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MISSOURI COOPERATIVE HIGHWAY RESEARCH PROGRAM

REPORT **69-3**

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"STATIC BEHAVIOR IN THE NEGATIVE-MOMENT REGION OF COMPOSITE BEAMS USING BOLTS AS SHEAR CONNECTORS"

MISSOURI STATE HIGHWAY DEPARTMENT
UNIVERSITY OF MISSOURI - COLUMBIA
BUREAU OF PUBLIC ROADS

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Prepared for

MISSOURI STATE HIGHWAY DEPARTMENT

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in cooperation with

U.S. DEPARTMENT OF TRANSPORATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

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PB 193 106

ABSTRACT

This report describes a further investigation of the use of high strength bolts for a more rigid connection between a concrete slab and a steel beam. This investigation covers the behavior of the negative moment region of continuous - composite members loaded to failure. Each of the three members tested consisted of a simply - supported span continuing into a cantilever. Two of the members were designed according to AASHO specifications with only one of the two members having 3/4 inch - bolted connectors over its negative moment region. The third member was designed in order to yield the steel beam completely in compression.

Load-deflection curves, strain profiles, and slip distribution curves were used in investigating the composite behavior of the members. Predicted loads were compared with the actual loads obtained. The members with and without connectors in the negative moment regions were compared. Only 67% of the steel beam of the third composite member was successfully yielded in compression, but the predicted ultimate capacity of this member was exceeded.

Six tension - pushout specimens were tested. Load-slip curves were plotted and compared with those obtained from the composite members.

The following conclusions were made from this investigation:

1. Because of the nature of the load-slip per bolt data for the composite members, a conclusion for the validity of using tension pushouts to predict connector behavior in a member can not be justified.
2. For those members with connectors over their negative moment regions, there was practically no slip at their working loads, and very little slip (less than .01 inches) even at their yielding loads.
3. Elementary beam theory (assuming complete composite action) can be used to satisfactorily predict the behavior at the working loads in the negative moment region of members with connectors over the negative moment regions.
4. Simple ultimate strength theory can be used to satisfactorily predict the ultimate capacity in the negative moment region of members with connectors over the negative moment regions.
5. The ultimate moment capacity of the member without connectors over the negative moment region exceeded the predicted ultimate moment capacity of the member with connectors over the negative moment region.
6. The elimination of connectors over the negative moment region of the composite member reduced the working and first yielding loads of that member by approximately 15%.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

1.1.1 Descriptive Reinforced concrete and structural steel can be commonly utilized in the construction of bridges (1)*. One type of construction consists of the two acting independently of each other. A reinforced concrete slab constitutes the roadway and distributes the loads in the transverse direction of the roadway to the girders. These steel girders then carry the entire load along the longitudinal axis of the bridge.

In the second type of construction the two components are connected. They are, therefore, dependent upon each other. They deform as a unit, thereby producing a stiffer member acting like a "T" section. The total load is necessarily carried more efficiently along the bridge axis.

The extent of the composite action obtained depends on the ability of connectors to carry the shear force between the two components. Ideally, the composite section acts as a unit which tolerates no slip between the slab and the steel beam.

1.1.2 Background Important economic advantages can be realized through composite construction (1) (2). It results in the saving of materials since smaller members can be justified. This in turn generates

---

* Numbers in parenthesis refer to entries in the entire bibliography.

further economy by extending the length of the span for which rolled shapes may be used.

Presently, "flexible" connectors are being used. These come in the standard shapes of studs, channels, or spirals. As "flexible" implies, with these connectors a relatively small shear force induces slip between the slab and the beam. The current AASHO specifications (3) recommends that this type of composite design shall be based on an elastic beam theory. Also, to insure a reasonable rigidity in the connection the code recommends a high factor of safety on the connectors. Apparently a more rigid connection is the key in unlocking the full economy of this composite section.

1.1.3 Previous Studies Presently at the University of Missouri, there is an investigation of the use of high-strength bolts for a more rigid connection. The bolts with a washer attached to the head are positioned in holes that have been pre-drilled in the top flange of the beam. The slab is cast and the bolts tightened after the slab has cured.

In the first phase of this research, twelve pushouts were investigated (2). The testing covered 1/2, 5/8, and 3/4 inch-high-strength bolts used as shear connectors. The conclusions drawn from this investigation were:

1. High strength pretensioned bolts exhibit practically zero slip in the working range of the load.
2. Bolts attain a critical load or useful capacity that is about twice that of studs. The ultimate load is also greater for bolts.

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1. High-strength pretensioned bolts exhibit practically zero slip in the working range of the load.
2. Bolts attain a critical load or useful capacity that is about twice that of studs. The ultimate load is also greater for bolts.

3. There is apparently no loss in prestress (bolt pretension) with time in high quality concrete.
4. More research is needed regarding the spacing and edge clearance requirements in haunch members due to the splitting observed in the tests of the "prestressed specimens".

In the second phase, the investigation moved to the testing of six composite beams (4). The members were simply supported and loaded to produce compression in the top and tension in the bottom of the member. The conclusions made from this phase were:

"1. The load-slip characteristics determined from the tests of the full-scale beams using one-half-and five-eighths-inch-bolt shear connectors compare very favorably with the load-slip characteristics of the companion pushout tests.
2. The ultimate strength of three-fourths-inch-bolt shear connectors is greater than that found from pushout tests.
3. The elementary beam theory, assuming complete interaction (no slip) between slab and stringer, accurately predicted the deflection and stresses of the composite beams at service load, even though the shear connectors were designed with little or no factor of safety against slip.
4. The modified ultimate strength theory (9)* accurately

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* This reference in this report is (12).

Six pushout specimens and three composite beams were tested. The pushouts were designed so that they would produce tensile forces in the concrete. Two of the beams were proportioned according to AASHO specifications. The only difference was that one did not have connectors over the negative moment region. The third beam was proportioned to obtain complete yielding of the steel beam.

1.3. NOTATION

Each composite member was given an alpha-numerical designation. The first character, N, denotes normal-weight concrete. The next two characters, CB, indicate a continuous beam. The fourth character, 6, signifies the diameter of the connector in eighths of an inch and the fifth, B, the high-strength bolts used. The final character was used in numbering this series of specimens.

As for the beams, the pushouts were also given alpha-numerical designations. The first, N, again implies normal-weight concrete. The first numerical character, 6, designates the diameter of the connector in eighths of an inch. The T following, differentiates these as tension pushouts. The next numerical character, 4, describes the number of connectors used in each pushout. The following two characters, TB, designates that the connectors used were high-strength bolts. The next character was used to correlate the pushout to its companion composite beam. The final lower case character distinguishes the different pushouts for any one beam specimen.

CHAPTER II

TEST SPECIMENS

2.1 DESIGN OF COMPOSITE BEAMS

2.1.1 Span All three of the test beams were 32 feet in length. Each test beam was an overhanging beam with a 24-foot simple span and a 7-foot cantilever. (See figures 5,6,7). This type of beam was chosen instead of a two-span continuous beam because of the lack of space to accommodate two 24-foot spans, as used previously (4).

2.1.2 Steel beam The steel beams used were 21 WF 55. The beams arrived in 36 foot lengths, but 4 foot sections were removed. The physical cross sectional measurements varied from those listed in the A.I.S.C. manual (5). The properties used for all calculations have been listed. (See Table I).

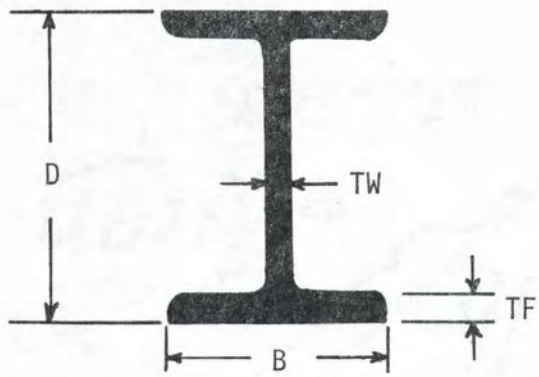
2.1.3 Reinforced concrete slab The concrete slab was 48 inches wide and 6 inches in depth. The length of the slab included the total length of the member. (See figure 1).

The longitudinal reinforcement of beams NCB6B1 and NCB6B2 was proportioned according to AASHO specifications. That of beam NCB6B3 was proportioned so that the reinforcement would yield in tension and the steel beam in compression at the ultimate load.

The transverse steel was designed to simulate that of an actual bridge slab. The steel was placed at a spacing of 6 inches. A continuous bar extended from the top, down one side, along the bottom, up the other side, and closed upon itself.

Table I

Actual Beam Dimensions



	Beam A.I.S.C.	Beam NCB6B1	Beam NCB6B2	Beam NCB6B3
TW (in.)	0.375	0.388	0.389	0.379
TF (in.)	0.522	0.491	0.480	0.488
D (in.)	20.800	20.875	20.802	20.875
B (in.)	8.215	8.359	8.297	8.375
I ($in.^4$)	1140.7	1107	1075	1098

Note: All of the measurements listed for the test beams are average quantities.

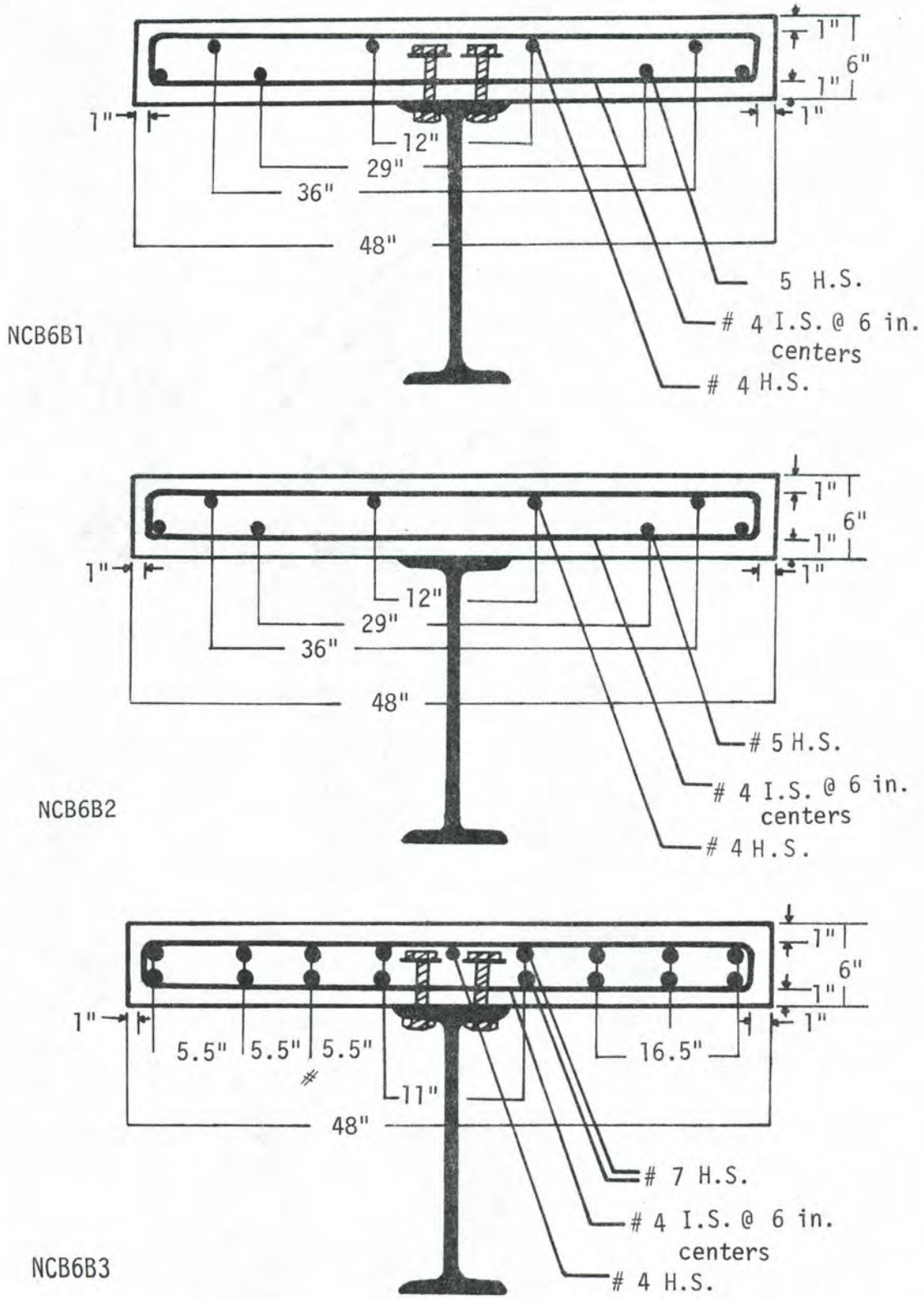


Figure 1 Cross Sections at the Negative Supports

2.1.4 Shear connectors The bolts used for connectors were 3/4 inch-high-strength bolts. Standard washers were spot welded to the under side of the head of the bolts. The bolts were placed in the holes so that none of the thread was exposed to the concrete. This exposure to the concrete could have caused a stress concentration at one of the threads.

The spacing of the connectors along beam NCB6B1 was according to AASHO. The capacity for a connector - used in conjunction with the AASHO specifications - was obtained from the results of tension pushout tests (7). The spacing for the connectors of NCB6B2 was identical to NCB6B1 for the positive moment region. The connectors were eliminated over the negative moment region of NCB6B2. (See figures 5, 6). The spacing of the bolts along beam NCB6B3 was determined by the tension pushouts using a factor of safety of about 2. For design purposes the spacing of the connectors was identical to NCB6B1. (See figure 7).

2.2 FABRICATION OF TEST BEAMS

2.2.1 Materials The steel beam met the requirements of ASTM A36-66.

The ready-mix concrete used consisted of normal-weight aggregates and type I portland cement. The fine aggregate was Missouri River sand. The course aggregate was crushed limestone with a maximum size of 3/4 inches. Typical mix proportions by weight were 1:1.85:3.5. The water-cement ratio was 5.0 gallons per sack. Air entrainment was added to the mix. This mix conformed with the class B-1 mix used by the State Highway Commission of Missouri for bridge construction.

High-strength-steel reinforcement with a yield of 60,000 psi was used in the longitudinal direction. All the bars satisfied the require-

ments of ASTM A32-62T. In the transverse direction, intermediate-strength-steel bars conforming to ASTM A15-62T were used.

The high-strength bolts met the specifications of ASTM A325-66. The washers were ordinary flat head washers.

2.2.2 Preparation for casting The holes in the top flange of the steel beam were drilled with a portable-magnet base drill. The bolts were then set and hand tightened against wire chairs. The beam was then stabilized.

Unshored-reusable forms were used for all three castings. The forms were wired to prevent separation of the forms from the steel beam. (See figure 2). The forms were supported by wedging two-by-fours between the bottom of the form to the inside of the bottom flange of the steel beam. This was done at approximately every two feet (See figure 3).

The top longitudinal steel was spaced and positioned above the forms. The transverse steel had been prefabricated into a stirrup arrangement. These were then hung on to the main reinforcement into their specified positions and welded. The remainder of the longitudinal steel was then placed into the bottom of the stirrups and welded. Completed, the reinforcement had formed a moveable cage. (See figure 4).

Before pouring, the forms were cleaned and coated with a form oil. The cage was set into the forms on one-inch-wire chairs. The sides of the cage were then wired to the forms in order to prevent any possible movement during casting.

2.2.3 Casting Upon arrival, the concrete was subjected to a slump test. When accepted, the material was transported from the ready-mix vehicle by a bucket mounted on a fork lift. Then it was poured, vibrated with a tube vibrator, leveled, and troweled. A final string-

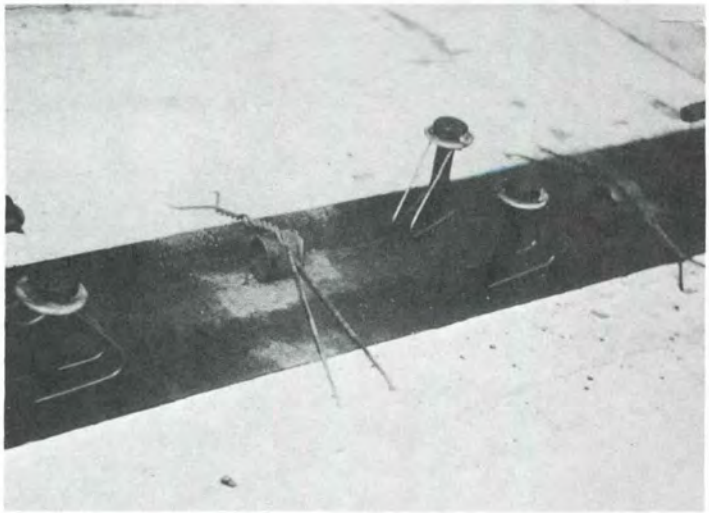


Figure 2 Wired Forms

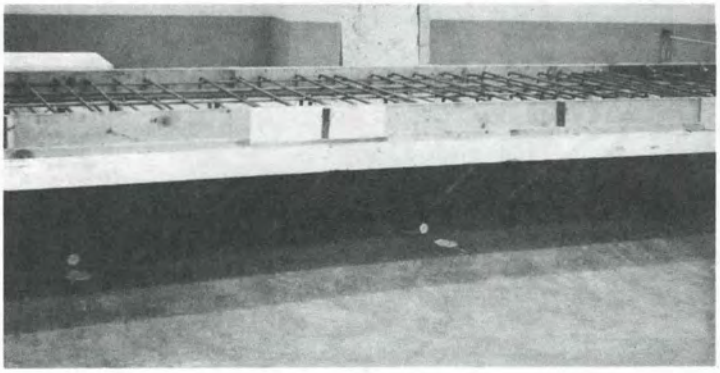


Figure 3 Form Support

Figure 4 Reinforcement Cage

line check was run.

2.2.4 Curing After the initial set, the slab was covered with polyethylene sheets. The next day, straw was spread over the slab. This was wetted and covered with the sheets. This curing proceeded for 28 days. The forms were not stripped until the end of the curing period.

2.2.5 Coring To facilitate the loading apparatus, it was necessary to core four sections from the slab. The 3 inch cores were extracted by a diamond-core bit mounted on a portable drill.

2.3 INSTRUMENTATION

2.3.1 Load indication The loading was accomplished by four-30 ton-hydraulic jacks. They were arranged two per load point. The jacks were connected to a pumping system containing a pressure cell. This cell contained resistance-strain gages to which a strain indicator was attached. For calibration the jacks were placed into a 100,000 lb. - mechanical-testing machine. A load increment was set on the testing machine. The jack(s) was then pumped until it balanced the pre-set load. When balanced, the strain indicator was read and recorded. In testing, the strain indicator was set and the jack was applied until the indicator balanced at the corresponding load. Three complete sets of calibrations were taken.

Since the capacity of the testing machine was only 100,000 lbs. the jacks were calibrated separately for their complete range (60,000 lbs). This was necessary in order to determine if the characteristic of the jack changed with increased load.

Two jacks were used at each load point on the beam. The mated jacks were commonly connected and then calibrated to the capacity of the machine. The history of the loading of both sets proved identical; therefore, equal loading at each load point during the experimentation was assured.

To insure equal and constant loading during the testing of the beams, the four jacks were to be connected to one pump. The four jacks were, therefore, calibrated together up to the capacity of the testing machine. The limited results were exact to those obtained by loading the jacks in pairs.

