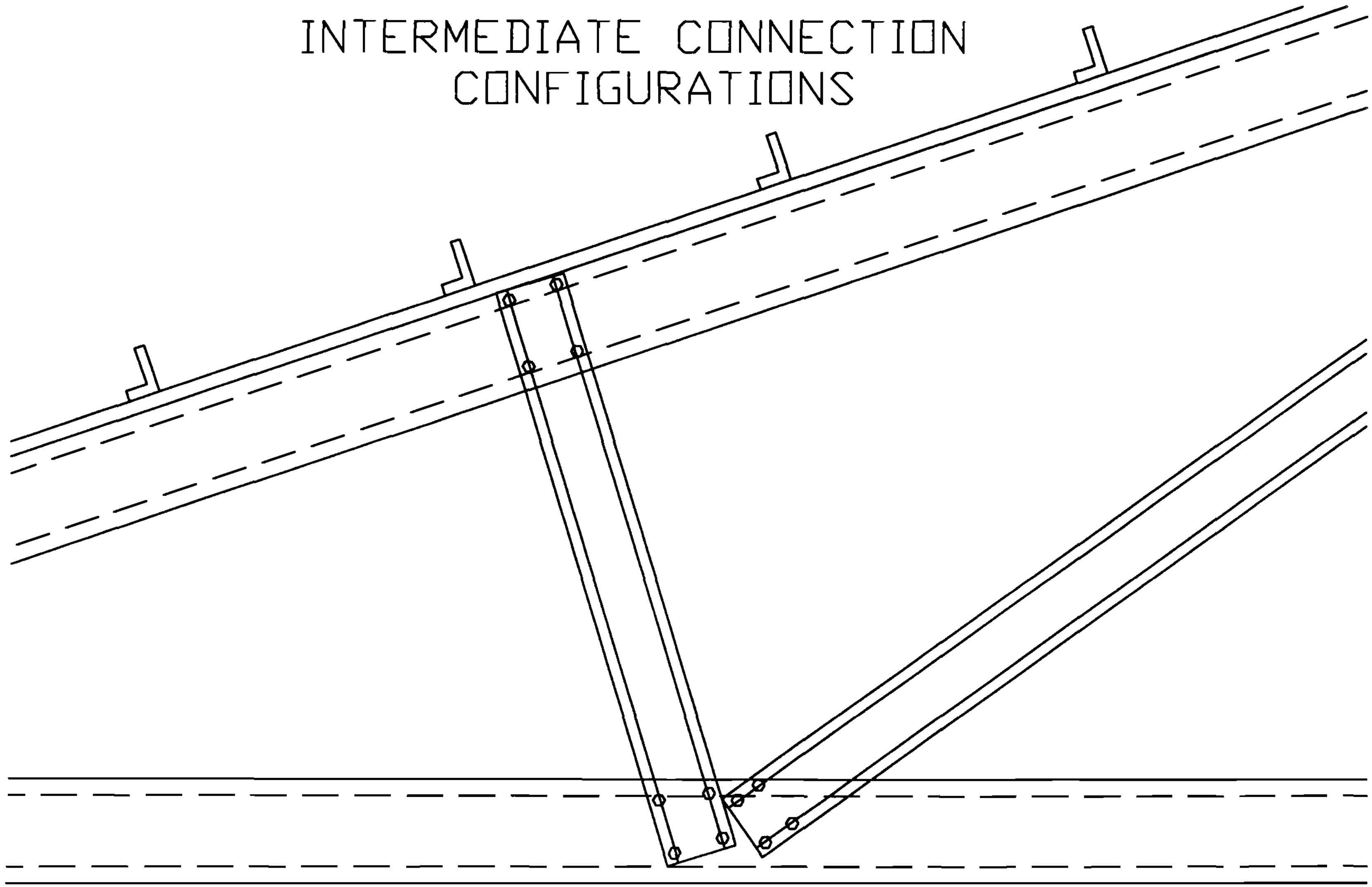


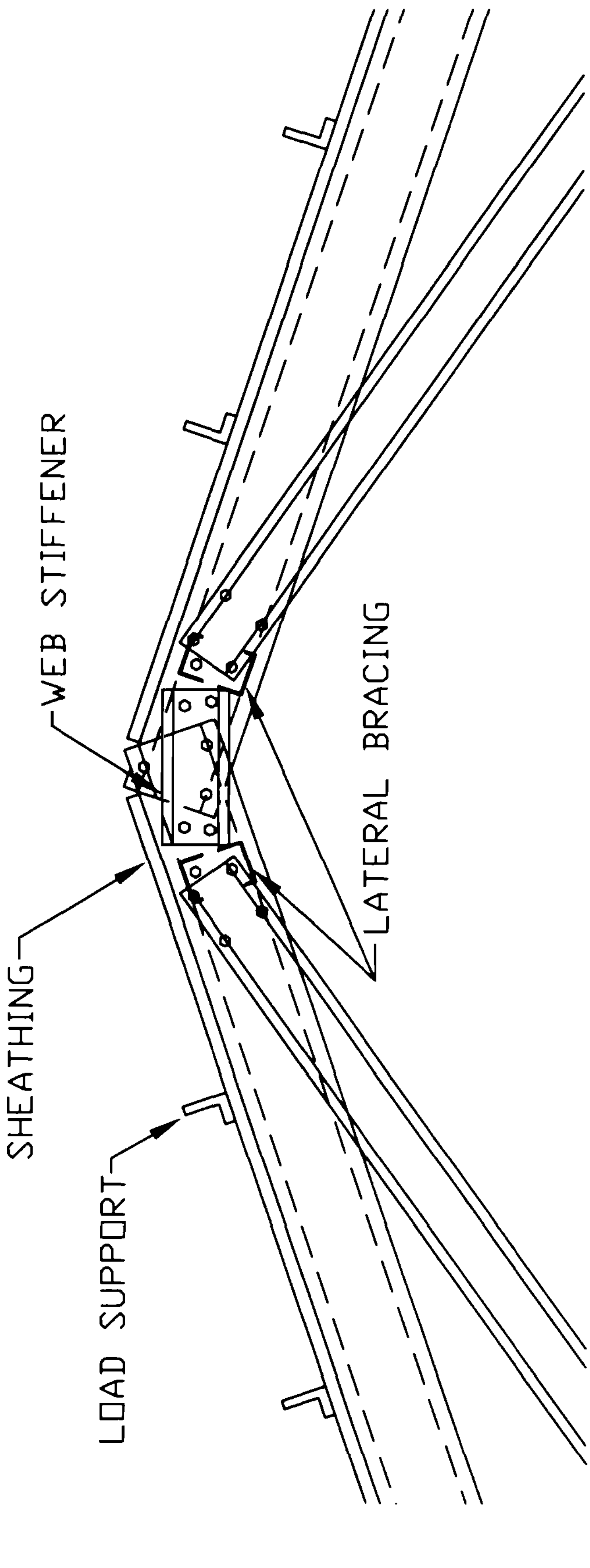
CENTERLINE - CENTERLINE JOINT COORDINATES
(AS USED FOR COMPUTER ANALYSIS)



HEEL CONNECTION CONFIGURATION




INTERMEDIATE CONNECTION
CONFIGURATIONS



RIDGE CONNECTION CONFIGURATION

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