# PROOF LOAD TEST OF R01 OF 61131
# M-37 OVER CSX RAILROAD, SOUTH OF BAILEY, MICHIGAN

MICHIGAN DEPARTMENT OF TRANSPORTATION

MATERIALS and TECHNOLOGY DIVISION

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# M•DOT

# PROOF LOAD TEST OF R01 OF 61131
# M-37 OVER CSX RAILROAD, SOUTH OF BAILEY, MICHIGAN

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Research and Technology Section
Materials and Technology Division
Research Project 94 TI-1731
Research Report No. R-1336


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# ACTION PLAN

1. Engineering Operations Committee

   A. Approve Report

2. Maintenance Division

   A. Continue to monitor the bridge on a six month inspection schedule, with emphasis on the fascia beams, condition of the deck, shear cracks in the end span beams, and condition of the piers.

3. Design Division

   A. As agreed upon with the Bridge Management Unit in November 1994, the bridge is currently open to traffic with the 45 ton rating for the two-unit vehicle. Due to our spring 1995 inspection findings, which show the new concrete overlay is deteriorating rapidly and has little bond to the existing deck, we reanalyzed the structure using the following assumptions:

      1.) Because the bond between the concrete overlay and the deck is very poor, and the overlay is separating from the deck at the piers, the structure's beams were analyzed as simple spans.
      2.) Because the horizontal shear is approximately zero near center-span of span 2, and there is no uplift of the overlay in this region, 50 percent of the actual flange thickness; (i.e. overlay thickness plus existing deck thickness) was used as the resisting section.
      3.) Since we found only small corrosion loss to the reinforcing steel, 90 percent of its cross-section was used.
      4.) During the load test, we found the maximum distribution of load to any beam is approximately 26 percent. This value was used rather than the 31.7 percent AASHTO distribution factor.

   Using these assumptions, we find the 45 ton, two-unit vehicle posting is still adequate. The Design Division should review this report's findings and the above referenced analysis to determine if they concur with the current bridge posting.

4. District 5

   A. If the structure remains in service for more than five years, the Structural Research Unit recommends that the department repair the two fascia beams and the interior beams that are in poor condition, and retest the structure to detect further deterioration and consequent loss of load carrying capacity.

## EXECUTIVE SUMMARY

In October 1994, the Bridge Management Unit requested a load test of the 68-year-old bridge carrying M-37 over CSX Railroad, 1.1 miles south of Bailey (structure number R01 of 61131). At that time, a contractor was replacing part of the deck on the reinforced concrete T-beam bridge. The bridge had a load restriction of 45 tons for a Michigan type, two-unit, 77-ton vehicle, but due to extensive deterioration, the Bridge Management Unit of Bridge Design was considering reducing the load carrying capacity to 34 tons. This action, however, would have presented a hardship to local manufacturers who use this route.

The Bridge Management Unit requested a load test of the bridge to examine the assumptions they used during analysis of the structure. The Structural Research Unit, after reviewing the feasibility of various types of load tests, decided to "proof load test" the bridge to determine its response and capacity. This type of test "proves" the ability of the structure to carry its full dead load plus a magnified live load that is larger than the bridge is expected to carry when in service. During this test, the Structural Research Unit monitored the structure for any signs of distress or overloading.

Before rehabilitation, the bridge was a three-span, simply supported structure composed of nine reinforced concrete beam stems composite with the reinforced concrete deck with a bituminous overlay. During the rehabilitation, the contractor removed the bituminous overlay, concrete wearing course, and the top 2½ inches of the existing deck. The top 2½ inches of the existing deck were then replaced and a new 5½ inch reinforced concrete wearing course, which extends through the expansion joints, was added.

The Structural Research Unit inspected the bridge before the load test, which involved rating each beam according to the Michigan Structure Inventory Appraisal Coding Guide. We found the fascia beams were in serious condition and the interior beams were in either poor or fair condition.

On November 8 and 9, 1994, we load tested the center span of the bridge. The structure successfully carried a proof load equivalent to an 82-ton two-unit vehicle, thus permitting a 45-ton two-unit vehicle posting. We also found that the bridge has significantly greater stiffness, (i.e., greater load carrying capacity) than estimated by the Bridge Management Unit. In order to determine the cause of this greater stiffness, we performed a post test analysis, and have concluded the following:

1. During the load test, the new reinforced concrete deck acted fully composite with the existing section.

2. Loss of reinforcing steel due to corrosion was less than estimated.

3. Reinforcing steel in the new wearing course extends through the expansion joints, thus during the load test the structure acted continuous rather than simply supported.

The south fascia beam (beam 9) showed less stiffness when compared to the north fascia beam (beam 1). As we removed the test load, both the south and north fascia beams did not return to their initial elevations, indicating that the fascia beams, particularly beam 9, were loaded beyond their yield point. Furthermore, it is likely that there is a loss in load carrying capacity of these beams due to deterioration. The bridge, having multiple beams and thus redundant load paths, is redistributing the live load from the weakened fascia beams to the interior beams.

During the load test, the structure showed considerably greater load carrying capacity than the Bridge Management Unit predicted. Because the bridge successfully carried the 82-ton proof load with no detectable distress during the test, we recommended to the Bridge Management Unit that the existing posted loading of 45 tons for a two-unit, 11 axle vehicle be continued for this structure. However, we believed the posting should not exceed 45 tons, due to the condition of the fascia beams. The Bridge Management Unit concurred with our recommendation, and the bridge was opened to traffic in November 1994.

In May 1995, the Structural Research Unit inspected the bridge deck, and used cores removed from the deck to perform a "pull-off bond test" to determine the bond strength between the new concrete overlay and the existing deck. This test was used to verify that the freeze/thaw action experienced during the winter of 1995 had not deteriorated the composite action between the overlay and the existing deck. During the inspection, we found extensive cracking in the concrete overlay where the average width of the cracks were .02 inches to .025 inches. When removing cores for the pull-off bond test each core fractured at the concrete overlay/existing deck interface. We inspected the core holes and found loose concrete debris at the bottom of the holes. Our inspection also found that there is a void between the overlay and existing deck. Based on this information, it is clearly evident that there is little bond between the new concrete overlay and the existing deck. The cause of the cracks in the overlay is the poor condition of the existing deck, which is providing a poor bond, and the reinforcing steel that extends through the expansion joints, which is causing the concrete overlay to separate from the existing deck at the piers.

Although the bridge demonstrated composite action of the new concrete overlay during the load test, our May 1995 inspection showed that this composite action may be unreliable as time passes because the overlay appears to be separating from the existing deck. Since this can change the load carrying capacity of the structure, we re-analyzed the structure to see if the 45 ton posting for the two-unit vehicle was still safe. To determine how the structure will respond to load in the future, we needed to determine why the concrete overlay, which is made of good strong concrete and reinforcing steel, is deteriorating so quickly, and why it is separating from the existing deck. We feel when the bridge receives live load, negative moment created by the overlay's reinforcing steel that extends through the expansion joint at each pier causes uplift of the overlay, causing it to separate from the existing deck. Large moments are developed in the overlay, thus causing it to crack. In the end-spans near abutment A and abutment B, and in the center portion of span 2, there is no uplift force; thus less cause for the overlay to separate from the existing deck.

The Bridge Management Unit found the weak component of the structure was mid-span of span 2 where they estimated the concrete in the deck, 3 in. estimate flange thickness, would fail in compression. With the addition of the reinforced concrete overlay, the actual flange thickness is 11 in. At the center one-third portion of span 2, we estimate the maximum live load horizontal shear stress at the bond interface is near zero. We also showed there is no uplift forces in the overlay in this region. Therefore, we feel the overlay can be considered partially composite with the existing deck at this location (i.e. center span of span 2).

We used the following assumptions for our May 1995 post-inspection analysis:

1.) The structure reacts as simple spans.
2.) At center-span of span 2, 50 percent of the actual flange thickness; (i.e. overlay thickness plus existing deck thickness) can be used as the resisting section.
3.) Since we found only small corrosion loss to the reinforcing steel, 90 percent of its cross-section can be used.
4.) During the load test, we found the maximum distribution of load to any beam is approximately 26 percent. This value can be used rather than the 31.7 percent AASHTO distribution factor.

Using these assumptions, we found the 45 ton, two-unit vehicle posting is still adequate.

The Design Division should review this report's findings and the above referenced analysis to determine if they concur with the current bridge posting.

The Maintenance Division should continue to monitor the structure on a six month inspection schedule, with emphasis on the fascia beams, condition of the deck, shear cracks in the end spans, and condition of the piers.

If the structure remains in service for more than five years we recommend that the department repair the two fascia beams and the interior beams that are in poor condition, and retest the structure to detect further deterioration and consequent loss of load capacity.

## INTRODUCTION

On October 27, 1994, the Bridge Management Unit requested a load test of the 68-year-old bridge carrying M-37 over the CSX railroad, 1.1 miles south of Bailey (structure number R01 of 61131). The structure, a three-span, reinforced concrete T-beam bridge, built in 1927 (see Figure 1), was in poor condition, as the 1994 bridge inspection report shows (see Figure 2). The "Structure Inventory and Appraisal" report (not shown) gave the bridge an overall appraisal rating of 2, which by definition states, "basically intolerable, requiring priority of replacement." The deck, stringers (beam stems), piers, and slope protection all received serious ratings. The bridge had a load restriction of 29 tons for a Michigan type one-unit vehicle, and a load restriction of 45 tons for a Michigan type two or three-unit vehicle. Figure 3 details these vehicle configurations.

Due to the extensive deterioration of the bridge, the Bridge Management Unit of Bridge Design was considering reducing the load rating from 45 to 34 tons for the two-unit vehicle. But a reduction in the load rating would have presented a hardship to local manufacturers, specifically the Gerber Corporation, whose trucks use this route for carrying their goods. The Bridge Management Unit based its analysis on numerous conservative assumptions about the extent of the structure's deterioration and how loads distribute through the structure. A detailed inspection, measuring actual material strengths, and a load test, measuring the structures response to a given load, would provide answers to many questions and demonstrate the bridge's actual response to a heavy load.

At the time of the request for the load test, the contractor had the bridge closed for rehabilitative construction, during which the existing bituminous overlay, the concrete wearing course, and 2½ inches of the existing deck were being removed. The contractor then replaced the 2½ inches of existing deck and added a new 5½ inch (minimum) reinforced concrete wearing course (see Figures 4 and 5 for existing and proposed section). The designers of the rehabilitation extended the steel reinforcement though the expansion joints making the bridge spans continuous rather than simply supported.

The department's district office stipulated that the bridge must be open to traffic by November 10, 1994. Due to the difficulty of obtaining a future detour, the Structural Research Unit agreed to conduct the load test immediately after completion of the rehabilitative work and to open the structure to traffic by November 10, 1994.

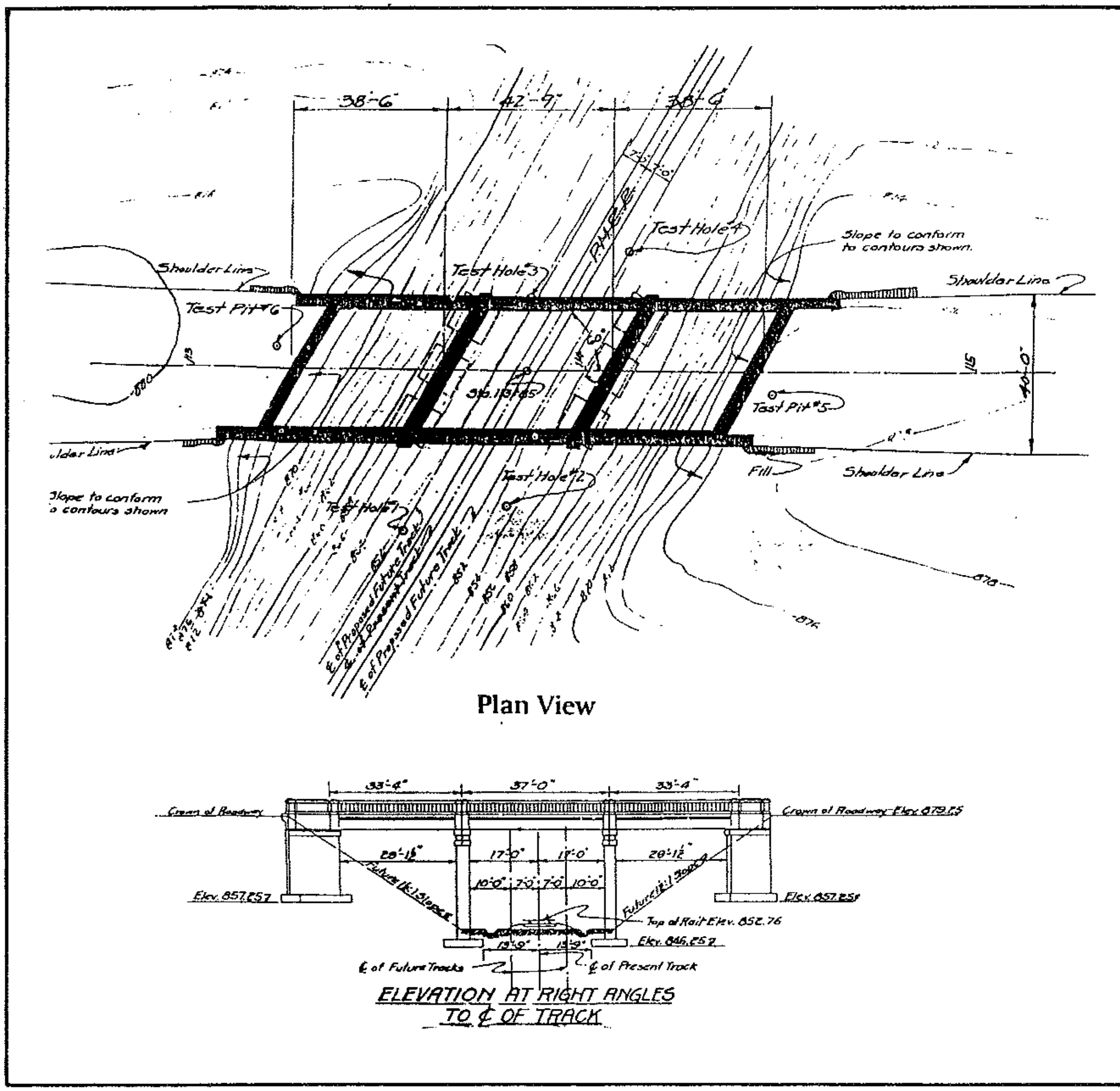



Figure 1: General Detail of Structure.

MICHIGAN DEPARTMENT OF TRANSPORTATION

# BRIDGE INSPECTION REPORT

P2502 (9/89)

| | | | | |
|---|---|---|---|---|
| DATE INSPECTED: | 6-11-93 | 11-30-93 | 4-22-94 | |
| INSPECTED BY: | AD | KT | AD | |

'IDGE NO. R01-61131 ROUTE M-37 OVER C&O RR
.JCATION 1.1 MI S OF BAILEY
DESCRIPTION: SPANS 3 = 120 RDWY. WIDTH 30.0 - 2
TYPE CONC T-BM DESIGN LOAD H-15

COUNTY MUSKEGON
DISTRICT 5
BUILT 1927
FOUNDATION:

1. REPAIRS MADE
2. ADDITIONAL INSPECTION EQUIPMENT
3. CRITICAL INSPECTION FEATURE posted 1U-29T, 2&3U-45T
4. PAINT CLASS: — YEAR/COLOR — 4-94 SIGNS IN PLACE
5. POSTING:

9 - - - - NEW
7-8 - - - - GOOD
5-6 - - - - FAIR
4 - - - - POOR
3 - - - - SERIOUS
2 OR LESS - CRITICAL

| UNIT | | RATING 93 | 93 | 94 | EXPLANATION OF CONDITIONS |
|---|---|---|---|---|---|
| | | | | | MATERIAL: SURFACE Bit DECK Conc APPROACH Bit. EXP. JOINT TYPE Cov'd MIN. OPENING — " — " |
| SUPERSTRUCTURE | 1. SURFACE YR. OVERLAY | 4 | 4 | 4 | |
| | 2. DECK | 3 | 3 | 3 | 1. Rough & uneven w/ some depressed areas. Many bit. patched & spalled areas. Heavy accumulation of dirt on deck. Spalled fascia's HVY SPALLS @ N SHOULDER |
| | 3. EXPANSION JOINTS | — | — | — | |
| | 4. OTHER JOINTS | 4 | 4 | 4 | 2. Heavy leaking thru deck w/ up to 6" stalactites. Several spalls to re-steel w/ some broken strands. |
| | 5. SIDEWALK & CURBS | 4 | 4 | 4 | |
| | 6. RAILINGS | 5 | 5 | 5 | 3. Ravelling @ JTS. Heavy leaking & spall along 4. bottom |
| | 7. UTILITIES | — | — | — | 5. Bit. to top of curb. |
| | 8. BEARING DEVICES | — | — | — | 6. Conc Parapet Railings (Low-22") Much leaching & spall to resteel on posts |
| | 9. DRAINAGE SYSTEM # | 4 | 4 | 4 | 9. No Extensions - Wood forms under some. MANY COVERED W/ DIRT |
| | 10. STRINGERS P.&H. # | 3 | 3 | 3 | 10. Severe spall to re-steel w/ some rusted thru. Many cracks & additional incip. spalls. Stalactites up to 8" long. Some beams spalled entire length of span. |
| | 11. PAINT | — | — | — | |
| | 12. SECTION LOSS | — | — | — | |
| SUB-STRUCTURE | 13. ABUTMENTS | 4 | 4 | 4 | 13. Scale in several locations w/ heavy leach stain & build up of leach material |
| | 14. PIERS | 3 | 3 | 3 | 14. Spall & scale to resteel w/ many stalactites up to 6" long. Heavy map cracking & corner crack |
| | 15. SLOPE PROTECTION | 3 | 3 | 3 | |
| APPROACHES | 16. PAVEMENT | 5 | 5 | 5 | 15. No - Washing out, SOME STONE FILL |
| | 17. SHOULDERS SIDEWALKS | 6 | 6 | 6 | 16. Cracked & sealed. |
| | 18. SLOPES | 6 | 6 | 6 | 19. Type B, Some Damage (12'6" post spacing) 5=90,11 |
| | 19. GUARD RAIL | 4 | 4 | 4 | DIRT ACCUMULATION TO BOTTOM OF RAIL |
| HYDRAULICS | 20. UNDERWATER INSP. (DESCRIBE) | — | — | — | SUPERSTRUCTURE HAS SHIFTED ALONG SKEW W END TO S, E END TO N ≈3/4 11-13 over 1"-E 3/4"-W |
| | 21. CHANNEL PROTECT. #61 | — | — | — | RECOMMENDATIONS: |
| | 22. CULVERT (OVER 20') #62 | — | — | — | Remove Structure or Replace on New Alignment. |
| | SI & A # 57 | 2 | 2 | | |
| | #58 ( ) | 3 | 3 | 3 | Programmed For 1992 (DELAYED) |
| | #59 ( ) | 3 | 3 | 3 | CLEAN DIRT OFF DECK & @ APPROACH GUARD RAILS |
| | #60 ( ) | 3 | 3 | 3 | |

Figure 2: Bridge Inspection Report

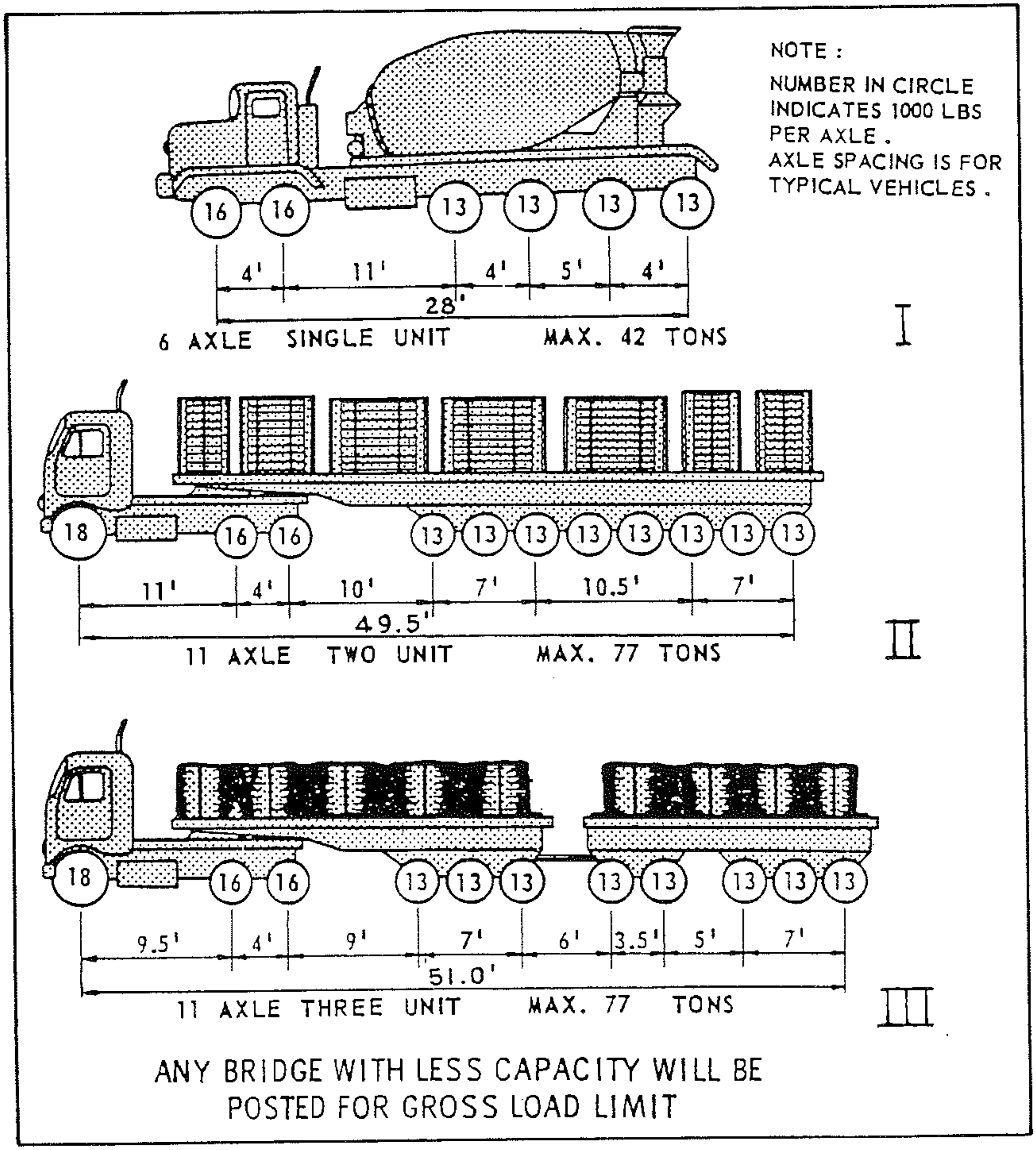



Figure 3: Truck Configurations used for Analysis.

