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EVALUATION OF LONG SPAN COMPOSITE JOISTS

by

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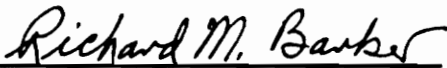
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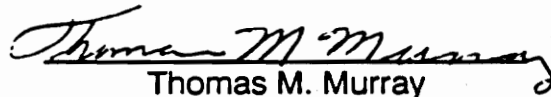
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**EVALUATION OF LONGSPAN
COMPOSITE JOISTS**

BY

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Civil Engineering

(ABSTRACT)

The strength and stiffness characteristics of longspan composite open-web steel joists are studied. Eight full scale single span composite joists were tested to failure to evaluate their performance in both the elastic and inelastic ranges. The test specimens' spans, depths, slab depth, and amount of shear connection was varied.

Analytical methods are presented to predict both the ultimate strength and the stiffness of the composite joist. A non-linear finite element model is presented to predict the behavior of the composite joist in the elastic and inelastic regions.

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CHAPTER I
INTRODUCTION

1.1 Background

This report presents the results of a research project conducted at Virginia Polytechnic Institute and State University (Virginia Tech) on composite long-span open-web steel joists. Composite long-span joists provide an economical way to obtain large, unsupported spans. The open web of the joist allows mechanical and electrical duct work to be run through the web. This allows for the lowering of floor to floor heights resulting in a savings of material. Also, after construction is completed, changes in the duct work can be easily made.

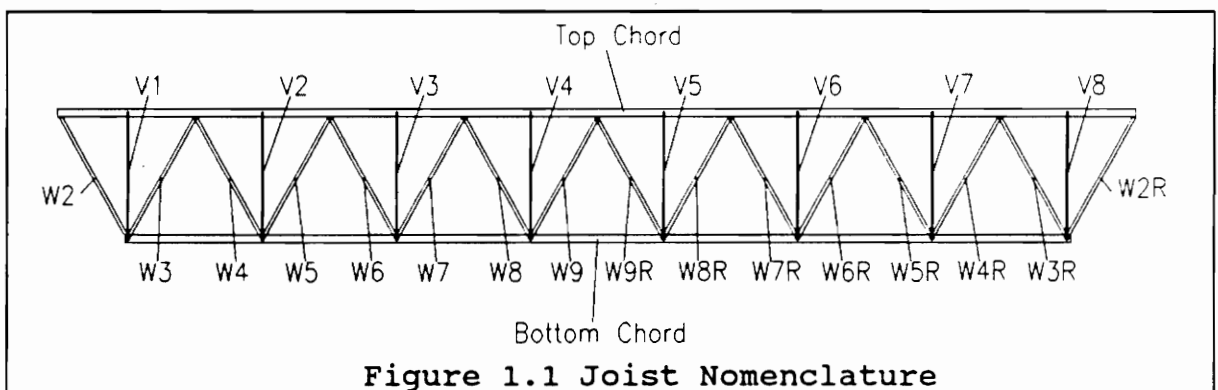
At this time (1990) there is no U.S. standard for the design of composite joists. This leaves the design engineer on his own in designing composite joists based on existing composite beam specifications. This study presents the results of eight full scale composite joist tests that were designed using a combination of the design provisions for non-composite joists and a composite design procedure that was developed in Canada.

1.2 Joist Nomenclature

The following nomenclature is used throughout this

report to reference the various members of the joist. Each web member of the joist is referred to by a particular number-letter combination. Diagonal web members are referred to with the letter "W" followed by a number. The number sequence starts with 2 at first diagonal web member near the end support. The diagonal web members are numbered sequentially from each end up to midspan. This numbering sequence means that all tension members are even numbered while all compression members are odd numbered. (See Figure 1.1) The diagonal web members on the side of midspan without the majority of the strain gages have a postscript of "R" added.

Vertical web members are denoted with the letter "V" and a number. The sequence starts at the end support with the majority of the strain gages and continues to the other end support.



1.3 Scope of Research

To analyze long-span composite joists, eight full scale test were performed at the Price's Fork Research Park and the Structures and Materials Research Laboratory at Virginia Tech. Test parameters that were varied include the span length, the depth of the joist, the configuration of the joist, the amount of composite connection provided, and the type of steel decking used.

The design method used for the joists is based on the following. The applied moment is assumed to be resisted by tension in the bottom chord and compression in the concrete to produce a couple. The top chord is assumed not to add to the composite moment strength of the section and is design only to resist the wet weight of the concrete. The web members are designed to resist all of the shear. The concrete slab is not considered to resist any shear. Shear connection is provided by headed shear studs.

Chapter II of this report reviews some of the literature dealing with composite joists and their components. The details of the individual tests are given in Chapter III. Chapter IV presents the results of the eight composite joist tests. These results are discussed in Chapter V. In Chapter VI a non-linear finite element model is presented which is used to model the observed behavior of the joists. A complete explanation of the methods used in designing the

joists is given in Chapter VII. Conclusions and recommendations are presented in Chapter VIII.

CHAPTER II
LITERATURE REVIEW

2.1 Introduction

The references for this project are grouped into two areas. The first deals with studies done on the various components of a composite joist with emphasis on shear stud strengths. (A detailed study of shear stud behavior and strength is being perform in a companion study. It will include a detailed literature review for shear studs.(Sublett 1991)) The second group considers studies of complete composite joists.

2.2 Research on Components

Much research has been done on the use of headed shear stud to provide composite action. Ollgaard et al. (1971) determined the shear strength for headed shear studs in normal and lightweight concrete. They preformed push out test on 48 specimens. From their data they concluded that:

$$Q_u = \frac{1}{2} A_s \sqrt{f'_c E_c} \leq A_s F_u \quad (2.1)$$

where Q_u = nominal shear stud strength, kips
 A_s = cross-sectional area of the shear stud, in²
 f'_c = concrete compressive strength, ksi
 E_c = modulus of elasticity for concrete, ksi
 F_u = tensile stress of the shear stud, ksi

Stud strength based on this equation was incorporated in the American Institute of Steel Construction (AISC) steel design specification in 1969. It remains in the current AISC specifications (Specifications 1989; Load 1986).

2.3 Research on Composite Joists

In an unpublished report by the Steel Joist Institute, a working stress design procedure for composite joists with 3/8 in. diameter shear studs is presented. (Composite Steel Joists 1971) The report suggest that minimum top chord thickness of 1/8 in. for 3/8 in. diameter headed shear studs be used. Stresses in the bottom chord and in the concrete are calculated by the elastic flexural formula. The composite section properties are calculated by using modular theory. This report is based directly on the conclusions drawn from research performed at Washington University (Tide and Galambos 1968, Nall et al. 1970) for which five composite joists were tested. The type and size of the top and bottom chords and the location and number of shear connectors were varied in testing. Web members were over-designed to prevent web failures.

Azmi tested six 50 ft. span composite joists and presented an ultimate strength design method for composite joists (Azmi 1972). The design method is based on three levels of shear connection, under-connected, balanced, and

over-connected. In the under-connected level, the shear strength of the studs is less than the tensile yield force of the bottom chord, which is defined as the area of the bottom chord multiplied by the steel yield stress. In the balanced case, the shear stud strength is equal to the tensile yield force in the bottom chord. In the over-connected case, the shear stud strength is greater than the tensile yield force in the bottom chord. These models provide good agreement to Azmi's test data.

Fahmy (1974) developed a finite difference method to analyze the behavior of composite joists in the elastic and inelastic regions. The test specimens that were analyzed were 50 ft. simply supported spans. Three different joist configurations were studied, which used puddle welds and shear studs to provide shear connection. The results from eight tests, two performed by Fahmy and six performed by Azmi (1972), were compared to the results of the finite difference method. There was good agreement between the model and test results.

Brattland and Kennedy (1986) studied the effect of concrete shrinkage on the behavior of composite joists. To this end they built two 38 ft. composite joists which were tested at 65 and 85 days. The specimens were tested to failure. They found that the majority of shrinkage occurred within the first 30 days. The failure loads obtained closely

match those that were predicted by an ultimate strength method with only the bottom chord in tension.

Leon and Currey (1987) discuss various design and analysis techniques used for composite joists. They compare test data from two 36 ft. test specimens to the different design methodologies. Of the methods studied, they concluded that ultimate strength methods for determining the moment capacity of a composite joist provided the best correspondence with the test data. Also, they noted that a ductile failure can be obtained if the joist is proportioned correctly, i.e. so that the bottom chord yields before the web members buckle.

A principal reference on composite joist design is Design and Construction of Composite Floor Systems (Chein and Ritchie 1984). Chapter five of this text deals with composite open-web steel joists and trusses. Design criteria are provided for strength, serviceability and connections based on ultimate strength methods. The methods presented by Chein and Ritchie were used in this report to calculate the strength and stiffness of the test specimens. See Appendix A for example calculations.

CHAPTER III

EXPERIMENTAL SETUP

3.1 General

To make an evaluation of long span composite joist behavior, eight full-scale tests were conducted. These tests were performed first at the Virginia Tech's Price's Fork Research Park and later at the newly constructed Structures and Materials Research Laboratory. Each test consisted of a single simply supported span, composed of a single steel joist, ribbed steel deck, headed shear connectors, and a concrete slab

Span lengths of 56 ft. and 40 ft. and joist depths of 36, 34, 20, 16, and 14 in. were used. Each steel joist was topped by a concrete slab placed on cold-formed ribbed steel decking. Composite action was obtained by welding shear studs through the steel deck to the joist's top chord.

The joists were fabricated entirely from hot-rolled angles. The top and bottom chords were double angles and the web members were either single- or double-angle sections. The joists were designed and fabricated by Vulcraft, a Division of Nucor Corporation, according to their design methodology, discussed in Chapter VI of this report.

A summary of the test parameters studied is presented in Table 3.1. The column title "Web" indicates the variations in the web members for the test; "Standard" indicates that the web members were not modified from the original design; "reinforced verticals" means that the vertical web members were replaced or reinforced to make them stronger; "30% over designed" denotes that the web members were designed to resist a load 30% greater than standard design.

Table 3.1
Test Matrix

Test	Span (ft)	Joist Depth (in)	# of Studs	Slab Depth (in)	Rib Height (in)	Slab Width (in)	Web
CLH-1	56	36	22	6	3	102	standard
CLH-2	56	36	38	6	3	102	reinforced verticals
CLH-3	40	16	66	6	3	81	standard
CLH-4	40	16	44	6	3	81	reinforced verticals
CLH-5	40	34	22	6	3	81	reinforced verticals
CLH-6	40	14	36	5	2	81	30% over designed
CLH-7	40	20	36	5	2	81	30% over designed
CLH-8	40	20	36	5	2	81	standard

3.2 Construction Methods and Details

The joists were designed and fabricated by Vulcraft and shipped to Virginia Tech for testing. Once at the appropriate testing location, the steel decking was placed on the joist. The decking was supported and shear studs were

then welded through the deck to the top chord of the joist, using automated stud welding equipment.

After the steel decking and studs were in place, cold-formed steel pour stop was placed around the edges of the decking. The size of pour stop used depended upon the rib height of the deck. For tests CLH-4 through CLH-8, the pour stop was attached so that it could be removed before the test began, so that the any formation of cracks in the concrete could be observed.

A series of single angles was used to brace each side of the decking while the concrete was placed and during curing. These angles were attached at one end to the edge of the decking and at the other end to the bottom chord of the joist. Approximately seven sets of the braces were placed along the joist in each test. These single angle braces were removed prior to testing the joist. This procedure was followed so that the joist would carry all of the coasting forces.

To keep the slab as level as possible, wooden braces were used to support the edge of the decking which prevented the deck from twisting. The wooden braces were removed after the wet concrete had been finished.

3.2.1 CLH-1 Details

CLH-1 was constructed at the Price's Fork Research Park. The joist had an overall length of 56 ft. and a depth of 36 in. The individual member sizes of the joist are listed in Table 3.2. The concrete slab had a nominal width of 102 in., a nominal depth of 6 in. and was placed on 3 in. ribbed metal deck. The specimen was tested in a specially designed, self-contained reaction frame.

Table 3.2
Nominal Member Sizes for CLH-1

Member	Size (in.)
Top Chord	2L-2.50x2.50x0.313
Bottom Chord	2L-3.50x3.50x0.313
Verticals	L-1.50x1.50x0.138
W2	2L-2.00x2.00x0.205
W3	2L-2.50x2.50x0.212
W4	2L-1.75x1.75x0.188
W5	2L-2.00x2.00x0.187
W6	L-2.00x2.00x0.205
W7	2L-1.50x2.50x0.145
W8	L-1.50x1.50x0.145
W9	2L-1.50x1.50x0.138

This specimen had a total of twenty-two 3/4 in. ϕ x 5 inch shear studs placed in an approximately uniform distribution. This number of shear studs provided a calculated shear stud strength, according to AISC specification (AISC 1986), that was slightly greater than the tension yield capacity of the bottom chord assuming a nominal yield force of 50 ksi. A true uniform distribution

of the shear studs could not be obtained since the number of ribs in the steel deck was not evenly divisible by the required number of shear studs.

3.2.2 CLH-2 Details

This test was very similar to CLH-1 and it was also constructed at Price's Fork Research Park. The joist used was a modified version of the joist used in CLH-1. The vertical web members were stiffened by welding 3/4 in. round bars to the top, middle, and bottom of the verticals. The member sizes are found in Table 3.3. The concrete slab was of the same dimensions as CLH-1, which was 102 inches wide, 6 inch deep and placed on 3 inch ribbed metal deck.

Table 3.3
Nominal Member Sizes For CLH-2

Member	Size (in.)
Top Chord	2L-2.50x2.50x0.313
Bottom Chord	2L-3.50x3.50x0.313
Verticals	L-1.50x1.50x0.138 + Rod-0.750 ϕ
W2	2L-2.00x2.00x0.205
W3	2L-2.50x2.50x0.212
W4	2L-1.75x1.75x0.188
W5	2L-2.00x2.00x0.187
W6	L-2.00x2.00x0.205
W7	2L-1.50x2.50x0.145
W8	L-1.50x1.50x0.145
W9	2L-1.50x1.50x0.138

A total of thirty-eight 3/4 in. ϕ x 5 in. shear studs were used in this test. This number of studs is provided a

shear stud capacity large enough to develop both the top and bottom chord in tension. Again, an approximately uniform distribution was used.

3.2.3 CLH-3 Details

The third test was conducted in the new Structures and Materials Research Facility. The test was performed on the structural reaction floor at the facility instead of in the self contained loading frame. The concrete slab width was reduced according to the dimensions of the reaction floor could be obtained.

Table 3.4
Nominal Member Sizes For CLH-3

Member	Size (in.)
Top Chord	2L-3.50x3.50x0.313
Bottom Chord	2L-5.00x5.00x0.438
Verticals	L-1.00x1.00x0.109
W2	2L-3.00x3.00x0.250
W3	2L-2.00x2.00x0.216
W4	2L-3.00x3.00x0.227
W5	2L-3.00x3.00x0.250
W6	2L-2.50x2.50x0.188
W7	2L-2.50x2.50x0.212
W8	2L-1.50x1.50x0.170
W9	2L-1.75x1.75x0.155
W10	2L-1.50x1.50x0.155

This test consisted of a 40 ft long joist with a depth of 16 in. The resulting span-to-depth ratio, 30, is at the upper limit of the Vulcraft's typical design span-to-depth ratio. The slab width for this test was reduced to 81 in.

The total slab thickness was 6 in. placed on 3 in. ribbed metal decking. A uniform distribution of sixty-six 3/4 in. ϕ x 5 in. shear studs was used. The member sizes are listed in Table 3.5

3.2.4 CLH-4 Details

This test had dimensions similar as to those of CLH-3. The center three vertical web members were removed and replaced with 1 in. square bars. The vertical web members at each end were reinforced with 1 in. diameter circular bars. These bars were welded at the top, middle, and bottom to the existing vertical members.

A total of forty-four 3/4 in. ϕ x 5 in. shear studs, enough to develop the bottom chord yield force, were attached in an approximately uniform distribution.

Table 3.5
Nominal Member Sizes for CLH-4

Member	Size (in.)
Top Chord	2L-3.50x3.50x0.313
Bottom Chord	2L-5.00x5.00x0.438
Verticals	SQ-1.00x1.00
W2	2L-3.00x3.00x0.250
W3	2L-2.00x2.00x0.216
W4	2L-3.00x3.00x0.227
W5	2L-3.00x3.00x0.250
W6	2L-2.50x2.50x0.188
W7	2L-2.50x2.50x0.212
W8	2L-1.50x1.50x0.170
W9	2L-1.75x1.75x0.155
W10	2L-1.50x1.50x0.155

3.2.5 CLH-5 Details

The joist for this test had of a span length of 40 ft. and a depth of 34 in. The span-to-depth ratio of 14.1 represents the lower limit of Vulcraft's typical design span-to-depth ratio.

The specimen had a nominal concrete slab width of 81 inch as in CLH-3 and CLH-4. Twenty-two 3/4 in. ϕ x 5 in. shear studs arranged in an approximately uniform configuration were used develop composite action. This number of shear studs was enough to develop the bottom chord yield force. Joist member sizes are tabulated in Table 3.6

Table 3.6
Nominal Member Sizes for CLH-5

Member	Size (in.)
Top Chord	2L-3.50x3.50x0.313
Bottom Chord	2L-3.50x3.50x0.313
Verticals	2L-1.25x1.25x0.133
W2	2L-2.00x2.00x0.250
W3	2L-2.50x2.50x0.212
W4	2L-2.00x2.00x0.187
W5	2L-2.00x2.00x0.187
W6	2L-1.50x1.50x0.170
W7	2L-2.00x2.00x0.216
W8	2L-1.50x1.50x0.170
W9	2L-2.0-0x2.000.205

3.2.6 CLH-6 Details

This test specimen (and the following tests specimens) was designed based on results from the first five tests. It

was decided to change the design for CLH-6 in three ways. First, the web members were designed to carry loads 30% greater than required by design calculations. Second, the joist configuration included no vertical web members. Third, no single angle web members were used. All web members were double angle members.

The joist had a depth of 14 in. and span of 40 ft. The slab had a total depth of 5 in. on a 2in. deep steel deck and a width of 81 in. To provide shear connection, thirty-six 3/4 in. ϕ x 5 in. shear studs, placed in a uniform distribution, were used. The dimensions of the joist members are given in Table 3.7.

Table 3.7
Member Sizes for CLH-6

Member	Size (in.)
Top Chord	2L-3.00x3.00x0.313
Bottom Chord	2L-4.00x4.00x0.438
W2	2L-2.50x2.50x0.250
W3	2L-2.00x2.00x0.187
W4	2L-2.00x2.00x0.163
W5	2L-2.00x2.00x0.187
W6	2L-2.00x2.00x0.163
W7	2L-2.00x2.00x0.187
W8	2L-1.50x1.50x0.155
W9	2L-1.75x1.75x0.155
W10	2L-1.50x1.50x0.143
W11	2L-1.50x1.50x0.170
W12	2L-1.25x1.25x0.109
W13	2L-1.25x1.25x0.133
W14	2L-1.00x1.00x0.109
W15	2L-1.25x1.25x0.109
W16	2L-1.00x1.00x0.109
W17	2L-1.25x1.25x0.109

3.2.7 CLH-7 Details

Like CLH-6 this test had web members designed to resist a load 30% greater than required by Vulcraft's design calculations. No vertical web members were used. The joist had a depth of 20 in. and a span of 40 ft. The concrete slab had a width of 81 in. and a depth of 5 in. and was placed on a 2 in. deep ribbed steel decking. Thirty-six 3/4 in. ϕ x 5 in. shear studs, enough to develop the bottom chord yield force, in an approximately uniform distribution were used to provide composite action.

Table 3.8
Nominal Member Sizes for CLH-7

Member	Size (in.)
Top Chord	2L-3.00x3.00x0.313
Bottom Chord	2L-4.00x4.00x0.438
W2	2L-2.50x2.50x0.250
W3	2L-2.00x2.00x0.187
W4	2L-2.00x2.00x0.163
W5	2L-2.00x2.00x0.187
W6	2L-2.00x2.00x0.163
W7	2L-2.00x2.00x0.187
W8	2L-1.50x1.50x0.155
W9	2L-1.75x1.75x0.155
W10	2L-1.50x1.50x0.143
W11	2L-1.50x1.50x0.170
W12	2L-1.25x1.25x0.109
W13	2L-1.25x1.25x0.133
W14	2L-1.00x1.00x0.109
W15	2L-1.25x1.25x0.109
W16	2L-1.00x1.00x0.109
W17	2L-1.25x1.25x0.109

3.2.8 CLH-8 Details

CLH-8 was identical to CLH-7 in all ways except one. The web members were not over designed. The depth, span, slab dimensions and stud arrangement were as in CLH-7. The member sizes are given in Table 3.9.

Table 3.9
Nominal Member Sizes for CLH-8

Member	Size (in.)
Top Chord	2L-3.00x3.00x0.313
Bottom Chord	2L-4.00x4.00x0.438
W2	2L-2.50x2.50x0.250
W3	2L-2.00x2.00x0.187
W4	2L-2.00x2.00x0.163
W5	2L-2.00x2.00x0.187
W6	2L-2.00x2.00x0.163
W7	2L-2.00x2.00x0.187
W8	2L-1.50x1.50x0.155
W9	2L-1.75x1.75x0.155
W10	2L-1.50x1.50x0.143
W11	2L-1.50x1.50x0.170
W12	2L-1.25x1.25x0.109
W13	2L-1.25x1.25x0.133
W14	2L-1.00x1.00x0.109
W15	2L-1.25x1.25x0.109
W16	2L-1.00x1.00x0.109
W17	2L-1.25x1.25x0.109

3.3 Instrumentation

Each set setup had a similar instrumentation scheme. Linear displacement transducers were used to determine the centerline and quarter point deflections of the top chord of the joist. In CLH-4 and subsequent tests, deflections of both the top and bottom chord were measured. Displacement

transducers were also used to determine the end rotation of the joist.

Slip between the concrete and joist was measured by dial gages at each end of the span. Slip along the span length was measured by displacement transducers. These transducers measured the movement between the joist and steel tabs inserted through a hole in the decking into the concrete at various locations along the span.

Strains at various locations along the bottom and top chord were measured in each test as well as several web members. Strain gages were used to determine the force in selected members of the joist. On members to be monitored, one gage was placed at the midwidth of each angle leg in the member. Double angle members had four strain gages on them, one on each leg of each angle.

The steel joists were painted with a white wash so that yielding in the joist could be observed. The white wash, which a mixture of lime and water adheres to the mill scale on the steel. When the steel yields the mill scale flakes off taking the white wash with it. Thus, the observance of white wash indicates locations where the steel has yielded.

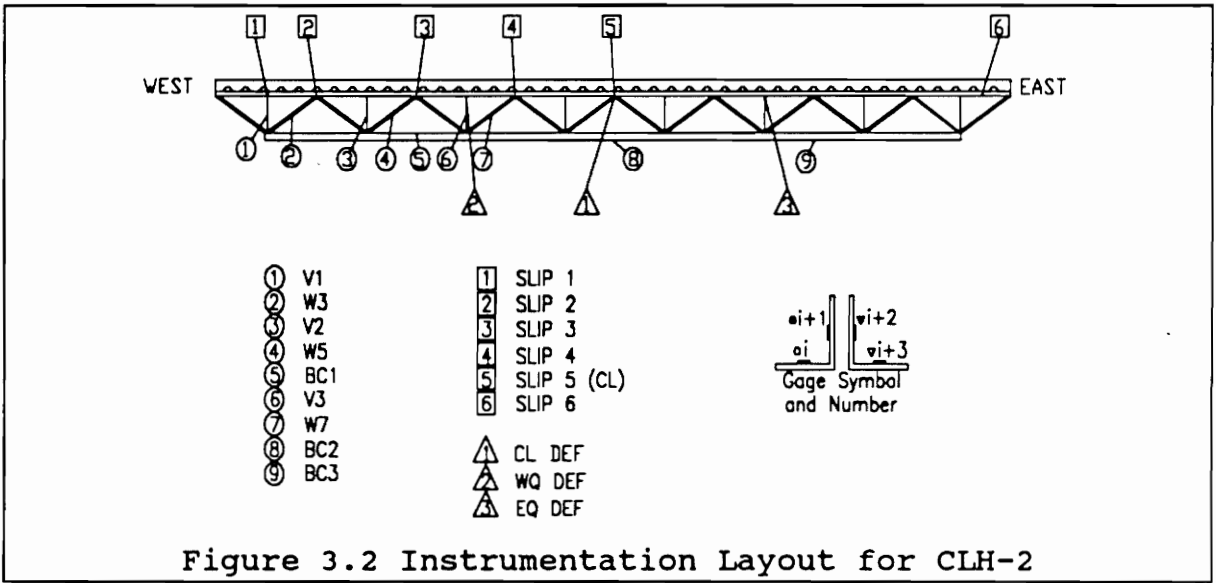
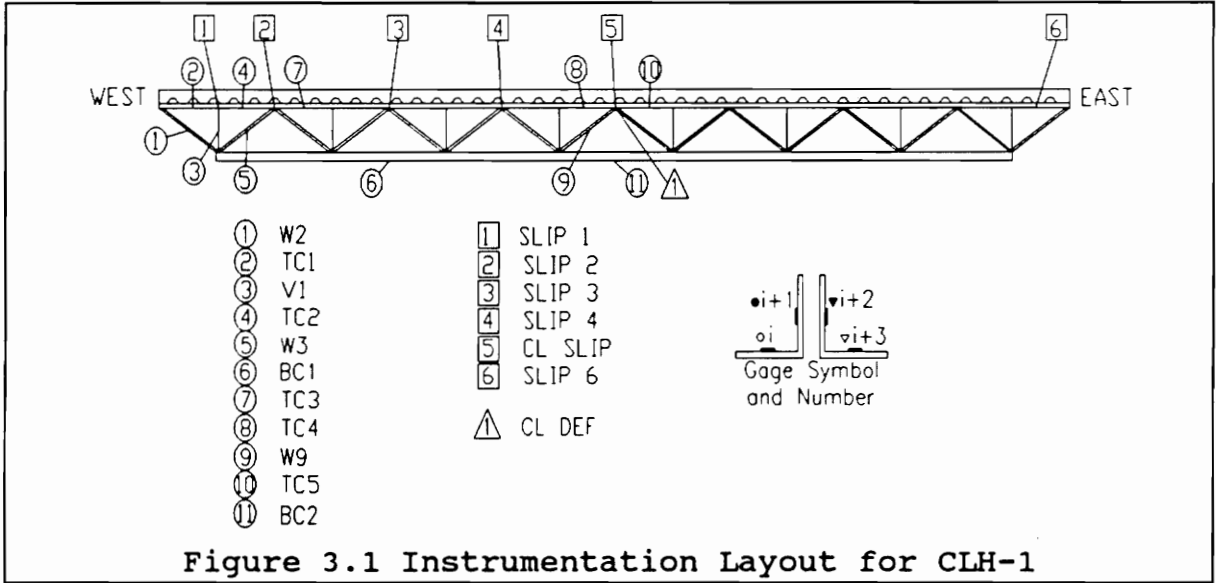
All data, except for the dial gage readings, was monitored, recorded, and reduced using a PC-based computer controlled data acquisition system.

3.3.1 Instrumentation for CLH-1

For test CLH-1, a total of 42 strain gages were used to measure the strains in various members, as shown in Figure 3.1. Seven displacement transducers were used to measure the vertical deflections of the test specimen as well as the reaction frame. Dial gages were used to determine the slip between the joist and the concrete slab at each end. Locations for the instrumentation are shown in Figure 3.1.

3.3.2 Instrumentation for CLH-2

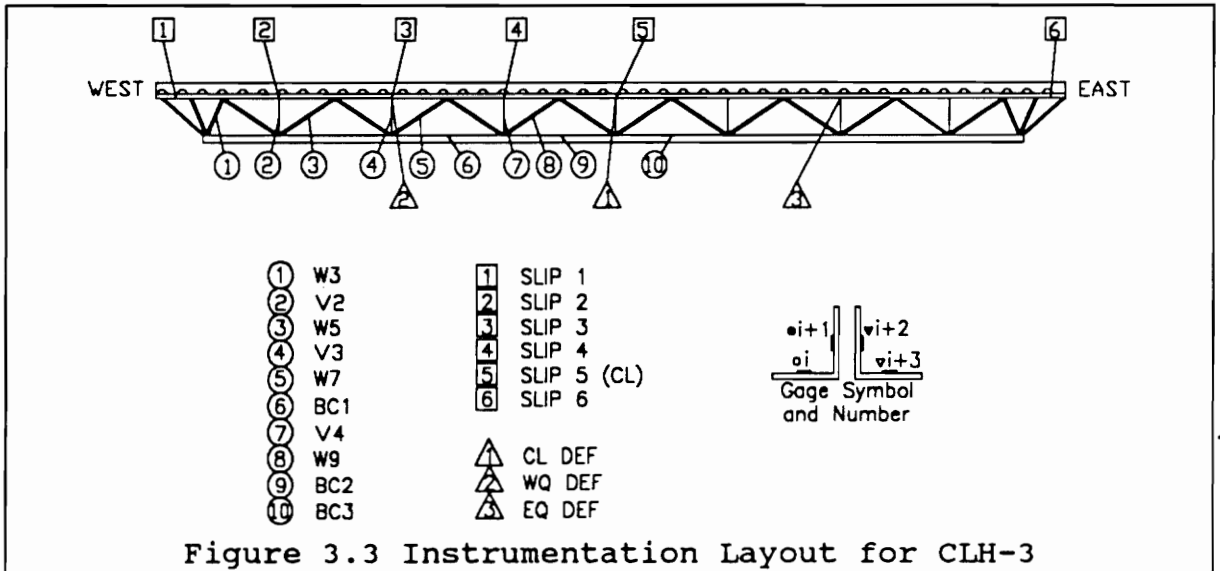
Test CLH-2 used 30 strain gages. The data collected in test CLH-1 was used to guide the placement of the gages. The deflections were measured at the quarter and center points with transducers. Slip between the concrete and the joist was also measured with dial gages placed at each end of the test specimen. Six transducers were used to measure the slip along the length of the specimen. See Figure 3.2. for instrumentation locations



3.3.3 Instrumentation for CLH-3

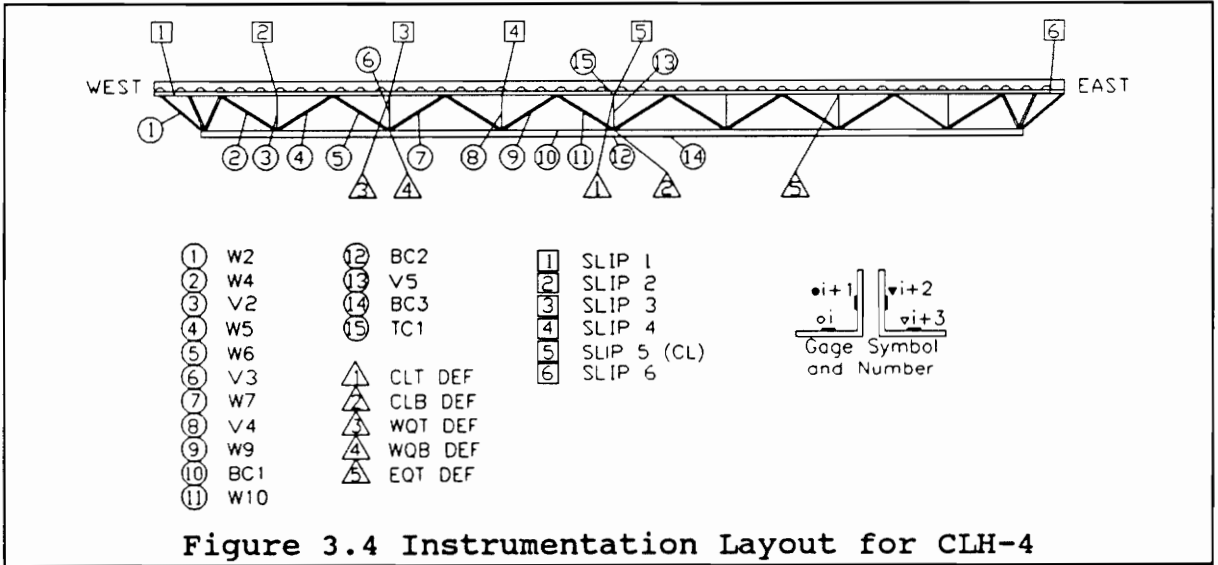
Strain gages were placed on the compression members and the bottom chord on half of the test specimen CLH-3 for a total of thirty-two strain gages on the specimen. Slip and

deflections were measured as in CLH-1 and CLH-2. The instrumentation layout is shown in Figure 3.3.



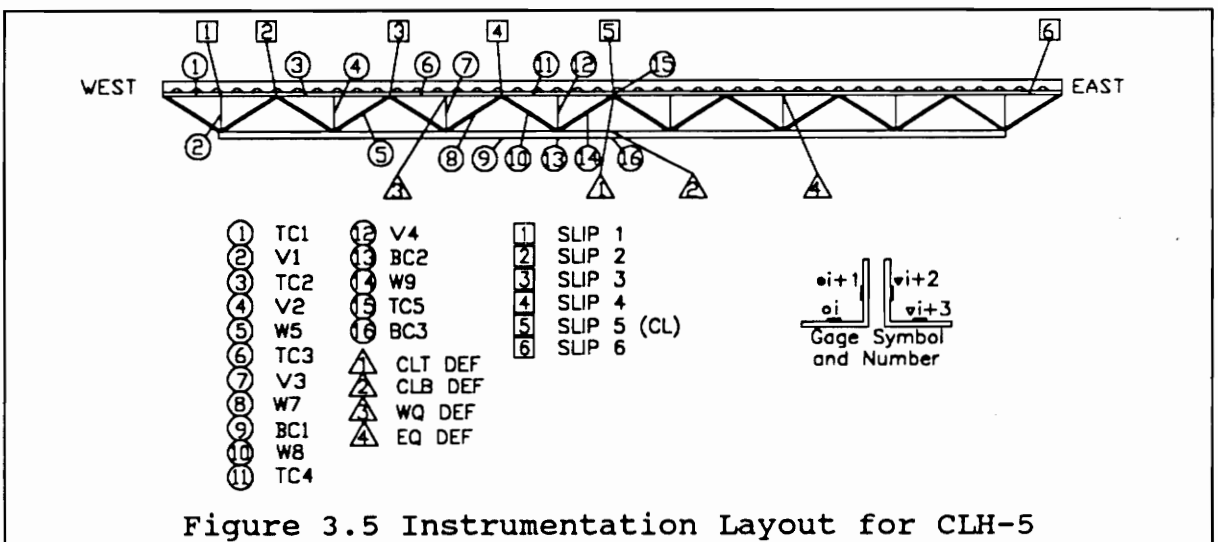
3.3.4 Instrumentation for CLH-4

For CLH-4, a total of 50 strain gages were placed using information gained from CLH-3. More gages were placed near the centerline of the specimen, as this is where the failure occurred in CLH-3. In addition to the normal slip and deflection measurements, the deflections of both the top and bottom chord was measured at the centerline and the west quarter point. Strains and deflections were measured during placement of the concrete. See Figure 3.4 for instrumentation locations.



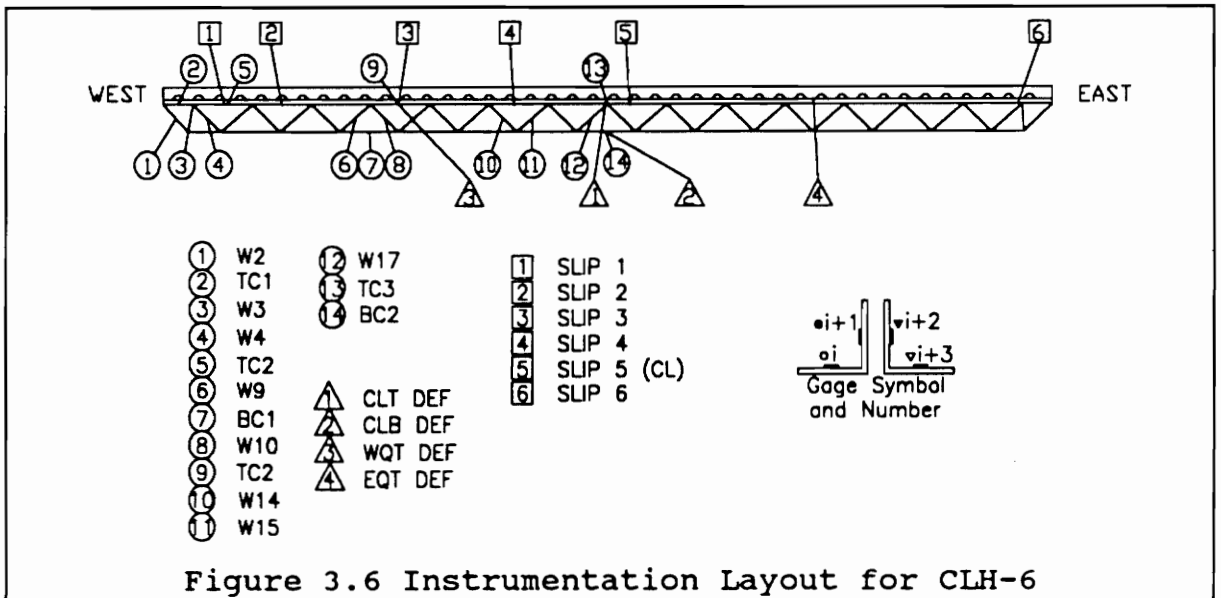
3.3.5 Instrumentation for CLH-5

A total of forty-four strain gages were used in the test CLH-5. The gages were placed along the top and bottom chords and on various compression members as shown in Figure 3.5. Strains, slips and deflections were measured during the placement of the concrete as in CLH-4.



3.3.6 Instrumentation for CLH-6

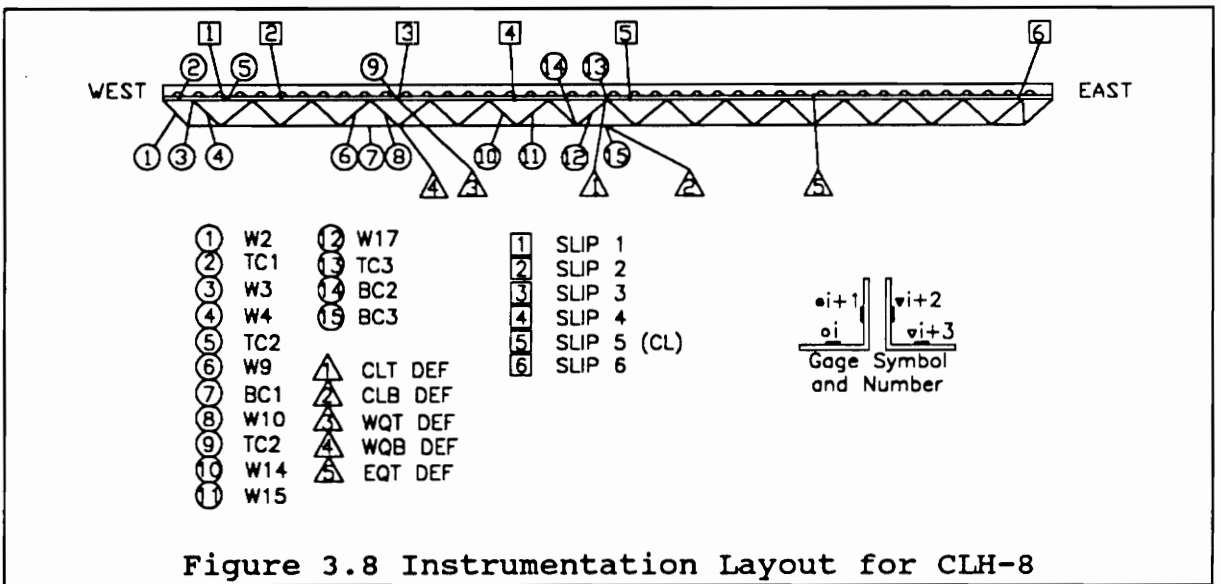
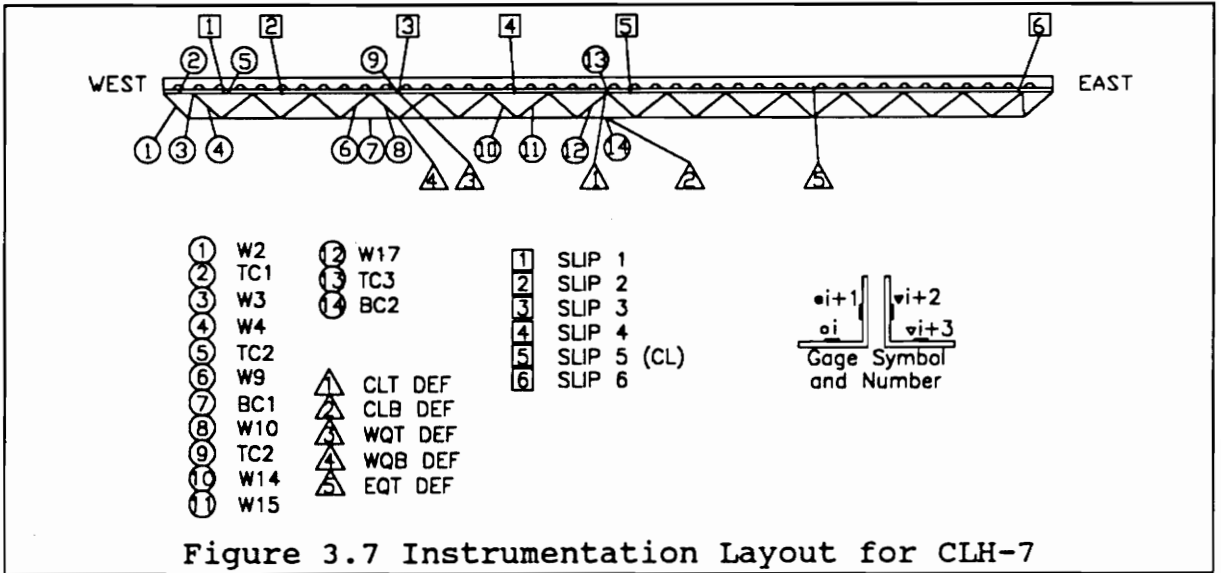
A total of 54 strain gages were placed on the joist top chord, web members, and bottom chord. The actual distribution is given in Figure 3.6. Deflections and slip were measured during the placement of concrete as in previous two tests.



3.3.7 Instrumentation for CLH-7

Strain gages were placed in similar positions as CLH-6. A total of 54 strain gages were used. Both top and bottom chord deflections were measured at the centerline and west quarter point. Slip between the concrete and the joist was measured with potentiometers and dial gages as usual. Strains and deflections were measured during placement of

the concrete. The instrumentation layout is shown in Figure 3.7



3.3.8 Instrumentation for CLH-8

A total of fifty-six strain gages were used. Fifty-four were placed in the same locations as in CLH-7. Two extra gages were added on the bottom chord at the panel point near the centerline. This location is where yielding in the bottom chord was first observed in previous tests. The locations of the strain gages are given Figure 3.8. Strains and deflections were measured during the placement of the concrete.

3.4 Loading Apparatus

The loading pattern for each test consisted of eight equally spaced point loads along the longitudinal centerline of the test specimen. This arraignment gave a close approximation to a uniformly distributed loading for shear, moment and deflection.

The load was applied by a single hydraulic ram and distributed into eight point loads by a three tiered binary loading pyramid (see Figure 3.9). This pyramid consisted of a W27x94 on the top tier, two W12x53s in the middle tier, and four W6x25s on the bottom tier. Each beam in the load chain rested on two 2 in. diameter rollers. The hydraulic ram applied load at the midspan of the W27x94, which in turn applied a load, through its end reactions, to the midspan of

each W12x53s on the second tier. The W12x53s, in turn, applied load to the midspan of each W6x25 on the bottom tier. The end reactions of the four W6x25s provided the eight point loads along the centerline of the specimen. The load chain was stabilized by lateral braces. These braces allowed the beams in the load chain to move vertically but prevented any lateral motion. The lateral braces also aided in adjusting the load chain so that it was centered vertically over the joist.

The W27x94 was laterally braced at the quarter points and midspan of the specimen. The W12x53s were braced at their midspan, which corresponded to the quarter points of the specimen. Sections of angle were welded to the flanges of the W27x94 and the W12x53s to add to the stability of the load chain. Sections of angle were also bolted to the concrete slab to laterally brace the bottom of the load chain. These angles helped align the loading beams with each other and hold the rollers in place.

3.5 Testing Procedure

Load was applied in each test in a similar sequence. Initially, the test specimens were loaded to 15 kips in 5 kip increments and then unloaded. This was done to ensure that the load chain, hydraulics, and instrumentation were all working correctly. After this initial loading, the

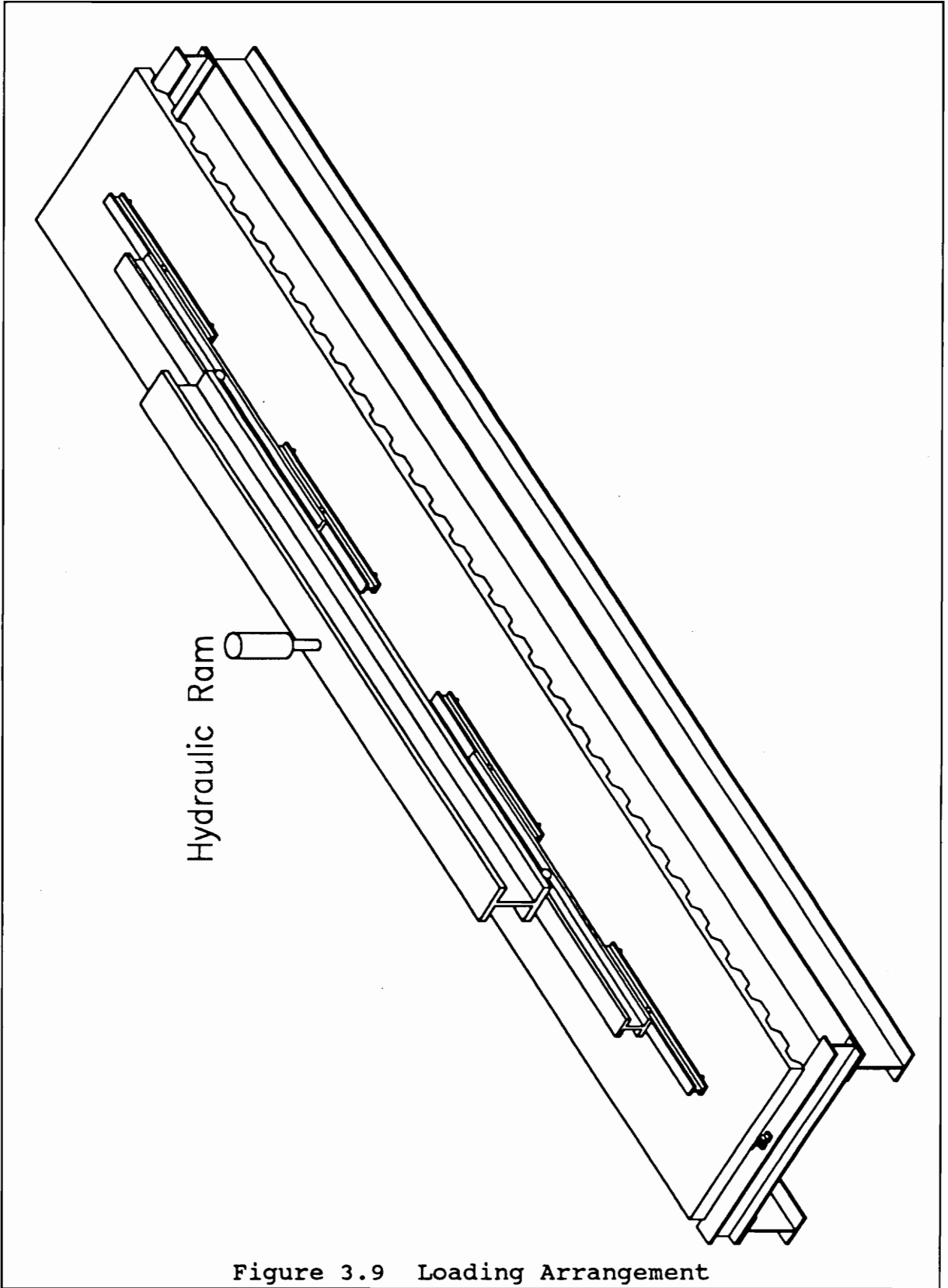
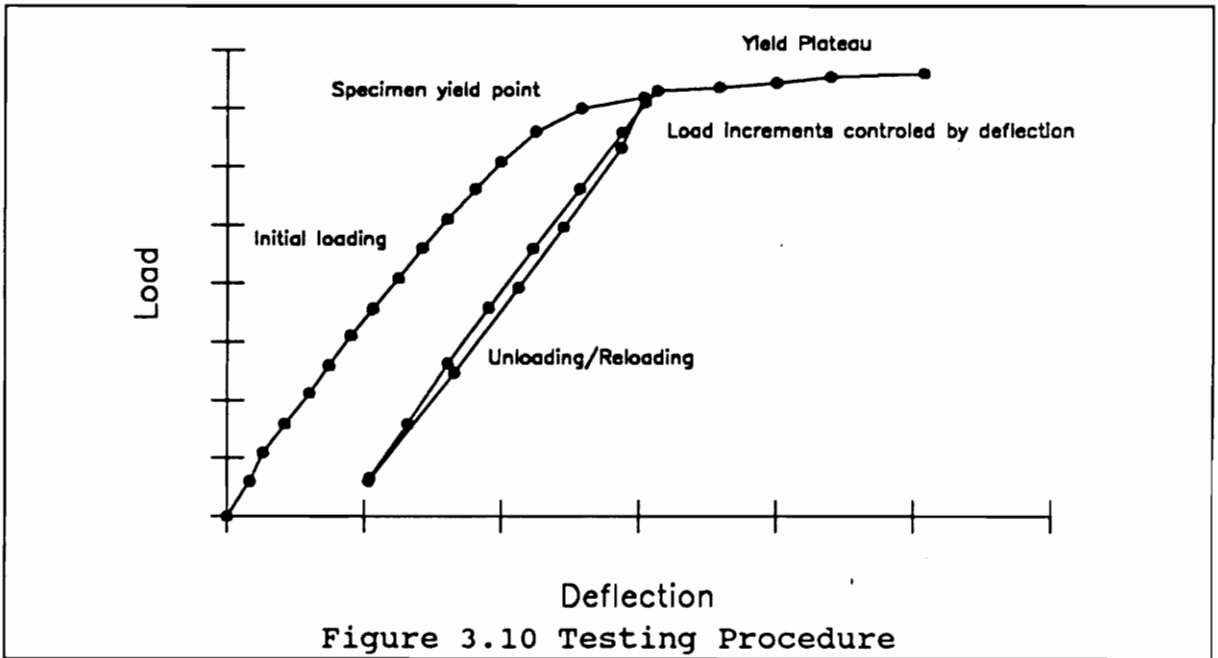


Figure 3.9 Loading Arrangement

specimen was then loaded again in 5 kip increments until initial yielding of the joist occurred. This was evident by a change in slope of the load vs. centerline deflection curve. At this point the specimen was again unloaded. Load was then reapplied to the specimen until the initial yielding load was again reach. At this stage in the test, further load increments were then applied based on centerline displacement control. (see Figure 3.10)



In test CLH-6 through CLH-8 the above procedure was slightly modified. The test specimens were loaded to their design load and then unloaded. Then, the specimens were loaded to their ultimate load, assuming nominal section and material properties, and then unload. After these two

loading cycles the test proceeded as described above. Both of these unloading cycles were in linear portion of the load-deflection curves for tests CLH-6 through CLH-8.

3.6 Supplemental Tests

Standard concrete cylinders were used to determine the compressive strength of the concrete. Two sizes of cylinders were used. In tests CLH-1 through CLH-4, 6 in. ϕ cylinders were used. 4 in. ϕ cylinders were used for CLH-5 through CLH-8.

The steel coupons tested at Virginia Tech were fabricated according to ASTM standard A370-88a. In the steel coupon tests performed by Vulcraft, the entire angle cross-section was used.

CHAPTER IV

TEST RESULTS

4.1 General

All the data collected during each test was reduced and compiled from a common reference point. The dead weight of the load chain was added to the load provided by the hydraulic ram to generate the total applied load. A value of 0 kips applied load indicates a fully composite section subjected only to its own dead weight. For tests where the strains were measured while the concrete was being placed, the loads would change from a negative value (the wet weight of the concrete). Test summaries are included in Appendix B. Each test has a summary sheet describing the test and the test results followed by plots of the deflection, slip and strain gage data. The plots were generated comparing the total applied load to the strains and deflections of the test specimens.

Theoretical deflection calculations were determined using a modified stiffness method as suggested by Chein and Ritchie (1984). The method accounts for slip of the shear connectors and deformation of the joist web. The predicted ultimate strength of the specimen was calculated by determining the maximum moment that could be resisted by the couple formed by tension in the bottom chord and compression

in the concrete slab. See Appendix A for sample calculations.

Strength characteristics for various concrete and joist members are listed in Table 4.1 and Table 4.2. Table 4.1 shows the concrete compressive strength of the cylinder at the time of testing. Table 4.2 shows the yield and ultimate stresses for several steel members from all tests performed.

Table 4.1
Concrete Compressive Strength Data

Test	f'c (psi)
CLH-1	4430
CLH-2	4330
CLH-3	4000
CLH-4	3110
CLH-5	5860
CLH-6	4430
CLH-7	5720
CLH-8	5380

4.2 CLH-1 Test Results

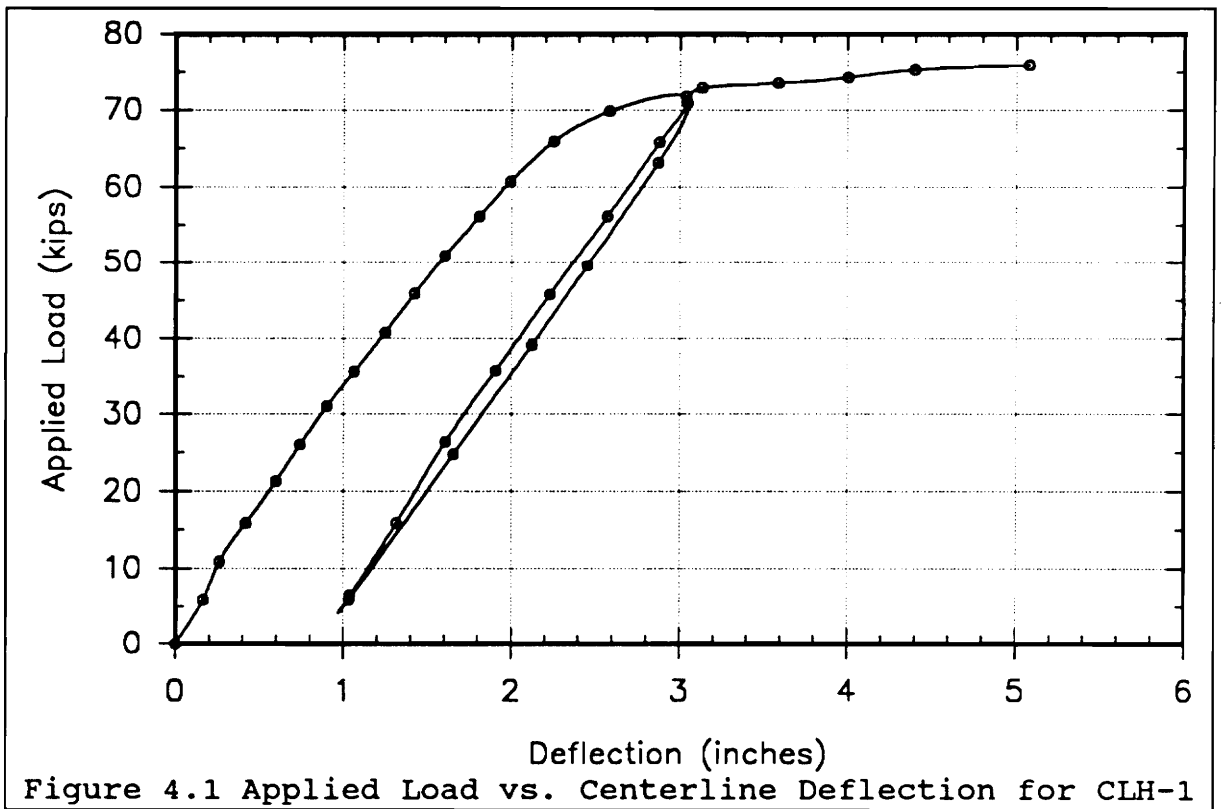
CLH-1 was conducted at the Price's Fork Research Park in a self-contained reaction frame. The test was conducted using the procedure presented in Section 3.5.

The specimen behaved linearly until an applied load of approximately 69 kips. The deflections in the linear range corresponded closely to the calculated theoretical values. Beyond the elastic range, the specimen deflected along a yield plateau (see Figure 4.1)

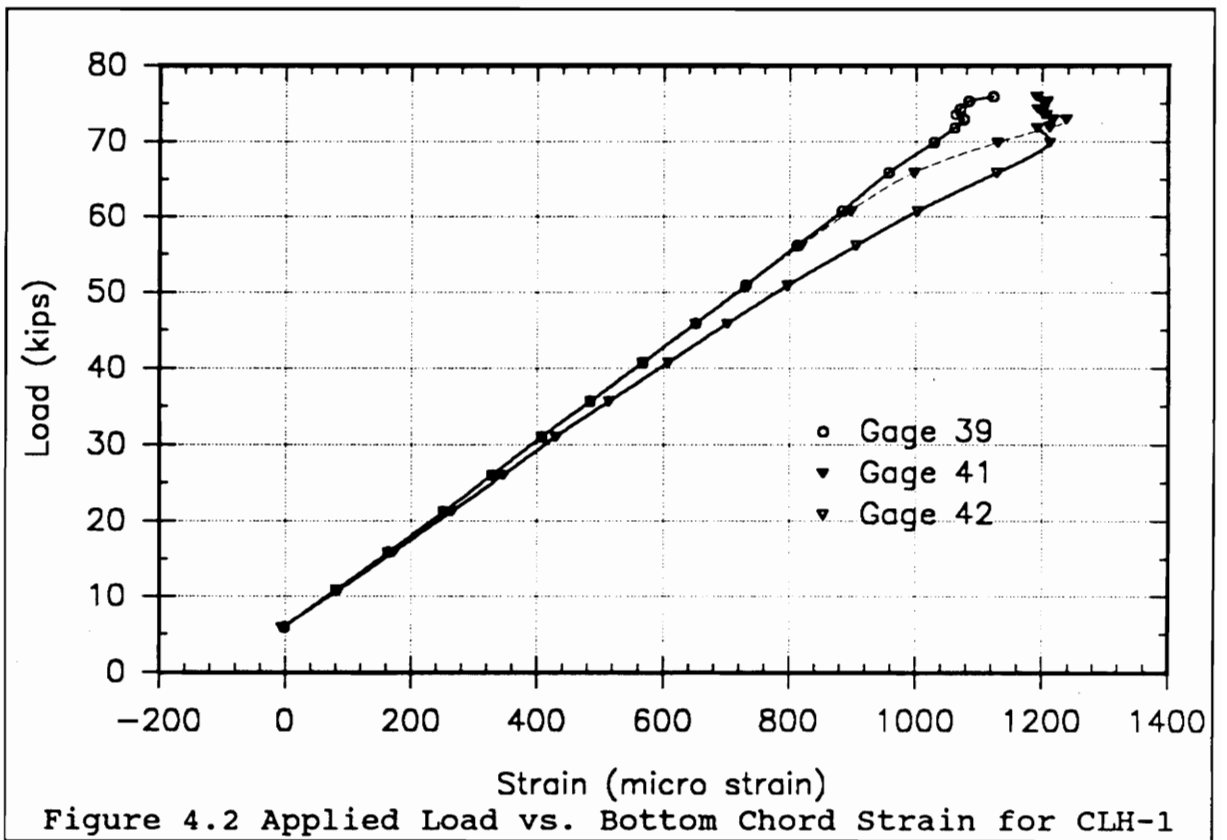
Table 4.2
Steel Coupon Strength Data

Specimen	Test Location	Fy (psi)	Fu (psi)
CLH-1-BC	VPI	53730	80530
CLH-1-BC	VULCRAFT	56430	79910
CLH-1-TC	VULCRAFT	56990	88750
CLH-1-W2	VULCRAFT	56170	80980
CLH-1-W3	VULCRAFT	60890	86110
CLH-1-W4	VULCRAFT	57950	78330
CLH-1-W5	VULCRAFT	57220	79240
CLH-1-W7	VULCRAFT	56280	83820
CLH-1-V1	VULCRAFT	56710	78230
CLH-2-BC	VPI	57320	80750
CLH-2-BC	VULCRAFT	57810	81800
CLH-2-TC	VULCRAFT	----	97340
CLH-2-W2	VULCRAFT	5681	80590
CLH-2-W3	VULCRAFT	61180	85030
CLH-2-W4	VULCRAFT	60670	83790
CLH-2-W5	VULCRAFT	57220	78260
CLH-2-W7	VULCRAFT	58210	82610
CLH-3-BC	VPI	54140	80900
CLH-3-BC	VULCRAFT	51200	78090
CLH-3-BC	VULCRAFT	50990	77670
CLH-4-BC	VPI	62110	89950
CLH-4-BC	VULCRAFT	55250	85590
CLH-4-BC	VULCRAFT	56330	87250
CLH-4-TC	VPI	55240	81240
CLH-4-TC	VULCRAFT	56330	80230
CLH-4-W4	VPI	57590	81900
CLH-4-W5	VPI	58090	81080
CLH-4-W6	VPI	57720	80760
CLH-4-W7	VPI	57110	86920
CLH-4-W8	VPI	58560	83640
CLH-5-BC	VPI	60450	88080
CLH-5-BC	VULCRAFT	59290	80600
CLH-5-TC	VPI	59290	81990
CLH-5-W2	VPI	60120	86700
CLH-5-W3	VPI	59640	88000
CLH-5-W4	VPI	60450	82720
CLH-5-W4	VULCRAFT	57920	78680
CLH-5-W6	VPI	56960	83680
CLH-5-W7	VPI	56360	88990
CLH-5-W9	VPI	54730	88640
CLH-5-V1	VPI	63270	93210
CLH-6-BC	VULCRAFT	56160	80160
CLH-7-BC	VULCRAFT	57140	82130
CLH-8-BC	VULCRAFT	56310	81140

The centerline bottom chord strains also behaved linearly elastically up to the specimen's yield load. The yield load is defined as the intersection of a tangent to the load deflection curve along the initial linear portion and a tangent along the yield plateau. After the yield load, however, the strain in the bottom chord at midspan did not increase under increasing load. (See Figure 4.2) Other members which were strain gaged showed a change in behavior at the specimens yield load. They were linearly up to the specimen's yield load. However, after the yield load, they displayed a higher rate of strain for a given load increment.



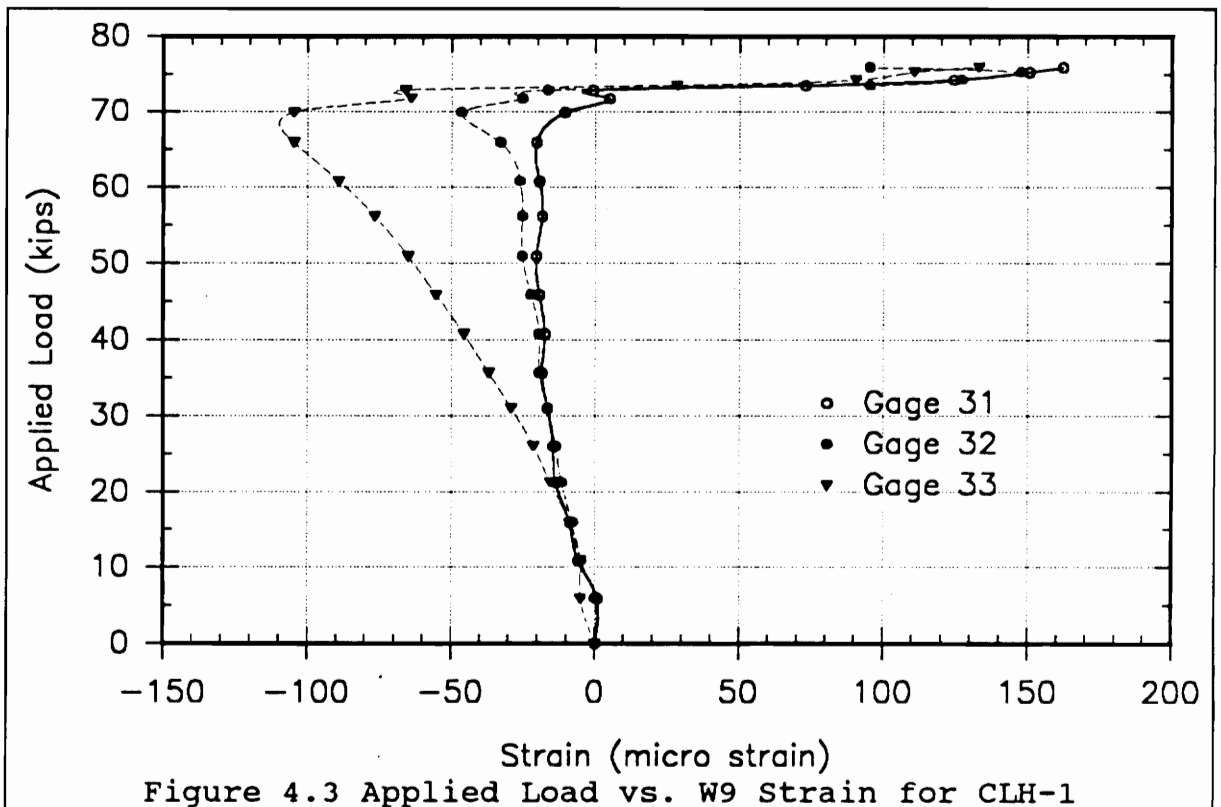
This can be seen in Figure 4.3. In a standard truss analysis member W9 is a compression member. The strain gages show that the member is in compression until the specimen's yield load. After this point however, member W9 moves into tension.



Visual observations made during the test agree with the strain gage information. Yield lines first appeared in the white wash around the panel points near the centerline on the bottom chord. The yielding expanded away from the panel

points but did not reach the strain gages placed mid-way between the panel points (shown in Figure 4.2).

The specimen in CLH-1 failed at 75.9 kips of applied load and at 5.07 in. of midspan deflection. At this load member V5, a vertical web member just to the west of the midspan, buckled causing member W9R is also buckle. The failure of the web caused the top chord to fail.



Three transverse cracks formed in the concrete at failure. These cracks were caused by sudden changes in deflection of the top chord. They formed over locations where the top chord failed.

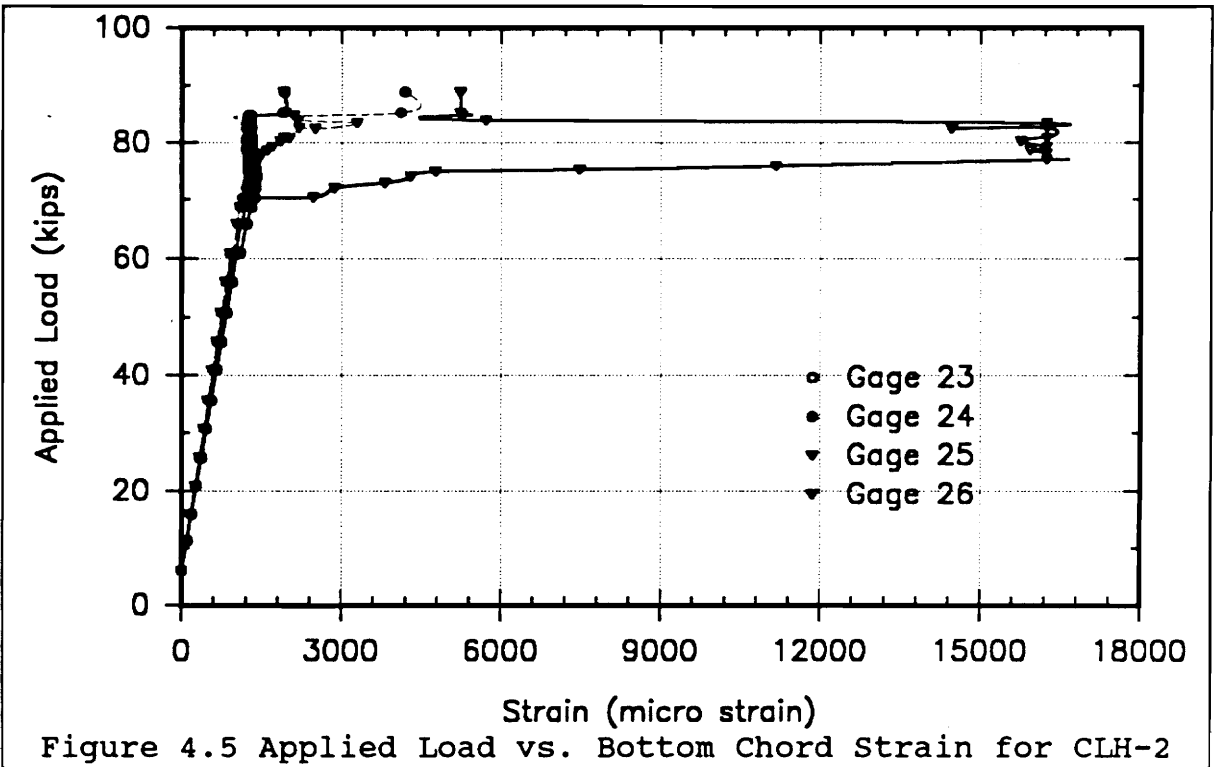
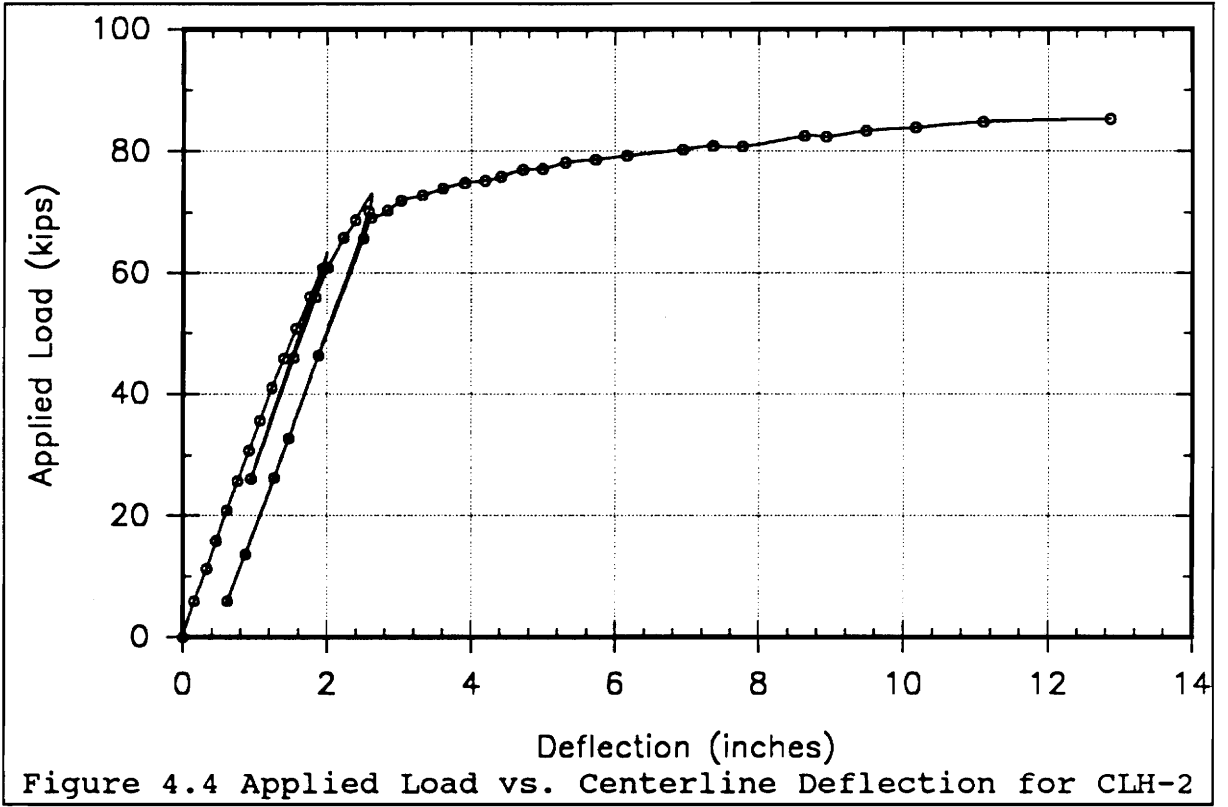
4.3 CLH-2 Test Results

CLH-2 had dimensions similar to CLH-1. The difference being that the vertical members were stiffened by 3/4 in. diameter rods. The number of shear studs was also increased from 22 to 38 for this test. This number is based on developing both the top and bottom chord yield strength.