2.3.2 Deflection The deflection of the beam was measured mid way between the supports and under the load point of the cantilever portion. (See figures 5, 6, 7). A dial graduated at .0001 inch was placed at each location. An additional 4 inch dial graduated at .001 inch was placed at the cantilever location since the deflection would exceed the travel of the smaller dial.

The dials were mounted on magnetic bases, which were then attached to a beam stub. The stub had been previously grouted to the test floor.

2.3.3 Slip The slip between the slab and the steel beam was measured by dials graduated to either .0001 or .001 inch. The dials were bolted to brackets, which were then cemented to the inner face of the beam flange with a contact cement. Small stops fabricated from an aluminum angle were then cemented to the slab. An expoxy filler was then placed around both the bracket and the stop.

Slip dials were placed at both ends of the beams. They were also attached at the shear connectors along predetermined locations of the

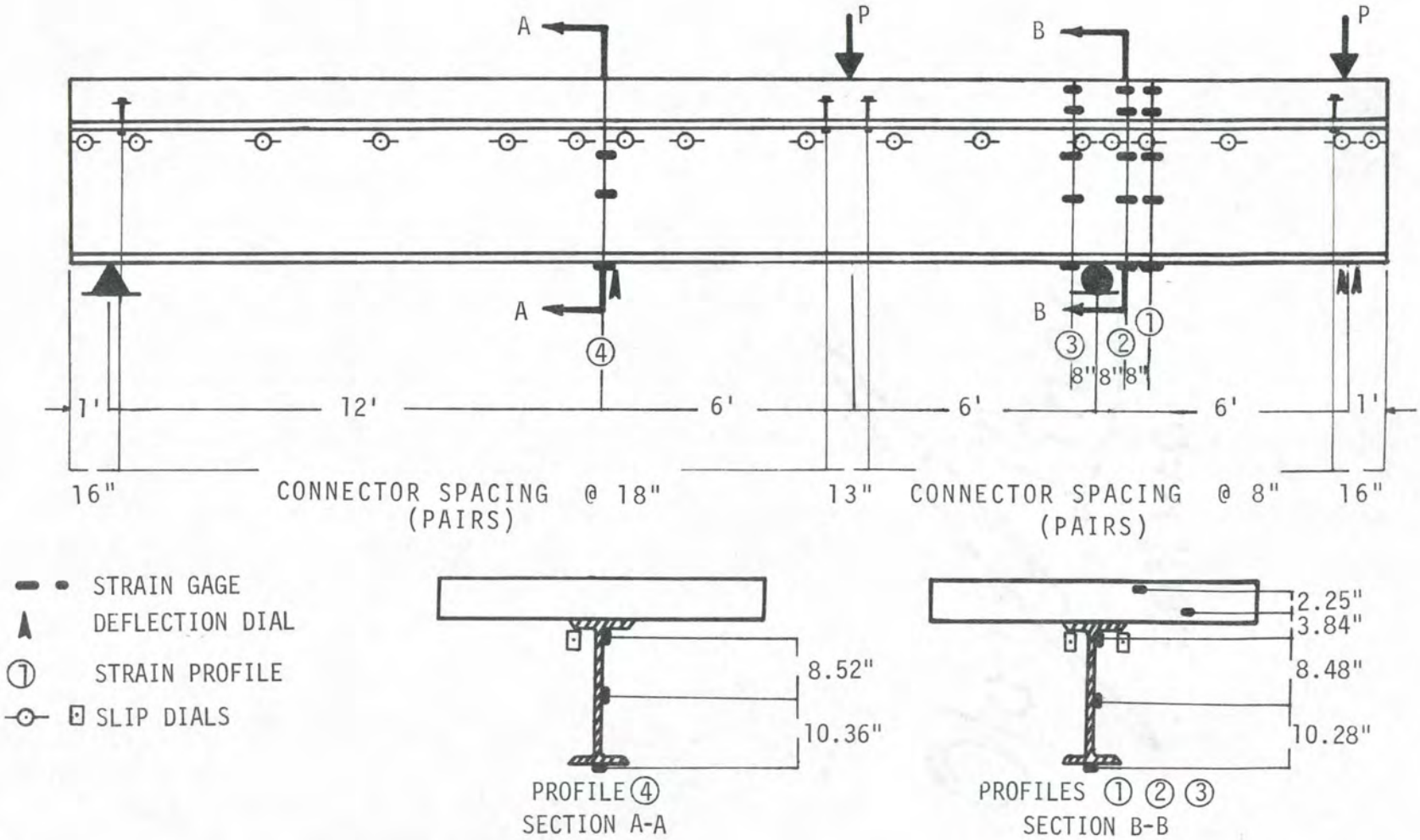


Figure 5 Instrumentation - NCB6B1

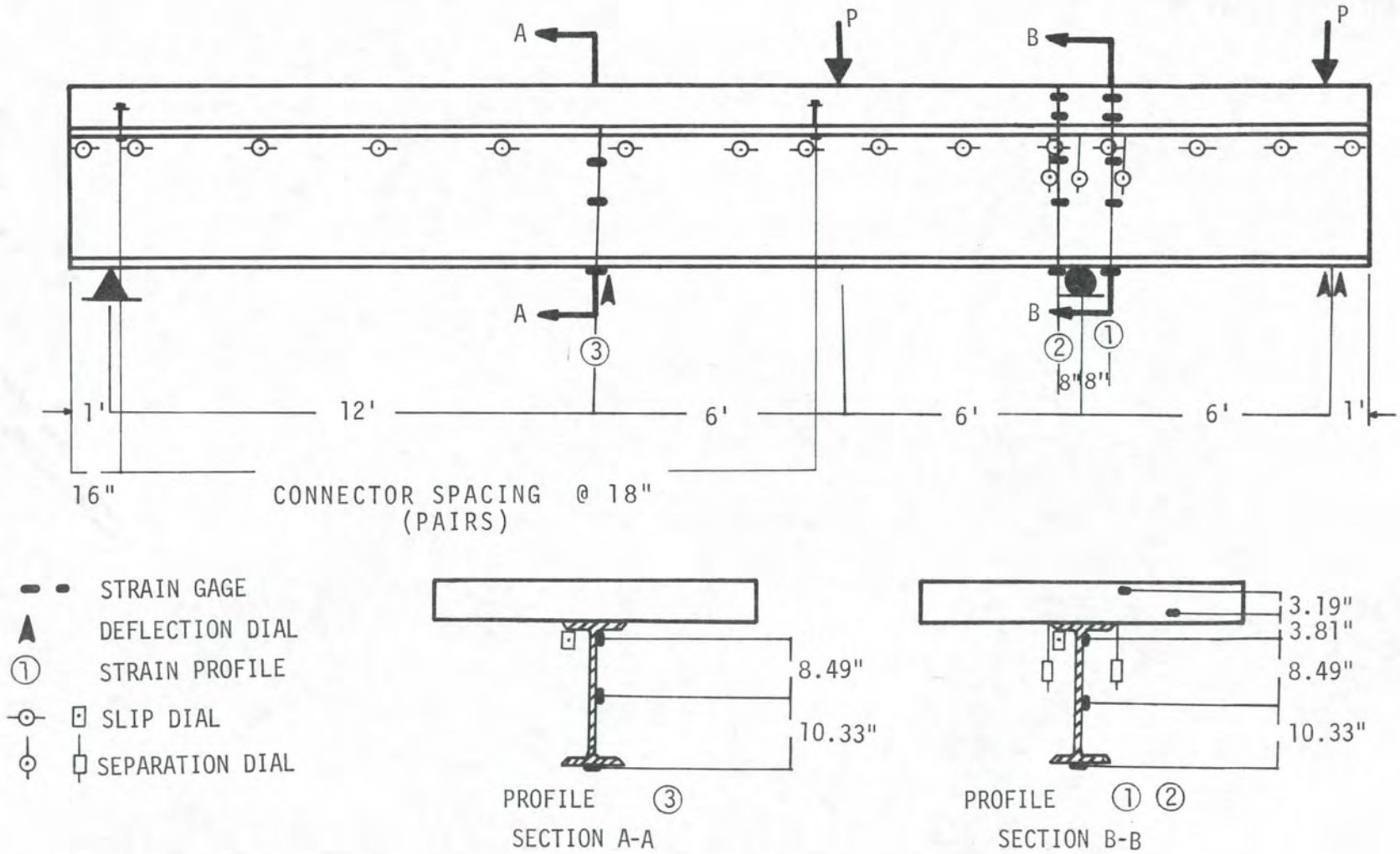


Figure 6 Instrumentation - NCB6B2

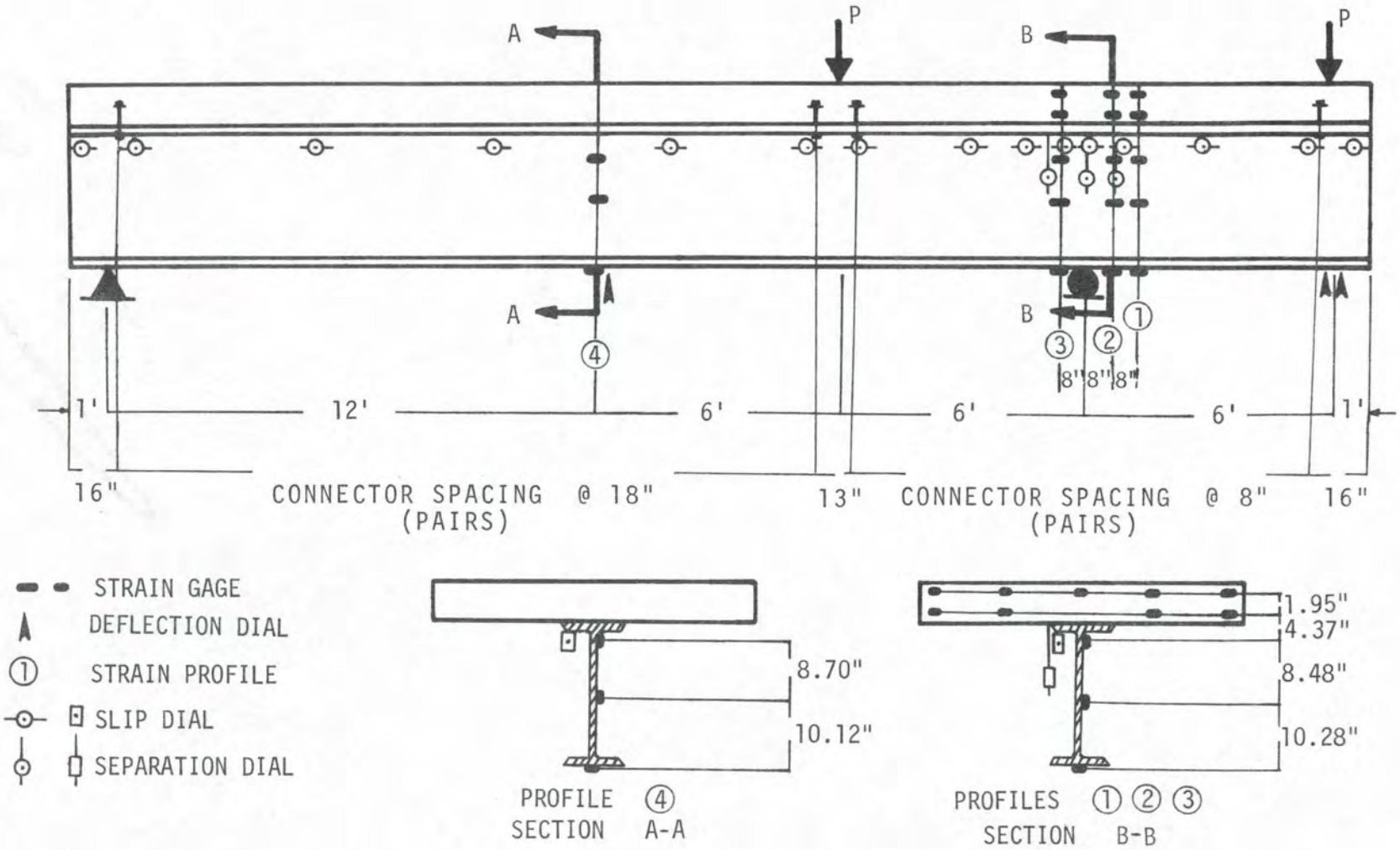


Figure 7 Instrumentation - NCB6B3

member. The only exception was in NCB6B2, where, because of the absence of connectors in the negative moment region, the dials were placed at selected distances. (See figures 5, 6, 7).

2.3.4 Steel strain, steel beam Strain in the steel beams was measured by Baldwin strain gages. The type used was classified as a wire, paper base, SR-4 gage. The gages were installed according to the Baldwin manual (Bulletin 279-B) prior to the fabrication of the slab forms. After the surface was prepared, the gage was attached with Nitrocellulose cement and dried accordingly. After 24 hours, the gage was uncovered, checked, wired, and coated with a rubber-base-waterproofing compound.

The wiring completed, the steel beam was mounted and loaded to half the theoretical yield. The resulting strain profiles were checked to determine whether the gages were functioning properly.

A strain profile consisted of three gages. The locations of the profiles along the steel beam were the same for NCB6B1 and NCB6B3. One of the profiles was eliminated for NCB6B2. (See figures 5, 6, 7).

2.3.5 Steel strain, reinforcing bars The strain in the reinforcing steel was measured by strain gages. The strain in the slab was determined by assuming perfect bond between the bars and the concrete.

The gages were attached before the steel was placed into the forms. The area that was to receive a gage was first ground to remove all bar deformations. These deformations would have induced stress concentration. The area was then sanded to the required finish. As in the steel beam, SR-4 gages were used. These gages were installed according to the manual, except, that the pressure on the gages was applied by taping

them completely. After the bars were welded into position, the coverings were stripped, the gages wired, and checked. Since the gages were to be embedded in the concrete, protection had to be provided. Numerous layers of waterproofing compound were applied not only to keep the gages dry, but also to form a protective covering. Also a layer of clear epoxy was surfaced over the waterproofing.

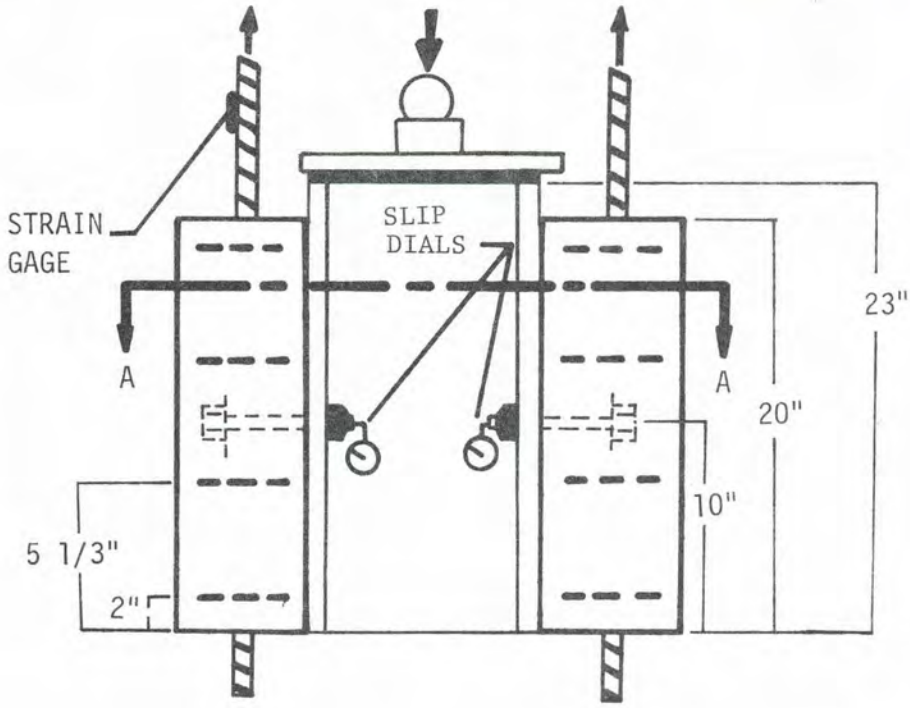
The gages were placed on selected bars at the same locations as the steel beam profiles. (See figures 5, 6, 7).

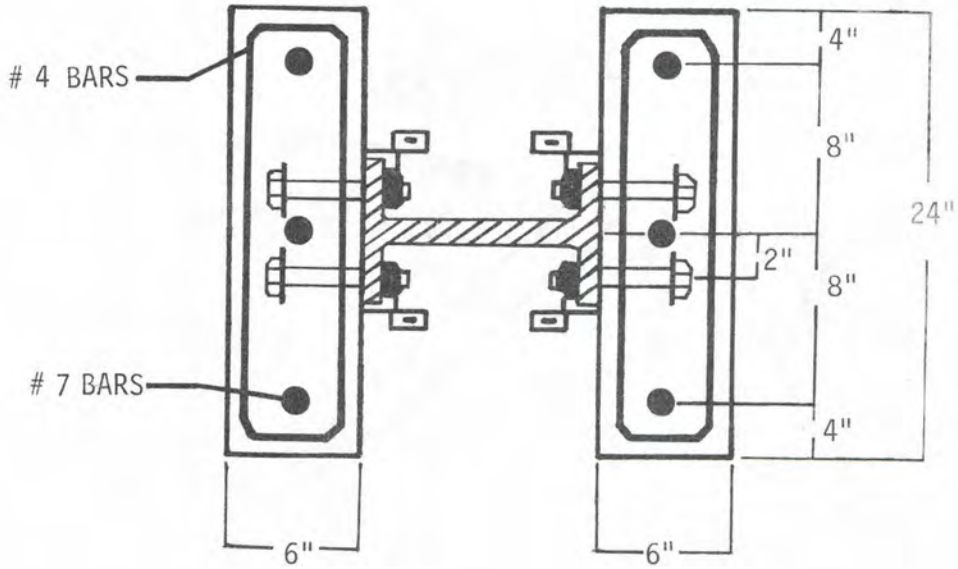
2.4 COMPANION SPECIMENS

2.4.1 Tension pushouts A total of six companion pushouts were cast, two for each composite member. (See figure 8).

The steel beam stub used was a 23 inch - 8 WF48 - section. The stub had reinforced-concrete slabs connected to each flange. The slab measured 20 x 24 x 6 inches. The main reinforcement consisted of three bars that extended beyond the slab. They were #7-high-strength-bars conforming to ASTM A32-62T. The transverse steel was composed of four-long stirrups fashioned like those in the composite beams. The connector and the concrete were of the same specifications as those used in the main members.

In the construction of the pushouts, the beam stub had to be first cleaned of any mill scale and then have two holes drilled in each flange. The holes were drilled 10 inches from the selected bottom of the stub. The bolts were placed as in the main beam members. The beam was then placed in the reusable-slab forms. The main reinforcement was cut into the necessary lengths in compliance with the testing apparatus. The main bars were fixed in the forms and the secondary





SECTION A-A

Figure 8 Details of Pushout

bars were welded to them. The pouring and curing of the pushouts were performed in conjunction with their companion members.

Instrumentation of the pushouts required slip dials and strain gages. A slip dial was placed at each of the four shear connectors according to the procedures described for the beams. To determine the distribution of forces in the pushout, SR-4 strain gages were installed at the three main bars of one of the two slabs. Installation of the strain gages was according to the procedures described for the bars in the composite beam, except that the extra layers of waterproofing compound and the epoxy were not necessary.

2.4.2 Concrete cylinders Six concrete cylinders were cast with each pour. The cylinders were cast in pairs spaced at the beginning, middle, and end of the pouring of the slab. They were made according to the provisions of ASTM C192-66.