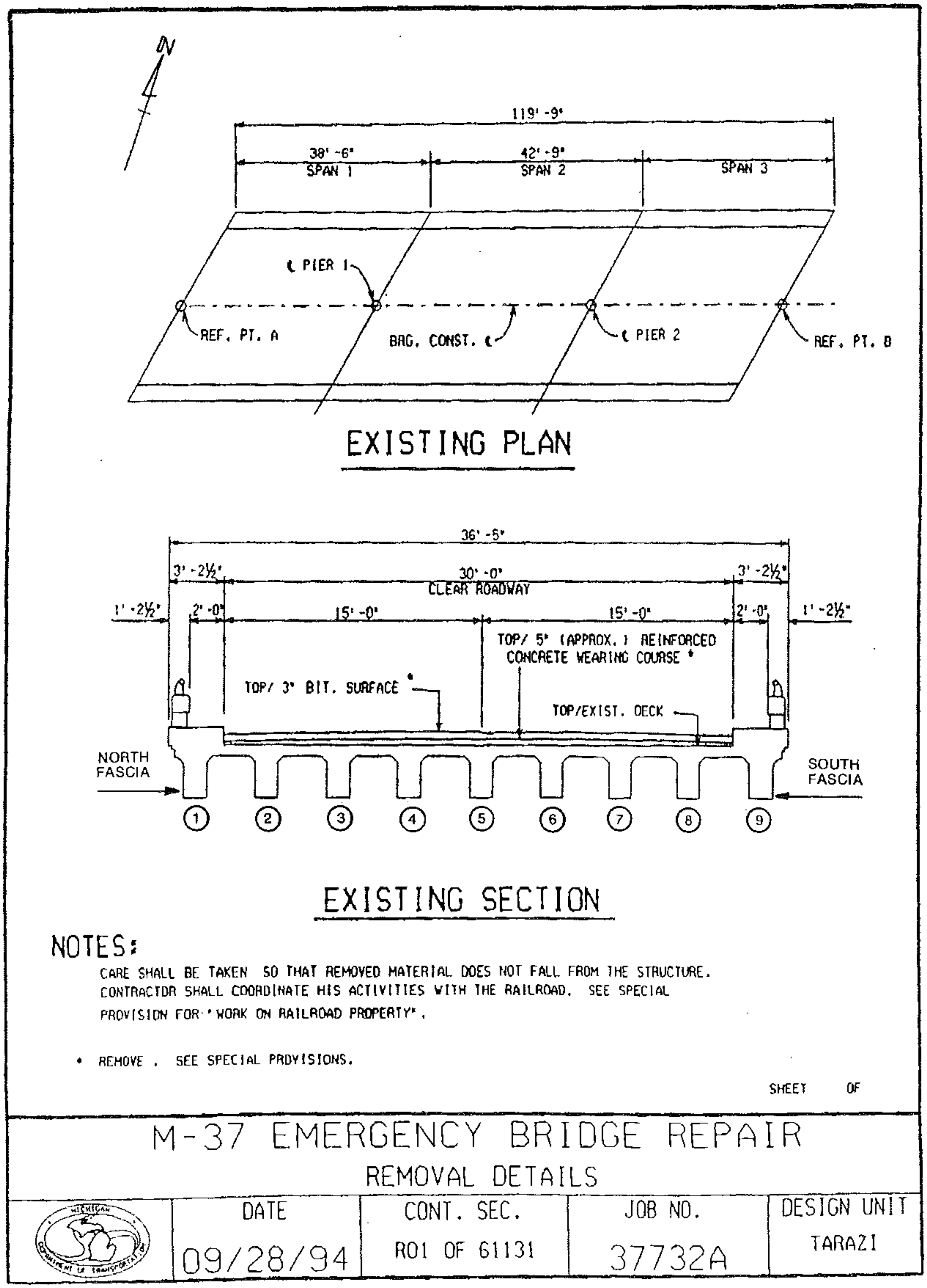
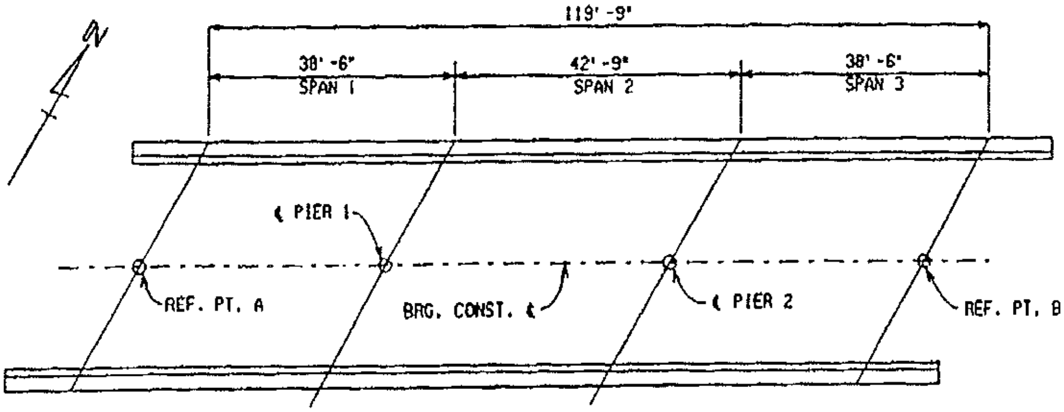
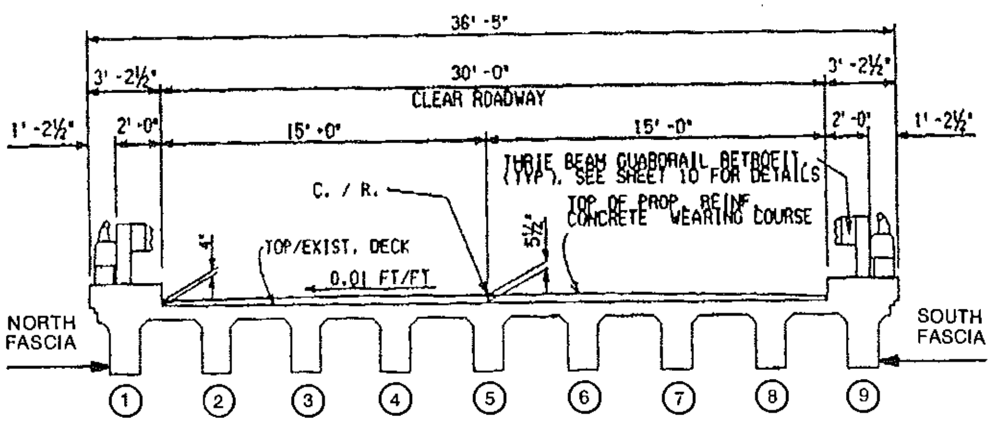



Figure 4: Existing Section




## PROPOSED PLAN

CENTER SPAN DECK DRAINS TO BE PLUGGED WITH SUPERSTRUCTURE CONCRETE.
SAW CUT ¾" DEEP AND SEAL OVER PIERS. NOT PAID FOR SEPARATELY BUT INCLUDED IN SUPERSTRUCTURE CONCRETE.




NOTES:

## PROPOSED SECTION

AREA REMOVED FULL DEPTH SHALL BE FORMED PAID FOR AS FORMING FOR PATCHES AND FILLED WITH SUPERSTRUCTURE CONCRETE.

SUPERSTRUCTURE CONCRETE SHALL BE A 9 SACK PER CUBIC YARD MIX.

SUPERSTRUCTURE CONCRETE SHALL BE PROTECTED AGAINST COLD WEATHER IN ACCORDANCE WITH SECTION 5.03.12 OF THE STANDARD SPECIFICATIONS, EXCEPT THE COST WILL NOT BE PAID FOR SEPARATELY, BUT INCLUDED IN THE PAY ITEM "SUPERSTRUCTURE CONCRETE". M-37 TRAFFIC SHALL BE DETOURED OVER EXISTING ROADS.

STEEL REINFORCEMENT, EPOXY COATED - 2" MIN. COVER. TRANSVERSE - 238 - EA043406 SPA. @ 6",
LONGITUDINAL - 30 - EA043009 SPA @ 1' - 0", MIN. LAP 1' -4"

EXISTING BRIDGE PLANS ARE AVAILABLE IF REQUESTED.

SHEET OF

# M-37 EMERGENCY BRIDGE REPAIR

## PROPOSED BRIDGE CONSTRUCTION DETAILS

| | DATE | CONT. SEC. | JOB NO. | DESIGN UNIT |
|---|---|---|---|---|
| | 09/28/94 | R01 OF 61131 | 37732A | TARAZI |

Figure 5: Proposed Section

## TEST PROCEDURE

The Structural Research Unit tested the bridge by applying a "proof load." A proof load is an estimated load based on the live load the bridge is expected to carry, plus an additional load to account for infrequent overloads and impact. We followed the guidelines in NCHRP Project 12-28(13)A, "Bridge Rating Through Nondestructive Load Testing" for this test. Basically, this test involves placing increments of load on the structure and measuring its response. The structure was continually loaded until distress was indicated or until the estimated "proof load" was achieved. We plotted load versus deflection diagrams for each increment of load placed on the structure. These deflections were monitored for any sign of non-linear displacement that would show that the elastic limit of the beam had been surpassed. Using these diagrams, we also compared actual stiffness of the beams to predicted stiffness. Stiffness, a function of the moment of inertia of the beam, the modulus of elasticity of the materials, composite action between the beam and slab, and continuity between the spans, can be directly related to strength.

The Structural Research Unit established two target proof loads for this structure. The first target, a proof load of 62 tons, was based on the 34 ton, two-unit vehicle capacity, which is the reduced load rating the Bridge Management Unit was considering. The second proof load was 82 tons, based on the 45 ton, two-unit vehicle capacity, which corresponded to the existing load rating on the structure before the rehabilitative construction. The requests from Bridge Design were to, first, target the proof load for the 34-ton rating and if the bridge exhibited signs of greater strength with no distress, target the proof load for the 45-ton load rating.

### Pre-Test Analysis

To ensure safety, we conducted a pre-test analysis of the structure, which included a shear and moment analysis of the beams and a check of the load capacity of the piers. We compared this to the resisting section that the Bridge Management Unit estimated for the interior beams for applied moment (see Figure 6). Due to the poor condition of the existing deck, the Bridge Management Unit used only three inches of the slab for the resisting section in their analysis. Also, because the bottom row of reinforcement was exposed and highly corroded, they used only 50 percent of its area for the resisting section. Their estimate for yield strength of the steel was 33,000 psi, and their estimate for concrete strength was 2,500 psi. This resisting section resulted in a gross moment of inertia equal to 63,400 $in.^4$ for the interior beams. Since the bending moment at the middle of the center span controlled the load carrying capacity of the structure, this span was chosen for the load test. Due to time constraints, we only tested the center span's response to load. However, during the test procedure, we drove a 59 ton slow-moving truck load across the north end-span, thus proving its ability to carry that given load.

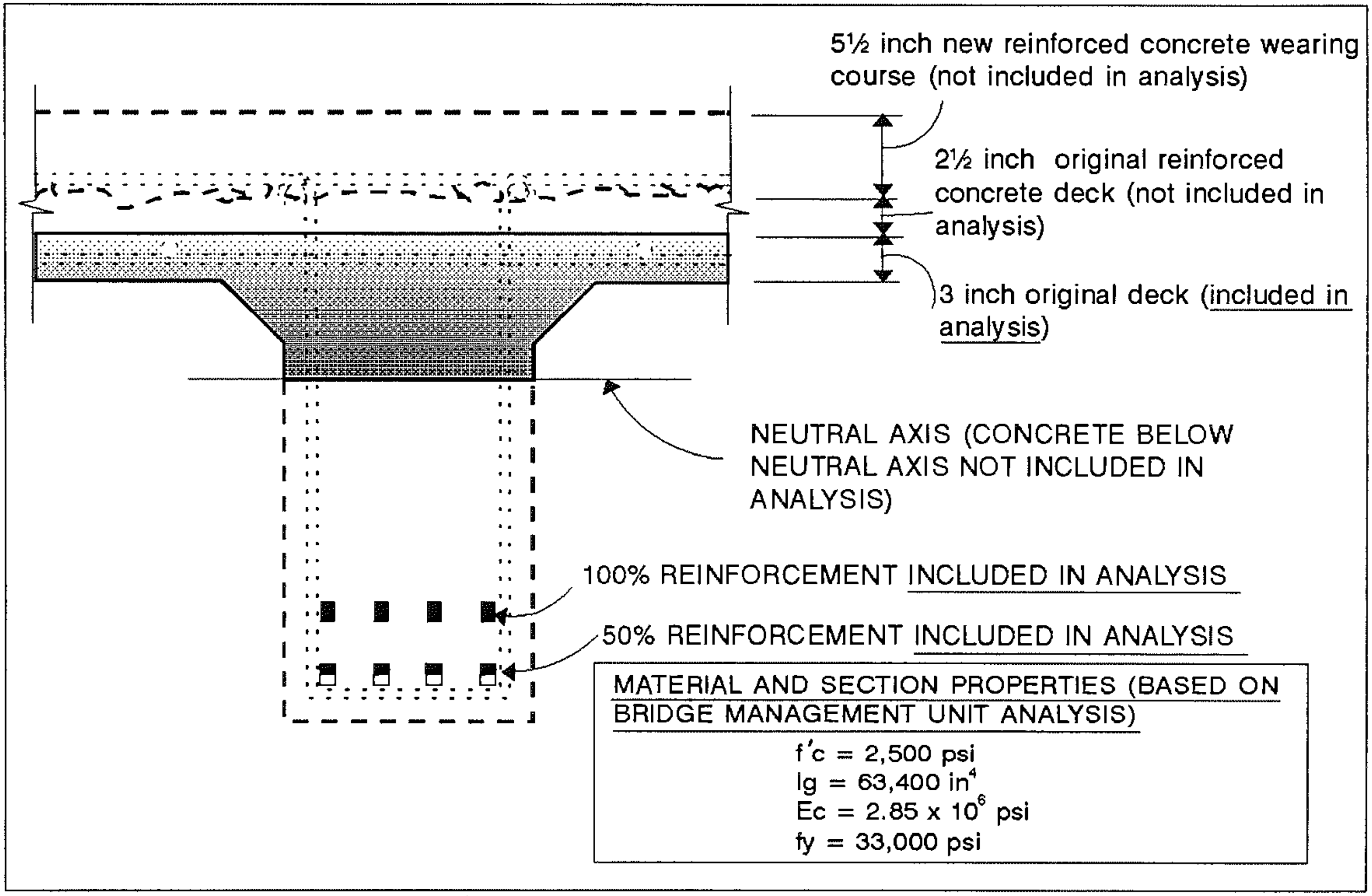



**Figure 6 - PRE-TEST ESTIMATED SECTION FOR INTERIOR BEAMS**

## Initial Bridge Inspection - Monday, November 7, 1994

Before load testing, the Structural Research Unit conducted an inspection of the bridge, which included a general inspection of the structure and a detailed inspection of the beam stems of the center span. We measured the width of several vertical cracks near the midspan of the fascia beams. We also removed rust from the exposed reinforcing steel and measured the width of the remaining steel. Finally, the overall condition of each beam was noted. All initial inspection information appears in Table 1. Figures 7 through 10 show the condition of the beams in the center span of the structure. Note that the bottom row of reinforcement is fully exposed in the south fascia beam (beam 9). When tapped with a chipping hammer, the reinforcing bars moved without restraint from the concrete, a clear indication that this beam was the weakest load carrying component of the structure.

| | *Pre-Test Inspection* | | *Post-Test Inspection* | |
|---|---|---|---|---|
| **Beam Number** | **Reinforcement Width (inch)** | **Crack Width (mm.) *(see below for coding.)** | **Crack Width (mm) *(see below for coding.)** | **General Rating and Comments** |
| 1 (North Fascia) | 1.201<br>1.239 | | | Serious Condition.<br>Spalling full length of beam, exposing reinforcement in areas. |
| 2 | | .08 (s) | .08 (s) | Fair Condition. |
| 3 | | .10 (n) | .10 (n) | Poor Condition.<br>8 foot long spall exposing bottom row of reinforcement. |
| 4 | | .05 (s) | .05 (s) | Fair Condition. |
| 5 | | .10 (n) | .10 (n) | Poor Condition.<br>Large spall at midspan of beam exposing bottom row of reinforcement. |
| 6 | | .20 (s) | .20 (s) | Fair Condition. |
| 7 | | .30 (s) | .30 (s) | Fair Condition. |
| 8 | | .20 (s) | .20 (s) | Fair Condition. |
| 9 (South Fascia) | 1.240<br>1.167<br>1.208 | .30 (s)<br>.30 (s)<br>.20 (n)<br>.20 (n) | .30 (s)<br>.30 (s)<br>.20 (n)<br>.20 (n) | Serious Condition.<br>Concrete spalling full length of beam.<br>Reinforcement is exposed for full length of beam. |
| Inspected by: DAJ on 11/7/94 and 11/9/94.<br>* s = south vertical face of beam stem.<br>n = north vertical face of beam stem. | | | | |

***Table 1, Pre-Test and Post-Test Inspections***

## Material Testing

We removed a sample of the steel reinforcement from one of the bridge beams and analyzed it in the department's laboratory. From this sample, three tensile specimens were prepared and tested to determine yield and ultimate strength. These results are in Table 2, which show the reinforcing steel meets the strength requirement of ASTM A15 structural steel, commonly used for steel reinforcement during the era that this bridge was built.