CLH-2 exhibited behavior similar to CLH-1. The specimens midspan deflection and member strains all show a close correspondence to CLH-1. The strains in the bottom chord, as seen in Figure 4.5, showed similar behavior to those in the first test. However, gages 25 and 26 indicate that the yielding had begun to occur at the centerline of the bottom chord at 70.4 kips total applied load.

The added steel rods prevented the joist's vertical members from buckling at the load they did in CLH-1. This was very apparent during the test. The 90 degree angle between the legs of the single angle portion of several vertical web members widen significantly and would have buckled if they had not been reinforced by the rods.

Since the vertical web members were restrained from failing, the specimen was able to sustain a greater centerline deflection. At an applied load of 85.4 kips, the specimen deflected 12.25 in. At this deflection the test was



stopped since the bottom chord was touching the bottom of the testing frame. No cracking of the concrete was observed during the test.

4.4 CLH-3 Test Results

CLH-3 was tested in the Structures and Materials Research Facility. This provided access to an overhead crane, enabling the strains caused by the weight of the loading frame to be easily measured.

Although the joist in CLH-3 had a different span to depth ratio than in previous tests, the members exhibited similar behavior. The specimen load deflection curve is shown in Figure 4.6. During this test the specimen was loaded and unloaded several times. These portions of the load-deflection plot show the same effective stiffness as the initial loading.

In the elastic range of the specimen the members behaved linearly elastic. Upon reaching the yield load of the specimen, the members displayed either constant strain or a rapidly increasing rate of strain. The strains in the bottom chord are shown in Figure 4.7. In Figure 4.8 the strains of member V4 are shown. After reaching the specimen yield load, the strains in V4 show a dramatic increase.

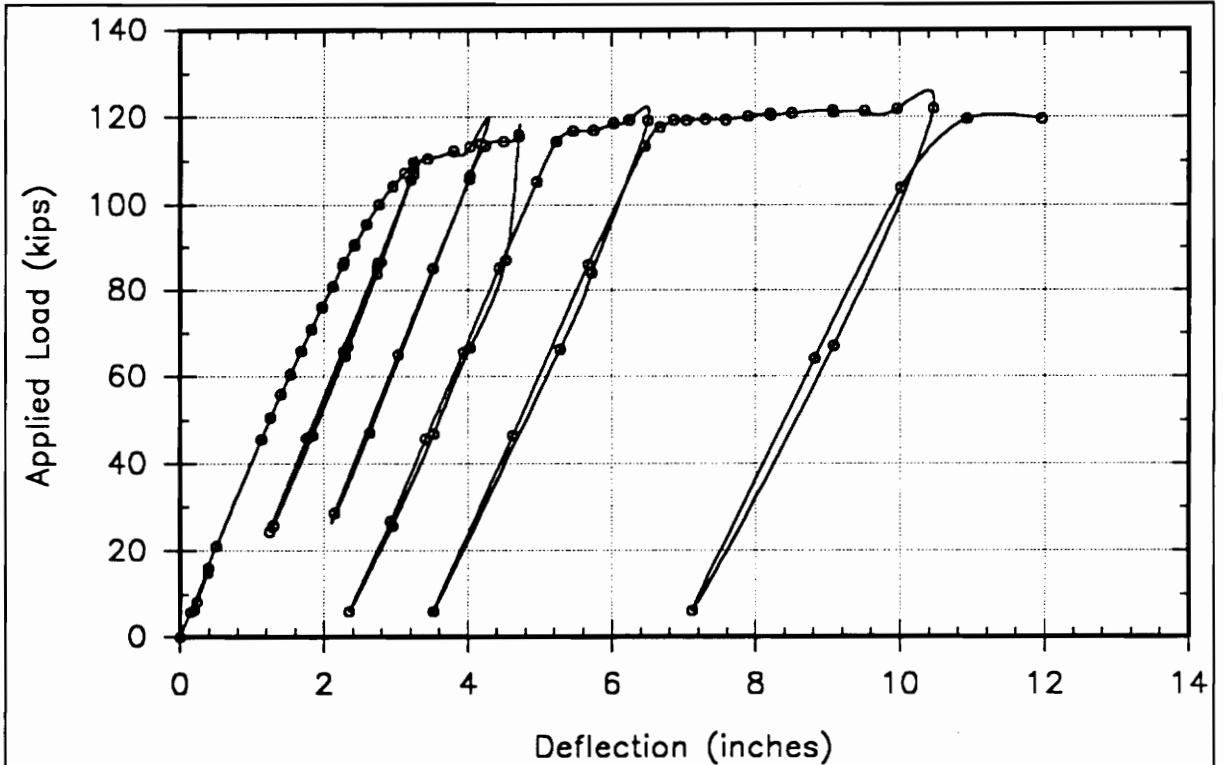


Figure 4.6 Applied Load vs. Centerline Deflection for CLH-3

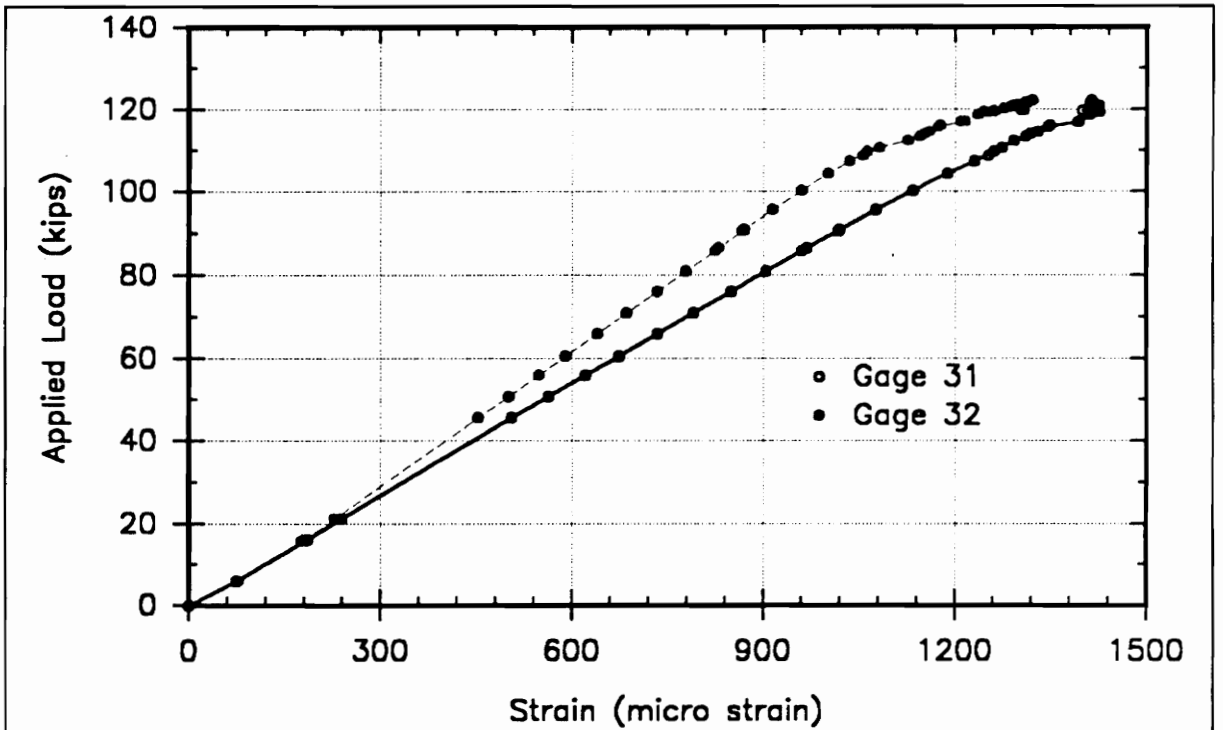
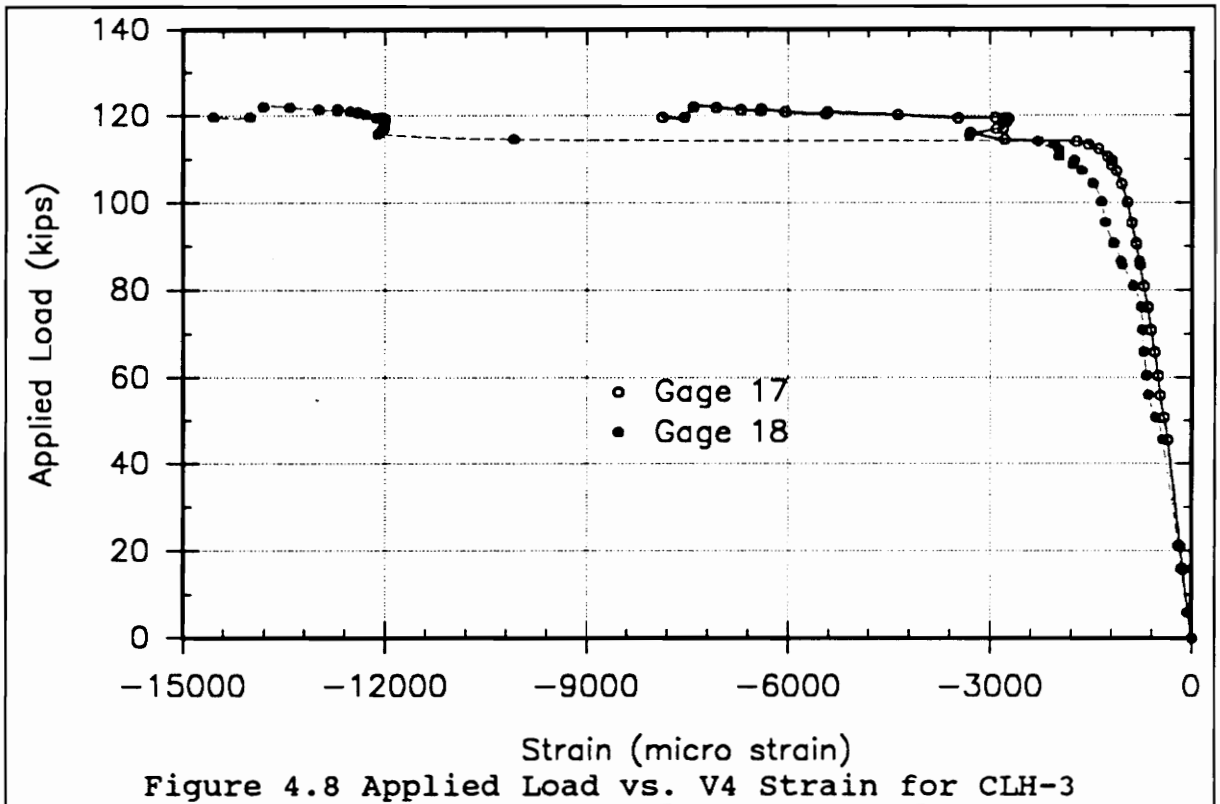


Figure 4.7 Applied Load vs. Bottom Chord Strain for CLH-3

Failure was caused by the top chord moving toward the bottom chord, significantly altering the depth of the joist. This relative motion caused the buckling of the center vertical and the two center diagonal web members, W10 and W10R. In a truss these members are tension members. The buckling of the web members caused the concrete to crack at mid-span.



4.5 CLH-4 Test Results

CLH-4 was similar in configuration to CLH-3. However, the three center verticals were replaced with 1 in. square

bars and members V1 and V8 were reinforced with 1 in. diameter rods welded at the top, middle, and bottom. The number of shear studs was reduced from 66 to 44.

Failure was caused by the local buckling of the top chord between members V6 and W6R. Both angles of the top chord rotated and buckled locally at the locations of shear studs. Since the studs were welded on alternating top chord angles, the two different top chord angles did not buckle in the exact same location. The locations were offset by one rib width of the decking.

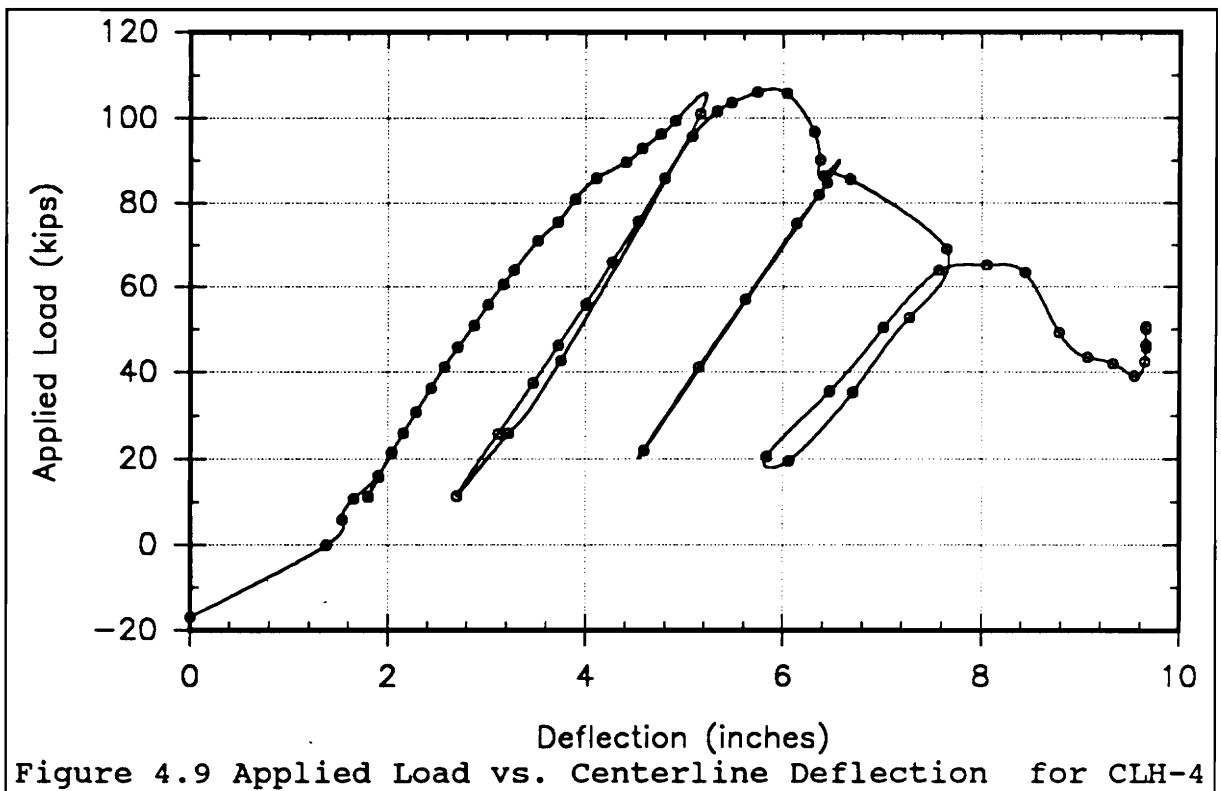
The failure of the specimen culminated in the shearing of the shear studs from the buckled section of the top chord to the end of the span. This brought about a loss of composite action between the joist and the concrete slab. Examination of the failed shear studs showed that the studs tore away from the joist, leaving a cupped hole in the top chord.

The load in the top chord at failure was a combination of axial load and moment. Both of these actions were applied by the shear studs. This was readily apparent since the failures occurred at the exact location of shear studs.

This was the only test in this series that did not reach a yield plateau. Figure 4.9 shows the test specimens load-deflection curve. As soon as the top chord began to fail, the specimen was unable to maintain its load carrying

capacity. Examining the loading cycles, the loss of composite action can be seen in the change in the effective stiffness of the test specimen.

The top chord midspan strains, Figure 4.10, show the loss of composite action at failure. After the initial compressive strain caused by the casting of the concrete slab the strains remain relatively constant until loss of composite action where the strains increase in compression. The strains in the bottom chord remain linear through the test. The bottom chord strains are shown in Figure 4.11.



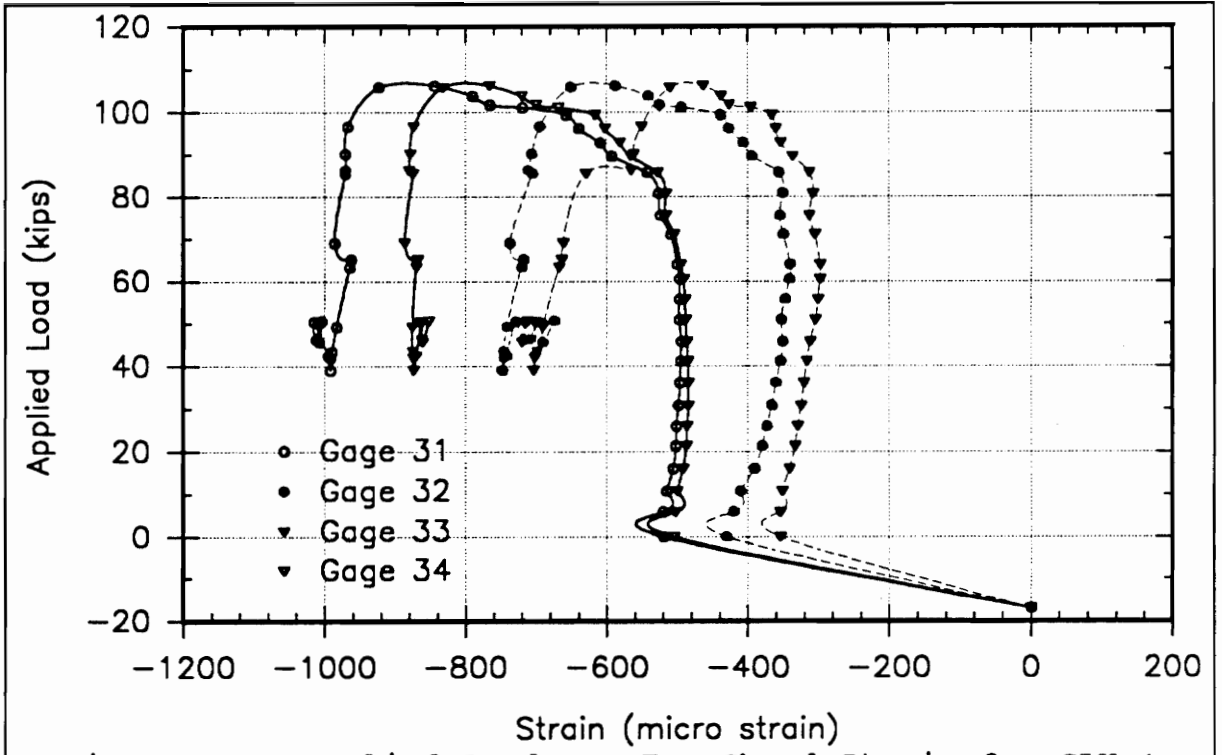


Figure 4.10 Applied Load vs. Top Chord Strain for CLH-4

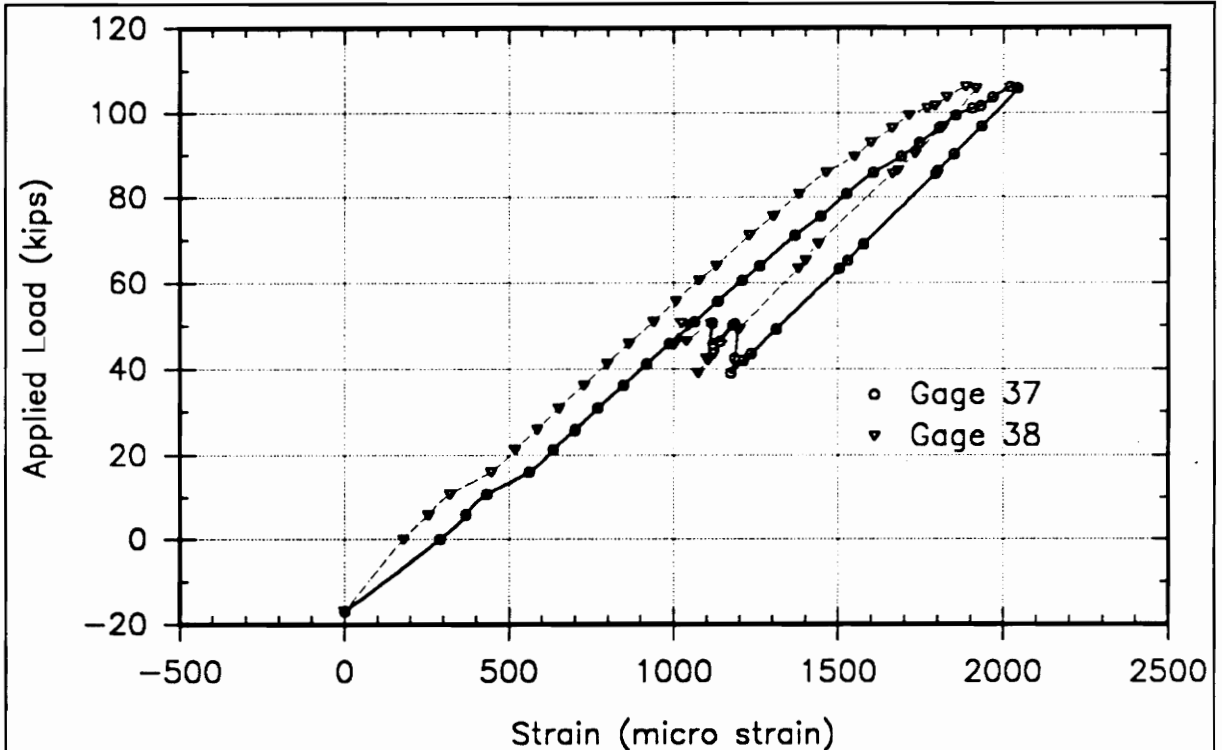


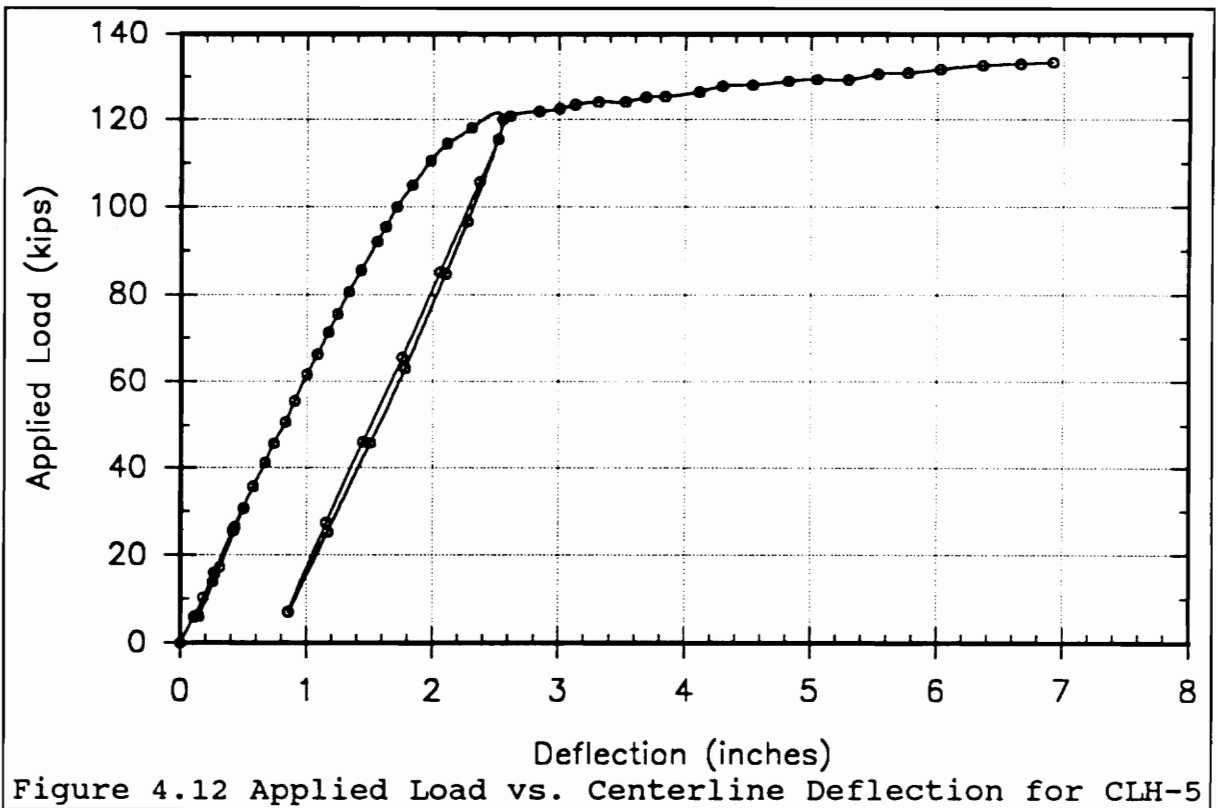
Figure 4.11 Applied Load vs. Bottom Chord Strain for CLH-4

4.6 CLH-5 Test Results

CLH-5 failed at an applied load of 133.5 kips and a centerline deflection of 7.2 in.. Failure was caused by the buckling of member W5R. The failure of this member also caused V3 to buckle.

Noting the failures that occurred on unreinforced joists (CLH-1 and CLH-3), the single angle vertical web members were replaced by double angle members before any load was applied.

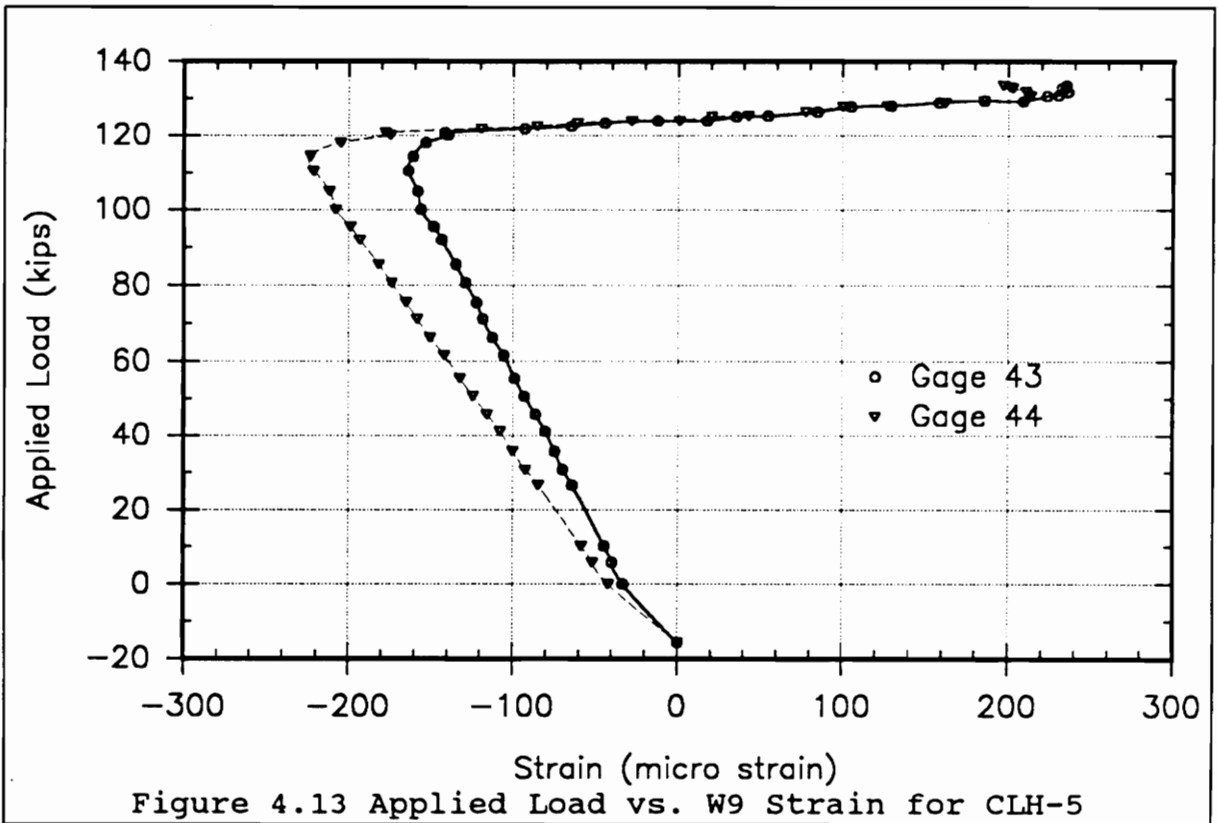
CLH-5 displayed similar behavior to the previous tests. Figure 4.12 shows the test specimens load-deflection curve.

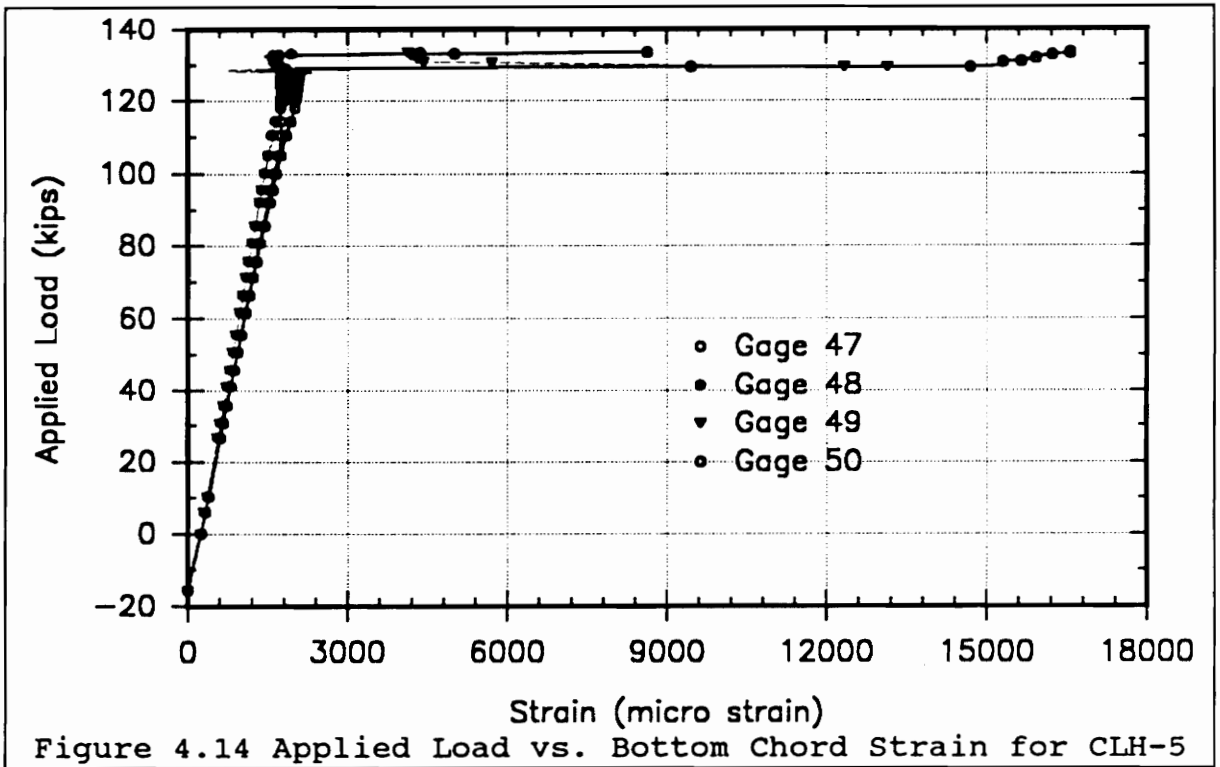


The web members behaved linearly in the linear region of the test specimen. After the specimen yielded, the members no longer behaved linearly elastically.

This can best be seen in Figure 4.13. This plot shows the strains in member W9. This is normally a compression member and in the linear portion of the load-deflection curve the strains are compressive. After the specimen's yield load, however, W9 goes into tension. This is very similar behavior to that of member W9 in CLH-1.

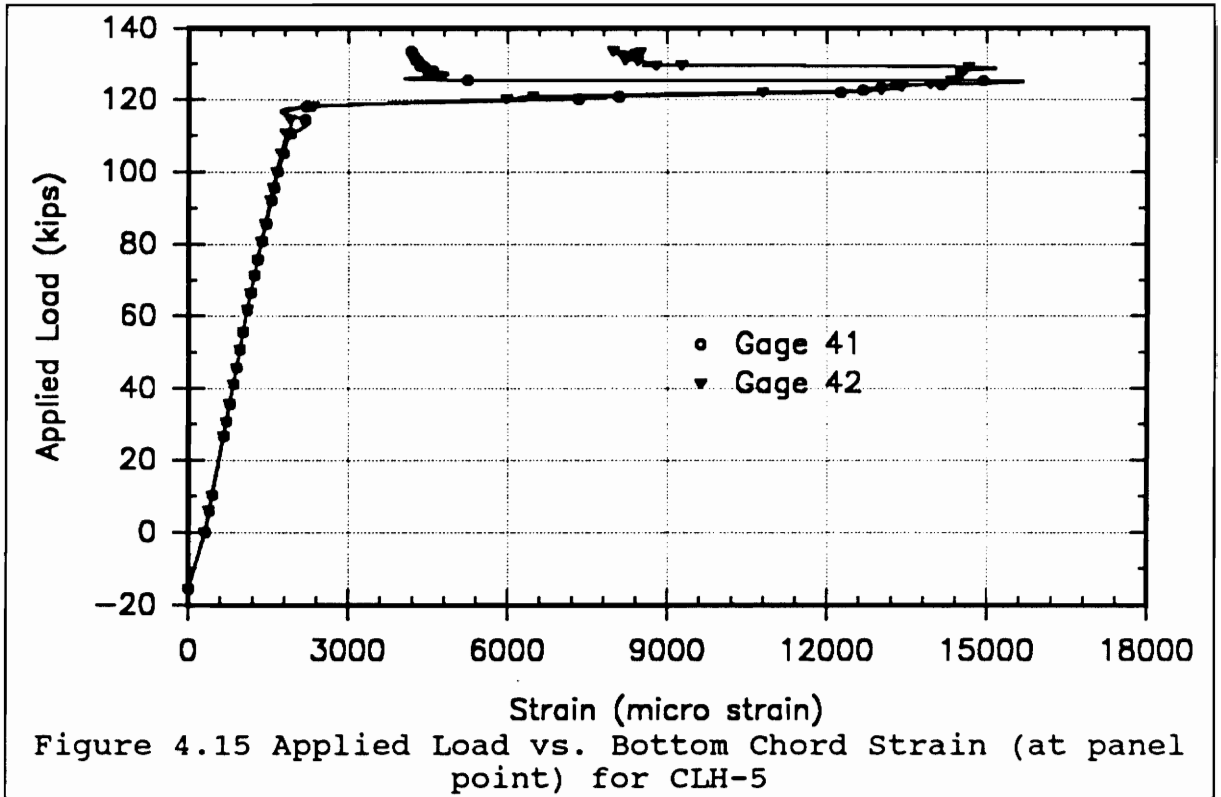
Bottom chord strains at the centerline show that yielding was reached towards the end of the test. (See Figure 4.14) It is interesting to note the behavior at a





panel point on the bottom chord. As seen in Figure 4.15, strains show that the bottom chord yielded at the panel point before yielding at midspan. Once the bottom chord yielded at midspan, the strains at the panel point actually decreased.

The strains in the top chord top chord at the midspan were approximately zero at failure. The compressive strains caused by the placement of the concrete slab were canceled by a small tensile force, leaving almost no strain in the top chord at failure.

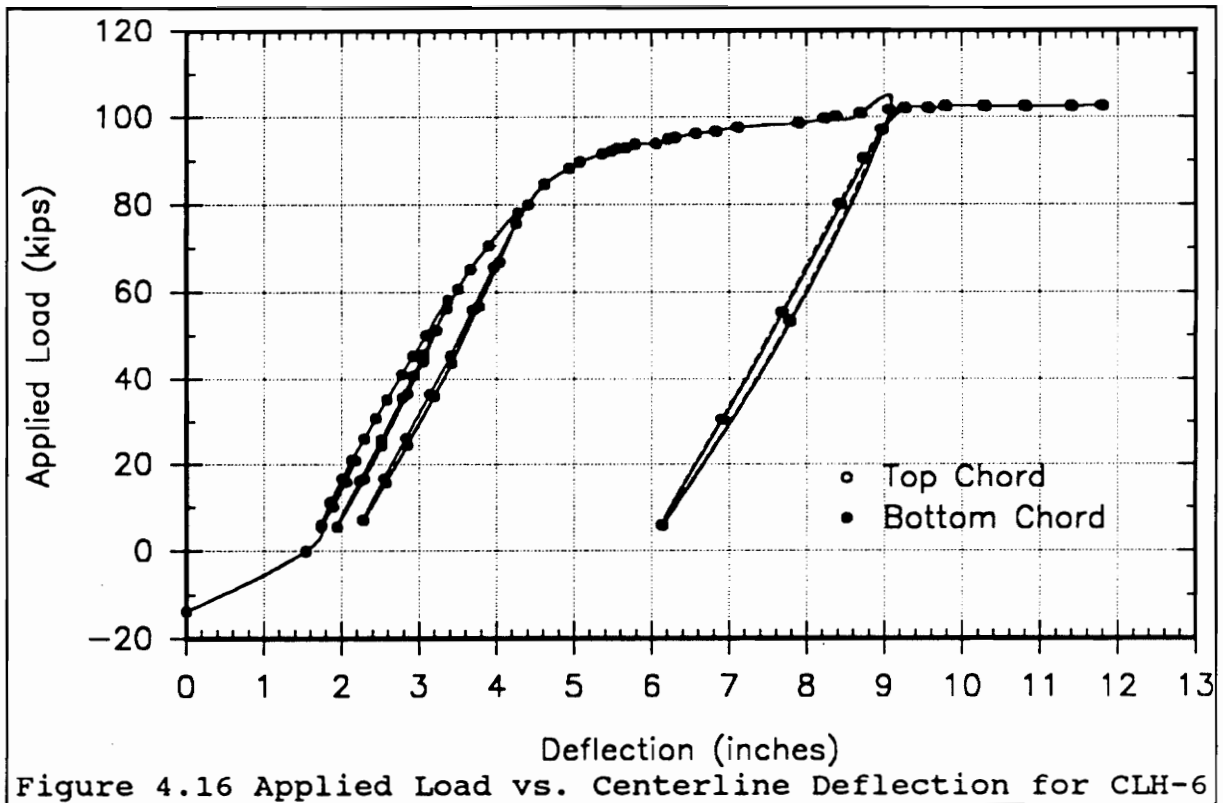


4.7 CLH-6 Test Results

The CLH-6 specimen was different from previous specimens in two major ways. The web members were designed for a load 30% greater than normally designed for and no vertical web members were used.

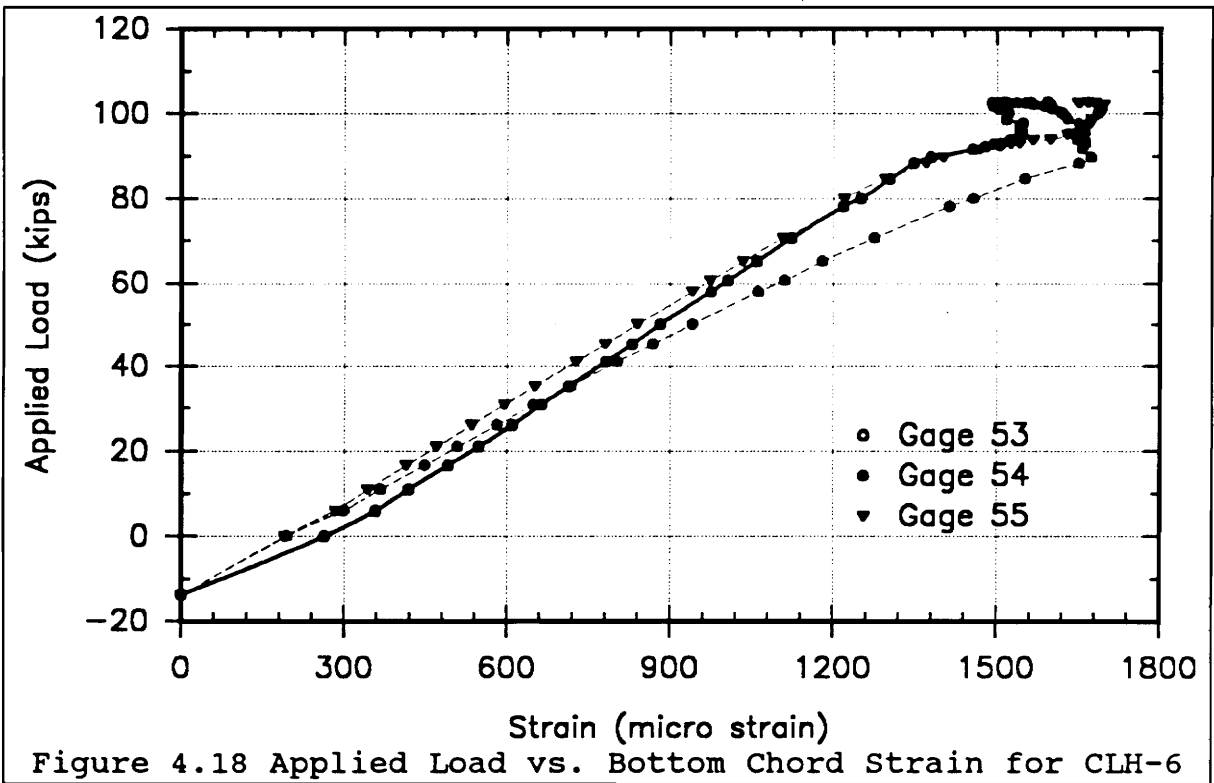
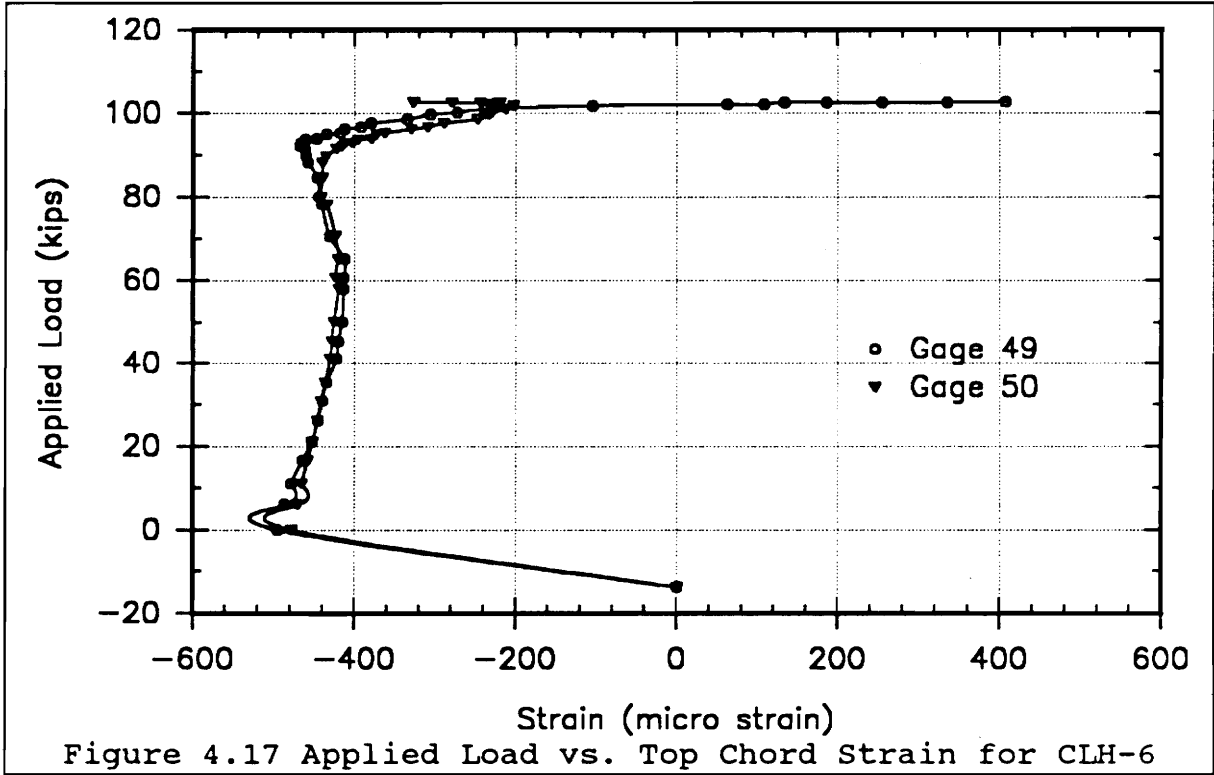
The specimen failed at an applied load of 102.5 kips and a midspan deflection of 14.1 in. Failure was characterized by local buckling of the top chord.

The load-deflection curve, Figure 4.16 is comparable to those of the previous tests. Even though the web members were larger than normal, their behavior was similar to previous tests.



This test was the first test to reach a load that was higher than predicted. After the specimens yield load, the strains in the top chord at midspan moved from compression to tension. The strains in the top chord indicate that the top chord is helping to carry the extra load. (See Figure 4.17)

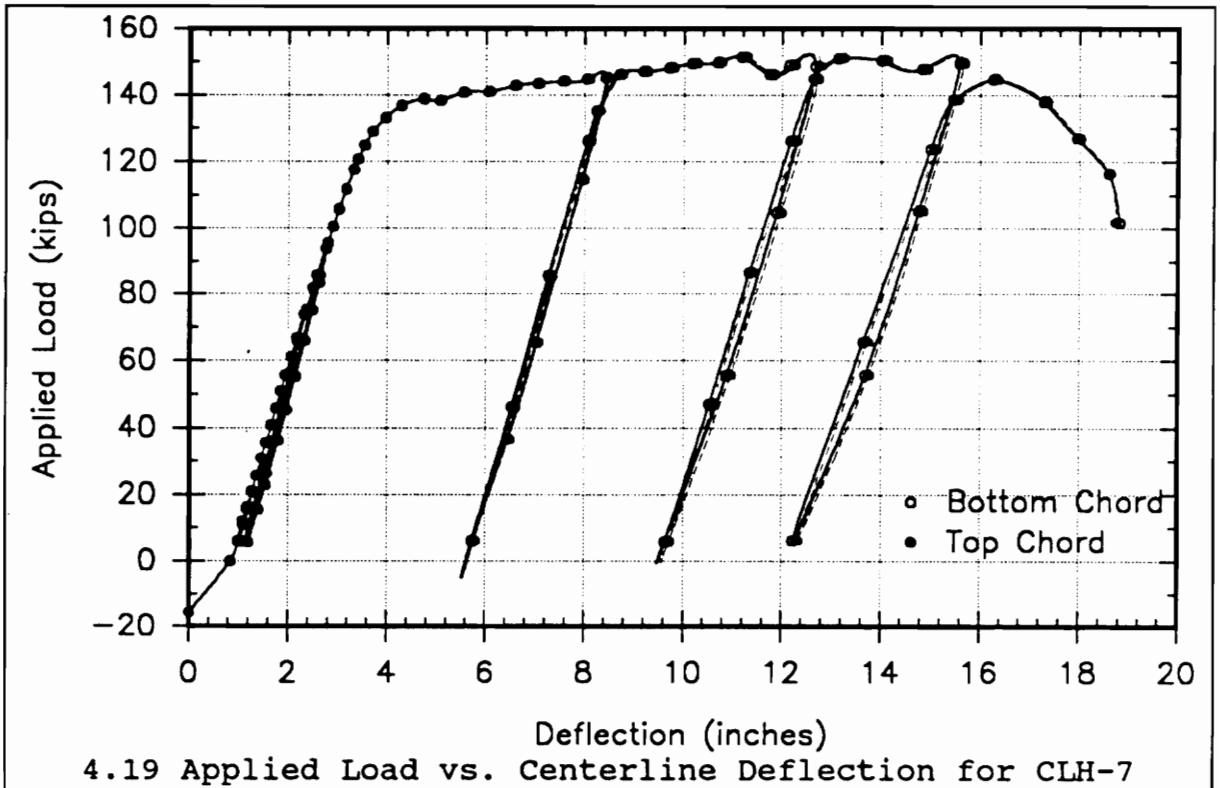
The bottom started to yield at the panel points as in previous tests. The yielding zone did not reach the strain gages located at the mid span, see Figure 4.18. Again, the bottom chord strains at midspan approach but do not exceed the yield strain of 1870 micro strains.



4.8 CLH-7 Test Results

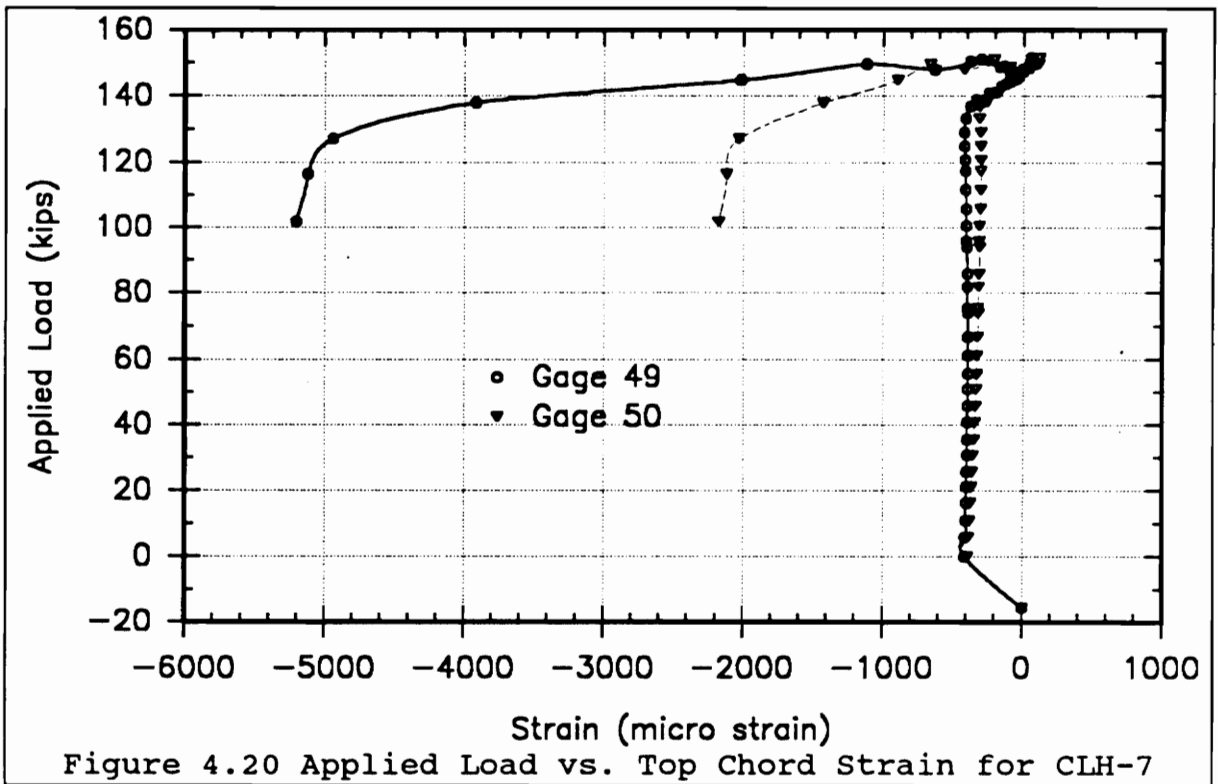
Like CLH-6, the joist used in this test also had web members designed to take 30% more load than normally design for. The test failed at a load of 151.6 kips and a midspan deflection of 18.8 in. At this load the specimen lost composite action.

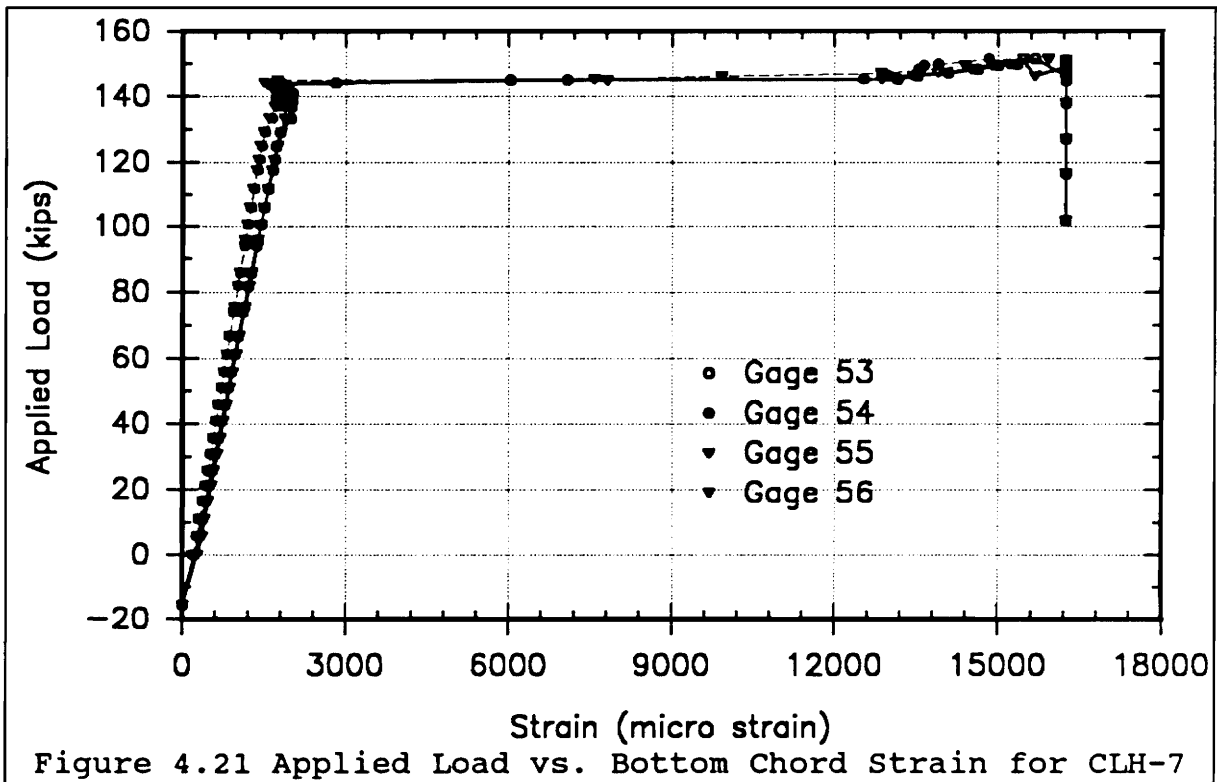
The loss in composite action can be seen in the load-deflection curve, Figure 4.19. The effective stiffness of the section, as seen in the unloading cycles, decreases as the test progresses. Also, the load that the specimen can carry decreases towards the end of the test.



The strains in the top chord, Figure 4.20, show the lose of composite action. After the initial compressive strains caused by the placement of the slab, the strains in the top chord remain constant. At the specimens yield load the top chord begins to take a tensile force. Once composite action is lost, the strain in the top chord rapidly becomes more compressive.

The bottom chord behaved as in previous tests. Yielding first occurred at the panel points. However, unlike earlier tests, the bottom chord completely yielded at the centerline. The strains in the bottom chord are shown in Figure 4.21



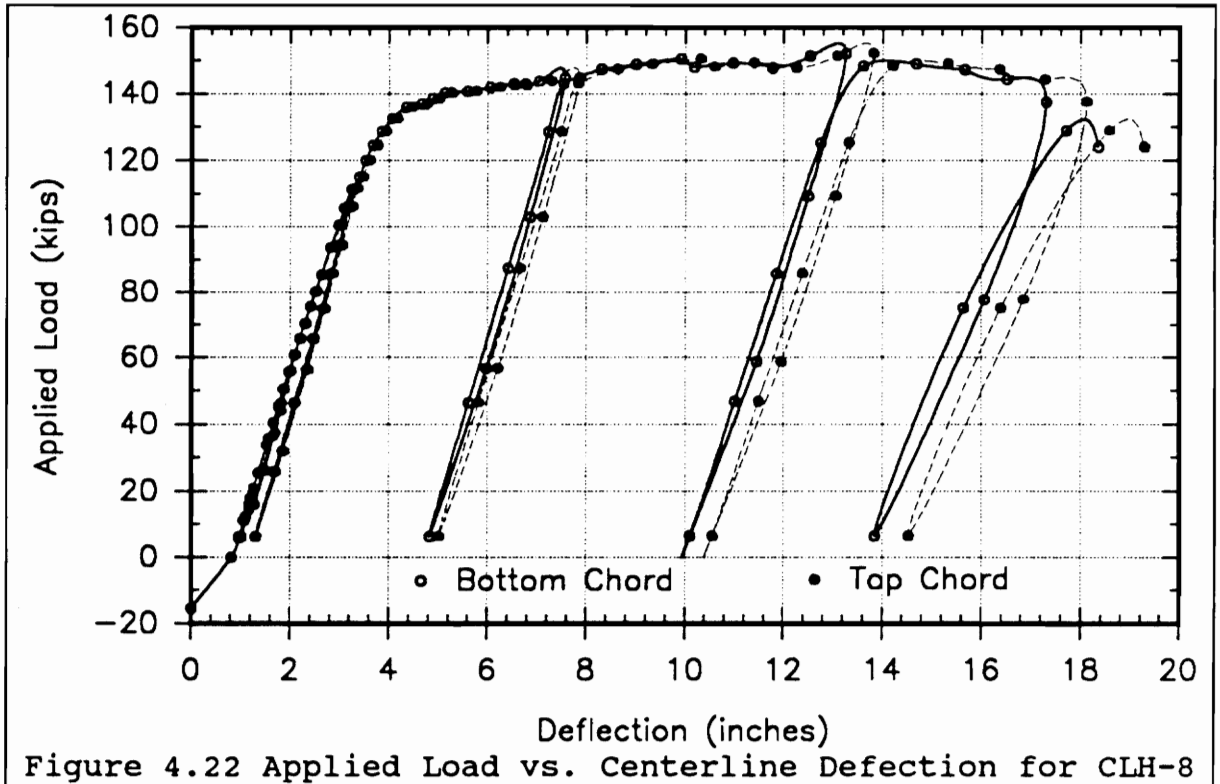


4.9 CLH-8 Test Results

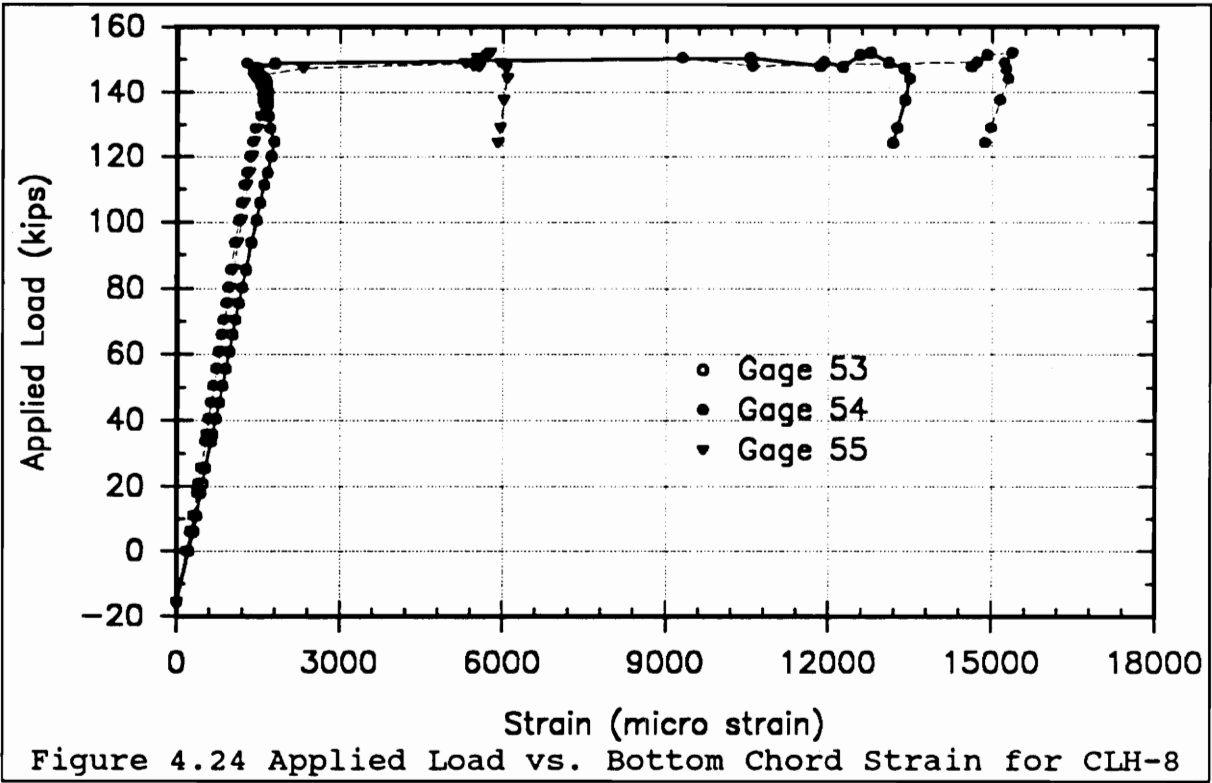
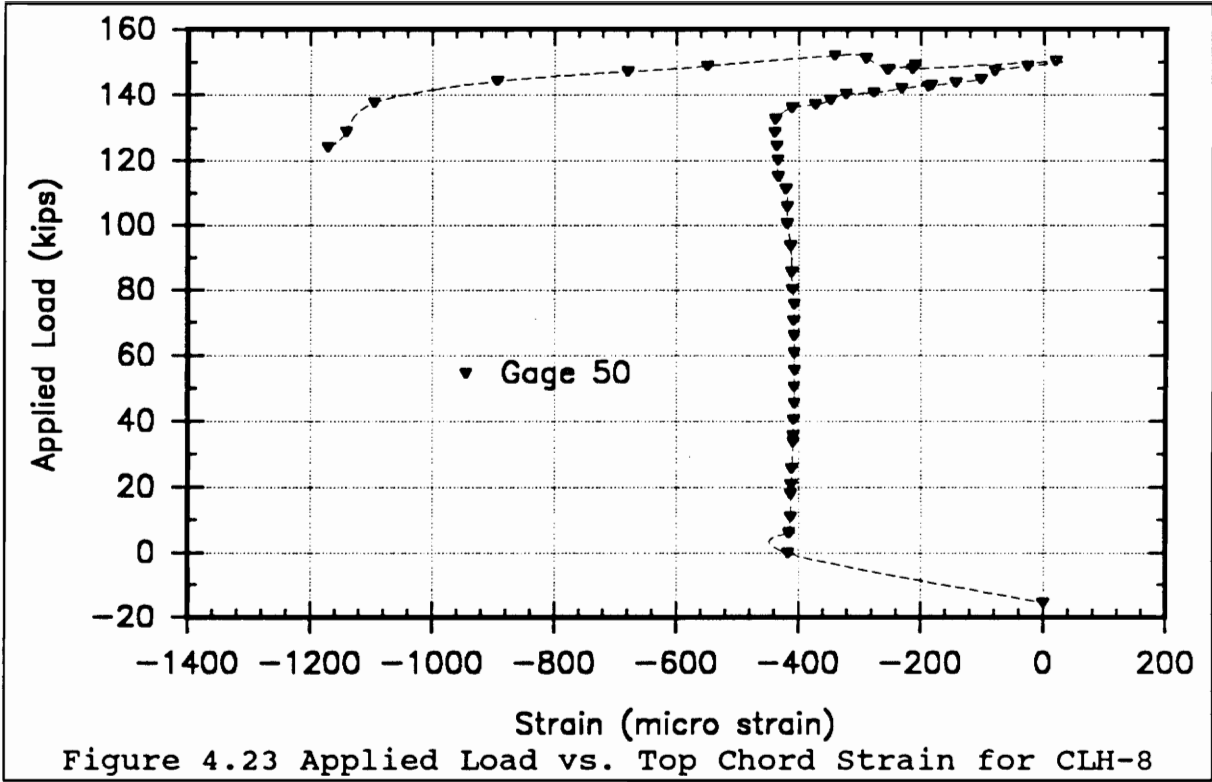
This was the last test performed in this series. Specimen CLH-8 had the same configuration as specimen CLH-7. However, unlike CLH-6 and CLH-7 the web members were not over-designed.

The specimen failed at a applied load of 152.2 kips and a centerline deflection of 18.8 in. The specimen carried a load greater than was predicted. As in CLH-7, failure was caused by the loss of composite action between the concrete slab and the steel joist.

The load-deflection curve, Figure 4.22, shows the loss of effective stiffness in the unloading cycles in the later portion of the test. As in CLH-7, the specimen load carrying capacity was reduced by the loss of composite action.



The top chord strains, shown in Figure 4.23, are similar to those of CLH-7. The strain stays virtually the same during the elastic loading portion. After the specimen has yielded, the top chord develops a tensile force. Once composite action is lost, the strains move rapidly into compression.



The bottom chord started to yield at the panel points near midspan. The yield zone expanded to midspan of the bottom chord. The strains in the bottom chord at midspan are shown in Figure 4.24. It is seen that yielding at midspan occurred after the specimens yield load.

CHAPTER V

DISCUSSION

5.1 General Observations

The overall behavior of all test specimens was similar. General points can be made as to the behavior of the major members of the composite joist from the data gathered during the tests. These include the bottom chord, top chord, web members, shear studs, and the concrete slab.

5.2 Load-Deflection Curves

The load-deflection curves for the specimens had two distinct regions. At low load levels the specimens displayed linear behavior. Once a specimen had reached a yield load, the load-deflection curve moved along a yield plateau.

5.3 Bottom Chords

The overall behavior of a composite joist can be directly related to the behavior of the bottom chord. The yield load of the specimen is directly related to the yielding of the bottom chord.

The bottom chords began to yield at the panel points nearer to the midspan in all tests. As the test progressed, the yielding zone in the bottom chord expanded. In tests with large deflections (CLH-2, CLH-6, CLH-7, and CLH-8) this

yielded zone reached the strain gages placed at midspan between the panel points.

The bottom chord begins to yield at the panel point due to the combined stress fields which were caused by the framing in of the web members. The strains at midspan approach the yield strain and then remain constant. The elongation of the bottom chord is occurring principally at the panel points. As the elongation of the bottom chord increases the yielded zone also increases.

5.4 Top Chords

The behavior of the top chord depends on the ratio of shear to moment. For a single span system the shear is high at the end and zero at midspan. The moment is the opposite, highest at midspan and zero at the ends.

Near midspan, where the moment-to-shear ratio is high, the strains remain relatively constant in the linear region of the specimen. The placement of the concrete on the non-composite joist puts the top chord in compression at midspan. After the initial yielding of the bottom chord, the top chord moves into tension. In tests where the ultimate calculated load was not exceeded, the tension increase is of the same magnitude as the compression caused by the placement of the concrete slab. This increase in strain accounts for the slight increase in load carry capacity

during the yield plateau.

In tests CLH-4, CLH-7 and CLH-8, where composite action was lost, the top chord moves back into compression. Since the top chord can no longer pass the compressive force into the concrete, it must resist the load itself.

In regions of high shear and low moment, near the end of the span, the top chord strains display linear behavior.

In tests where the top chord buckled locally, the failure occurred near the quarter point of the specimen. These failures were located where shear studs were welded to the top chord. Prior to failure, yielding could be observed in the bottom of the outstanding leg of the top chord around the shear studs. This was observed all along the top chord.

5.5 Web Members

The applied load vs. web member strain plots are similar to the load-deflection curve. In the linear region of the specimen, the strains in the various members are linear.

Once the bottom chord begins to yield, the behavior of the web members vary with their distance from the midspan. Members near midspan, where the bottom chord is yielding, show a remarkable change. Compression members move into tension and tension members shift into compression. This is particularly evident in CLH-3. In this test, the failure was

caused by the lateral buckling of the web tension members at midspan.

Web members near the end of the specimen maintain their linear behavior after the bottom chord begins to yield. Here, the bottom chord and other members are not yielding and the behavior of the members does not change.

5.6 Ultimate Load Calculations

The ultimate load calculations were made using an ultimate strength method. Tension in the bottom chord and compression in the concrete are assumed to provide the moment resisting couple. See Appendix A for a detailed example. Five tests did not exceed their calculated capacity, while three did exceed their capacity.