2.4.3 Tensile coupons Before fabrication each of the three steel beams had a 4-foot section removed. Seven coupons were machined from these sections. Two were taken from the top flange, three from the web, and two from the bottom flange. The coupons were standard-rectangular coupons with an 8 inch-gage length. They conformed to the specifications of ASTM A370-67.

2.4.4 Reinforcing bar sections Nine sections were cut from the high-strength-reinforcing steel used. They included three sections from each of the different sizes used (#4, #5, and #7). The sections were 18 inches long.

CHAPTER III

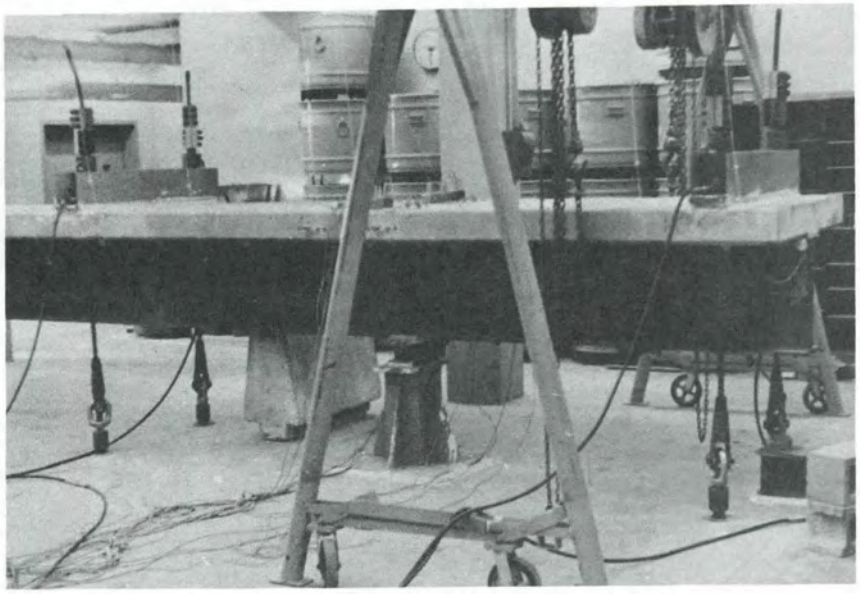
TEST PROCEDURES

3.1 COMPOSITE BEAMS

3.1.1 Preparation of composite beams In the preparation of the composite members for testing, the connectors first had to be tensioned. They were pretensioned according to the "turn-of-nut" method (6). The bolts were first hand tightened with a 12-inch-crescent wrench. They were then tensioned an additional half-turn using a larger wrench with an extension.

The positioning of the member then followed. The steel piers were grouted to the test floor. The rollers were attached to the piers. A fixed roller was placed at the positive support, and a free roller at the negative support. The test member was then lowered onto bearing plates placed on the top of the supports. These plates were grouted to the beam, but they rested freely on the rollers. Because of a cable loading system, alignment of the member with respect to the test floor was critical in eliminating possible excentricity in loading.

3.1.2 Preparation of loading system Transverse distributor beams were grouted to the test slab at its load points. (See figure 9). Slots had been cut in these beams to coincide with the cores in the test slab. Four 30-ton jacks were used to load the test specimens. The jacks had hollow shafts through them. Each jack was positioned in the distributor beam so that the shaft coincided with the slot in the beam. High-strength cables were connected to the test floor. Each cable was then threaded through the slab core, beam slot, and jack shaft immediately above it. Two clamps were placed on each



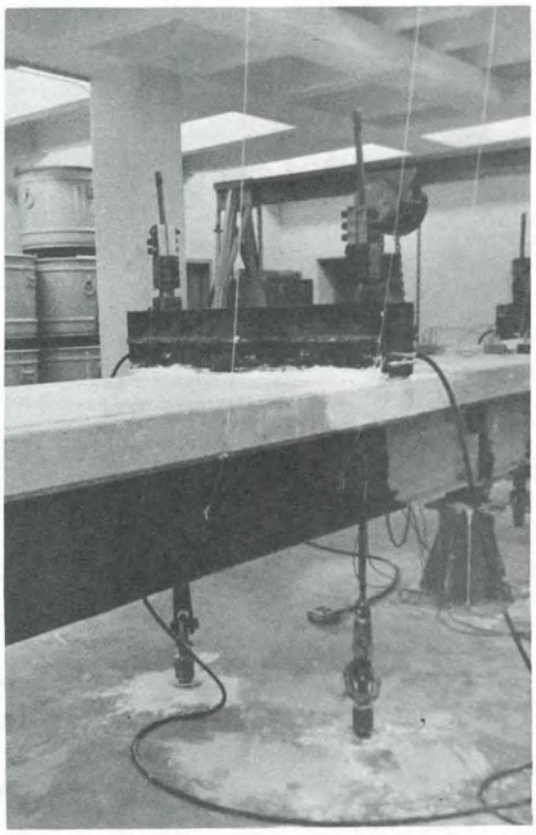


Figure 9 Beam Loading System

cable against the jack. The cable was pulled taut and the clamps were fixed against the jack. Hydraulic hoses were then connected from each jack to a common pump.

As previously described, a strain indicator was connected to the pump. The load was applied by balancing previously calibrated quantities on the indicator.

3.1.3 Loading of test beams The members tested were subjected to a minimum of three loading cycles. The load increments were usually five kips per load point, but finer increments were used at the critical ranges.

For each increment, the load was stabilized, while deflection, slip, and strain were recorded.

3.2 COMPANION SPECIMENS

3.2.1 Tension pushouts In previous tension pushout experiments, premature failure of the specimens was caused by a separation of the slabs from the stub and a twisting of the specimen due to the loading system (7). The new testing system, (see figure 10), consisted of a confining frame and a 100 ton-hydraulic jack.

The connectors were pretensioned by the "turn-of-nut" method after the concrete had reached its 28-day strength. The pushout was then placed in the test frame. The pushout specimen was temporarily set on four screw jacks, so that it sat above the bottom of the test frame. The bottom bars extending from the slab had been positioned so that they were confined by the bottom cross members of the frame. The top of the frame with the jack was then positioned over the top bar extensions. The specimen was aligned and the beam stub leveled with the four jacks. Steel plates were then placed over the top of the rein-

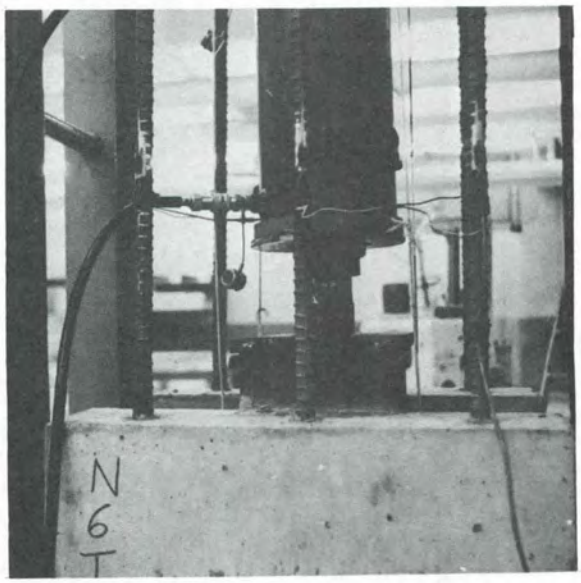


Figure 10 Pushout Loading System

forcing bars and welded. To simulate the continuity of the reinforcement in a slab, bar sections were welded from a bottom extension of one slab to the corresponding bar of the other slab. The screw jacks were removed, thereby leaving the specimen hanging from the top of the frame. The specimen was then checked for proper alignment. Lead strips were placed on the end of the beam stub, and a distributor plate with a ball attachment was placed over the lead strips.

A minimum of one loading cycle was performed on each specimen. The load increment was usually 5 kips, but a finer increment was used at the critical ranges. At each increment the load was stabilized, while slip and strain* readings were recorded.

3.2.2 Concrete cylinders The cylinders were capped with a sulfur, compound and tested in a 300,000 pound-hydraulic-testing machine.

Of the six cylinders cast with each pour, three were tested with the beam and three with the pushouts. In each set of three cylinders, one was loaded to find the compressive strength of the concrete according to ASTM C39-66. The remaining two were loaded to find the modulus of elasticity according to ASTM C469-65, and then continued until failure. All the cylinders for the beams were run within 48 hours of the companion test. Those for the pushouts were run within one week of their respective pushouts.

3.2.3 Tensile coupons The steel coupons were tested in a 60,000 pound mechanical testing machine. All data was gathered by an autographic recorder. A 2-inch recorder was used.

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* See instrumentation of companion section (section 2.4.1).

The specimen was loaded to determine its modulus of elasticity, its yield point, and its static yield point. The static yield point was determined by stopping the testing machine after the yield had been reached and letting the load reach a stable value (8). This was done three times for each coupon specimen. (See figure 11). The rate of loading recommended by ASTM was disregarded, because the rate did not represent that of the companion member. A slower rate of 0.025 inch per minute was applied.

3.2.4 Reinforcing bar specimens The testing of the bar specimens was in accordance with that of the coupons, except that the static yield was not determined.

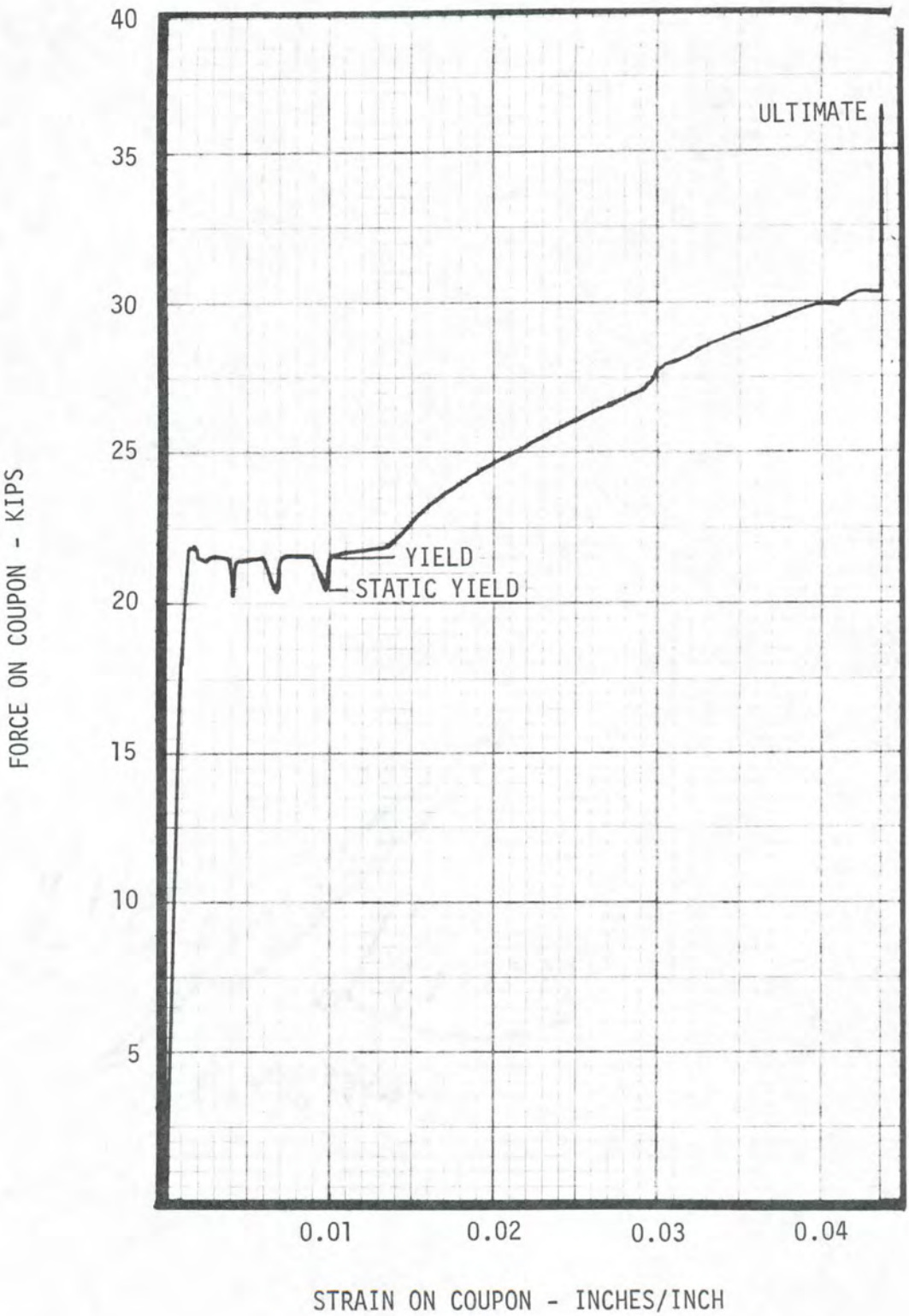

Figure 11 Autographic Chart Strip

CHAPTER IV

RESULTS AND COMPARISONS

4.1 COMPANION SPECIMEN RESULTS

4.1.1 Concrete Cylinders The results of the cylinder tests for the composite test beams and the companion pushout specimens are presented in Tables II and III, respectively. In both tables the compressive strengths and moduli of elasticity are average values.

4.1.2 Tensile Coupons The results of the coupon tests are presented in Tables IV and V. Values listed for the top flange, web, and bottom flange are the average of all respective coupon results. The final column contains the weighted average of these three components. The averages are weighted according to their respective areas. The yield strengths are entered in Table IV and the static yields in Table V.

The coupon results for the modulus of elasticity differed substantially from the expected values. This was due to the relatively large scale used by the autographic plotter. As seen in figure 11 an accurate determination of the slope is impossible. For this reason a uniform value of 29.5×10^6 psi was used for the modulus of elasticity, which is an average of coupon tests from 21 WF 55 sections used in previous research (4).

4.1.3 Reinforcing bar specimens The averaged results of the reinforcing bar specimens are listed in Table VII. This table lists the

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* See section 2.3.4

Table II

Results of Cylinder Tests

For Test Beams

Specimen	Compressive strength psi	Modulus of Elasticity psi x 10^6
NCB6B1	7313 ± 130*	4.60 ± .16
NCB6B2	7785 ± 247	4.69 ± .11
NCB6B3	5473 ± 159	4.21 ± .11

Table III

Results of Cylinder Tests

For Pushout Specimens

Specimen	Compressive strength psi	Modulus of Elasticity psi x 10^6
N6T4TB1a N6T4TB1b	7953 ± 87	4.59 ± 0
N6T4tB2a N6T4TB2b	8292 ± 79	4.99 ± .28
N6T4TB3a N6T4TB3b	6523 ± 79	4.28 ± .04

* Standard deviation

Table IV

Coupon Results

Yield Strength- ksi

Specimen	Top Flange	Web	Bottom Flange	Average
NCB6B1	38.7 ± .0*	39.4 ± 1.3	38.8 ± 1.3	39.0
NCB6B2	39.6 ± .2	41.2 ± .9	38.9 ± .7	40.2
NCB6B3	36.8 ± 1.4	37.3 ± 1.2	36.9 ± .9	37.1

Table V

Coupon Results

Static Yield Strength- ksi

Specimen	Top Flange	Web	Bottom Flange	Average
NCB6B1	36.5 ± .6	36.4 ± 1.8	37.0 ± 1.6	36.6
NCB6B2	37.5 ± .1	39.1 ± .9	37.4 ± .6	38.3
NCB6B3	34.7 ± 1.4	35.1 ± 1.2	34.8 ± 1.6	34.9

* Standard deviation

yield strength and the modulus of elasticity for the high-strength reinforcing bar sizes used.

The yield strengths of the #4 and #5 bars were determined from well defined yield points on the autographic recorder. The yield strength of the #7 bar was determined by a 1% offset.

4.1.4 Tension pushouts The ultimate loads attained in the tension pushouts far exceed previous results (7). This can be directly attributed to the improved loading system and the introduction of a means of limiting the separation of the slabs from the beam although there still existed some eccentricity in the loading system. It had been noted that the slab receiving the greater portion of the force would crack first. Approximately 5 to 10 kips later, the second slab would crack. The failure of the pushouts was caused by the failure of their concrete slabs. It was noted that the ultimate load closely followed the

Table VII

Reinforcing Bar Specimen Results

High Strength Steel

Specimen	Yield Strength ksi	Modulus of Elasticity psi x 10^6
#4 Bars	66.6 ± .7*	29.3 ± .6
#5 Bars	64.4 ± .1	27.7 ± .5
#7 Bars	67.6 ± .2	27.5 ± 2.2

* Standard deviation

separation of the slabs from the beam stub. (See figures 12, 13). There was only one exception. In one case the failure was due to the shearing of two connectors. (See figure 14).

The results of the six tension pushout tests are summarized in Table VIII. The compressive strength obtained from the cylinder tests is re-entered into this table. Next, the load and slip per bolt at the first cracking of either slab are listed. The next two entries are the last recorded load and slip per bolt before the failure of the specimen. Following next are the ultimate load per bolt and the type of failure of each specimen.

Before proceeding further, an explanation of the calculation of the load and slip per bolt quantities used in this report is in order. Because of the eccentricity in the loading of the pushouts, the forces on the connectors were not evenly distributed. With the strain gages on the reinforcing bars of one of the two slabs, the force on that slab was computed. The force on the other slab was the difference of the applied load and the load in the instrumented slab. The force per bolt was taken as half the force per slab. The distribution of the load to each bolt at the ultimate was assumed to have been according to same ratio as that for the last recorded load increment. The corresponding slip for each slab was the average slip of the two bolts connecting the slab. The load per bolt and slip per bolt quantities presented in this report are those for the slab of each pushout that received the greater component of the applied load.