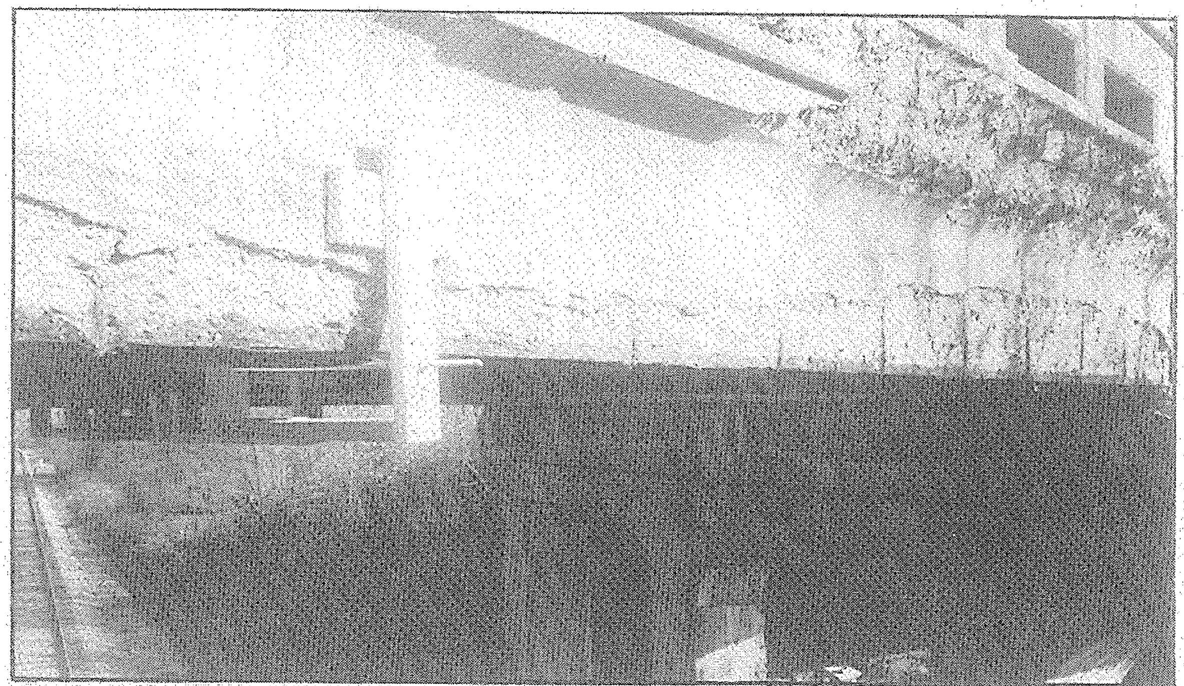


Figure 7: South Fascia Beam (Beam 9) Exposed Reinforcement Full Length of Span 2.

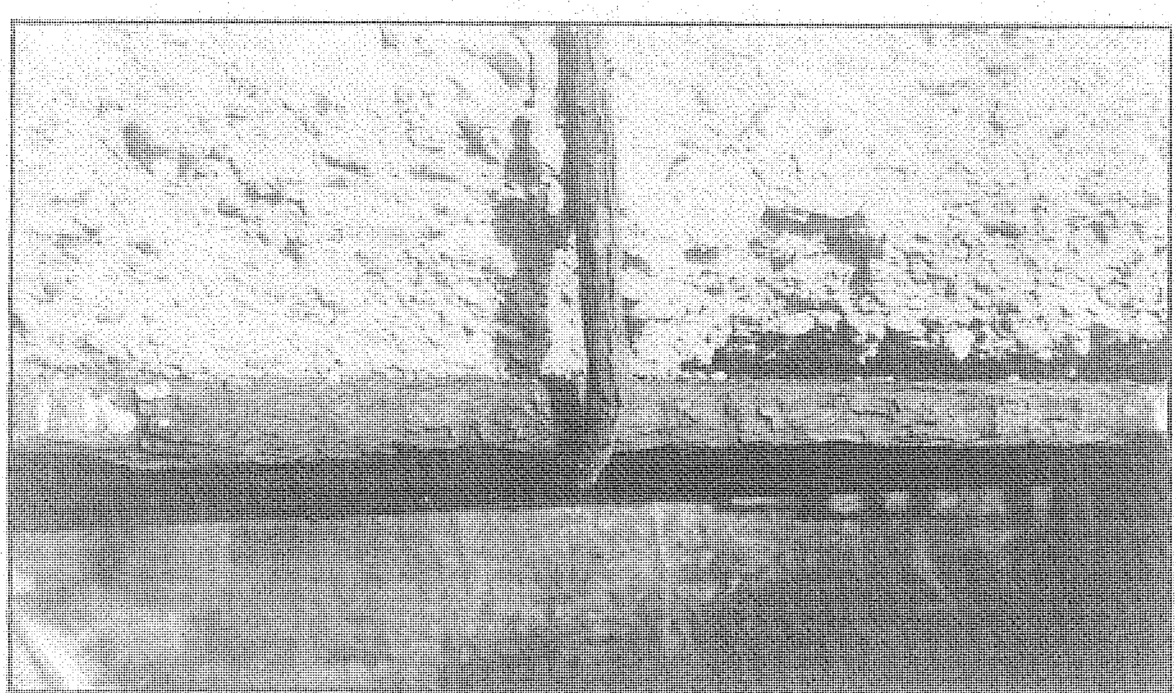


Figure 8: Exposed 1-1/4" Square Reinforcing Steel. Beam 9, Span 2.

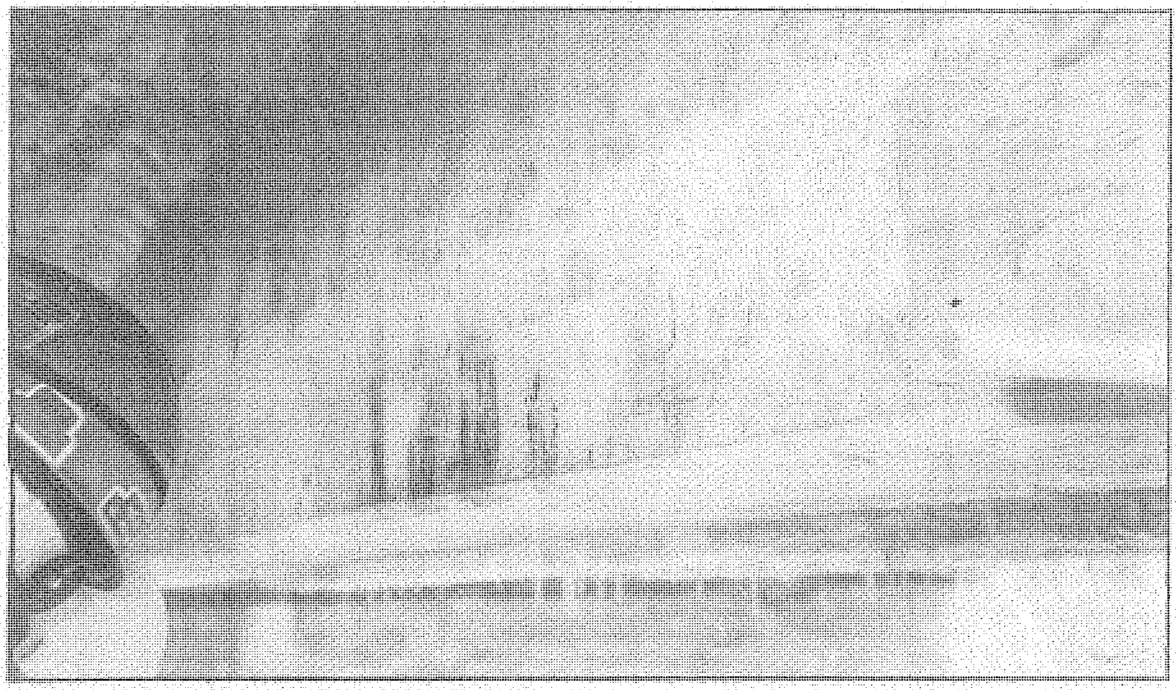


Figure 9: Interior Beam, Span 2, in Good Condition.

Figure 10: Beam 3, Span 2. Large Spall, Exposing Steel Reinforcement at Bottom of Beam.

| Specimen I.D. | 1 | 2 | 3 | |
|---|---|---|---|---|
| Yield Strength (psi.) | 37300 | 58400 | 31300 | |
| Ultimate Strength (psi.) | 93000 | 94000 | 94000 | |
| Elongation | 28% | 29% | 27% | |
| Reduction of Area | 44% | 44% | 45% | |

**Comments**

Average Yield Strength for Samples = 42,000 psi.
Standard deviation for yield strength of 3 samples = 14,200 psi. Note - This is high due to value of sample 2.
For analysis purposes, use Yield Strength = 33,000 psi..

**Table 2 - Material Properties of Reinforcing Steel.**

The construction field office prepared concrete cylinders from the new reinforced concrete wearing course to determine its strength during the load test (see Table 3 for results) and measure the modulus of elasticity of the new wearing course for use during the post analysis deflection estimates (see Table 5 for results). They also prepared test beams from the wearing course to determine the modulus of rupture (see Table 4 for results), which is commonly used to determine when concrete has reached a desired strength.

| Specimen I.D. | 2-1 | 2-4 | 2-7 | 2-10 | |
|---|---|---|---|---|---|
| Date Tested | 11-7-94 | 11-7-94 | 11-7-94 | 11-7-94 | |
| Age, Days | 12 | 12 | 12 | 12 | |
| Strength (psi.) | 5164 | 4893 | 5694 | 5942 | |
| Average Strength (psi.) | | | | | 5400 |

**Table 3 - Concrete Wearing Course.**
**Compression Strength (6" x 12" Cylinders).**

| Specimen I.D. | 2A | 2D | 2I | |
|---|---|---|---|---|
| Date Tested | 11-7-94 | 11-7-94 | 11-7-94 | |
| Age, Days | 12 | 12 | 12 | |
| Modulus of Rupture (psi.) | 883 | 920 | 896 | |
| Average Modulus of Rupture (psi.) | | | | 900 |

**Table 4 - Concrete Wearing Course.
Modulus of Rupture (6" x 6" Beams).**

| Specimen I.D. | 2-11 | 2-8 | 2-5 | |
|---|---|---|---|---|
| Date Tested | 11-8-94 | 11-8-94 | 11-8-94 | |
| Age, Days | 13 | 13 | 13 | |
| Modulus of Elasticity (psi.) | 5.10E+06 | 4.50E+06 | 5.05E+06 | |
| Average | | | | |
| Modulus of Elasticity (psi.) | | | | 5.00E+06 |

**Table 5 - Concrete Wearing Course.
Compression Modulus of Elasticity (6" x 12" Cylinders).**

## Test Load and Weighing Procedure

It is important that the load configuration of the test vehicle is similar to the vehicle used in posting the bridge for load rating. Load distribution to the structure varies with spacing of the axles, tire pressure, pressure in the air-suspension system, width of the vehicle, and position of the load on the vehicle's trailer. For this load test, we used a two-unit, 11 axle, 77 ton (gross) vehicle with a flat-bed trailer (see Figure 11 for vehicle configuration). Additional loads were applied to the trailer with temporary concrete barrier.

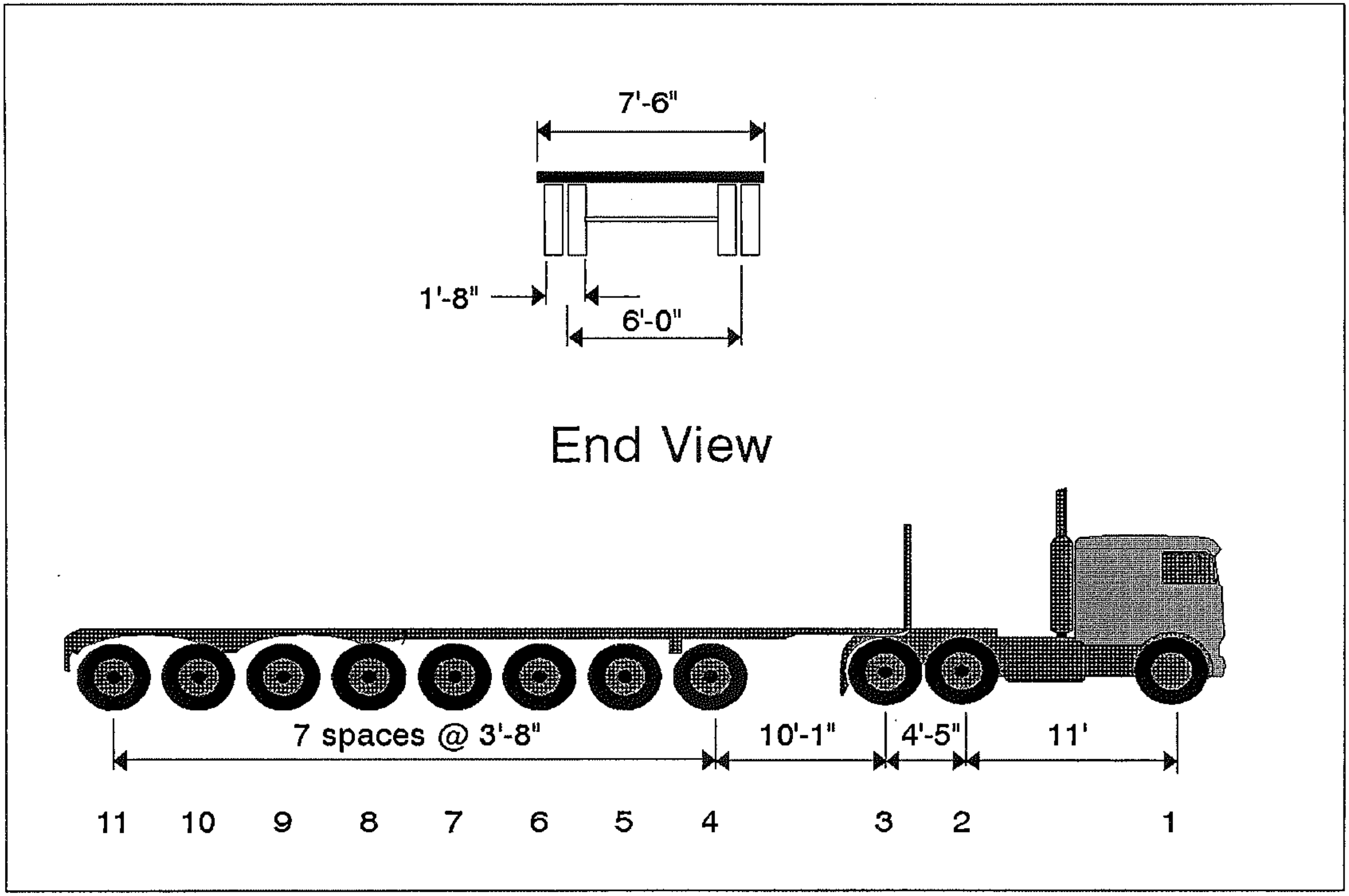



Figure 11 - TRUCK CONFIGURATION

The barriers were weighed using portable scales (see Figures 12 and 13) provided by the Michigan Department of State Police.

Knowing both the accuracy of the load and the distribution of the load to the structure is important. We used two methods to provide an independent check of the weighing procedure. For the first method, we weighed each axle of the empty vehicle using the portable scales (see Table 6 for portable scale axle weights), while for the second method, we weighed each axle of the empty vehicle at a scale located at Michigan Agricultural Commodities (MAC), Inc. in Newaygo, Michigan (see Table 7 for MAC axle weights). A comparison of the axle weights and the gross vehicle weight showed a discrepancy between the two weighing methods. Tables 6 and 7 show the portable scales registered the gross vehicle weight as 53,050 pounds, while the MAC scale measured 46,480 pounds.

**Empty truck**

Weighed 11-8-94 by DAJ,MCI

| Axle # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Side 1 | 4850 | 2650 | 2400 | 4450 | 4600 | 850 | 700 | 800 | 900 | 750 | 3450 |
| Side 2 | 5050 | 2450 | 2400 | 4600 | 5000 | 750 | 750 | 800 | 800 | 750 | 3300 |
| Axle weight | 9900 | 5100 | 4800 | 9050 | 9600 | 1600 | 1450 | 1600 | 1700 | 1500 | 6750 |

Total truck weight **53,050**

**Day 1 - Load Case 8**

Weighed 11-8-94 by MCI, NAL, CPD

| Axle # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Side 1 | 5800 | 4500 | 4150 | 5900 | 6300 | 4450 | 5400 | 6800 | 7800 | 10000 | 6350 |
| Side 2 | 5400 | 4100 | 4050 | 6000 | 6550 | 5050 | 5850 | 8300 | 9500 | 10600 | 6150 |
| Axle weight | 11200 | 8600 | 8200 | 11900 | 12850 | 9500 | 11250 | 15100 | 17300 | 20600 | 12500 |

Total truck weight **139,000**

**Day 2 - Load Case 8**

Weighed 11-9-94 by MCI, NAL, CPD

| Axle # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Side 1 | 6300 | 5000 | 4500 | 6100 | 6700 | 7200 | 7600 | 8100 | 8000 | 9000 | 5750 |
| Side 2 | 5900 | 4750 | 4650 | 7100 | 7700 | 5700 | 6400 | 7800 | 7950 | 7900 | 6000 |
| Axle weight | 12200 | 9750 | 9150 | 13200 | 14400 | 12900 | 14000 | 15900 | 15950 | 16900 | 11750 |

Total truck weight **146,100**

**Table 6 - Truck axle weights (lbs.) from portable scales**

Last Scale Calibration 10/19/94 by the State Of Michigan.

Weighed 11/8/94 by DAJ.

| Axle # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Axle weight | 10480 | 5900 | 5520 | 6260 | 7460 | 660 | 1180 | 1420 | 1240 | 0 | 6360 |

Total truck weight **46,480**

**Table 7: Truck axle weights (lbs.), empty. From MAC Inc. Scales**

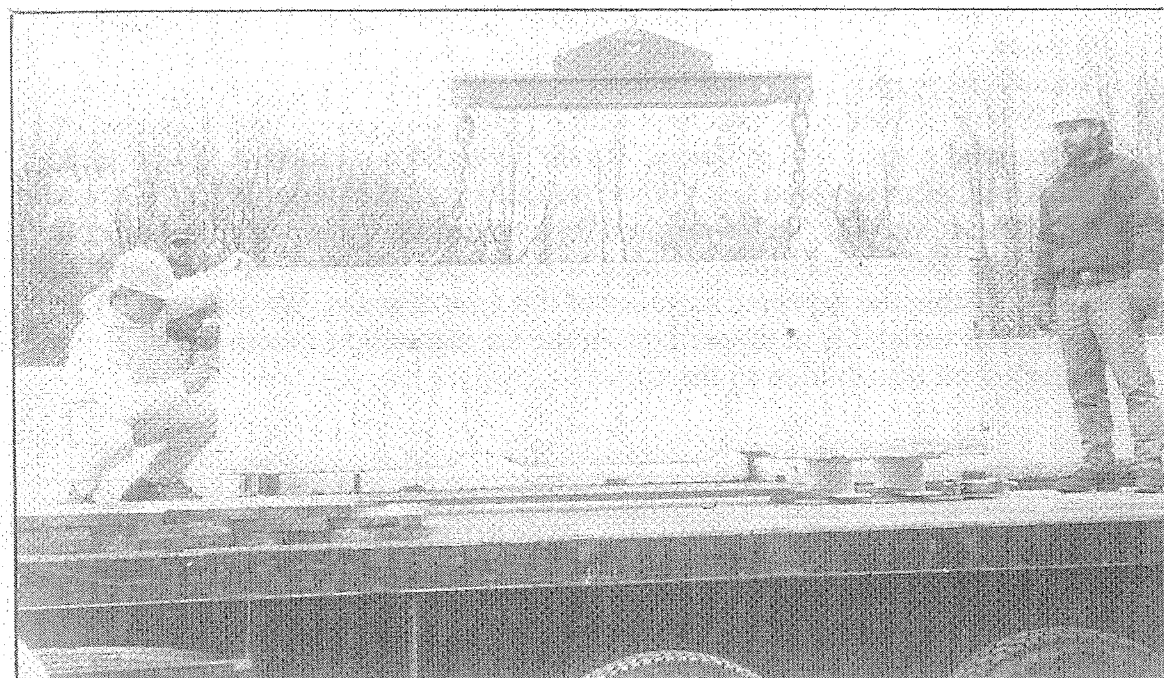


Figure 12: Structural Research Unit Personnel Weighing Temporary Barrier.