The ratios of the total shear stud capacity to tension yield force in the bottom chord, $\Sigma Q_n/T_{bc}$, and the actual applied moment to the theoretical ultimate moment, M_a/M_u , are listed in table 5.1 for each test. Comparing the first five tests (those which did not exceed the calculated capacity), it can be seen that as the value of $\Sigma Q_n/T_{bc}$ increases so does the value of M_a/M_u . For the last three tests, the differences between $\Sigma Q_n/T_{bc}$ and M_a/M_u are too small to make a comparison.

The exception to all this is CLH-4. The value for $\Sigma Q_n/T_{bc}$ is less than 1.0. That is, there was not sufficient

shear connection to fully develop the bottom chord in tension. At the peak load, the shear connectors failed.

The ratio of M_a/M_y is between 1.15 and 1.24 for all tests, (except, of course CLH-4) with an average of 1.19. Thus once the specimen starts to yield it has approximately 15% more load carry capacity. In this respect, the results from the first five tests correspond to those of the last three tests.

Table 5.1
Summary of Load Relationships

TEST	M_a/M_{ud}	M_a/M_u	M_a/M_y	$\Sigma Q_n/T_{bcd}$	$\Sigma Q_n/T_{bc}$
CLH-1	1.05	0.888	1.152	1.51	1.34
CLH-2	1.18	0.972	1.239	2.58	2.23
CLH-3	0.974	0.961	1.161	1.75	1.68
CLH-4	0.887	0.757	1.068	1.13	0.975
CLH-5	1.13	0.918	1.208	1.52	1.27
CLH-6	1.28	1.10	1.211	1.57	1.39
CLH-7	1.36	1.15	1.174	1.58	1.38
CLH-8	1.36	1.17	1.182	1.58	1.40

where

- M_a = Maximum applied moment
- M_u = Calculated ultimate moment with actual material Properties
- M_{ud} = Calculated ultimate moment with Design Material Properties
- M_y = Moment at which the test specimen yielded
- ΣQ_n = sum of shear stud strengths from the point of maximum moment to the point of zero moment
- T_{bc} = $A_{bc} F_y$
- A_{bc} = Area of bottom chord
- F_y = actual yield stress of the bottom chord
- T_{bcd} = $A_{bc} F_{yd}$
- F_{yd} = Design yield stress of bottom chord, 50 ksi

5.7 Stiffness and Deflection Calculations

The effective stiffness calculations closely approximated the measured stiffness of the specimens as determined from the load-deflection curves. From Table 5.2 it can be seen that the ratios of the actual moment of inertia to the calculated moment of inertia, I_{effa}/I_{effc} , are higher for joists with shallower depth. The exception is CLH-4 which failed to reach a yield plateau.

Table 5.2
Stiffness Comparisons

TEST	depth (in)	I_{effa} (in ⁴)	I_{effc} (in ⁴)	I_{effa}/I_{effc}	D_{max}/D_y
CLH-1	36	4342	4884	0.889	2.124
CLH-2	36	4327	4878	0.887	5.227
CLH-3	16	2037	2049	0.994	3.814
CLH-4	16	1607	1989	0.808	2.919
CLH-5	34	3012	4347	0.693	4.315
CLH-6	14	1561	1260	1.239	2.781
CLH-7	20	2527	2366	1.068	6.431
CLH-8	20	2385	2354	1.013	6.717

where I_{effa} = Effective moment of inertia of the test specimen
 I_{effc} = Calculated effective moment of inertia
 D_{max} = Maximum deflection of test specimen
 D_y = Calculated yield deflection:calculated using the specimens calculated ultimate load and effective moment of inertia

Comparing the stiffness results, the first five tests had a stiffness less than predicted. While, the last three tests had a stiffness greater than predicted.

Looking at the first five tests alone, the shallower the joist the closer the theoretical stiffness predictions

were to the actual stiffness. In the stiffness calculations, the moment of inertia of the composite section is reduced by value equal to 15% of the steel joists moment of inertia. This reduction is to account for the shear deformation of the web under loading. The amount of shear deformation in the web members is related to the depth. With a deeper joist, there is greater shear deformation. Hence, the joist is not as stiff as the theoretical calculations predict.

A measure of ductility is the ratio of D_{\max}/D_y or the ratio of maximum deflection to the yield deflection. A structure where this value is greater than 3 is commonly considered to be ductile. It can be seen in Table 5.2 that for five of the eight test performed the value of D_{\max}/D_y was substantially greater than 3. The ductility of the specimens was obtained by two actions, the yielding of the bottom chord and the slip between the concrete and the joist.

5.8 Normalized Load-Deflection Curves

In Figure 5.1, the normalized load-deflection curves for the tests in the CLH series are presented. The normalized moment is calculated by dividing the actual moment by the ultimate theoretical moment. The normalized deflection is calculated by dividing the actual deflection by theoretical elastic deflection at the ultimate moment.

Therefore, a theoretical elastic-plastic load-deflection curve would go from the point (0.0 load, 0.0 deflection) to (1.0 load, 1.0 deflection) horizontally to (1.0 load, infinite deflection).

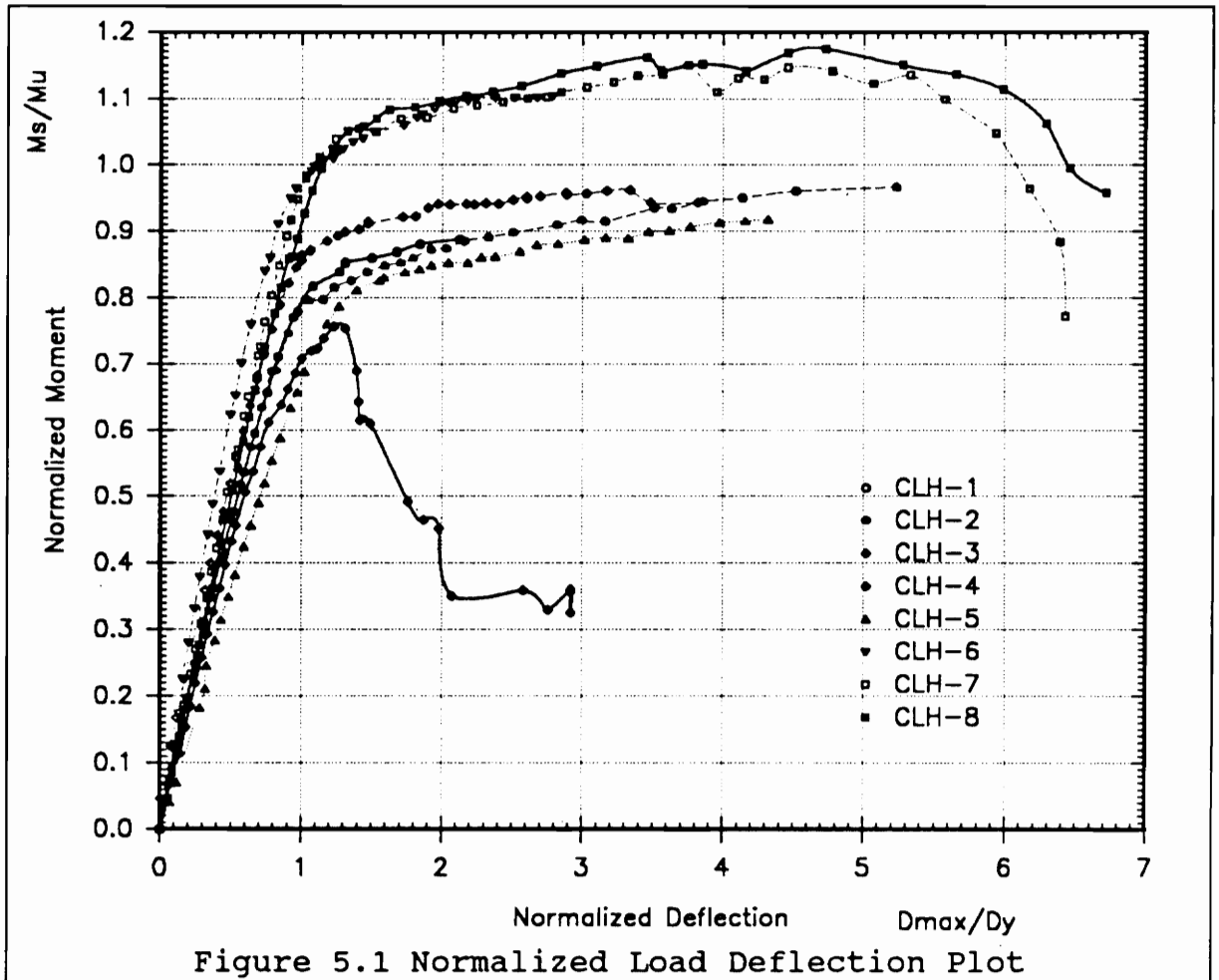


Figure 5.1 gives a graphical representation of the stiffness and load carry capability of the test specimens. From this figure, it can be seen that the data falls into two different groups. In the first group, which includes the

first five tests, the stiffness was less than calculated and the maximum load reached was also less than theoretically predicted. In the second group, consisting of the last three tests, the opposite was true. The actual stiffnesses and load carry capacity were greater than theoretically predicted.

CHAPTER VI

FINITE ELEMENT ANALYSIS

6.1 Introduction

A non-linear finite element model was prepared. This model was based on the material properties and configuration of CLH-1. Results, specifically the load-deflection curve, from the finite element model were compared to the results obtained from the test specimen.

The non-linear behavior was obtained from the plastic beam elements used for the bottom chord. The non-linear slip behavior of the shear connectors was not modeled.

6.2 Model

The model consisted of non-linear and linear elements. The symmetry of the test specimen was used. Therefore, only half of the joist was modeled.

Elastic beams were used to model the top chord and the shear connectors. The shear studs were small elastic beams with a stiffness approximately 1000 times greater than the stiffest member of the model. This gave the shear studs an effectively infinite stiffness.

Truss elements were used to model the concrete and the web members. The nominal member cross-sectional areas and moments of inertia were used for the web members. The

concrete section properties were derived by transforming the concrete into an equivalent steel section using the modular ratio. See Figure 6.1.

A non-linear plastic beam element was used to represent the bottom chord. This element provided the non-linearity of the model. The yield stress used for the element was the actual measured yield stress of the bottom chord minus the calculated stress in the bottom chord due to the placement of the concrete on the non-composite joist. This allowed for a direct comparison between the load in the model and the applied load used for the tests.

Since symmetry was used, load was applied to the model using four point loads. These loads were placed at the same locations of the loading used in the test.

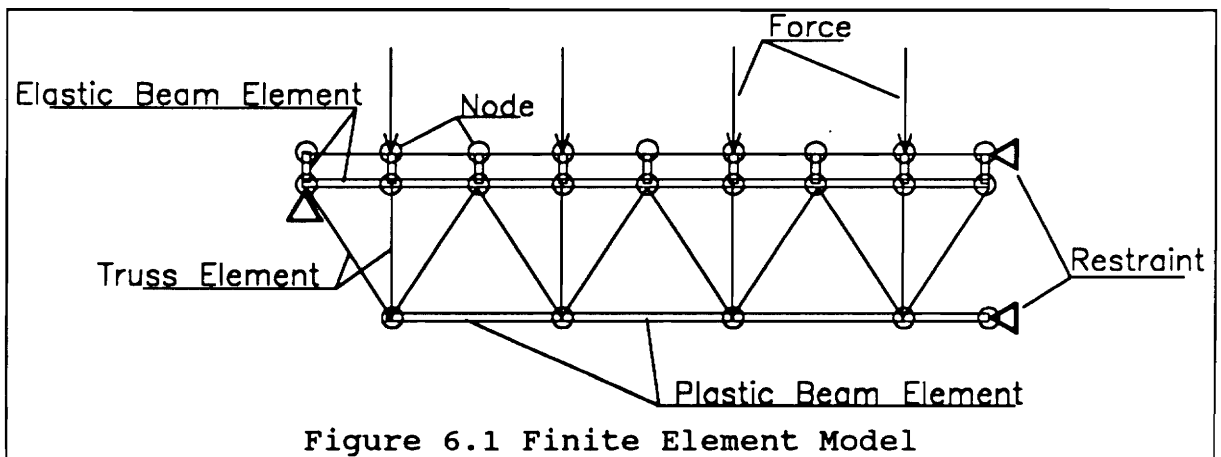
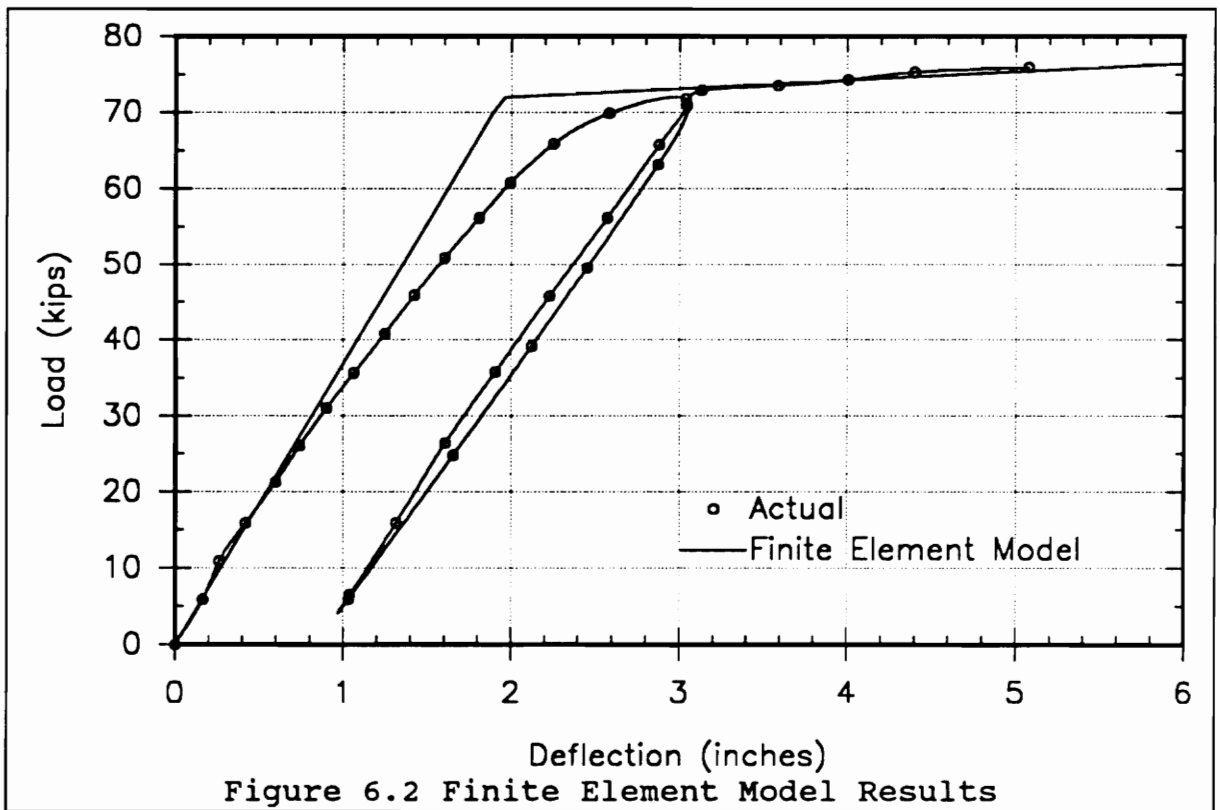


Figure 6.1 Finite Element Model

6.3 Results

In the elastic range, the finite element model was

slightly stiffer than the actual specimen. (See Figure 6.2) This is somewhat expected since computer models of this nature are generally stiffer. Also, the shear connectors were effectively infinitely stiff, causing the model to be stiffer. However, the finite element model displayed elastic deflections close to those predicted by hand calculations.



The specimen yielded at a lower load than the model did. This reflects that the bottom chord of the specimen started to yield at a lower load than predicted due to stress concentrations at the panel points. Also, the non-linear nature of the shear connectors would effect the yield load.

The model closely approximates the behavior of the specimen along the yield plateau. More detailed analysis of the test results will be conducted in a future study.

CHAPTER VII

DESIGN METHODOLOGY

7.1 Introduction

This chapter discusses the methodology used by Vulcraft to design the composite joists used in this research project. This is a standard design method used by Vulcraft for composite and non-composite joist design. Allowable stress design methodologies used were based on the Steel Joist Institute's "Standard Specifications for Longspan Steel Joists, LH-Series" and Chien and Ritchie (1984).

In the design, it is assumed that the joist is an unshored simply supported span. The top chord is considered fully braced by the concrete slab. Shear connection is provided by shear studs.

Following is a description of design methods for each member of the joist and for the overall moment resistance. For a complete example of design calculations see Appendix A

7.2 Steel Top Chord

The steel top chord of the joist is designed for two different loadings, non-composite action before the concrete

has hardened and composite action after the concrete has hardened.

The top chord of the joist is designed to resist construction loads and the wet weight of the concrete slab non-compositely. In this case, the top chord forms the compressive component of the resisting moment.

Once the concrete has hardened the top chord transfers the horizontal web forces through the shear studs into the concrete. The top chord also passes the vertical web forces from one web member to another web member. In the end panel, the top chord is design to resist all the vertical shear. Here, the top chord does not have any web members to transfer the load to. Thus, the force in the top chord due to this shear is limited to 40% of the yield stress.

7.3 Steel Bottom Chord

The bottom chord provides the tension force in the resisting moment. It provides the tension component for both composite and non-composite action.

The maximum moment that the joist can withstand can be determined by using the tensile yield stress of the bottom chord. This method is presented by Chein and Richie (1984). A resisting couple is formed by the tension force in the bottom chord and the compressive force of an equivalent stress block in the concrete.

The design moment strength is less than 60 percent of the ultimate moment capacity. Also, the total stress in the bottom chord due to non-composite dead loads and composite loads is limited to less than 90% of the yield stress.

$$M_d = 0.6 M_u \quad (7.1)$$

where: M_d = design moment
 M_u = theoretical ultimate moment

7.4 Concrete Slab and Steel Deck

The concrete slab is considered as a transformed concrete section with a limited effective width. The effective width either side of the centerline is the lesser of an eighth of the span length or one half the spacing between joists on each side of the joist.

For design, the average stress in the concrete does not exceed $0.45f'_c$ under design loads. The concrete slab must also be able to resist and anchor the shear studs.

7.5 Steel Web Members

The web members are designed to carry all the vertical shear at their location. The concrete slab, top and bottom chord, and steel deck are not be designed to resist any vertical shear. Panel lengths greater than 2.2 times the depth are avoided.

7.6 Steel Shear Studs

Sufficient shear studs are used to develop the full tensile yield load of the bottom chord. The amount of horizontal shear that is needed to be resisted between the point of maximum moment and zero moment is the lesser of the yield load of the bottom chord divided by 1.67 or the compressive resistance of the concrete slab divided by 1.67.

The shear resistance provided by one shear stud is given by AISC (Load 1986). The reduction factor for the shape and orientation of the decking is used.

Studs are placed in a uniform distribution along the span length. Those studs which do not fit into the uniform distribution are placed near the end of the span. The studs were assumed to be placed in the middle of the rib or towards the side closer to the end of the span. This insures that a sufficient cone of concrete can form on the compression side of the stud.

7.7 Serviceability Considerations

The serviceability of the joist must be considered at several times in the load history of the joist. The deflection of the non-composite joist under the wet weight of the concrete. The composite joist should be checked under occupancy loads and any additional partitions. Floor vibrations should also be considered.

The joist should be cambered such that under the wet weight of the concrete the top chord of the joist is flat. This avoids concrete ponding problems.

CHAPTER VIII

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary

The purpose of this study was to evaluate the behavior of long-span composite joists. Eight full scale tests were performed. Strains in the top chord, bottom chord, and various web members as well as the applied load and deflections were measured.

Despite varying the span, joist depth, joist configuration, and the amount of shear connection, the test specimens showed similar behavior. All the tests which had sufficient shear connection were able to reach a yield plateau. Calculations made for the elastic stiffness and ultimate moment capacity were close to the test results.

8.2 Conclusions

From the eight tests conducted, the following conclusions can reasonably be drawn.

1. The ultimate strength method used in this study is sufficiently accurate for calculating the moment carry capacity of the composite section. Only the bottom chord is considered to be fully developed in tension. The top chord is not considered to carry any load in calculating moment capacity.

2. The method proposed by Chein and Richie (1984) provides a sufficiently accurate and simple method to calculate the elastic stiffness of a composite joist.

3. The load-deflection curves from the tests show that composite joists have great ductility.

8.3 Recommendations

1. The ratio of $\Sigma Q_n / T_{bc}$ should be greater than 1.5 using design properties. All the tests with $\Sigma Q_n / T_{bc} > 1.5$ achieved a yield plateau. Keeping $\Sigma Q_n / T_{bc} > 1.5$ insures ductile behavior. The lone test where $\Sigma Q_n / T_{bc} < 1.5$, CLH-4, did not achieve a yield plateau in the load-deflection curve.

2. Vertical web members should not be used. Test specimens design and built without vertical web members did better than those specimens which included them.

3. The ultimate load method should be used to determine the ultimate moment that a composite joist can resist. The top chord should not assumed to carry any load.

8.3 Areas of further Research

The analysis of the test data has shown areas of further research.

1. The joists without vertical web members showed greater relative strength and stiffness. This was true

wether the web members were over-designed or not. The design methodology used should be studied to determine why this happened.

2. A more complete non-linear finite element model should be developed. This model should include non-linear elements to model the load-slip behavior of the headed shear studs.

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APPENDIX A
DESIGN EXAMPLES

A.1 Design Procedure

The following procedures were used in this report to calculate the theoretical ultimate moment and the theoretical elastic stiffness. The methods used here were presented by Chein and Richie (1984).

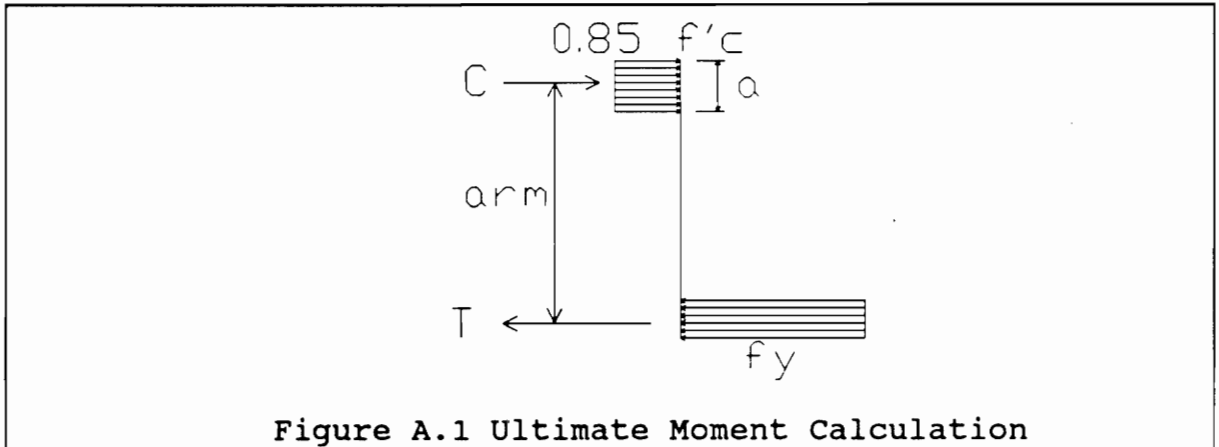


Figure A.1 Ultimate Moment Calculation

The ultimate moment capacity is equal to the resistance provided by the bottom chord in tension and an equivalent area of concrete in compression provided that there is adequate shear connection. See Figure A.1. The tension force is equal to the yield stress multiplied by the area of the bottom chord.

$$T = A_S F_Y \quad (A.1)$$

where:

T = tensile force
A_S = area of bottom chord
F_Y = yield stress

The effective depth of concrete in compression is calculated by using the Whitney equivalent stress block.

$$a = T/0.85f'_c b \quad (A.2)$$

where:

a = depth of stress block
f'_c = concrete compressive strength
b = width of stress block

The effective moment of inertia of the composite section is calculated from a modular transformation of the specimens section properties. From this moment of inertia, 15% of the steel joists moment of inertia is subtracted. This is to approximately account for the shear deformation of the web of the joist. Finally the moment of inertia is divided by a factor of 1 plus 15% for slip and 15% for creep. Due to the time frame of the test performed for this report only the slip reduction was considered.

$$I_{eff} = (I_{tran} - 0.15I_S)/(1 + sf + cf) \quad (A.3)$$

where:

I_{eff} = effective moment of inertia of the composite joist
I_{tran} = moment of inertia of the transformed composite section
I_S = moment of inertia of steel joist
sf = reduction factor for slip = 0.15
cf = reduction factor of creep = 0.15

A.2 Sample Problem Description

The following shows the calculation of the moment capacity for CLH-1 using a steel yield stress of 50 ksi and concrete strength of 3000 psi.

Given:

Joist Properties:

span = 56 ft.

depth (out-to-out) = 36 in.

Top Chord = 2L-2.50x2.50x0.313

$A_s = 2.93 \text{ in}^2$

$I_z = 1.70 \text{ in}^4$

$y = 0.740 \text{ in.}$

Bottom Chord = 2L-3.50x3.50x0.313

$A_s = 4.186 \text{ in}^2$

$I_x = 4.90 \text{ in}^4$

$y = 0.991 \text{ in}$

$F_y = 50000 \text{ psi}$

Slab Dimensions:

Width = 102 in.

Depth of concrete = 6 in.

$f'_c = 3000 \text{ psi}$

Deck Dimensions:

$h_r = 3 \text{ in.}$

$w_r = 6 \text{ in.}$

Stud Dimensions:

$3/4 \text{ in. } \phi$

$H_s = 5 \text{ in.}$

$F_u(\text{stud}) = 65000 \text{ psi}$

Total number of shear stud = 22

A.3 Moment Capacity

Shear stud strength:

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq A_{sc} F_u$$

$$Q_n = 0.5(0.442)\sqrt{(3)(3156)} = \underline{21.5 \text{ k}} \leq (0.442)(65) = 28.7 \text{ k}$$

$$\Sigma Q_n = 11(21.5) = \underline{236.5 \text{ k}}$$

Bottom chord tension:

$$T \leq \begin{cases} A_{bc} F_y = 4.186(50) = \underline{209.3 \text{ k}} \leq \text{governs} \\ \Sigma Q_n = 236.5 \text{ k} \end{cases}$$

Effective width of the slab:

$$b \leq \begin{cases} \frac{1}{4} \text{ span} = 56(12)/4 = 168 \text{ in.} \\ \text{width} = \underline{102 \text{ in.}} \end{cases}$$

Equivalent depth of concrete in compression:

$$a = \frac{T}{0.85 f'_c b} = \frac{209}{0.85(3)102} = \underline{0.805 \text{ in.}}$$

Moment capacity:

$$\text{arm} = 36 + 6 - \frac{0.805}{2} - 0.991 = \underline{40.61 \text{ in.}}$$

$$M_u = \text{arm } T = 40.58(209.3) = 8493 \text{ k-in} = \underline{708 \text{ k-ft}}$$

A.4 Stiffness Calculations

Transformed Section:

$$n = E_s / E_c = 29000/3156 = \underline{9.19}$$

$$b_{\text{eff}} = b/n = 102/9.19 = \underline{11.10 \text{ in.}}$$

$$A_c = B_{\text{eff}}(T_c - h_r) = 11.10(6-3) = \underline{33.3 \text{ in}^2}$$

$$y = A_y/A = [4.186(0.991) + 33.3(36+6-1.5)]/(4.186+33.3) \\ = \underline{36.09 \text{ in.}}$$

$$I_{\text{tran}} = 4.90 + 4.186(36.09-0.991)^2 + (11.10(3)^3)/12 \\ + 33.3(40.5-36.09)^2 = \underline{5834 \text{ in}^4}$$

Steel Joist:

$$y = A_y/A = [(2.93)(36-.740) + (4.186)(0.991)]/(4.186+2.93) \\ = \underline{15.07 \text{ in}}$$

$$I_s = 4.90 + 4.186(15.07-0.991)^2 + 1.70 \\ + 2.93(36-.740-15.07)^2 = \underline{2072 \text{ in}^4}$$

Effective moment of inertia:

$$I_{\text{eff}} = (I_{\text{tran}} - 0.15 I_s)/(1 + 0.15) \\ = [5834 - 0.15(2072)]/1.15 = \underline{4809 \text{ in}^4}$$

APPENDIX B

TEST DATA

Following is the data obtained for the eight composite joist tests performed. The data is presented in a series of plots with the applied load on the vertical axis and either strain, slip, or deflection on the horizontal axis. The first page for each test is a test summary sheet which provides a description of the test results as well as a figure showing the locations of the various strain, deflection and slip measurements taken during the test.

TEST SUMMARY SHEET

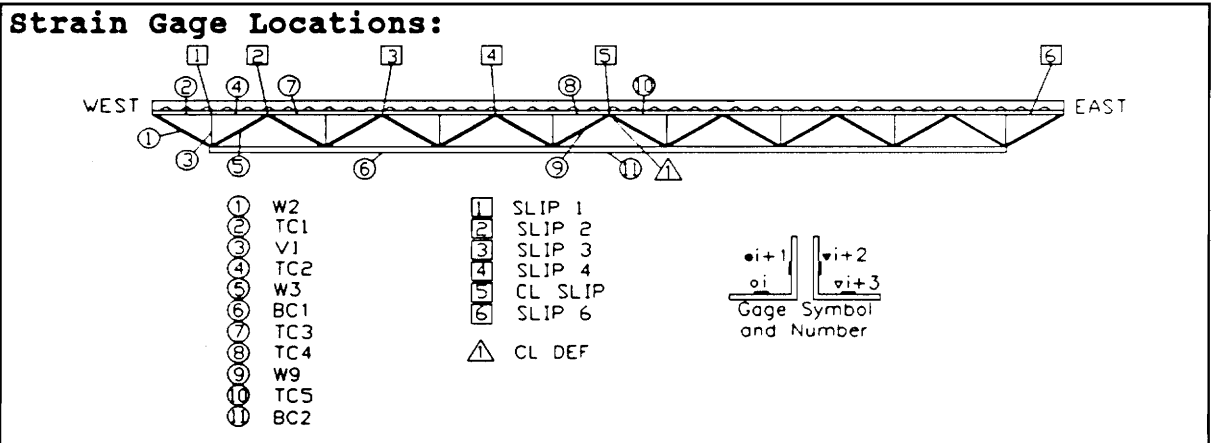
TEST DESIGNATION: CLH-1 TEST DATE: 28 July, 1989
 TEST LOCATION: Price's Fork Research Park

JOIST DESCRIPTION
 Depth: 36 in Top Chord: 2L-2.5x2.5x0.313
 Length: 56 ft Bottom Chord: 2L-3.5x3.5x0.313
 Number of Studs: 22 Width of Slab: 102 in
 Dead Weight of Joist and Slab: 516 plf

CALCULATED CAPACITIES (using measured material properties)
 Theoretical Max Applied Load: 85.5 kips
 Theoretical Moment of Inertia: 4884 in⁴
 Theoretical Elastic Deflection: 2.40 in

TEST RESULTS
 Max Applied Load: 75.9 kips Max Deflection: 5.08 in
 Yield Load: 65.9 kips Deflection at Yield: 2.24 in
 Average Max Bottom Chord Strain: 1168 micro strain
 Bottom Chord Fy: 56.42 ksi f'c: 4.43 ksi
 Concrete Pour Strains Recorded: NO
 Maximum Slip: 0.093 in
 Failure: Web members W7R and V6 buckled.

COMPARISON OF ACTUAL TO THEORETICAL
 Actual Stiffness/Theoretical Stiffness: 0.889
 Actual Max Load/Theoretical Max Load: 0.888



Comments: This test showed behavior that was to typify the following tests. The bottom started to yield at the panel points first. The yield zone did not extend to where the strain gages were placed. The gage readings were zeroed with the load beams in place (5.9 kips applied load).

CLH - 1

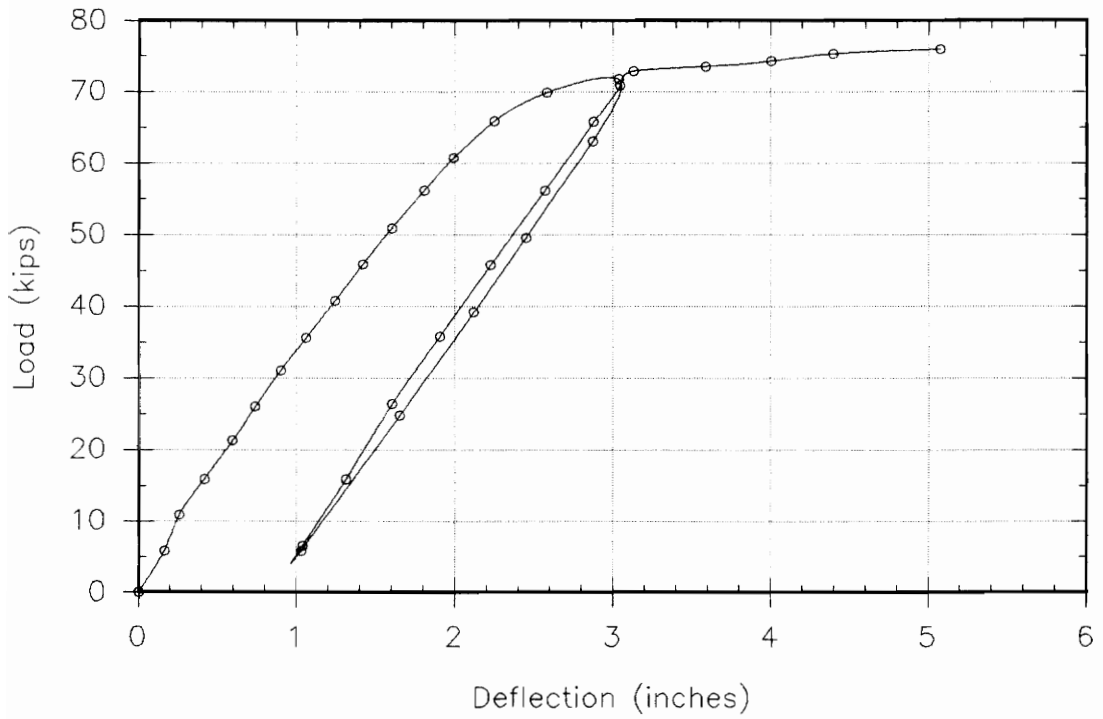


Figure B.1.1 Applied Load vs. Centerline Deflection

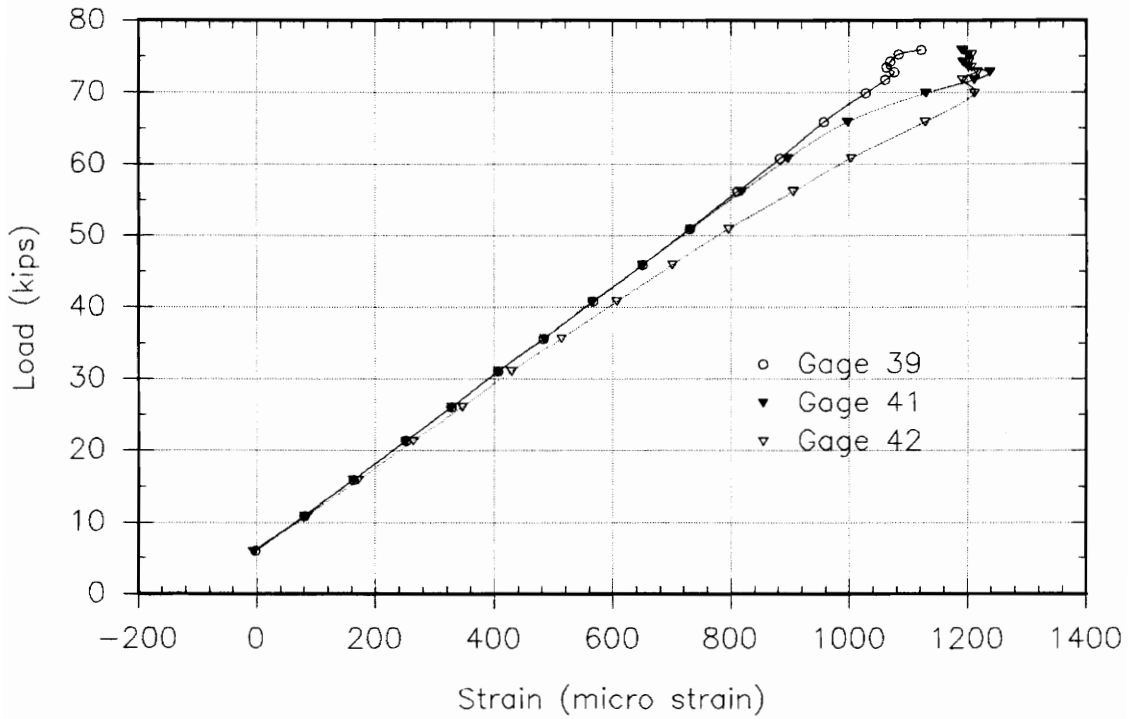


Figure B.1.2 Applied Load vs. Bottom Chord Strain (BC2)

CLH - 1

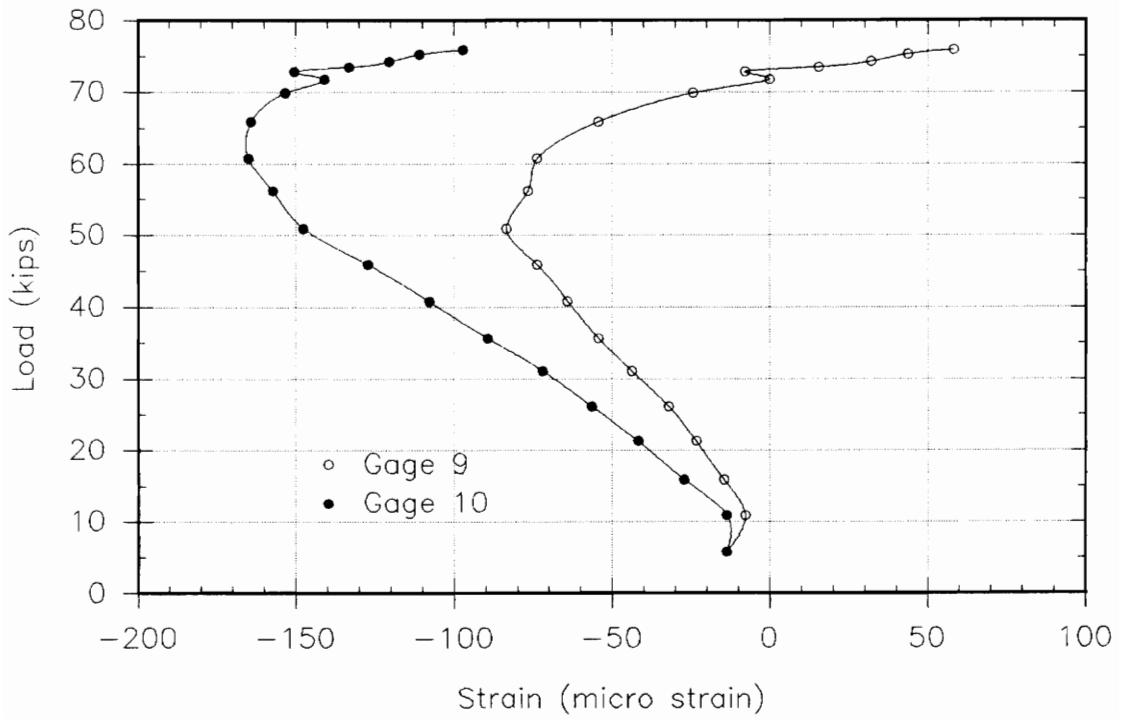


Figure B.1.3 Applied Load vs. V1 Strain

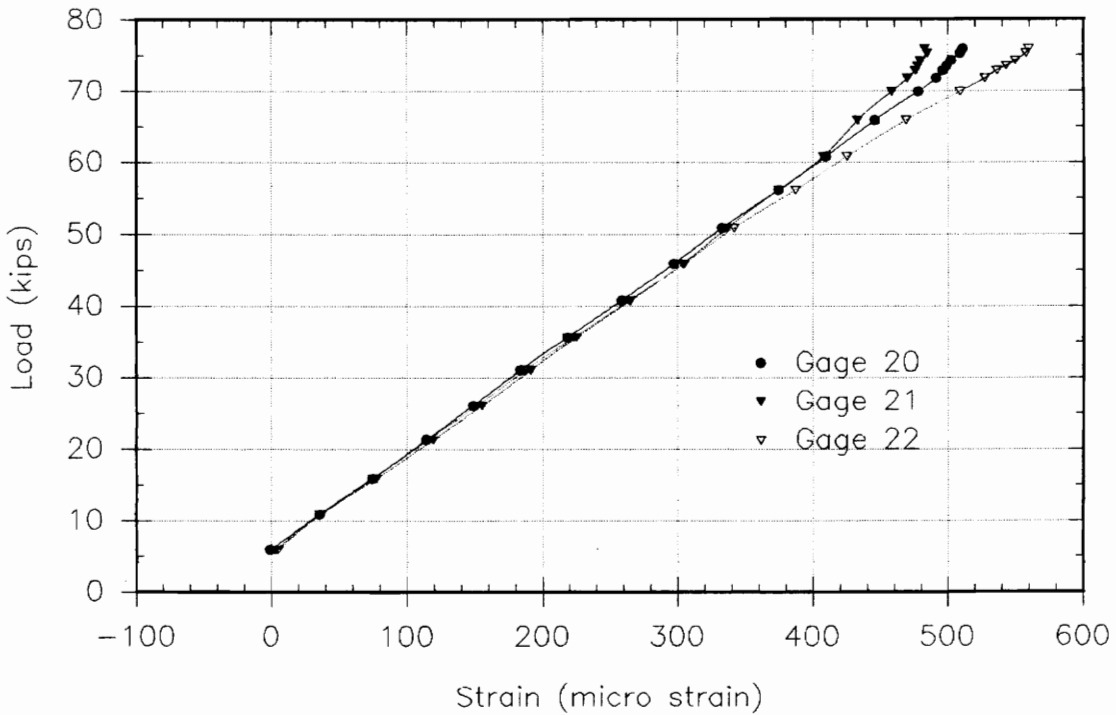


Figure B.1.4 Applied Load vs. Bottom Chord Strain (BC1)

CLH - 1

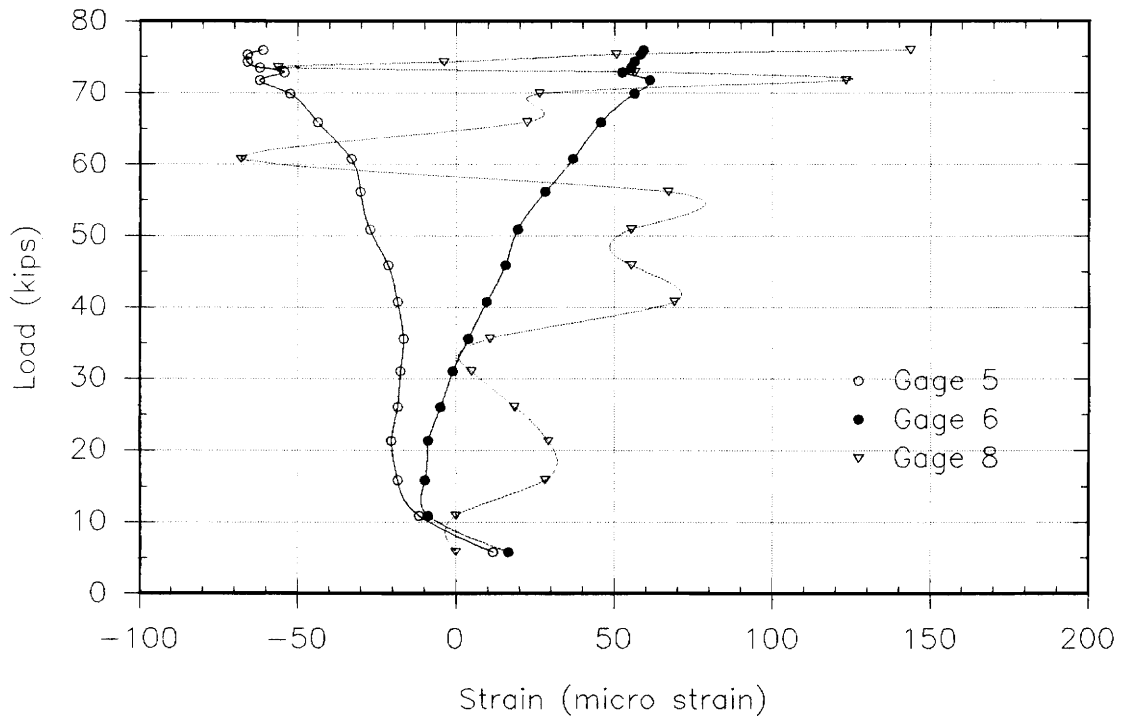


Figure B.1.5 Applied Load vs. Top Chord Strain (TC1)

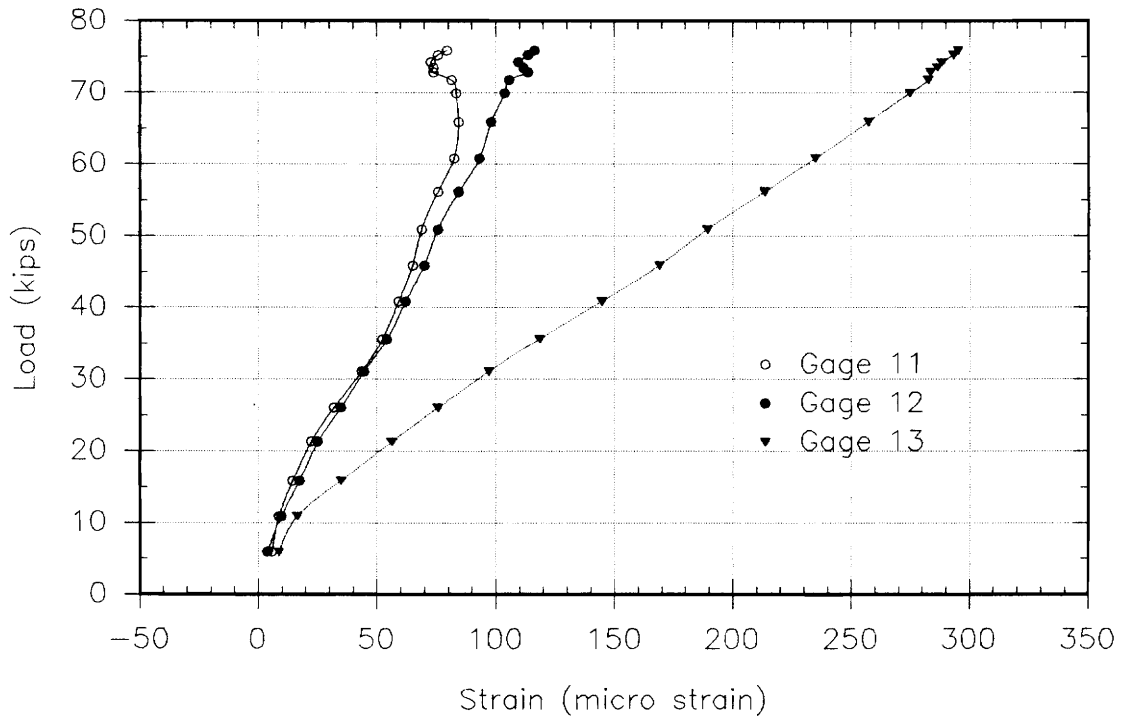


Figure B.1.6 Applied Load vs. Top Chord Strain (TC2)

CLH - 1

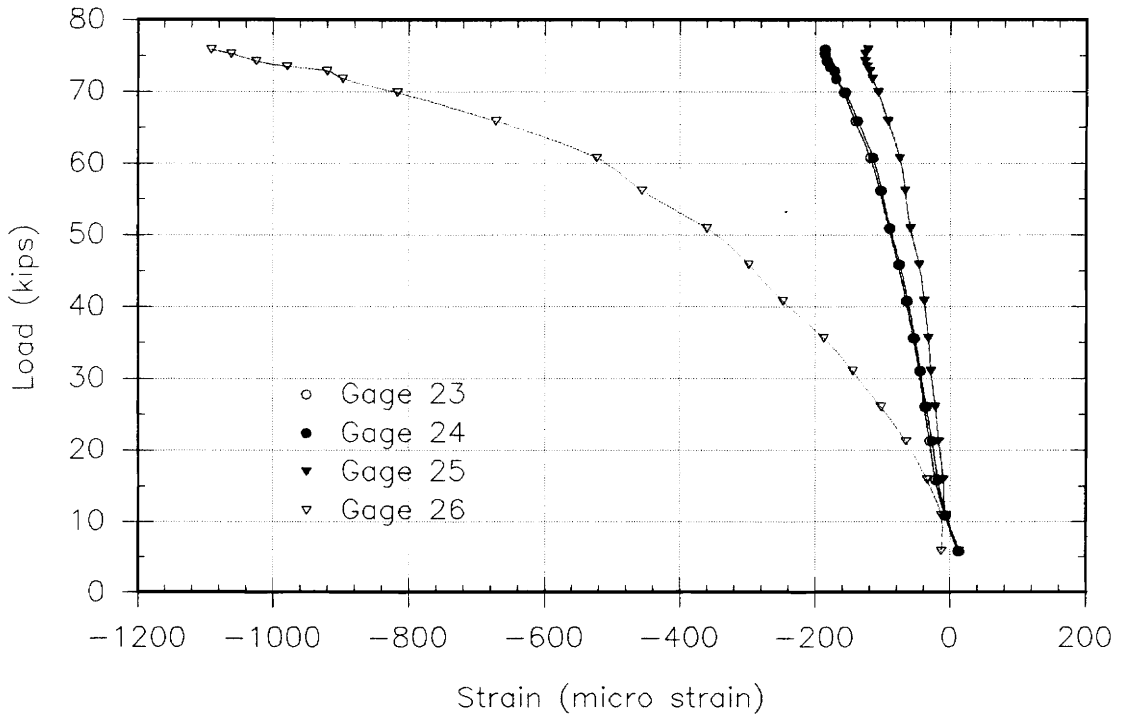


Figure B.1.7 Applied Load vs. Top Chord Strain (TC3)

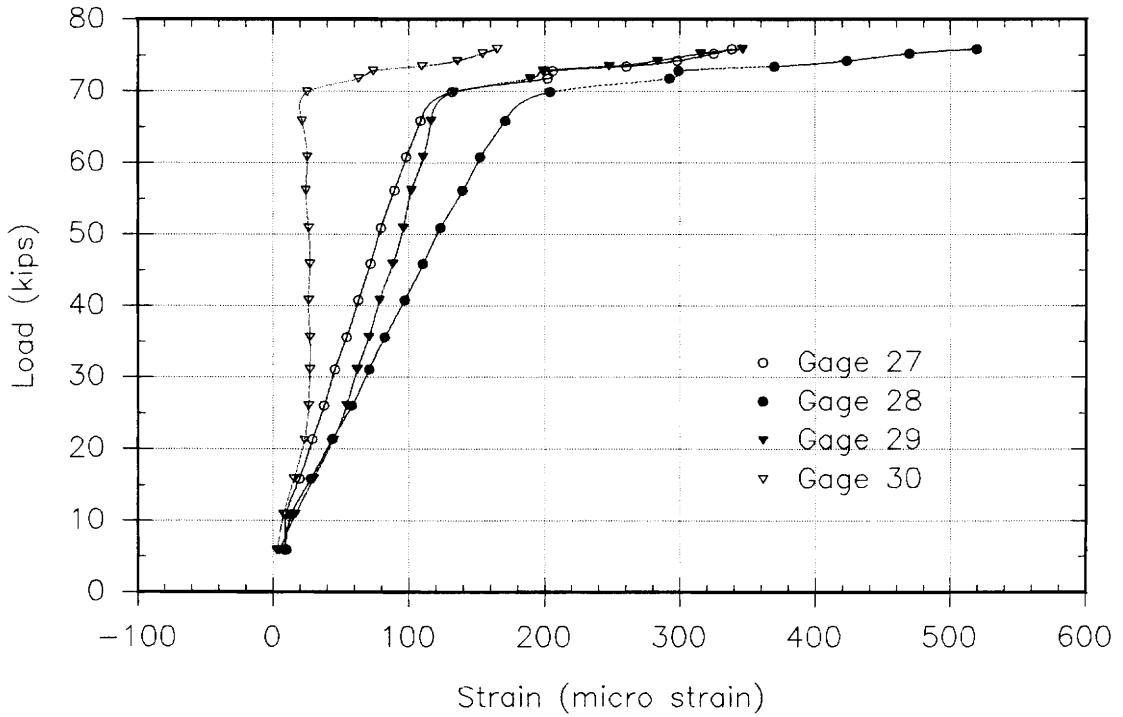


Figure B.1.8 Applied Load vs. Top Chord Strain (TC4)

CLH - 1

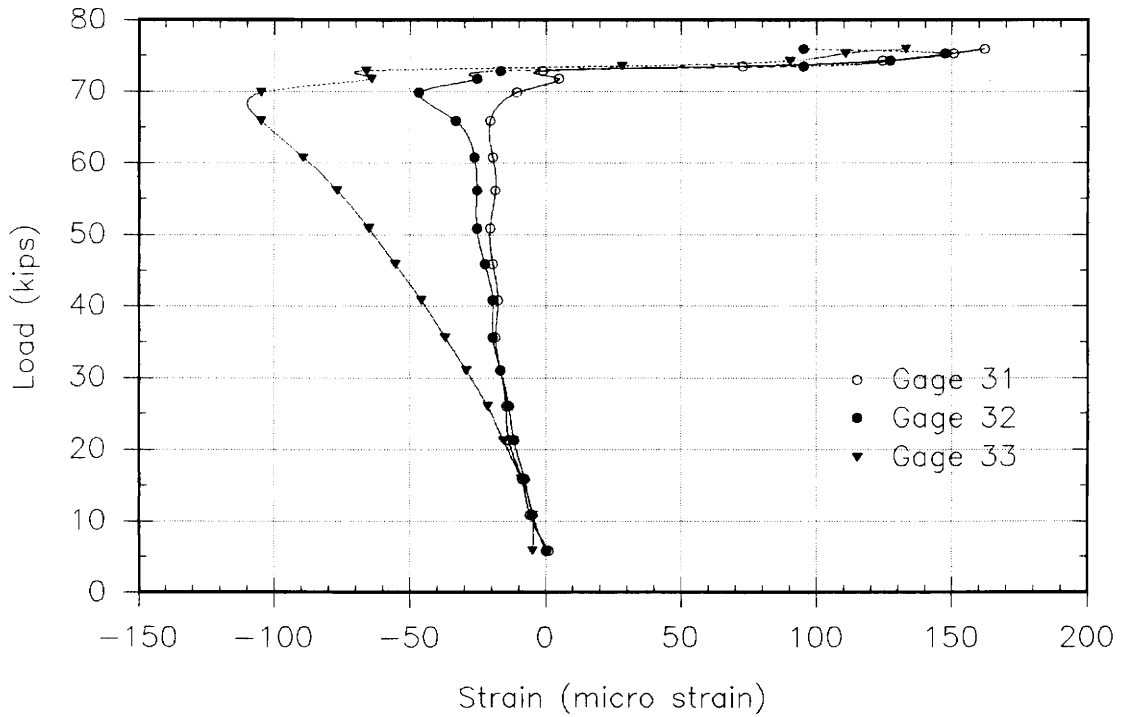


Figure B.1.9 Applied Load vs. W9 Strain

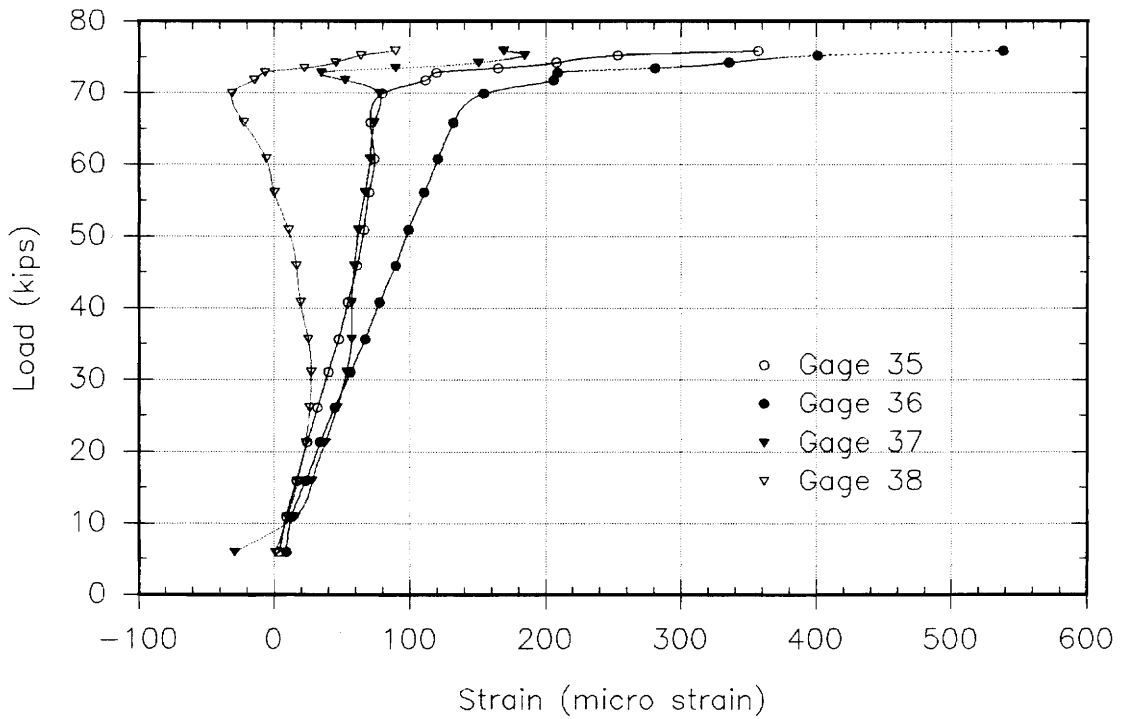


Figure B.1.10 Applied Load vs. Top Chord Strain (TC5)

CLH - 1

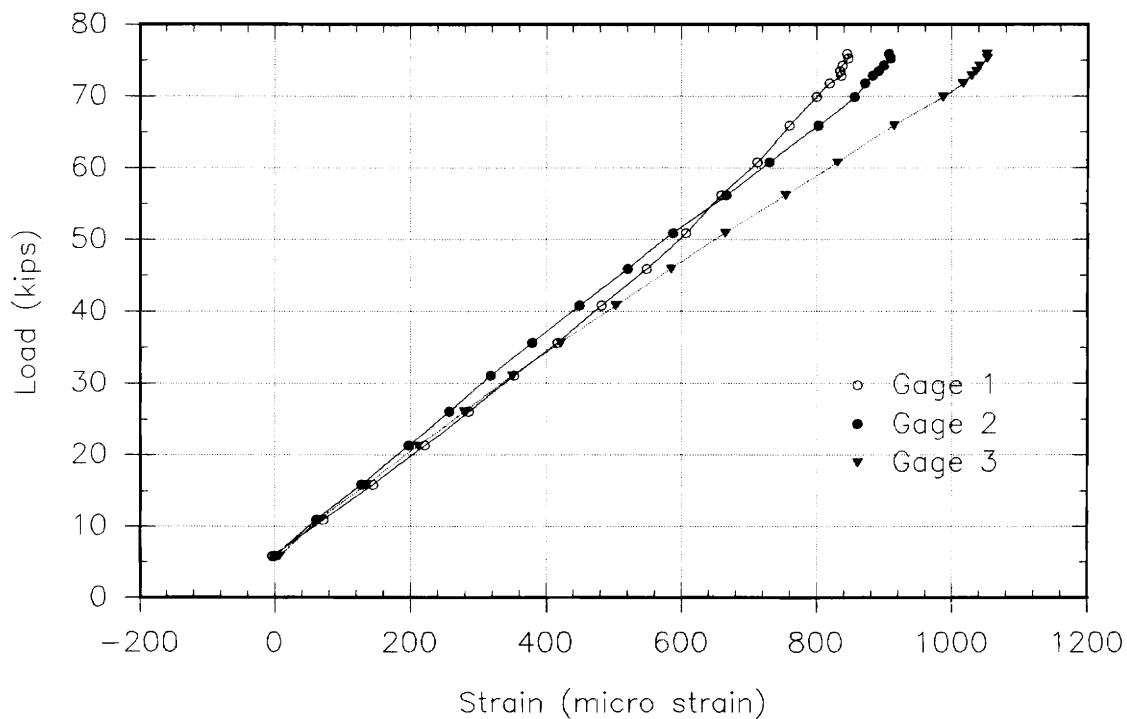


Figure B.1.11 Applied Load vs. W2 Strain

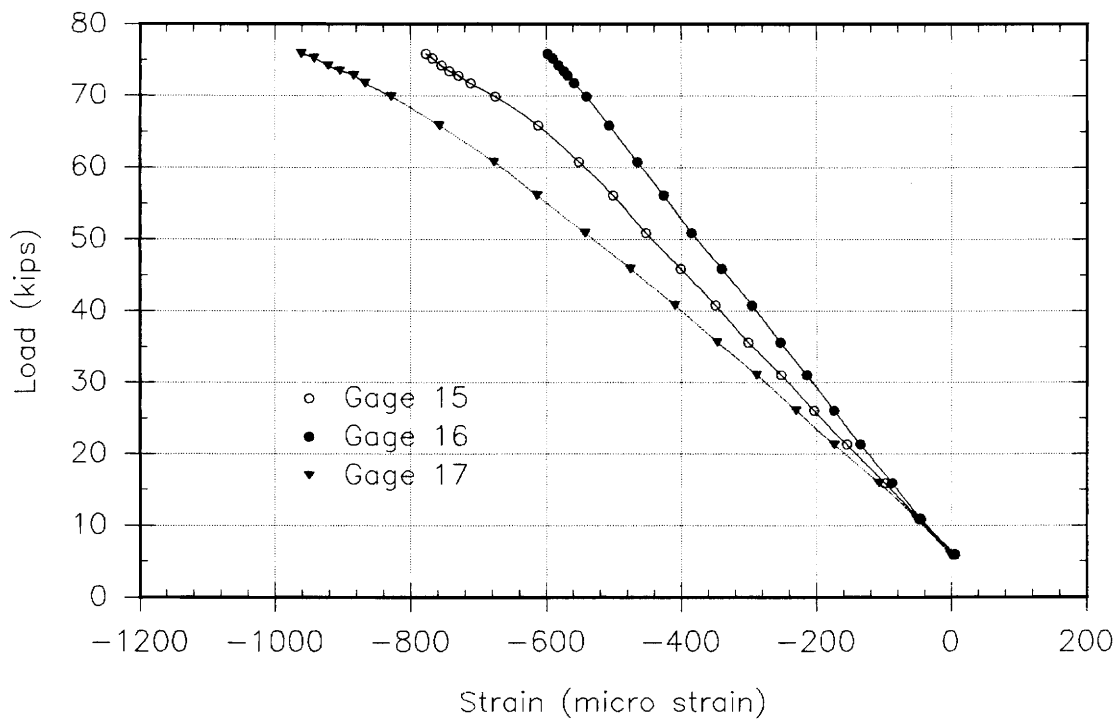


Figure B.1.12 Applied Load vs. W3 Strain

CLH - 1

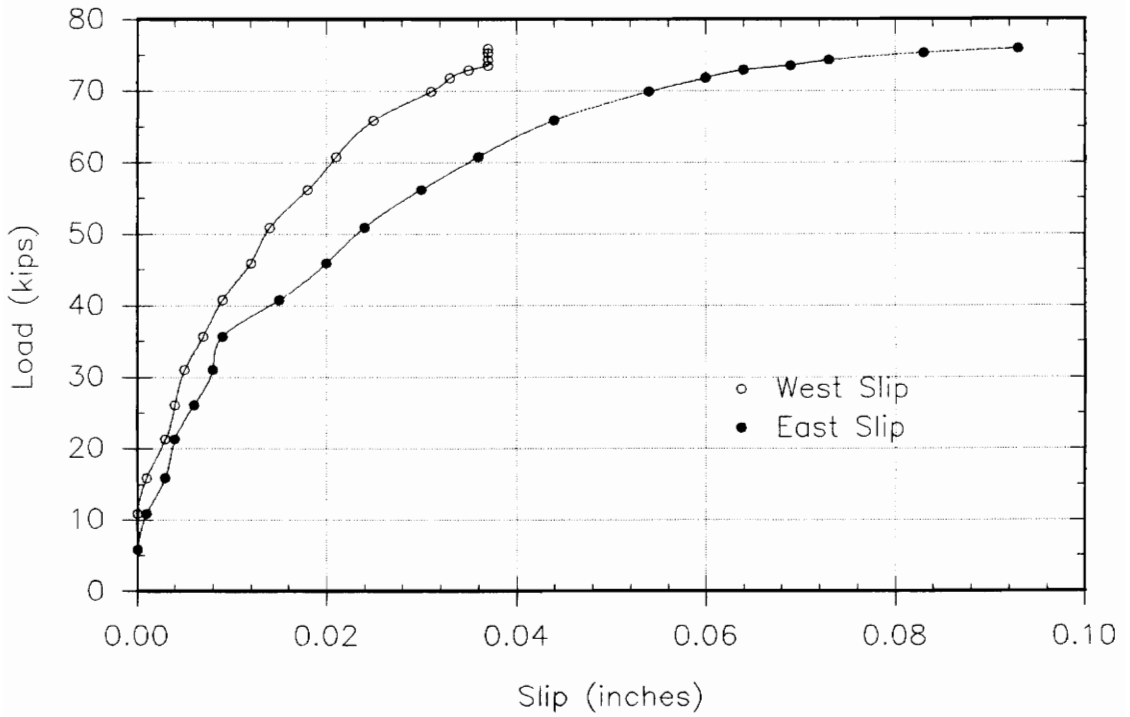


Figure B.1.13 Applied Load vs. End Slip

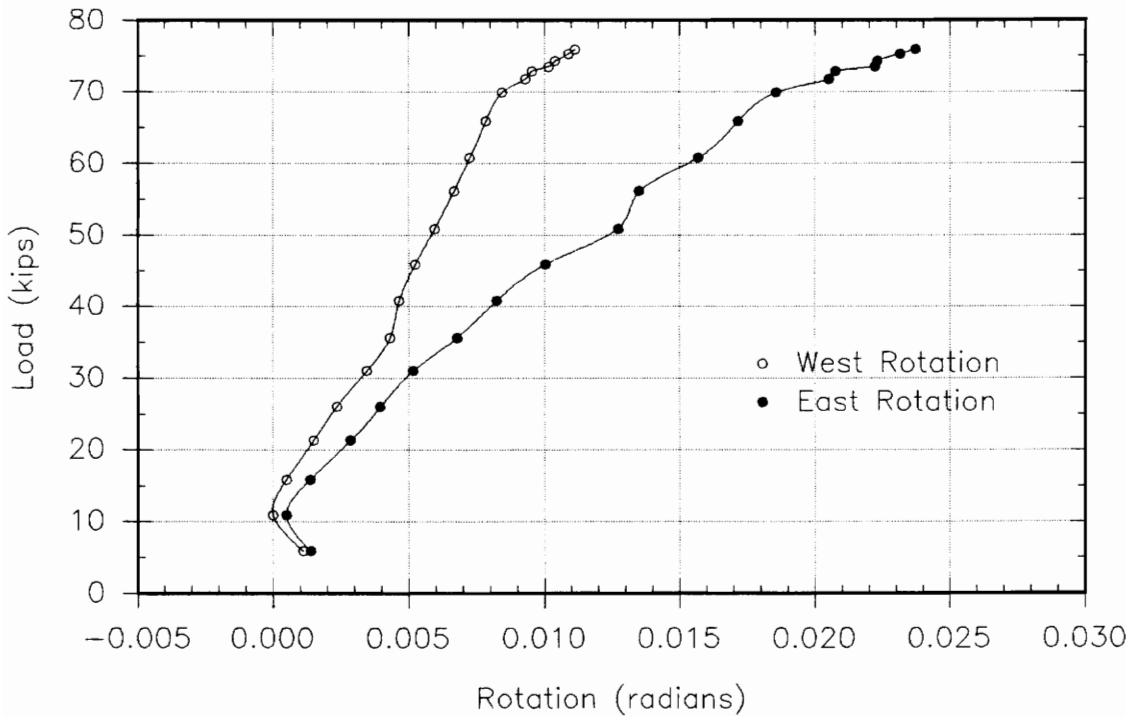


Figure B.1.14 Applied Load vs. End Rotation

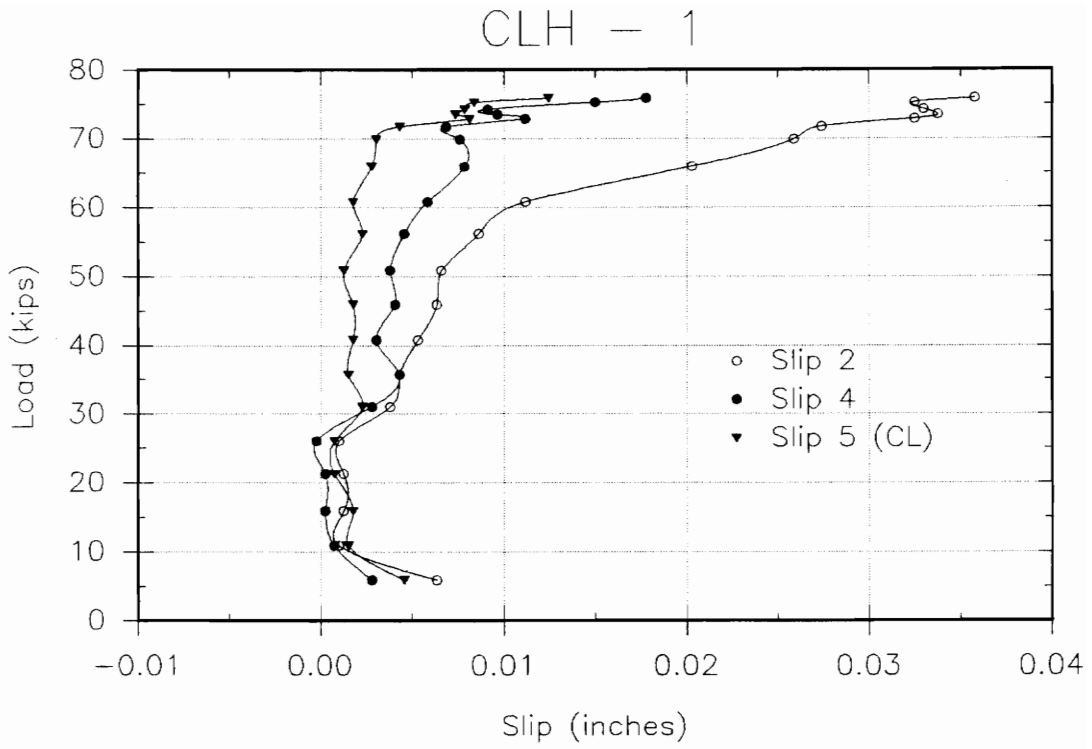


Figure B.1.15 Applied Load vs. Slip

TEST SUMMARY SHEET

TEST DESIGNATION: CLH-2 TEST DATE: 4 October, 1989
 TEST LOCATION: Price's Fork Research Park

JOIST DESCRIPTION

Depth: 36 in Top Chord: 2L 2.5x2.5x0.313
 Length: 56 ft Bottom Chord: 2L 3.5x3.5x0.313
 Number of Studs: 38 Width of Slab: 102 in
 Dead Weight of Joist and Slab: 516 plf

CALCULATED CAPACITIES (using measured material properties)

Theoretical Max Applied Load: 87.8 kips
 Theoretical Moment of Inertia: 4878 in⁴
 Theoretical Elastic Deflection: 2.46 in

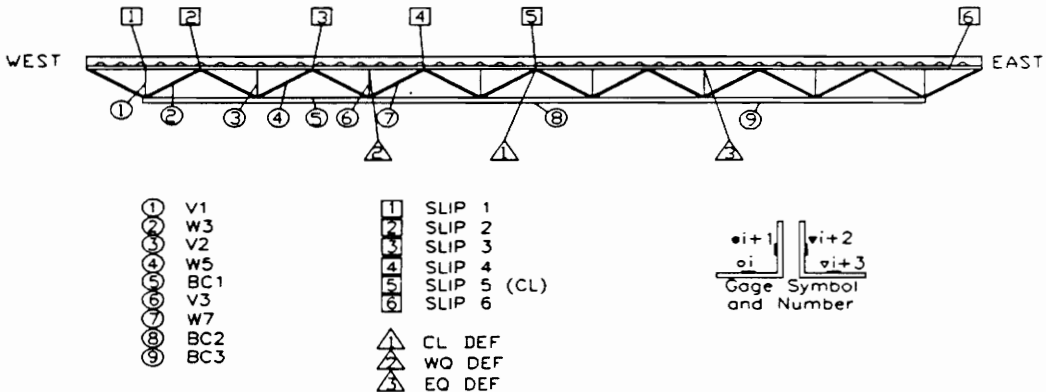
TEST RESULTS

Max Applied Load: 85.3 kips Max Deflection: 13.60 in
 Yield Load: 68.8 kips Deflection at Yield: 2.39 in
 Average Max Bottom Chord Strain: 1910 micro strain
 Bottom Chord Fy: 57.81 ksi f'c: 4.18 ksi
 Concrete Pour Strains Recorded: NO
 Maximum Slip: 0.042 in
 Failure: Excessive deflection.