The desired result from the pushout tests was the development

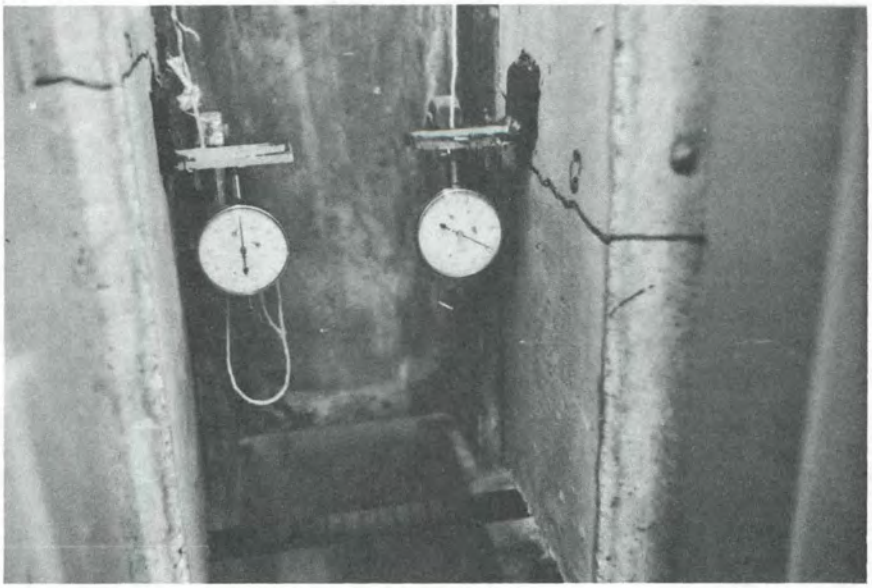


Figure 12 Separation of Slabs

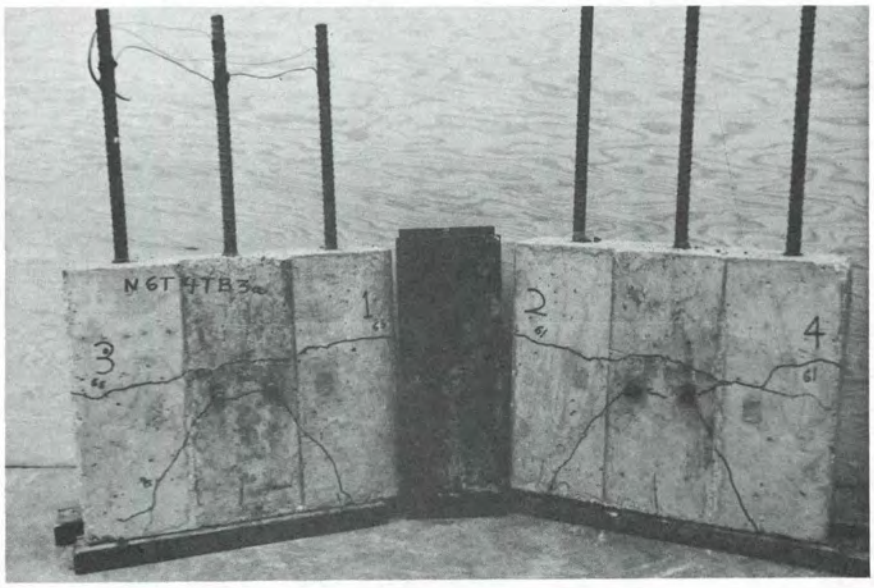
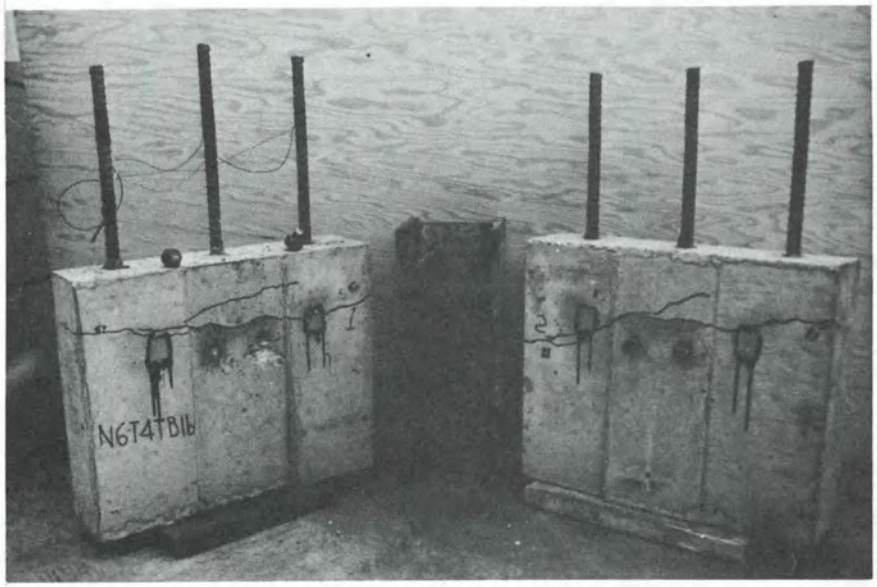


Figure 13 Tension Pushout
Concrete Slab Failure



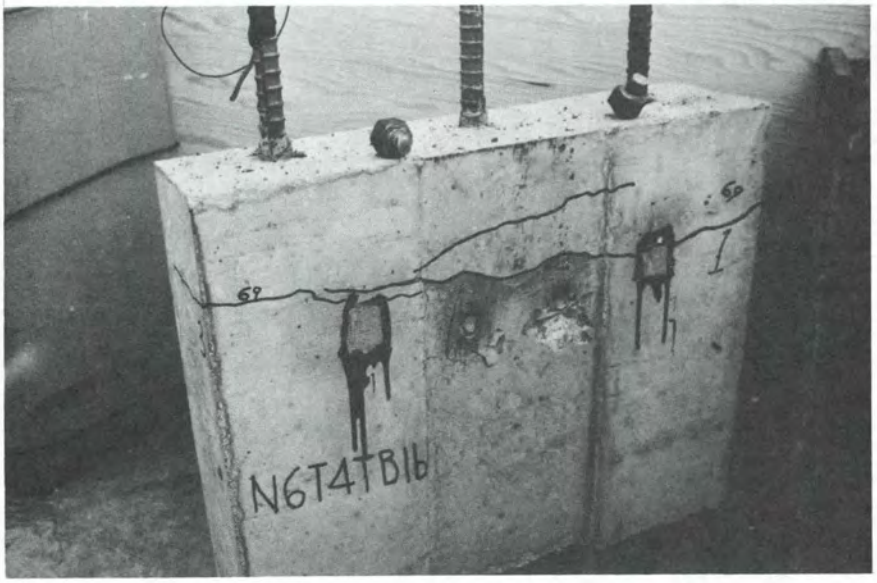


Figure 14 Tension Pushout
Connector Failure

Table VIII

Tension Pushout Specimen Results

	N6T4TB1a	N6T4TB1b	N6T4TB2a	N6T4TB2b	N6T4TB3a	N6T4TB3b
f'c	7953 (psi)	7953	8292	8292	6523	6523
Crack Load Per Bolt	17.45 (kips)	19.70	18.95	16.75	14.65	14.35
Crack Slip Per Bolt	0.0010 (in.)	0.0107	0.0262	0.0015	0.0026	0.0000
Maximum Load Recorded Per Bolt	33.55 (kips)	37.30	37.45	29.10	26.95	31.20
Maximum Slip Recorded Per Bolt	0.1071 (in.)	0.2486	0.1332	0.0548	0.0948	0.0256
Ultimate Load Per Bolt	34.39 (kips)	37.30	38.16	30.15	27.49	32.70
Type of Failure	Concrete	Connector	Concrete	Concrete	Concrete	Concrete

of the load-slip curve. The curves of the two companion pushouts for each composite member are presented together. (See figures 15, 16, 17).

4.2 COMPOSITE BEAM RESULTS

4.2.1 Observations during testing Certain general observations, made during the testing of the members, shall be included in this section. This is done in order to familiarize the reader with the performance of the members before the presentation of the formal results.

The composite members had wiring protruding from the sides of their slabs, because of the presence of strain gages on the reinforcing bars. (See figure 18). It was noticed that the first cracks in the concrete appeared along this wiring. Further observations on the cracking patterns shall be discussed in a later portion of this chapter.

Cracks also formed on the bottom of the slabs at higher loads. These cracks usually formed at the shear connectors. (See figure 19).

On the simply supported section of the continuous span, the load point was 6 feet from the negative moment support. During the tests, there was no recorded slip for that portion of the composite member extending from this load point toward the other support.

NCB6B2 did not have connectors over its negative moment region. The slab appeared to slip in relatively large quantities, especially in the early loading

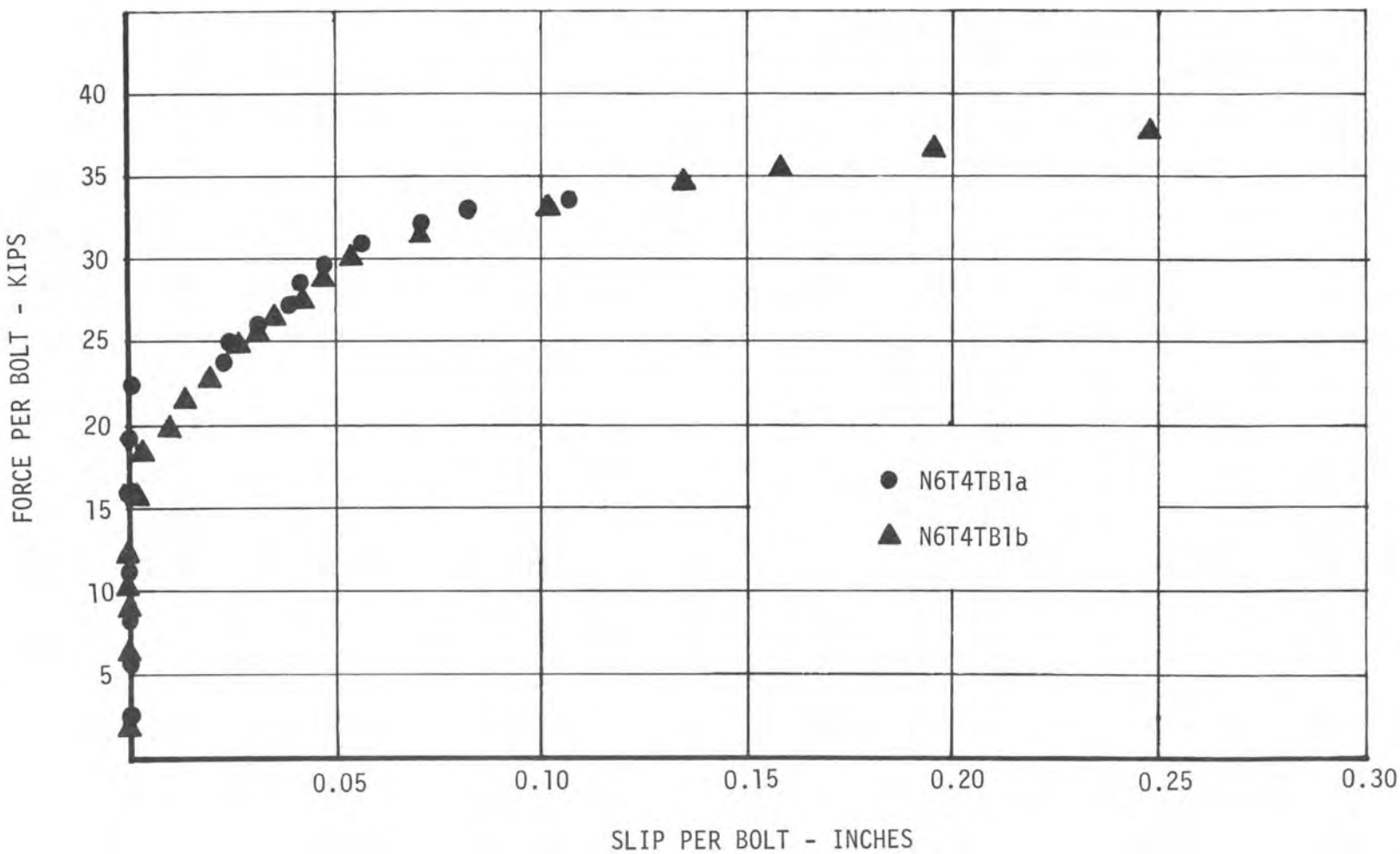


Figure 15 Load Versus Slip - N6T4TB1a, b

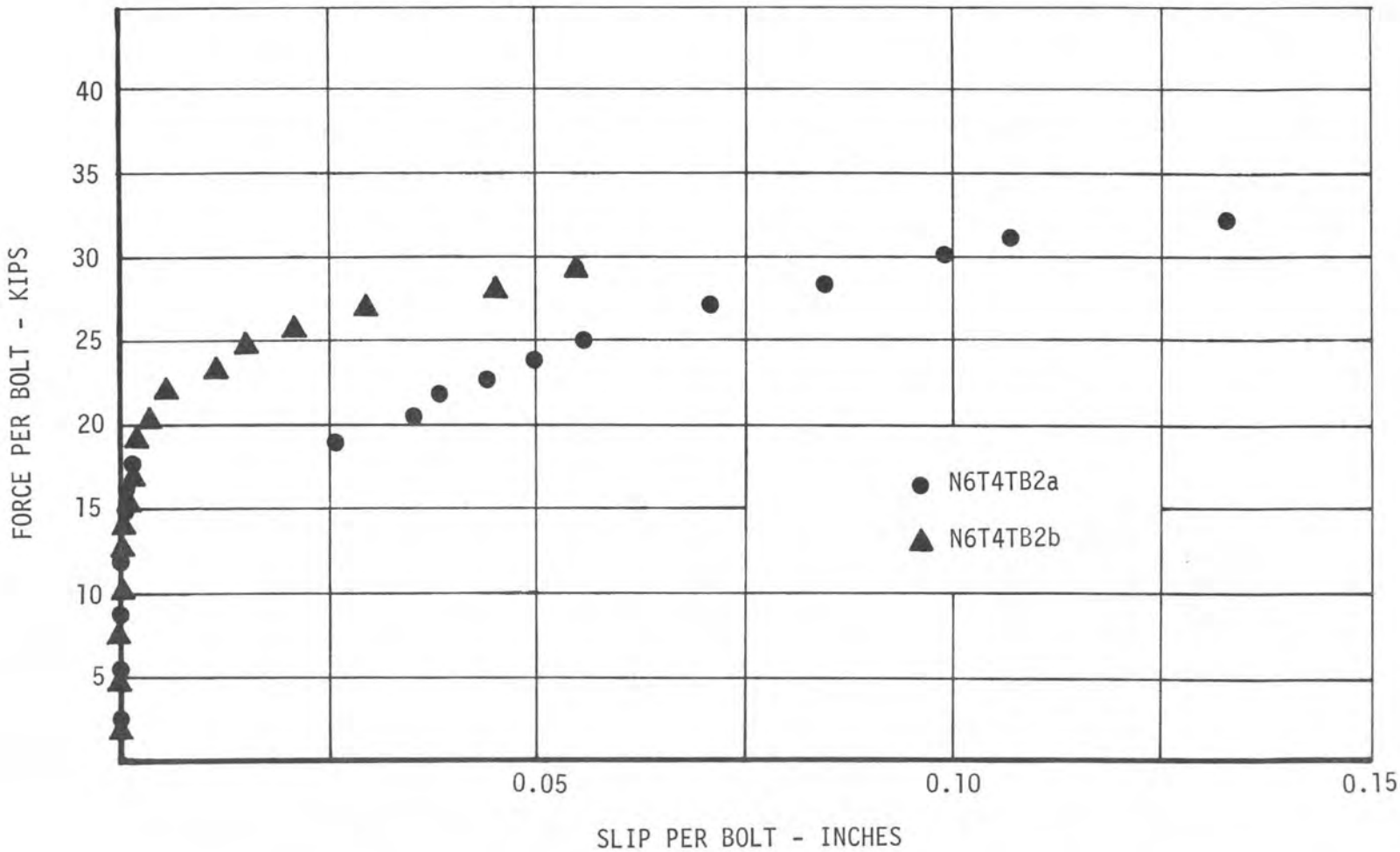


Figure 16 Load Versus Slip - N6T4TB2a, b

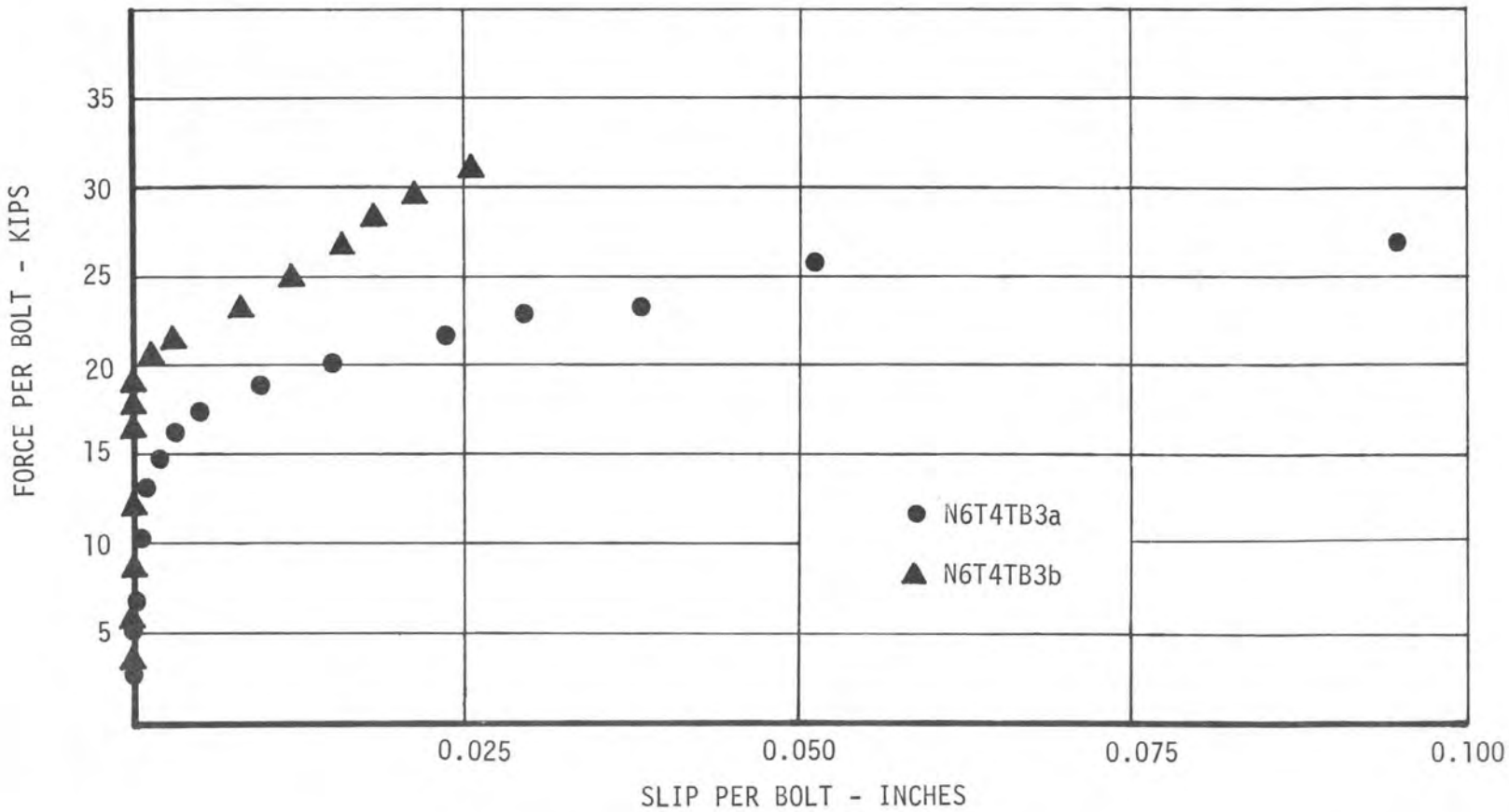


Figure 17 Load Versus Slip - N6T4TB3a, b

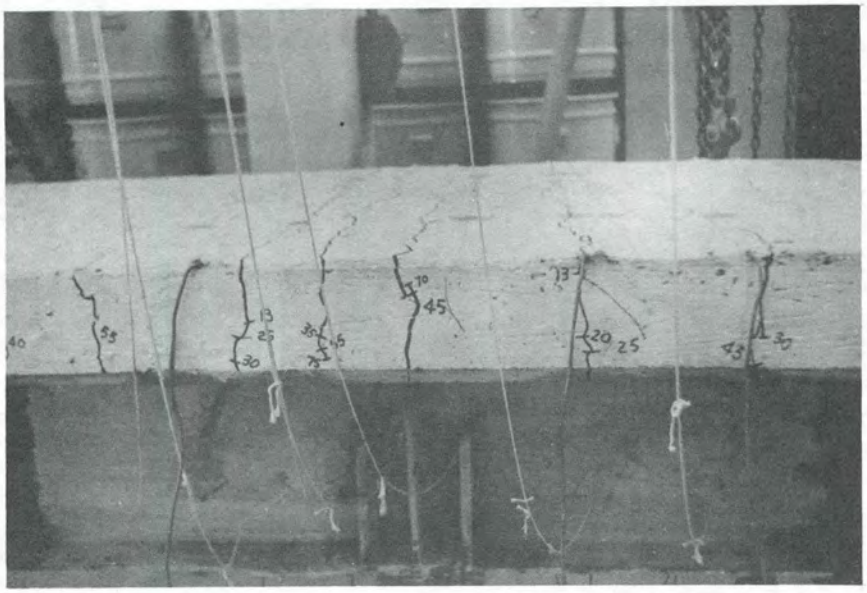


Figure 18 Gage Wiring (NCB6B3)

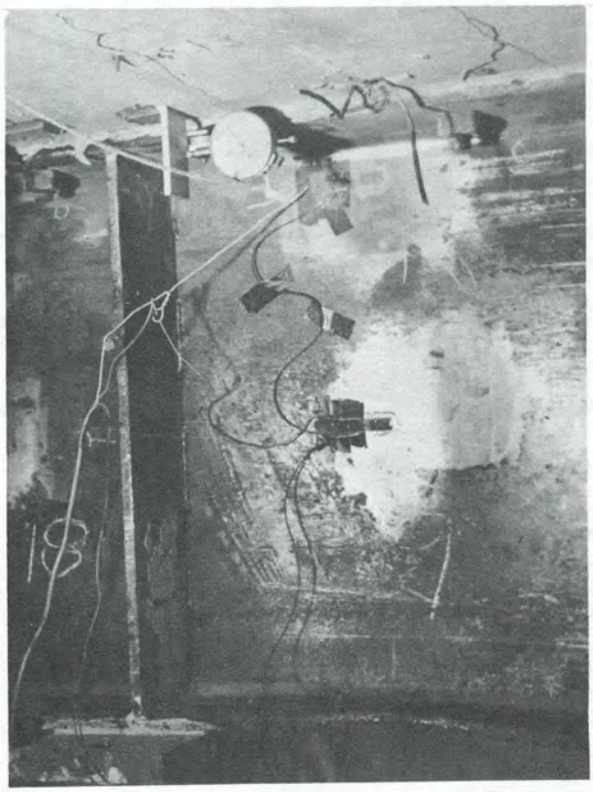


Figure 19 Cracking at Connectors
Buckling of the Flange (NCB6B1)

stages. At the load increments during which audible sounds were observed, a large increase in slip was recorded.

