SCHOOL OF CIVIL ENGINEERING

JOINT HIGHWAY RESEARCH PROJECT

JHRP-75-15

USE OF PRECAST-PRESTRESSED CONCRETE FOR BRIDGE DECKS

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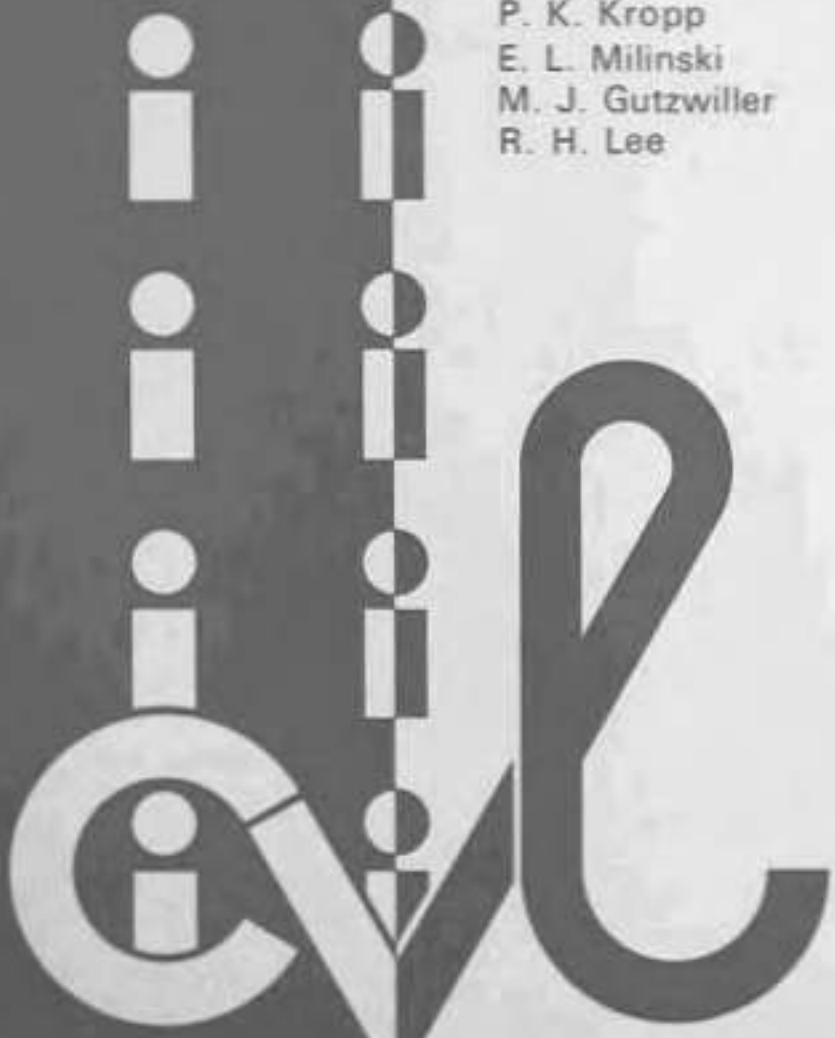

PURDUE UNIVERSITY
INDIANA STATE HIGHWAY COMMISSION

Final Report

USE OF PRECAST-PRESTRESSED CONCRETE FOR BRIDGE DECKS

TO: J. F. McLaughlin, Director
Joint Highway Research Project

July 30, 1975
Revised Dec. 1976
Project: C-36-56N

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

File: 7-4-14

Attached is the Final Report on the HPR Part II Research Study titled "Use of Precast-Prestressed Concrete for Bridge Decks". This Report has been authored by Messrs. P. K. Kropp, E. L. Milinski, M. J. Gutzwiller and R. H. Lee. Professors Gutzwiller and Lee together with Professor C. F. Scholer were the principal investigators and directed the research.

The investigator has shown that precast prestressed concrete bridge decks produce a durable deck which can be constructed in a minimum of time. Some water leakage through the joints is occurring and recommendations include application of a waterproof membrane over the deck or sealing of the joints and then an asphalt overlay both for the two experimental bridges now four years old and for future bridges.

This Report is submitted for acceptance as fulfillment of the objectives of this research. It will also be forwarded for review, comment and similar acceptance to ISHC and FHWA.