COMPARISON OF ACTUAL TO THEORETICAL

Actual stiffness/theoretical stiffness: 0.887
 Actual Max Load/Theoretical Max Load: 0.972

Strain Gage Locations:



Comments: The reinforced vertical members did not buckle. This enabled the specimen to withstand larger deflections. The test was stopped when specimen touched the bottom of the testing frame. The gages were zeroed with the loading beams in place (5.9 kips applied load).

CLH - 2

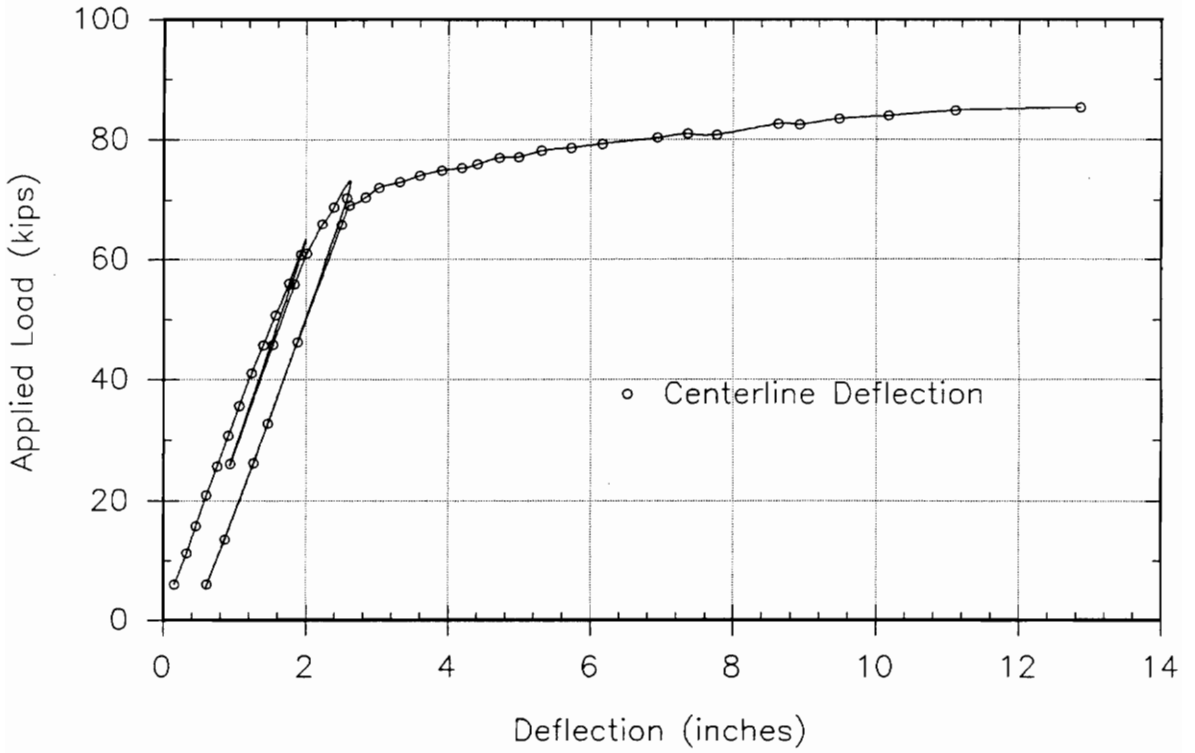


Figure B.2.1 Applied Load vs. Centerline Deflection

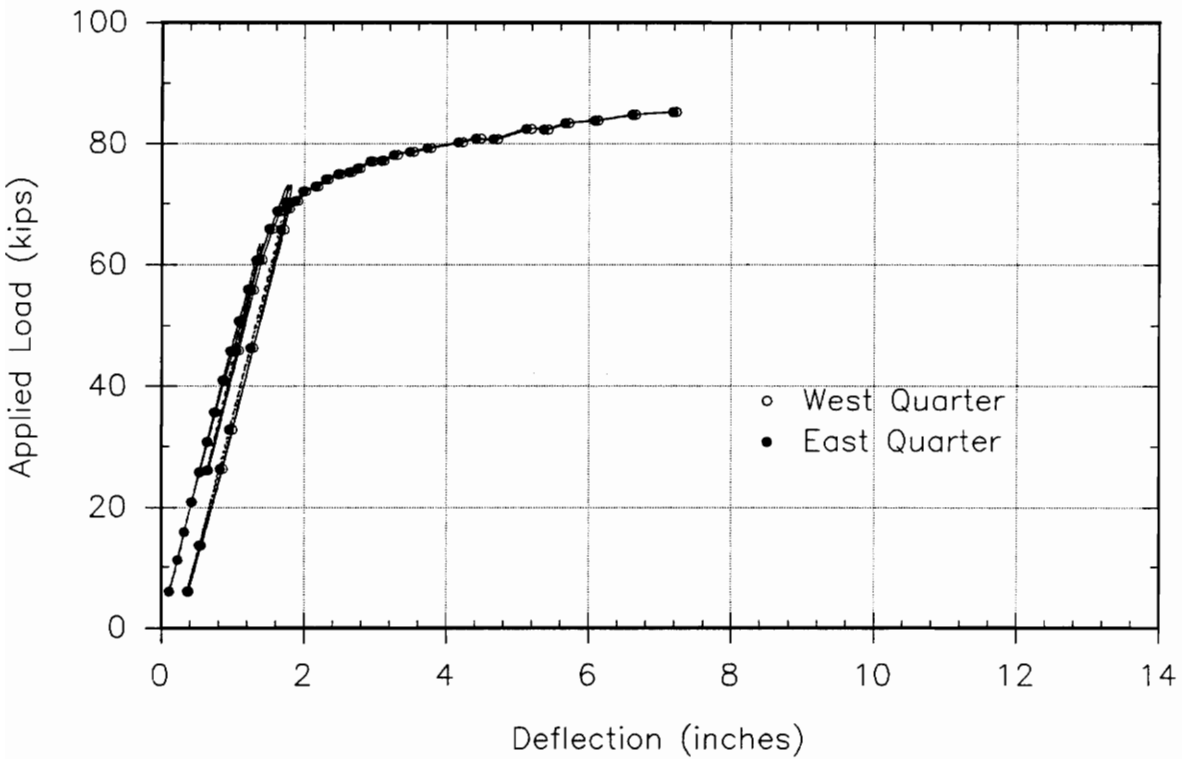


Figure B.2.2 Applied Load vs. Quarter Point Deflection

CLH - 2

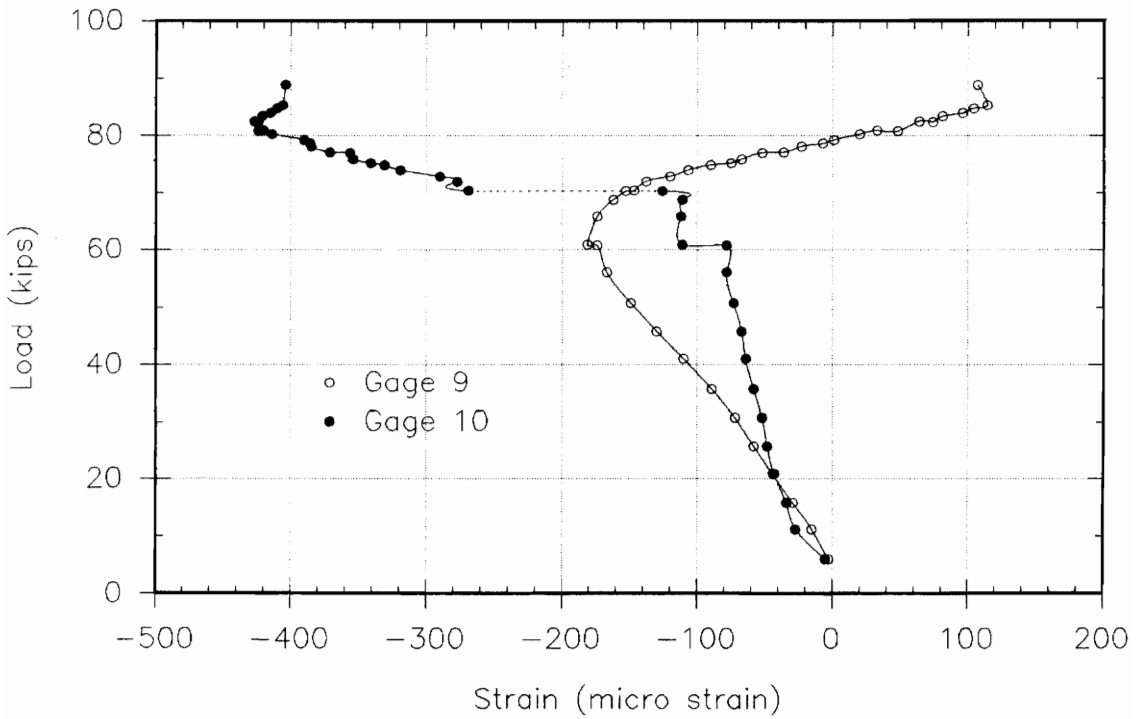


Figure B.2.3 Applied Load vs. V1 Strain

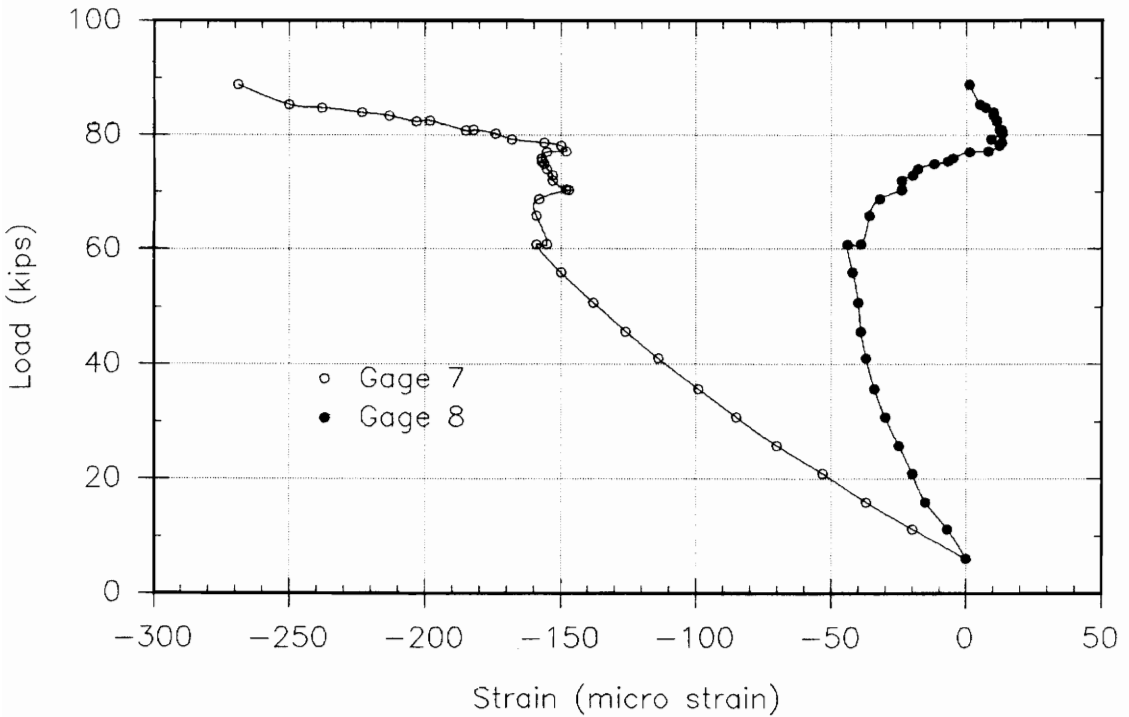


Figure B.2.4 Applied Load vs. V2 Strain

CLH - 2

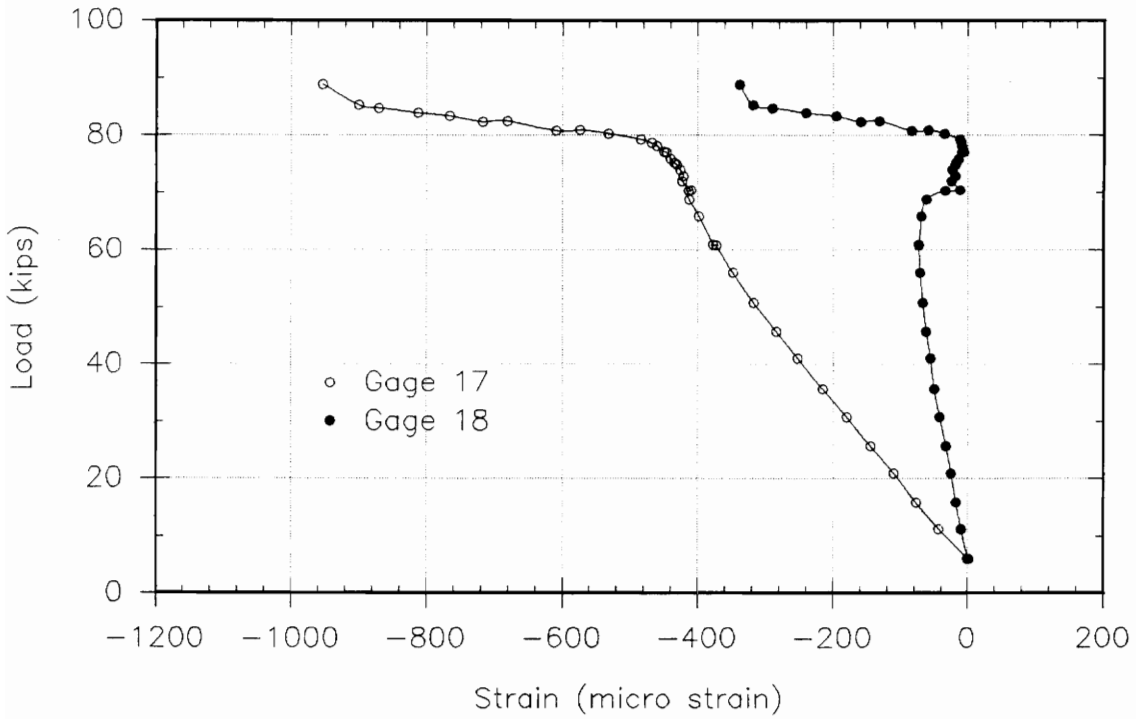


Figure B.2.5 Applied Load vs. V3 Strain

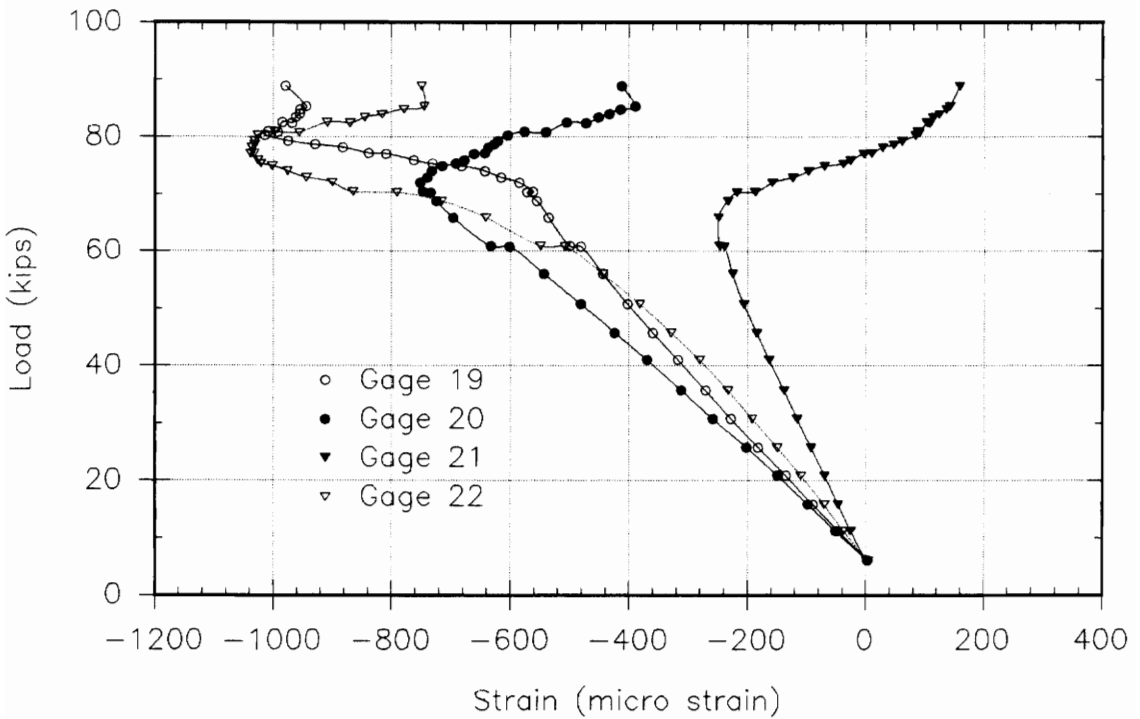


Figure B.2.6 Applied Load vs. W7 Strain

CLH - 2

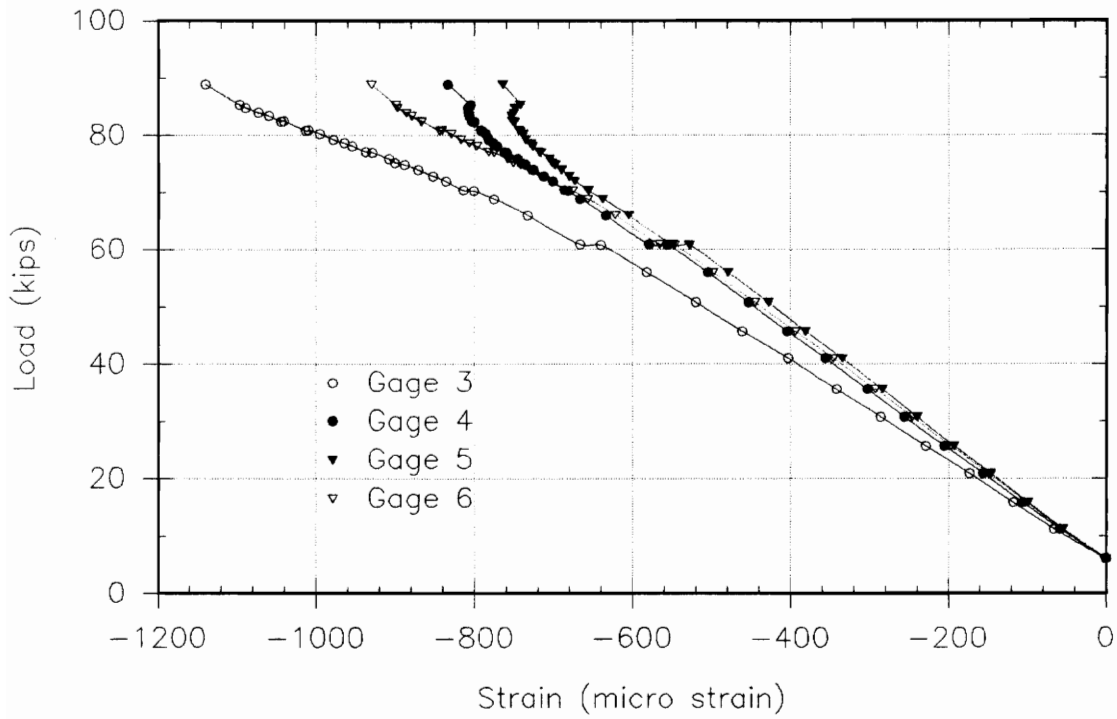


Figure B.2.7 Applied Load vs. W3 Strain

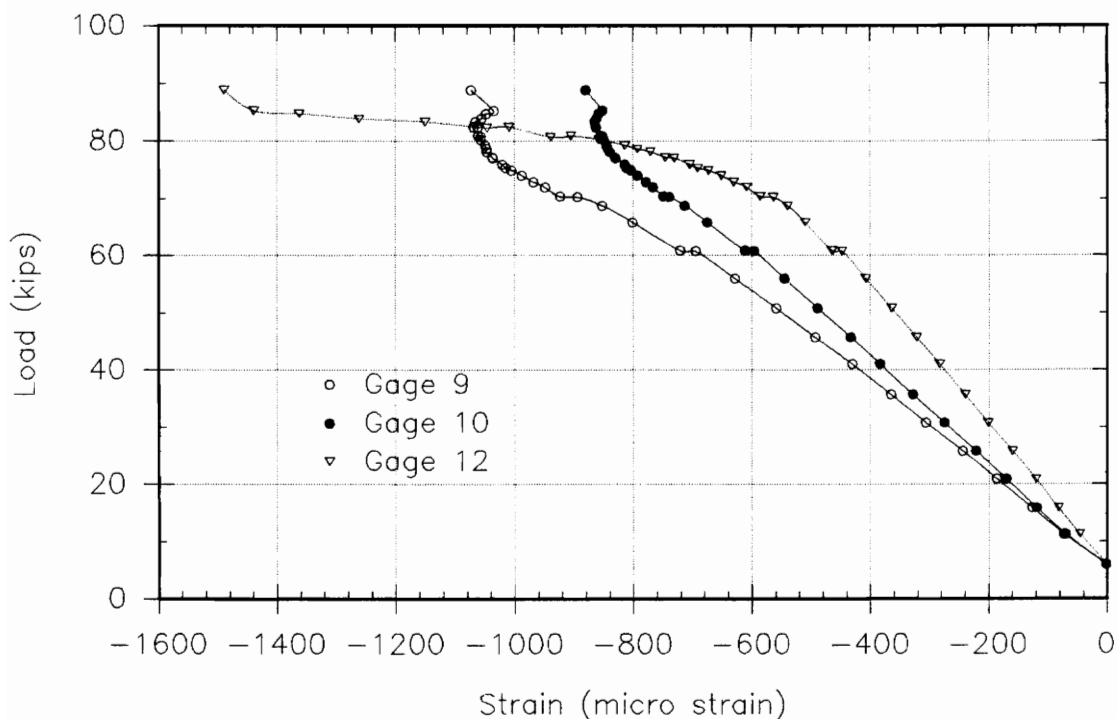


Figure B.2.8 Applied Load vs. W5 Strain

CLH - 2

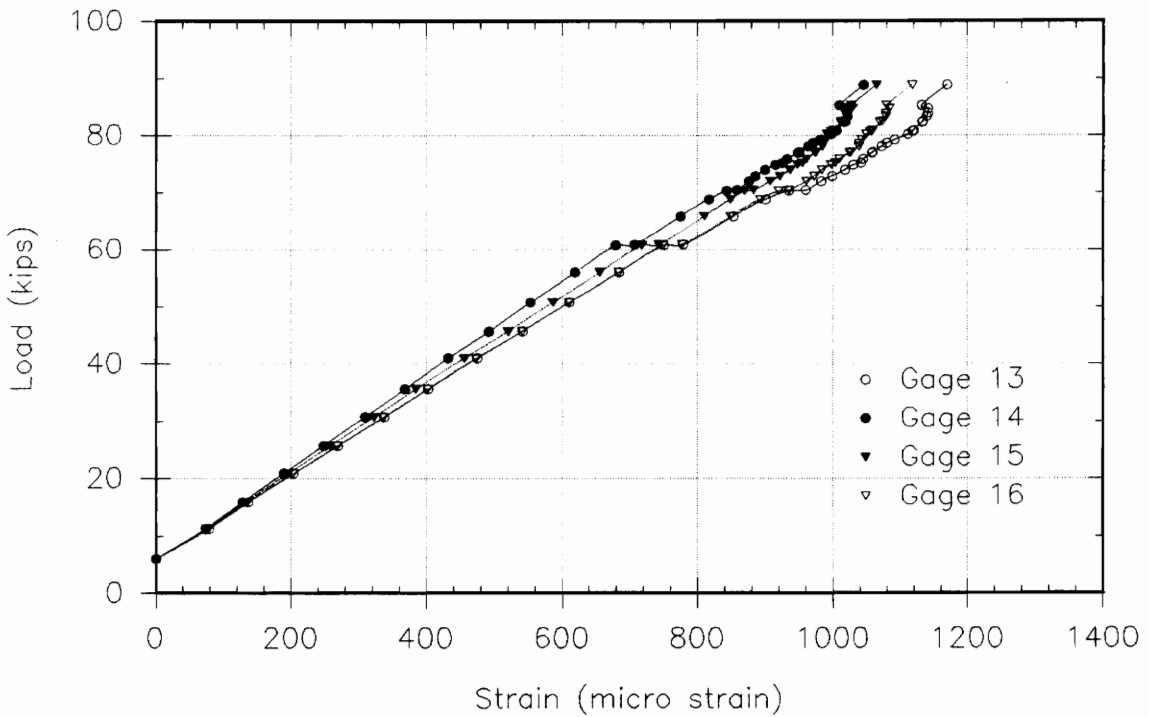


Figure B.2.9 Applied Load vs. Bottom Chord Strain (BC1)

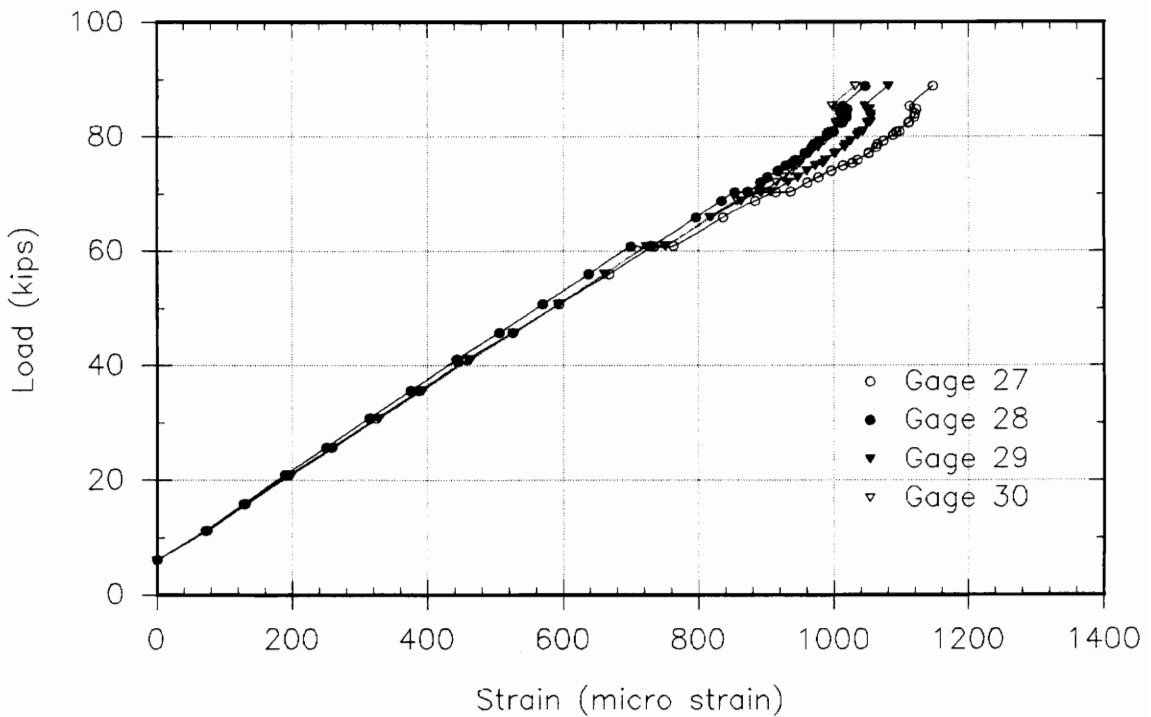


Figure B.2.10 Applied Load vs. Bottom Chord Strain (BC3)

CLH - 2

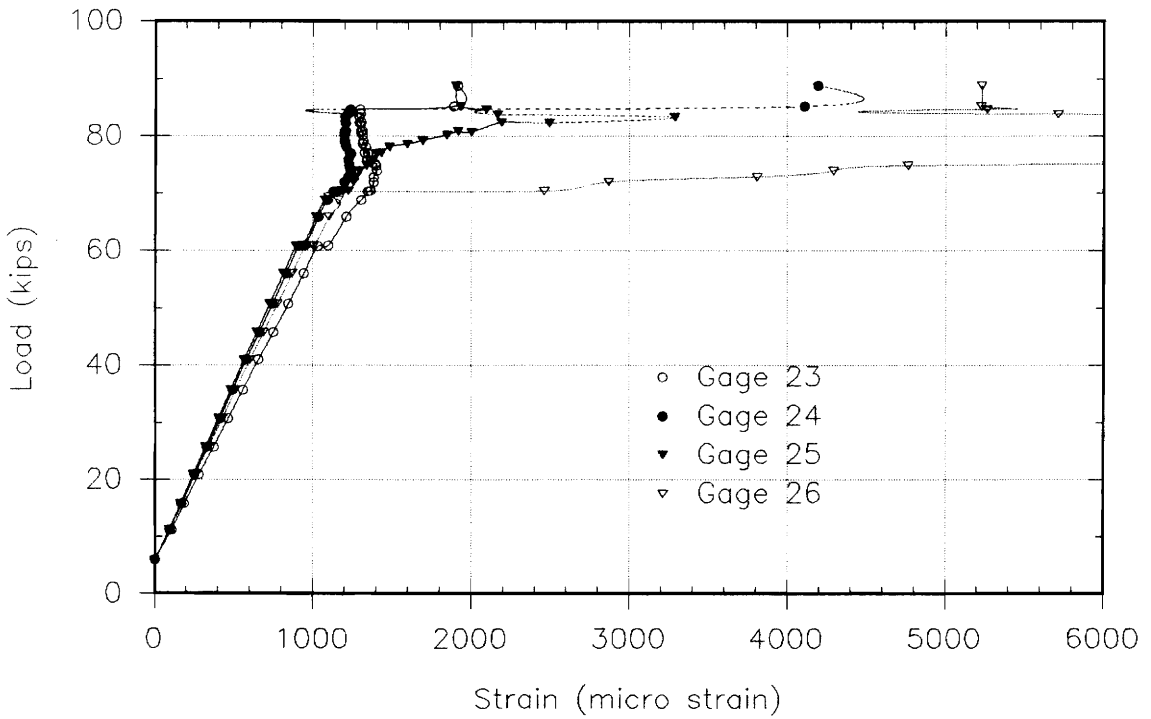


Figure B.2.11 Applied Load vs. Bottom Chord Strain (BC2)

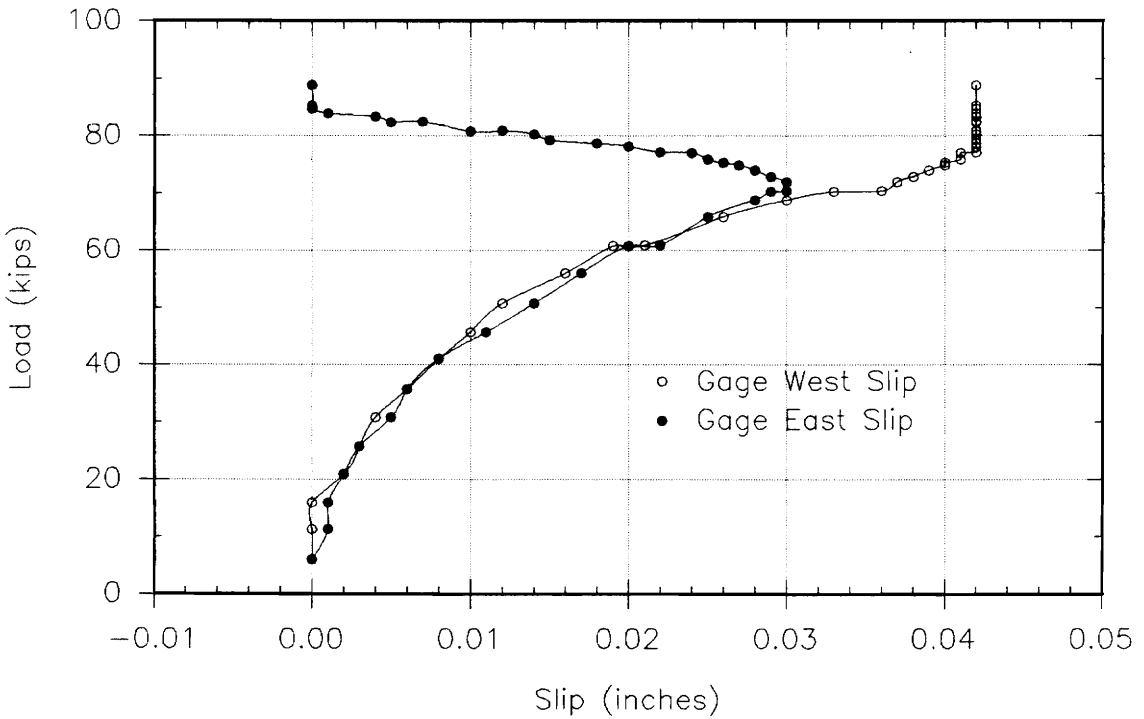


Figure B.2.12 Applied Load vs. End Slip

CLH - 2

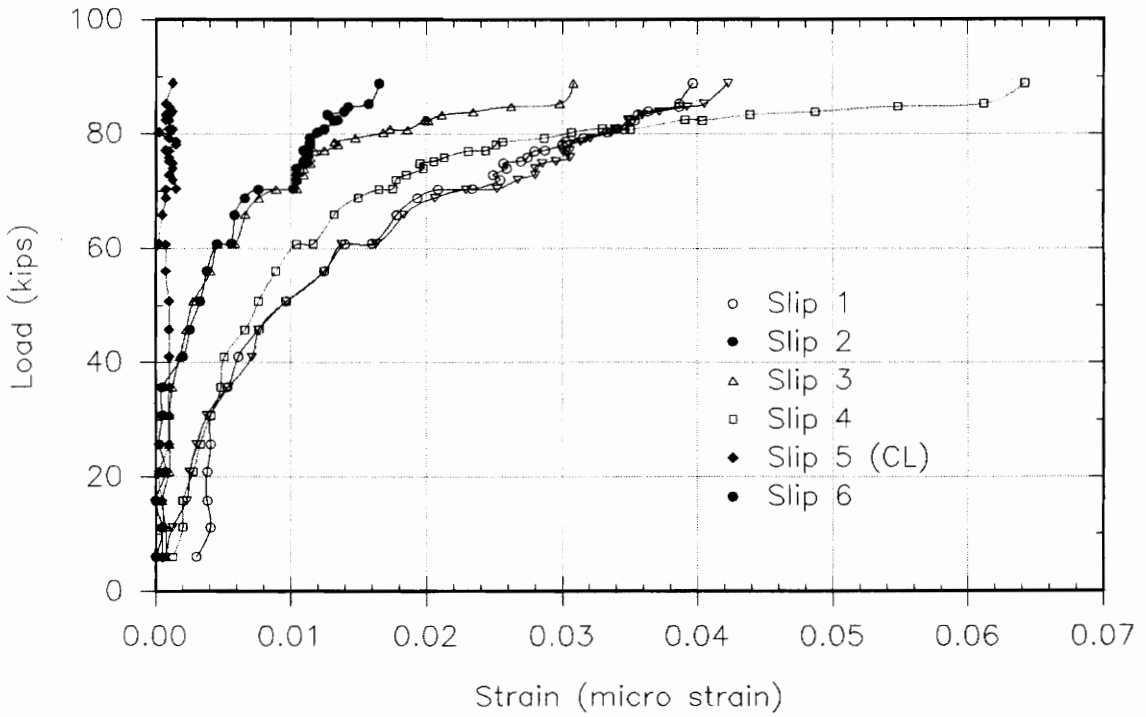


Figure B.2.13 Applied Load vs. Slip

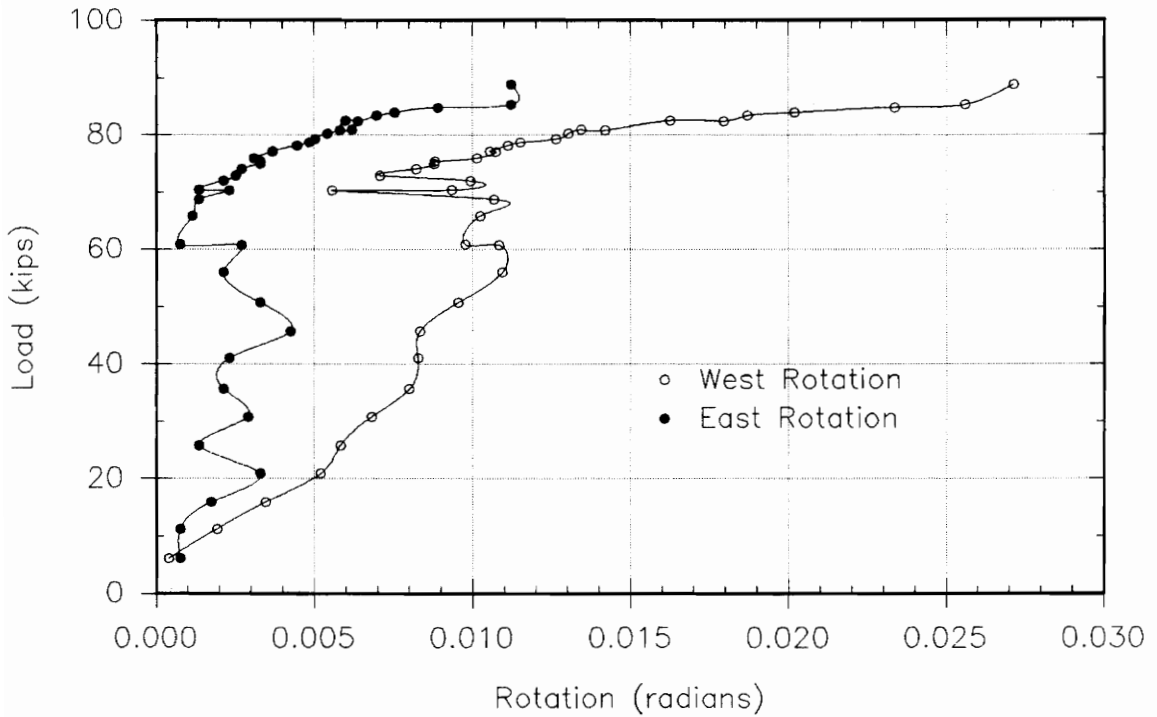


Figure B.2.14 Applied Load vs. End Rotation

TEST SUMMARY SHEET

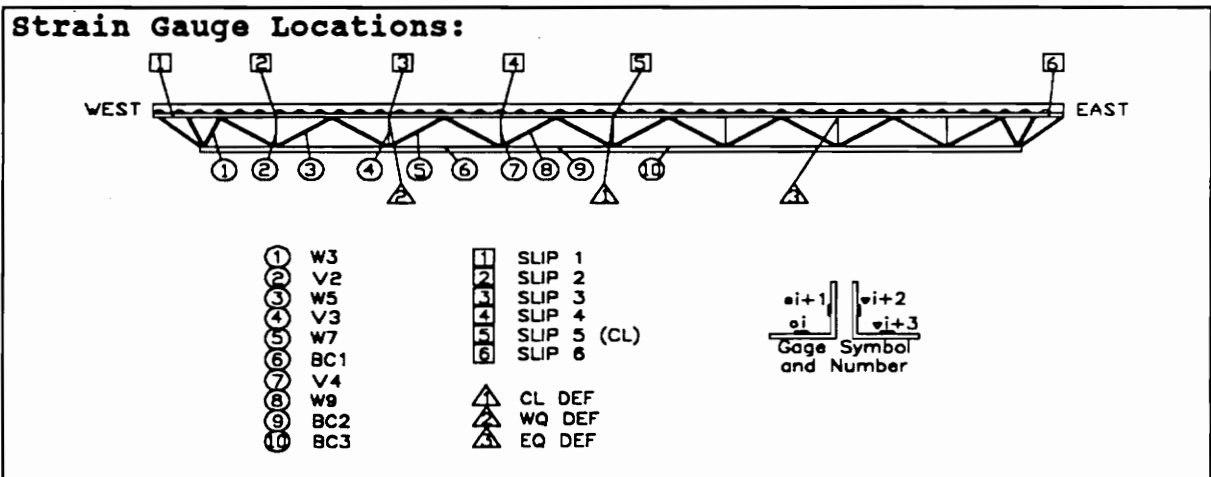
TEST DESIGNATION: CLH-3 TEST DATE: 18 January, 1990
 TEST LOCATION: Structures and Materials Research Laboratory

JOIST DESCRIPTION	
Depth: <u>16 in</u>	Top Chord: <u>2L 3.5x3.5x0.313</u>
Length: <u>40 ft</u>	Bottom Chord: <u>2L 5x5x0.438</u>
Number of Studs: <u>66</u>	Width of Slab: <u>81 in</u>
Dead Weight of Joist and Slab: <u>423 plf</u>	

CALCULATED CAPACITIES (using measured material properties)
Theoretical Max Applied Load: <u>127.0 kips</u>
Theoretical Moment of Inertia: <u>2045 in⁴</u>
Theoretical Elastic Deflection: <u>3.10 in</u>

TEST RESULTS
Max Applied Load: <u>128.1 kips</u> Max Deflection: <u>11.96 in</u>
Yield Load: <u>110.3 kips</u> Deflection at Yield: <u>2.95 in</u>
Average Max Bottom Chord Strain: <u>1310 micro strain</u>
Bottom Chord F _y : <u>52.11 ksi</u> f' _c : <u>4.00 ksi</u>
Concrete Pour Strains Recorded: <u>NO</u>
Maximum Slip: <u>0.324 in</u>
Failure: <u>Buckling of members W10 and W10R.</u>

COMPARISON OF ACTUAL TO THEORETICAL
Actual Stiffness/Theoretical Stiffness: <u>0.994</u>
Actual Max Load/Theoretical Max Load: <u>0.961</u>



Comments: This test was characterized by deformation of the web. The top and bottom chords moved closer together. This caused members W10 and W10R (tension members) to buckle. The gauges were zeroed at no applied load.

CLH - 3

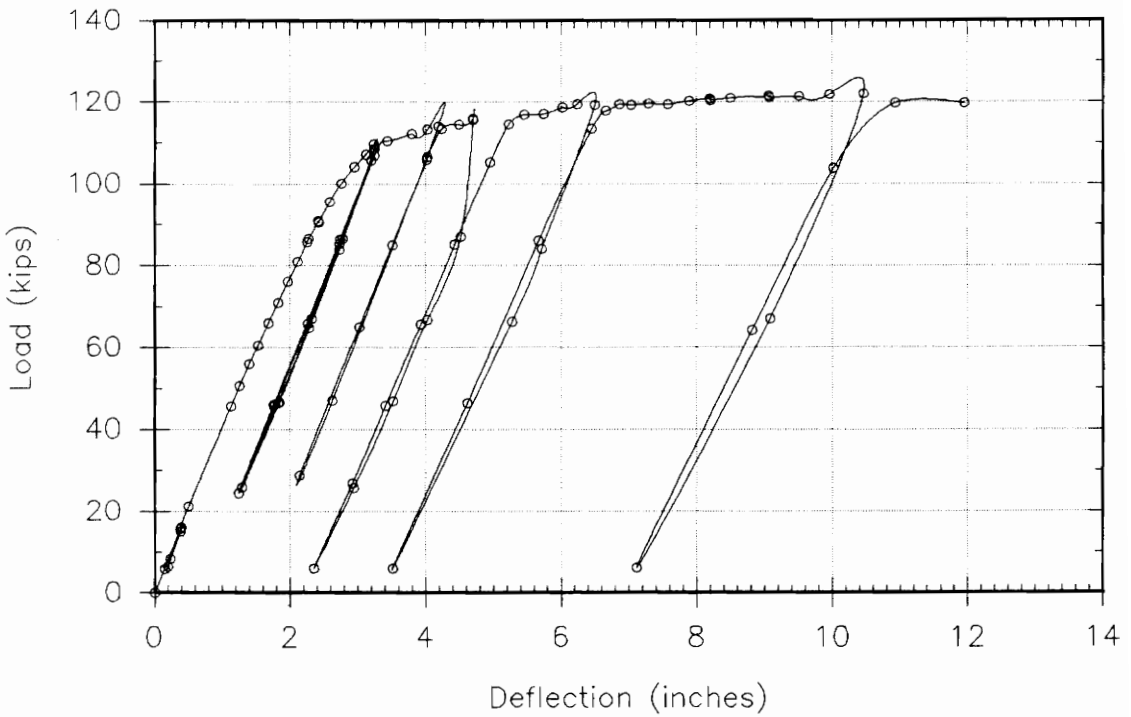


Figure B.3.1 Applied Load vs. Centerline Deflection

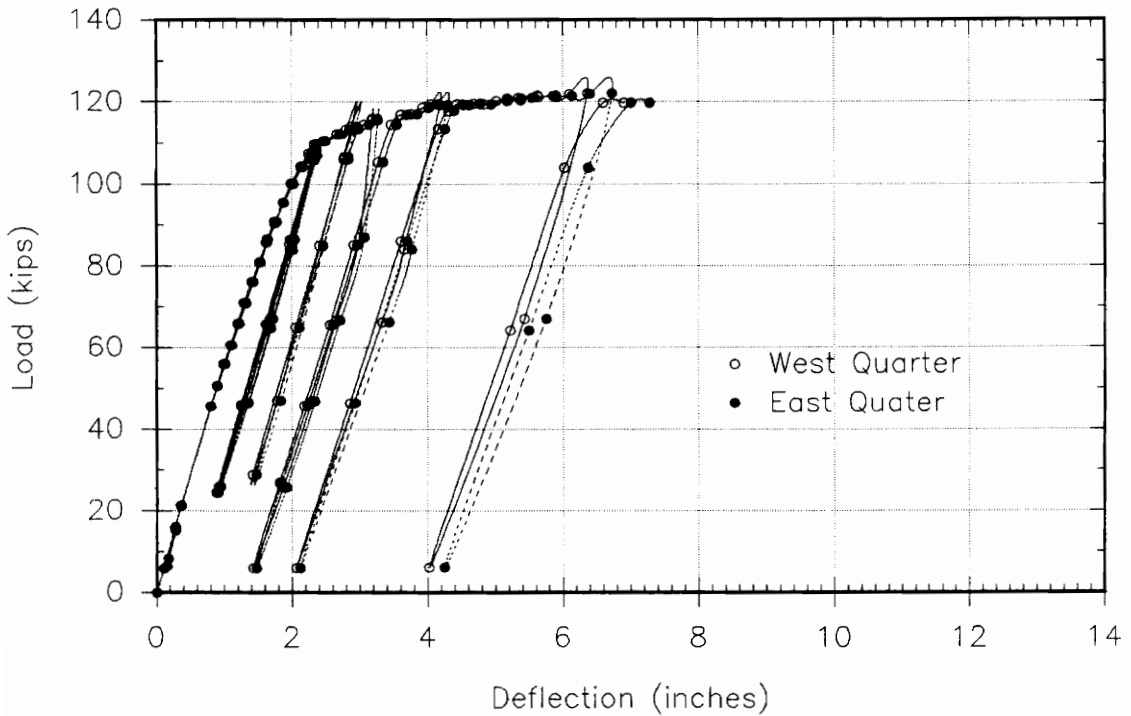


Figure B.3.2 Applied Load vs. Quarter Point Deflection

CLH - 3

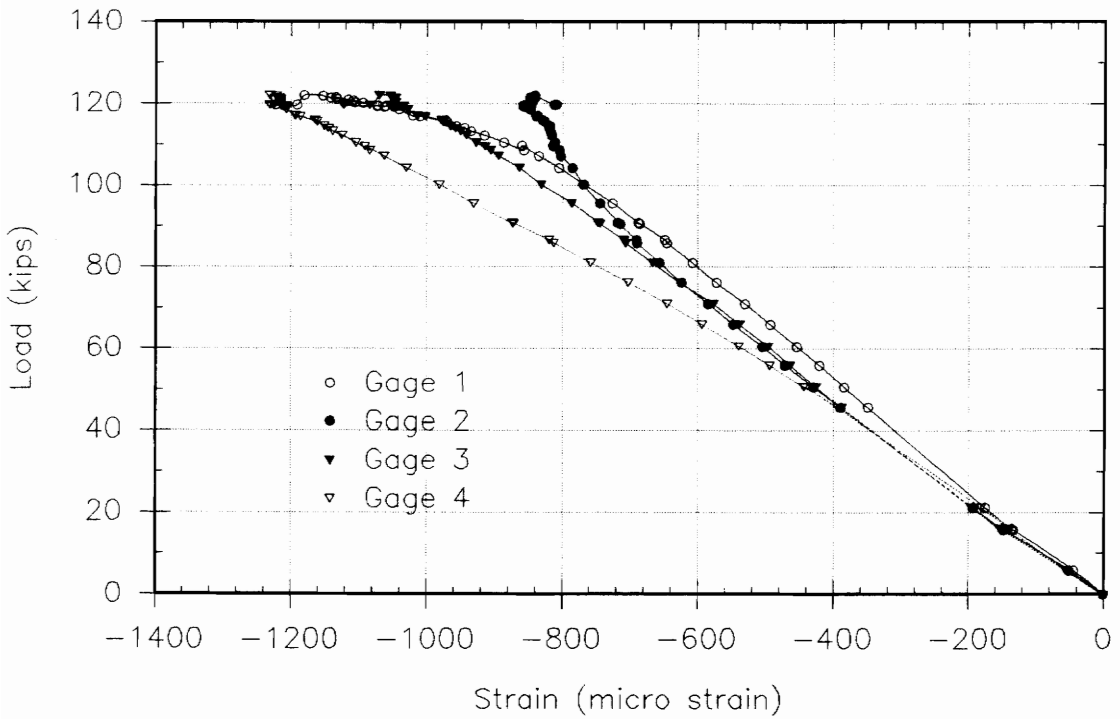


Figure B.3.3 Applied Load vs. W3 Strain

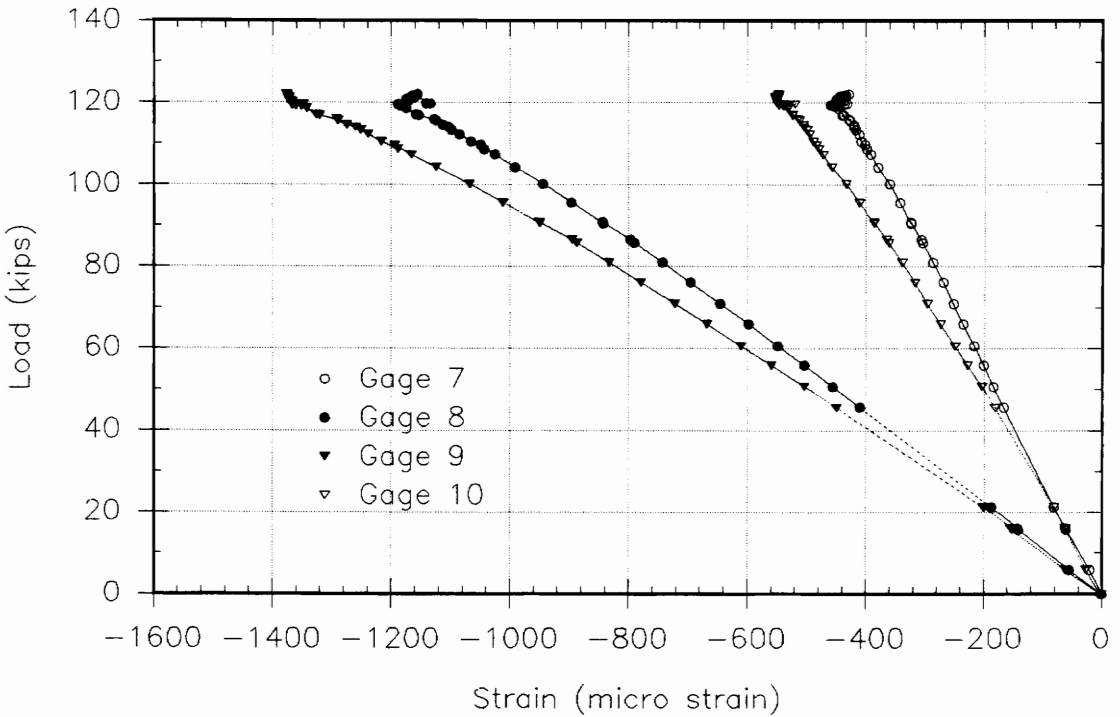


Figure B.3.4 Applied Load vs. W5 Strain

CLH - 3

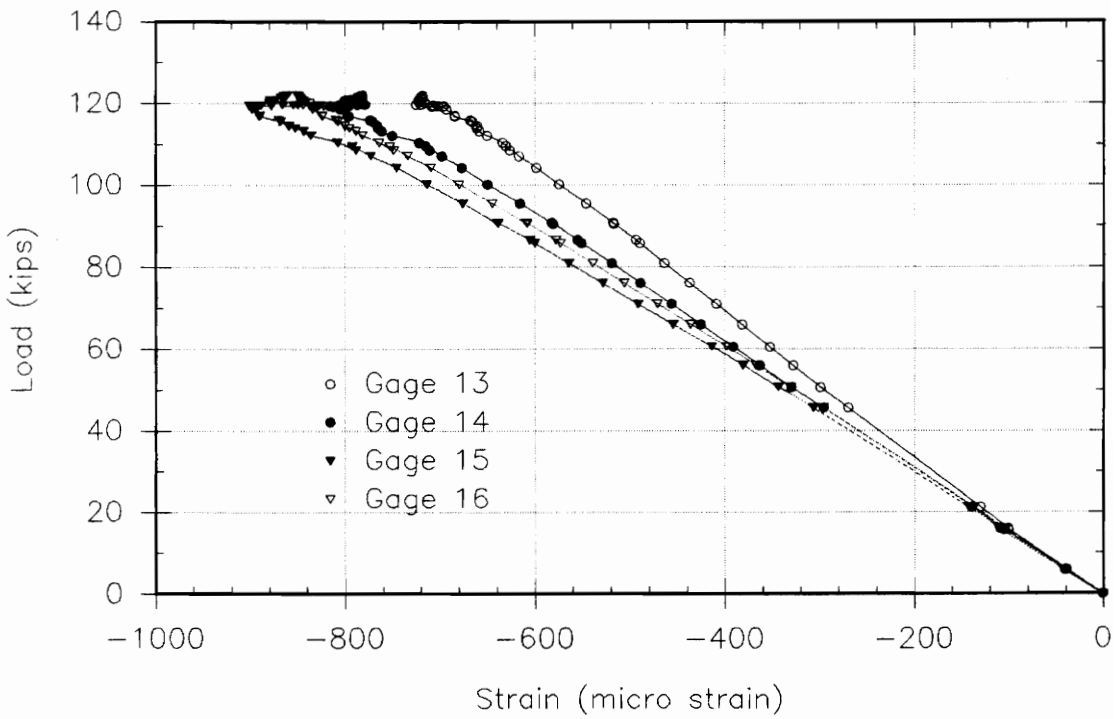


Figure B.3.5 Applied Load vs. W7 Strain

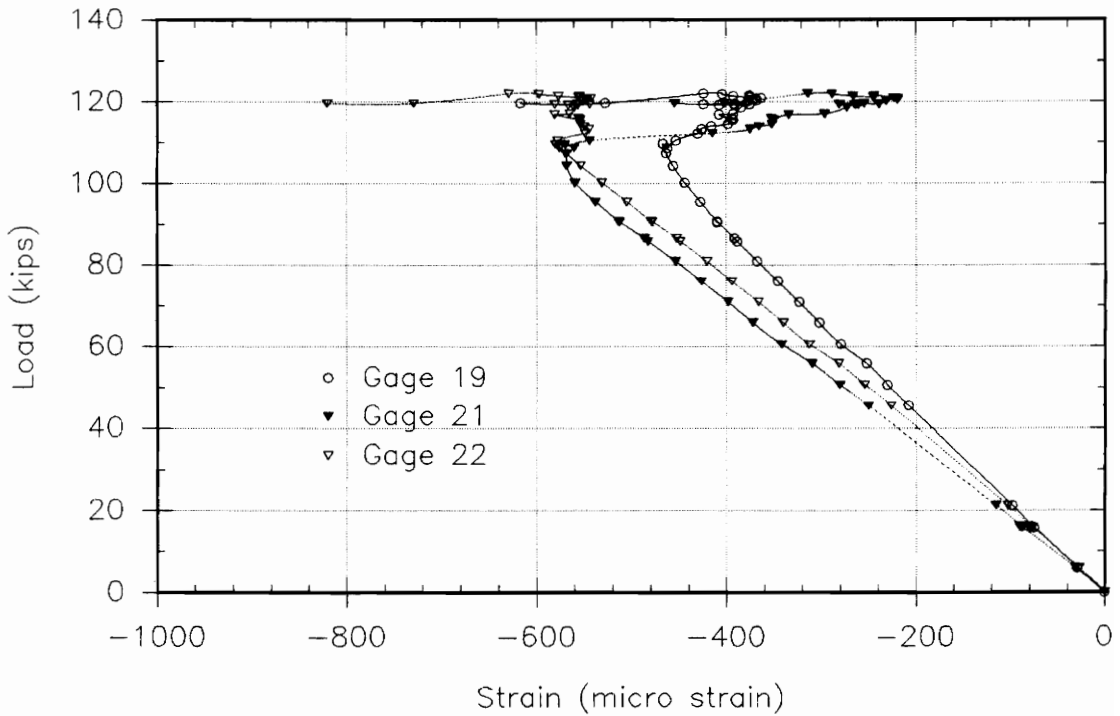


Figure B.3.6 Applied Load vs. W9 Strain

CLH - 3

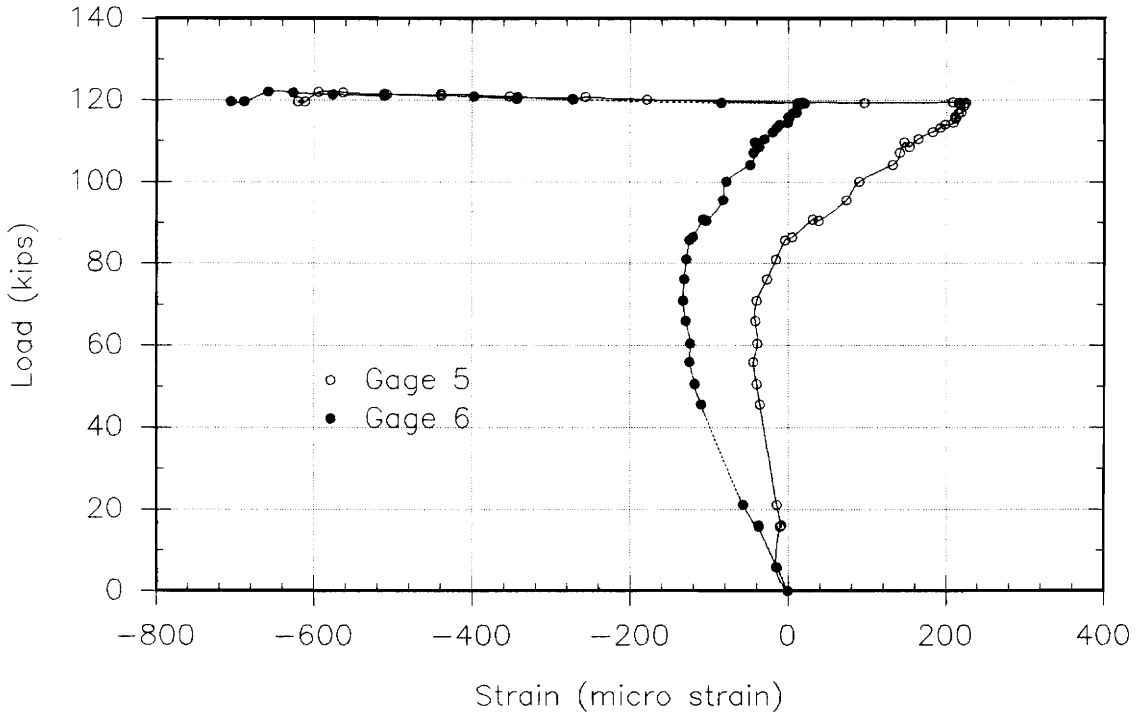


Figure B.3.7 Applied Load vs. V2 Strain

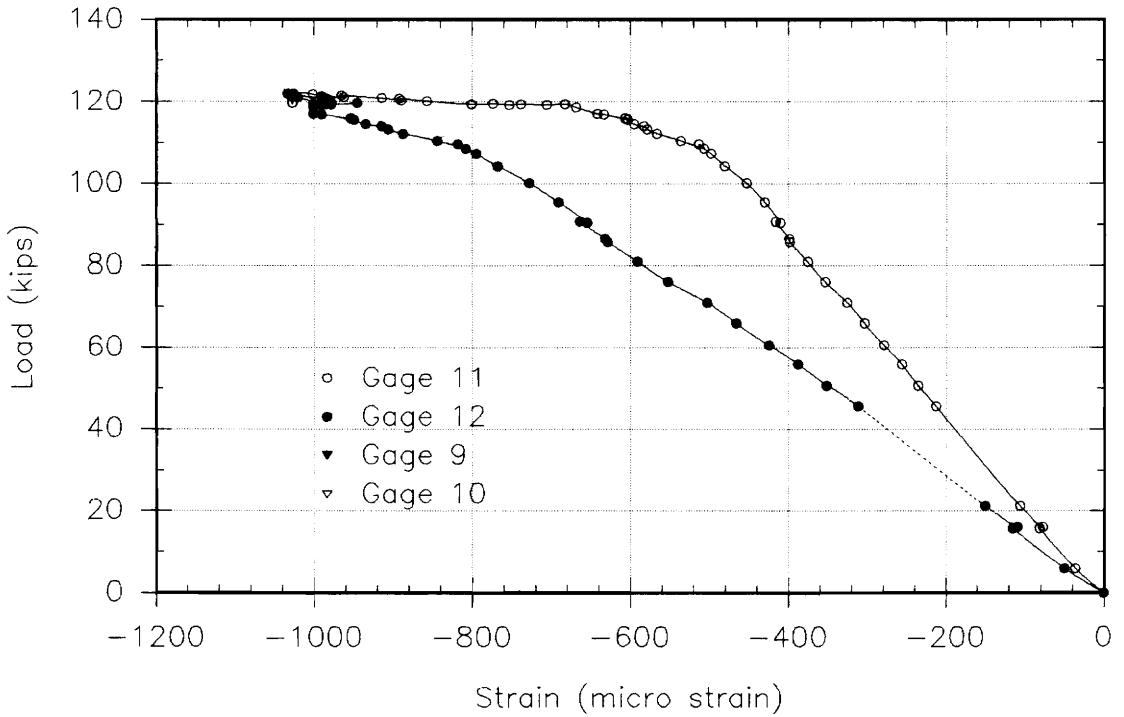


Figure B.3.8 Applied Load vs. V3 Strain

CLH - 3

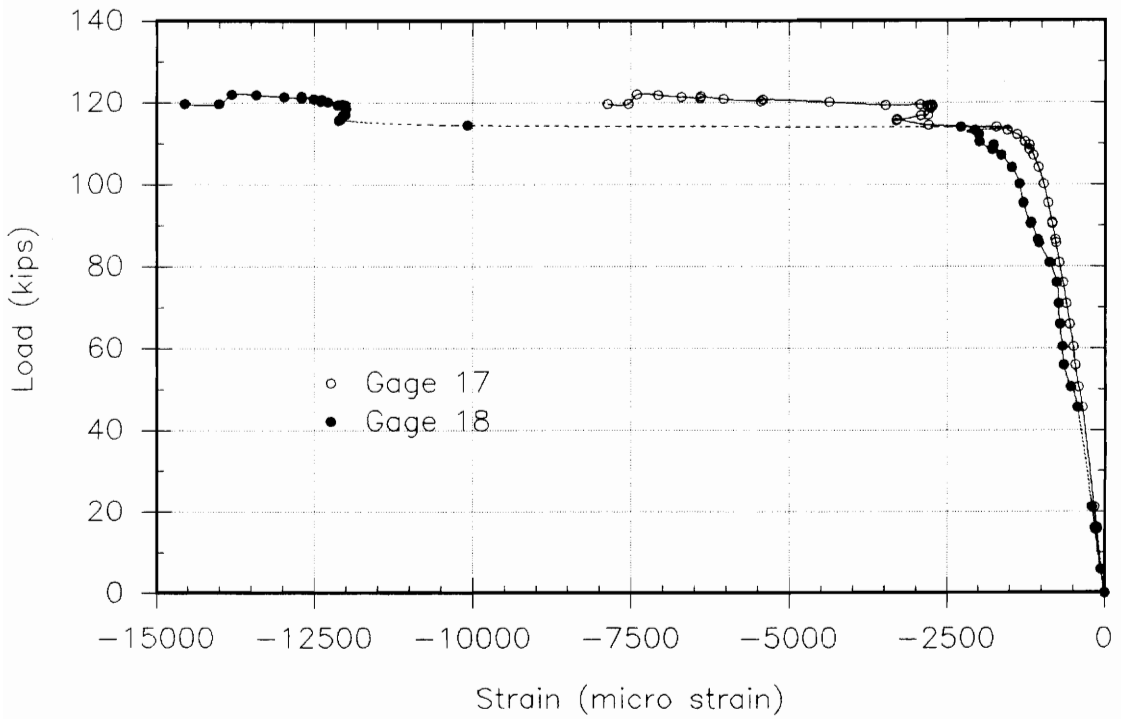


Figure B.3.9 Applied Load vs. V4 Strain

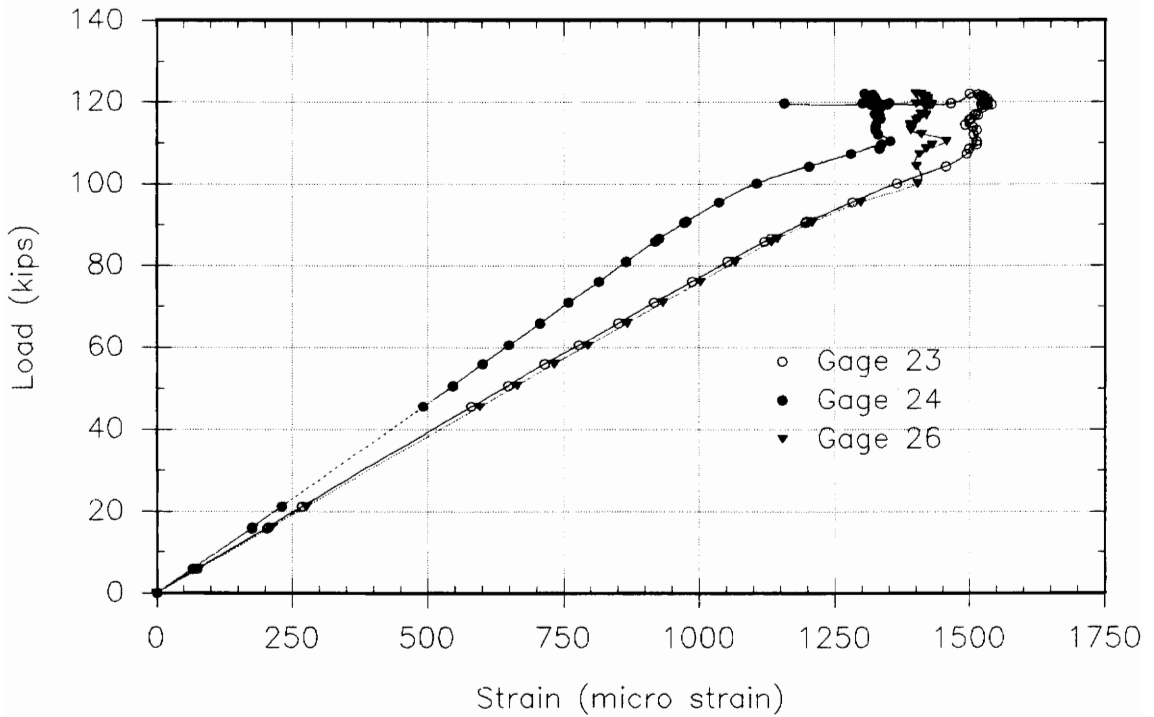


Figure B.3.10 Applied Load vs. Bottom Chord Strain (BC1)