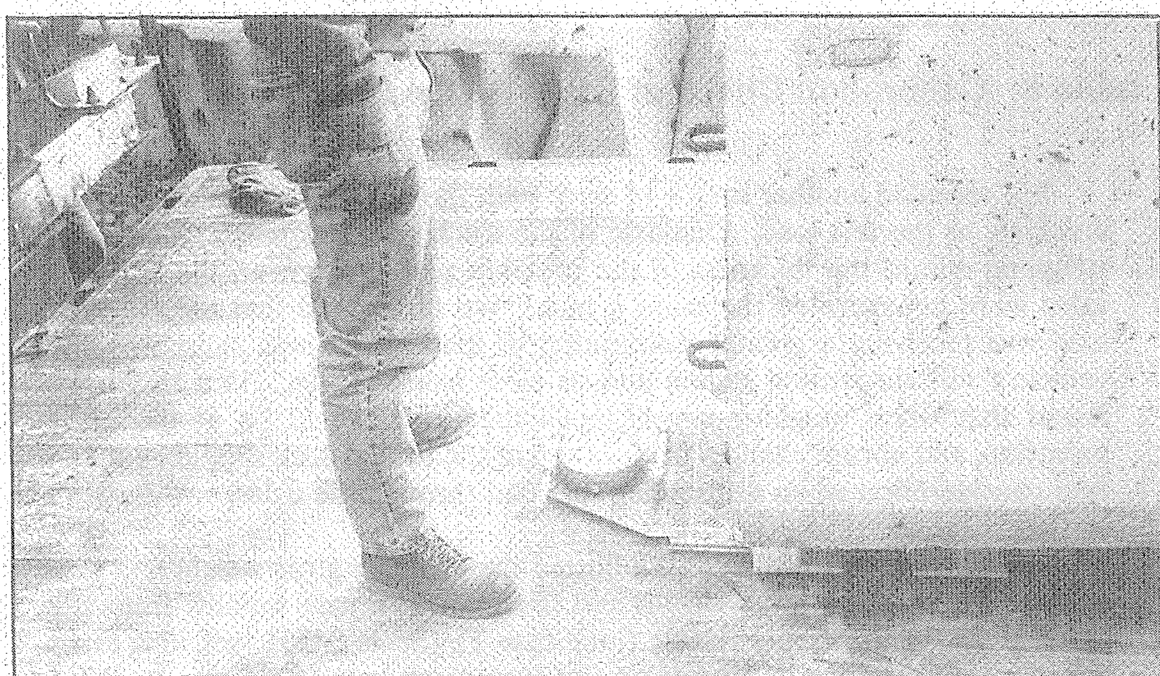


Figure 13: Portable Scale Weighing Temporary Barrier.

While investigating this error, we believe that we found the discrepancy between the two weighing methods. The MAC scale, which is used for commercial purposes, was last calibrated by the State of Michigan on October 19, 1994. It must be accurate within ±0.2 percent to be a registered commercial scale. Due to time constraints, we did not calibrate the portable scales until after the load test. When we calibrated the portable scales, it was found that scale 3423 "under-weighed" by approximately six percent (see Table 8). Scale 2392, in contrast, showed a calibrated accuracy of one percent, which is within the expected accuracy of this type of scale. We also found that changes or variability of air pressure in the air suspension system will alter the trailer's load distribution to the axles.

| scale # 2392 | | | scale # 3423 | | |
|---|---|---|---|---|---|
| WEIGHT POUNDS | SCALE READING POUNDS | PERCENT DIFFERANCE | WEIGHT POUNDS | SCALE READINGS POUNDS | PERCENT DIFFERANCE |
| 1000 | 1000 | 0 | 1000 | 950 | 5 |
| 2000 | 2000 | 0 | 2000 | 1850 | 7 |
| 3000 | 3000 | 0 | 3000 | 2800 | 7 |
| 4000 | 4100 | 2.5 | 4000 | 3750 | 6 |
| 5000 | 5075 | 1.5 | 5000 | 4700 | 6 |
| 6000 | 6050 | 1 | 6000 | 5650 | 6 |
| 7000 | 7050 | 1 | 7000 | 6600 | 6 |

Note: Scale # 3423 out of calibration
Checked by NAL & CPD 11\10\94

## Table 8: Calibration Check of State Police Scales

We revealed another source of error with the portable scales when using a vehicle as the test load. One axle of the truck was elevated ¼ inch during weighing due to the thickness of the portable scales; however, the adjacent axles were not elevated the same ¼ inch. We discovered that the elevated axle was receiving a greater distribution of load, which was caused by the vehicle's stiff suspension system and its close axle spacing. As a result, we found that when weighing a vehicle all the axles should be at the same elevation, and abrupt changes in elevation of the bridge deck should be taken into consideration when distributing the load through the axles.

For the post-test analysis, we adjusted the vehicle's gross weight and axle loads. We used the gross weight and the individual axle weights from the MAC scale for the empty vehicle (load case 1). Scale 3423's weighing error was compensated for by increasing the temporary barrier weights by three percent (see Table 9). During the load test, the temporary barrier was applied to the structure in 10 increments designated as load cases. We adjusted the vehicle gross weight for each load case by adding the adjusted temporary barrier weights to the MAC empty vehicle weight (see Tables 10 and 11). The method we used to adjust the axle weights for each load case is shown below:

| BARRIER | AS MEASURED WITH PORTABLE SCALES 11-7-94 | | | |
|---|---|---|---|---|
| | WEIGHT 1 | WEIGHT 2 | TOTAL | * ADJUSTED WEIGHT |
| 1 | 2450 | 2600 | 5050 | 5202 |
| 2 | 2500 | 2450 | 4950 | 5099 |
| 3 | 2350 | 2500 | 4850 | 4996 |
| 4 | 2700 | 2300 | 5000 | 5150 |
| 5 | 2400 | 2500 | 4900 | 5047 |
| 6 | 2850 | 2050 | 4900 | 5047 |
| 7 | 2300 | 2650 | 4950 | 5099 |
| 8 | 2150 | 2800 | 4950 | 5099 |
| 9 | 2450 | 2850 | 5300 | 5459 |
| 10 | 2250 | 2700 | 4950 | 5099 |
| 11 | 2550 | 2400 | 4950 | 5099 |
| 12 | 2200 | 2800 | 5000 | 5150 |
| 13 | 2850 | 2350 | 5200 | 5356 |
| 14 | 2400 | 2600 | 5000 | 5150 |
| 15 | 2500 | 2700 | 5200 | 5356 |
| 16 | 2600 | 2350 | 4950 | 5099 |
| 17 | 2300 | 2700 | 5000 | 5150 |
| 18 | 2600 | 2350 | 4950 | 5099 |

*Total weight increased by 3% due to one scale weighting 6% light,as per scale calibration

**Table 9: Adjusted Barrier Weights**

1. During load case 8, we weighed each axle of the vehicle using the portable scales (see Table 6) and computed percentages of total load for each axle.

2. Load case 8 axle loads were adjusted by multiplying the percentage of each axle load, obtained in step 1, to the load case 8 adjusted gross weight.

3. For the remaining load cases, we adjusted the axles loads by interpolating between the load case 8 adjusted axle weights and the MAC empty vehicle (load case 1) axle weights given the adjusted gross weight of the vehicle. These values are listed in Tables 10 and 11, and shown graphically in Figures 14 and 15.

<u>Deflection Measurements</u>

The Structural Research Unit measured deflections of the beams using six Schaevitz 2000 HCD linear variable differential transformers (LVDTs), which

| | Axle Number | MAC Scale (lbs.) Truck Empty | Adjusted (lbs.) | Adjusted (lbs.) | Adjusted (lbs.) | Adjusted (lbs.) | Adjusted (lbs.) | Adjusted (lbs.) | Adjusted (lbs.) | Adjusted (lbs.) | Adjusted (lbs.) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Tractor*** | 1 | 10480 | 10351 | 10229 | 10109 | 9985 | 9864 | 9741 | 9618 | 9496 | 9373 |
| | 2 | 5900 | 6123 | 6332 | 6540 | 6753 | 6963 | 7175 | 7385 | 7596 | 7808 |
| | 3 | 5520 | 5748 | 5963 | 6176 | 6394 | 6609 | 6826 | 7042 | 7258 | 7475 |
| ***Trailer*** | 4 | 6260 | 6853 | 7412 | 7966 | 8533 | 9093 | 9657 | 10219 | 10781 | 11346 |
| | 5 | 7460 | 7996 | 8500 | 9000 | 9513 | 10018 | 10528 | 11035 | 11543 | 12053 |
| | 6 | 660 | 1783 | 2842 | 3890 | 4965 | 6024 | 7094 | 8158 | 9223 | 10292 |
| | 7 | 1180 | 2347 | 3448 | 4538 | 5655 | 6756 | 7868 | 8974 | 10081 | 11192 |
| | 8 | 1420 | 3150 | 4780 | 6395 | 8050 | 9681 | 11328 | 12968 | 14607 | 16254 |
| | 9 | 1240 | 3280 | 5203 | 7106 | 9059 | 10982 | 12924 | 14857 | 16790 | 18732 |
| | 10 | 0 | 2650 | 5148 | 7621 | 10158 | 12656 | 15180 | 17691 | 20202 | 22725 |
| | 11 | 6360 | 7015 | 7633 | 8245 | 8872 | 9490 | 10114 | 10735 | 11356 | 11980 |
| ***Sum (lbs.)*** | | 46480 | 57295 | 67492 | 77586 | 87938 | 98135 | 108435 | 118683 | 128932 | 139232 |
| ***Sum (tons)*** | | 23.2 | 28.6 | 33.7 | 38.8 | 44.0 | 49.1 | 54.2 | 59.3 | 64.5 | 69.6 |
| *Equivalent Proof Load (tons)* | | | | | | | | | 69.2 | 77.1 | 85.0 |

***Table 10 - Adjusted Axle Weights and Equivalent Proof Loads, Day 1***

| | Axle Number | Load Case 1 MAC Scale(lbs.) Truck Emty | Load Case 2 Adjusted (lbs.) | Load Case 3 Adjusted (lbs.) | Load Case 4 Adjusted (lbs.) | Load Case 5 Adjusted (lbs.) | Load Case 6 Adjusted (lbs.) | Load Case 7 Adjusted (lbs.) | Load Case 8 Adjusted (lbs.) | Load Case 9 Adjusted (lbs.) | Load Case 10 Adjusted (lbs.) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Tractor*** | 1 | 10480 | 10397 | 10318 | 10238 | 10160 | 10081 | 10001 | 9922 | 9844 | 9764 |
| | 2 | 5900 | 6203 | 6487 | 6780 | 7062 | 7349 | 7636 | 7923 | 8209 | 8497 |
| | 3 | 5520 | 5807 | 6076 | 6353 | 6621 | 6892 | 7164 | 7436 | 7706 | 7979 |
| ***Trailer*** | 4 | 6260 | 6929 | 7556 | 8202 | 8826 | 9460 | 10093 | 10727 | 11357 | 11994 |
| | 5 | 7460 | 8095 | 8691 | 9304 | 9897 | 10499 | 11100 | 11702 | 12301 | 12905 |
| | 6 | 660 | 2130 | 3510 | 4931 | 6303 | 7696 | 9090 | 10483 | 11869 | 13270 |
| | 7 | 1180 | 2706 | 4138 | 5613 | 7038 | 8484 | 9931 | 11377 | 12816 | 14270 |
| | 8 | 1420 | 3141 | 4756 | 6420 | 8027 | 9658 | 11290 | 12921 | 14544 | 16184 |
| | 9 | 1240 | 2994 | 4640 | 6336 | 7974 | 9636 | 11299 | 12962 | 14616 | 16287 |
| | 10 | 0 | 2056 | 3984 | 5971 | 7890 | 9838 | 11786 | 13734 | 15672 | 17629 |
| | 11 | 6360 | 6837 | 7285 | 7746 | 8192 | 8644 | 9096 | 9548 | 9998 | 10453 |
| ***Sum(lbs.)*** | | 46480 | 57295 | 67441 | 77895 | 87989 | 98238 | 108486 | 118735 | 128932 | 139232 |
| ***Sum(Tons)*** | | 23.2 | 28.6 | 33.7 | 38.9 | 44.0 | 49.1 | 54.2 | 59.4 | 64.5 | 69.6 |
| *Equivalent Proof Loa (Tons)* | | | | | | | | | 68.8 | 76.6 | 84.4 |

***Table 11 - Adjusted Axle Weights and Equivalent Proof Loads, Day 2***

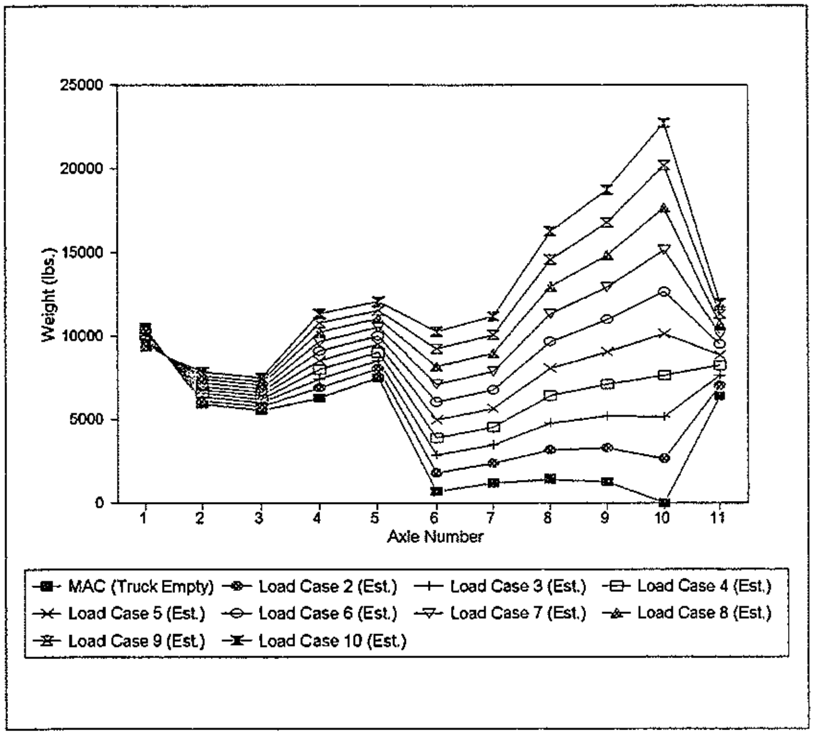



Figure 14 - Day 1, Axle Weight Distribution.

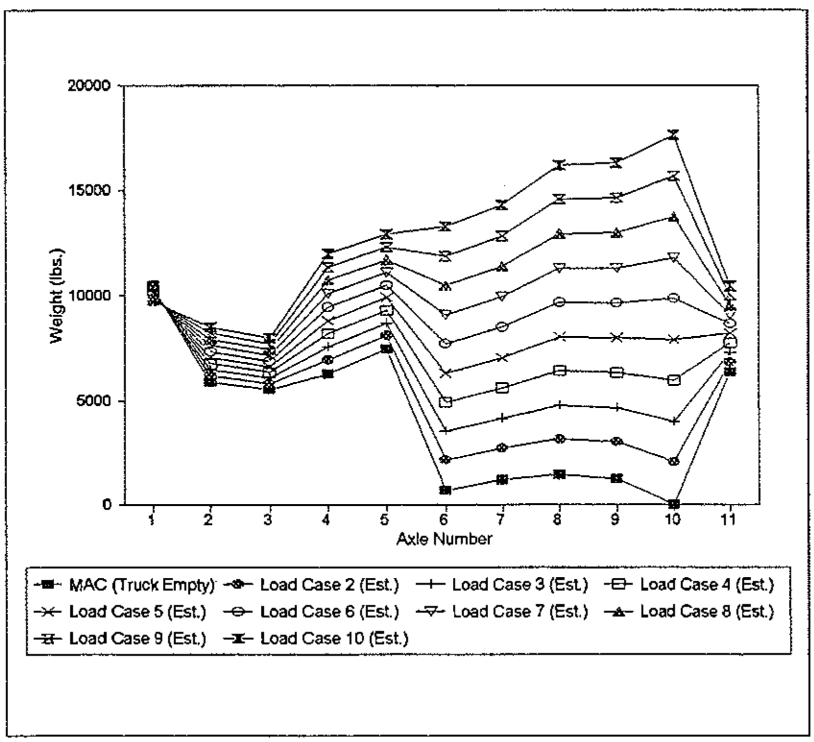



Figure 15 - Day 2, Axle Weight Distribution.

have a two inch range of motion in each vertical direction. We suspended the core of each LVDT from the bridge beams with 1/16 inch diameter aircraft cable, a ten pound weight (to keep the line taught), and a swivel connection between the LVDT core and the weight (to allow for minor imperfections in alignment). The barrel of the LVDT was attached to a 50 pound weight, which rested on the railroad bed under the structure (see Figure 16). We found that the LVDTs in this configuration had an accuracy of 0.001 inches.

Deflection measurements were also collected with a Wild NA2000 electronic level to check the data received from the LVDTs. The Wild NA2000 level uses a "bar code" rod that has an accuracy of approximately 0.012 inches. Level rods were attached to each of the nine beams under the bridge (see Figure 17) and readings taken for each load case. After review of the data, we determined that the accuracy of the level was not adequate for this application because the change in beam deflection for each load case was in the range of 0.002 to 0.004 inches. Therefore, the level rod data were not used for interpretation of the results.

## Data Collection

During the load test, the Structural Research Unit entered the deflection data, shown in Figure 18, directly into a lap-top computer. We plotted load versus deflection diagrams for all instrumented beams as each load case was placed on the structure, and monitored the diagrams for possible elastic, non-linear behavior and relative stiffness of the beams. This computational ability to monitor the structure's response and to compare it to theoretical response, was critical to the successful and safe completion of the load test.

## Monitoring Cracks

The Structural Research Unit used a Celestron 30X Spotting Scope to observe the fascia beams during the loading sequence for changes in crack dimensions, as shown in Figure 19. We also monitored the bridge deck for changes of crack widths and for initiation of new cracks.

## Loading Procedure

We applied the 82 ton proof load to the bridge in ten increments of approximately 10,000 pounds each by placing two temporary concrete barriers on the test vehicle for each load case (see Figures 20 to 25). In order to achieve maximum moment, the bridge was loaded with the eight axles of the trailer centered on span 2, which has a length of 41 feet. In this alignment, however, the semi-tractor was actually off the span. Therefore, its weight was not included in the pre-analysis (simple span) moment calculations.