The failure of all three members was caused primarily by the buckling of the bottom flange at the negative support. This mode of failure appeared at the side of the pier facing the simply supported section. (See figure 19).

There were no noticeable effects on the connectors during the testing. All connectors were fully intact at the end of each test.

4.2.2 Quantitative analysis For the presentation of the results of the testing, the working load, yielding load, and the ultimate load of each composite member was determined both theoretically and experimentally. The assumptions and theory used in the computation of these quantities shall now be described. The actual beam measurements, and the information obtained from the coupon and cylinder tests were used in all the theoretical and experimental calculations.

The theoretical working and yielding loads for NCB6B1 and NCB6B3 were based on elastic theory and section transformations (1) (9). In determining the working load, the allowable values were based on the AASHO specifications (3). The yielding load was based on the first yield of any of the steel in the cross section. The theoretical ultimate load for the member was based on the assumption that all the steel

had yielded at the failure of the critical cross section. The theoretical working and yielding loads for NCB6B2 were based on the assumption that there was no interaction between the slab and the steel beam. It was also assumed that the curvature of the two components were equal at any load (10). It was, therefore, necessary to find the working load of the steel beam. Then the curvature which it produced was equated to that for the slab, and the corresponding load in the slab was determined. Then the working load of the slab was found and the curvature equated to that for the beam. The condition producing the more critical load governed. This procedure was also followed in establishing a first yielding load. In obtaining the ultimate condition for NCB6B2, it was assumed that at failure both the steel beam and the slab must attain their ultimate condition. Ultimate strength theory was used in the analysis of the slab (9). Again, it was assumed that the steel beam had completely yielded at ultimate.

The experimental working and yielding loads for member NCB6B3 was determined from the recorded strain gage data. The loads corresponding to the strains at working stress and first yield stress were designated the working load and yielding load, respectively. In beams NCB6B1 and NCB6B2 the strain data was unreliable and therefore the working and yielding loads were determined from the deflection data. The theoretical deflection at the overhang was calculated for the theoretical working and yielding loads. The applied loads which produced these deflections were designated the experimental working and yielding loads for NCB6B1 and NCB6B2.

The results of the theoretical and experimental analyses for the three members are listed in Table IX.

4.2.3 Deflection results The deflection of the members under the load point at the cantilevered section are presented in figures 20, 21, 22. A solid line has been included, which represents the theoretical deflection computed using elastic beam theory (cracked slab) assuming complete interaction between the slab and the steel beam for NCB6B1 and NCB6B3. For NCB6B2, this line represents no interaction between the slab and the steel beam.

NCB6B2 seems to have acquired a little composite action from the friction between the slab and the steel beam.

4.2.4 Slip results The slip distribution along the members are presented next. (See figures 23, 24, 25). The slip has been plotted against the distance from either side of the negative support. The distance included was the 7 foot overhang and 7 feet of the simply supported portion. As was previously noted, there was no slip recorded beyond the interior load point toward the other support. The loads, for which the information is presented, are the working load, the yielding load, and the ultimate load of the composite member.

It can be seen from the figures that most of the slip occurs at the overhang portion of each member. The distribution of the slip in NCB6B2 follows a definite pattern, but that in NCB6B1 and NCB6B3 is distributed randomly along the member.

4.2.5 Strain results Strain profiles for the three beams are introduced next for investigation. (See figures 26, 27, 28). The strain

Table IX

Analysis Result For

Test Beams

NCB6B1

	Theoretical	Experimental
Working Load	31.6 (k)	33.0^k
Yielding Load	59.8	52.5^k
Ultimate Load	80.8	87.3

NCB6B2

	Theoretical	Experimental
Working Load	27.5 (k)	27.3
Yielding Load	53.9	52.1
Ultimage Load	66.2	84.7

NCB6B3

	Theoretical	Experimental
Working Load	36.7^k	39.1
Yielding Load	65.5	67.3
Ultimate Load	103.5	101.3