CLH - 3

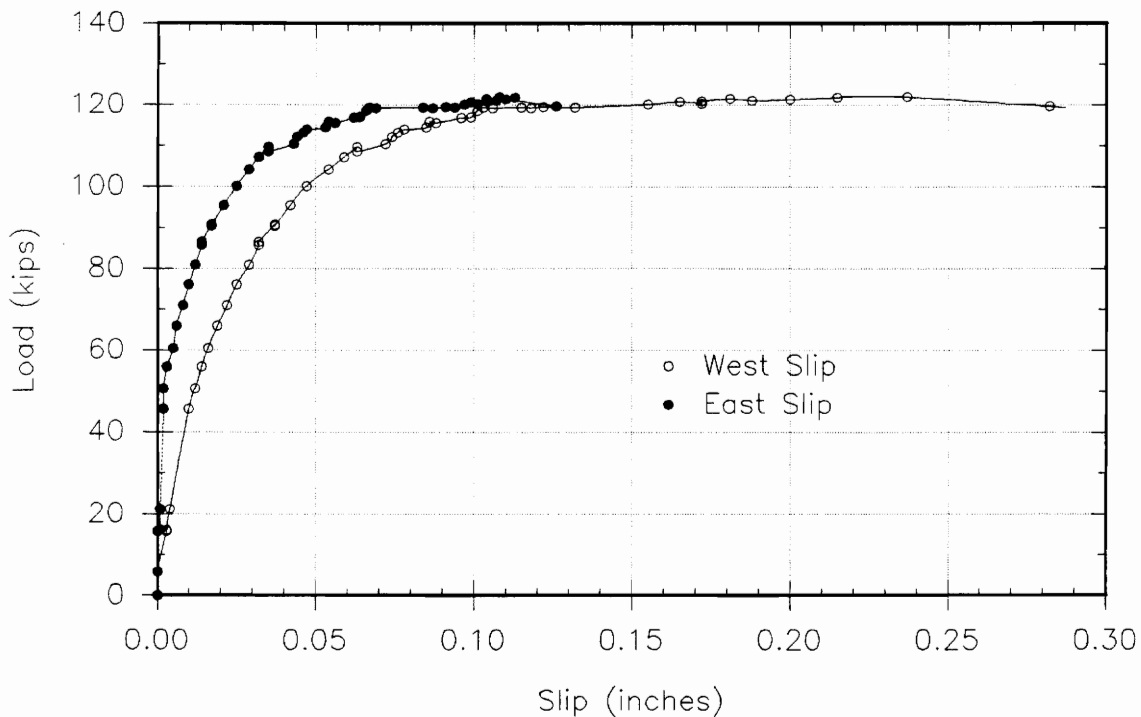


Figure B.3.11 Applied Load vs. End Slip

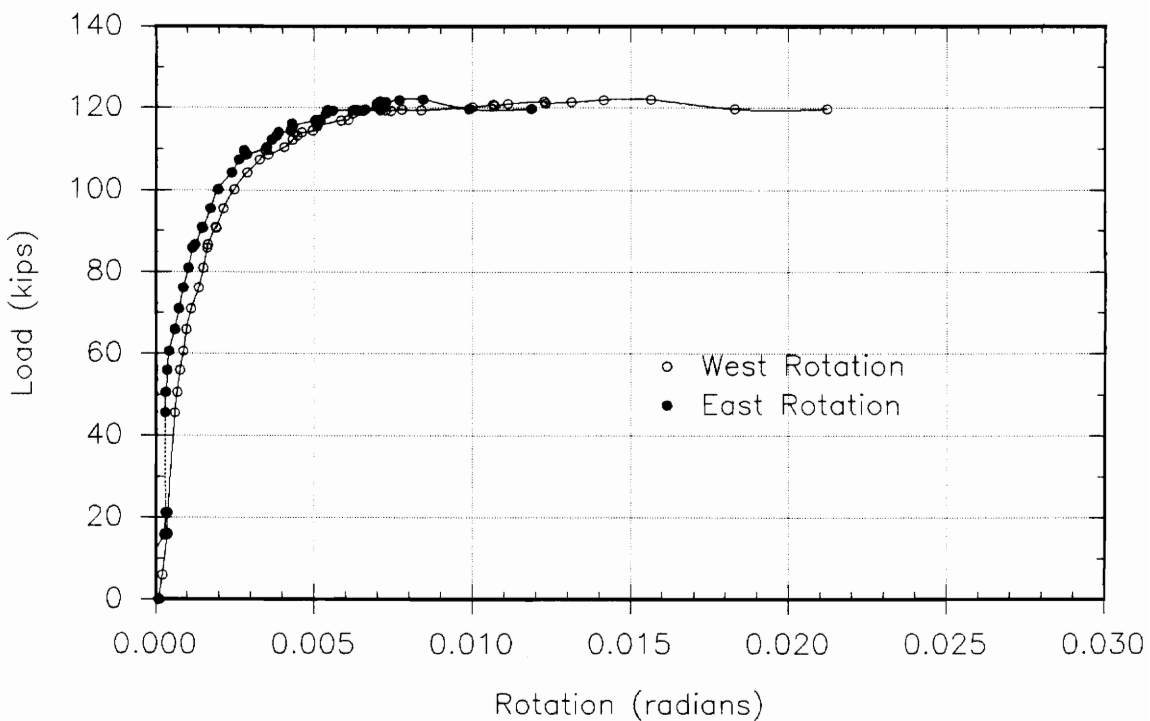


Figure B.3.12 Applied Load vs. End Rotation

CLH - 3

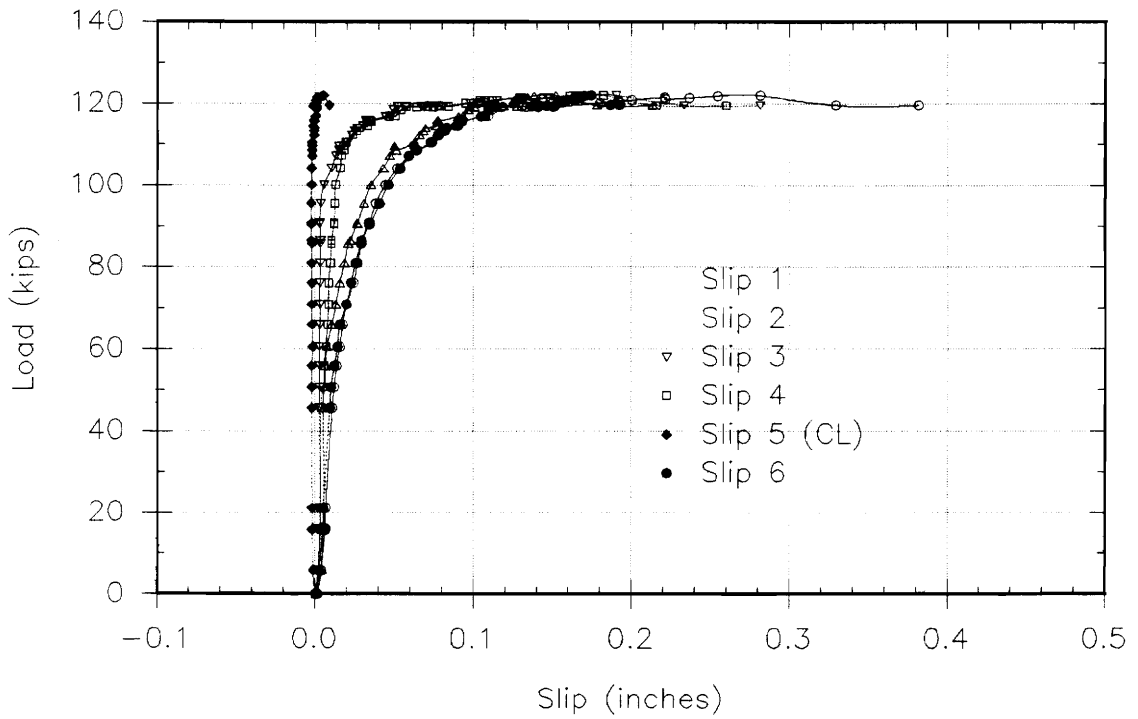


Figure B.3.13 Applied Load vs. Slip

TEST SUMMARY SHEET

TEST DESIGNATION: CLH-4 TEST DATE: 15 February, 1990
 TEST LOCATION: Structures and Materials Research Laboratory

JOIST DESCRIPTION

Depth: 16 in Top Chord: 2L 3.5x3.5x0.313
 Length: 40 ft Bottom Chord: 2L 5x5x0.438
 Number of Stud: 44 Width of Slab: 81 in
 Dead Weight of Joist and Slab: 423 plf

CALCULATED CAPACITIES (using measured material properties)

Theoretical Max Applied Load: 140.3 kips
 Theoretical Moment of Inertia: 1989 in⁴
 Theoretical Elastic Deflection: 3.53 in

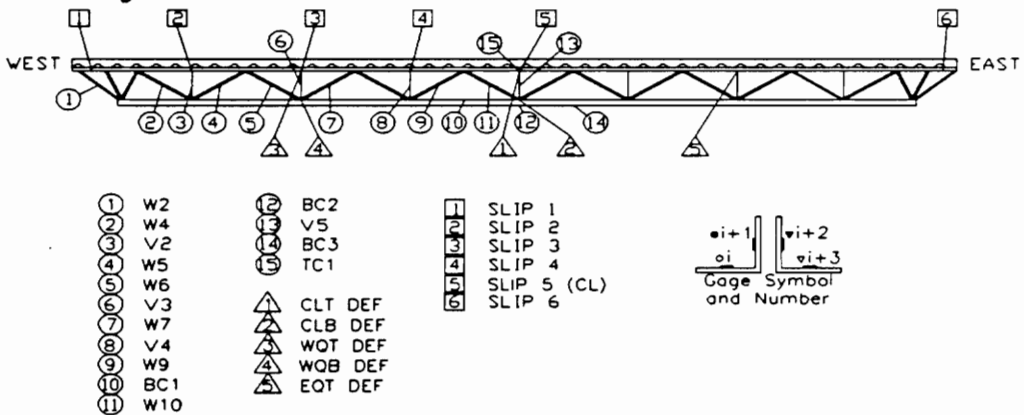
TEST RESULTS

Max Applied Load: 106.2 kips Max Deflection: 11.67 in
 Yield Load: 99.4 kips Deflection at Yield: 4.90 in
 Average Max Bottom Chord Strain: 1980 micro strain
 Bottom Chord Fy: 57.90 ksi f'c: 3.10 ksi
 Concrete Pour Strains Recorded: YES
 Maximum Slip: 3.597 in
 Failure: Local buckling of top chord.

COMPARISON OF ACTUAL TO THEORETICAL

Actual Stiffness/Theoretical Stiffness: 0.808
 Actual Max Load/Theoretical Max Load: 0.757

Strain Gauge Locations:



Comments: Failure was caused by the shear studs producing a localized moment in the top chord causing the top chord to buckle at the studs. Gauges were zeroed with no applied load. Some measurements were taken during the concrete placement.

CLH - 4

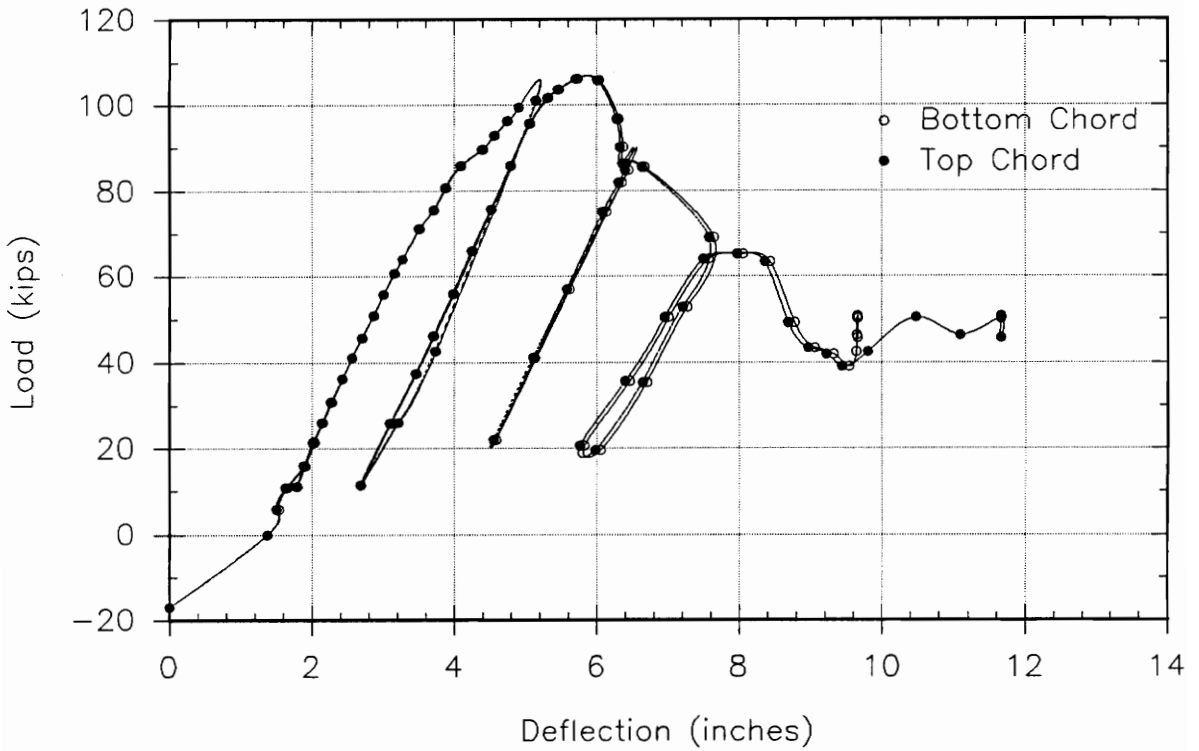


Figure B.4.1 Applied Load vs. Centerline Deflection

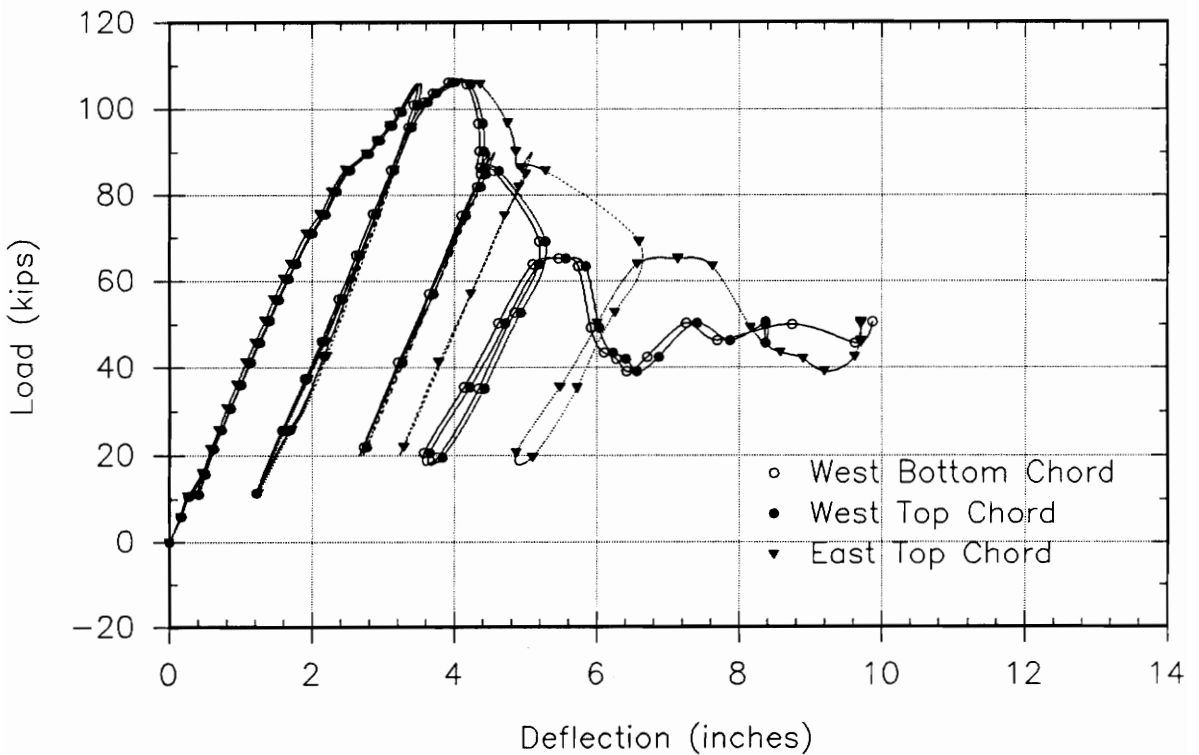


Figure B.4.2 Applied Load vs. Quarter Point Deflection

CLH - 4

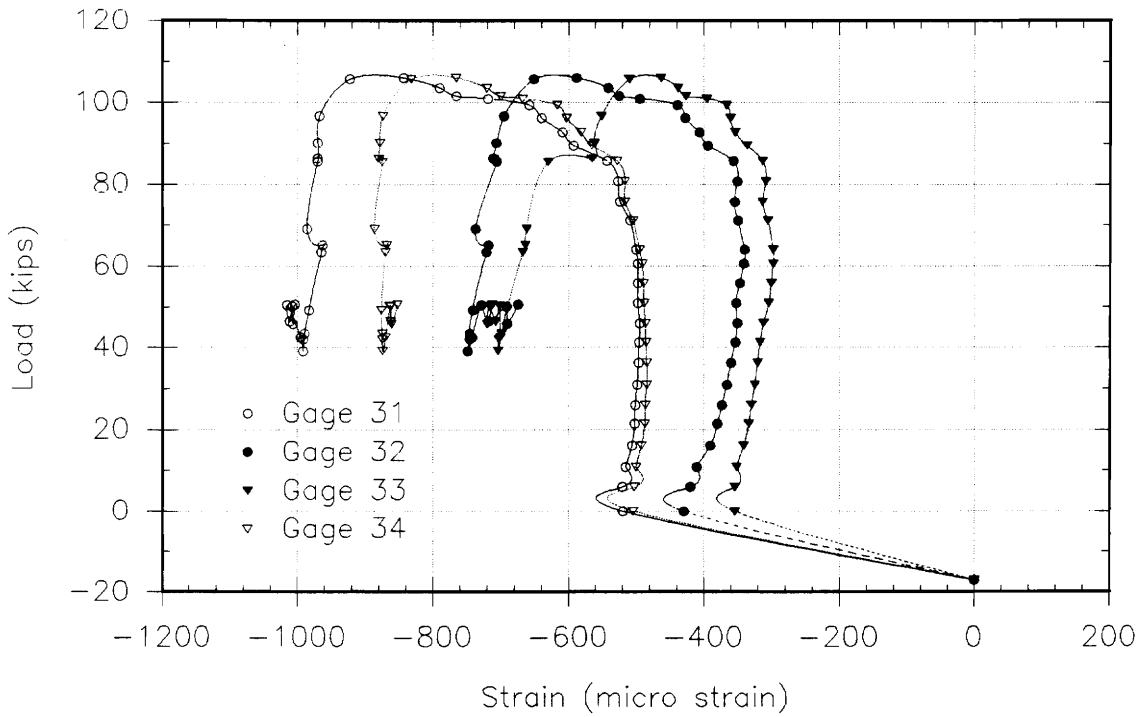


Figure B.4.3 Applied Load vs. Top Chord Strain (TC1)

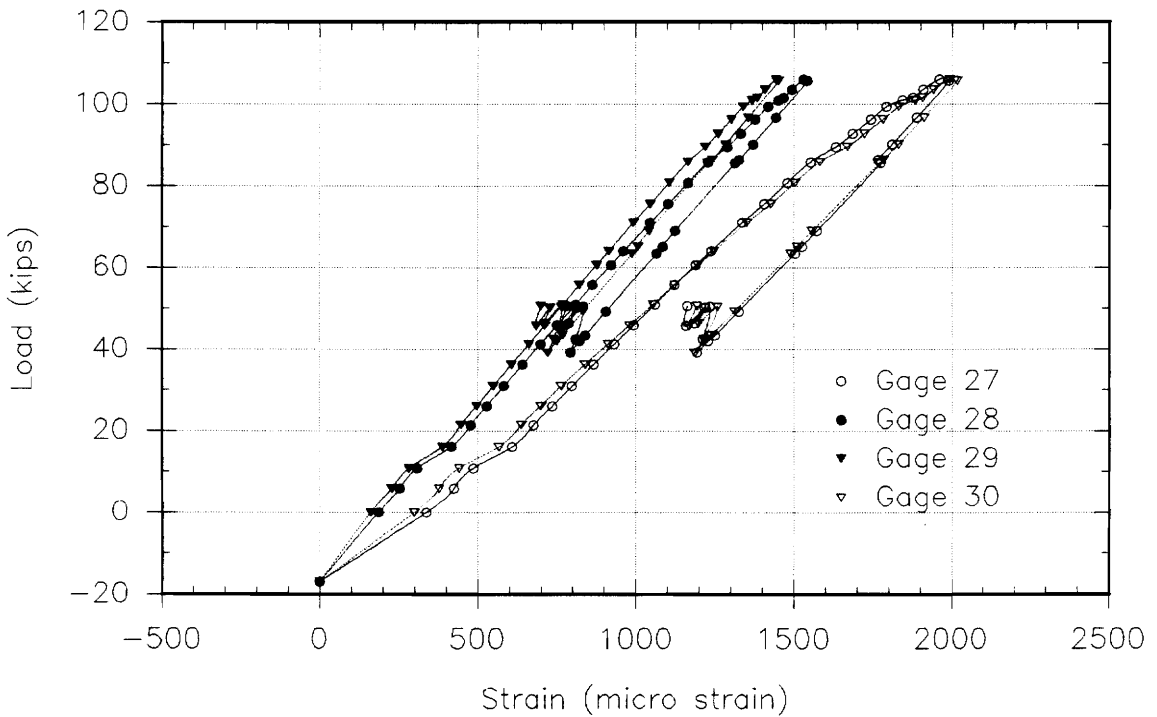


Figure B.4.4 Applied Load vs. Bottom Chord Strain (BC1)

CLH - 4

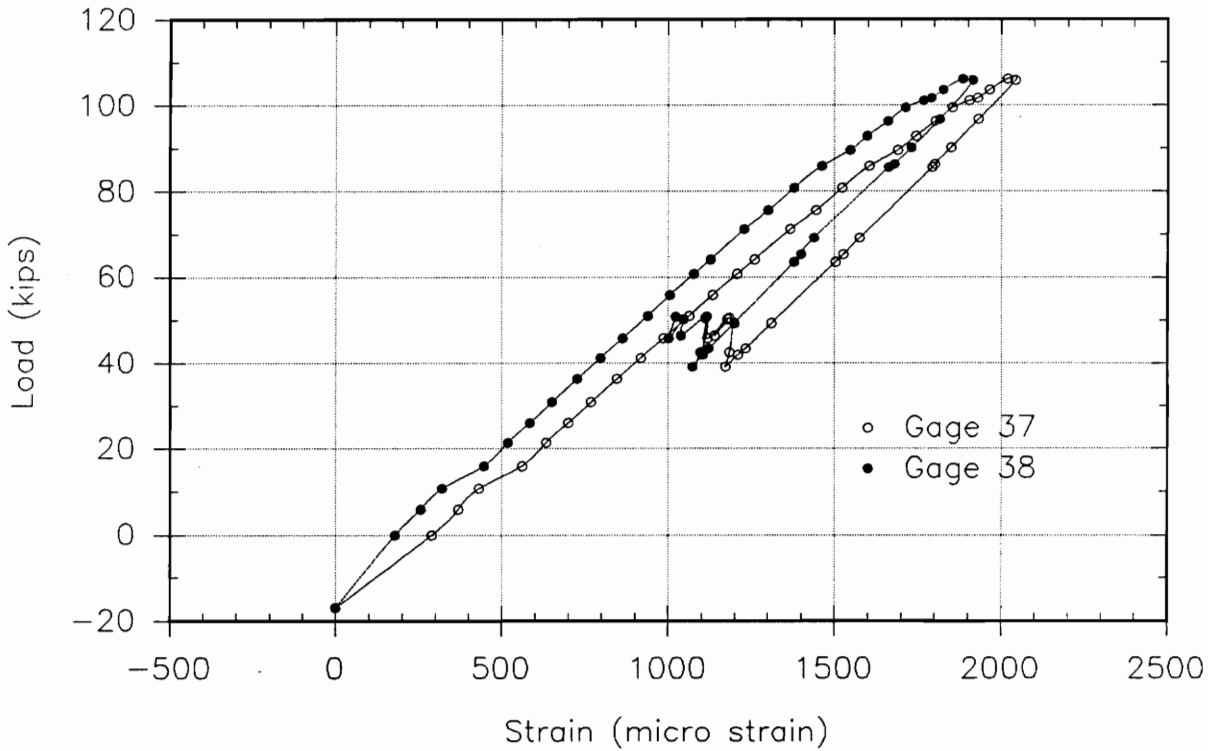


Figure B.4.5 Applied Load vs. Bottom Chord Strain (BC2)

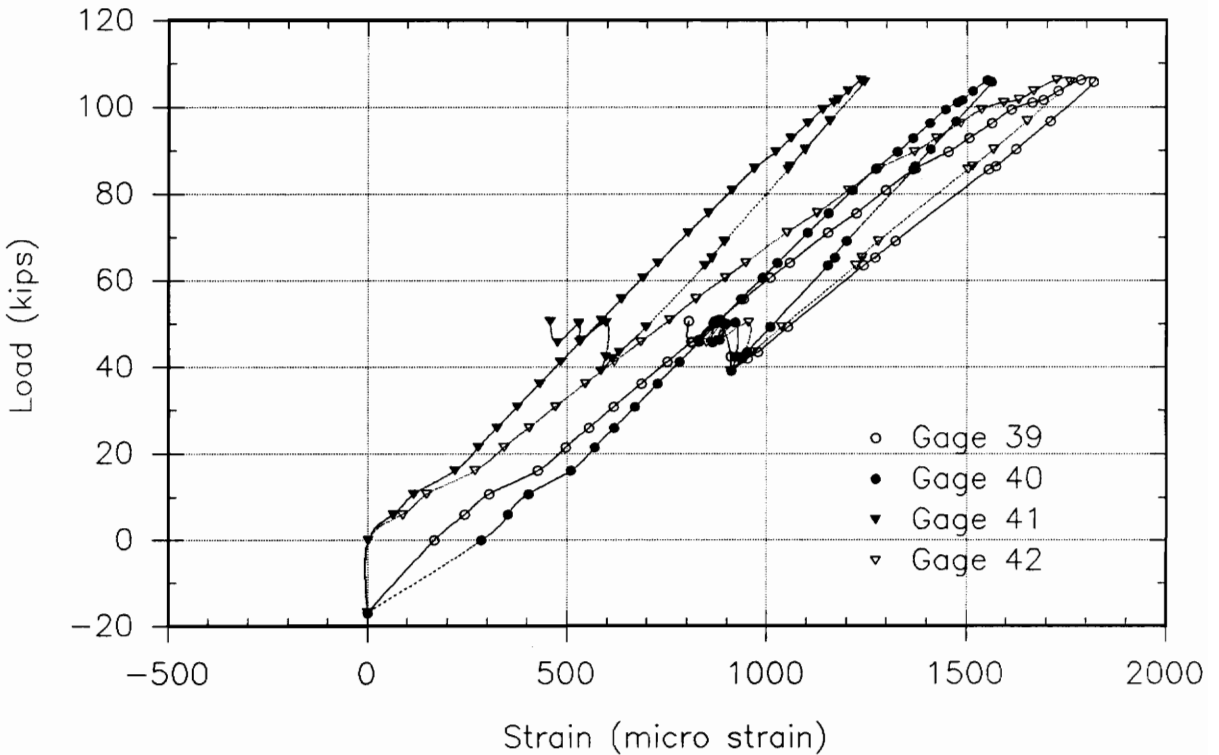


Figure B.4.6 Applied Load vs. Bottom Chord Strain (BC3)

CLH - 4

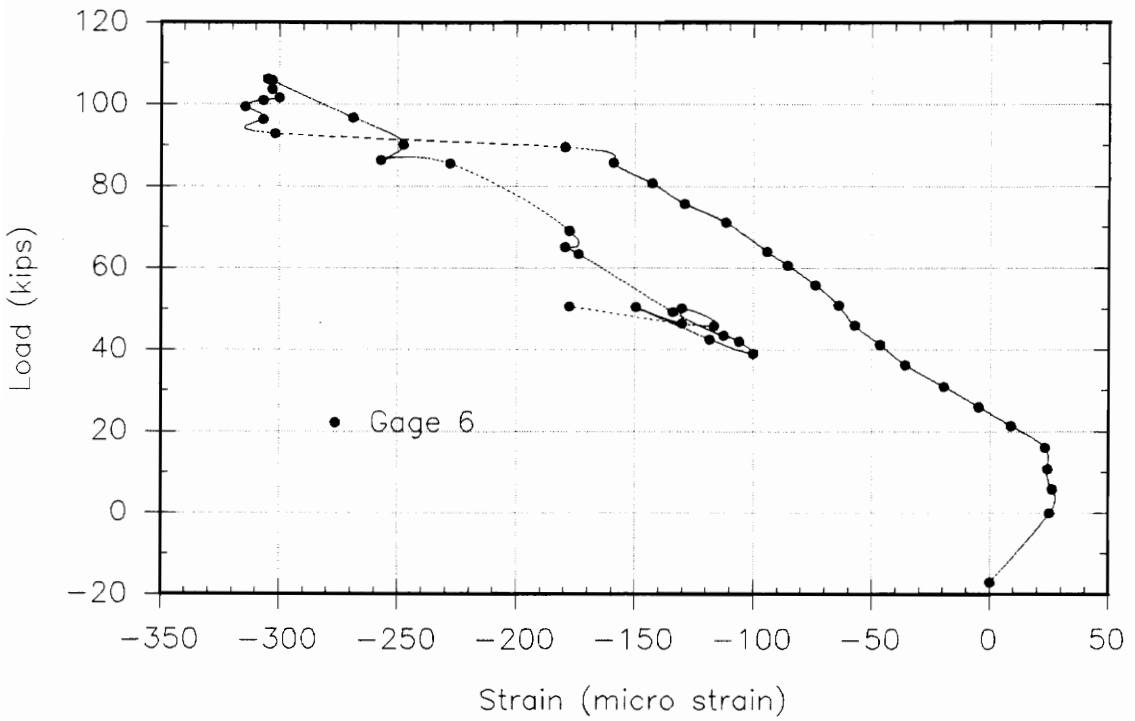


Figure B.4.7 Applied Load vs. V2 Strain

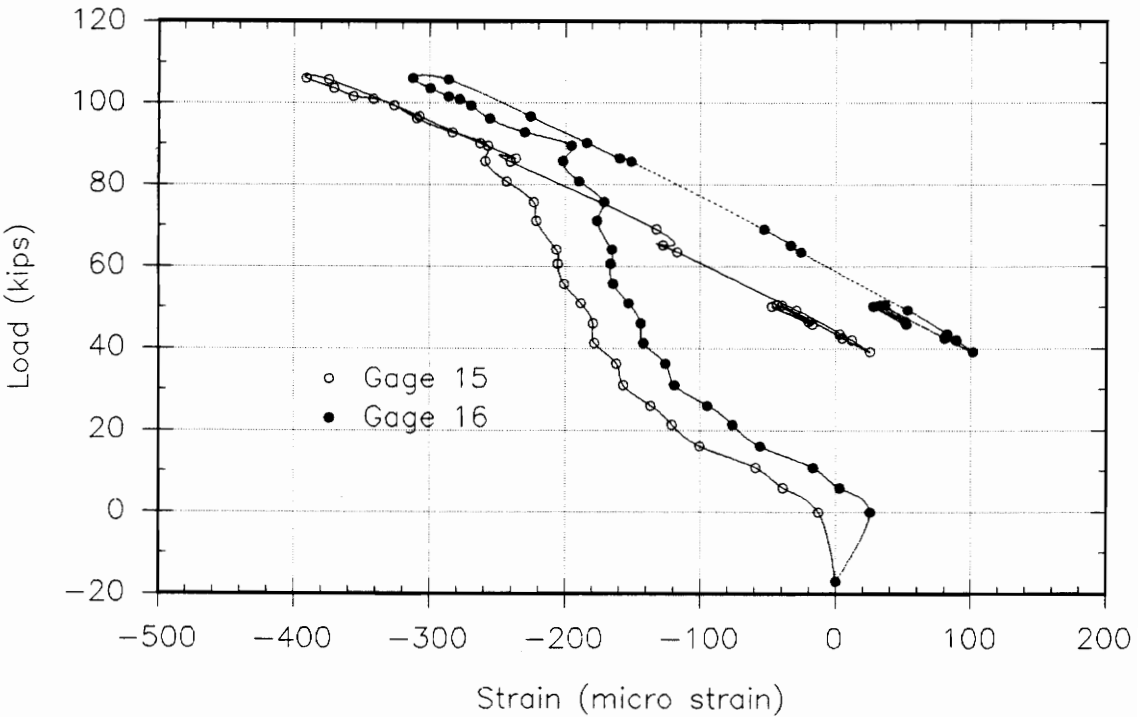


Figure B.4.8 Applied Load vs. V3 Strain

CLH - 4

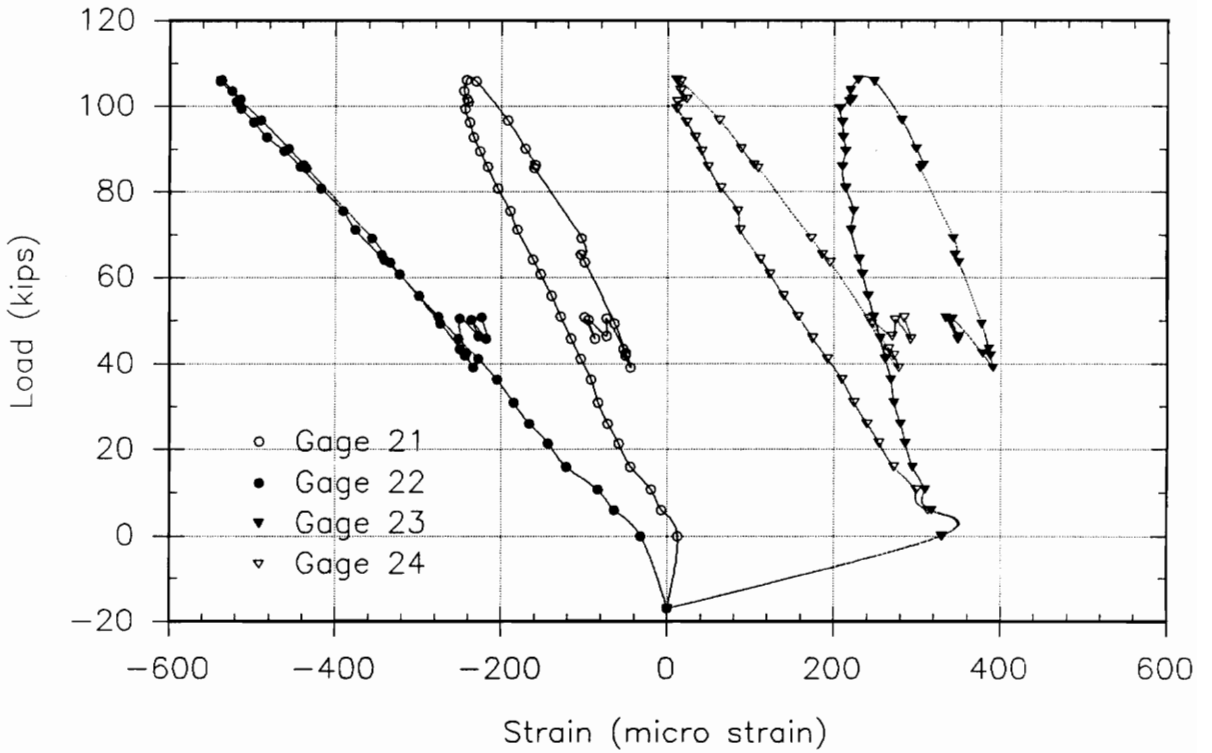


Figure B.4.9 Applied Load vs. V4 Strain

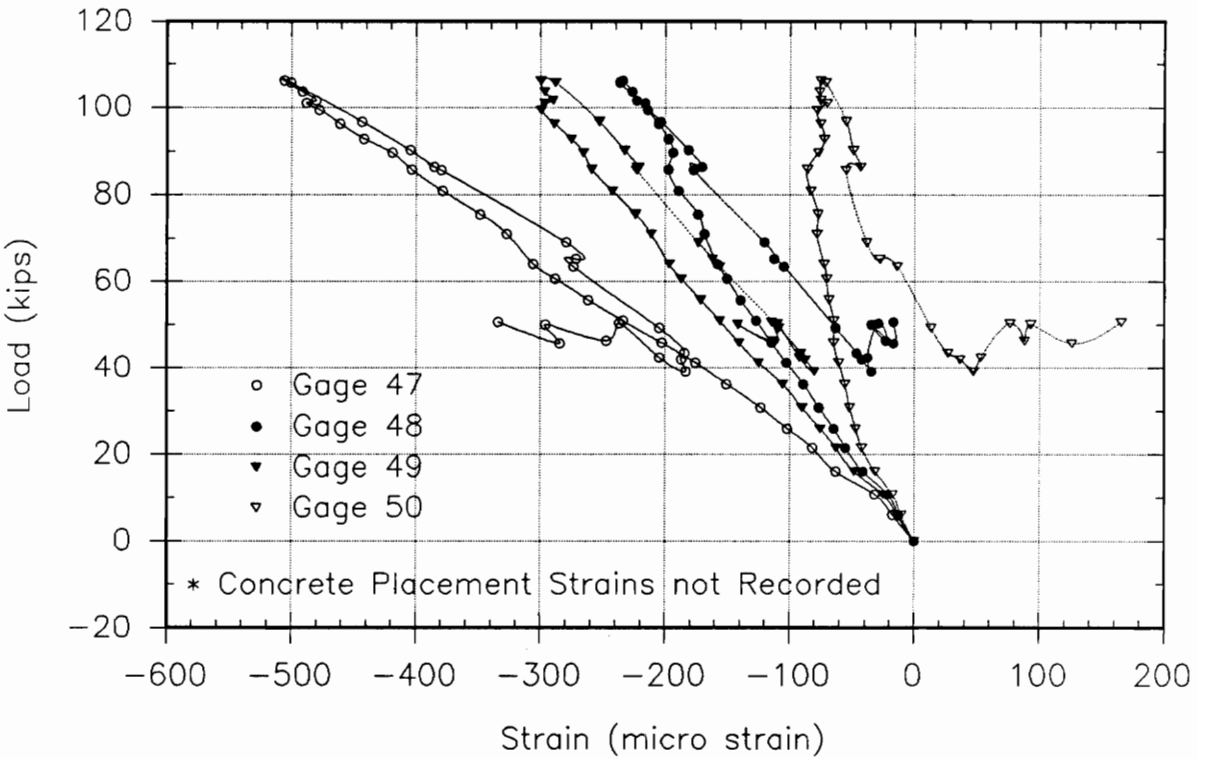


Figure B.4.10 Applied Load vs. V5 Strain *

CLH - 4

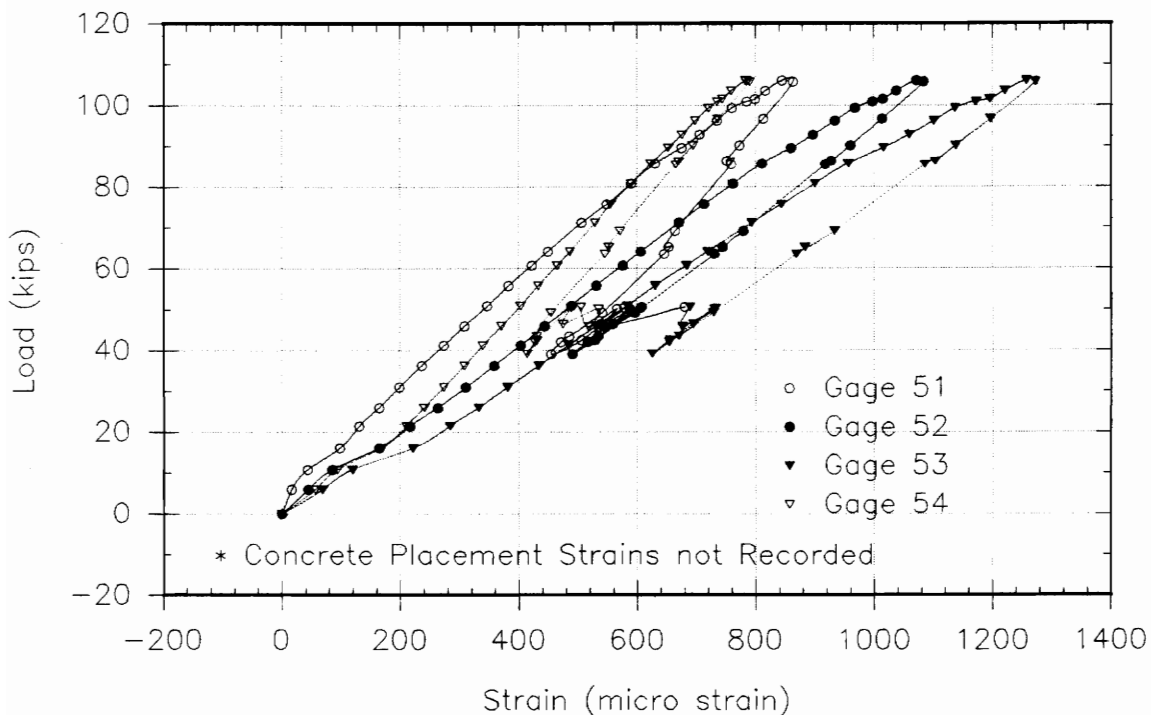


Figure B.4.11 Applied Load vs. W2 Strain *

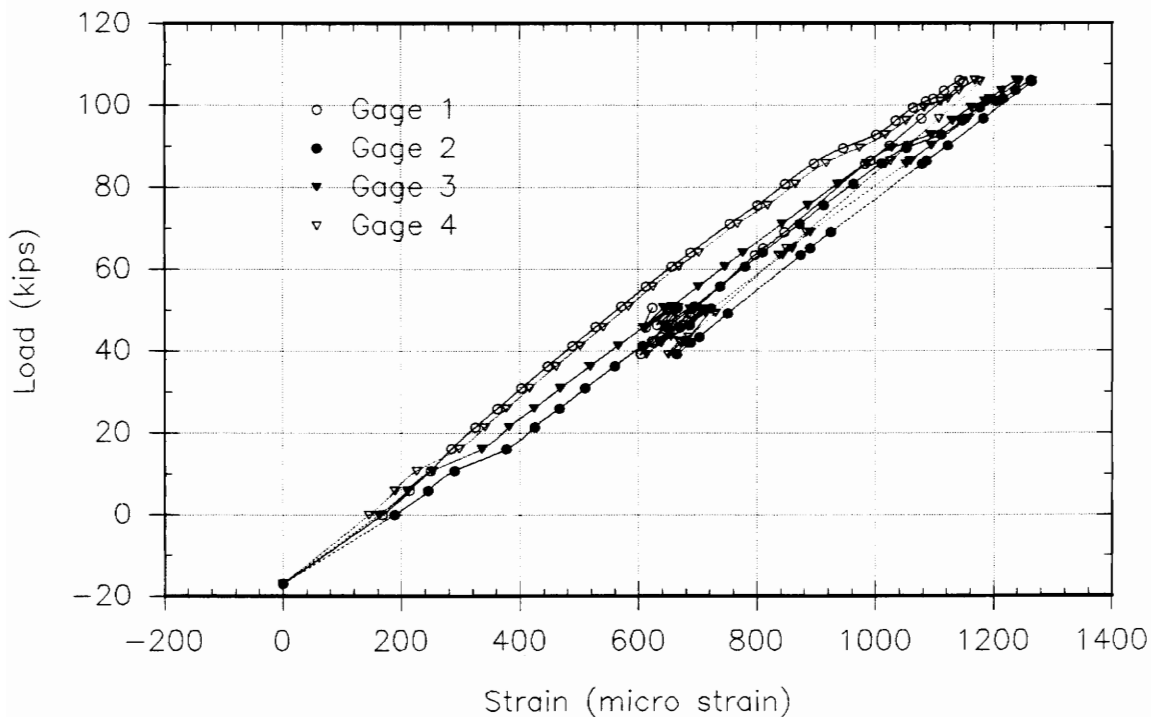


Figure B.4.12 Applied Load vs. W4 Strain

CLH - 4

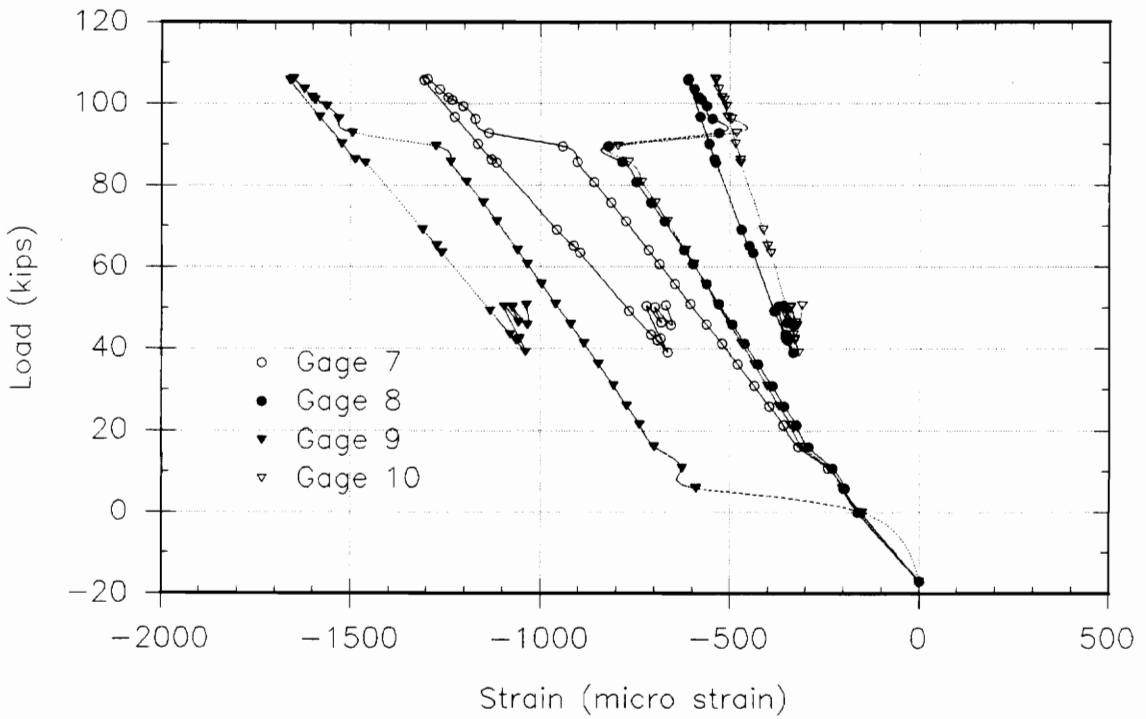


Figure B.4.13 Applied Load vs. W5 Strain

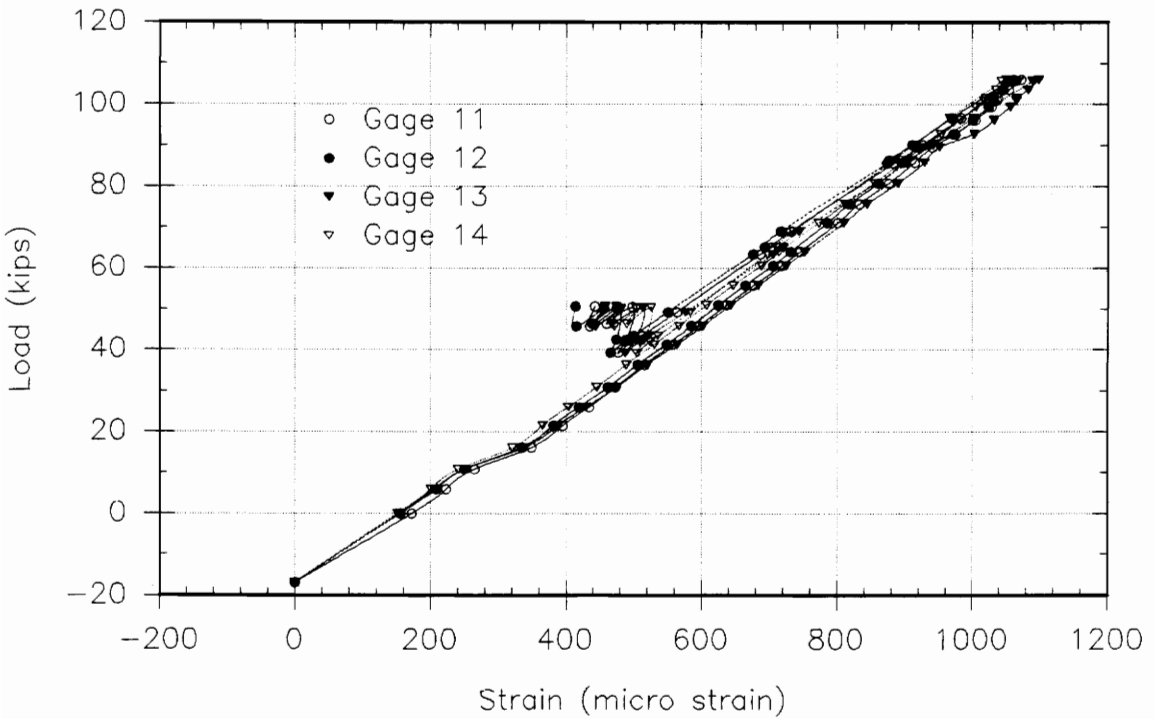


Figure B.4.14 Applied Load vs. W6 Strain

CLH - 4

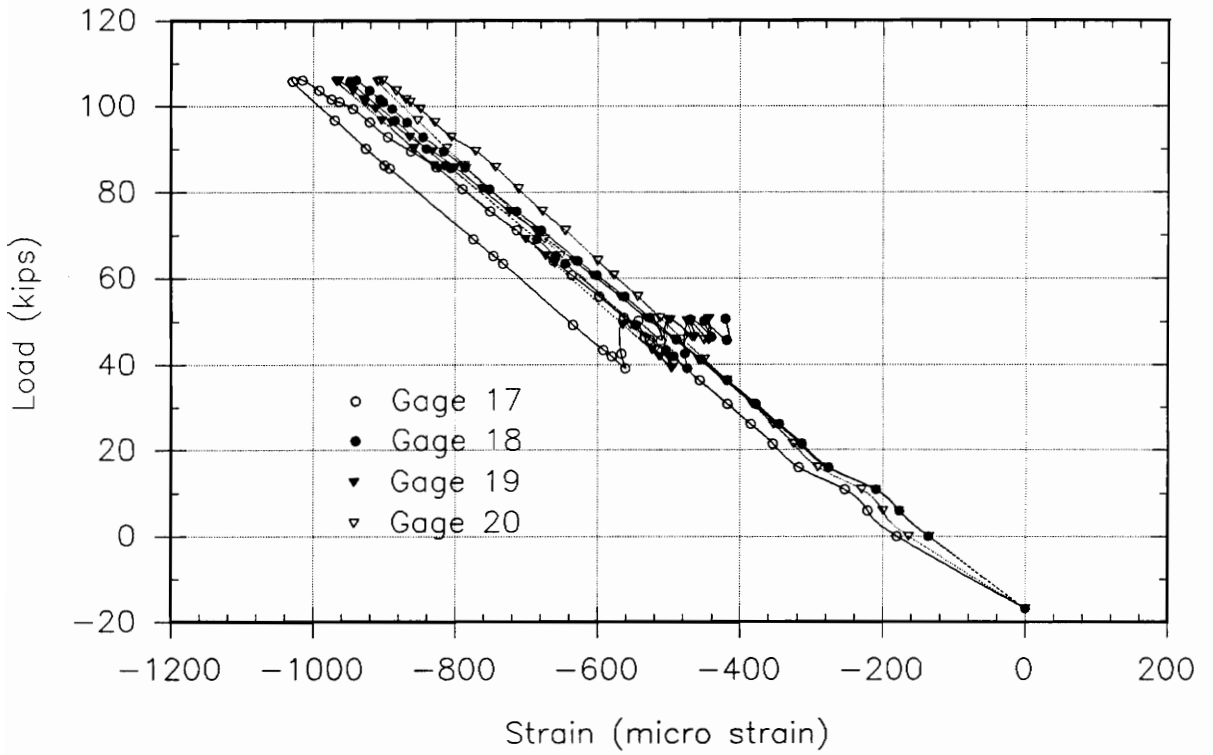


Figure B.4.15 Applied Load vs. W7 Strain

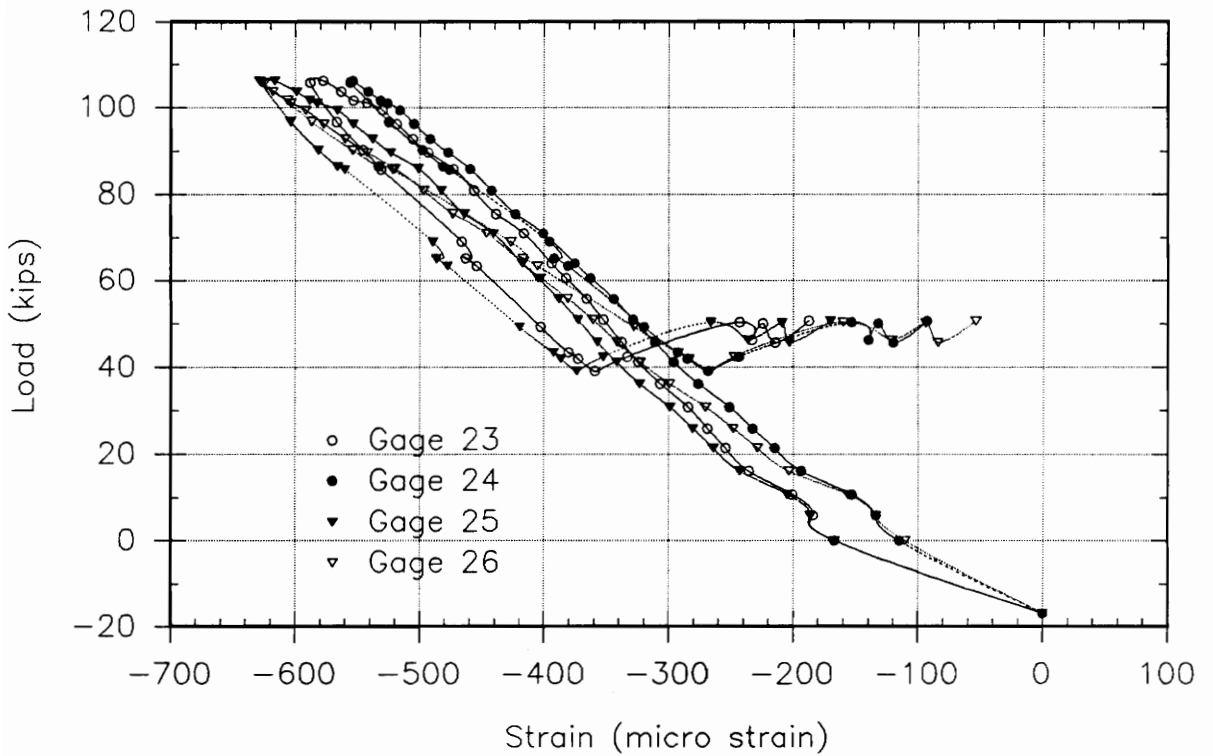


Figure B.4.16 Applied Load vs. W9 Strain

CLH - 4

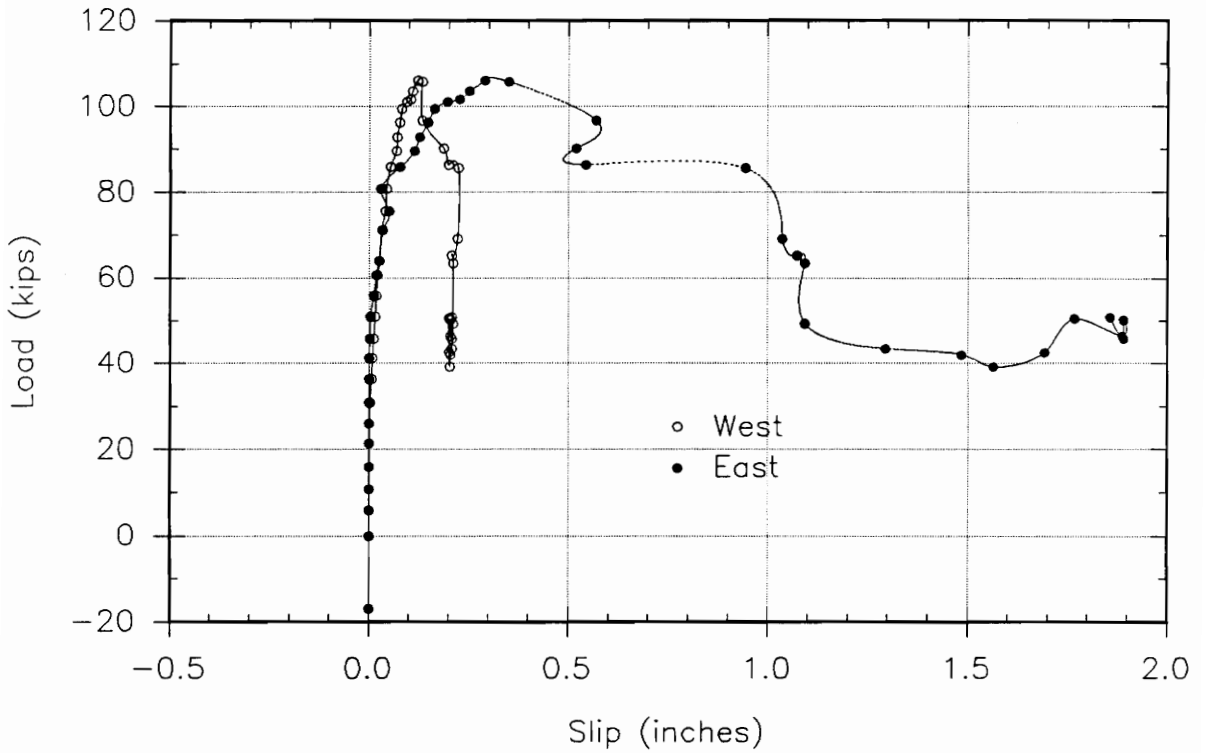


Figure B.4.17 Applied Load vs. End Slip

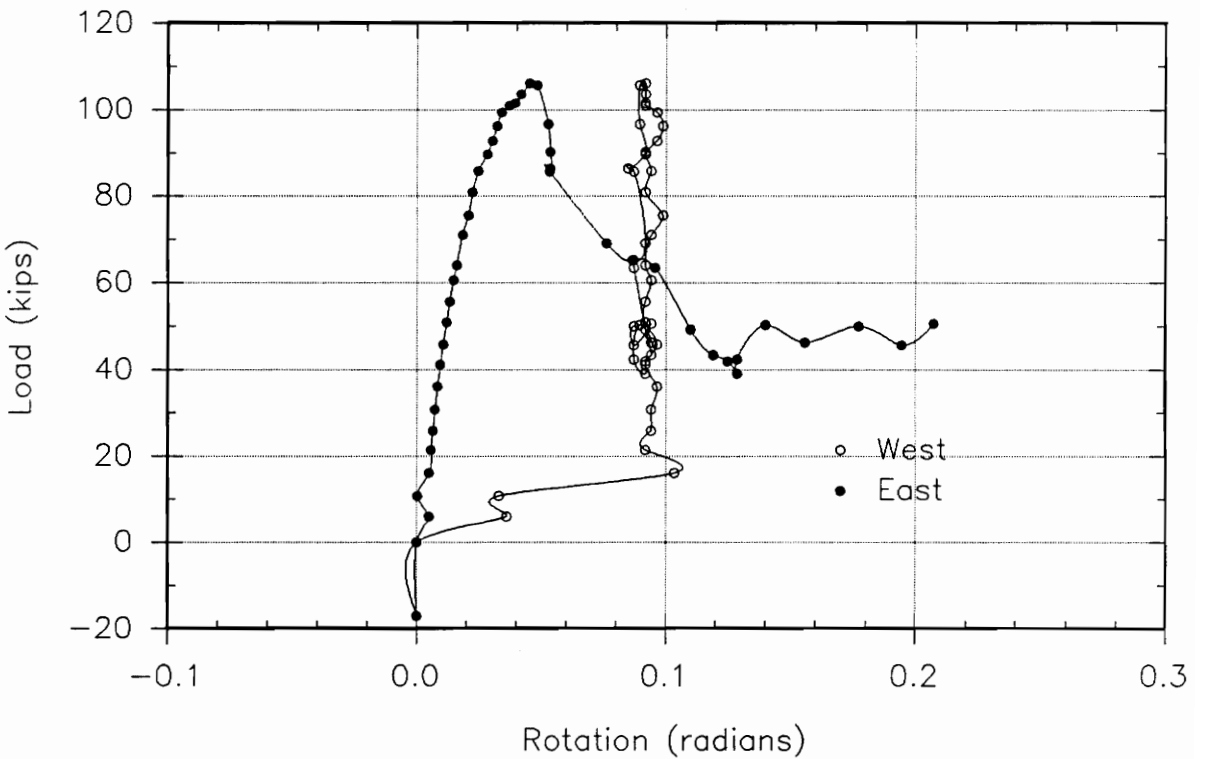


Figure B.4.18 Applied Load vs. End Rotation

TEST SUMMARY SHEET

TEST DESIGNATION: CLH-5 TEST DATE: 16 May, 1990
 TEST LOCATION: Structures and Materials Research Laboratory

JOIST DESCRIPTION

Depth: 34 in Top Chord: 2L 3.5x3.5x0.313
 Length: 40 ft Bottom Chord: 2L 3.5x3.5x0.313
 Number of Studs: 22 Width of Slab: 81 in
 Dead Weight of Joist and Slab: 406 plf

CALCULATED CAPACITIES (using measured material properties)

Theoretical Max Applied Load: 145.4 kips
 Theoretical Moment of Inertia: 4347 in⁴
 Theoretical Elastic Deflection: 1.67 in

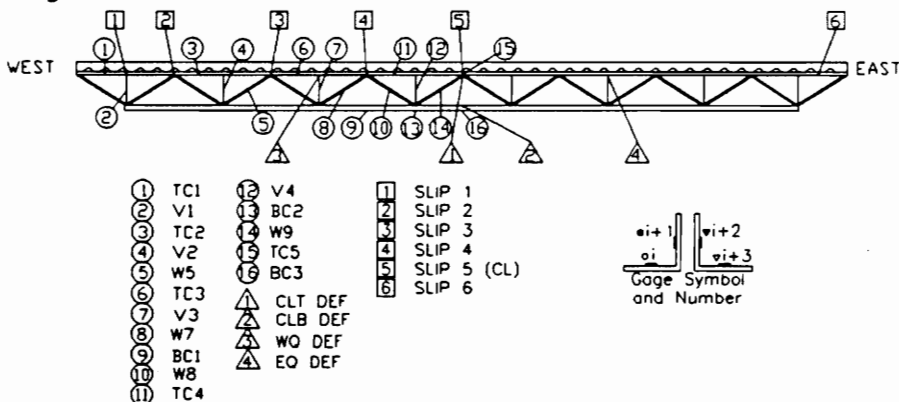
TEST RESULTS

Max Applied Load: 133.5 kips Max Deflection: 7.207 in
 Yield Load: 110.5 kips Deflection at Yield: 1.98 in
 Average Max Bottom Chord Strain: 1774 micro strain
 Bottom Chord fy: 59.87 ksi f'c: 5.86 ksi
 Concrete Pour Strains Recorded: YES
 Maximum Slip: 0.287 in
 Failure: Buckling of W5R.

COMPARISON OF ACTUAL TO THEORETICAL

Actual Stiffness/Theoretical Stiffness: 0.642
 Actual Max Load/Theoretical Max Load: 0.918

Strain Gage Locations:



Comments: The bottom chord started yielding at the panel points. The yielding zone did not reach the strain gages placed at midspan. Gages were zeroed at no applied load. Measurements were taken during the concrete placement.