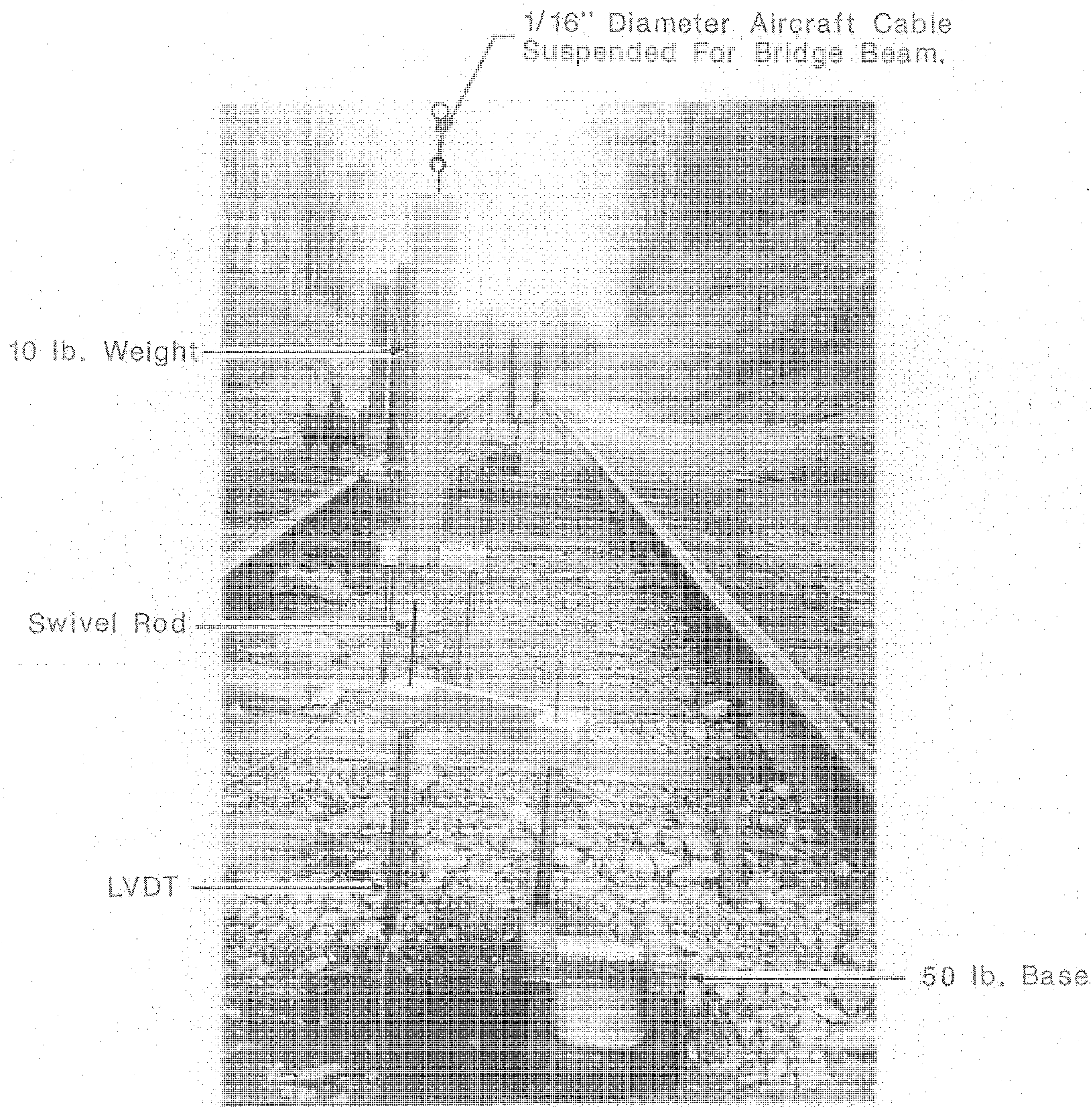



Figure 16: LVDT Set-Up For Measuring Deflection of Bridge Beams.

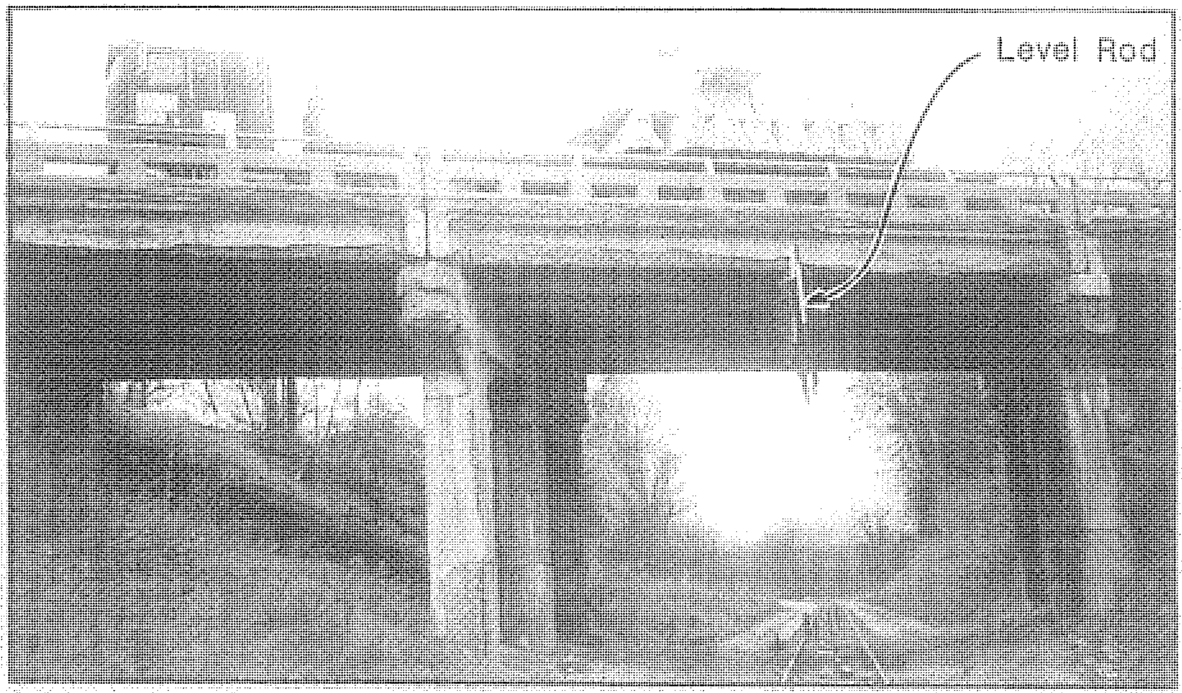



Figure 17: Level Rods Attached to Each Beam.

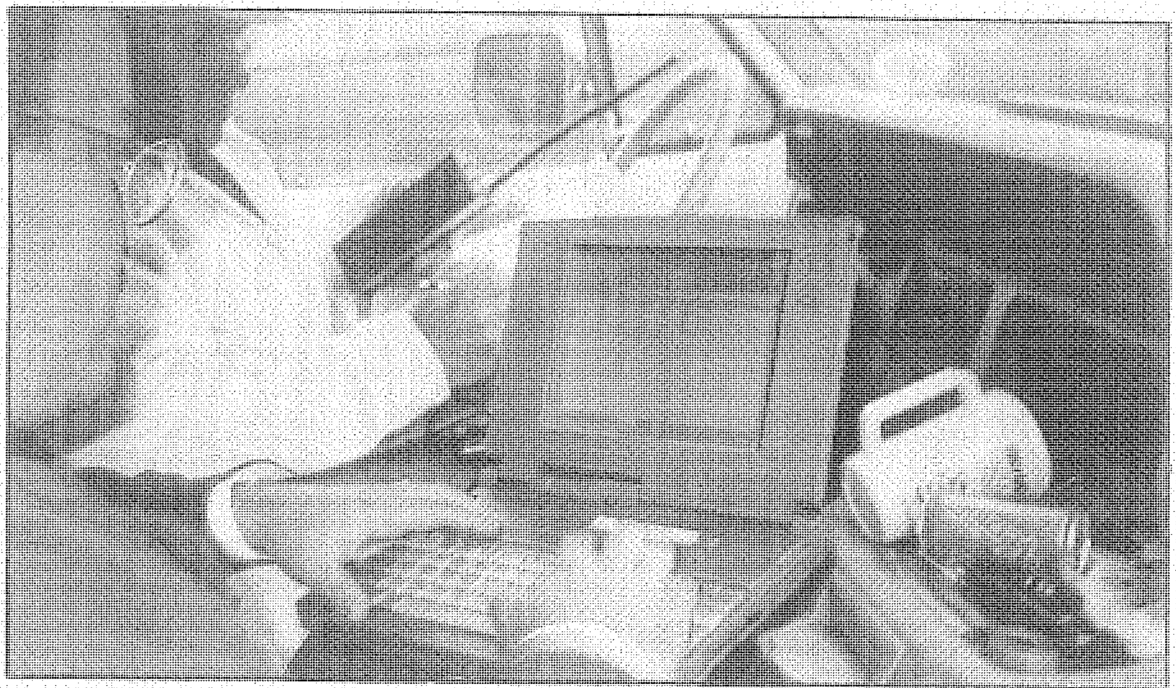


Figure 18: Lap Top Computer Used to Input Deflection Data and Plot Load vs Deflection Diagrams.

Figure 19: A Spotting Scope Was Used to Monitor Cracks in the Fascia Beams.

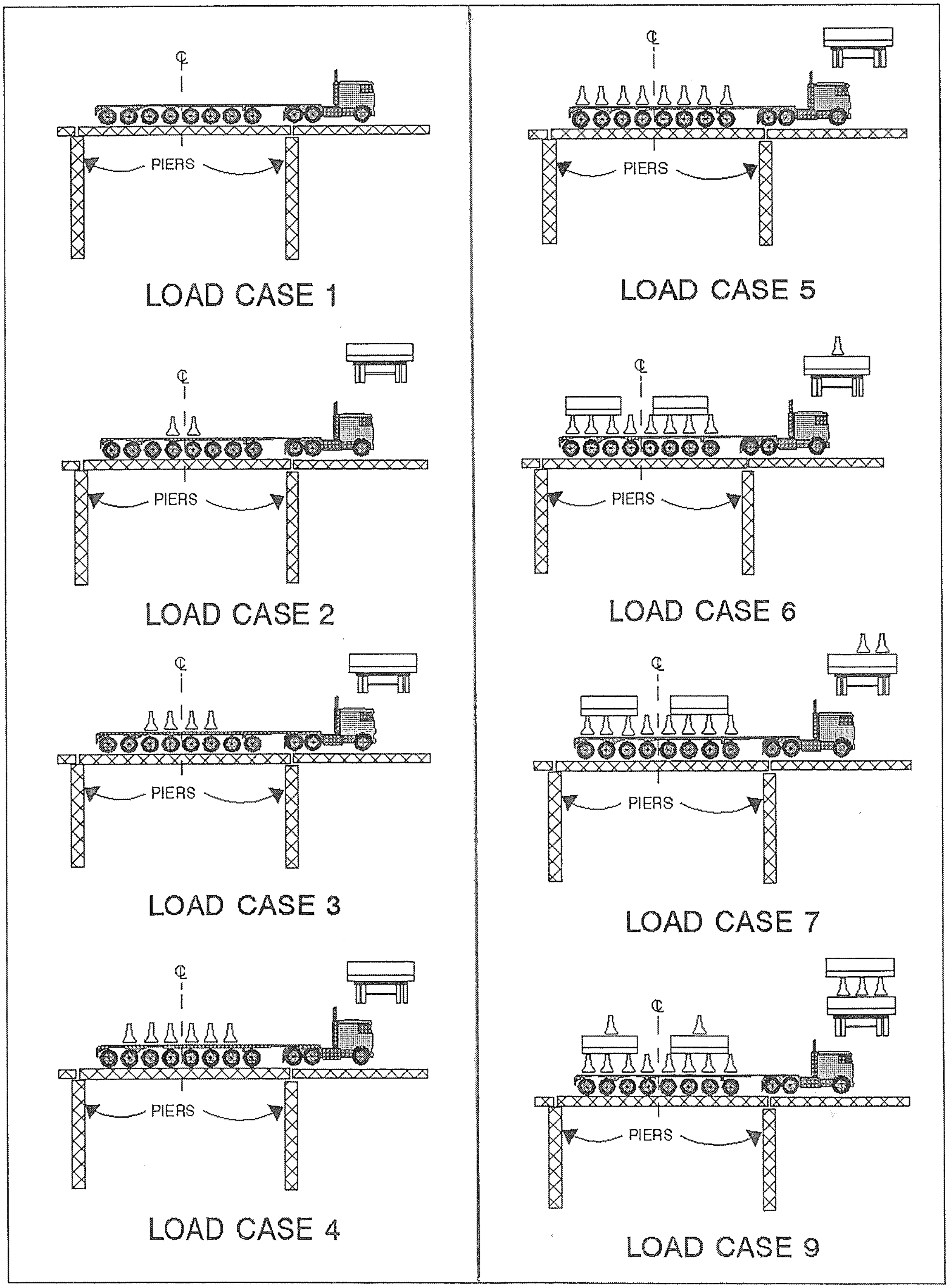



FIGURE 20 - DAY 1

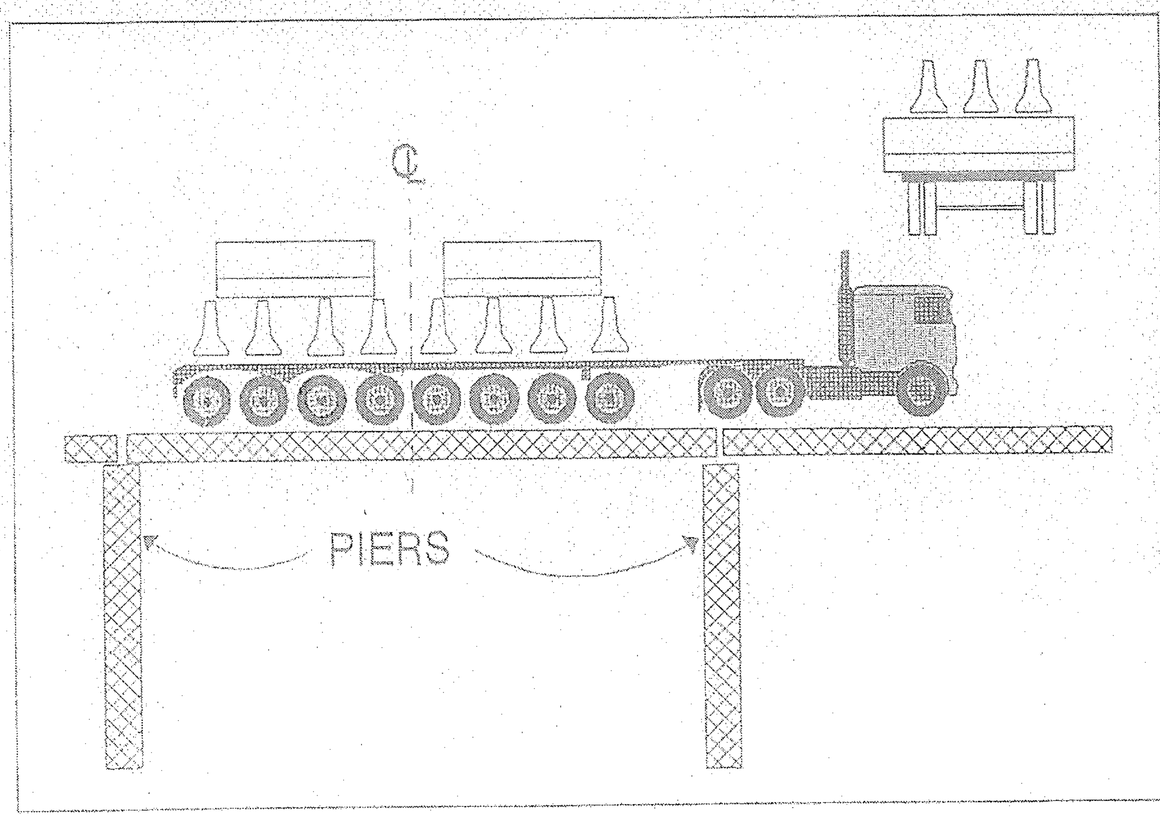
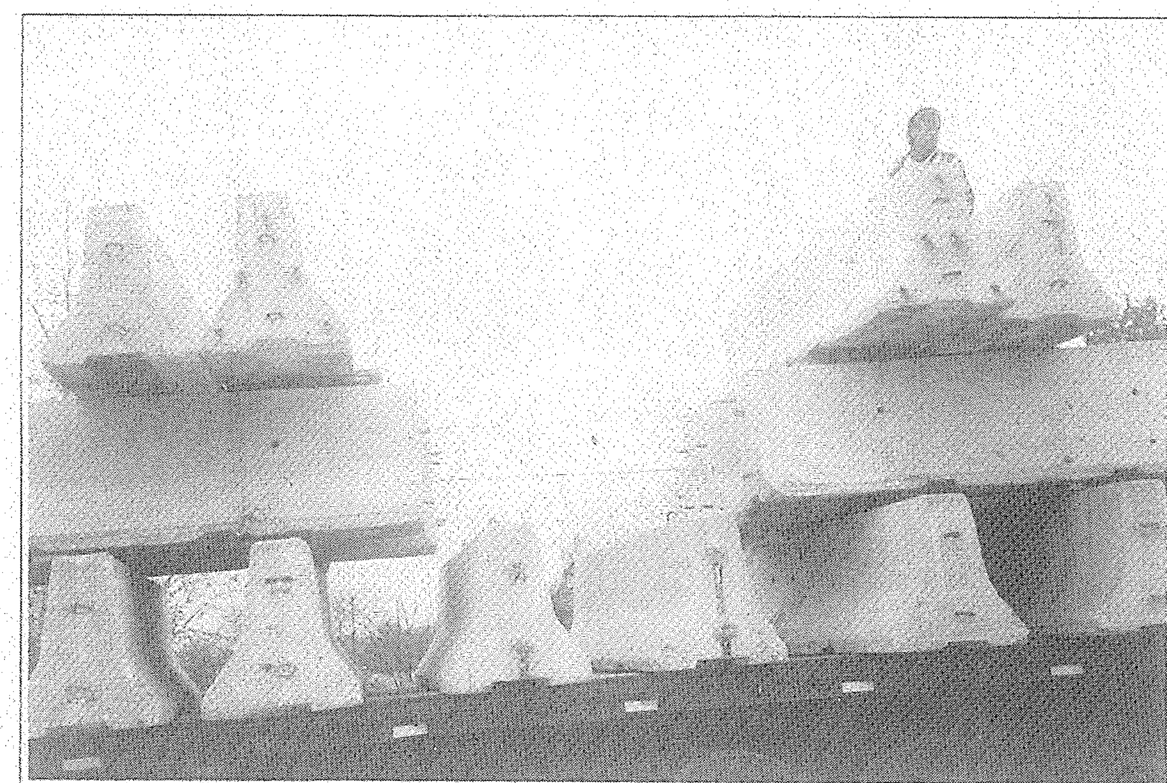