Respectfully submitted,

Harold L. Michael
Associate Director

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Final Report

USE OF PRECAST-PRESTRESSED CONCRETE FOR BRIDGE DECKS

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16. Abstract

A precast, prestressed concrete bridge deck system has been designed to be used either on new bridges or for the replacement of deteriorated decks. The report summarizes the construction activities, static load tests, and the present physical conditions of the two field installations constructed during the summer of 1970.

This investigation has shown that precast prestressed concrete bridge decks provide a durable deck which can be constructed in a minimum amount of time. The actual time required to place the plank was two days at both field installations. Static load tests and visual inspections have indicated no significant changes in deck performance over the past five years.

17. Key Words	18. Distribution Statement
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FINAL REPORT

Use of Precast-Prestressed Concrete for Bridge Decks

INTRODUCTION

The repair of concrete bridge decks on many of our highways continues to be an expensive and complex problem. The use of precast prestressed concrete for bridge decks can partially solve this problem by increasing the durability of the concrete through plant quality control and by reducing the time required to construct a bridge deck.

A complete deck consists of precast prestressed slabs, each having a minimum thickness of 6", a length equal to the transverse dimension of the bridge, and a width of at least four feet. The slabs are placed transversely to the bridge stringers and connected to the top flanges of the stringers by means of spring clips which are bolted to inserts cast in the concrete. A pretensioned stress is applied in the longitudinal direction of the slabs.

Two bridges were constructed in 1970 using the above described concept; one was a replacement deck for an existing bridge near Bloomington, Indiana (See Figure No. 1); and the second was a deck for a new bridge near Knightstown, Indiana (See Figure No. 2). Since 1970 both bridges have been periodically monitored through the use of permanent strain gage installations mounted on both the steel beams and the precast slabs. Periodic visual inspections were also conducted to assess the condition of the joints and slabs. To date both bridges have performed satisfactorily with the exception of a few minor problems.

The following paper is the third and final report to be published on the research conducted at Purdue University on the subject of precast, prestressed bridge decks. The first report was written by James Ford in 1969

Figure #1. Bloomington Bridge SR-37

Figure #2. Knightstown Bridge SR-140

and described the laboratory research conducted on prototype slabs. An interim report by Peter K. Kropp was published in March 1973 which described the construction and initial load tests conducted on both the Bloomington and Knightstown bridges. This final report will first describe the construction procedures used to complete both bridge decks. These procedures were also described in the interim report but in some cases these procedures are important in understanding why certain problems have arisen. The present physical condition of the bridges will also be discussed in terms of the areas where problems have arisen, the possible solutions to these problems, and recommendations for further research and for future projects.

Bloomington Bridge

During the summer of 1970 the above described precast prestressed deck was installed as a replacement deck on a 2 lane bridge located 3 miles north of Bloomington, Indiana on SR 37. The existing structure is an eight-panel thru type pony truss with a single span of 125 feet over Bean Blossom Creek.