CLH - 5

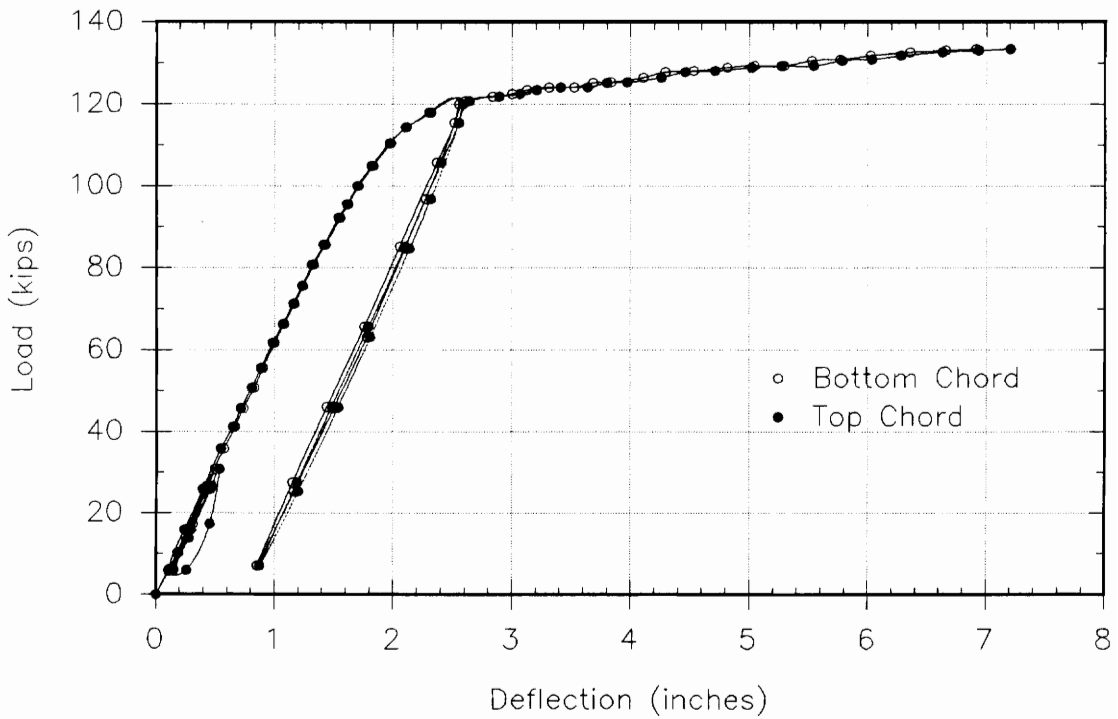


Figure B.5.1 Applied Load vs. Centerline Deflection

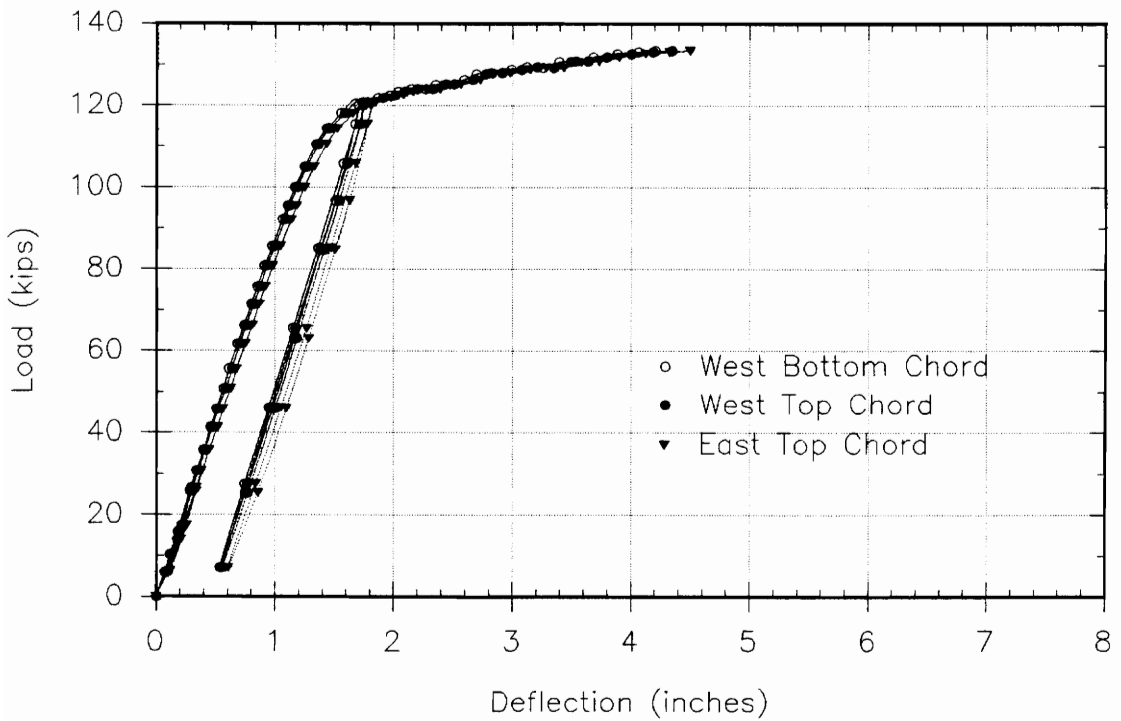


Figure B.5.2 Applied Load vs. Quarter Point Deflection

CLH - 5

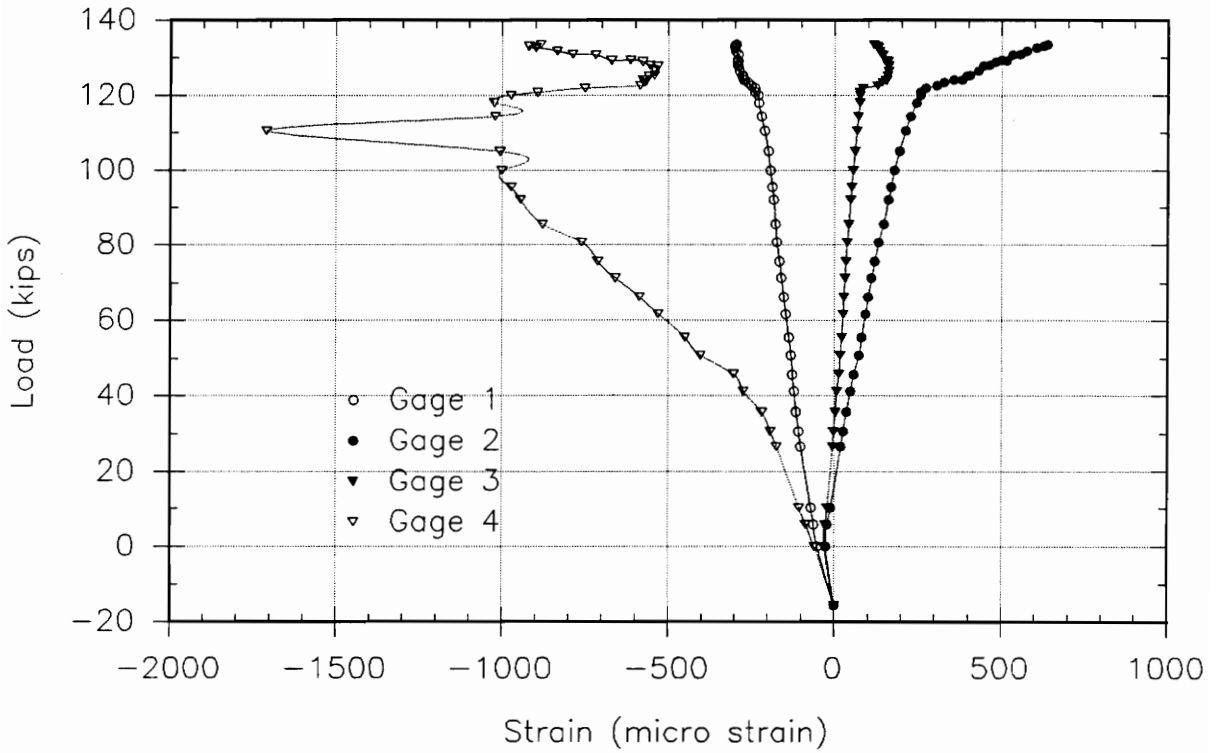


Figure B.5.3 Applied Load vs. Top Chord Strain (TC1)

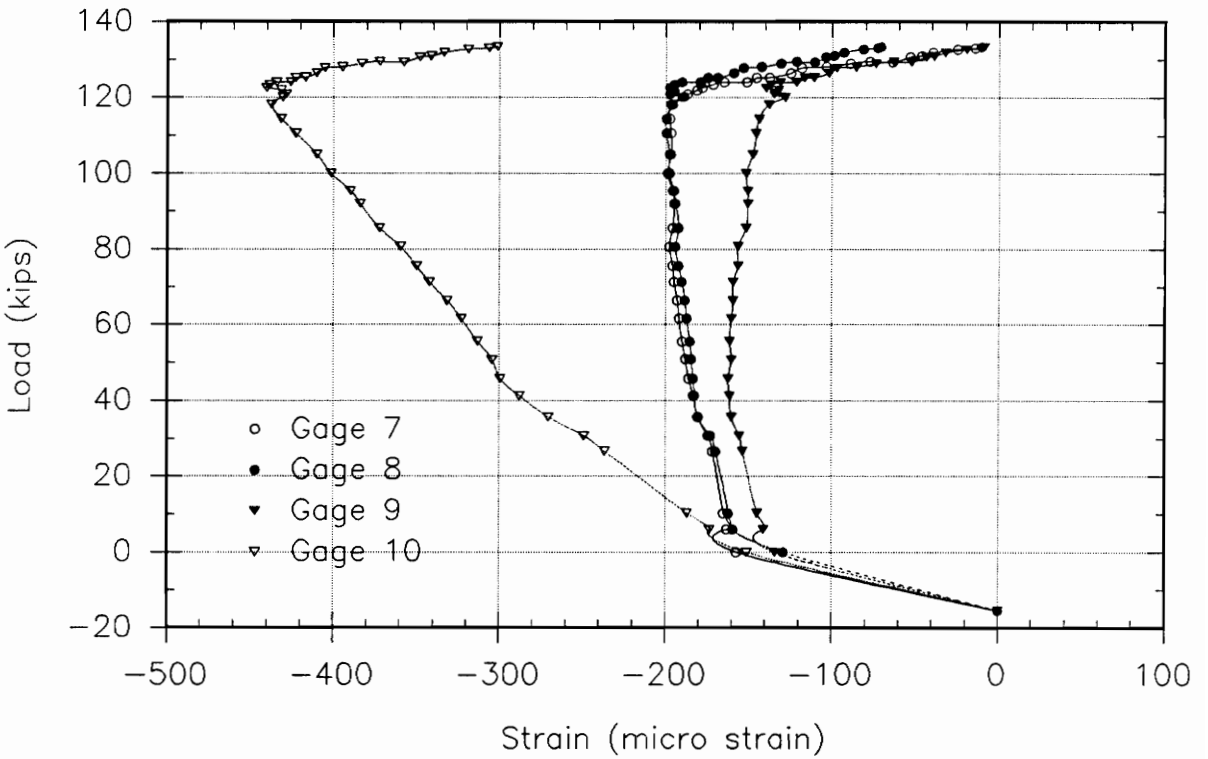


Figure B.5.4 Applied Load vs. Top Chord Strain (TC2)

CLH - 5

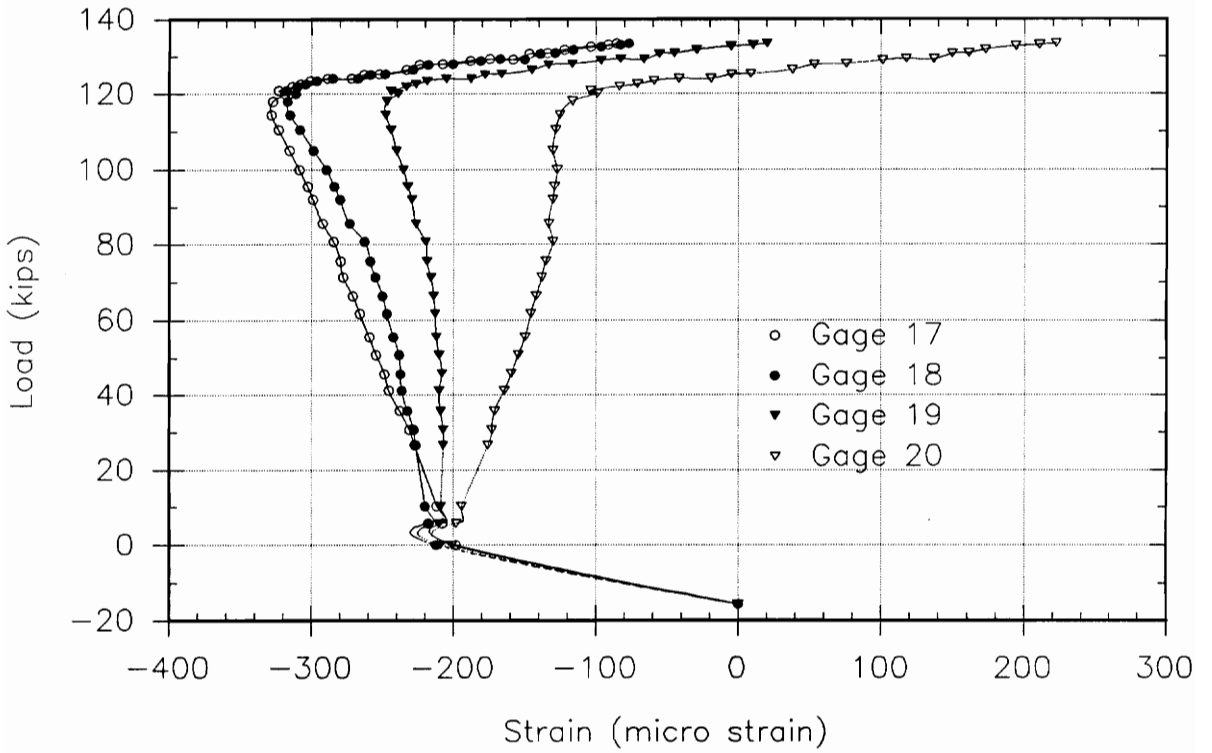


Figure B.5.5 Applied Load vs. Top Chord Strain (TC3)

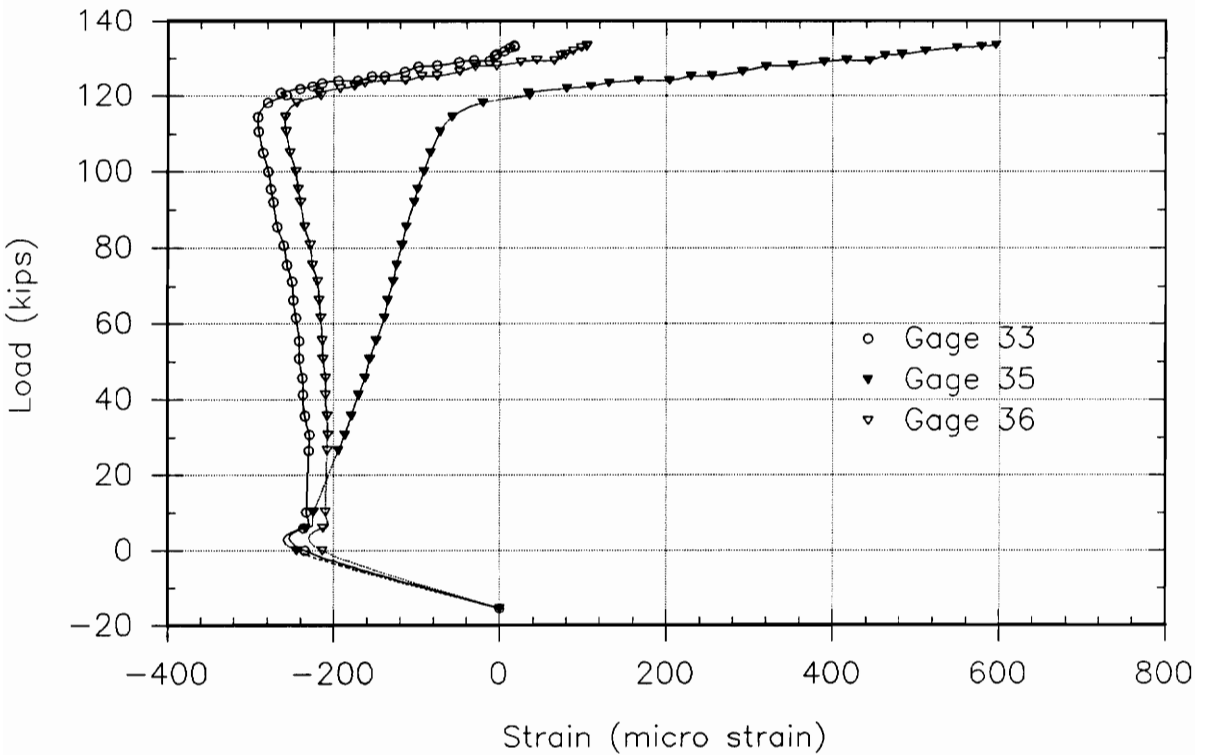


Figure B.5.6 Applied Load vs. Top Chord Strain (TC4)

CLH - 5

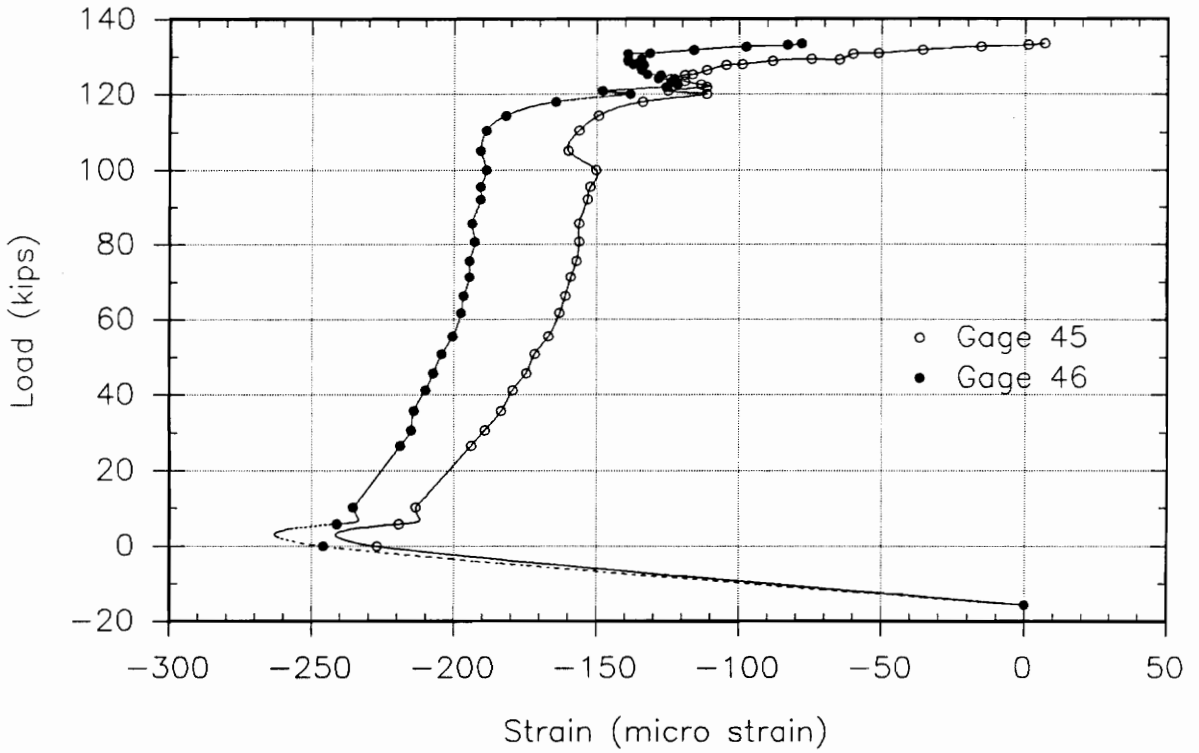


Figure B.5.7 Applied Load vs. Top Chord Strain (TC5)

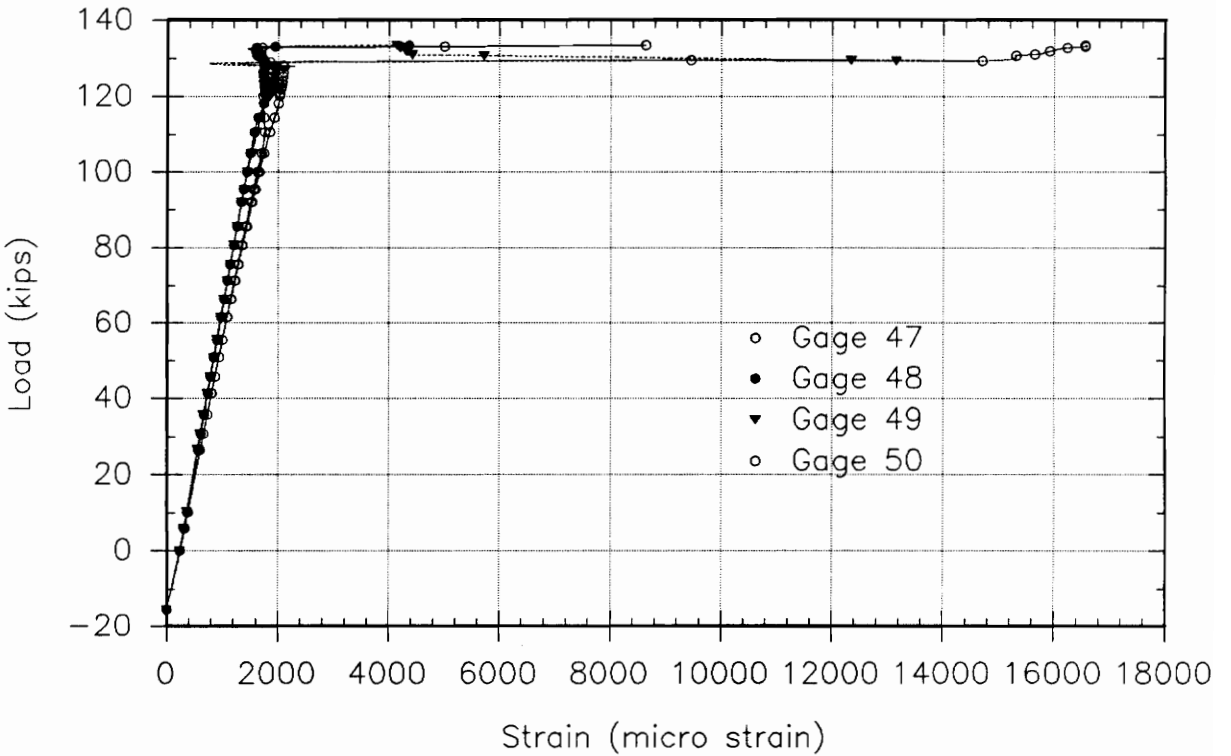


Figure B.5.8 Applied Load vs. Bottom Chord Strain (BC3)

CLH - 5

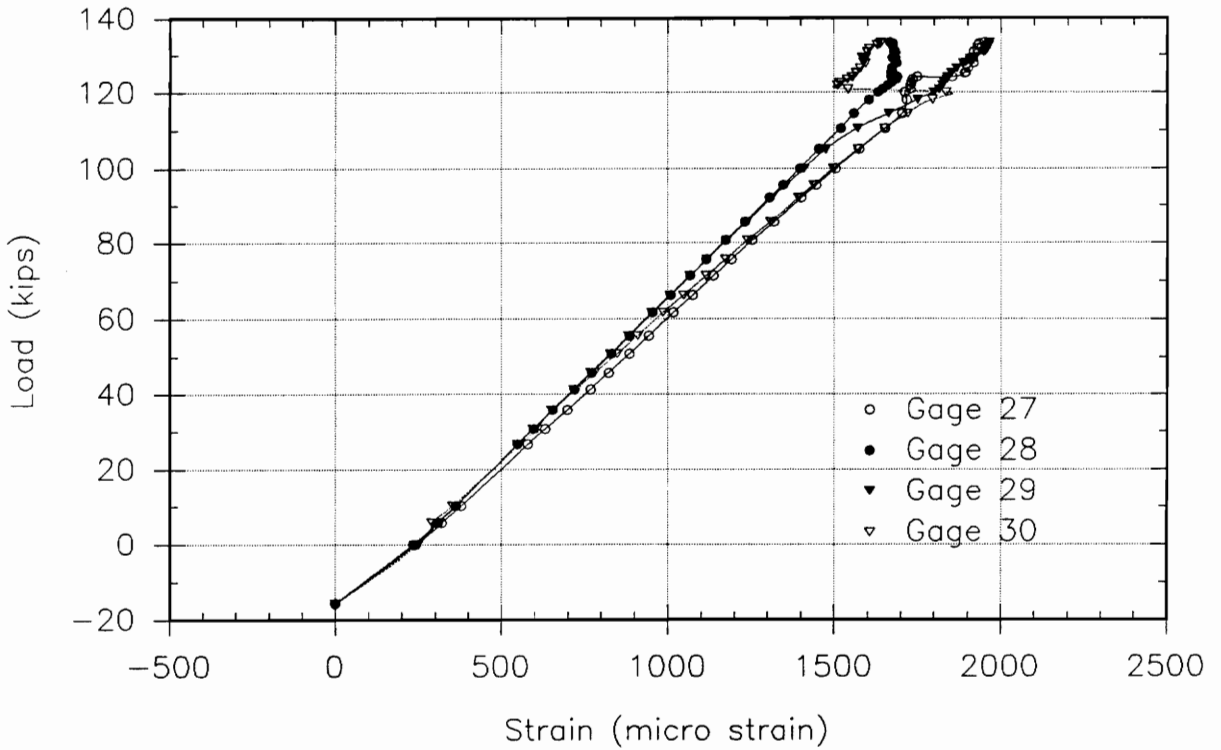


Figure B.5.9 Applied Load vs. Bottom Chord Strain (BC1)

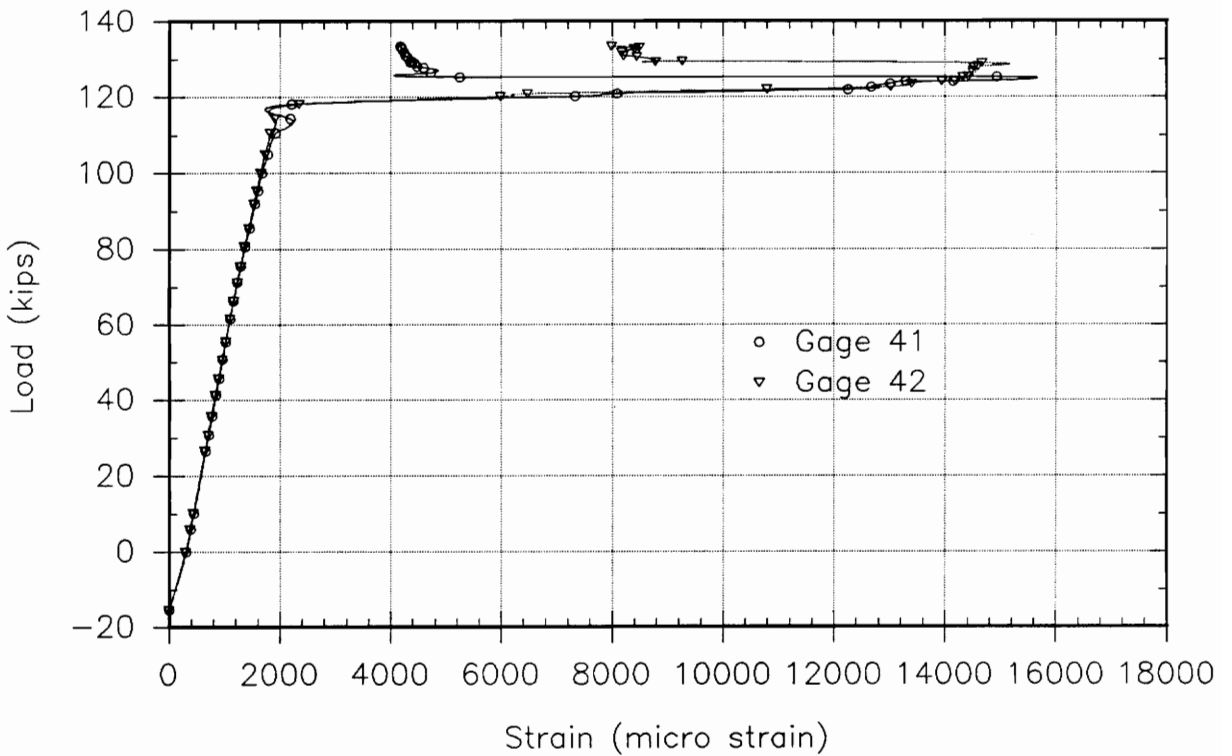


Figure B.5.10 Applied Load vs. Bottom Chord Strain (BC2)

CLH - 5

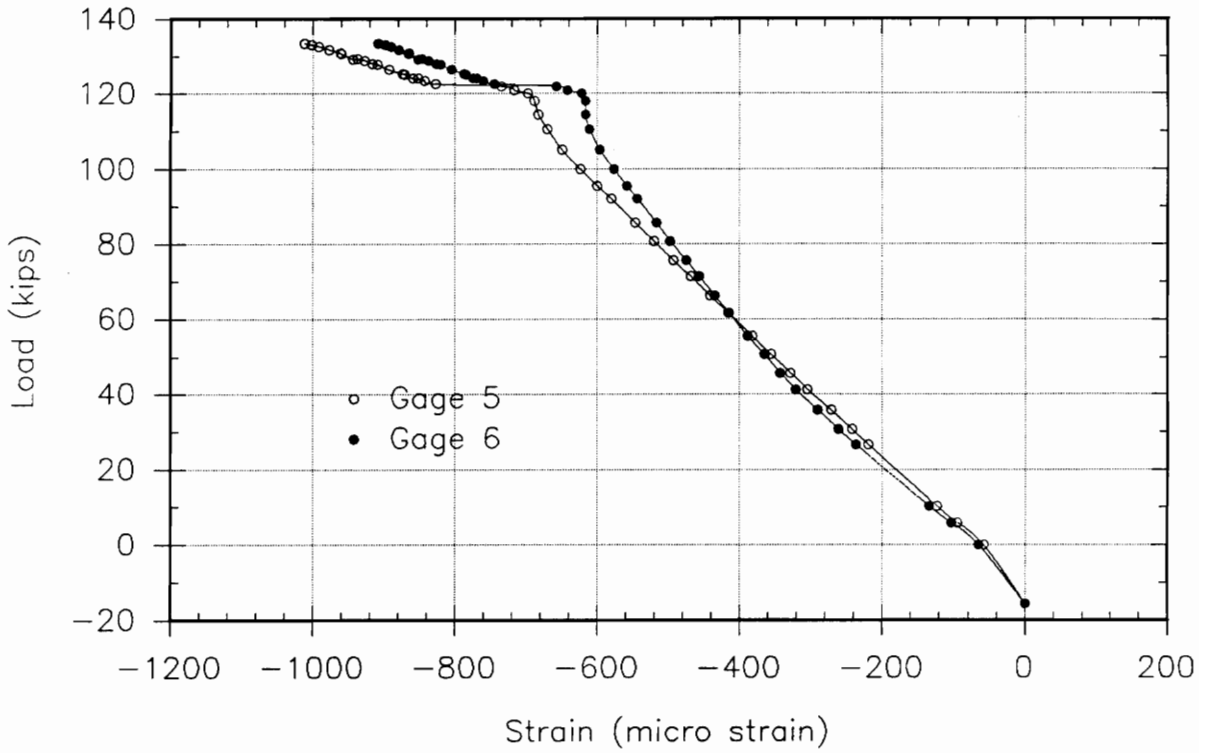


Figure B.5.11 Applied Load vs. V1 Strain

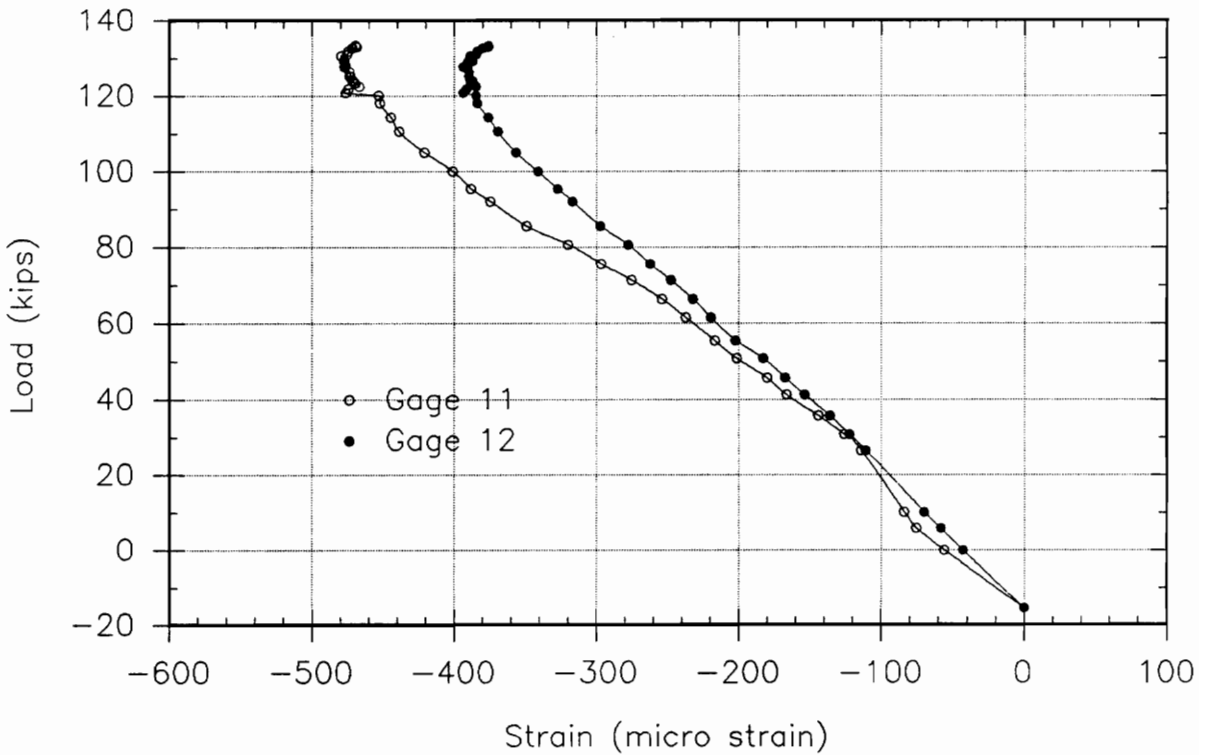


Figure B.5.12 Applied Load vs. V2 Strain

CLH - 5

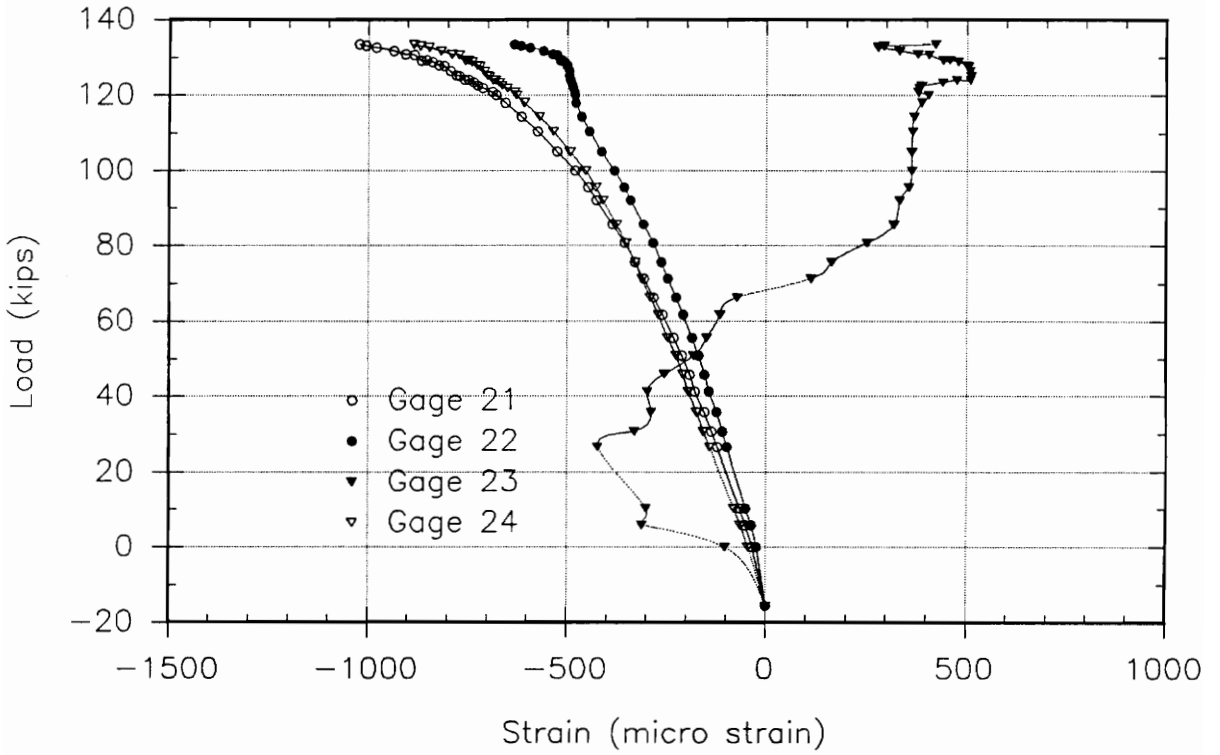


Figure B.5.13 Applied Load vs. V3 Strain

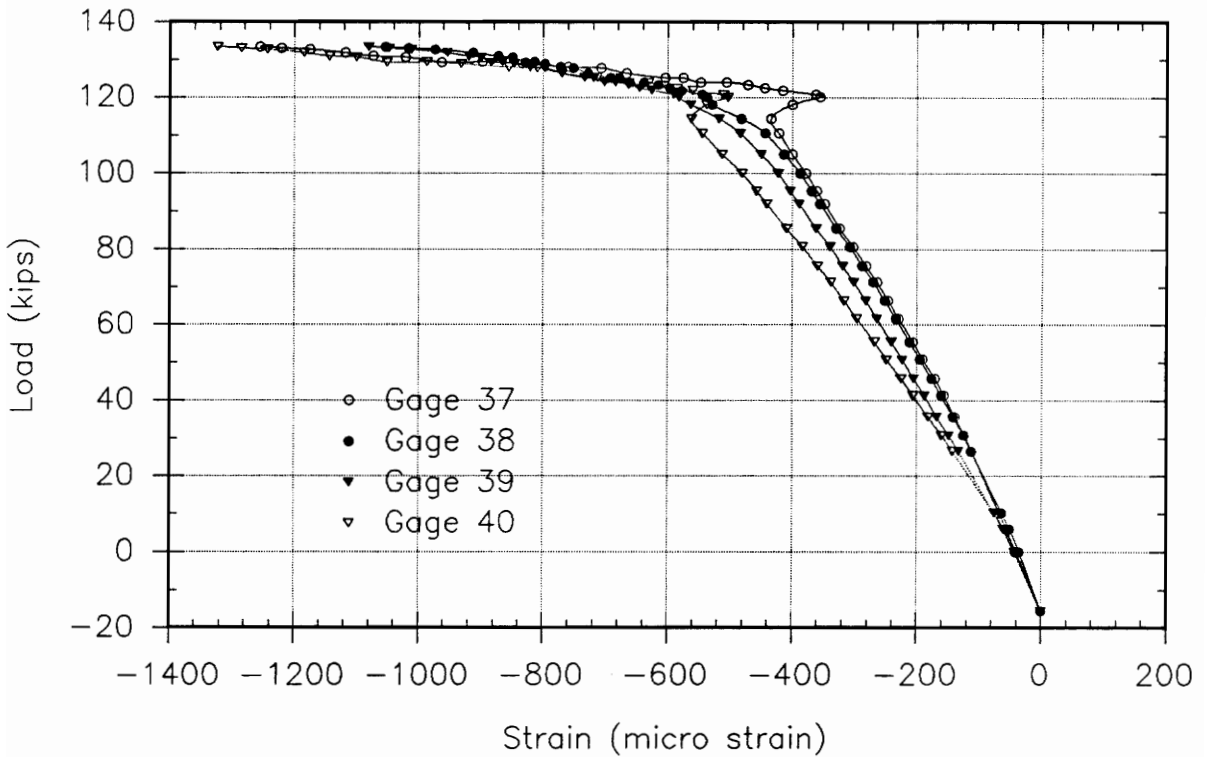


Figure B.5.14 Applied Load vs. V4 Strain

CLH - 5

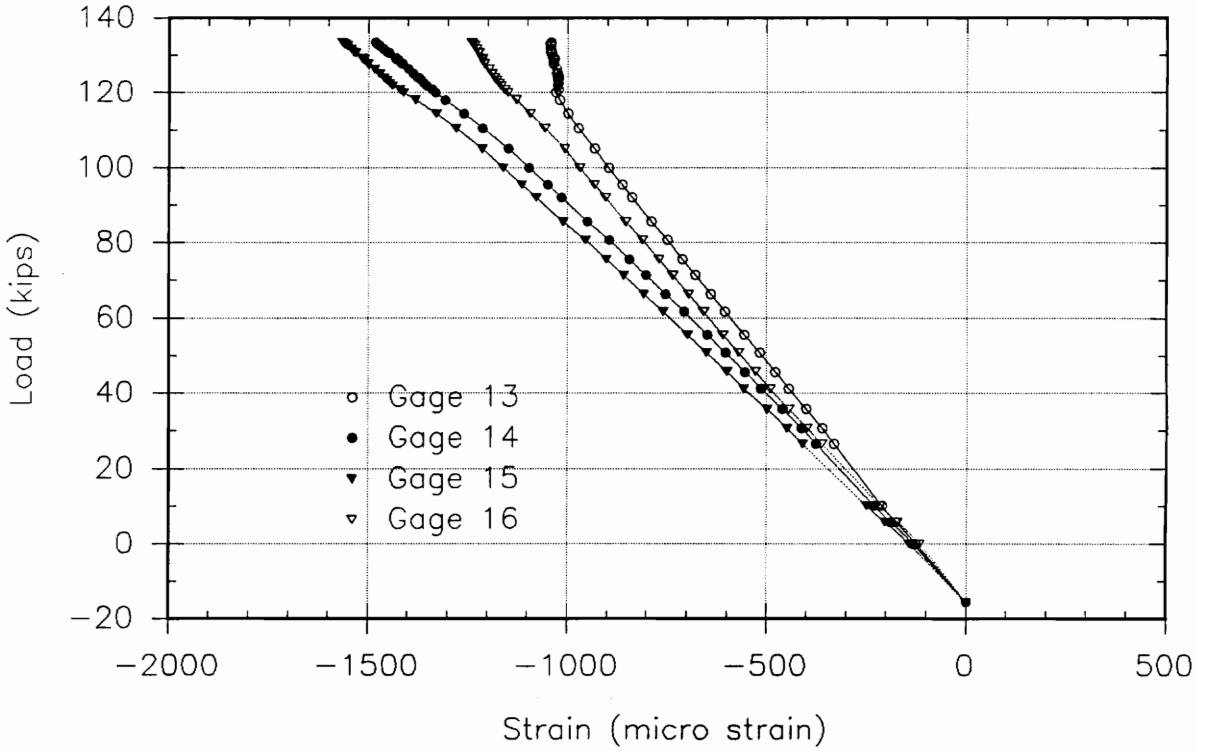


Figure B.5.15 Applied Load vs. W5 Strain

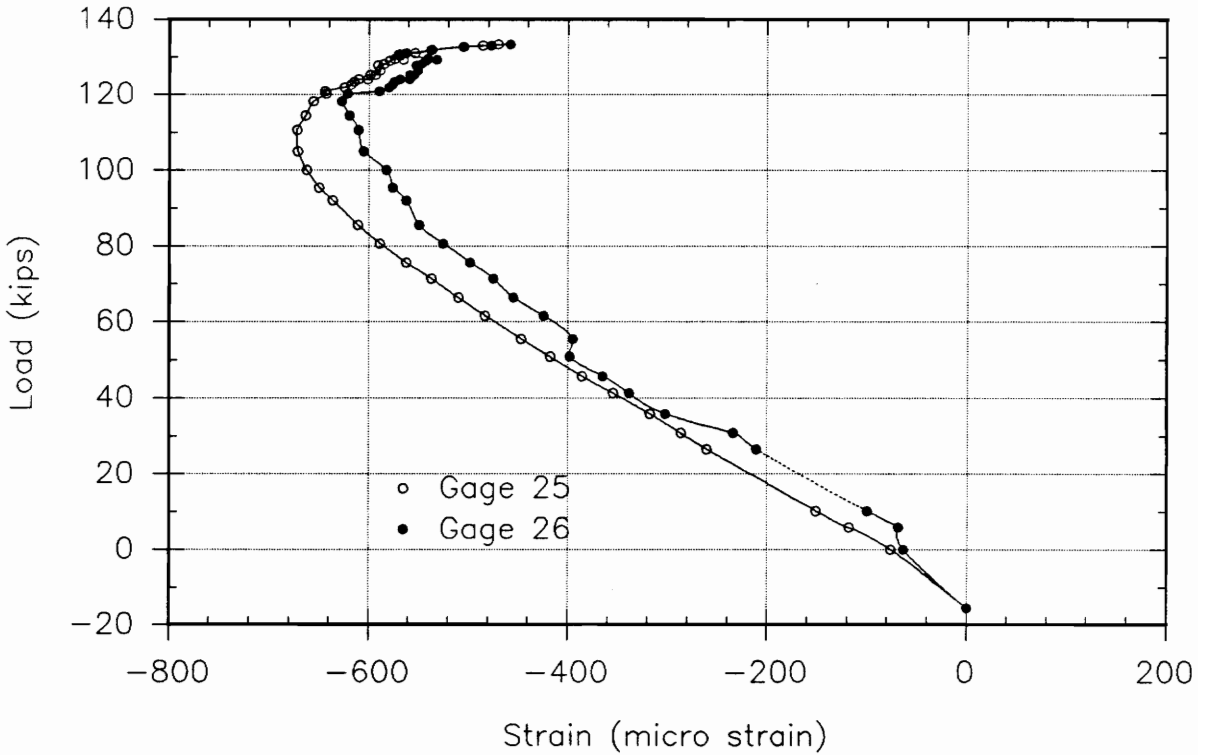


Figure B.5.16 Applied Load vs. W7 Strain

CLH - 5

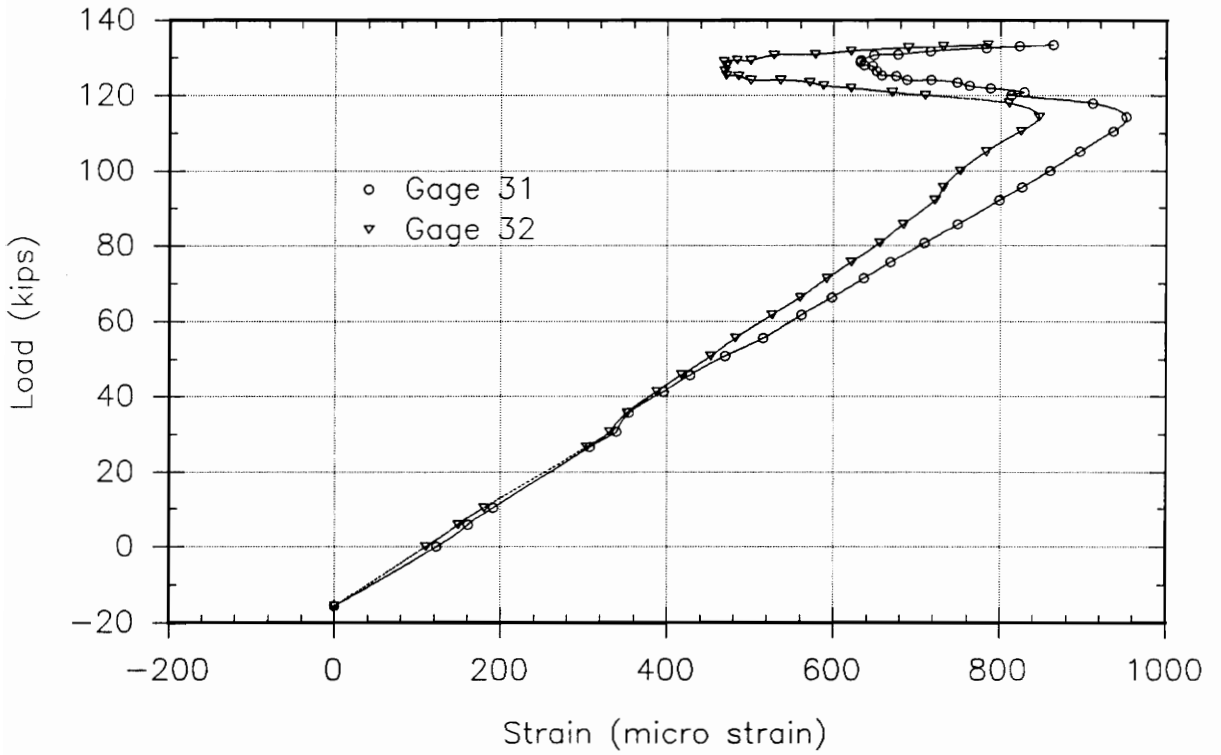


Figure B.5.17 Applied Load vs. W8 Strain

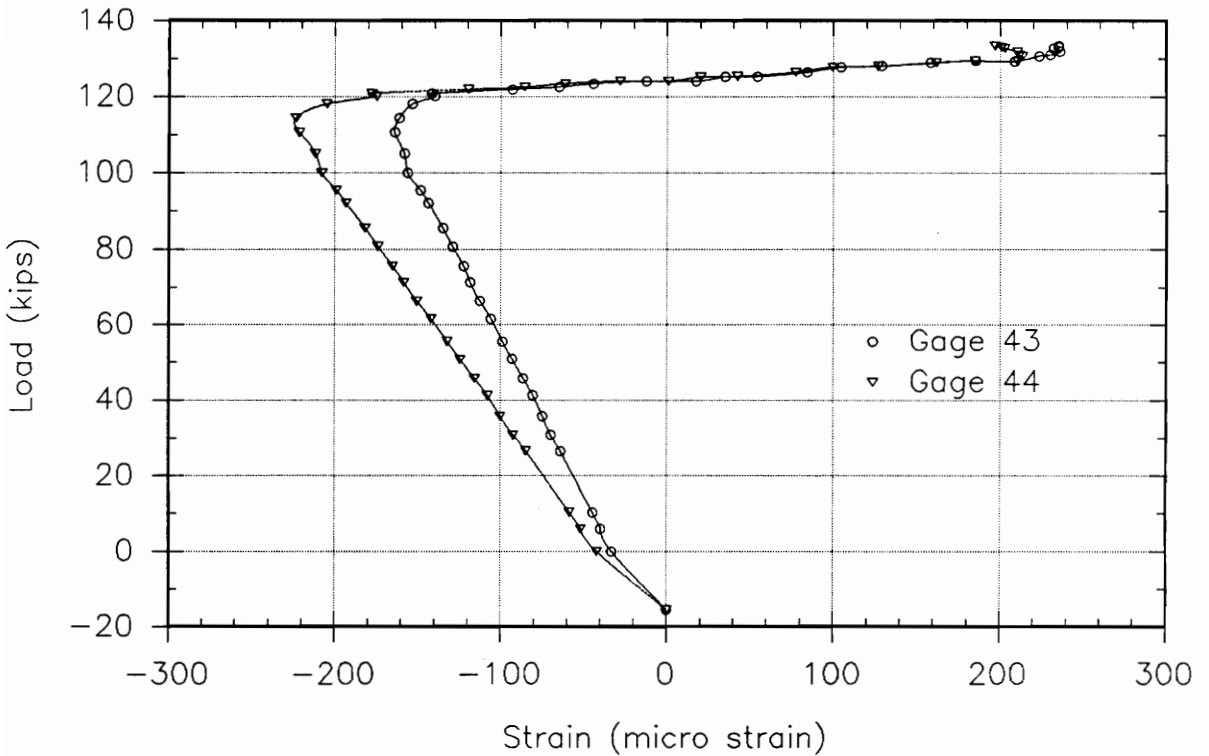


Figure B.5.18 Applied Load vs. W9 Strain

CLH - 5

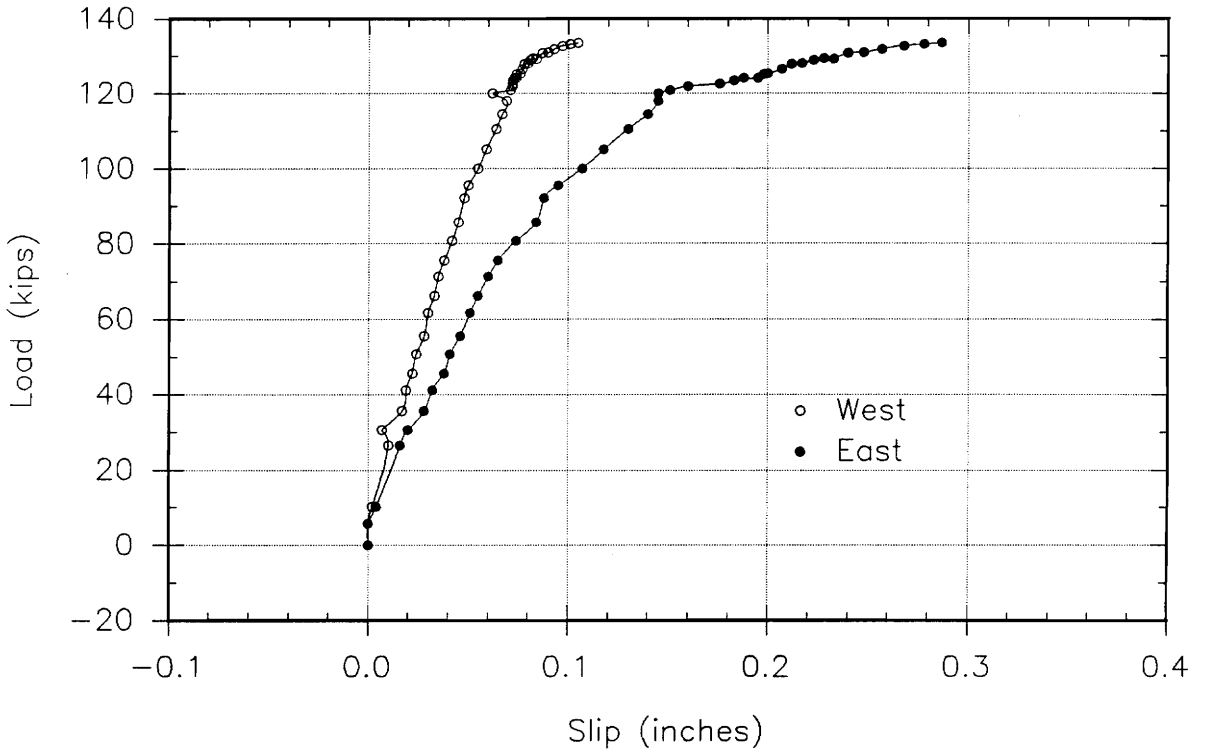


Figure B.5.19 Applied Load vs. End Slip

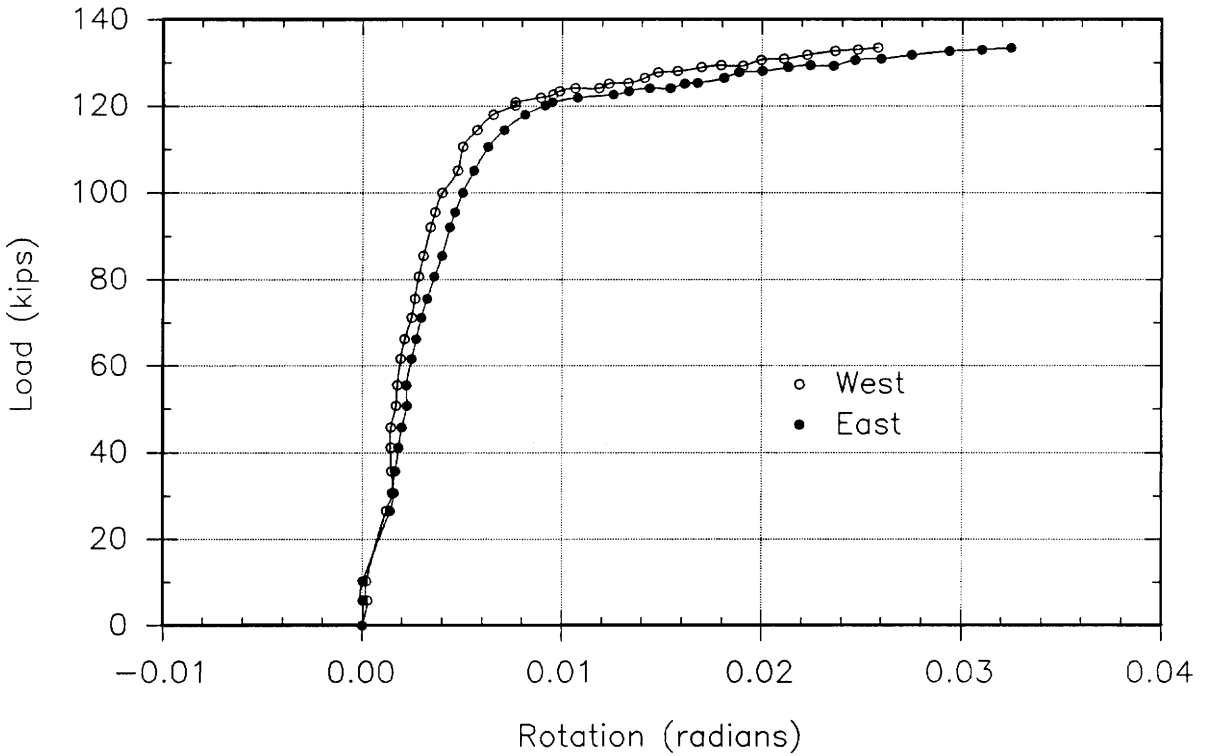


Figure B.5.20 Applied Load vs. End Rotation

CLH - 5

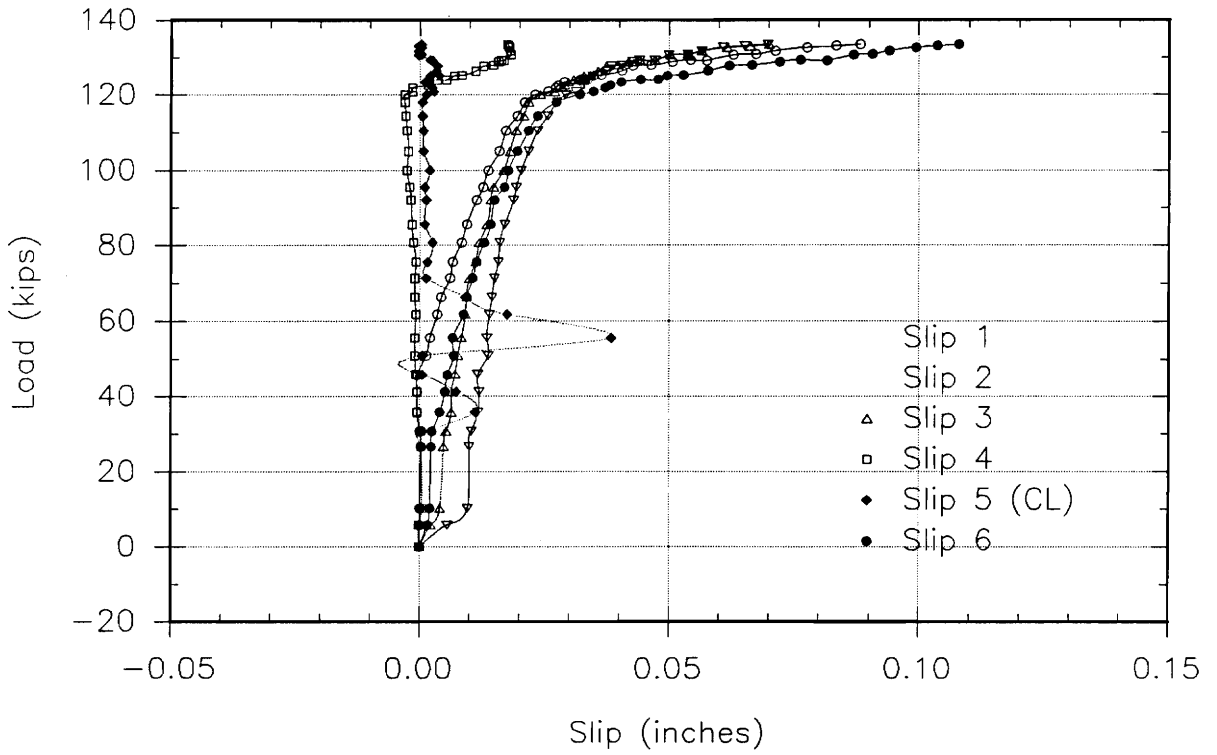


Figure B.5.21 Applied Load vs. Slip

TEST SUMMARY SHEET

TEST DESIGNATION: CLH-6 TEST DATE: 1 November 1990
 TEST LOCATION: Structures and Materials Research Laboratory

JOIST DESCRIPTION

Depth: <u>14 in</u>	Top Chord: <u>2L3.00x3.00x.313</u>
Length: <u>40 ft</u>	Bottom Chord: <u>2L4.00x4.00x.438</u>
Number of Studs: <u>36</u>	Width of Slab: <u>81 in</u>
Dead Weight of Joist and Slab: <u>344 plf</u>	

CALCULATED CAPACITIES (using measured material properties)

Theoretical Max Applied Load: 93.1 k
 Theoretical Moment of Inertia: 1260 in⁴
 Theoretical Elastic Deflection: 3.692 in

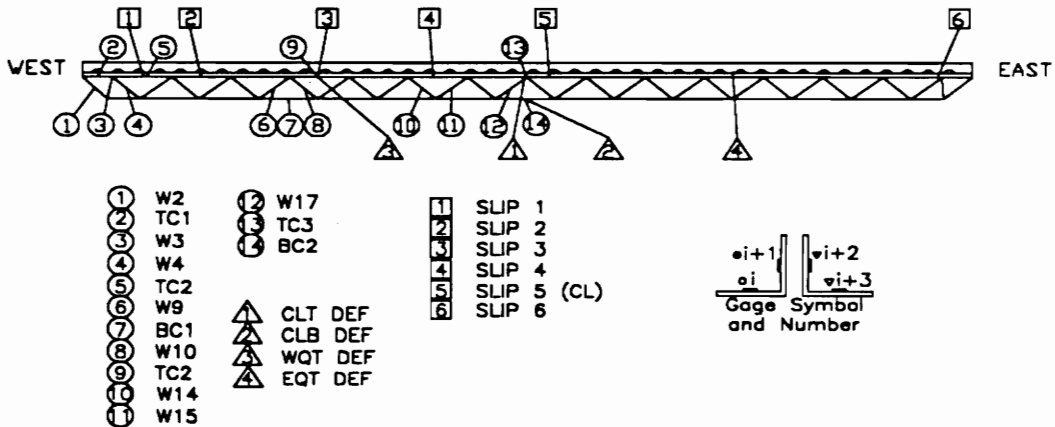
TEST RESULTS

Maximum Applied Load: 102.5 k Maximum Deflection: 14.09 in
 Yield Load: 84.7 k Deflection at Yield:
 Average Max Bottom Chord Strain: 4143 micro strain
 Actual Bottom Chord Fy: 56.16 ksi f'c: 4.43 ksi
 Concrete Pour Strains Recorded: YES
 Maximum Slip: 0.500 in
 Failure: Local Buckling of the Top Chord

COMPARISON OF ACTUAL TO THEORETICAL

Actual stiffness/theoretical stiffness: 1.24
 Actual Max Load/Theoretical Max Load: 1.10

Strain Gage Locations:



Comments: The web members were designed to carry 30% greater load than normal. Test reached a load higher than predicted. Bottom chord yield zone moved to the where the strain gages were placed at midspan.