FIGURE 21 - DAY 1, LOAD CASE 8



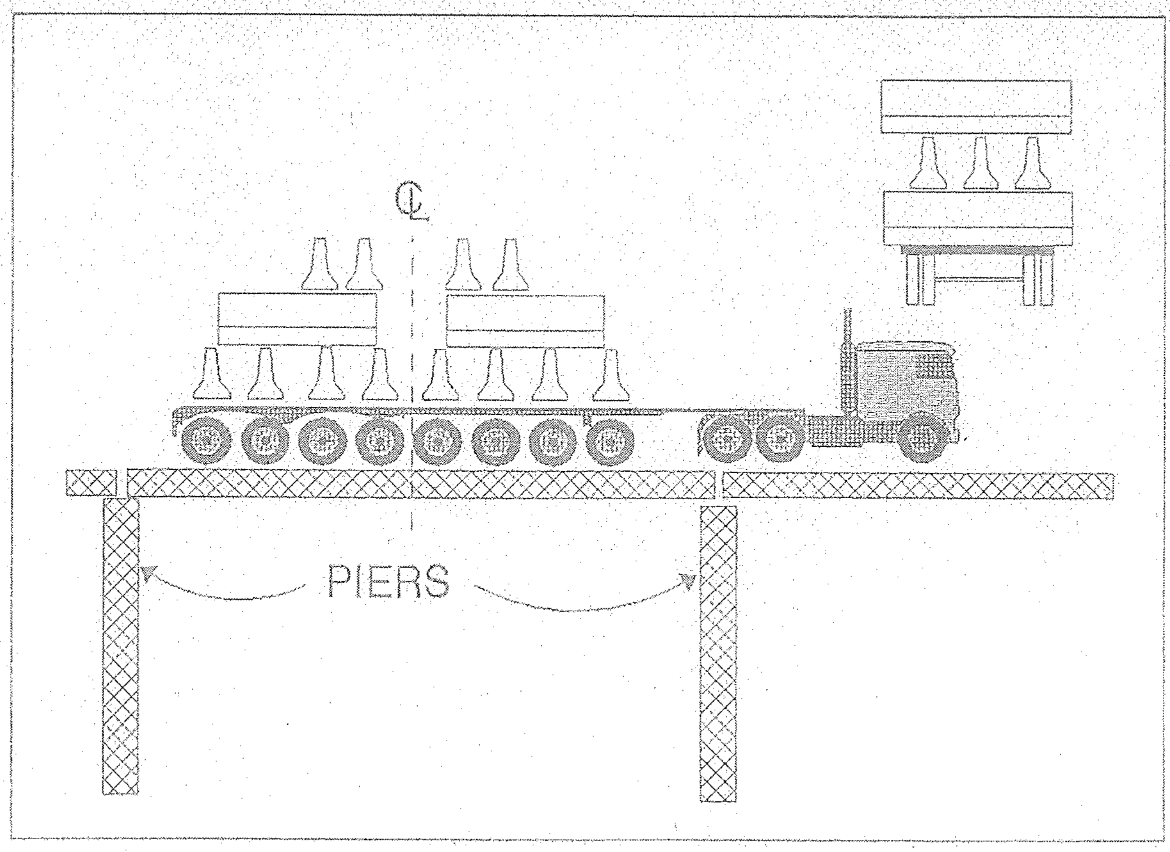



FIGURE 22 - DAY 1, LOAD CASE 10

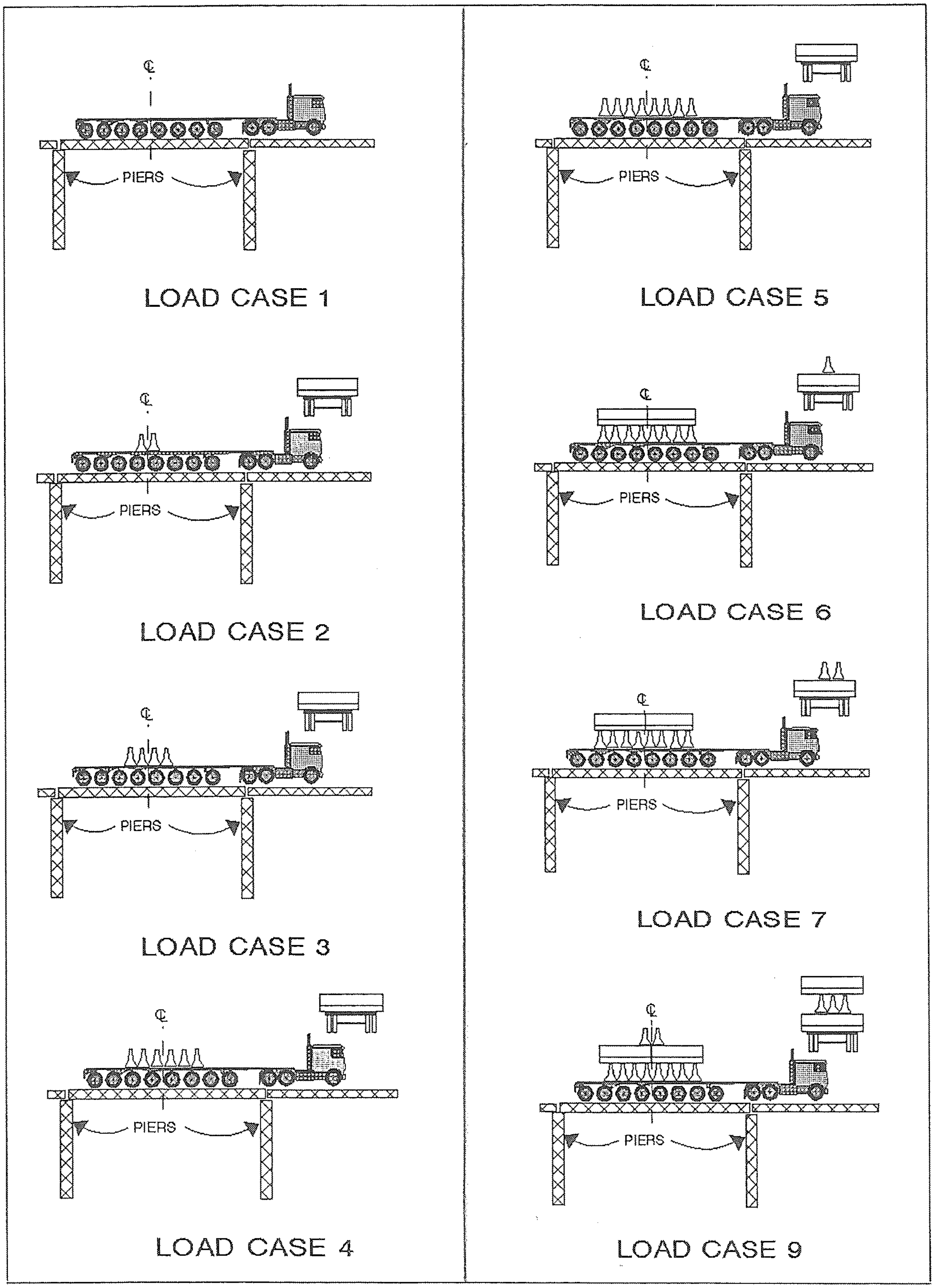



FIGURE 23 - DAY 2

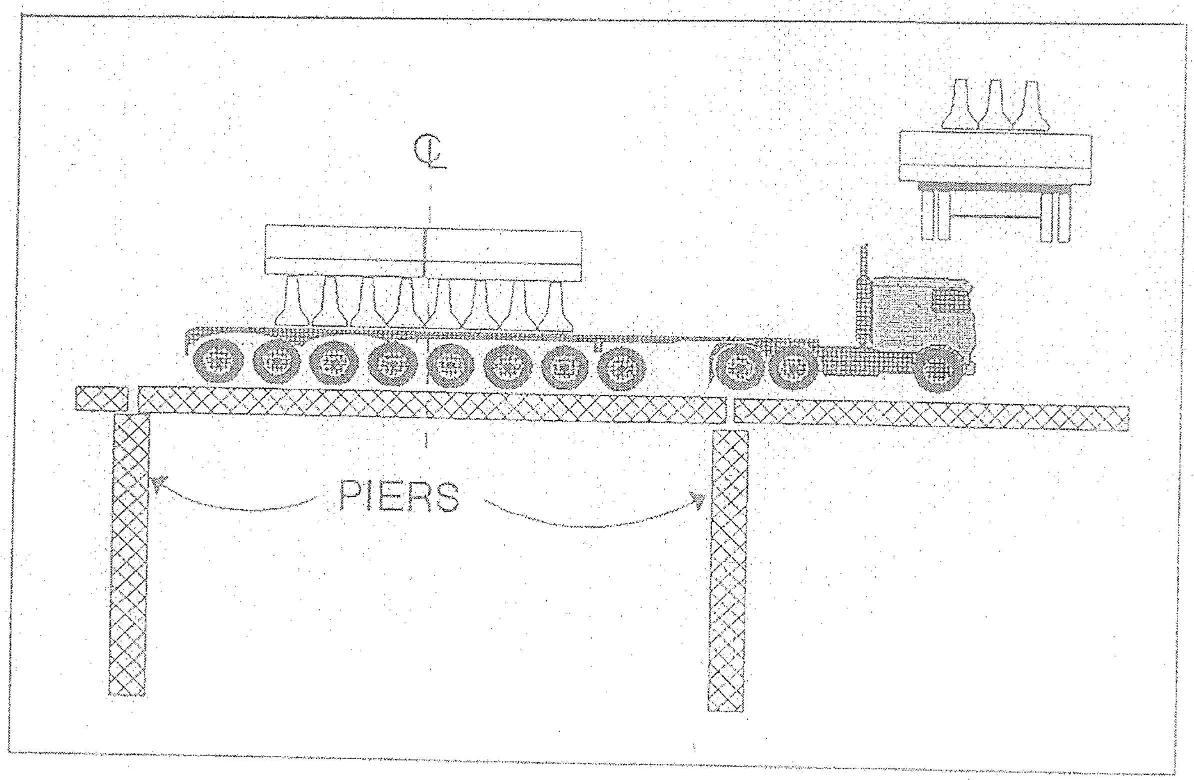




FIGURE 24 - DAY 2, LOAD CASE 8

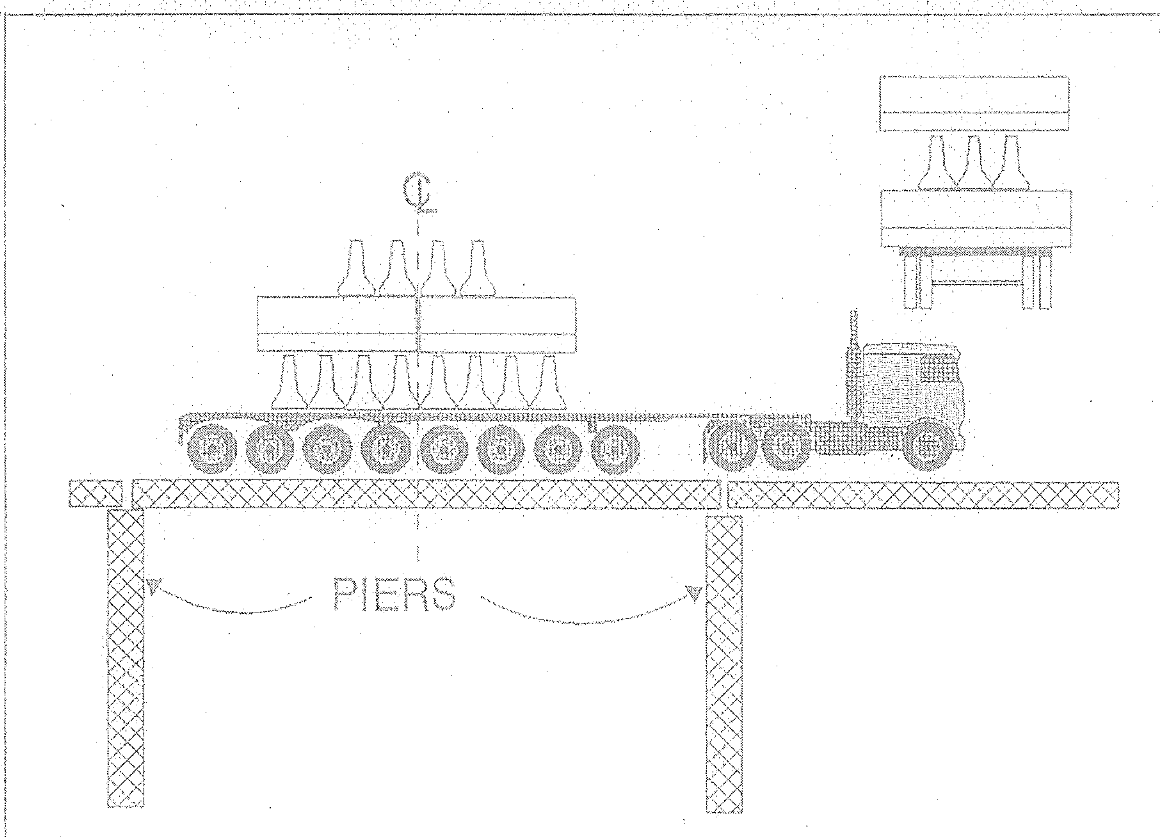



FIGURE 25 – DAY 2, LOAD CASE 10

The Structural Research Unit performed the load test on Tuesday, November 8, and Wednesday, November 9, 1994. Activities that took place on Tuesday are indicated in this report as "Day One", whereas activities that took place on Wednesday are indicated as "Day Two". The loading procedure for the test follows:

1. Initial deflection readings with no live load on the structure were taken.

2. The test vehicle was parked on the bridge with the trailer centered on span 2 with the trailer wheels flush to the curb. The contractor placed two temporary barriers on the trailer for each load case. For example, load case 1 consisted of the empty test vehicle parked on the bridge, whereas load case 2 had two temporary concrete barriers loaded on the vehicle's trailer, etc. Finally, load case 10 had 18 barriers loaded on the trailer for a gross vehicle weight of 139,000 pounds.

3. After the contractor placed load case 10, he removed four temporary barriers from the test vehicle to return to load case 8. The test vehicle was then driven off the bridge so it could be weighed using the portable scales.

4. The contractor drove the test vehicle back onto the bridge and parked it in the center of the driving lane on span 2, and deflection readings were recorded (Load Case 8R).

5. The contractor removed the test vehicle from the bridge. We allowed the bridge 15 minutes to rebound before the final (no-load) deflection readings were recorded

The loading procedure is explained in greater detail for each day of the test in the following section.

## TEST RESULTS

<u>Load Test - Day One, Tuesday, November 8, 1994</u>

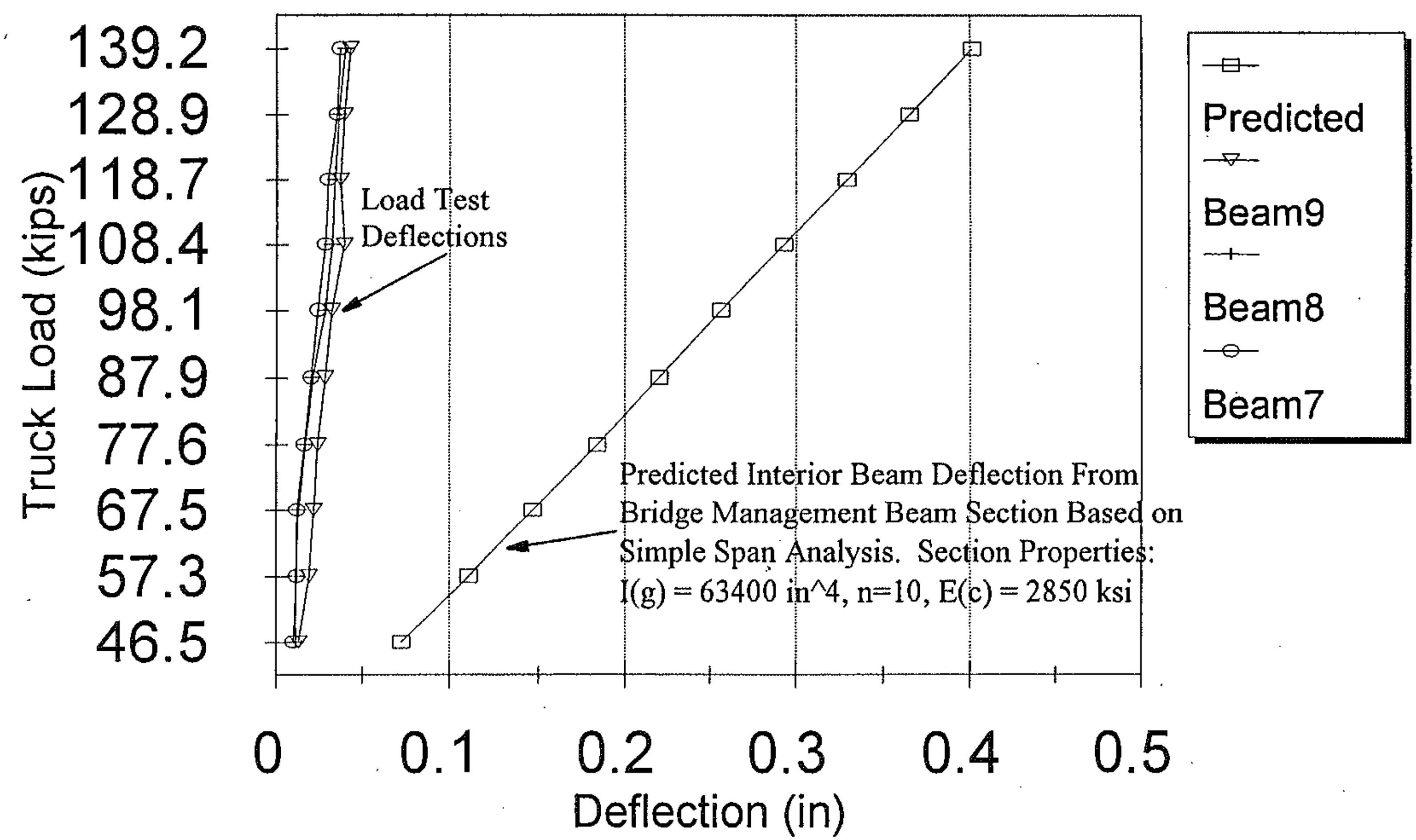
On Day One of the load test, we instructed the contractor to park the test vehicle on the structure in two positions. In the first position, the trailer wheels were flush to the south curb, and the trailer was positioned in the middle of the center span (span 2), as shown in Figure 26. We chose this position to distribute the maximum load to the fascia beam (beam 9), which we felt was the weakest member. In addition, this load position would also give the largest percentage of load distribution to any one beam.

The contractor applied load cases 2 through 10 to the structure by placing the temporary barriers directly over the axles as shown in Figures 20 through 22. For each load case, we recorded beam deflections, shown in Table 12 and plotted load versus deflection diagrams. The diagrams remained linear for each load case which showed that the structure was within its elastic load range. The deflection of the beams was far less than estimated, indicating greater stiffness and thus, greater load capacity. The bridge was loaded through load case 10 with no signs of distress. Load versus deflection diagrams for beams 7, 8 and 9 during Day One, Position One are shown in Figure 28. In this figure, we also show the computed deflections based on the Bridge Management Unit's estimated section. Figure 29 shows the deflection of each beam for load cases 8 through 10 and the distribution of load to the beams for load case 10. This figure reveals that the maximum distribution of load to any beam is approximately 26 percent, whereas the AASHTO distribution factor used for analysis is 31.7 percent. Figure 29 also shows that maximum deflection occurs in the fascia beam (beam 9). After the contractor removed the final load from the structure, we waited 15 minutes and then took deflection measurements that showed beam 9 had a deflection of 0.011 inches; a sign that permanent deformation may have resulted in this beam or that the beam needed more time to rebound.

The contractor then positioned the test vehicle on the bridge with the trailer wheels 6.8 feet from the curb (Position 2) and the trailer centered on span 2 (see Figure 27). The deflection readings can be found in Table 12 (load case 8R), and beam deflections in Figure 30.

Figure 26: Truck position on day 1, position 1.
(Trailer wheels adjacent to south curb.)


Figure 27: Truck position on day 1, position 2.
(Trailer wheels 6.8' from curb.)