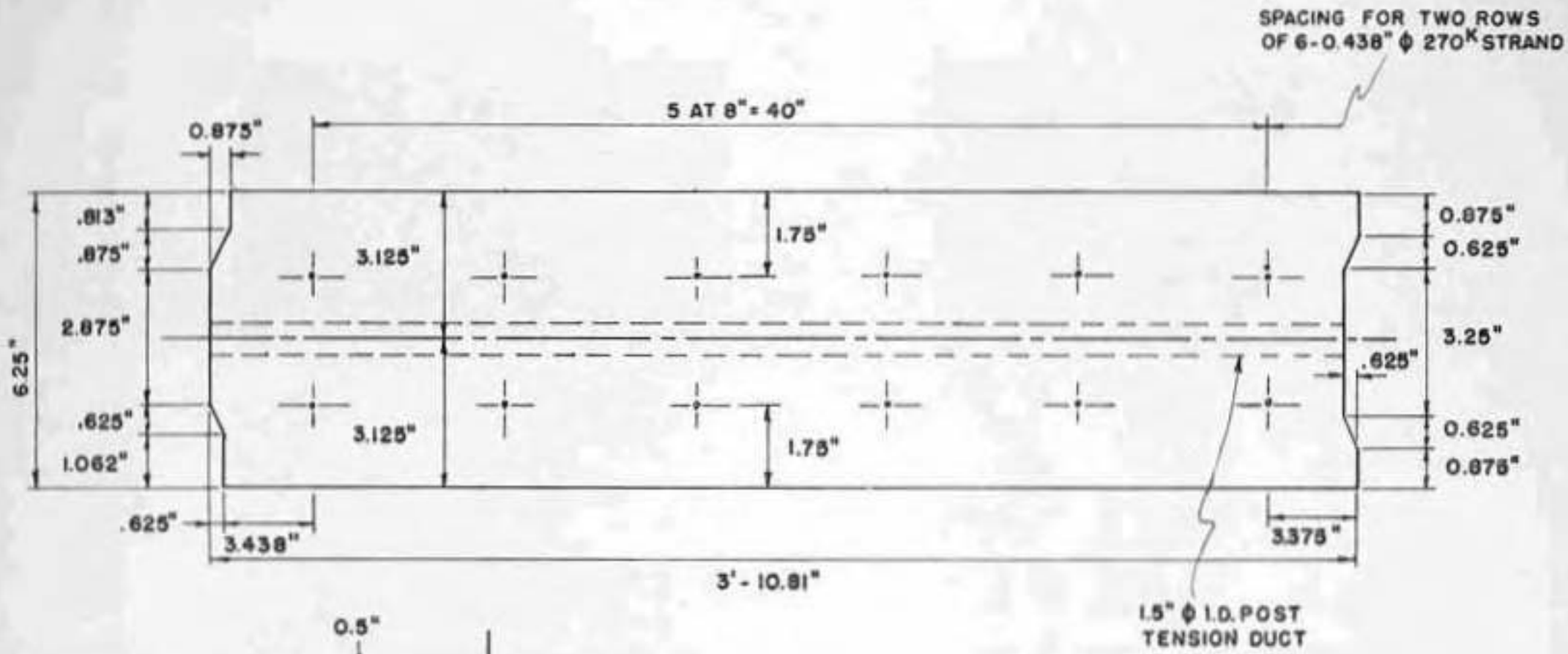
Construction was started on June 1, 1970 with the removal of the old reinforced concrete deck after which the top flanges of the stringers and floor beams were sandblasted and coated with epoxy. The top flanges were prepared in this manner in order that a layer of epoxy could be placed on the steel stringers to provide a proper bearing for the new precast slabs. Since the roadway crown had been built in with the stringers, the two exterior stringers were raised 2" and a 5/8" thick continuous plate was tack-welded to the flanges of the two first interior stringers. These adjustments, coupled with the existing stringer crown, allowed the precast slabs to deflect to conform to the adjusted contour without cracking. Since

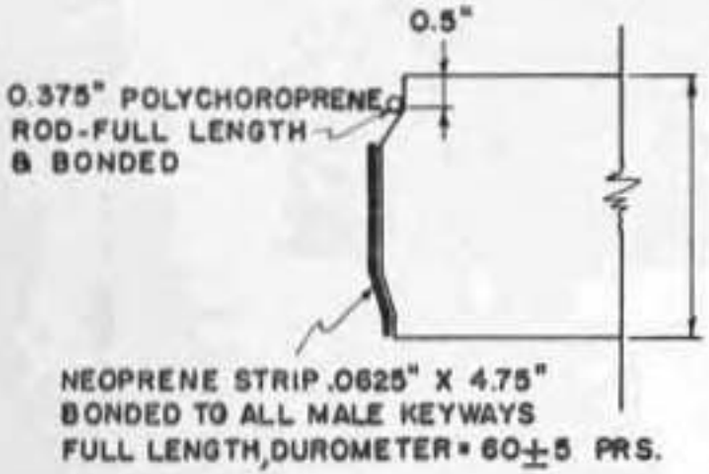
the top flanges of the stringers were higher than the floor beam flanges, steel shims were tack-welded to the floor beam flanges to the same elevation as the top of the stringers framing into the floor beam. This provided a continuous support for the slabs at the stringer locations.