CLH - 6

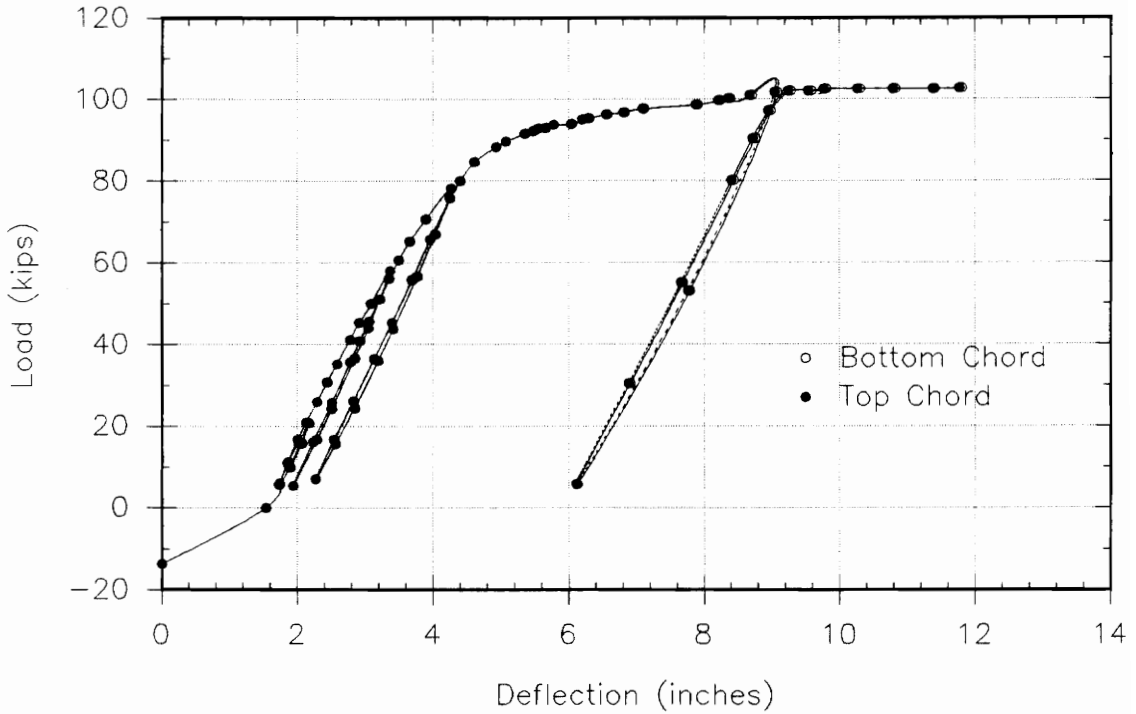


Figure B.6.1 Applied Load vs. Centerline Deflection

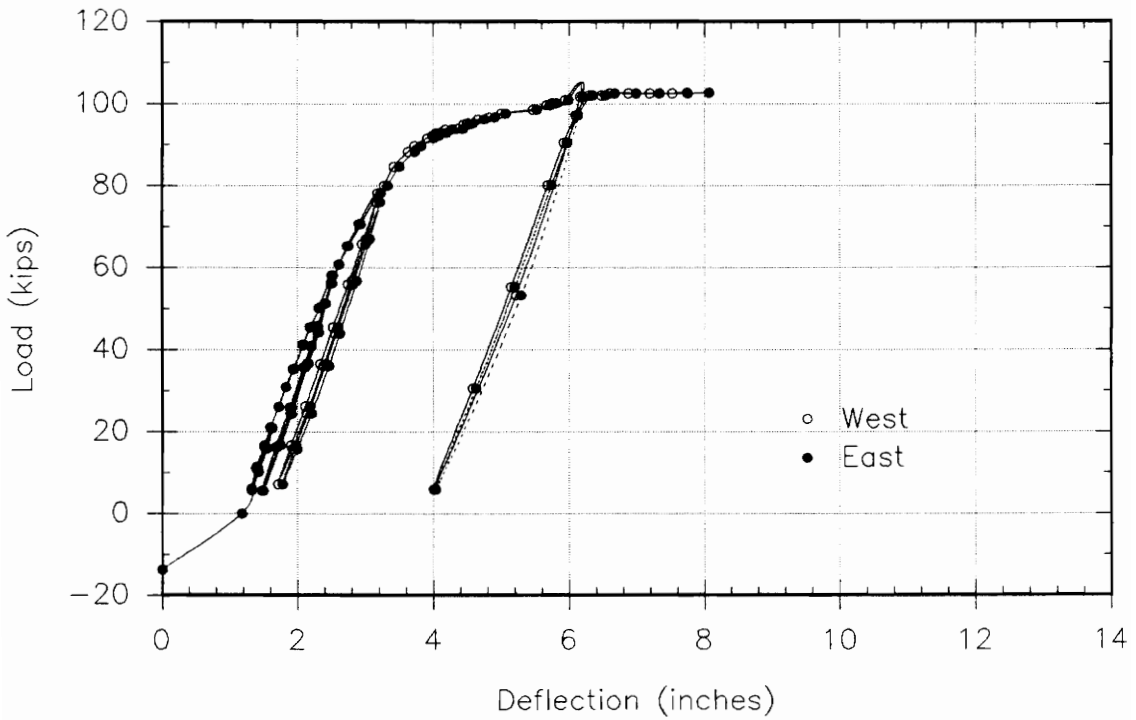


Figure B.6.2 Applied Load vs. Quarter Point Deflection

CLH - 6

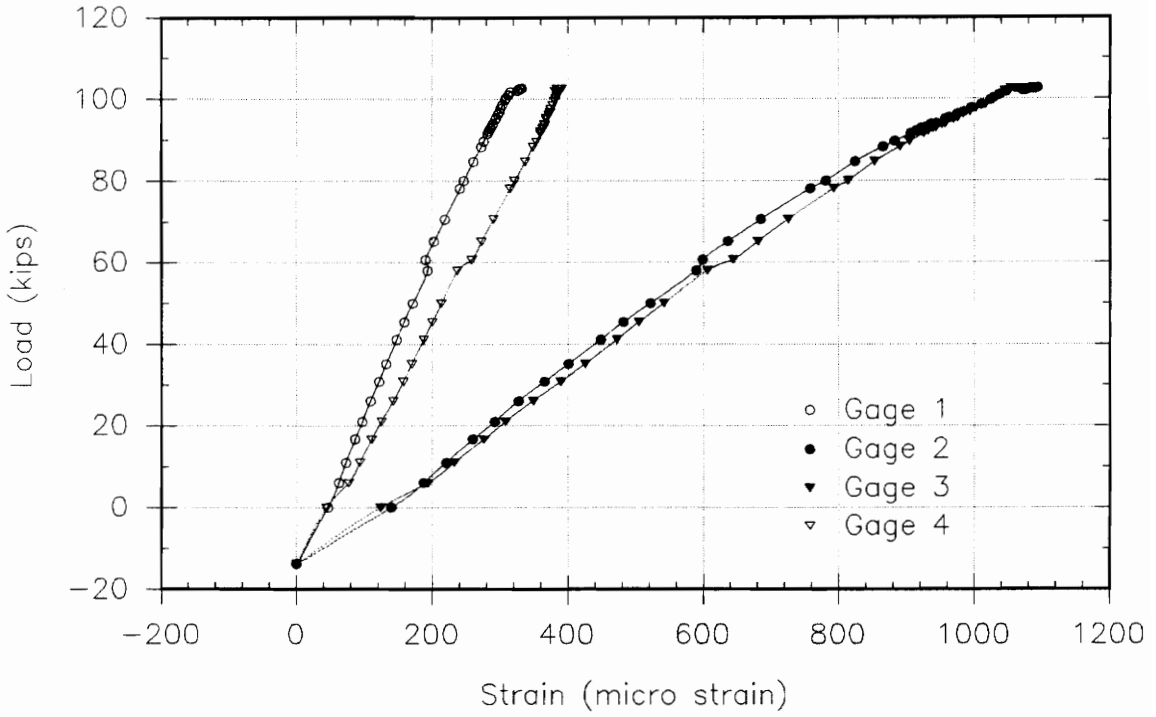


Figure B.6.3 Applied Load vs. W2 Strain

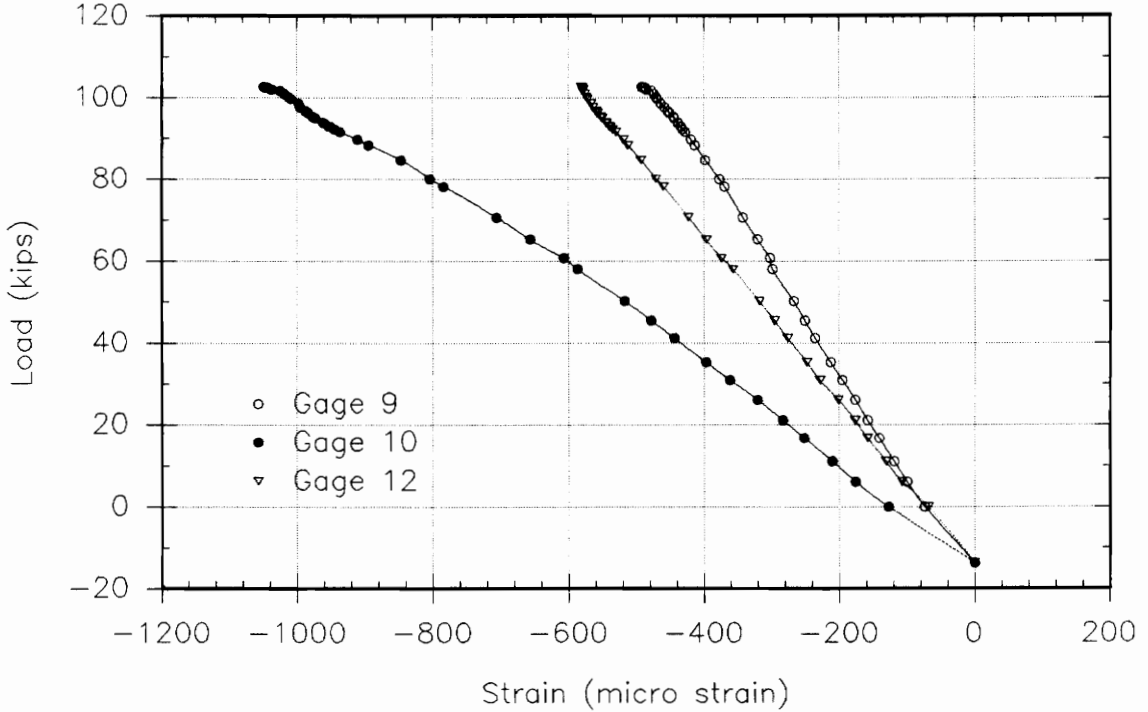


Figure B.6.4 Applied Load vs. W3 Strain

CLH - 6

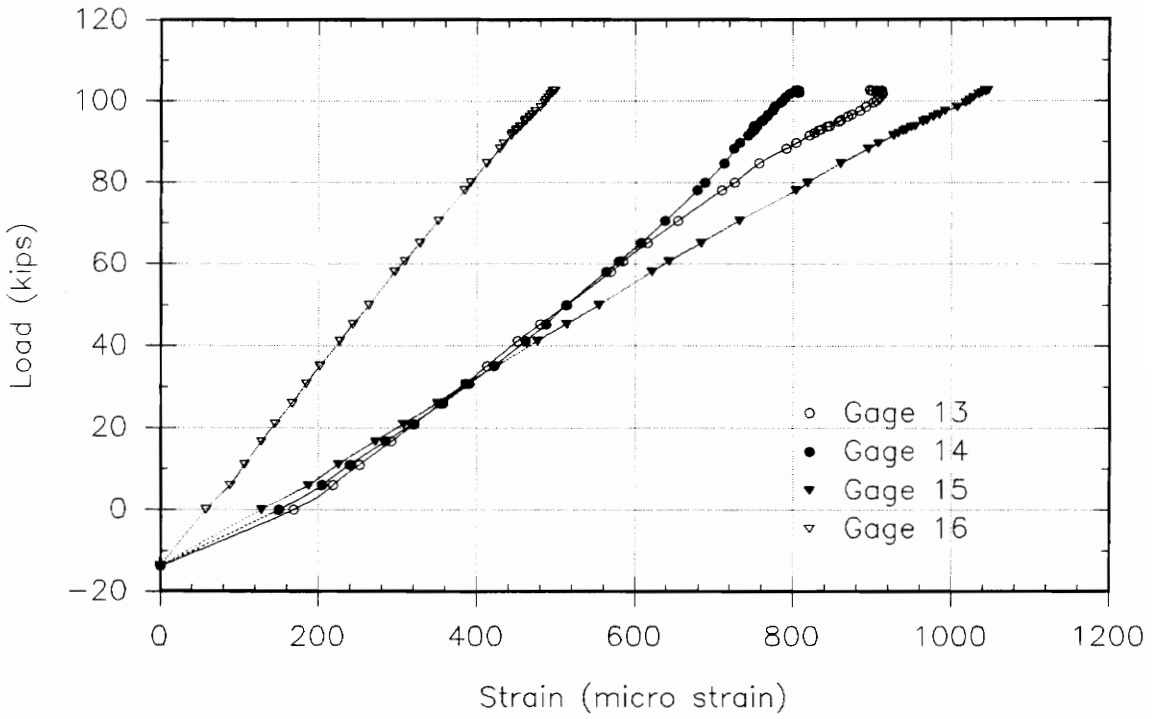


Figure B.6.5 Applied Load vs. W4 Strain

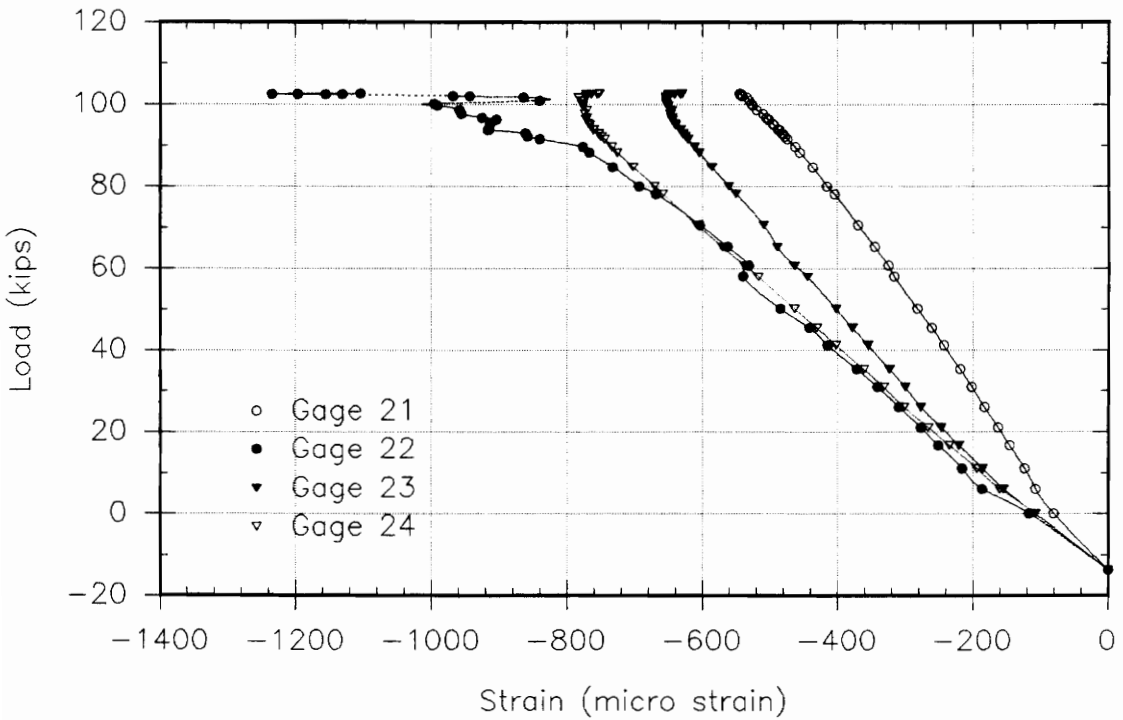


Figure B.6.6 Applied Load vs. W9 Strain

CLH - 6

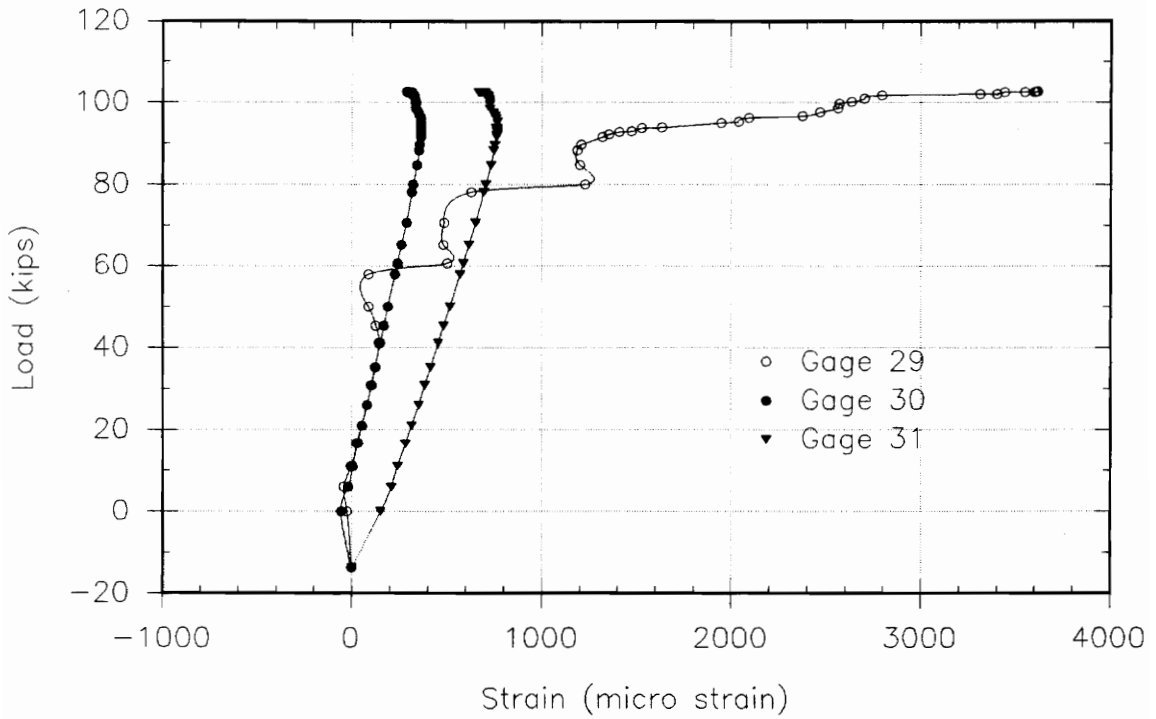


Figure B.6.7 Applied Load vs. W10 Strain

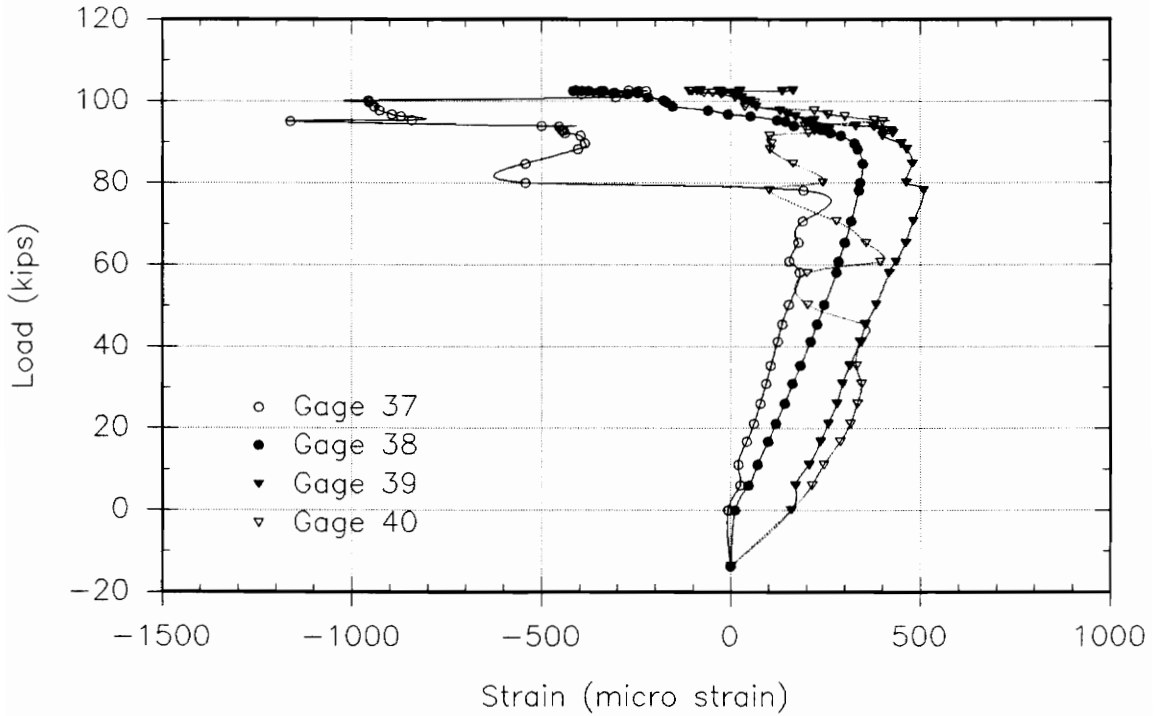


Figure B.6.8 Applied Load vs. W14 Strain

CLH - 6

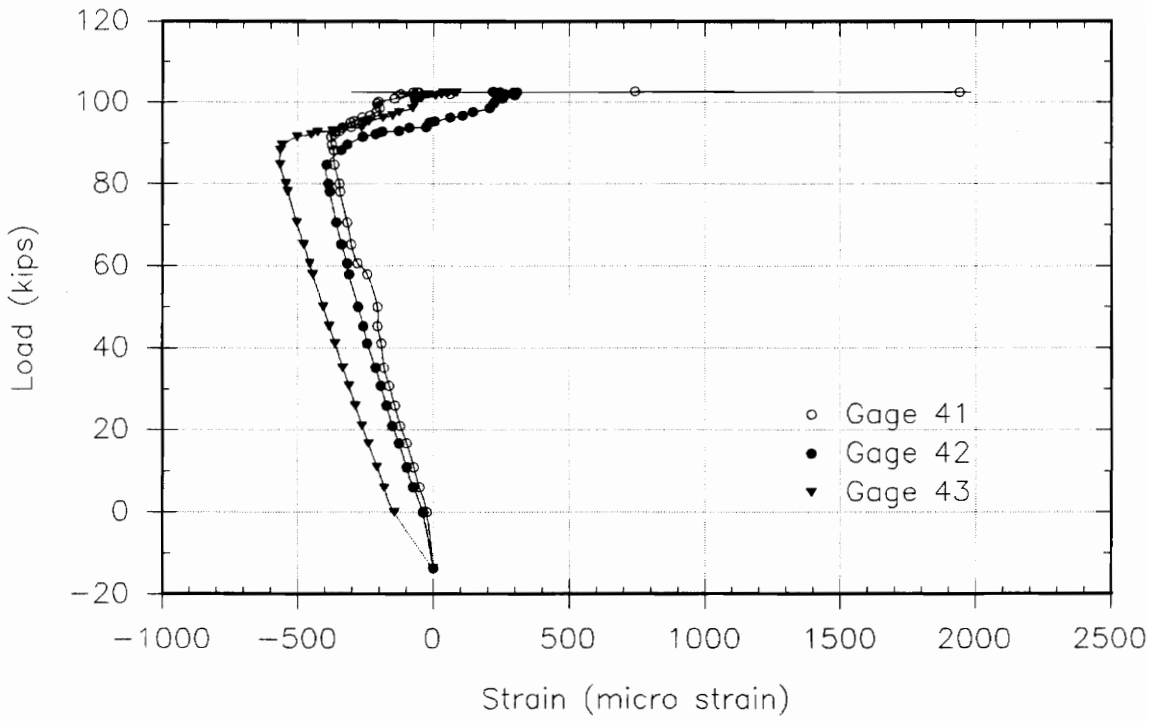


Figure B.6.9 Applied Load vs. W15 Strain

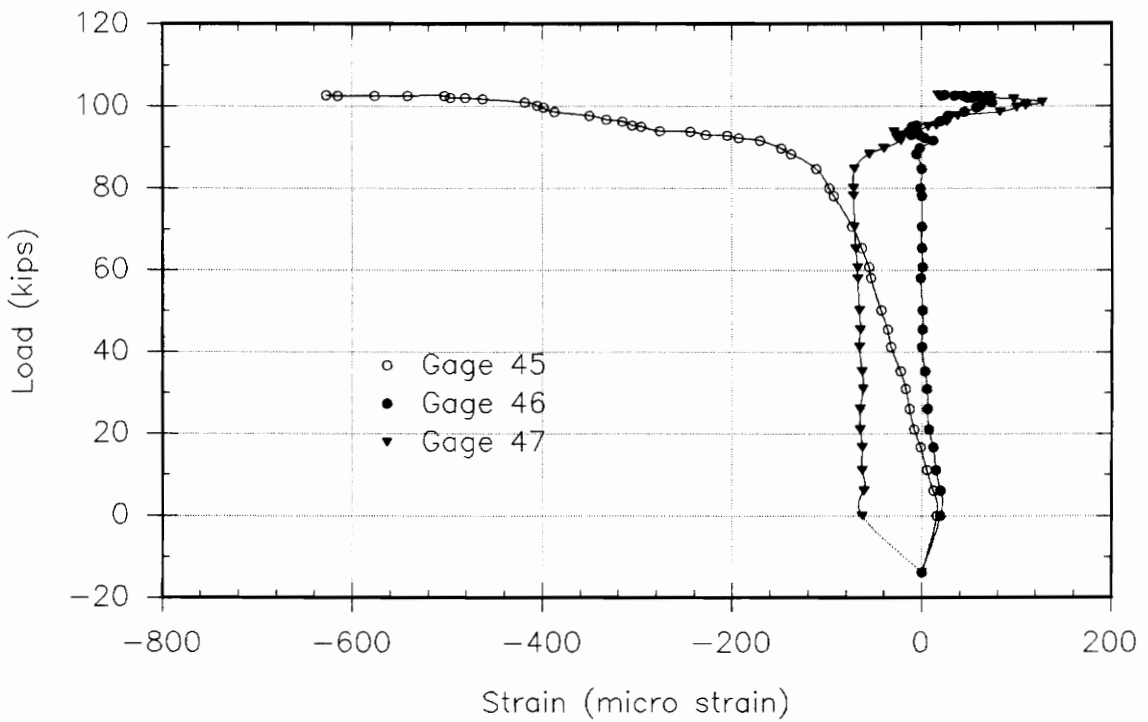


Figure B.6.10 Applied Load vs. W17 Strain

CLH - 6

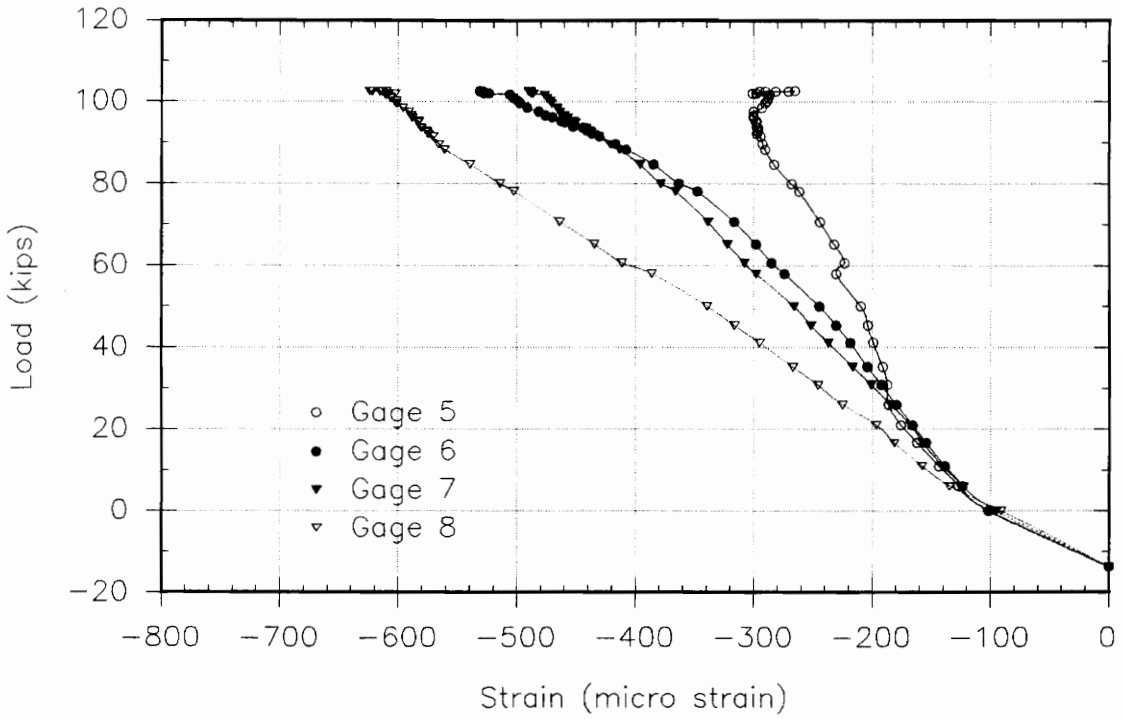


Figure B.6.11 Applied Load vs. Top Chord Strain (TC1)

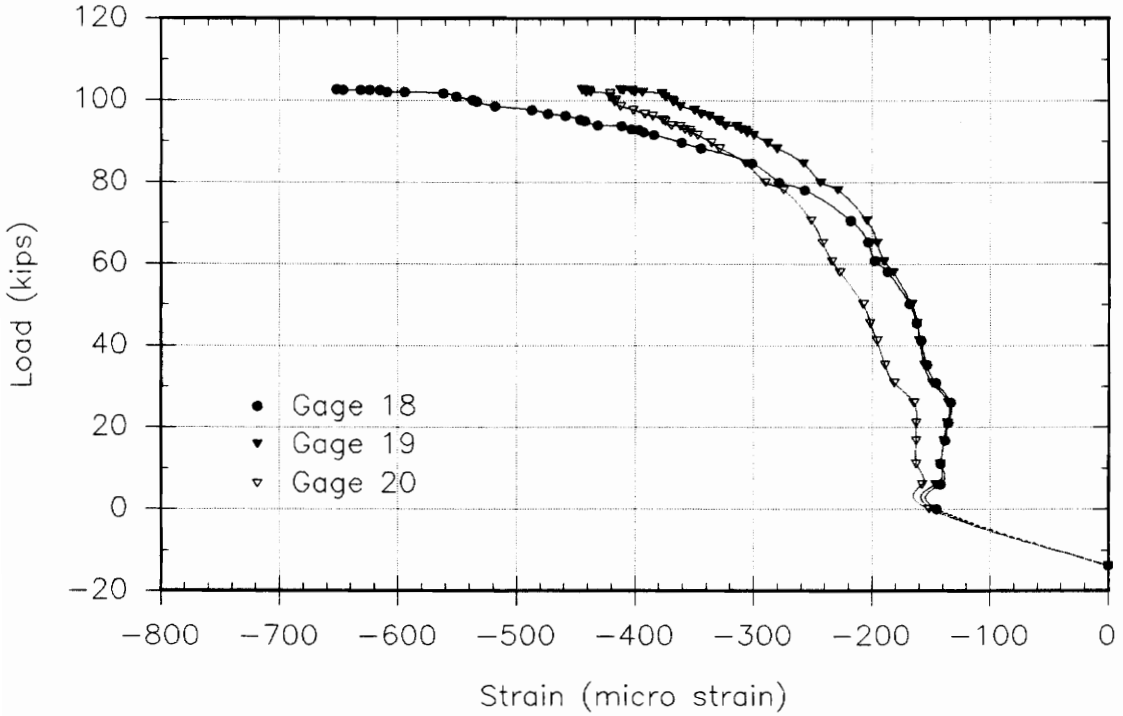


Figure B.6.12 Applied Load vs. Top Chord Strain (TC2)

CLH - 6

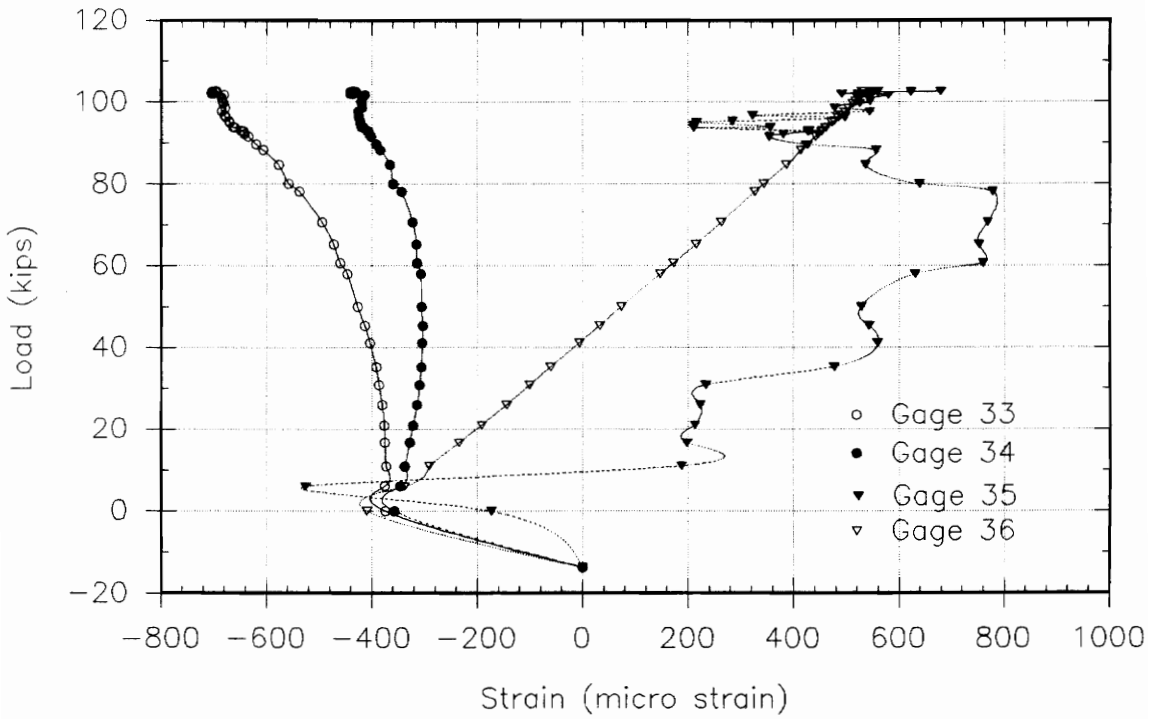


Figure B.6.13 Applied Load vs. Top Chord Strain (TC3)

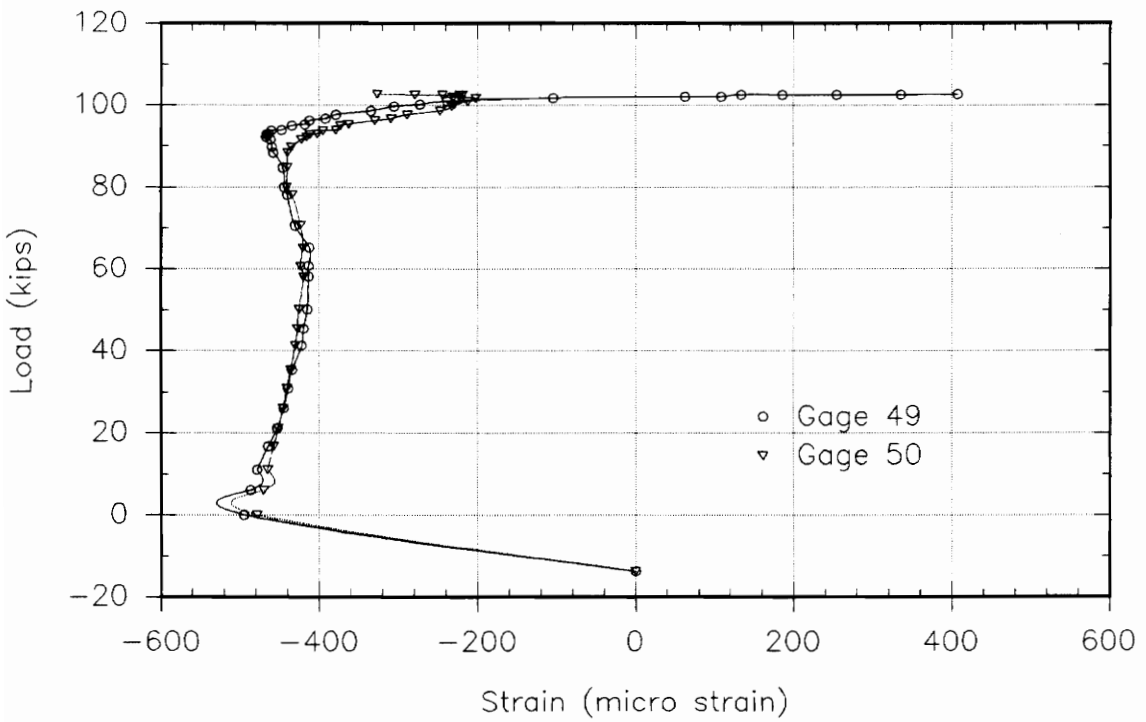


Figure B.6.14 Applied Load vs. Top Chord Strain (TC4)

CLH - 6

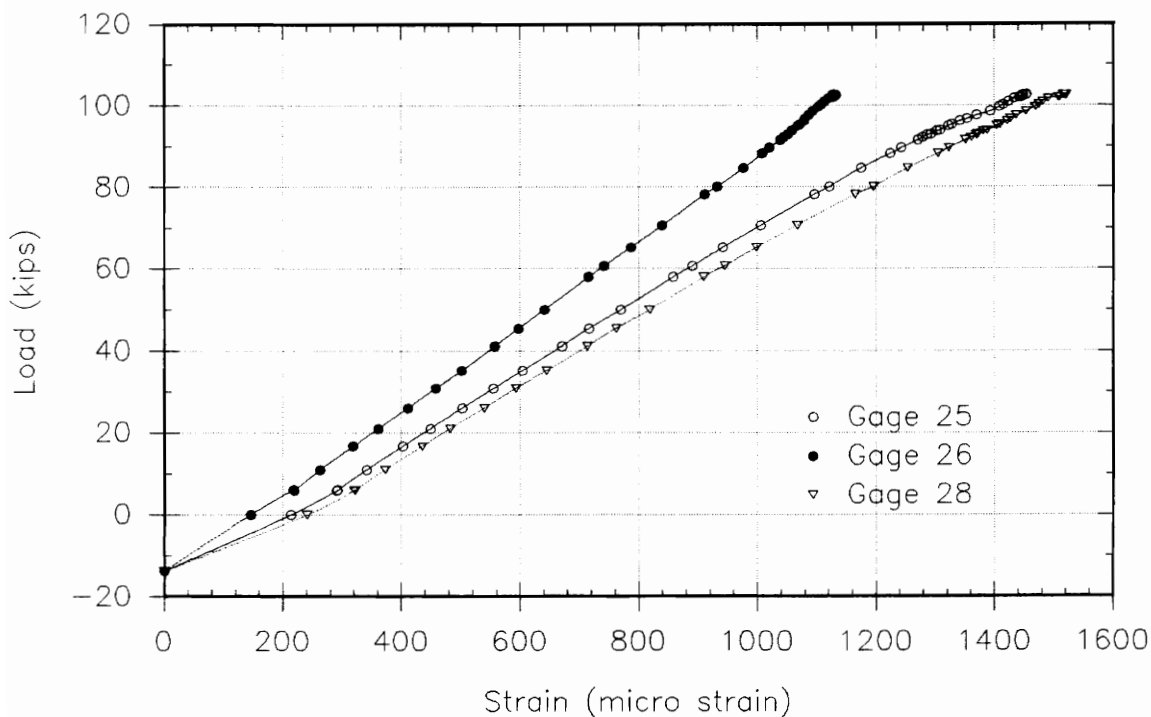


Figure B.6.15 Applied Load vs. Bottom Chord Strain (BC1)

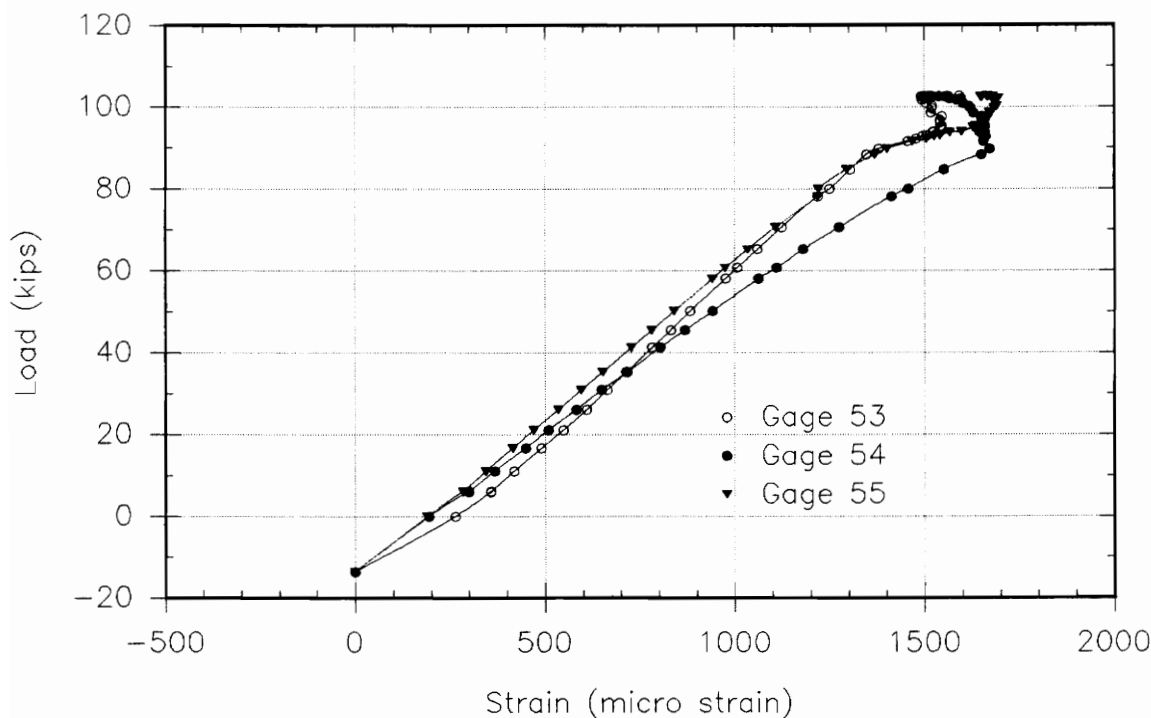


Figure B.6.16 Applied Load vs. Bottom Chord Strain (BC2)

CLH - 6

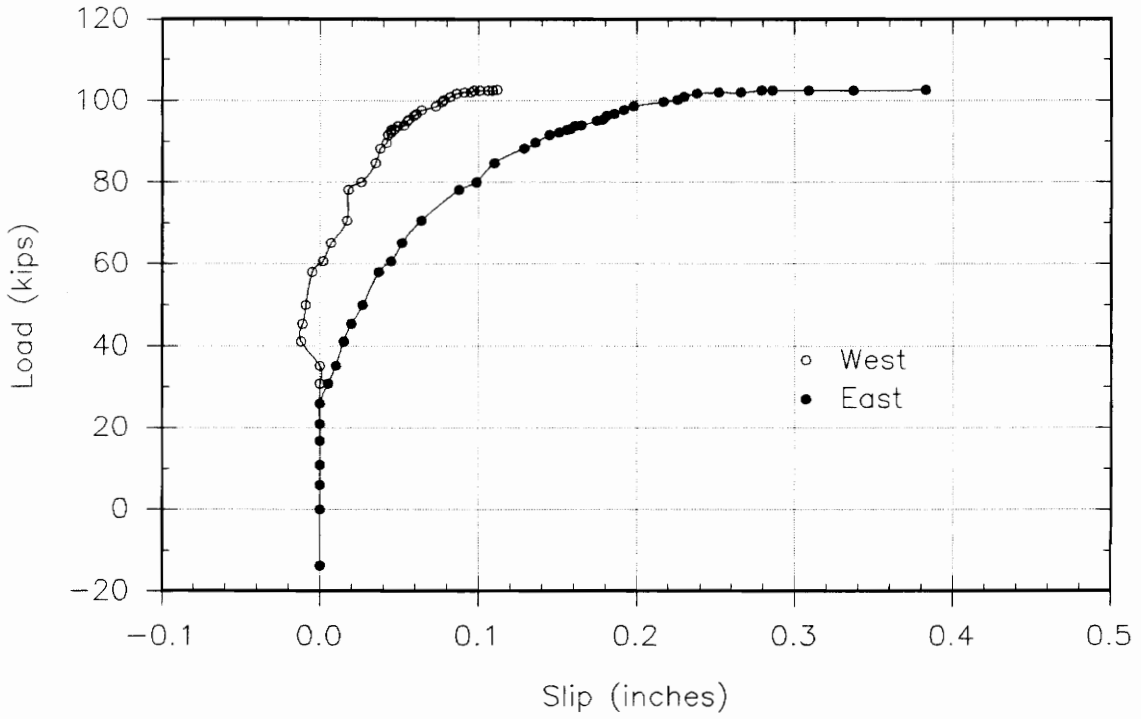


Figure B.6.17 Applied Load vs. End Slip

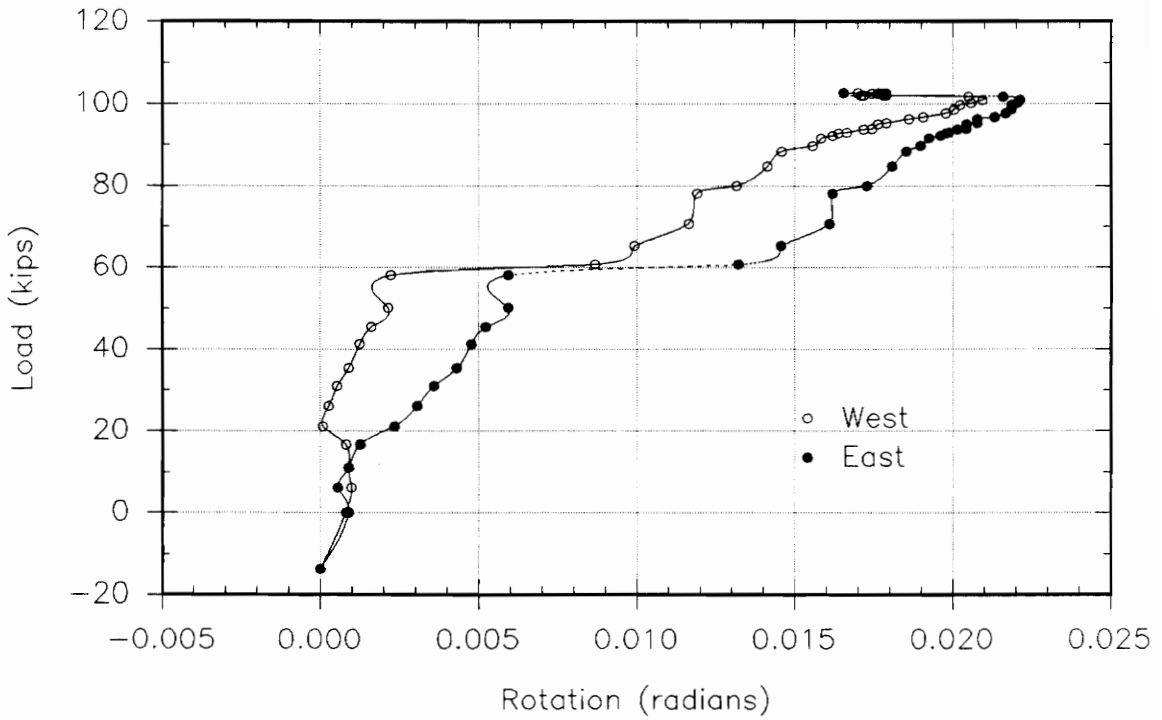


Figure B.6.18 Applied Load vs. End Rotation

TEST SUMMARY SHEET

TEST DESIGNATION: CLH-7 TEST DATE: 14 November 1990
 TEST LOCATION: Structures and Materials Research Laboratory

JOIST DESCRIPTION

Depth: 20 in Top Chord: 2L3.00x3.00x.313
 Length: 40 ft Bottom Chord: 2L4.00x4.00x.438
 Number of Studs: 36 Width of Slab: 81 in
 Dead Weight of Joist and Slab: 389 plf

CALCULATED CAPACITIES (using measured material properties)

Theoretical Max Applied Load: 131.79 k
 Theoretical Moment of Inertia: 2366 in⁴
 Theoretical Elastic Deflection: 2.783 in

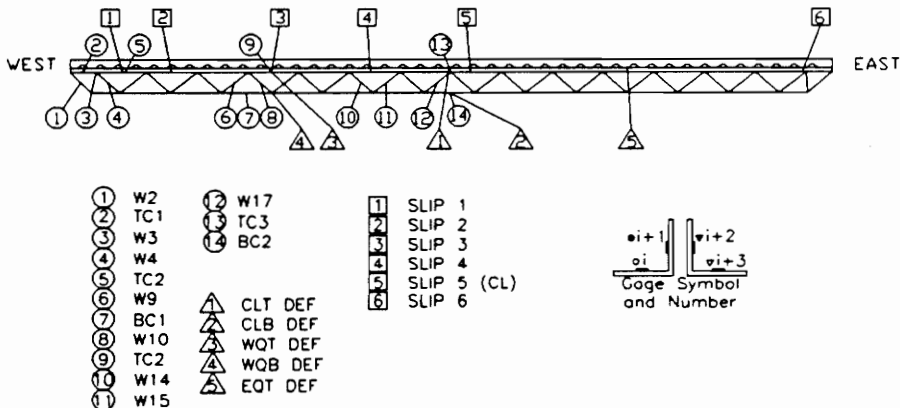
TEST RESULTS

Maximum Applied Load: 151.6 k Max Deflection: 18.814 in
 Yield Load: 129.1 k Deflection at Yield: 3.702 in
 Average Max Bottom Chord Strain: 16232 micro strain
 Actual Bottom Chord Fy: 57.14 ksi f'c: 5.72 ksi
 Concrete Pour Strains Recorded: YES
 Maximum Slip: 1.514 in
 Failure: Local Buckling of the along the entire Top Chord

COMPARISON OF ACTUAL TO THEORETICAL

Actual stiffness/theoretical stiffness: 1.07
 Actual Max Load/Theoretical Max Load: 1.15

Strain Gauge Locations:



Comments: Web members were designed to carry 30% greater load than normal. Bottom chord yielded completely at the centerline. Top chord buckled at multiple locations where shear studs were attached. Test carried higher load than predicted.

CLH - 7

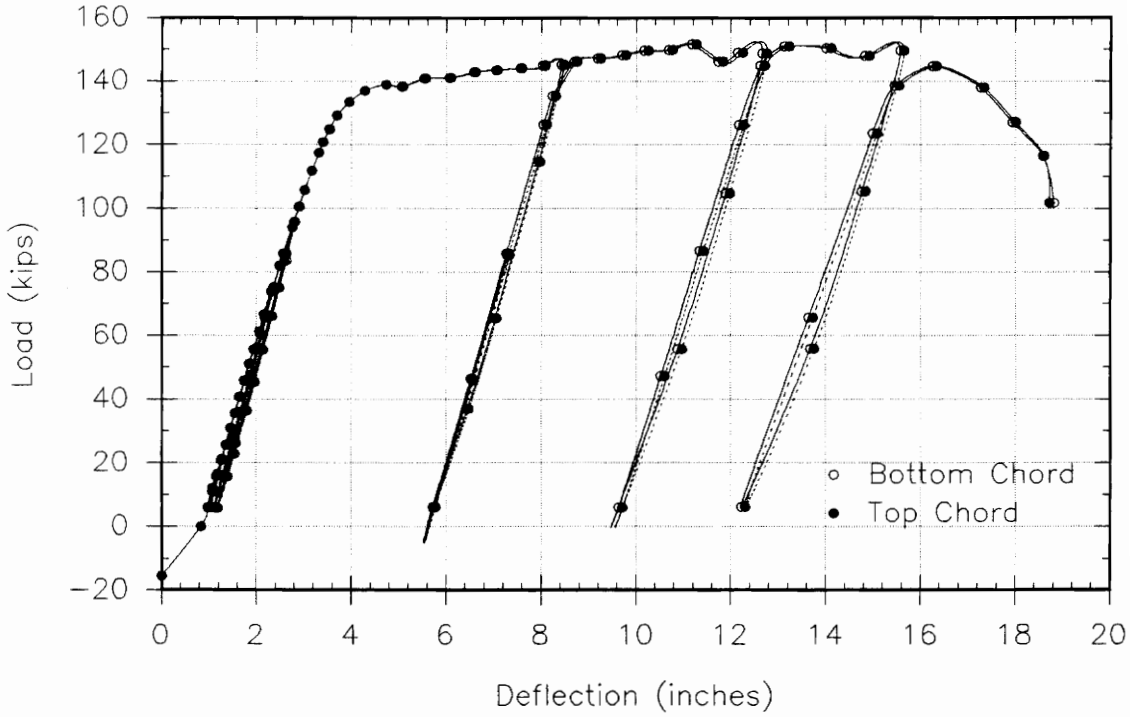


Figure B.7.1 Applied Load vs. Centerline Deflection

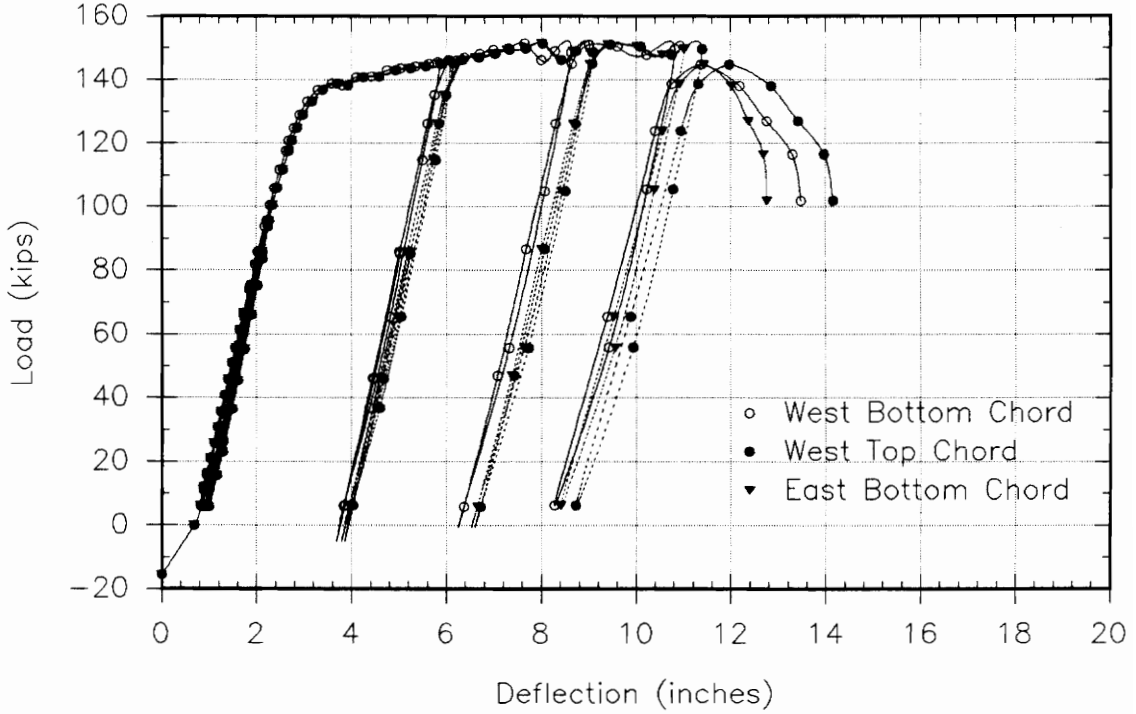


Figure B.7.2 Applied Load vs. Quarter Point Deflection

CLH - 7

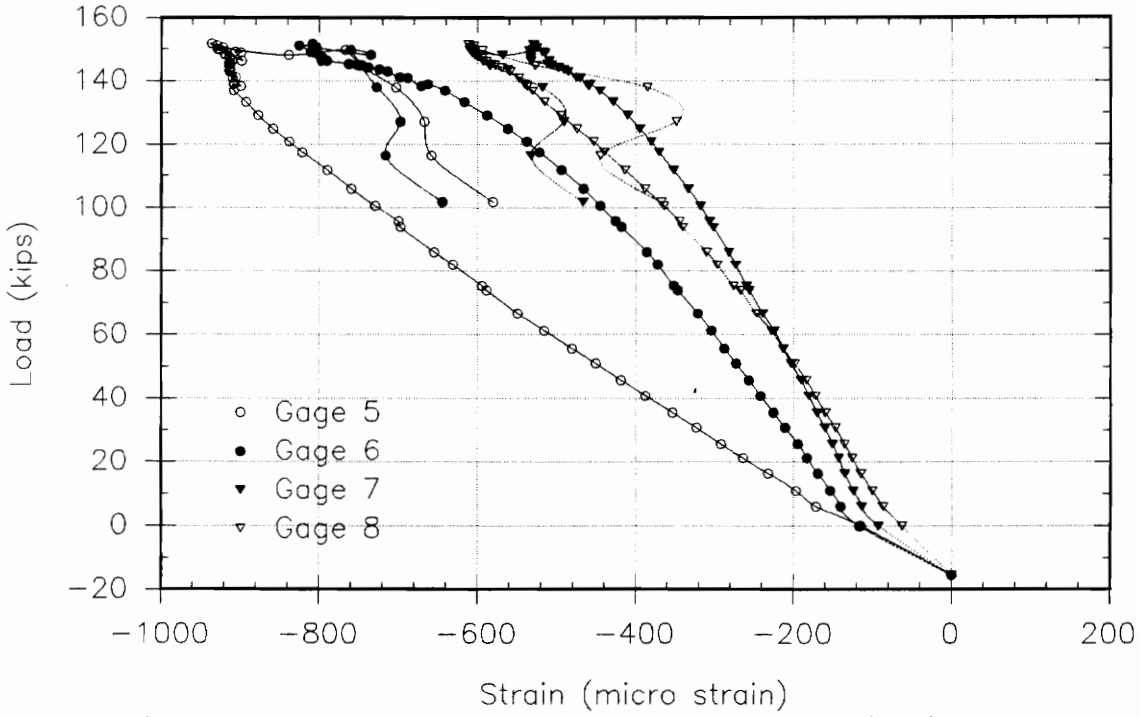


Figure B.7.3 Applied Load vs. Top Chord Strain (TC1)

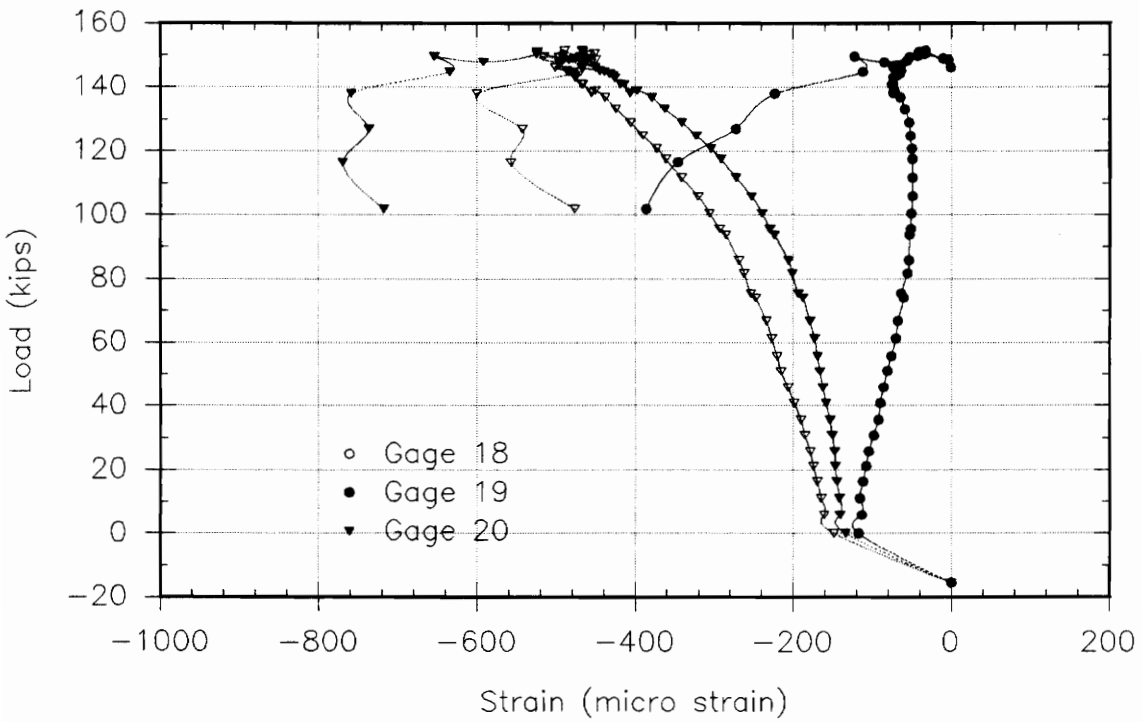


Figure B.7.4 Applied Load vs. Top Chord Strain (TC2)

CLH - 7

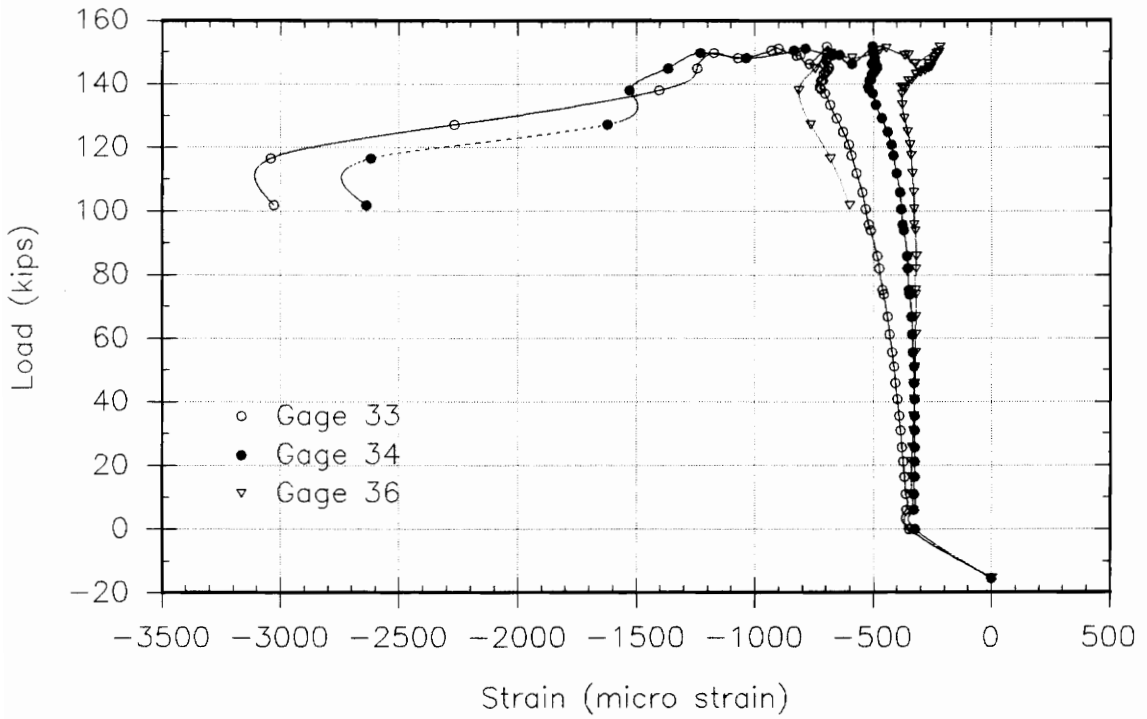


Figure B.7.5 Applied Load vs. Top Chord Strain (TC3)

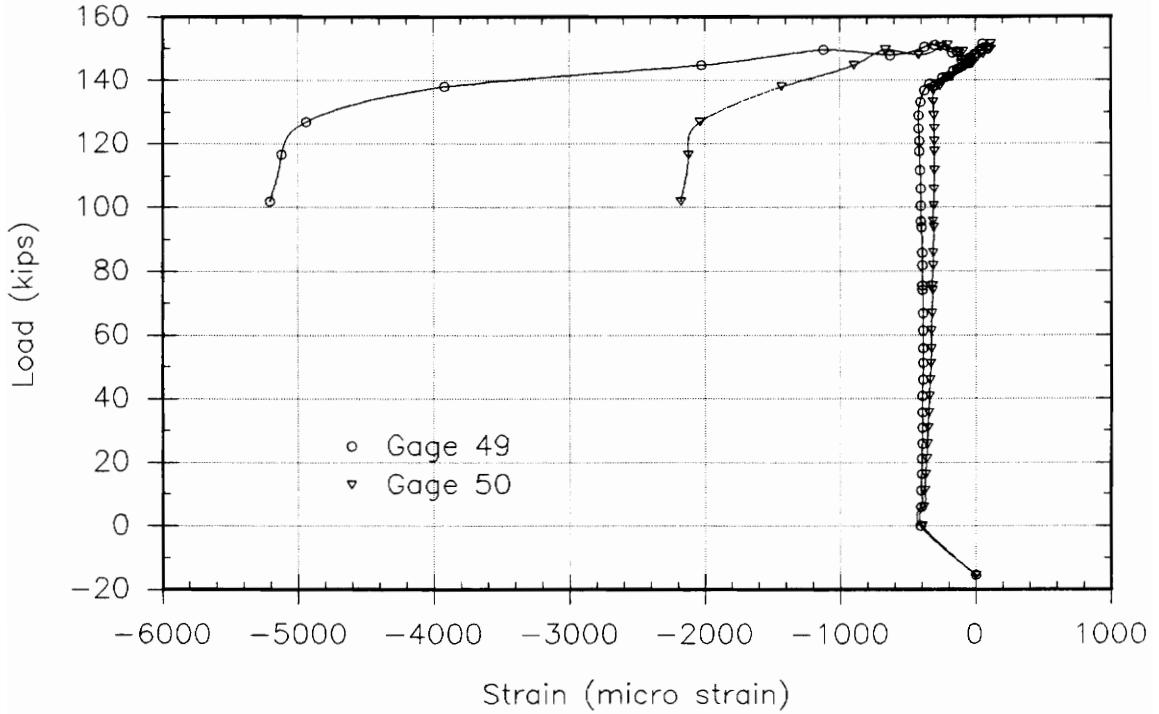


Figure B.7.6 Applied Load vs. Top Chord Strain (TC4)

CLH - 7

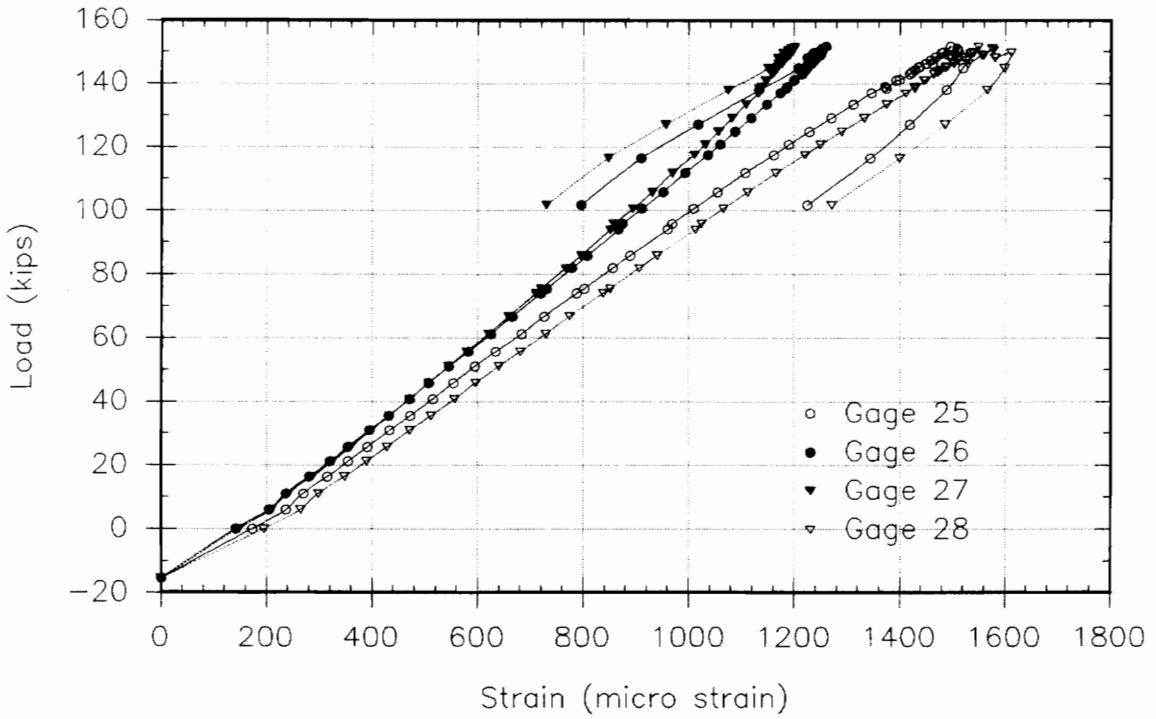


Figure B.7.7 Applied Load vs. Bottom Chord Strain (BC1)

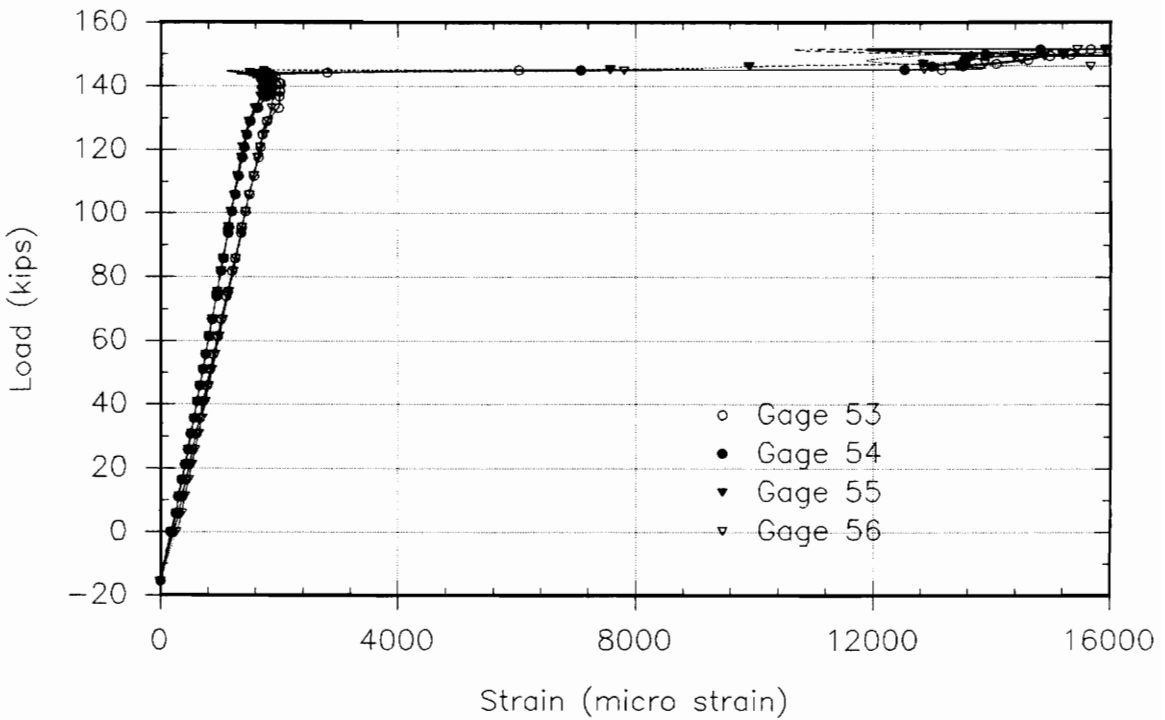


Figure B.7.8 Applied Load vs. Bottom Chord Strain (BC2)