**Bailey Bridge Load Test - Day 1**

Load vs. Deflection, Position 1

Figure 28

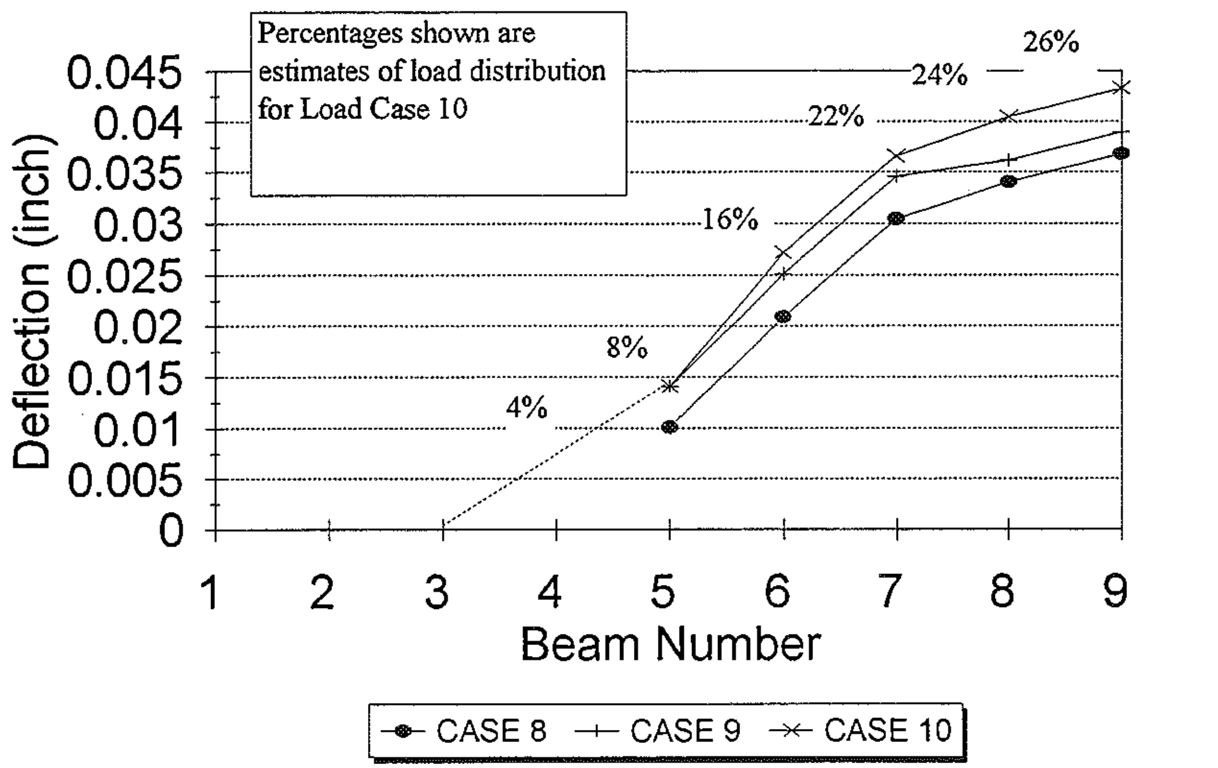



*Figure 29 - Load Distribution*
*Day 1, Position 1*

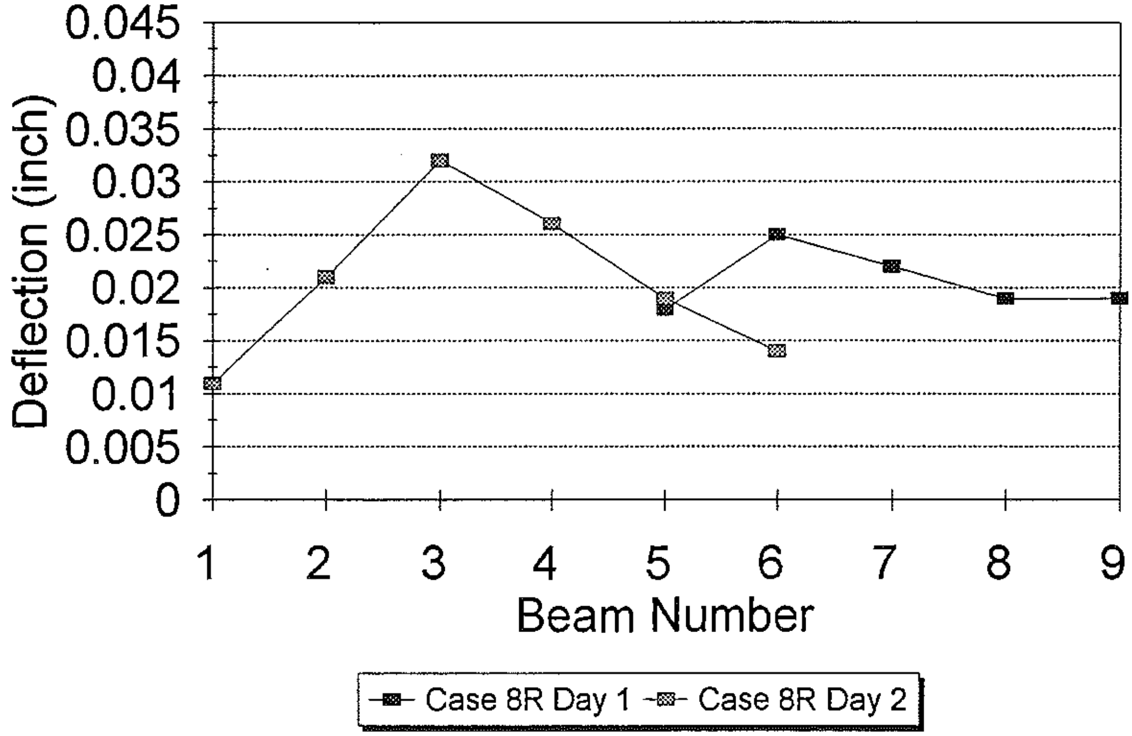



***Figure 30 - Load Distribution***
***Case 8R, Position 2***

| Delfections (inches) | | | | | |
|---|---|---|---|---|---|
| | BEAM 5 | BEAM 6 | BEAM 7 | BEAM 8 | BEAM 9 |
| CASE 1 | 0.006 | 0.006 | 0.010 | 0.011 | 0.013 |
| CASE 2 | 0.004 | 0.008 | 0.012 | 0.011 | 0.019 |
| CASE 3 | 0.004 | 0.010 | 0.012 | 0.013 | 0.022 |
| CASE 4 | 0.006 | 0.013 | 0.016 | 0.017 | 0.024 |
| CASE 5 | 0.008 | 0.015 | 0.020 | 0.021 | 0.028 |
| CASE 6 | 0.010 | 0.019 | 0.024 | 0.028 | 0.032 |
| CASE 7 | 0.010 | 0.021 | 0.028 | 0.032 | 0.039 |
| CASE 8 | 0.010 | 0.021 | 0.030 | 0.034 | 0.037 |
| CASE 9 | 0.014 | 0.025 | 0.035 | 0.036 | 0.039 |
| CASE 10 | 0.014 | 0.027 | 0.037 | 0.040 | 0.043 |
| UNLOADED * | -0.004 | -0.002 | 0.002 | 0.002 | 0.011 |
| CASE 8 R ** | 0.018 | 0.025 | 0.022 | 0.019 | 0.019 |

* No live load on bridge
** Trailer in position 2, trailer wheels 6.8' from curb.

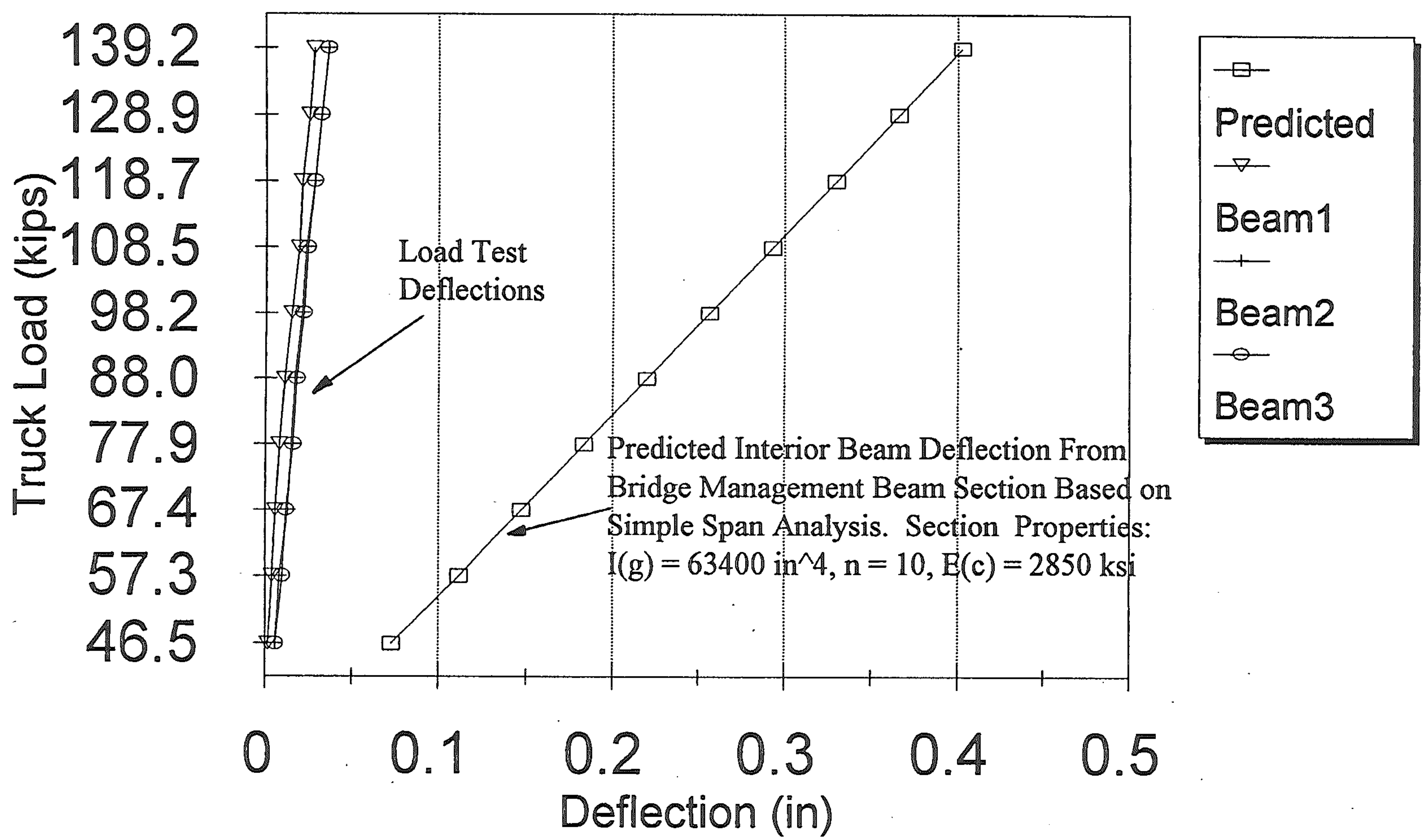
**Table 12: LVDT Deflections - Day 1**

Load Test - Day Two, Wednesday, November 9, 1994

On Day Two, we repeated the test for the north half of the structure, keeping in mind that the axle weights from Day One were unequally distributed from the front to the rear of the trailer. Since it was desired to equally distribute the load to all axles of the trailer, we concentrated the load closer to the center of the trailer as is shown in Figures 23 through 25. For Position One (see Figure 31), the contractor positioned the trailer so its wheels were adjacent to the north curb. We loaded the bridge through load case 10, and found no signs of distress or non-linear behavior (see Table 13 for deflections). Load versus deflection plots for beams 1, 2 and 3 are shown in Figure 33, along with the Bridge Management Unit's estimated deflections. As on Day One, the beams showed greater stiffness. After the contractor removed the load from the bridge, we took deflection readings (see Table 13). The contractor then placed the vehicle on the bridge in Position Two (see Figure 32), with the trailer wheels 5.4 in. from the curb. Recorded deflections can be found in Table 13, load case 8R, and the deflections for each instrumented beam in Figure 30.

Figure 31: Day 2, Position 1. Vehicle trailer wheels adjacent to North Curb.

Figure 32: Day 2, Position 2. Vehicle trailer wheels 5.4' from North Curb.




**Bailey Bridge Load Test - Day 2**

Load vs. Deflection - Position 1

**Figure 33**

| Deflections (inches) | | | | | | |
|---|---|---|---|---|---|---|
| | BEAM 1 | BEAM 2 | BEAM 3 | BEAM 4 | BEAM 5 | BEAM 6 |
| CASE 1 | 0.002 | 0.006 | 0.006 | 0.004 | 0.002 | 0.002 |
| CASE 2 | 0.004 | 0.008 | 0.010 | 0.006 | 0.002 | 0.002 |
| CASE 3 | 0.006 | 0.013 | 0.012 | 0.008 | 0.002 | 0.004 |
| CASE 4 | 0.008 | 0.015 | 0.016 | 0.008 | 0.004 | 0.004 |
| CASE 5 | 0.011 | 0.017 | 0.018 | 0.010 | 0.006 | 0.004 |
| CASE 6 | 0.015 | 0.021 | 0.022 | 0.014 | 0.006 | 0.006 |
| CASE 7 | 0.019 | 0.023 | 0.024 | 0.014 | 0.009 | 0.006 |
| CASE 8 | 0.021 | 0.028 | 0.028 | 0.016 | 0.009 | 0.010 |
| CASE 9 | 0.025 | 0.032 | 0.032 | 0.020 | 0.013 | 0.010 |
| CASE 10 | 0.028 | 0.036 | 0.036 | 0.022 | 0.013 | 0.010 |
| UNLOADED * | 0.006 | 0.002 | 0.004 | 0.000 | 0.000 | 0.002 |
| CASE 8 R** | 0.011 | 0.021 | 0.032 | 0.026 | 0.019 | 0.014 |
| UNLOADED * | 0.006 | 0.002 | 0.006 | 0.000 | 0.000 | 0.004 |

* No live load on bridge
** Trailer in position 2, trailer wheels were 5.4' from curb.

## Table 13: LVDT Deflections - Day 2

After the contractor removed the load from the bridge and the bridge was allowed to settle for 15 minutes, we found beams 1 and 3 were still deflected 0.006 inches, a sign of permanent deformation or perhaps the need for more rebound time. During the pre-test inspection (see Table 1), we rated beam 1 as "serious condition" and beam 3 as "poor condition." Figure 34 shows the deflection of each instrumented beam for load cases 8, 9 and 10, and that the estimated maximum load distribution to a given beam is approximately 24 percent.

### Loading of The End Span

We subjected the north end-span (span 3) to a 59-ton slow moving load on Day One and Day Two of the load test when the contractor drove the test vehicle across that span during load case 8. Although, we made no attempt to quantitatively measure the span's response to this load or monitor crack widths, we did not observed any visual distress due to the load.

### Post Test Inspection

After completion of the proof loading, we remeasured the cracks that were marked on the beams during the pre-test inspection and found no change in crack widths (see Table 1).

During the test, as expected, new cracks appeared in the slab over the piers. Because it had not been loaded previously, it was found that negative moment at the piers caused tension and cracking in the now continuous slab. We monitored these cracks for changes in width as the test progressed, but of the six cracks that were monitored, only one crack, running along the construction joint over pier one, increased in width (from 0.012 to 0.016 in.).

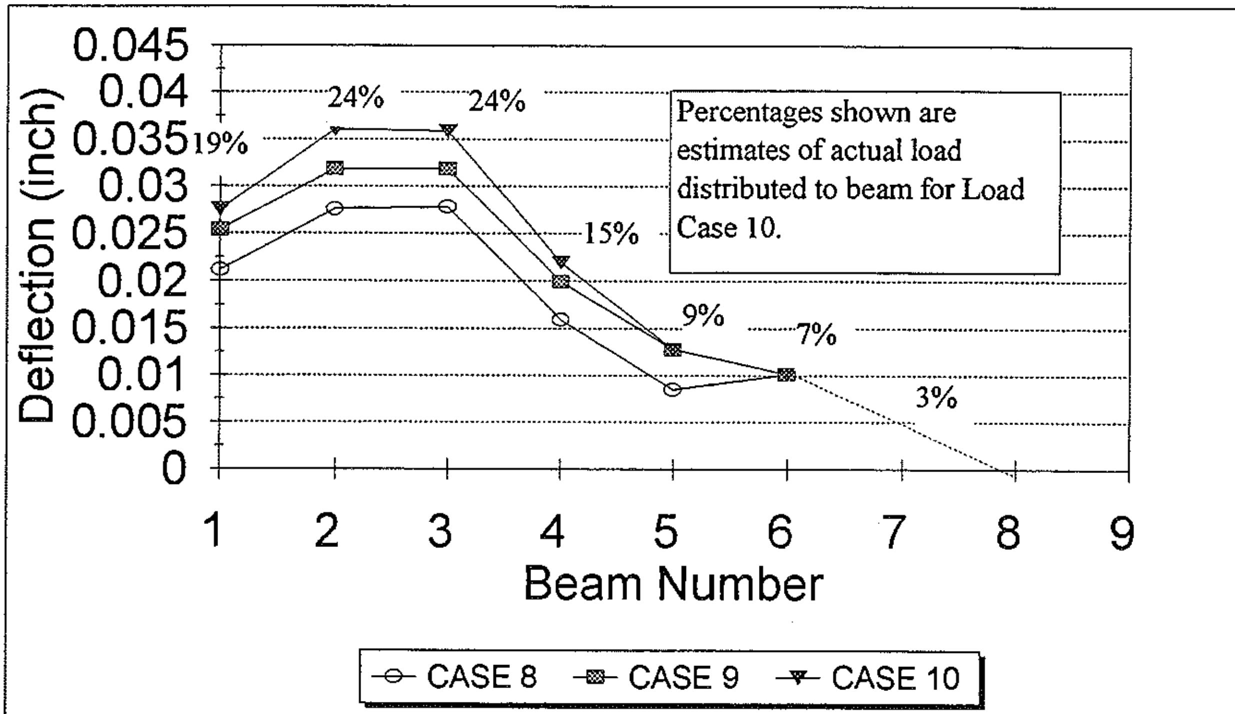



*Figure 34 - Load Distribution*
*Day 2, Position 1*

### Rating, Fascia Beams

We rated both fascia beams (beams 1 and 9) as serious condition. Clearly, the south fascia beam (beam 9) is in the worst condition of the two fascia beams. Because the bottom row of reinforcement is fully exposed, it cannot be considered in the analysis of the beam because there is little bond of the bars to the concrete for development of strength. As a result, this leaves only one reinforcing bar in the second row of the section to provide resistance, which can cause a significant loss in load carrying capacity of the beam. In fact, this was verified during the load test. When reviewing the deflections (shown in Tables 12 and 13, and Figures 29 and 34), we found beam 9 has less relative stiffness than beam 1, a clear indication that the load carrying capacity of beam 9 has been reduced due to deterioration of the reinforcement. Therefore, the live load normally carried by this beam is being redistributed to the first interior beam (beam 8), which was rated in fair condition.

The test data indicate that both fascia beams may have exhibited permanent deformation during the load because the beam failed to return to its original position. This hypothesis is supported by the fact that the interior beams, rated in fair condition, returned to their initial position immediately. Due to the possible permanent deformation, beam 1, which was rated in serious condition, likely has also lost load carrying capacity, albeit not as much as beam 9.

### Rating, Interior Beams

The pre-test inspection showed that five out of seven of the interior beams (2, 4, 6, 7, and 8) are in fair condition, while beams 3 and 5 are in poor condition. However, the load test demonstrated that the beams have far greater strength than Bridge Management estimates. The beams were stiffer than estimated, and all beams, with the exception of beam 3, which showed a permanent deflection of .004 inches, immediately returned to their original positions upon removal of the load. The pre-inspection rating of beam 3 as poor supports the theory that this beam may also have permanent deformations due to the load test.

### Overall Rating

The load test showed, overall, that the beams have far greater stiffness than the Bridge Management Unit estimates, as shown in Table 14. This can be attributed to the following:

1. During the load test, the new reinforced concrete deck acted fully composite with the existing section.

2. Load distribution to the beams was better than estimated.

| | Modulus of Elasticity $E_c$ (psi.) | Moment of Inertia, I (inch^4) | Calculated Stiffness ($S_t$) $S_t = E_c * I$ (lb.*inch^2) |
|---|---|---|---|
| Bridge Management Unit (Pre-Test). Estimated Section | 2.85E+06 (see note 1) | 63000 (see note 2) | 1.80E+11 |
| Post-Analysis Estimated Section Day 1 | 5.0E+06 (see note 3) | 135000 (see note 4) | 6.75E+11 |
| Post-Analysis Estimated Section Day 2 | 5.0E+06 (see note 3) | 150000 (see note 4) | 7.50E+11 |

<u>Notes</u>

1. Calculated value based on concrete compressive strength ($f'_c$) equal to 2500 psi.
2. From Bridge Management Unit analysis.
3. From modulus of elasticity tests for new wearing course concrete.
4. Back calculated given actual beam deflection at center span and actual distribution factor determined from load test. Continuous spans were used for calculation.

**Table 14 - Comparison of Beam Stiffnesses.**

3. Loss of reinforcing steel due to corrosion was less than estimated.

4. Reinforcing steel in the new deck, passing through the expansion joints at the piers allowed the structure to act continuous rather than simply supported.

Post-Test Analysis (November 1994)

The estimated distribution of load to the trailer axles and the actual axle spacings of the test vehicle was used to compute the maximum moment in the center span based on a simple supported span. We divided this moment by the moment produced by the two-unit, 11 axle, 77 gross ton capacity vehicle and then multiplied it by the gross weight of the vehicle to compute the "Equivalent Proof Load" for load case 8, 9 , and 10 (see Tables 10 and 11).

To explain why such small deflections resulted when compared to pre-test estimates, we used the information gathered during the detailed inspection and the load test to estimate a more accurate interior beam resisting section. For this estimate, we made the $5\frac{1}{2}$ in. reinforced concrete wearing course composite with the existing deck and included 100 percent of the steel reinforcement in the beam. This resulted in a gross moment of inertia equal to 149,000 in.$^4$ (see Figure 35). Given the maximum deflections for Day One and Day Two of the load test, we back-calculated the corresponding stiffness and moment of inertia using continuous spans rather than simple spans, and used the estimated distribution factors shown in Figures 29 and 34. The resulting moments of inertia are 135,000 in.$^4$ and 150,000 in.$^4$ for Day One and Day Two, respectively (see Table 14), which agree closely with the post-test estimated section.