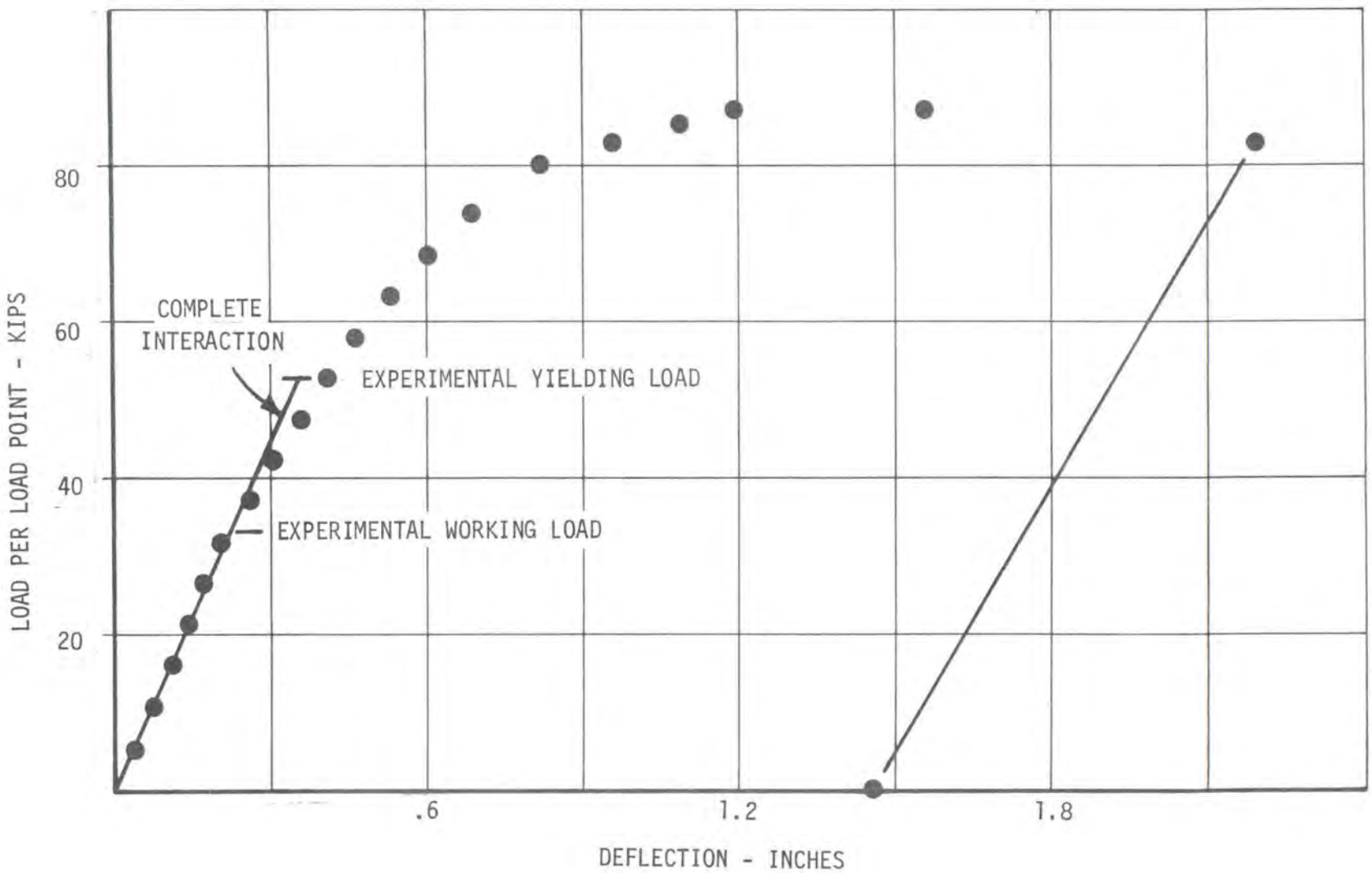


Figure 20 Overhang Deflection - NCB6B1

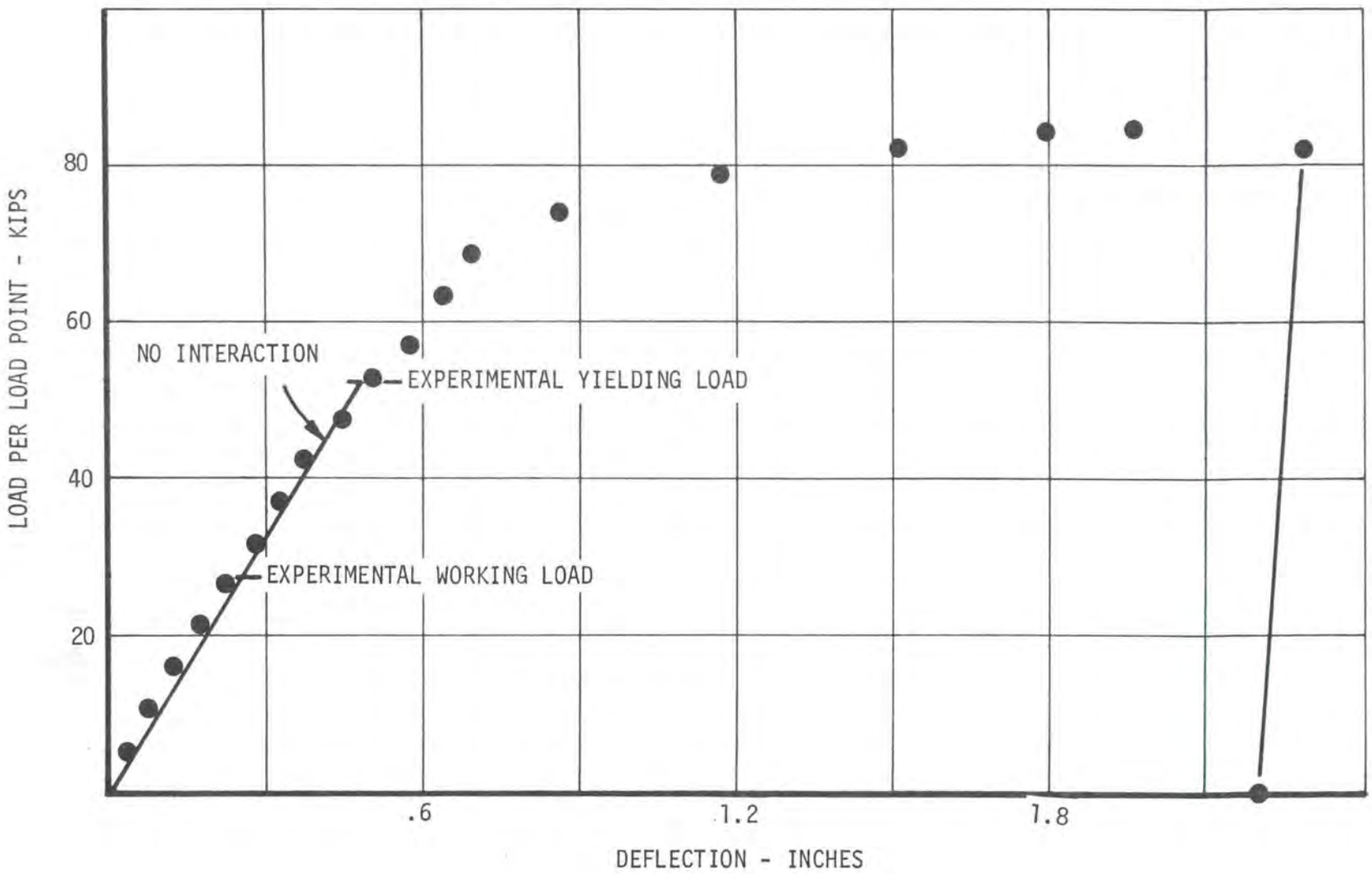


Figure 21 Overhang Deflection - NCB6B2

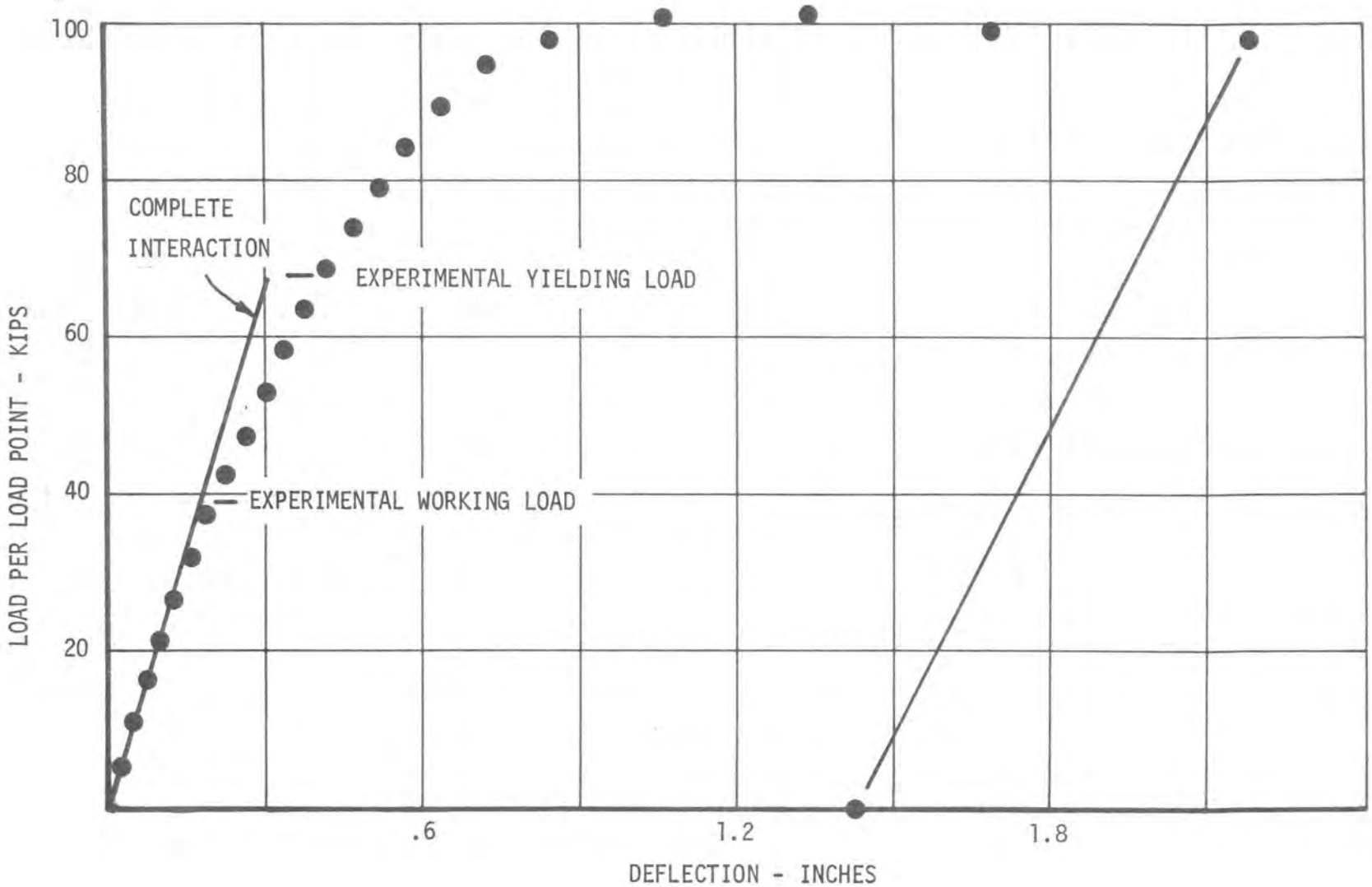


Figure 22 Overhang Deflection - NCB6B3

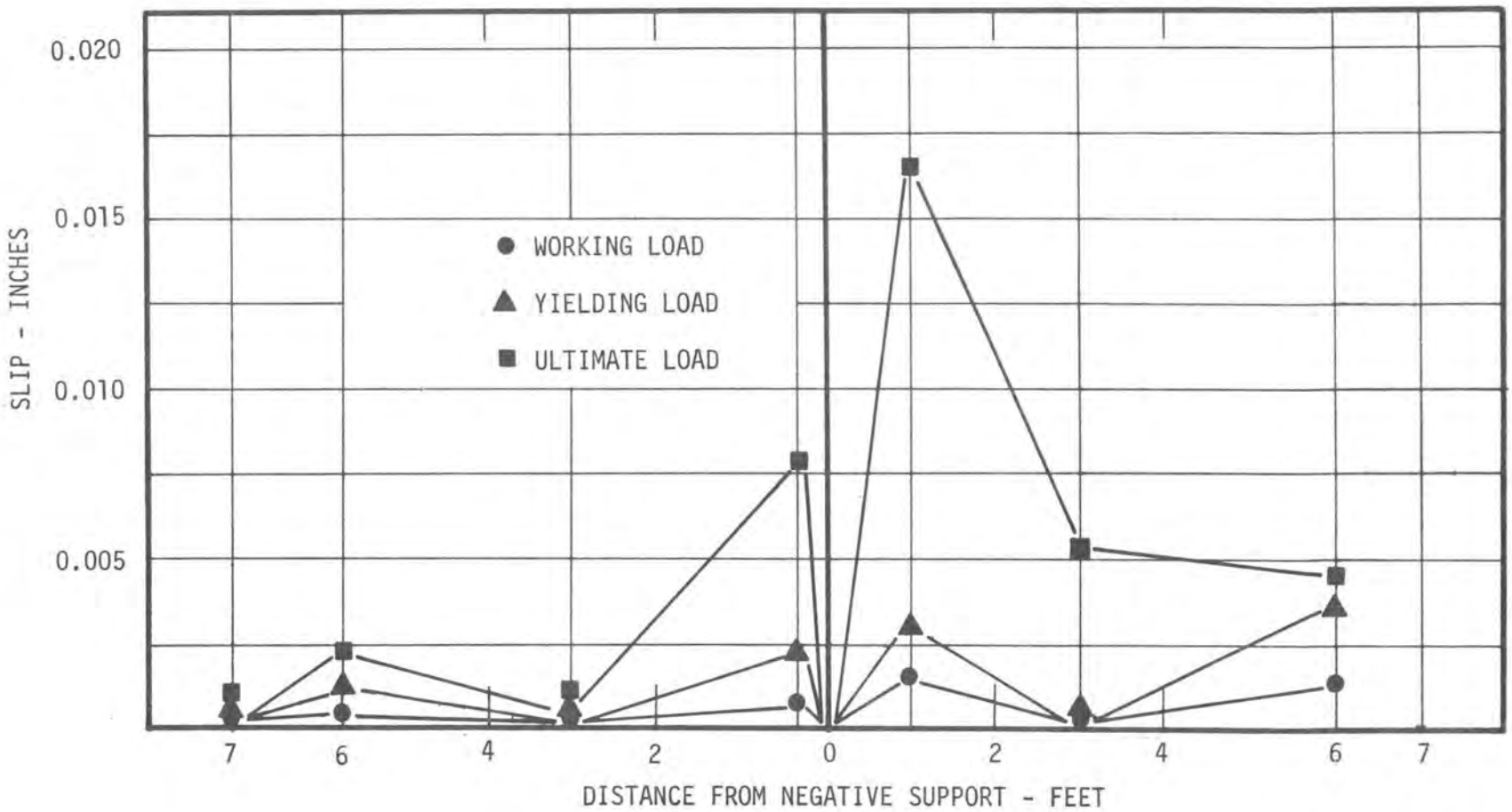


Figure 23 Slip Distribution - NCB6B1

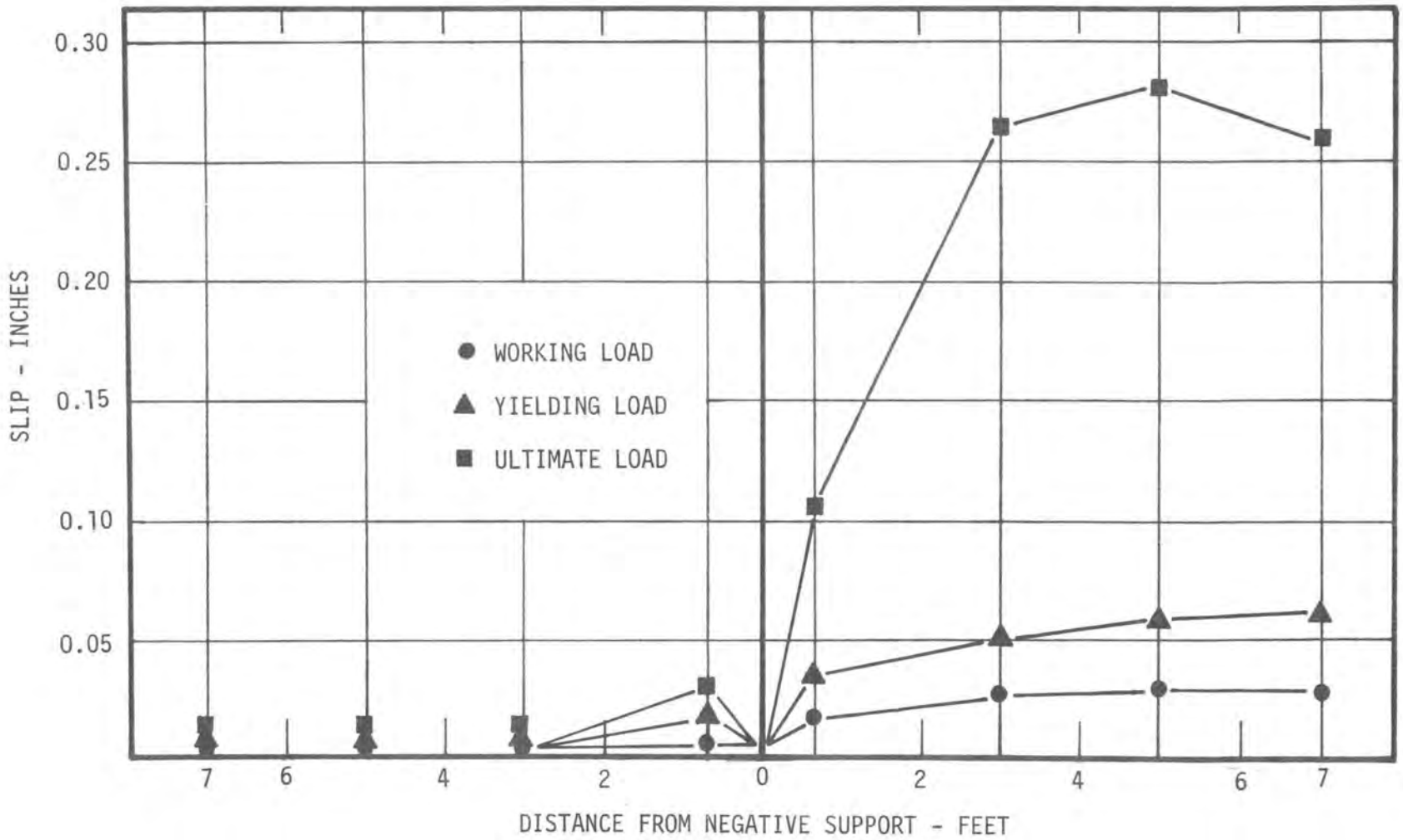


Figure 24 Slip Distribution - NCB6B2

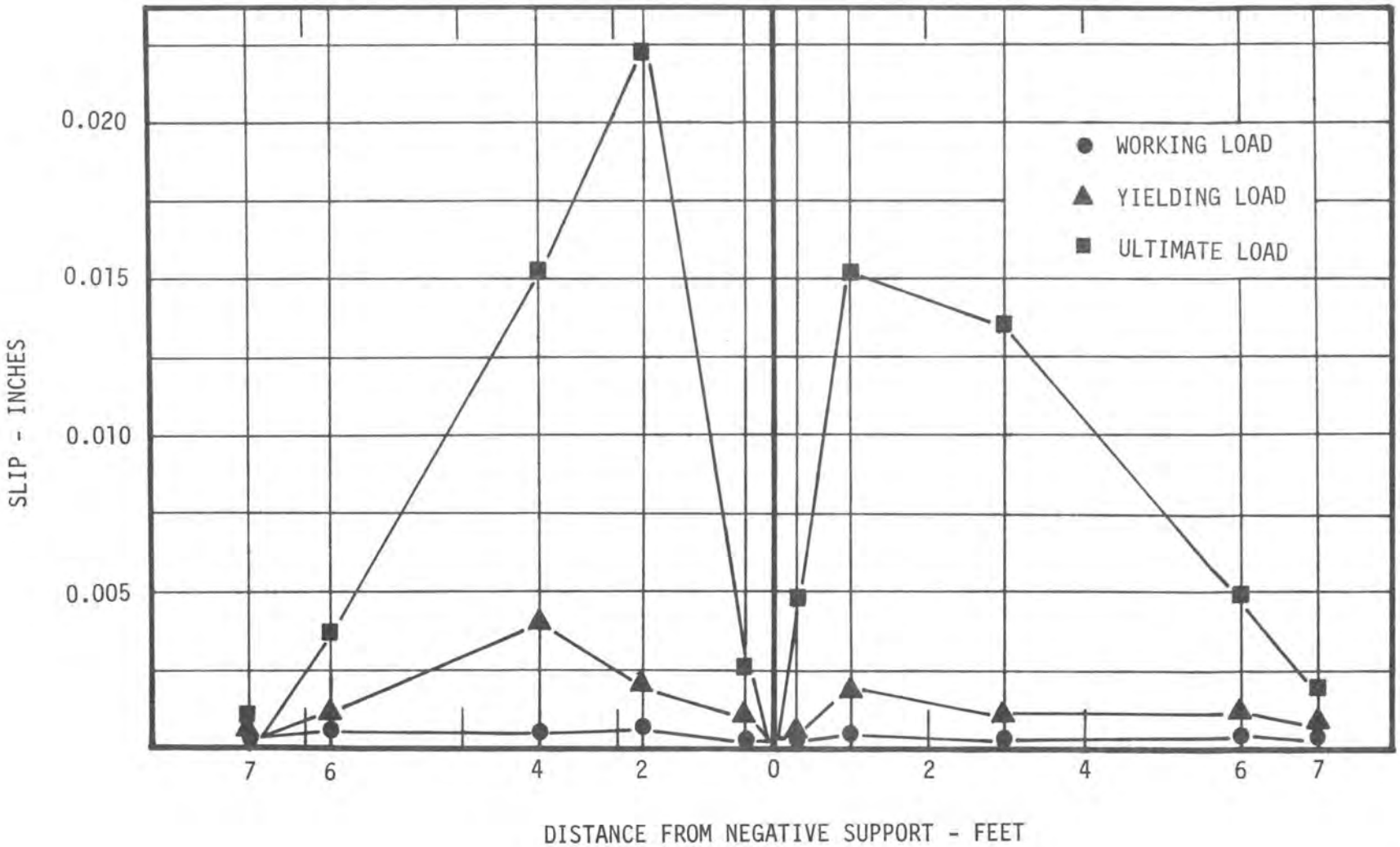


Figure 25 Slip Distribution - NCB6B3

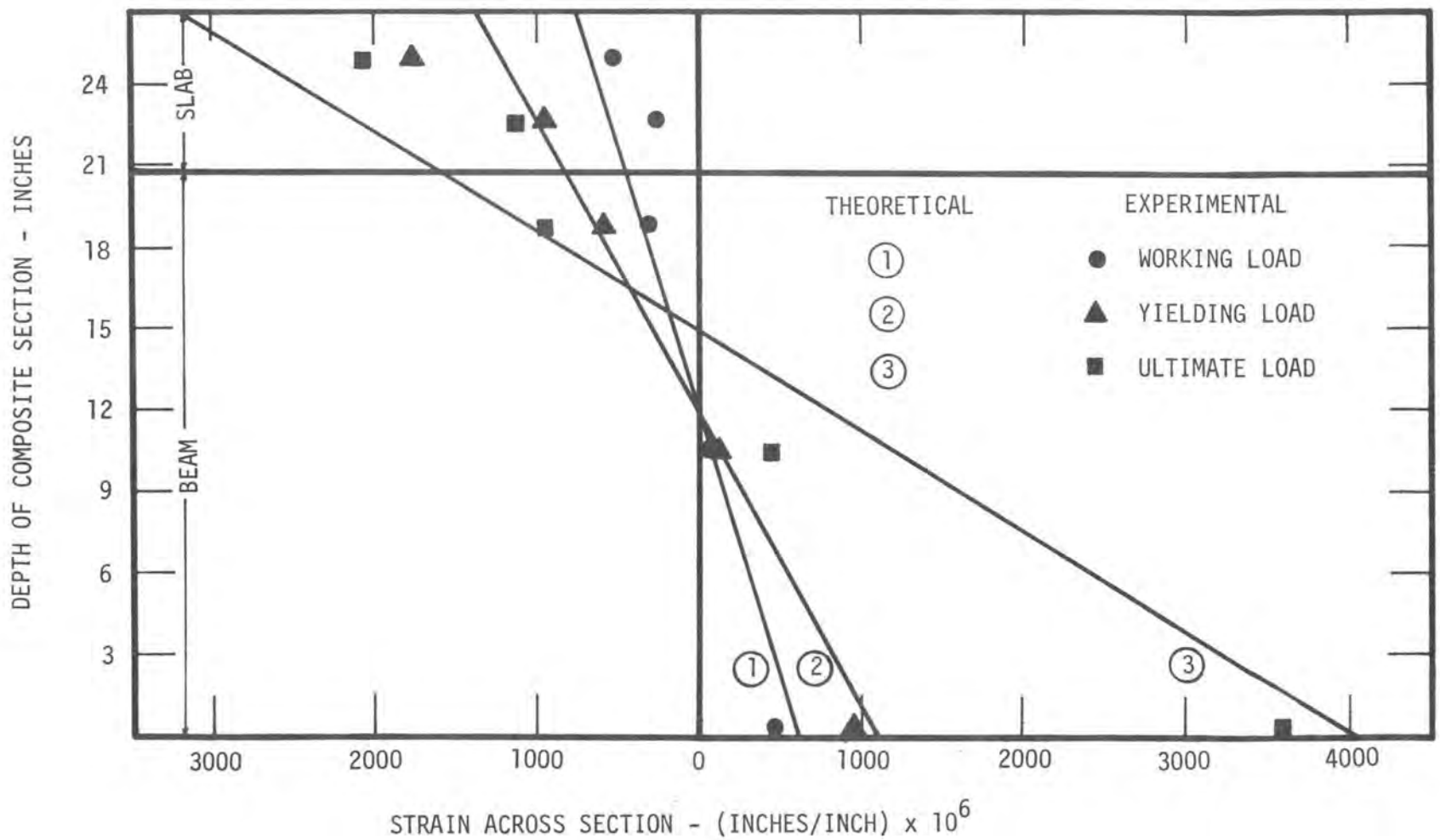


Figure 26 Strain Profile - NCB6B1

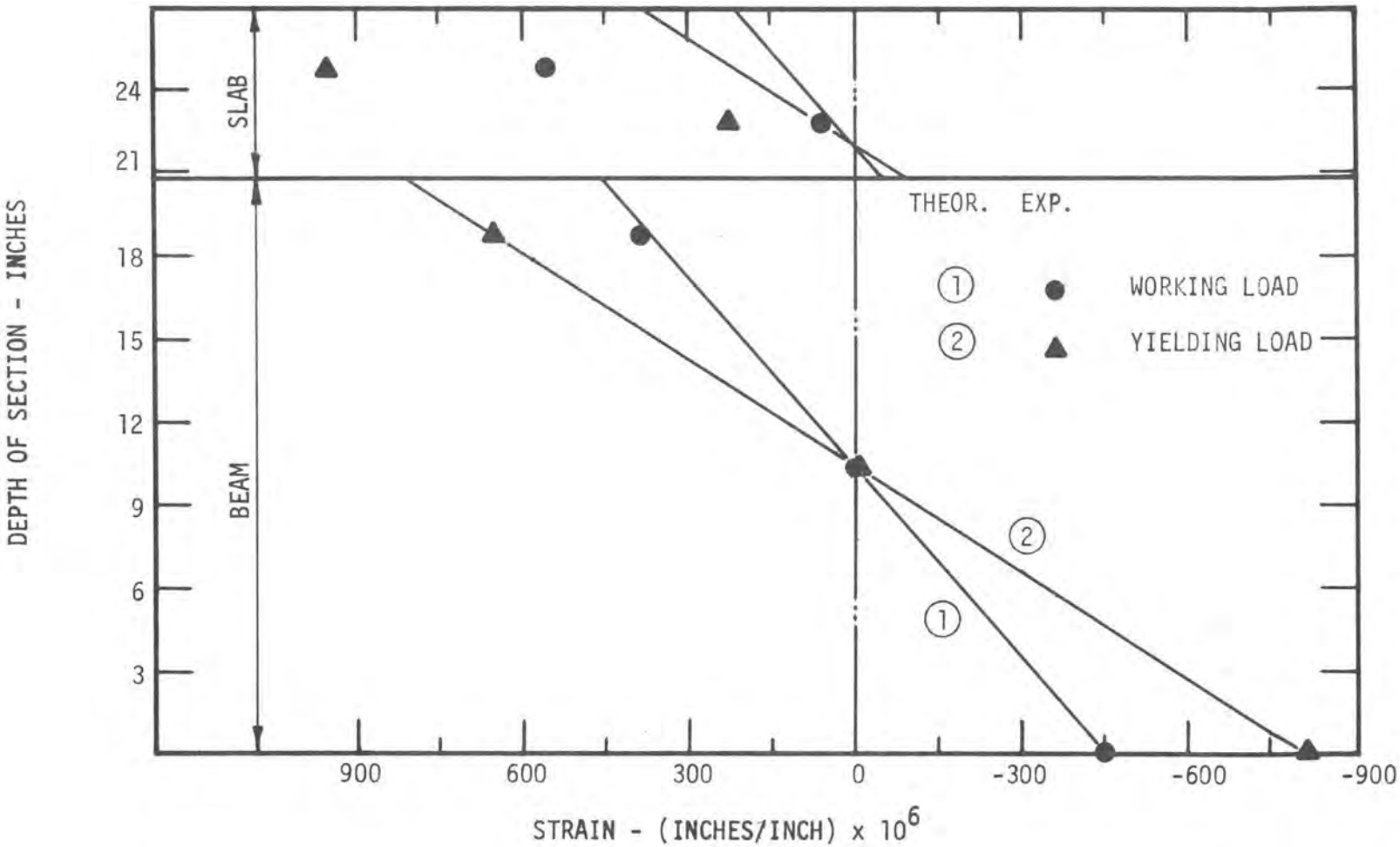


Figure 27 Strain Profile - NCB6B2

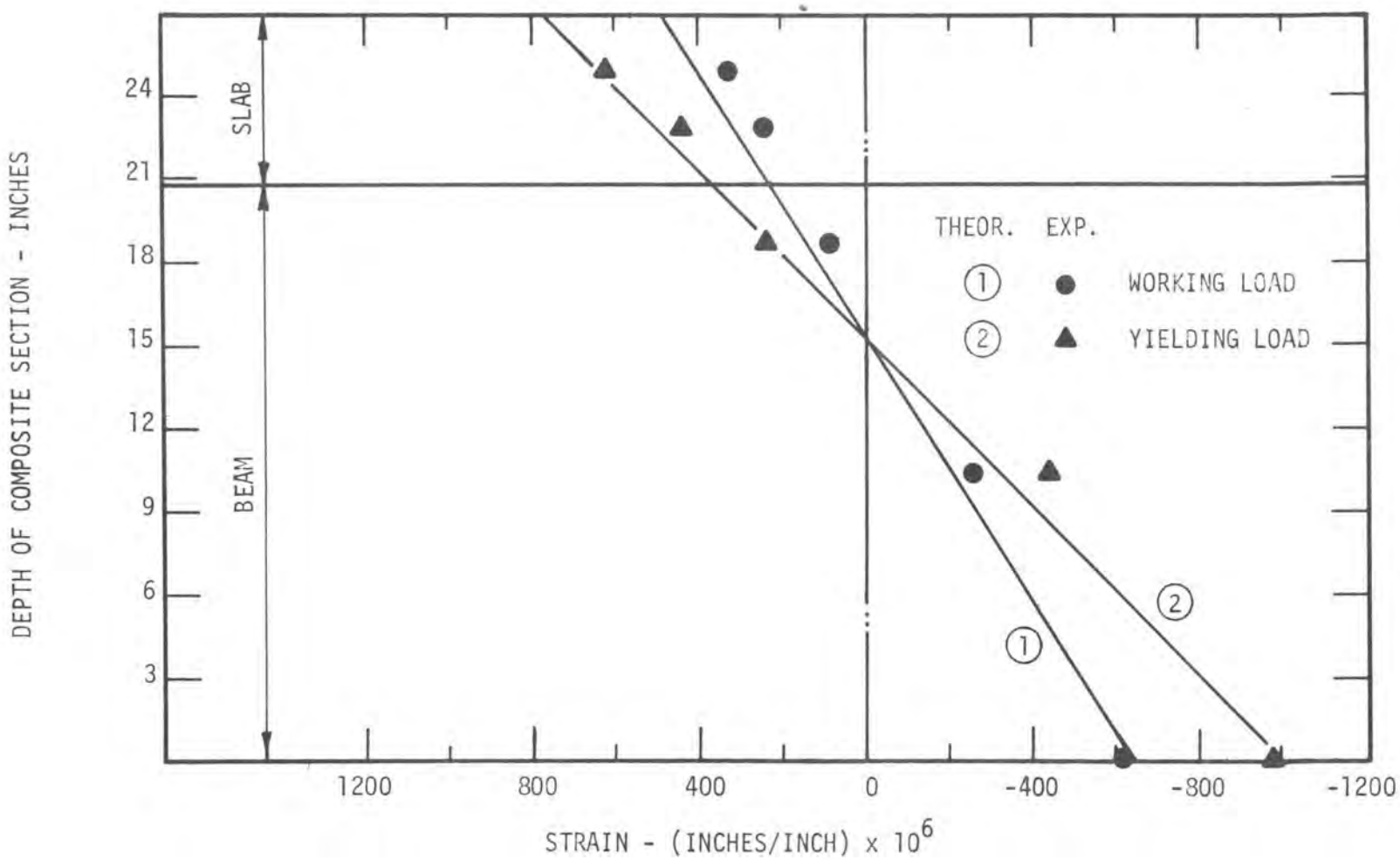


Figure 28 Strain Profile - NCB6B3

has been plotted against the depth of the member. An increase to the right from zero strain indicates compression and to the left indicates tension. The strain profiles were taken at 8 inches from the negative support on the cantilever section.