CLH - 7

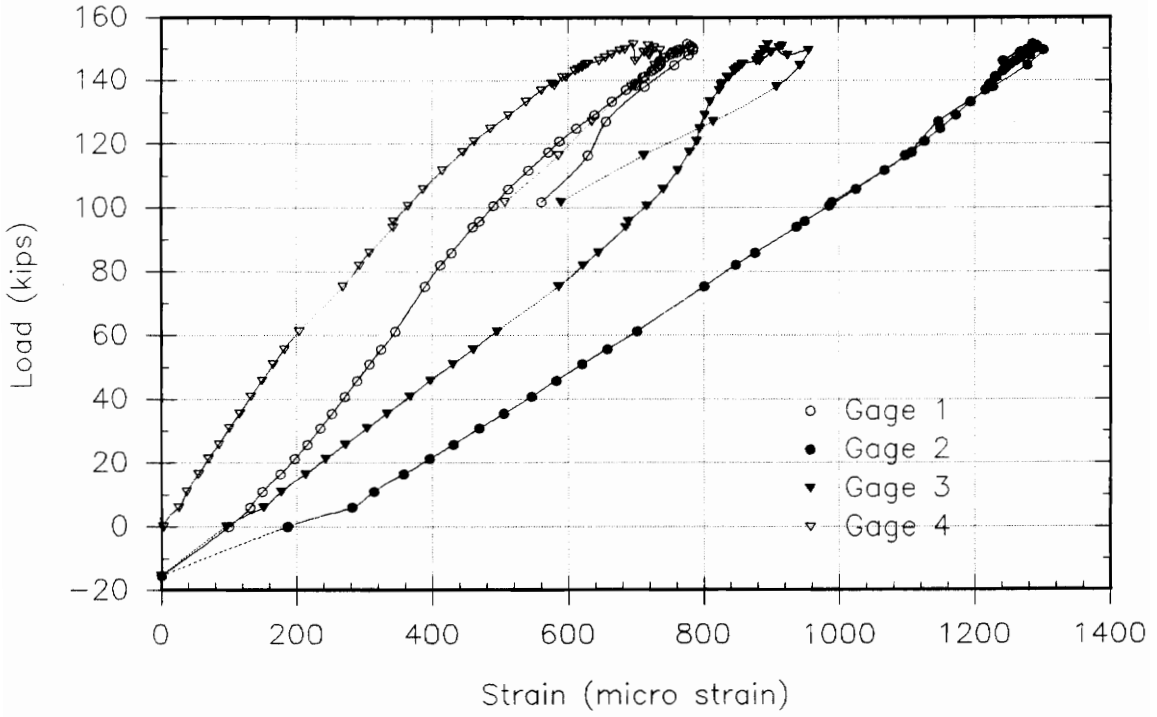


Figure B.7.9 Applied Load vs. W2 Strain

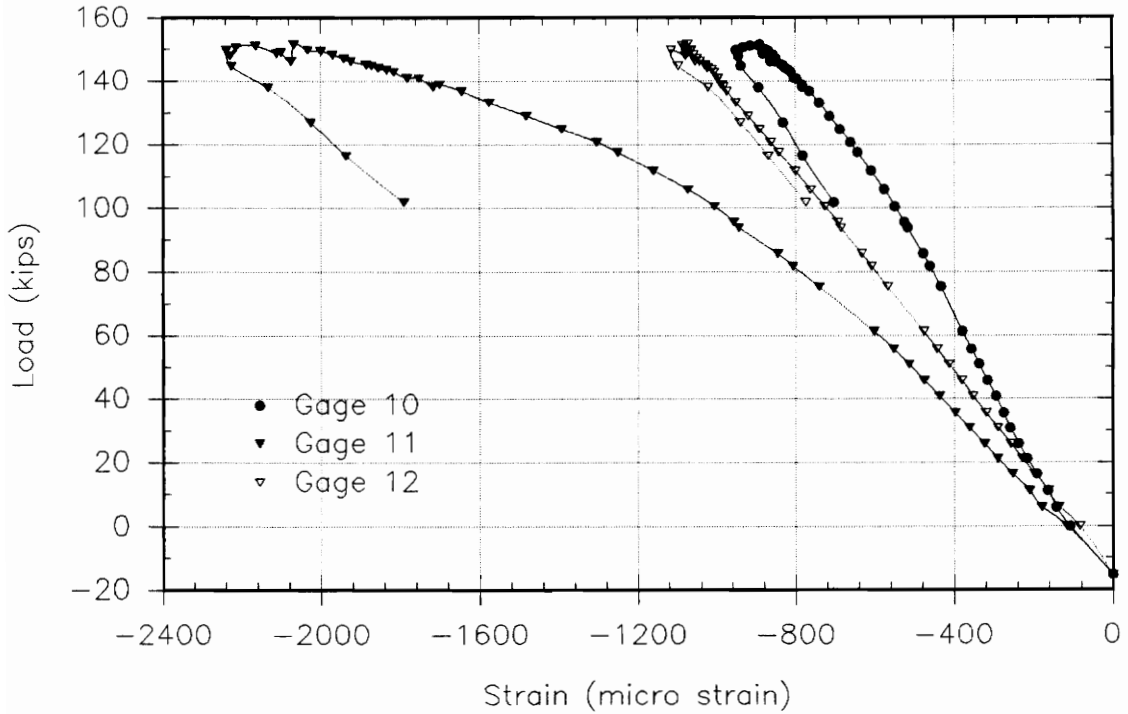


Figure B.7.10 Applied Load vs. W3 Strain

CLH - 7

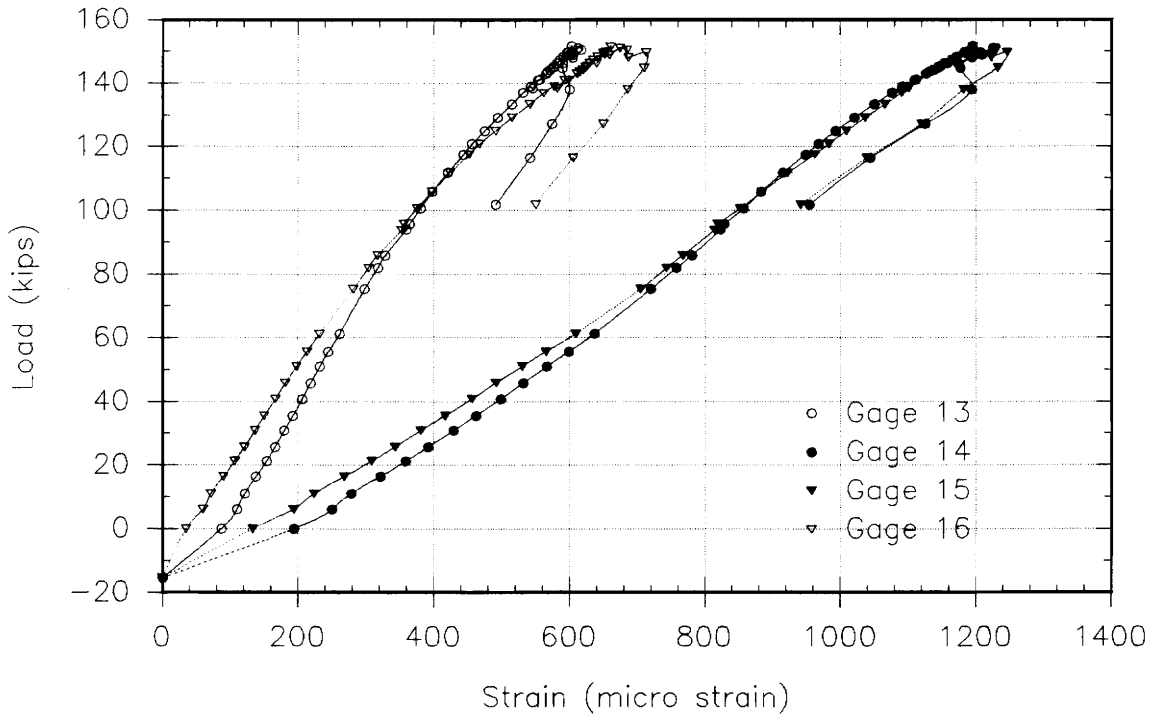


Figure B.7.11 Applied Load vs. W4 Strain

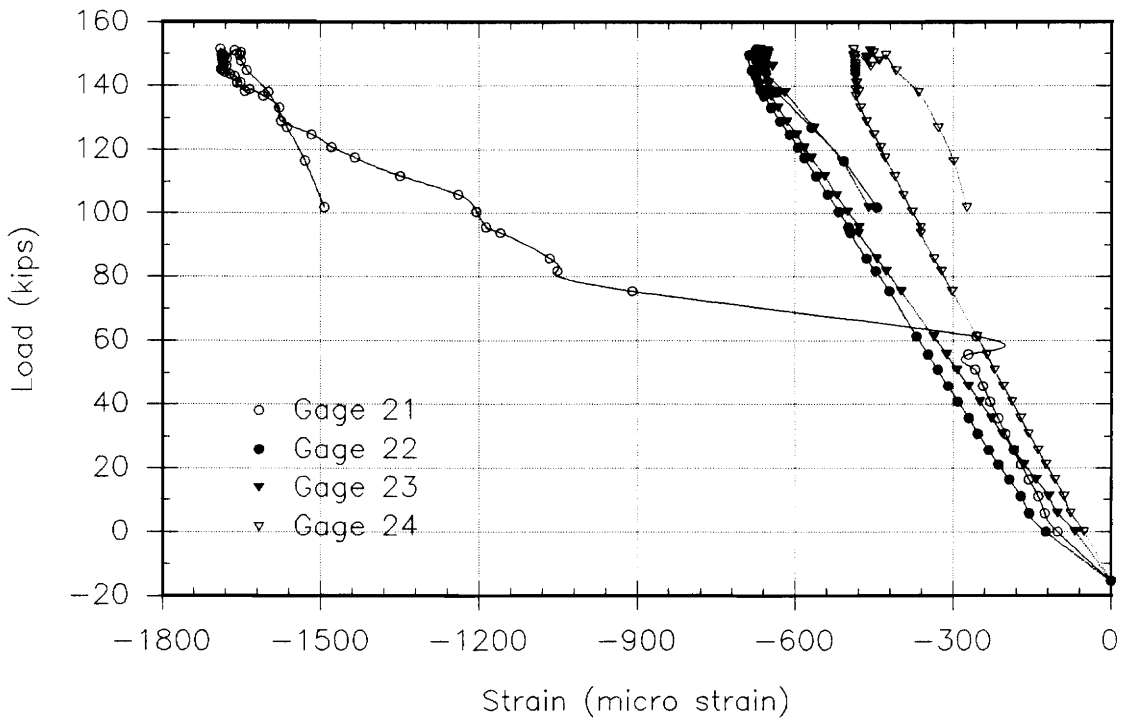


Figure B.7.12 Applied Load vs. W9 Strain

CLH - 7

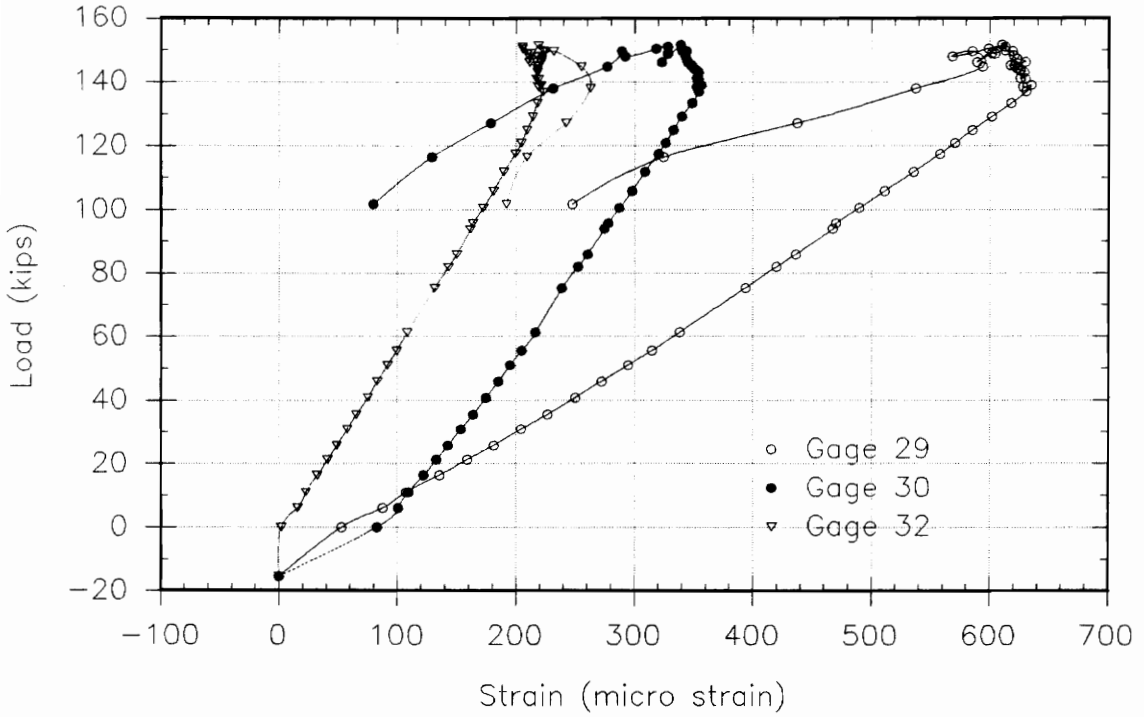


Figure B.7.13 Applied Load vs. W10 Strain

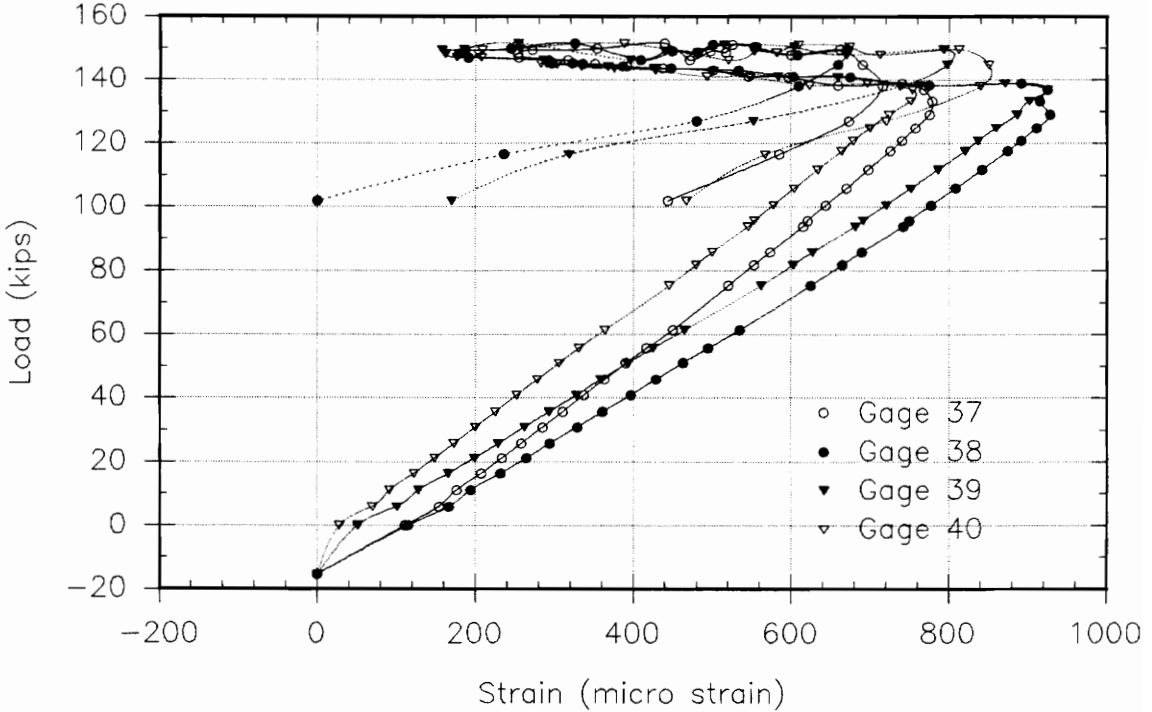


Figure B.7.14 Applied Load vs. W14 Strain

CLH - 7

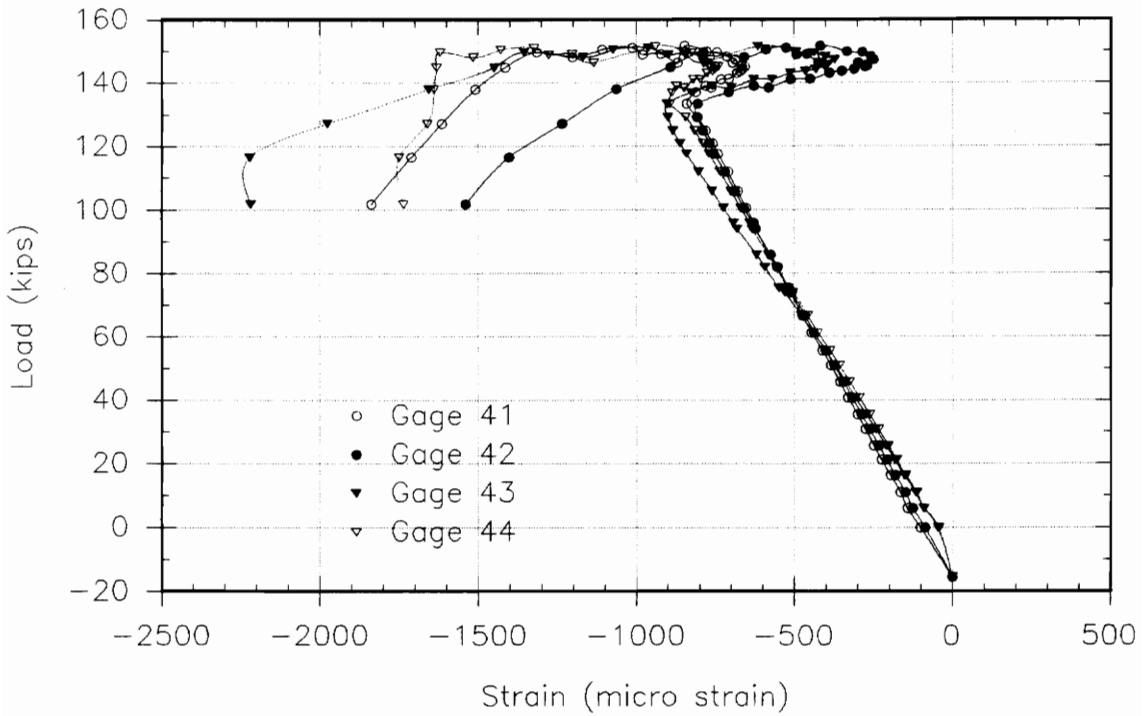


Figure B.7.15 Applied Load vs. W15 Strain

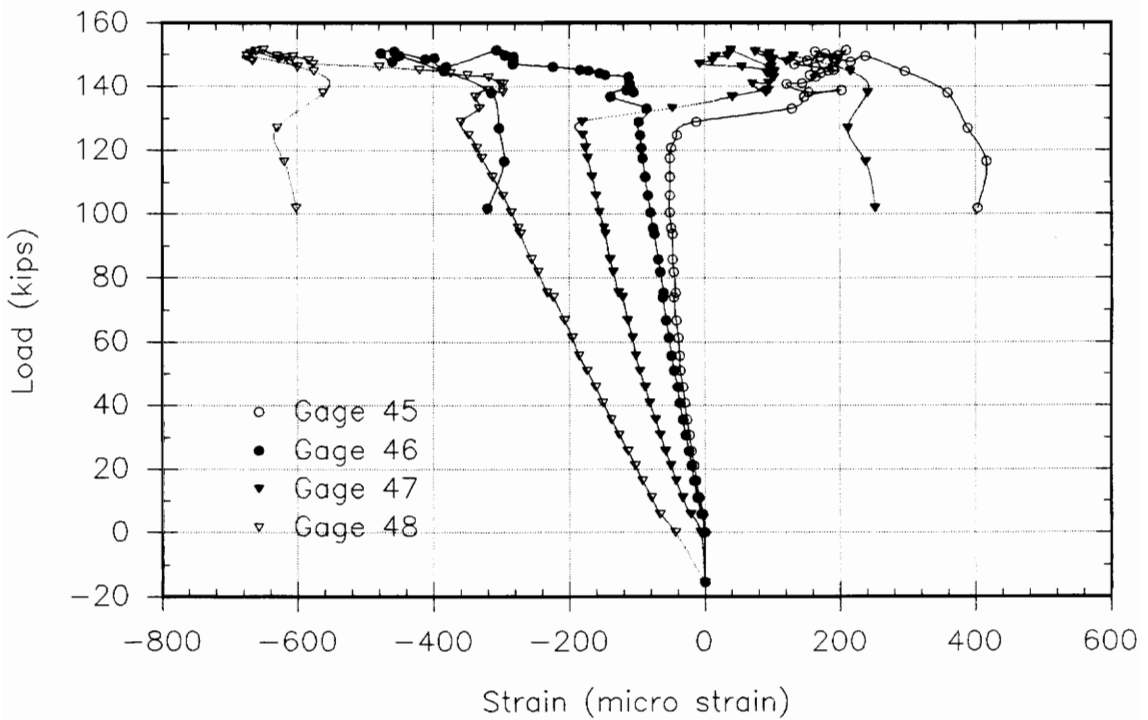


Figure B.7.16 Applied Load vs. W17 Strain

CLH - 7

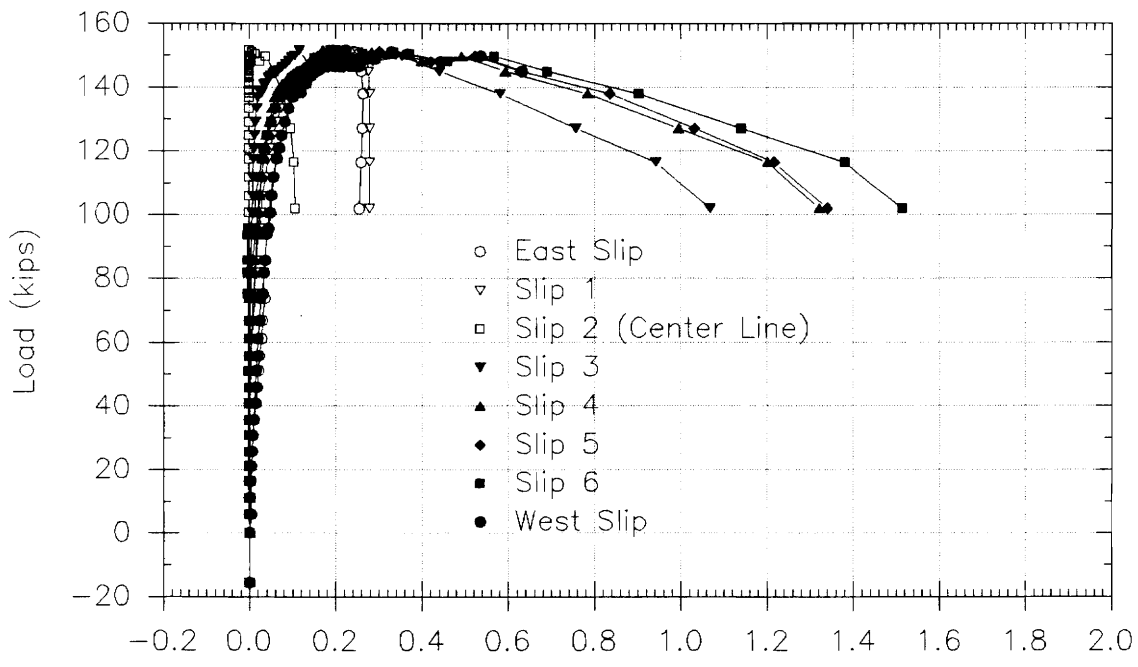


Figure B.7.17 Slip (inches)

TEST SUMMARY SHEET

TEST DESIGNATION: CLH-8 TEST DATE: 6 December 1990
 TEST LOCATION: Structures and Materials Research Laboratory

JOIST DESCRIPTION

Depth: 20 in Top Chord: 2L3.00x3.00x.313
 Length: 40 ft Bottom Chord: 2L4.00x4.00x.438
 Number of Studs: 36 Width of Slab: 81 in
 Dead Weight of Joist and Slab: 387 plf

CALCULATED CAPACITIES (using measured material properties)

Theoretical Max Applied Load: 129.6 k
 Theoretical Moment of Inertia: 2354 in⁴
 Theoretical Elastic Deflection: 2.751 in

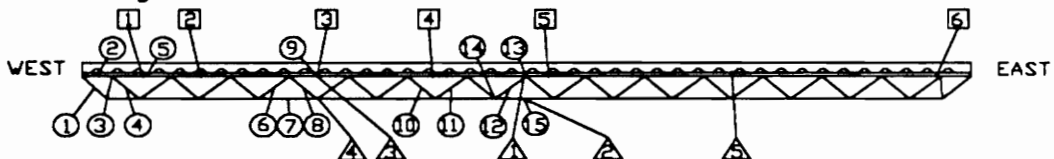
TEST RESULTS

Maximum Applied Load: 152.2 k Max Deflection: 18.791 in
 Yield Load: 128.8 k Deflection at Yield: 3.444 in
 Average Max Bottom Chord Strain: 11388 micro strain
 Actual Bottom Chord Fy: 5631 psi f'c: 538 psi
 Concrete Pour Strains Recorded: Yes
 Maximum Slip: 1.560 in
 Failure: Local Buckling of the along the entire Top Chord

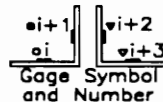
COMPARISON OF ACTUAL TO THEORETICAL

Actual stiffness/theoretical stiffness: 1.02
 Actual Max Load/Theoretical Max Load: 1.17

Strain Gauge Locations:



- | | | |
|-------|-----------|---------------|
| ① W2 | ⑫ W17 | ① SLIP 1 |
| ② TC1 | ⑬ TC3 | ② SLIP 2 |
| ③ W3 | ⑭ BC2 | ③ SLIP 3 |
| ④ W4 | ⑮ BC3 | ④ SLIP 4 |
| ⑤ TC2 | | ⑤ SLIP 5 (CL) |
| ⑥ W9 | ▲ CLT DEF | ⑥ SLIP 6 |
| ⑦ BC1 | ▲ CLB DEF | |
| ⑧ W10 | ▲ WQT DEF | |
| ⑨ TC2 | ▲ WQB DEF | |
| ⑩ W14 | ▲ EQT DEF | |
| ⑪ W15 | | |



Comments: Bottom Chord yielded completely at the centerline. Top chord buckled at multiple locations where shear studs were attached. Test carried higher load than predicted. Results similar to CLH-7.

CLH - 8

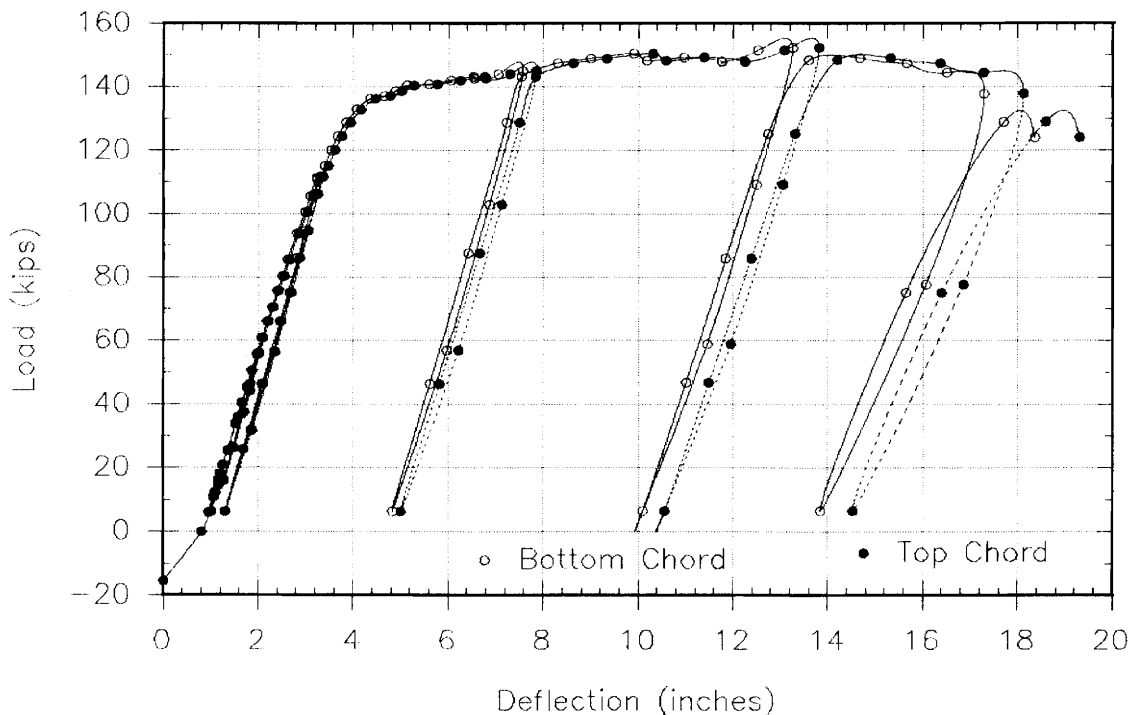


Figure B.8.1 Applied Load vs. Centerline Deflection

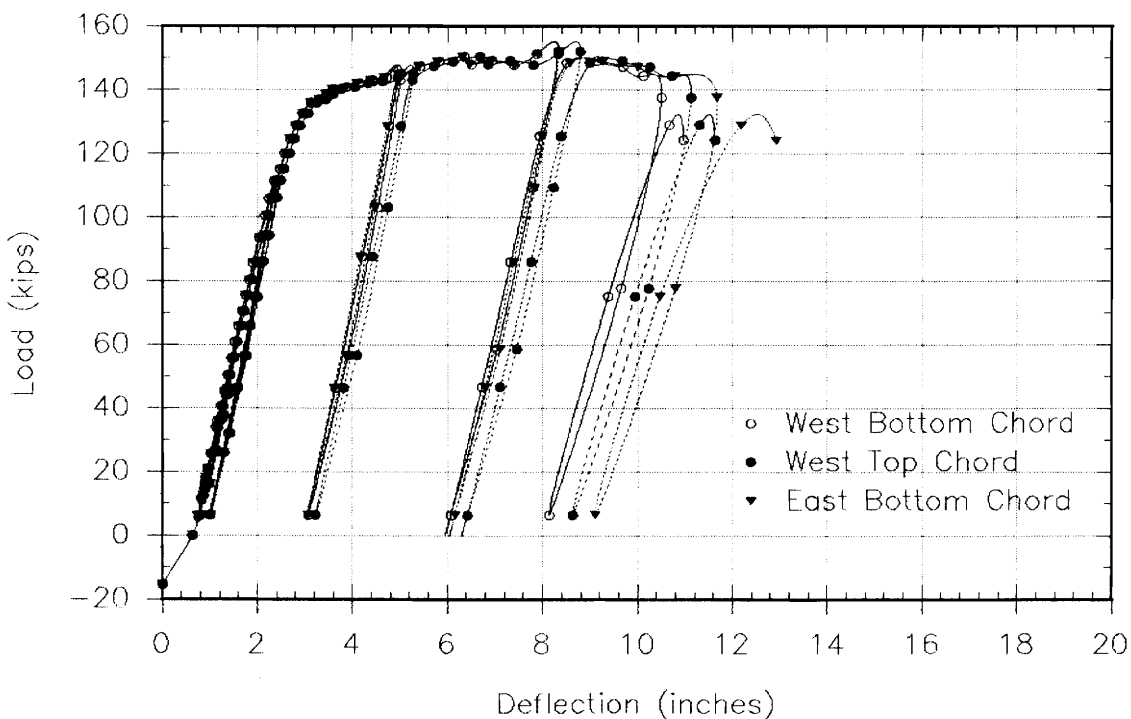


Figure B.8.2 Applied Load vs. Quarter Point Deflection

CLH - 8

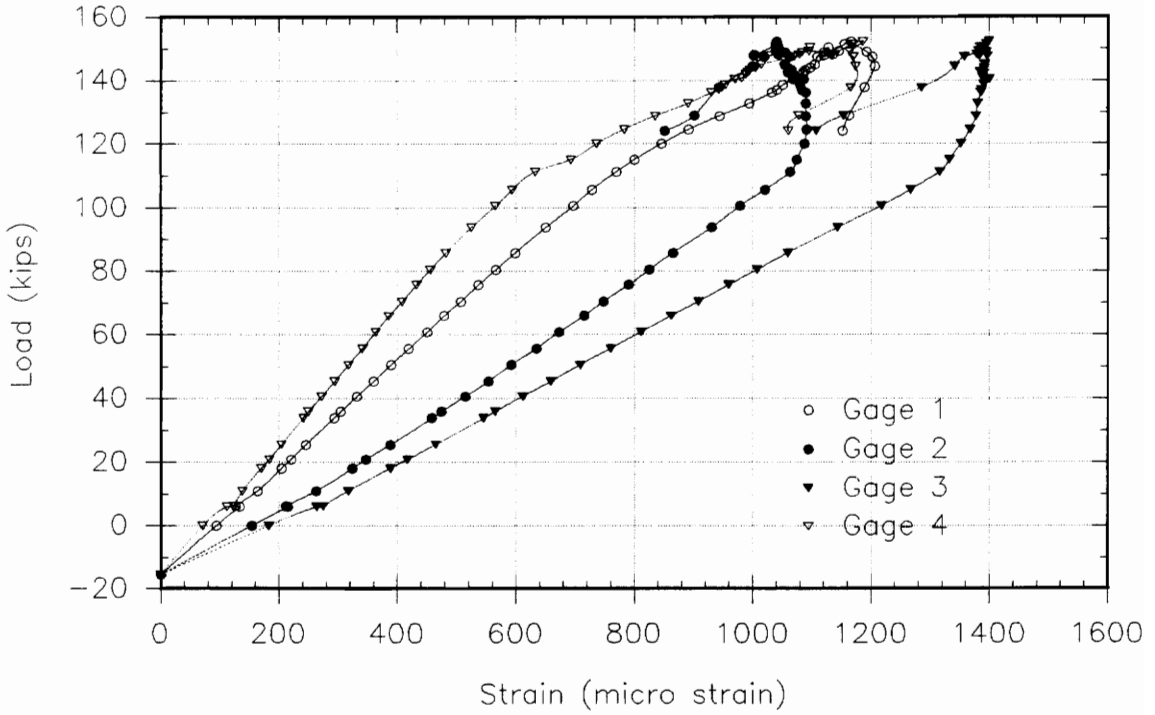


Figure B.8.3 Applied Load vs. W2 Strain

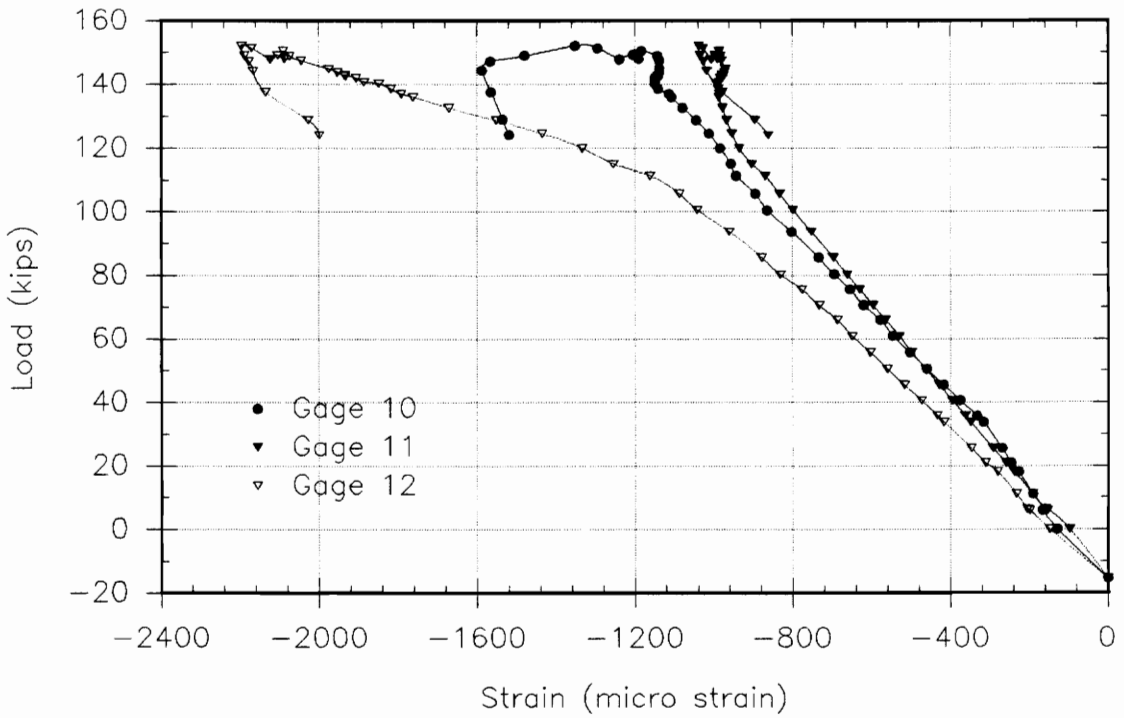


Figure B.8.4 Applied Load vs. W3 Strain

CLH - 8

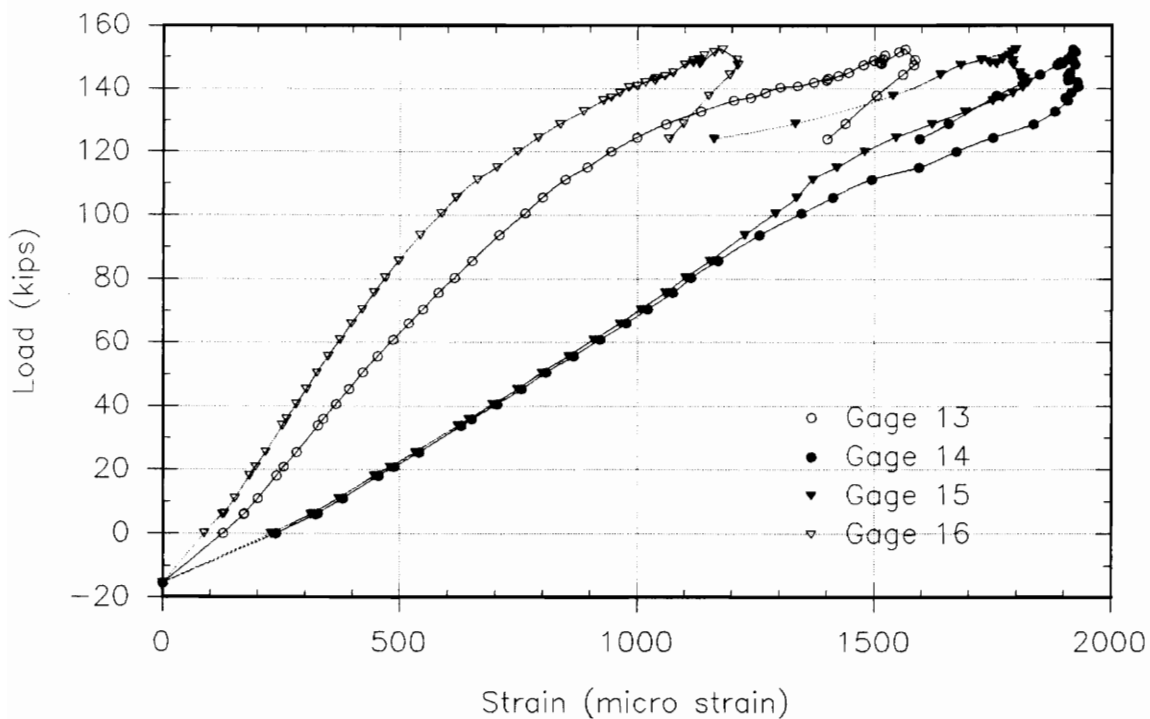


Figure B.8.5 Applied Load vs. W4 Strain

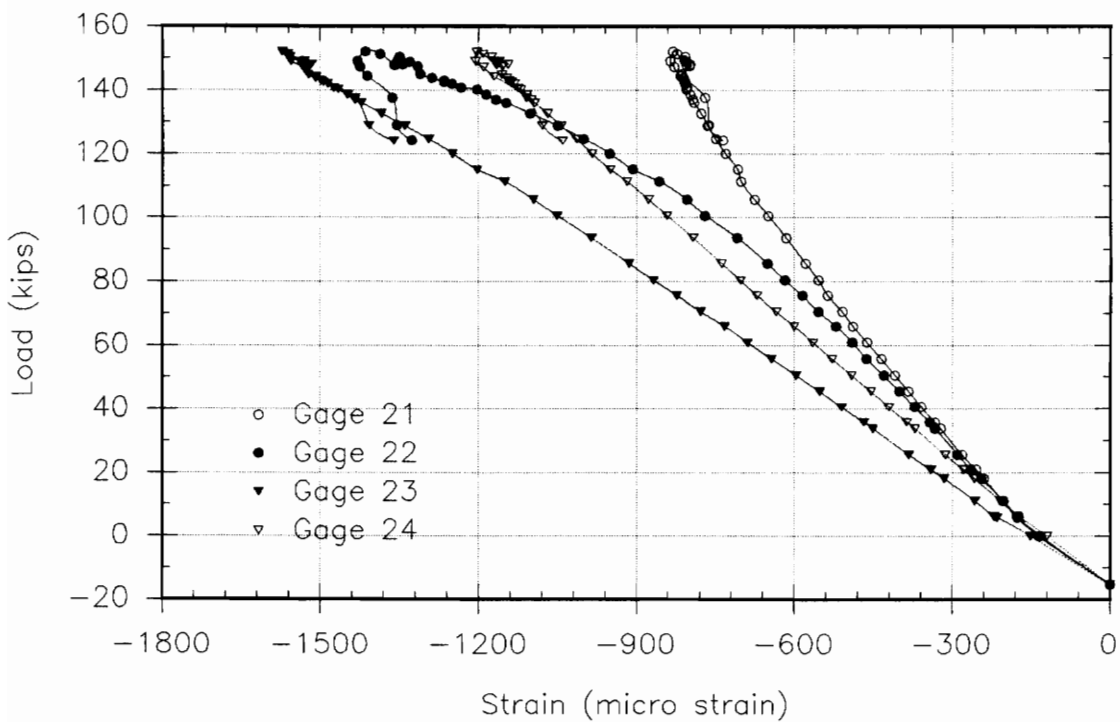


Figure B.8.6 Applied Load vs. W9 Strain

CLH - 8

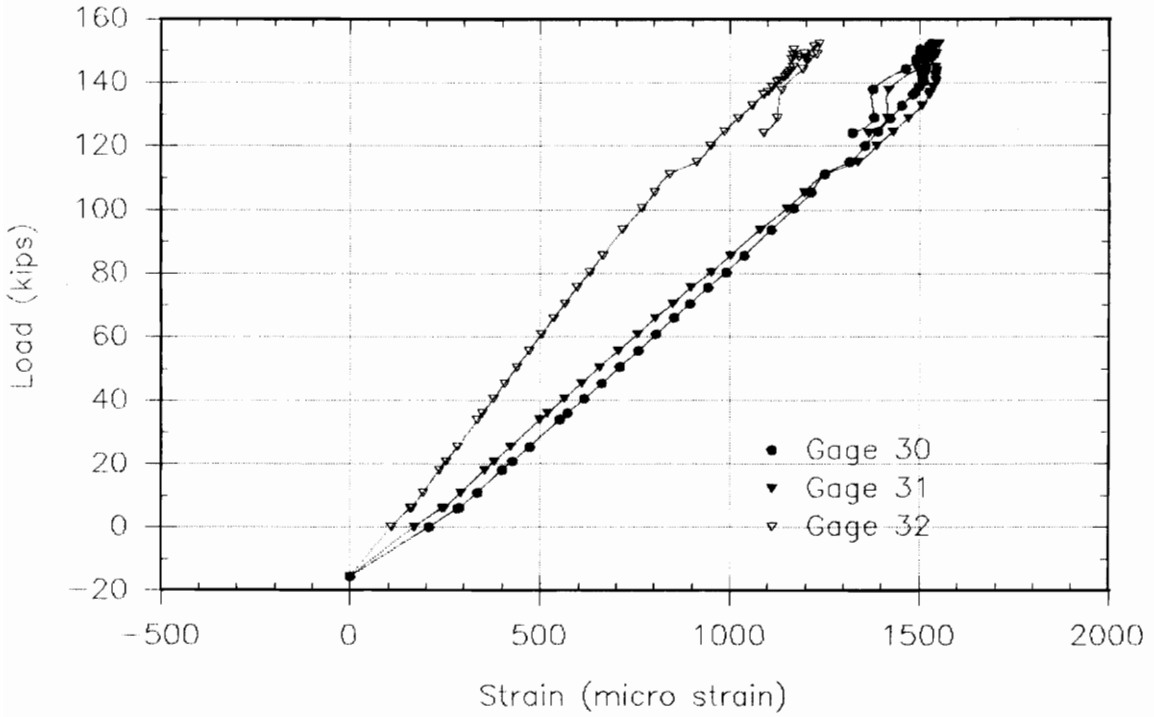


Figure B.8.7 Applied Load vs. W10 Strain

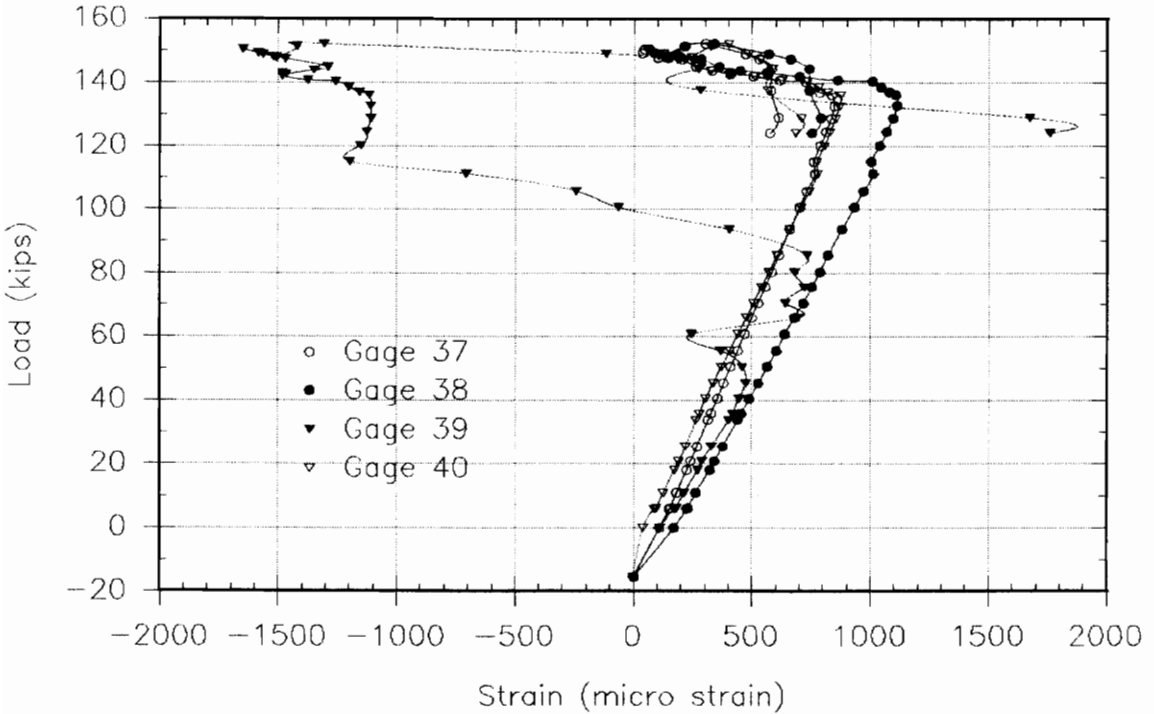


Figure B.8.8 Applied Load vs. W14 Strain

CLH - 8

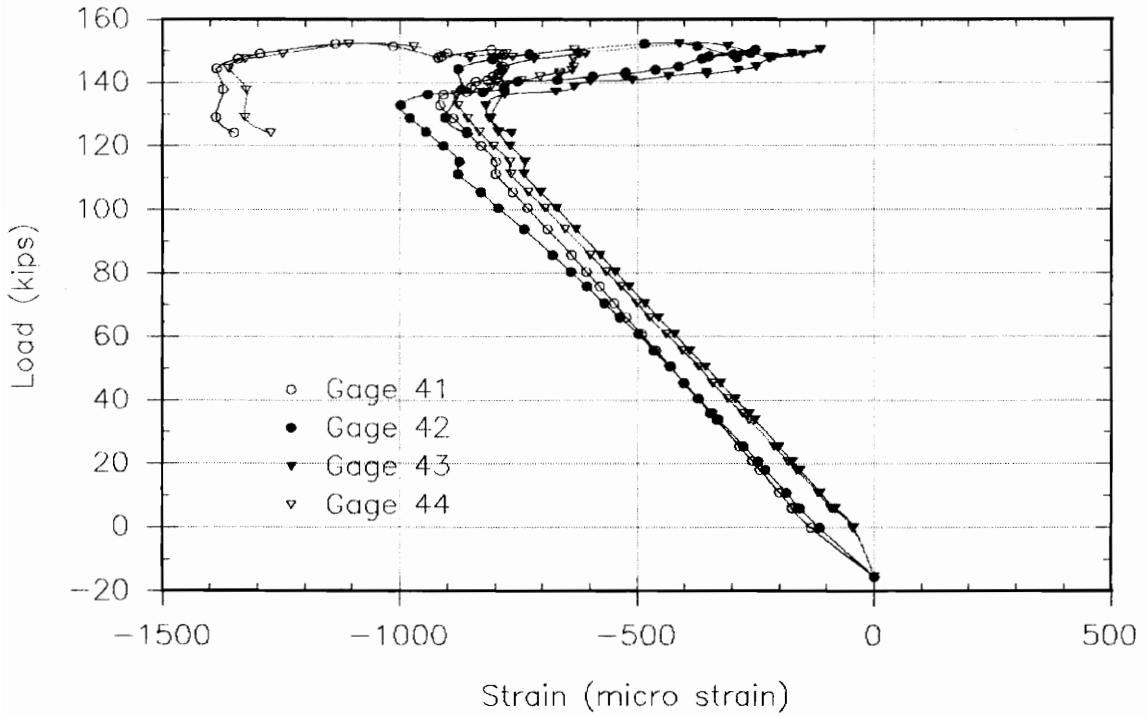


Figure B.8.9 Applied Load vs. W15 Strain

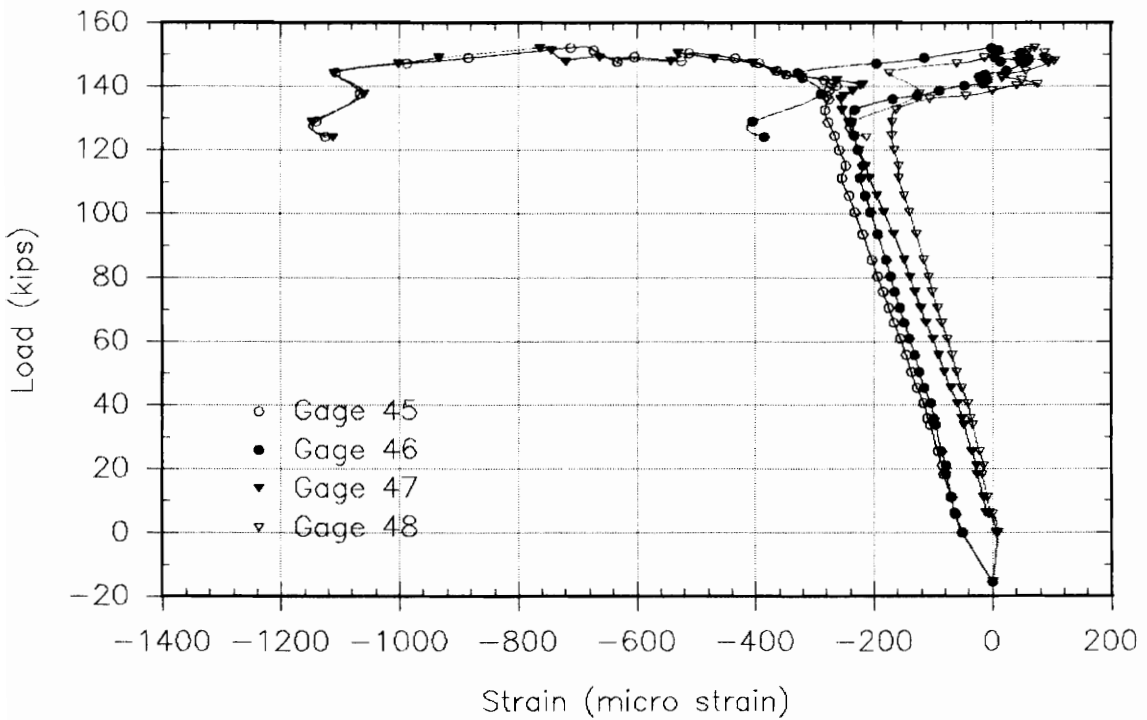


Figure B.8.10 Applied Load vs. W17 Strain

CLH - 8

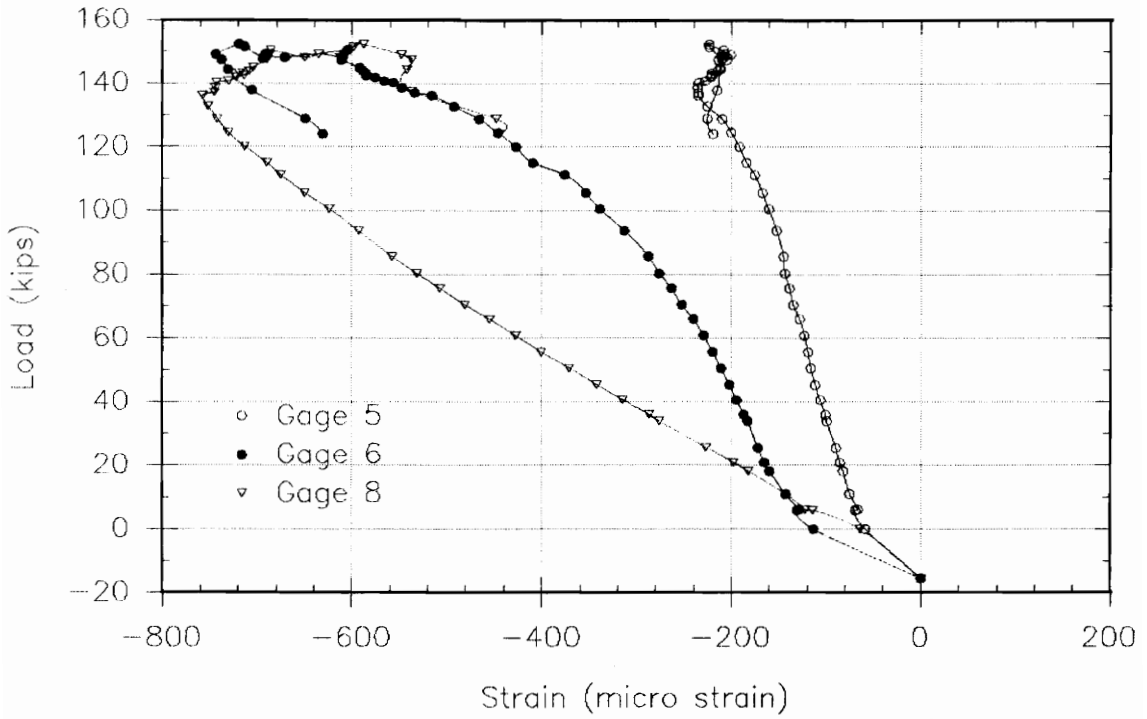


Figure B.8.11 Applied Load vs. Top Chord Strain (TC1)

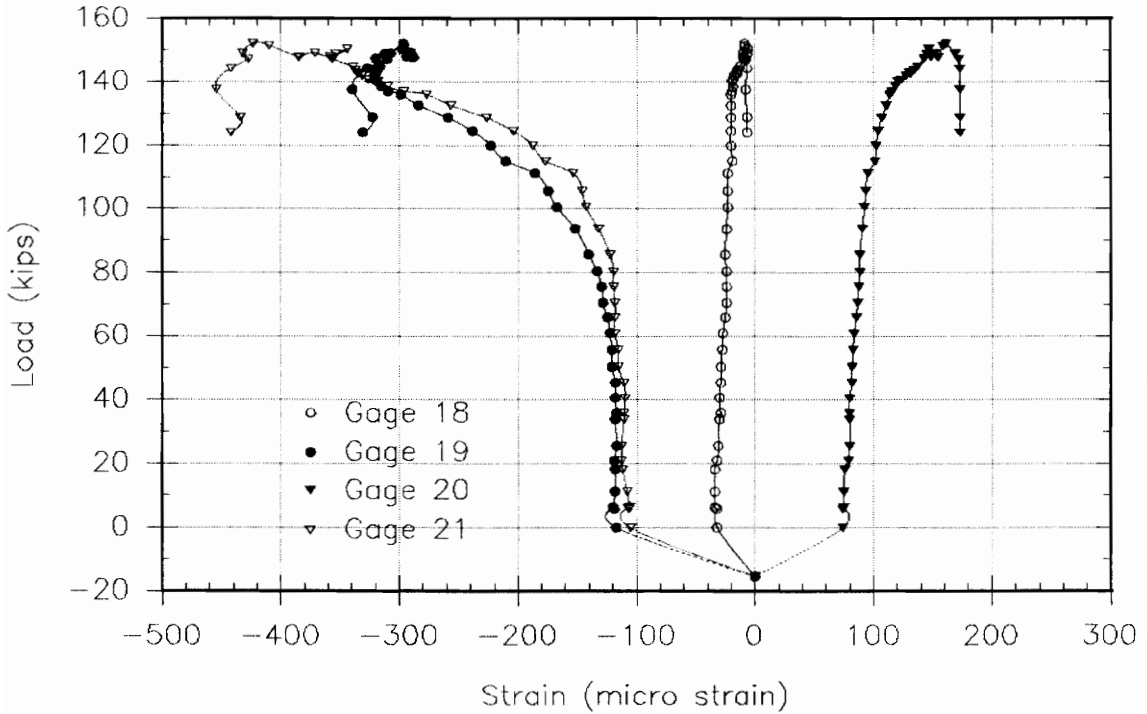


Figure B.8.12 Applied Load vs. Top Chord Strain (TC2)

CLH - 8

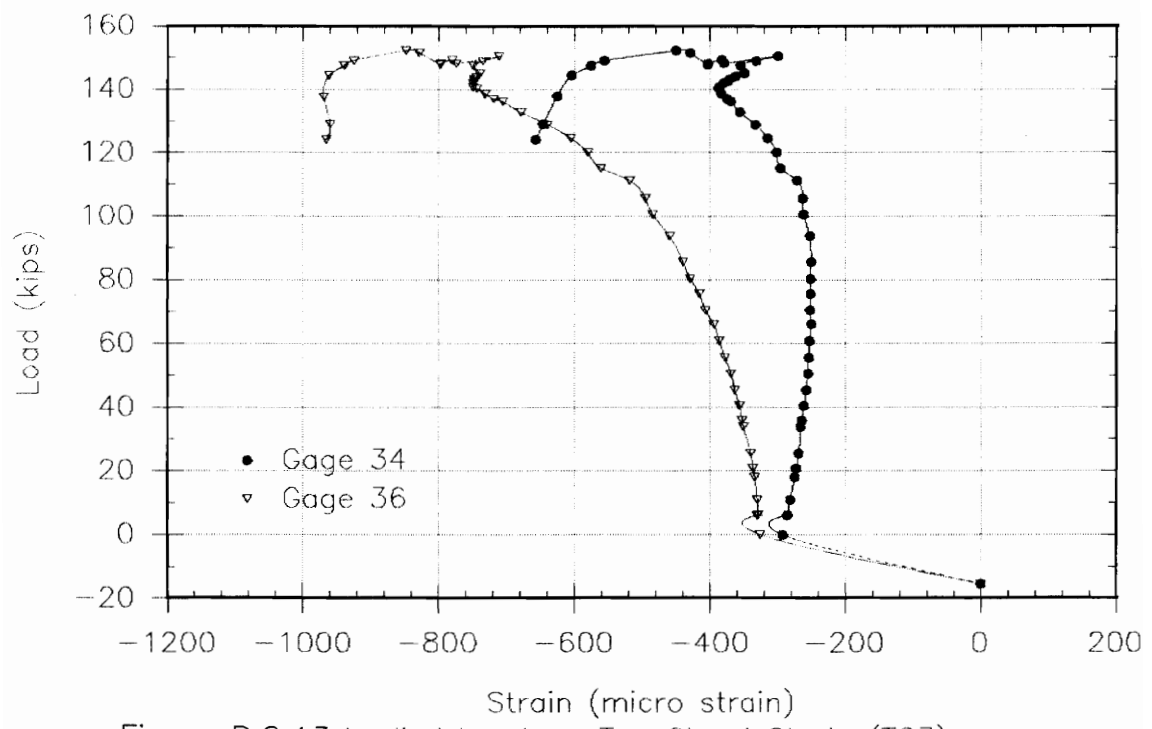


Figure B.8.13 Applied Load vs. Top Chord Strain (TC3)

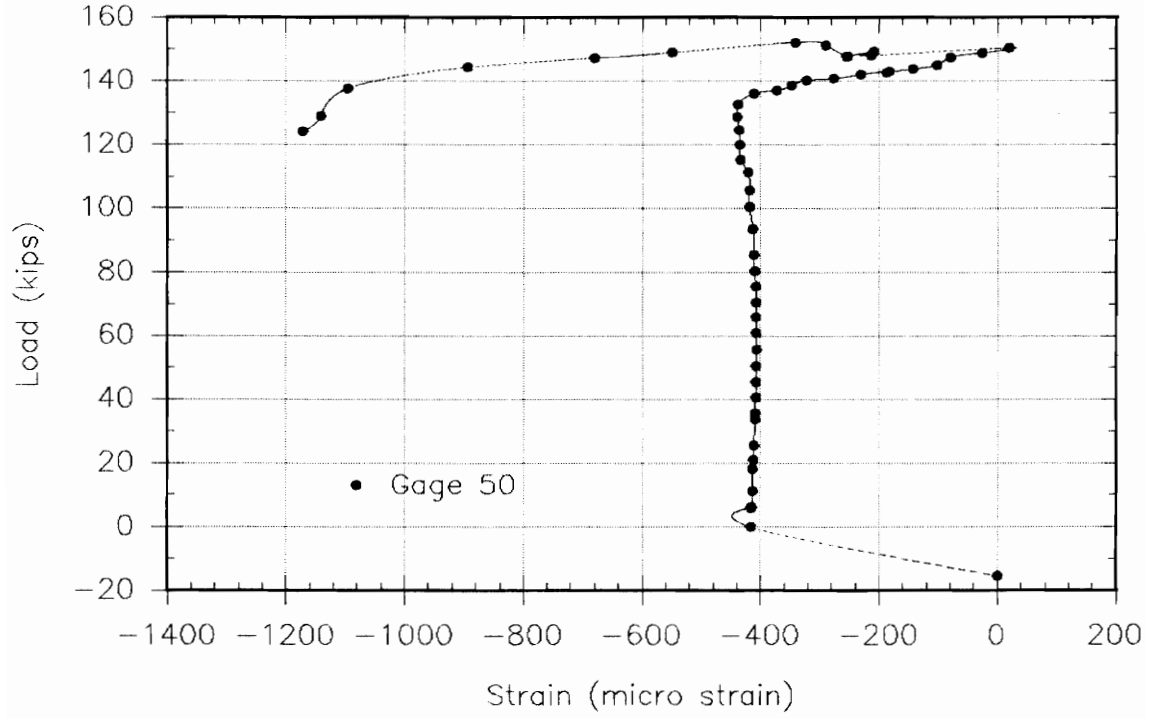


Figure B.8.14 Applied Load vs. Top Chord Strain (TC4)

CLH - 8

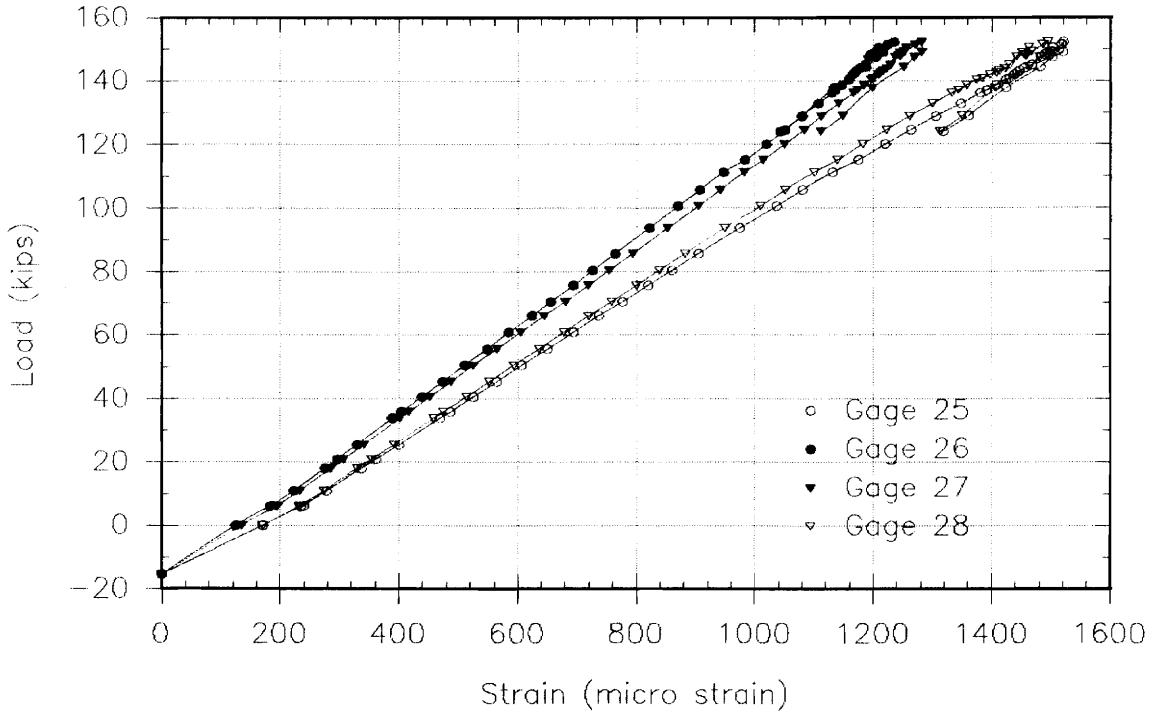


Figure B.8.15 Applied Load vs. Bottom Chord Strain (BC1)

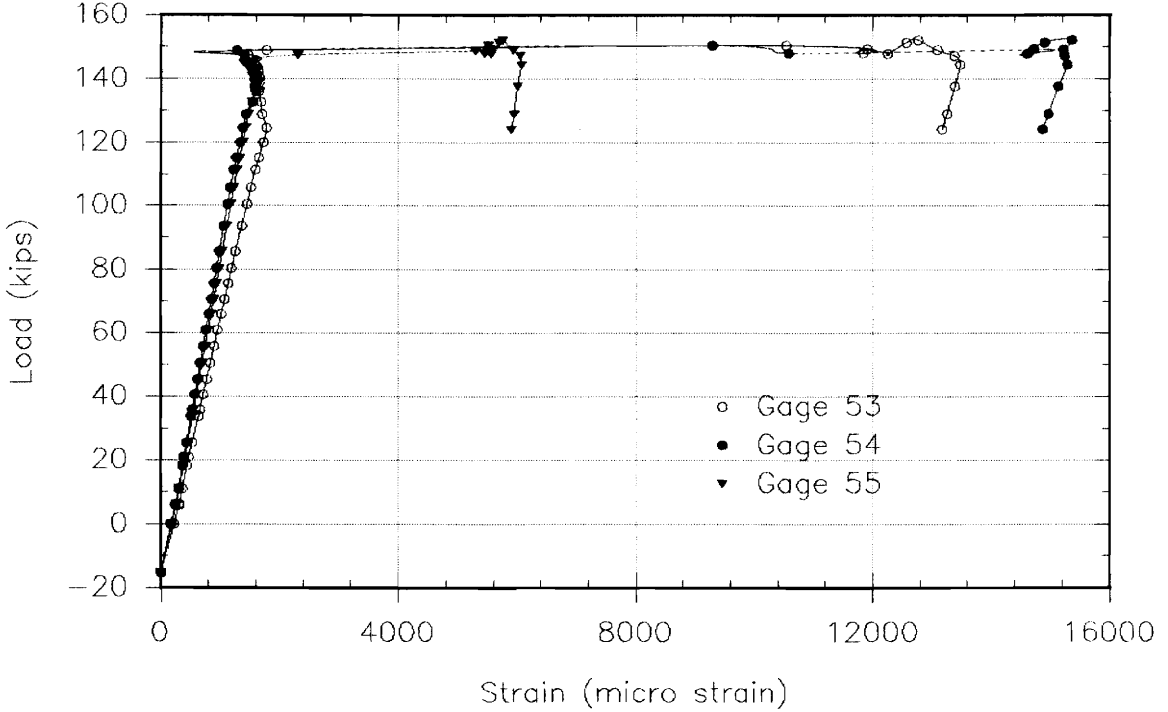


Figure B.8.16 Applied Load vs. Bottom Chord Strain (BC3)

CLH - 8

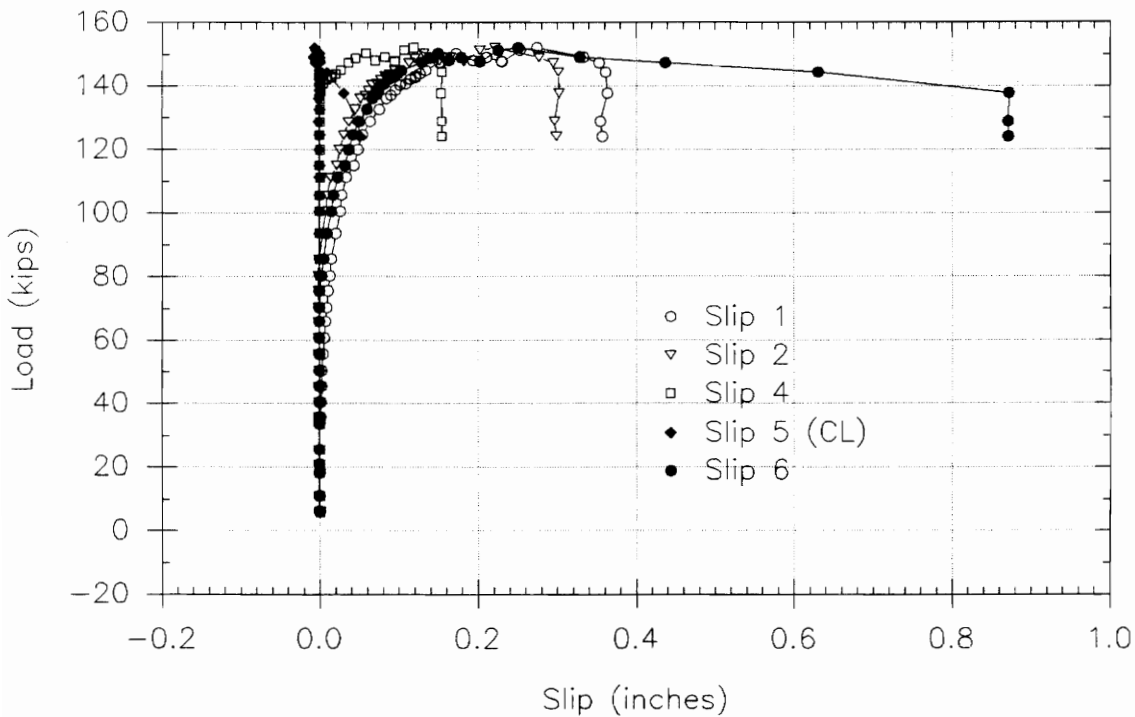


Figure B.8.17 Applied Load vs. Slip

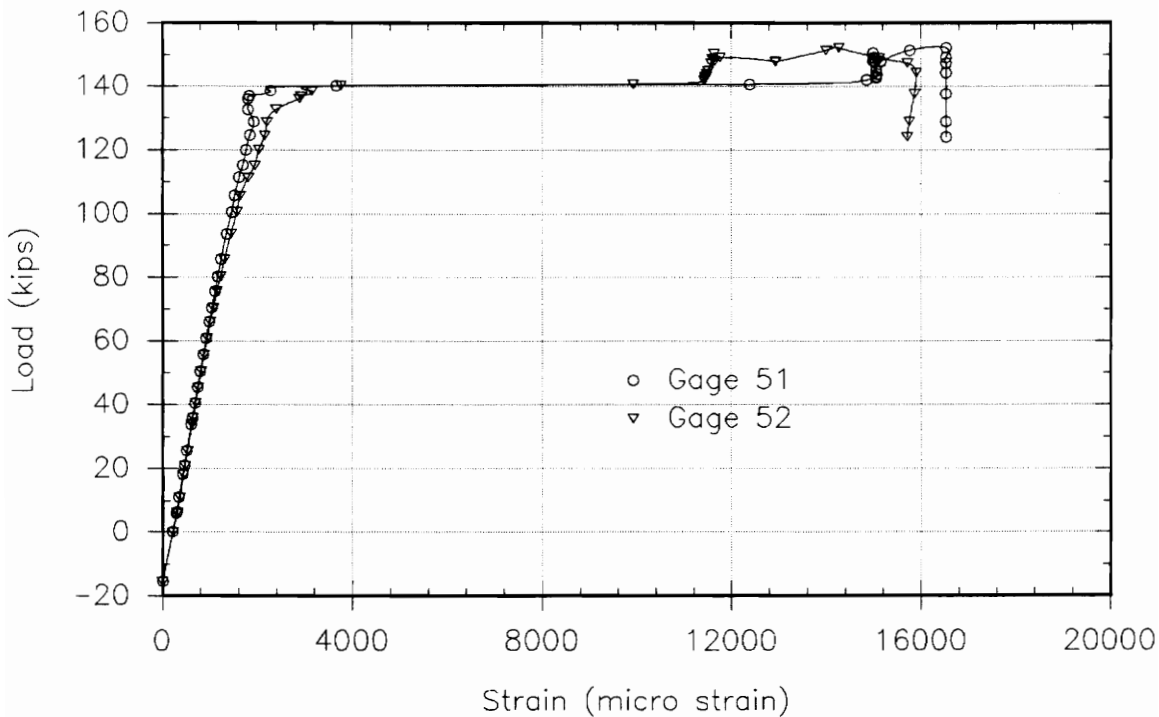


Figure B.8.18 Applied Load vs. Bottom Chord Strain (BC2)

CLH - 8

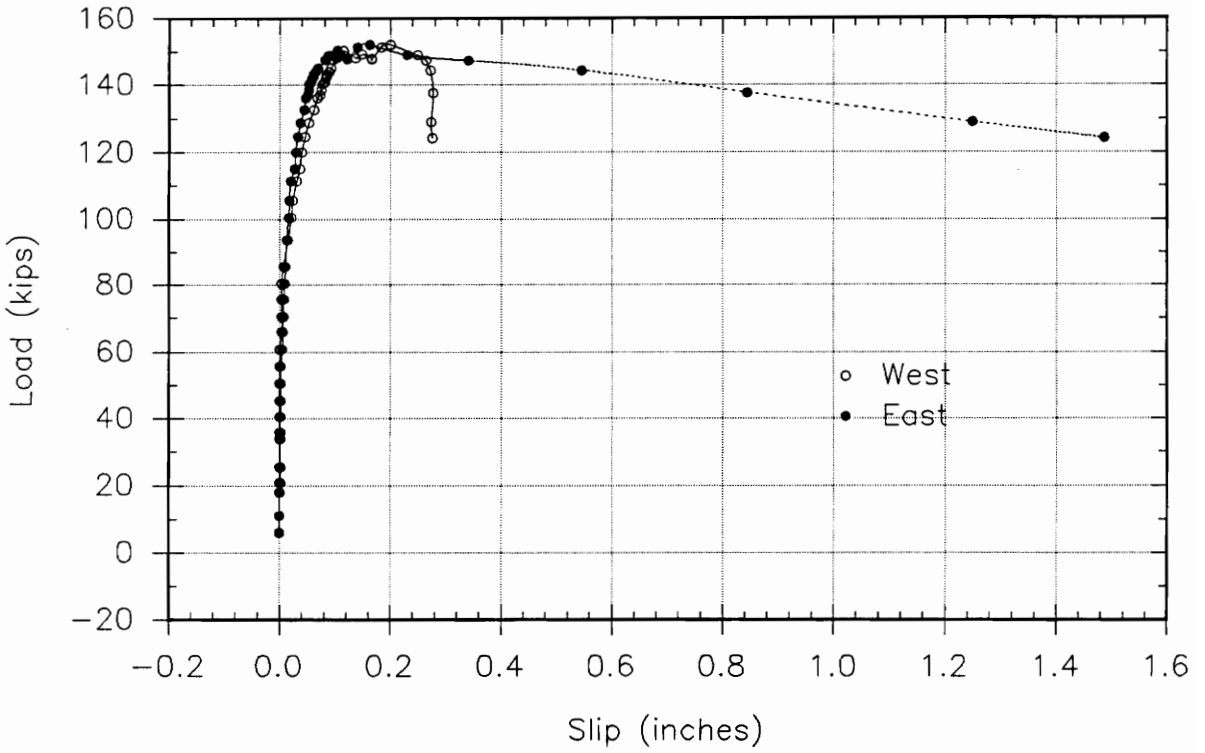


Figure B.8.19 Applied Load vs. End Slip

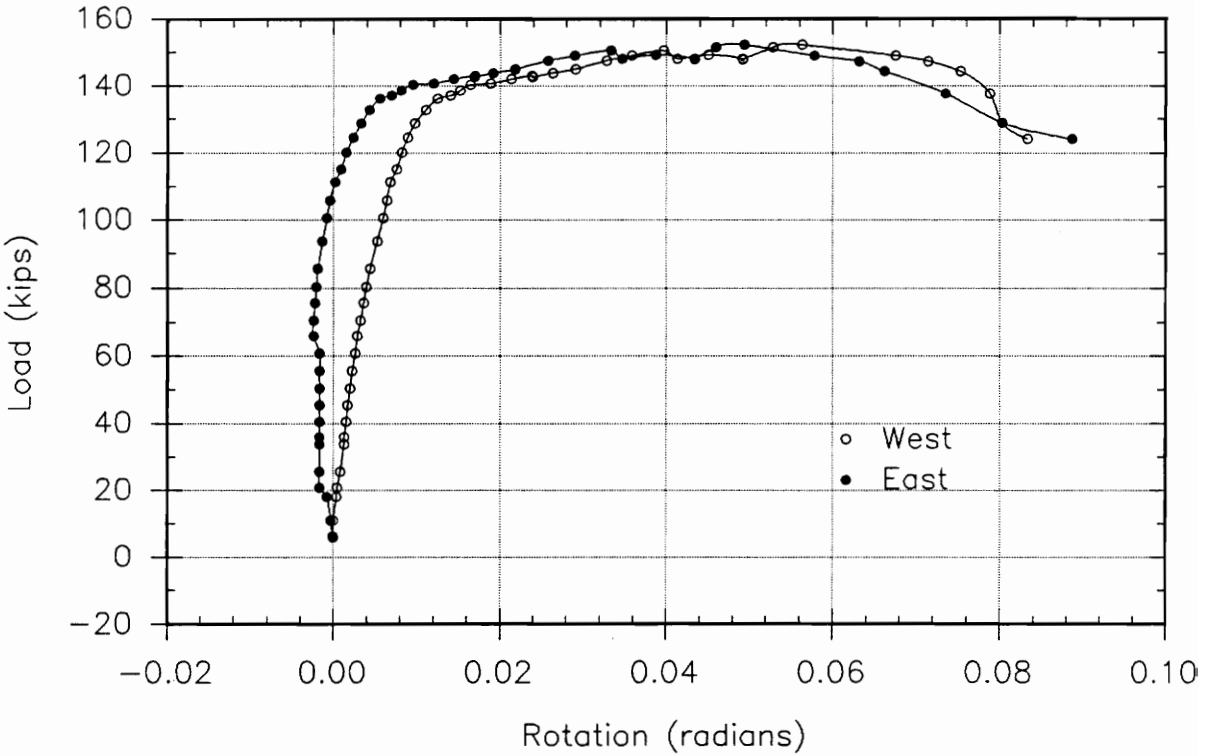


Figure B.8.20 Applied Load vs. End Rotation

VITA

David R. Gibbings was born on August 4, 1966 in Richmond, Virginia. He graduated from Kempsville High School in Virginia Beach, Virginia in June of 1984. He received the Bachelor of Science degree in Civil Engineering from the Virginia Military Institute in May, 1988. He enrolled in the graduate program at Virginia Tech in the fall of 1988.

David R. Gibbings