Inspection Results (May 1995)

In May 1995, the Structural Research Unit inspected the bridge deck, and used cores removed from the deck to perform a "pull-off bond test" to determine the bond strength between the new concrete overlay and the existing deck. We did this test to see if the freeze-thaw action during the winter of 1995 deteriorated the composite action between the overlay and the existing deck. During our inspection, we found extensive cracking in the concrete overlay where the average width of the cracks were .02 inches to .025 inches (see Figures 37 and 38). When drilling cores for the "pull-off bond test" each core fractured at the concrete overlay/existing deck interface. We inspected the core holes and found loose concrete debris at the bottom of the hole (see Figure 36). Feeling the edges of the core hole, we found a void between the overlay and existing deck. From this inspection, it is clearly evident that there is little bond between the new concrete overlay and the existing deck.

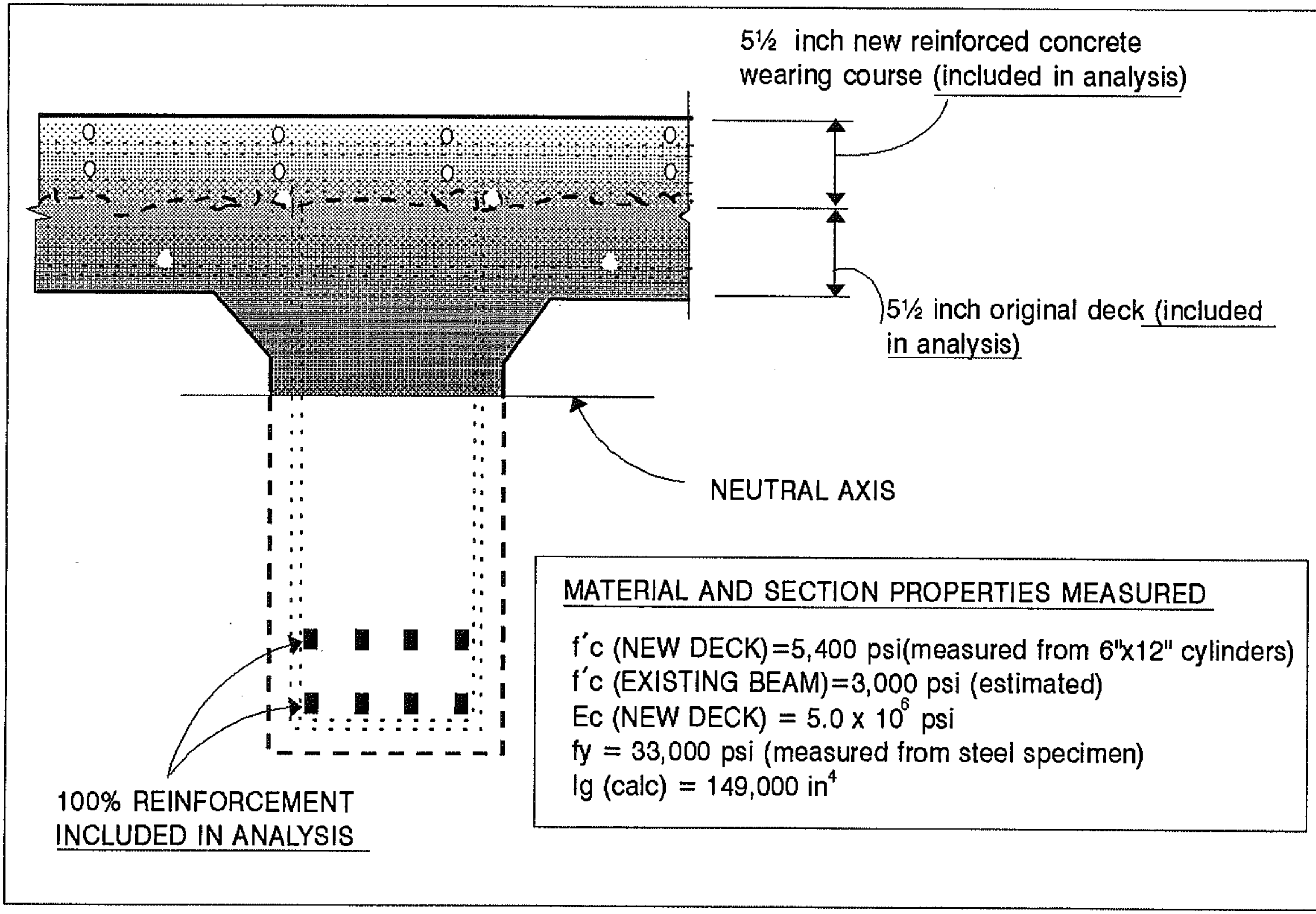



**Figure 35 - POST TEST ESTIMATED SECTION FOR INTERIOR BEAMS**

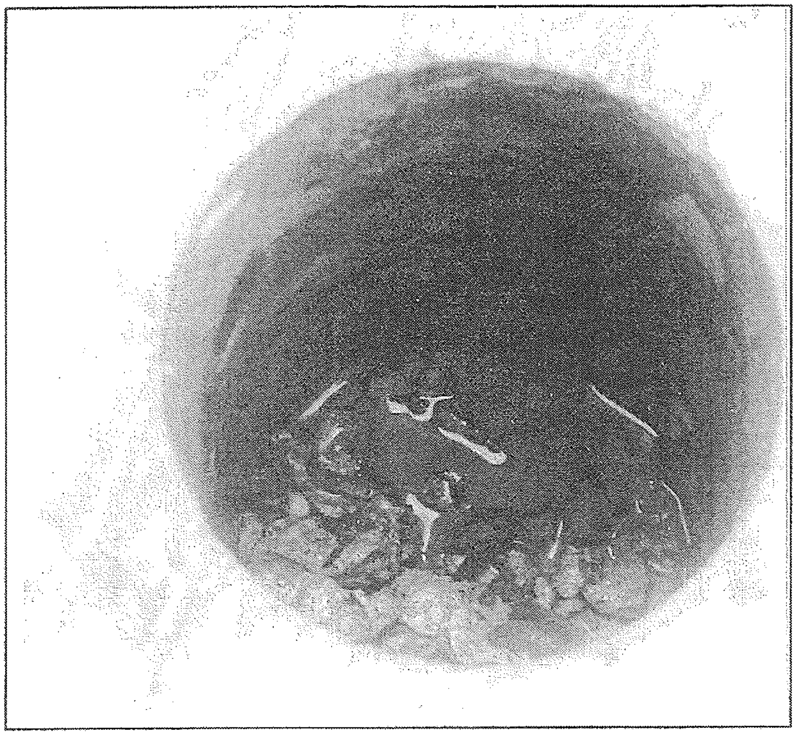


Figure 36

4" Diameter Core Hole for Pull-off Bond Test Showing Loose Aggregate at Bottom of Hole.

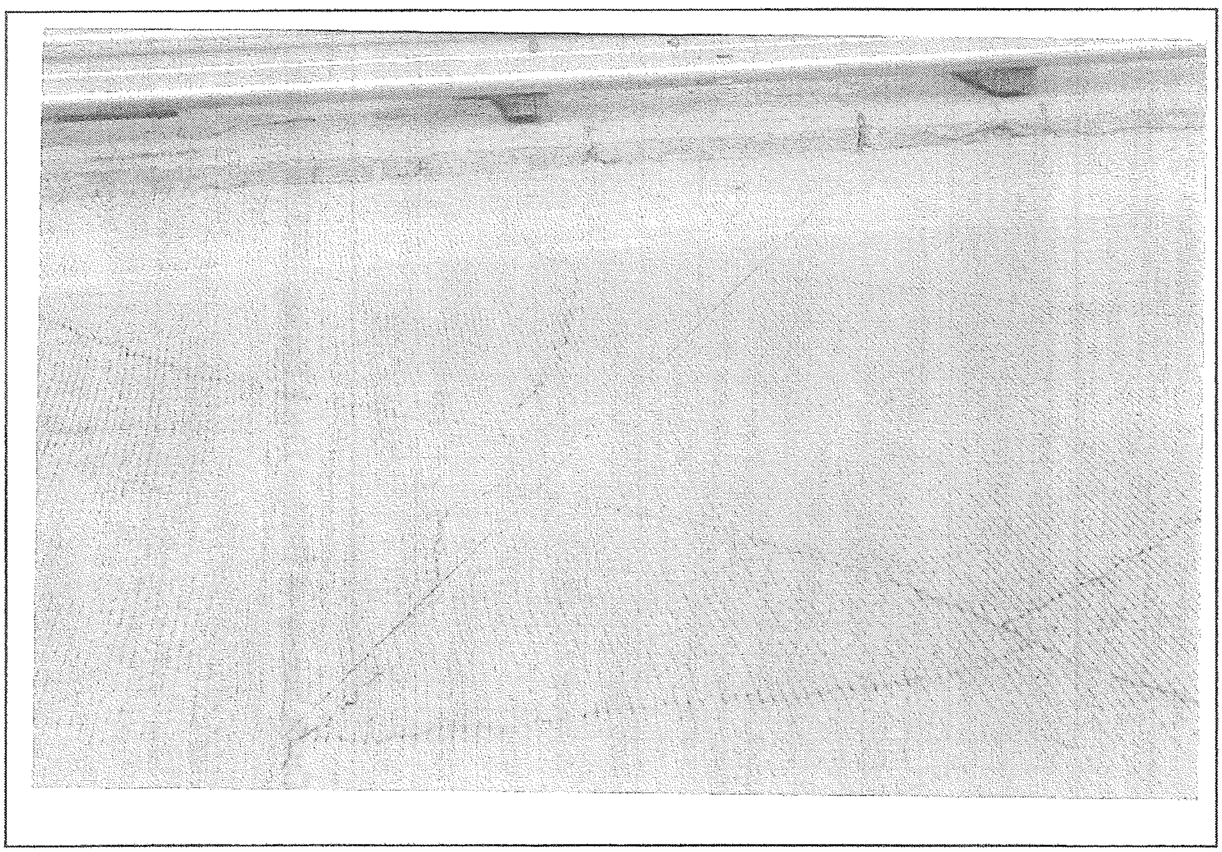


Figure 37 - Showing Cracked Bridge Deck. Spring 1995 Inspection

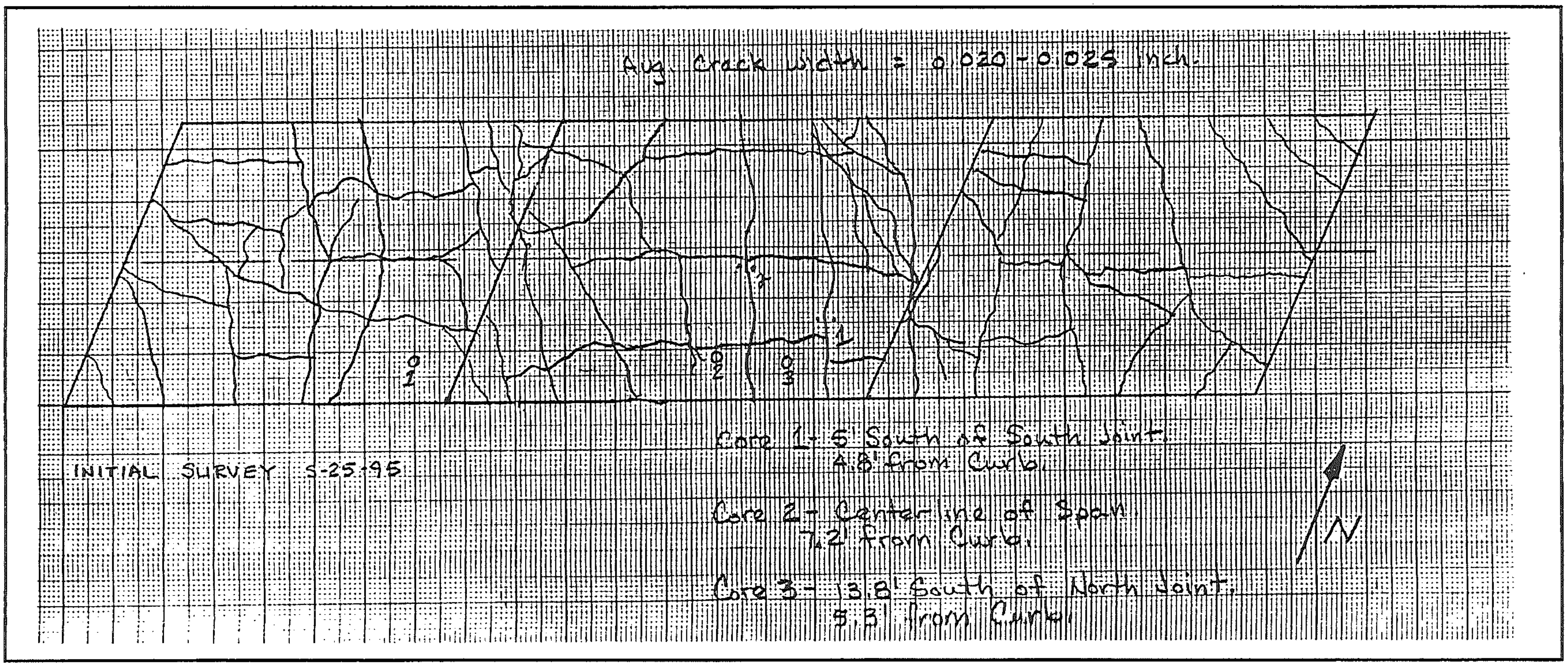



Figure 38 - Crack Survey of Bridge Deck Spring 1995 Inspection.

Post Inspection Analysis (May 1995)

Although the bridge demonstrated composite action of the new concrete overlay during the load test, our May 1995 inspection showed that this composite action may be unreliable as time passes because the overlay appears to be separating from the existing deck. Since this can change the load carrying capacity of the structure, we re-analyzed the structure to see if the 45 ton posting for the two-unit vehicle was still safe. To determine how the structure will respond to load in the future, we needed to determine why the concrete overlay, which is made of good strong concrete and reinforcing steel, is deteriorating so quickly, and why it is separating from the existing deck. We feel negative moment created by the overlay's reinforcing steel that extends through the expansion joint at each pier causes uplift of the overlay, causing it to separate from the existing deck (see Figure 39). Large moments are developed in the overlay over the piers, thus causing the overlay to crack. In the end-spans near abutment A and abutment B, and in the center portion of span 2, there is no uplift force; thus less cause for the overlay to separate from the existing deck.

The Bridge Management Unit found the weak component of the structure was mid-span of span 2 where they estimated the concrete in the deck, 2½ in. estimate flange thickness, would fail in compression. With the addition of the reinforced concrete overlay, the actual flange thickness is 11 in. At the center one-third portion of span 2, we estimate the maximum live load horizontal shear stress at the bond interface is near zero. We also showed there is no uplift forces in the overlay in this region. Therefore, we feel the overlay can be considered partially composite with the existing deck at this location (i.e. center span of span 2).

We used the following assumptions for our May 1995 post-inspection analysis:

1.) The structure reacts as simple spans.
2.) At center-span of span 2, 50 percent of the actual flange thickness; (i.e. overlay thickness plus existing deck thickness) can be used as the resisting section.
3.) Since we found only small corrosion loss to the reinforcing steel, 90 percent of its cross-section can be used.
4.) During the load test, we found the maximum distribution of load to any beam is approximately 26 percent. This value can be used rather than the 31.7 percent AASHTO distribution factor.

Using these assumptions, we found the 45 ton, two-unit vehicle posting is still adequate.

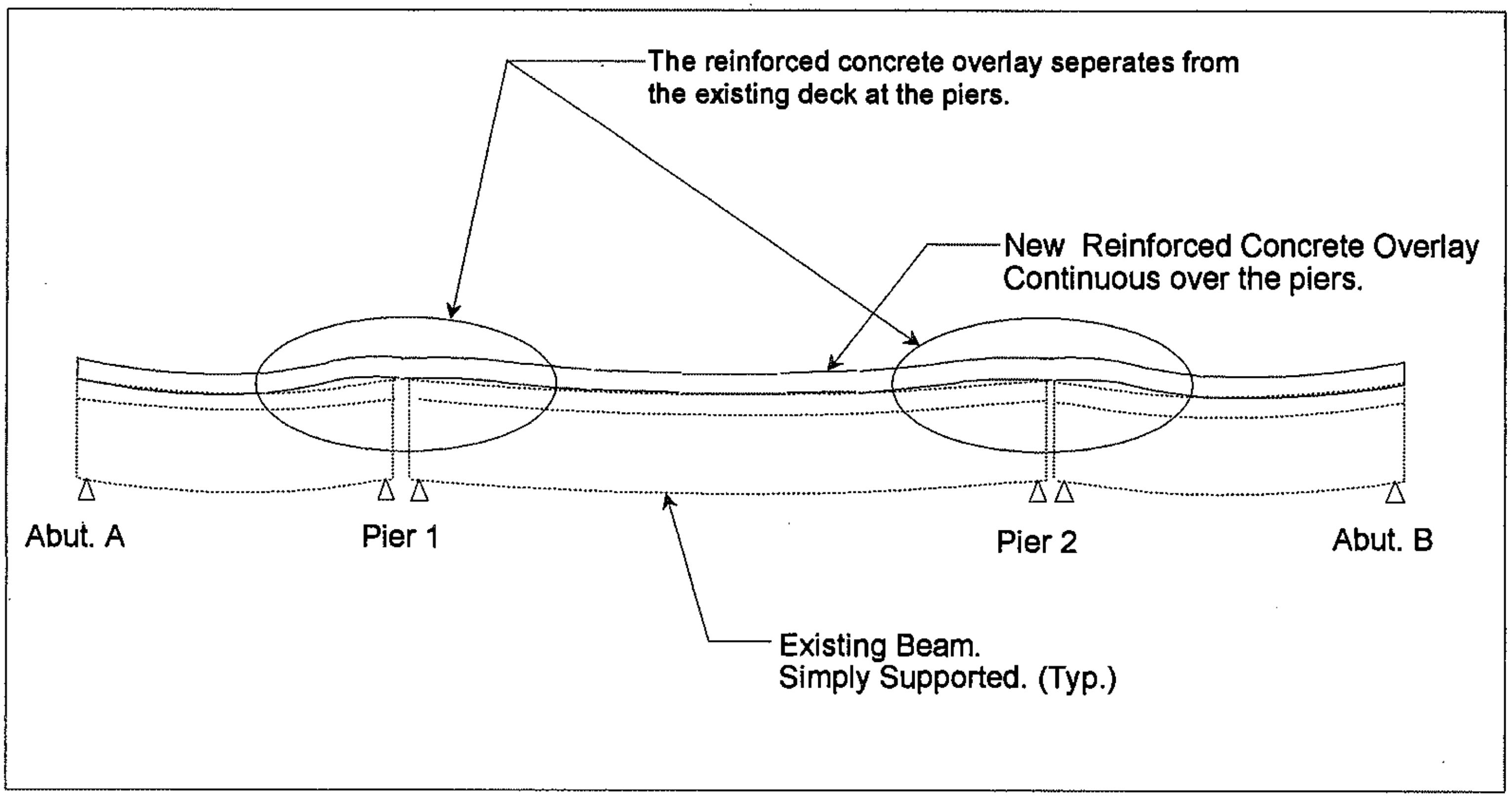



**Figure 39.**
**Showing the reinforced concrete overlay seperating form the existing deck.**

## CONCLUSIONS AND RECOMMENDATIONS

Because the bridge successfully carried the 82-ton proof load with no detectable distress during the load test in November 1994, we recommended the Bridge Management Unit keep the existing posted loading of 45 tons for a two-unit, 11 axle vehicle. However, we felt the posting should not exceed 45 tons, due to the condition of the fascia beams. The Bridge Management Unit concurred with our recommendation and the bridge was opened to traffic in November 1994.

In May 1995, we inspected the bridge to find the concrete overlay was deteriorating rapidly, and it was separating from the deck at the piers. This prompted us to reanalyze the structure with new assumptions. We found the 45 ton, two-unit vehicle posting is still adequate.

We recommend the department continue efforts to replace this bridge. The fascia beams and piers are in serious condition and the concrete overlay is deteriorating rapidly. The Design Division should review this report's findings and the above referenced analysis to determine if they concur with the current bridge posting.

The Maintenance Division should continue to monitor the structure on a six month inspection schedule, with emphasis on the fascia beams, condition of the deck, shear cracks in the end spans, and condition of the piers.

If the structure remains in service for more than five years we recommend that the department repair the two fascia beams and the interior beams that are in poor condition, and retest the structure to detect further deterioration and consequent loss of load capacity.