The profiles are plotted for the working load and the yield load of each member and the ultimate load of NCB6B1. The theoretical profiles are indicated by solid lines.

Generally the experimental strains plotted for the steel beams generate straight lines and agree well with the theoretical profiles. The exceptions occur at the ultimate loads and is due to the large slip between the concrete slab and the steel beam section. The experimental strains in the reinforcing bars of NCB6B1 and NCB6B2 do not generate lines parallel to the theoretical profiles. The bar strains of NCB6B3 form lines nearly parallel to the theoretical profiles. The quality of this (NCB6B3) data can be directly attributed to the number of gages placed on the bars at each strain profile. It was shown by this data that the distribution of strain - in the slab - transverse to the major axis of the member was parabolic. The insufficient information obtained in the other two members did not give average strains for the slab but merely strains for two isolated points along the parabola.

4.3 COMPARISONS

4.3.1 Cracking patterns As mentioned previously, the first cracks appeared at the strain gage wiring in the slab. All the cracking was confined to the area of the slab between the two load points. The cracks along the top of the slab at first ran perpendicular to the major axis of the member. The spacing between the cracks was approximately 8 inches. Figure 29 shows an early cracking pattern of NCB6B1. As the loading approached the ultimate, the transverse cracks were interconnected by longitudinal and diagonal components. (See figures 30, 31). The total degree of cracking was approximately the same for NCB6B1 and NCB6B2. (See figure 30). As expected, that for NCB6B3 was greater. (See figure 31).

The vertical cracks on the side of the slab are continuations of the evenly spaced cracks at the top. Toward the ultimate loading, there were scattered occurrences of horizontal cracking on the sides, especially for NCB6B1. (See figure 32).

4.3.2 Beam and companion pushouts The load-slip results of the pushouts were to be compared with that of a connector in the companion member.

Strain gages were placed in vertical profiles on both sides of a chosen set of connectors. For each load increment, the strain was integrated over the area of the beam (11). This was then multiplied by the modulus of elasticity to give the axial force in the beam at that profile. The difference between the forces at the two profiles was the shear carried by the two connectors. The results from these calculations were not consistent, and inferred that an increase in

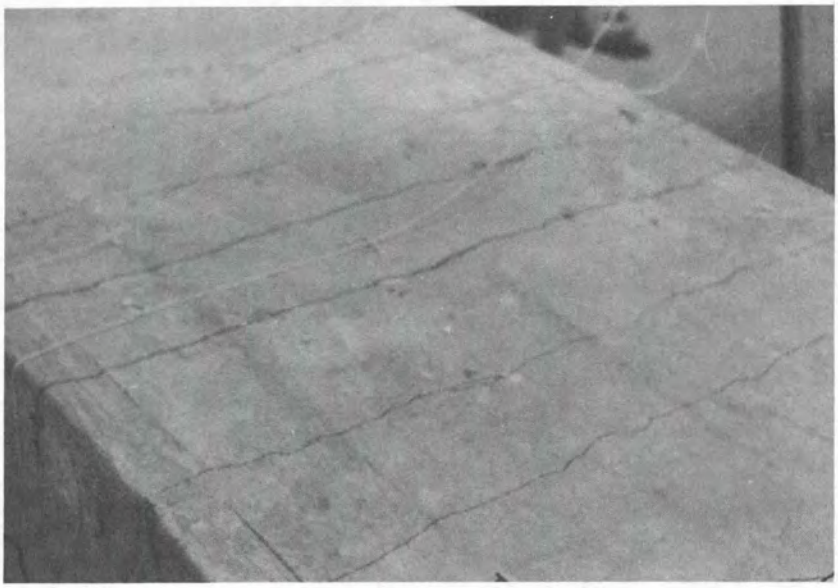


Figure 29 An Early Cracking Pattern (NCB6B1)

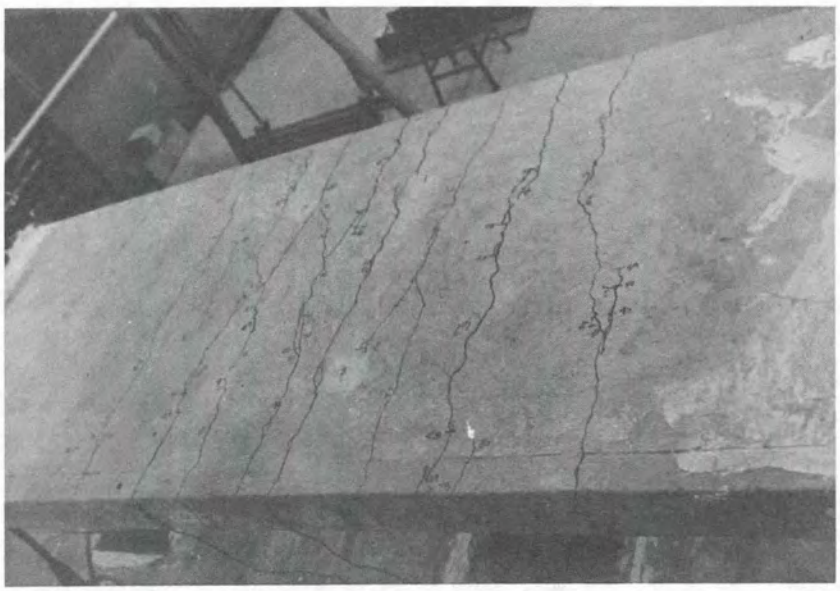


Figure 30 Cracking Pattern - NCB6B2

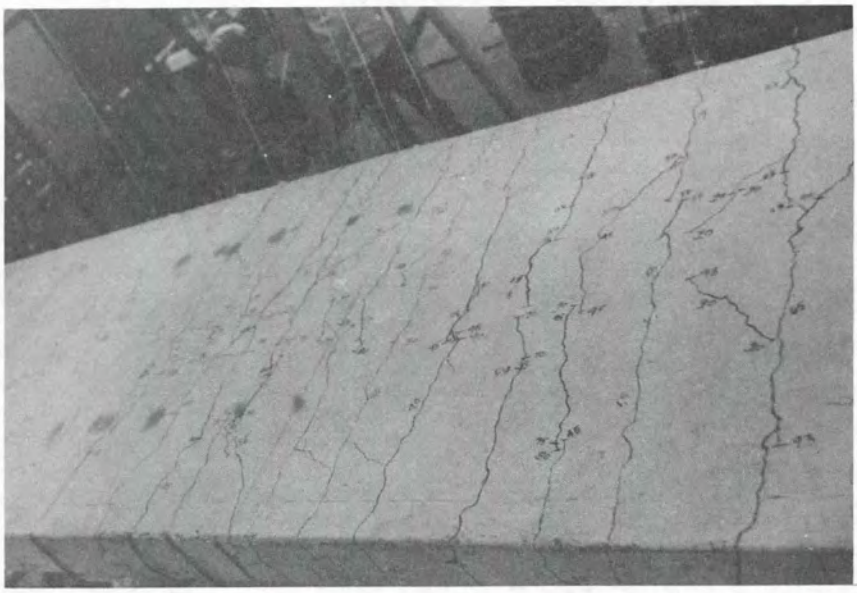


Figure 31 Cracking Pattern - NCB6B3

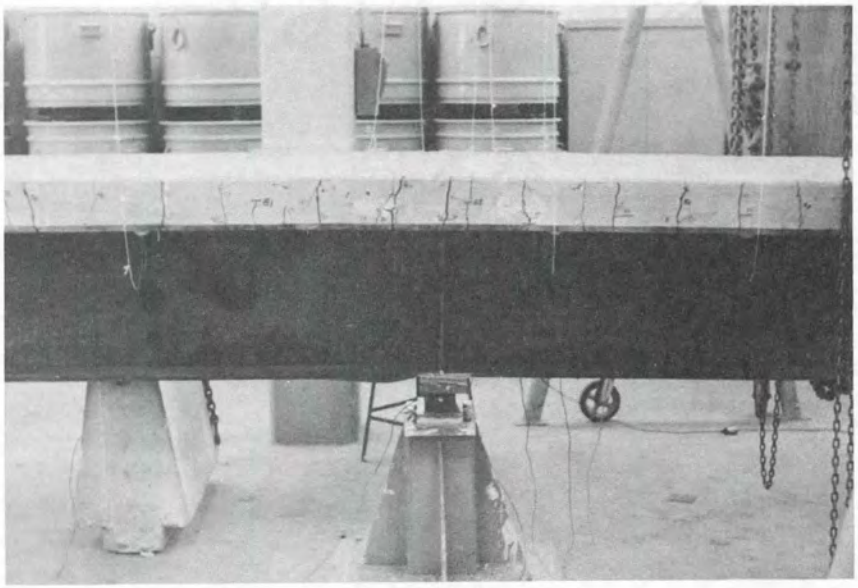


Figure 32 Vertical Cracking Pattern (NCB6B1)

load did not necessarily produce a corresponding increase in shear. This could have been due to the spacing and the positioning of the profiles. Because the connectors were spaced at 8 inches over the negative moment region, the profiles were only 4 inches on either side of the connectors. Also, the profiles were relatively close to the negative support. Because of the "symmetrical" loading of the composite member, the profiles were in a minimum slip region of the member as shown in figures 23 and 25.

In NCB6B3, there were a considerable number of strain gages placed on the reinforcing bars at each profile. The shears were, therefore, calculated by the differences of forces in the reinforcing steel. The results were more consistent.

The load per bolt versus the slip per bolt for the member and its pushouts are compared. (See figure 33). The curves for the pushouts were reproduced from those in section 4.1.4. The load per bolt for NCB6B3 is half the calculated shear. The corresponding slip was obtained from the dial readings.

4.3.3 NCB6B1 and NCB6B2 Next, an investigation of the difference in the behavior of a composite member for which the connectors had been omitted over the negative moment region shall be presented.

First, a comparison of the stiffness of the two members. (See figure 34). The curves are a reproduction of the overhang deflection curves in Section 4.2.3. It can be seen that there is considerably more stiffness when shear connectors are placed over the negative moment region.

The slips of the two members are compared at a common position along the cantilever section. The gages were placed in NCB6B1 after

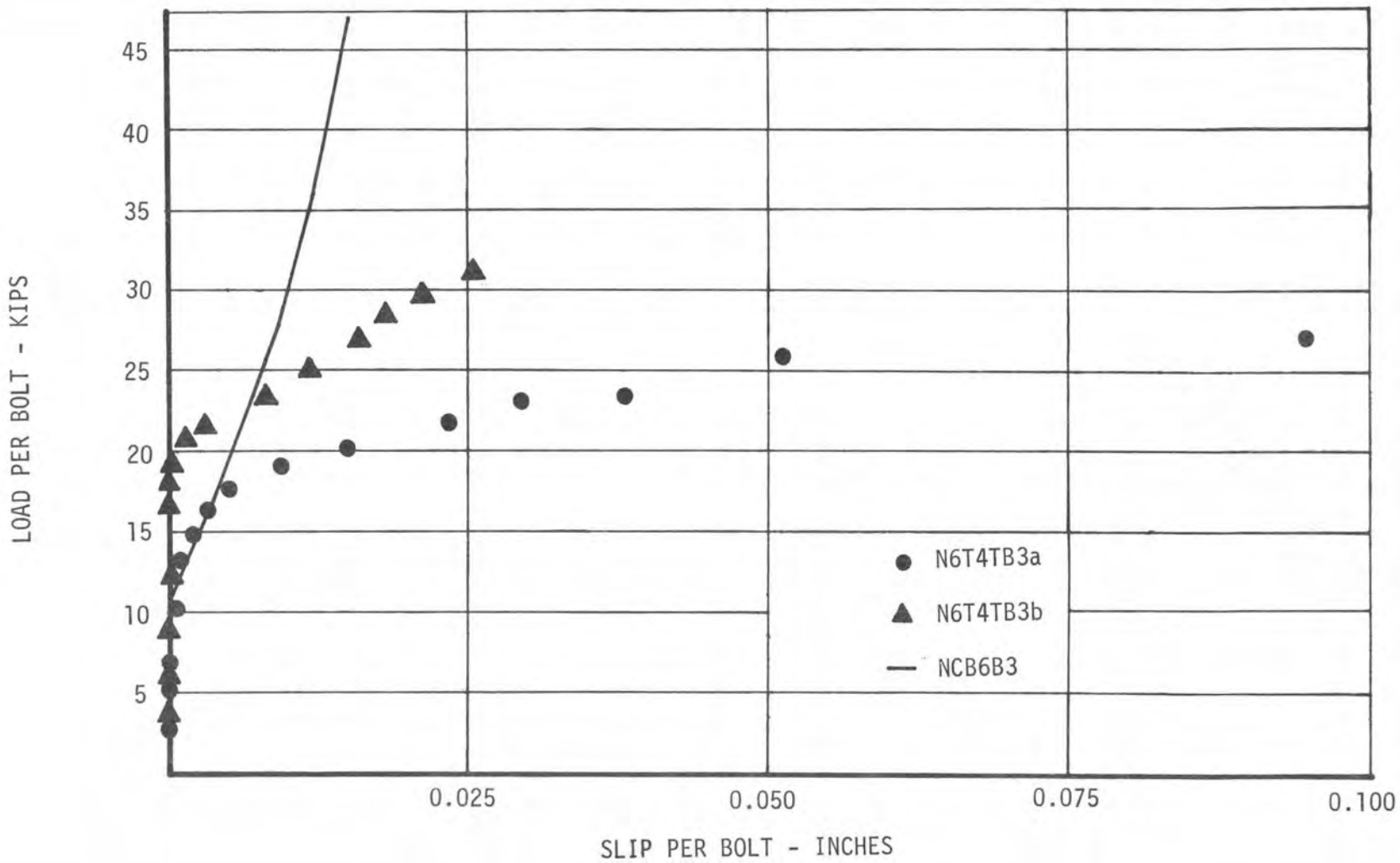


Figure 33 Comparison - NCB6B3 and N6T4TB3a, b

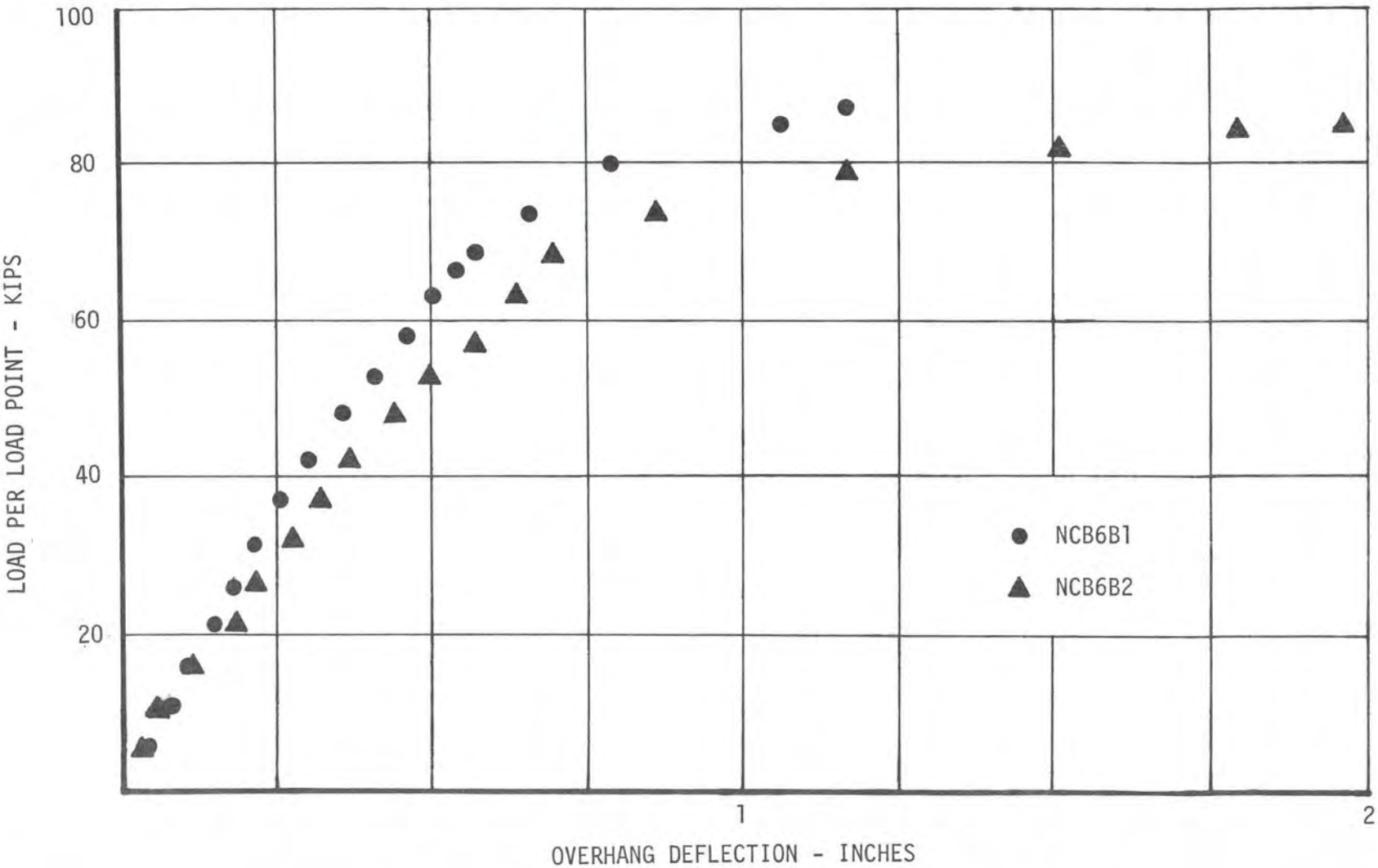


Figure 34 Comparison of Deflections - NCB6B1 and NCB6B2

the first set of connectors from the negative support and in NCB6B2 at a comparable distance. (See figure 35). As expected, the slip of NCB6B2 greatly exceeded that for NCB6B1.

For the final comparison, the strains at a common point along the cantilever section are presented. The gage under discussion had been placed on the bottom flange at the strain profile 8 inches from the negative support. (See figure 36). The results follow a pattern similar to that for the overhang deflections.

4.3.4 Ultimate strength NCB6B3 had been designed so that the entire steel beam would yield in compression and the reinforcing steel in tension.

The member failed prematurely by buckling of the bottom flange. (See figures 37, 38). At the ultimate load 67% of the steel beam had yielded. A strain diagram has been introduced here in order to illustrate the degree of yielding at the ultimate of the beam. (See figure 39). The shaded area represents the elastic strain region, and the hatched area the plastic region.

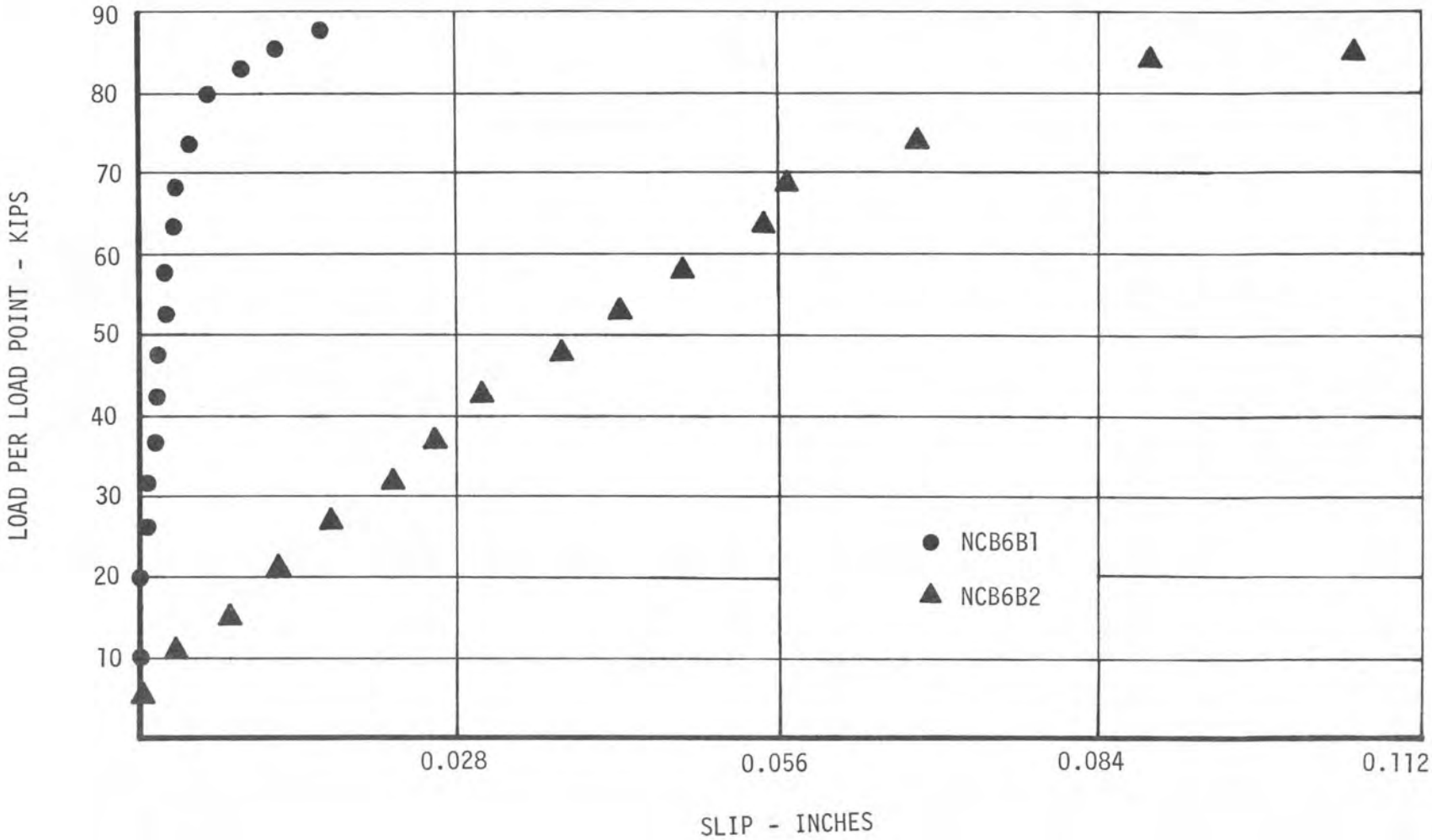


Figure 35 Comparison of slips - NCB6B1 and NCB6B2

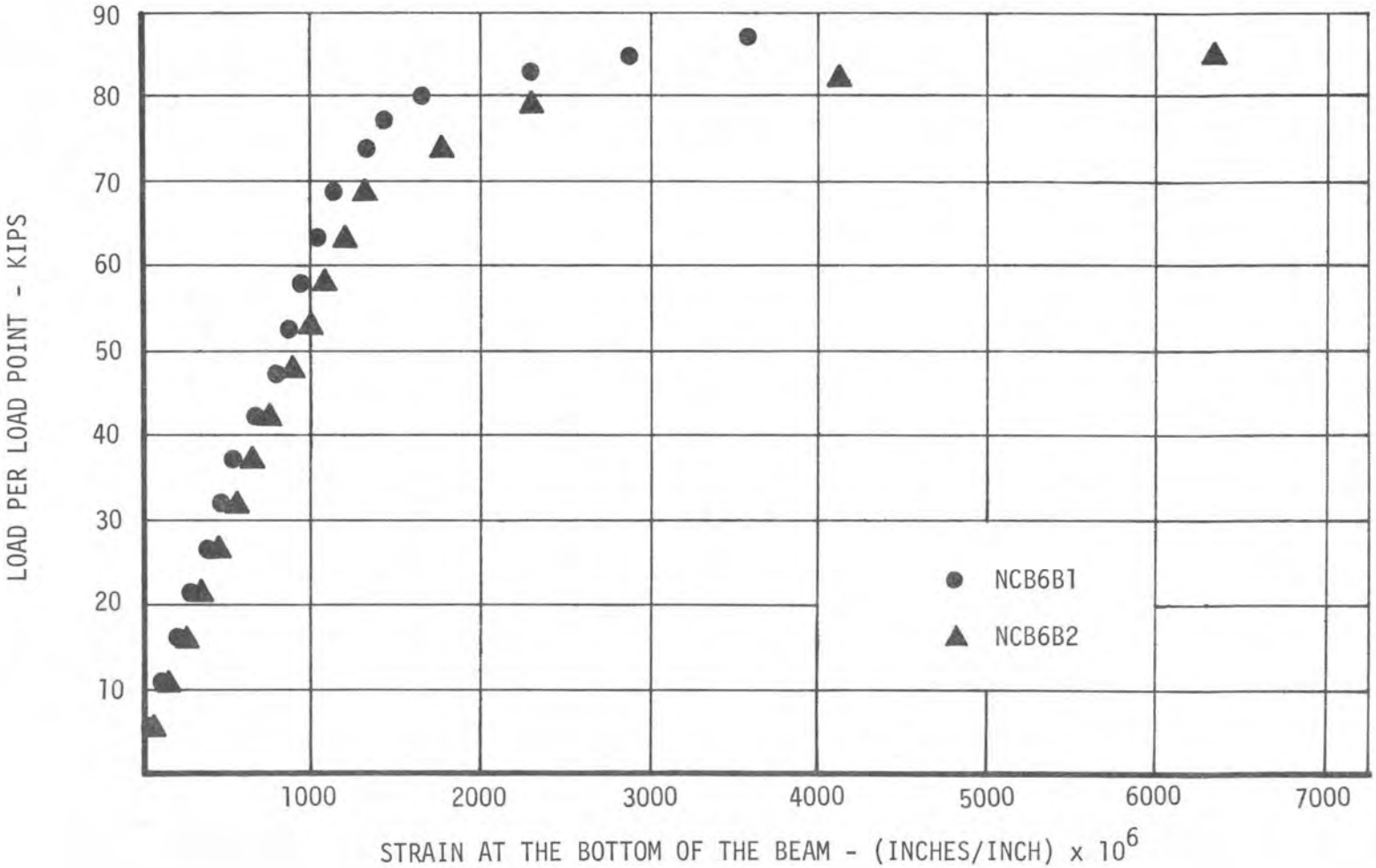


Figure 36 Comparison of Strain - NCB6B1 and NCB6B2

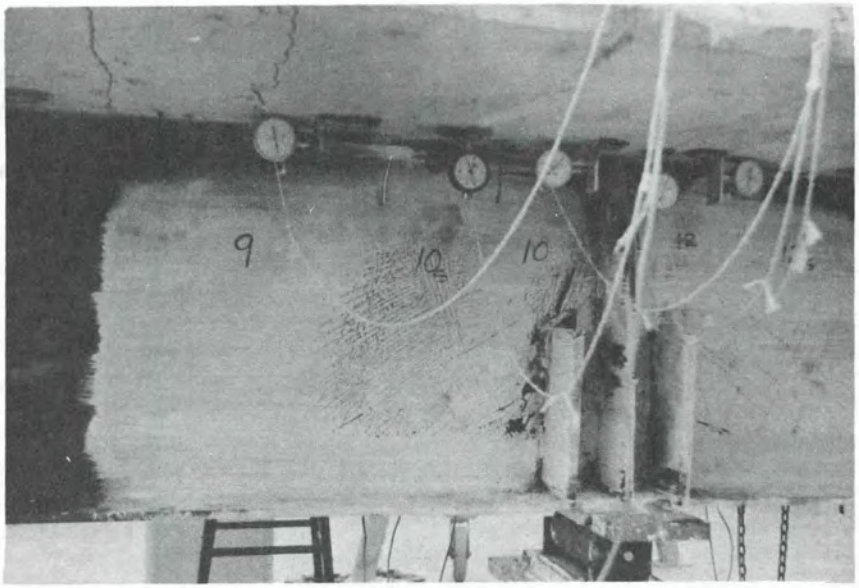


Figure 37 Luder Lines Before Failure - NCB6B3

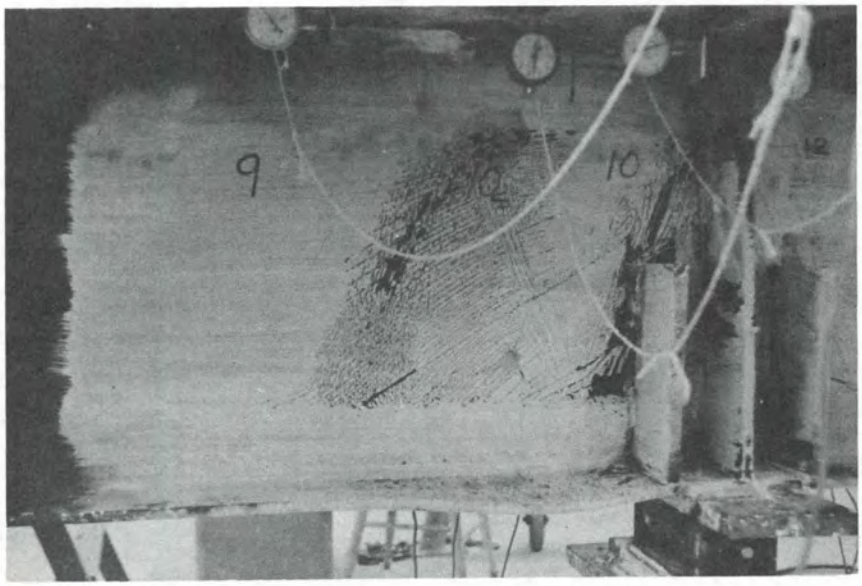


Figure 38 Secondary Failure - NCB6B3

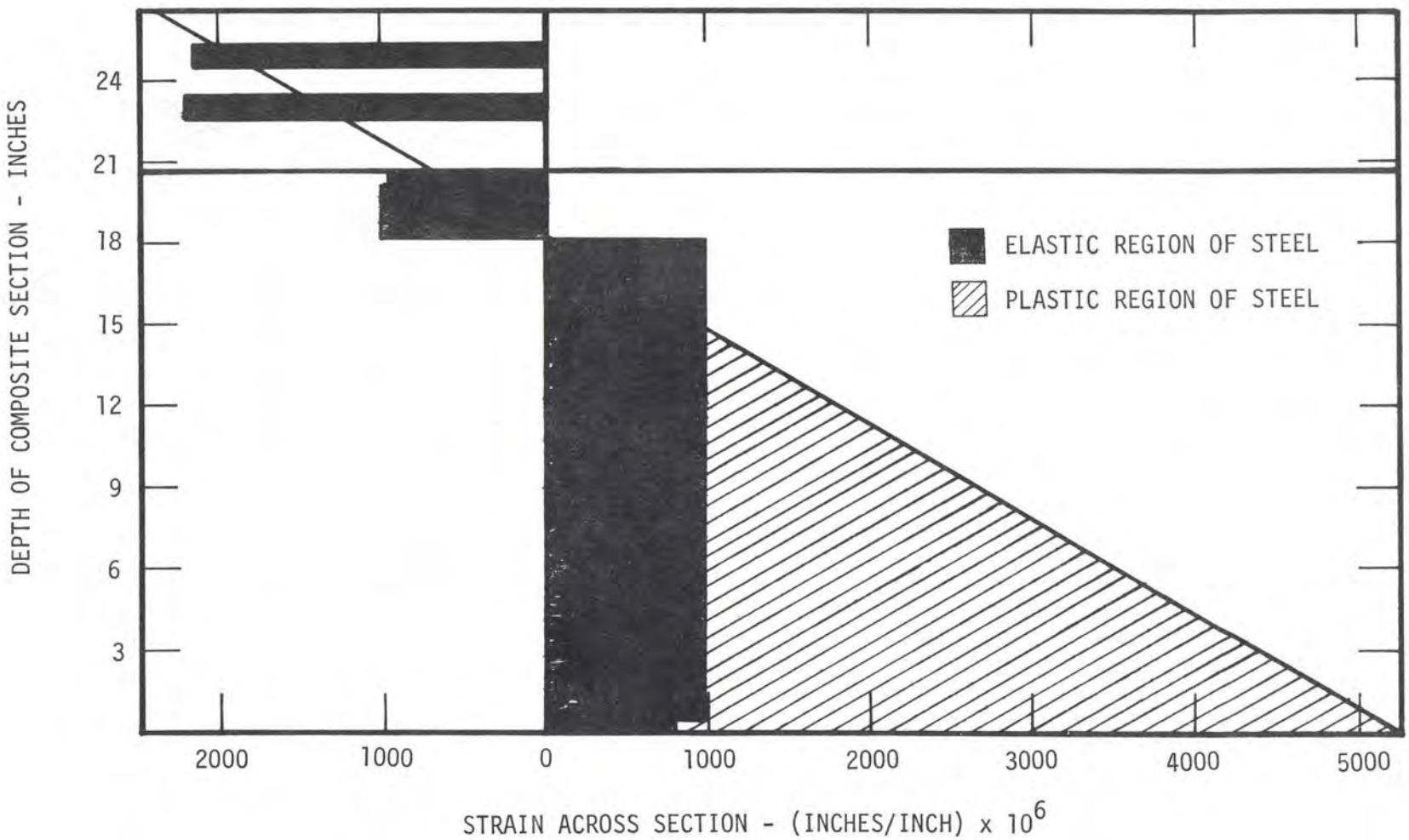


Figure 39 Critical Section at Ultimate - NCB6B3

CHAPTER V

SUMMARY AND CONCLUSIONS

5.1 SUMMARY

Three continuous-composite-bolted members were investigated. Two of the members were designed according to AASHO specifications with only one of the two members having 3/4 inch-bolted-shear connectors over its negative moment region. The third member was designed in order to yield the steel beam completely in compression.

Load-deflection curves, strain profiles, and slip distribution curves were used in investigating the composite behavior of the members. Predicted loads were compared with the actual loads obtained. The members with and without connectors in the negative moment regions were compared. Only 67% of the steel beam of the third composite member was successfully yielded in compression, but the predicted ultimate moment was exceeded.

Six tension-pushout specimens were investigated. Load-slip curves were investigated and also compared with those obtained from the composite members.

5.2 CONCLUSIONS

The following conclusions were made from this investigation:

1. Because of the nature of the load-slip per bolt data for the composite members, a conclusion for the validity of using tension pushouts to predict connector behavior in a member can not be justified.
2. For those members with connectors over their negative moment regions, there was practically no slip at their working loads, and very little slip

(less than .01 inches) even at their yielding loads.

3. Elementary beam theory (assuming complete composite action) can be used to satisfactorily predict the behavior at the working loads in the negative moment region of members with connectors over the negative moment regions.
4. Simple ultimate strength theory can be used to satisfactorily predict the ultimate capacity in the negative moment region of members with connectors over the negative moment regions.
5. The ultimate moment capacity of the member without connectors over the negative moment region exceeded the predicted ultimate capacity of the member with connectors over the negative moment region.
6. The elimination of connectors over the negative moment region of the composite member reduced the working and first yielding loads of that member by approximately 15%.

BIBLIOGRAPHY

1. Viest, I.M., Fountain, R.S., and Singleton, R.C., *Composite Construction in Steel and Concrete for Bridges and Buildings*, McGraw-Hill Book Company, Inc., New York, 1958.

2. Dallam, L. N., "Pushout Tests with High Strength Bolt Shear Connectors", Missouri Cooperative Highway Research Program Report 68-7, Engineering Experiment Station, University of Missouri - Columbia, 1968.

3. *Sandard Specification for Highway Bridges*, adopted by the American Association of State Highway Officials,, 1965.

4. Dallam, L. N., and Harpster, J. L., "Composite Beam Tests with High-Strength Bolt Shear Connectors", Missouri Cooperative Highway Research Program Report 68-3, Engineering Experiment Station, University of Missouri - Columbia, 1968.

5. *Manual of Steel Construction*, American Institute of Steel Construction, Inc., 1967.

6. "Specifications for Structural Joints Using ASTM A325 or A490 Bolts", American Institute of Steel Construction, Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation, New York, 1964.

7. Dallam, L.N., and Gaudini, P.B., "Study of Composite Bridge Stringers, Tension Pushout Tests with High-Strength-Pretensioned Bolts", Department of Civil Engineering, University of Missouri - Columbia, 1968.

8. The AASHO Road Test Report 2 Materials and Construction, HRB Special Report 6IB, Appendix E.

9. Winter, George, *Design of Concrete Structures*, McGraw-Hill Book Company, Inc., New YOrk, 1964.

10. Proctor, Melvin H., "Analytical and Experimental Study of Lightweight Concrete-Steel Composite Beams", M. S. Thesis, Department of Civil Engineering, University of Missouri - Columbia, 1963.

11. Baldwin, J. W., Henry, L. R., and Sweeney, G. M., "Study of Composite Bridge Stringers, Phase II", Technical Report for the State Highway Commission of Missouri and the U. S. Bureau of Public Roads, Engineering Experiment Station, University of Missouri - Columbia, May, 1965.

12. Slutter, R.G., and Driscoll, G.C., "Flexural Strength of Steel-Concrete Composite Beams", Journal of the Structural Division, American Society of Civil Engineers, Vol. 91 No. ST 2, April, 1965.

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