The slabs were cast by Construction Products Corporation of Lafayette, Indiana. A typical slab (See Figure #3) contained twelve 7/16" diameter 270^{k} strand with an initial pull of 21,700 lbs/strand. Each slab was 32 feet long, 6-1/4 inches deep and approximately 4 feet wide. After each slab was detensioned and removed from the form a 1/16" x 4-1/4" x 32' neoprene strip was bonded to the male keyway. Just before the slabs were placed on the bridge a 3/8" diameter x 32' polychloroprene flexible rod was bonded to the male keyway to serve as a back-up material for the joint sealant. The specified concrete strength was 5000 psi at 28 days with the actual cylinder strengths averaging 6900 psi with an air content of 5% to 5-1/2%. Each slab was lightly broomed longitudinally for added skid resistance.

The slabs were delivered to the bridge by truck with 4 slabs per truck. The slabs were lifted directly from the truck and placed on the bridge using a truck-mounted crane. On the first day, June 30, 21 slabs were set in 7-1/2 hours. On the following day the remaining 12 slabs were placed and the entire deck was post-tensioned.

Slab placement began at the north end of the bridge and proceeded southward. Figure 4 shows the placement operation. After each slab was in place, it was lightly fastened to the two outside stringers and to the middle stringer using 115-RE-F railroad clips and 3/4" diameter bolts. The bolts were not tightened so that during the post-tensioning stage the slab could move longitudinally, but would not buckle. When four slabs had been set, three strands of post-tensioning cable were fed through the slabs, one at each edge and one in the middle. A force of 2000 lbs. was then





JOINT MATERIAL DETAIL

FIGURE NO. 3 - TYPICAL PRECAST SLAB SECTION and JOINT MATERIAL DETAIL

BLOOMINGTON BRIDGE

Figure #4. Placement of the Precast Slabs at the Bloomington Bridge.

applied to each of the three strands to snug up the slabs. This process was repeated after another four slabs were placed until all the slabs were set.

After all the slabs had been placed, the remaining five post-tensioning cables were fed through the slabs. All eight post-tensioning strands were 1/2" diameter 270^{k} and were encased in waterproof tubing. An initial jacking force of 16,000 lbs. was applied to each of the eight strands using the two hydraulic jacks symmetrically about the bridge center line. During this initial post-tensioning, the truck crane which was used to set the slabs was parked at the north end of the bridge to keep the first eight or so slabs from developing an excessive compressive stress in the slab joints since the joints were fairly tight to begin with. These joints at the north end were tight due to the above mentioned "snugging" force applied after each set of four slabs were placed.

After the 16,000 lbs. was applied to each of the eight strands, the truck crane was driven up and down the center of the bridge. This operation was done to release any slabs which had somehow locked up on the stringers and caused the total length of the bridge deck to shorten by 3/8". The truck crane was then driven off the bridge and the final post-tensioning force of 28,900 lbs. was symmetrically applied to all eight strands. One of the problems encountered during this final post-tensioning from 16,000 lbs. to 28,900 lbs. was that the chucks on some of the strands would not immediately release their grip. The jaws of the chucks had become "set" in the strand at 16,000 lbs. and had the initial force level been higher than 16,000 lbs., it is doubtful whether the chucks would have released easily upon retensioning.

The only problem encountered during the actual slab placement came in placing the individual slabs which were directly over a floor beam. A

block-out to accommodate connection angles, which protruded from the top of each end of the floor beam, was not large enough to allow for "slack" in the slab placement. This difficulty was remedied by burning off the piece of each angle which was interfering with the slab block-out.

Following the final post-tensioning operation all of the slabs were permanently fastened to the stringer flanges using the 115-RE-F railroad clips bolted to 3/4" diameter inserts cast in the slabs (See Figure #5). The 115-RE-F clips used were different from those specified and could not be drawn up against the lower part of the top flange because the offset spacer on the clip was deeper than the flange was thick. All of the offset spacers had to be burned off, the bolts were greased, and the bolts were tightened with an electric torque wrench to approximately 50 ft lbs. of torque.

The joints were then prepared for the joint sealant. Each joint was brushed and cleaned with an air jet, and DuPont primer VM-22 was applied to the walls of the joint, and the sealant was then placed in the joints. Two types of polyurethane elastomeric joint sealant were used. An elastomeric sealant could be used in this case since the relative movements at the joints is minimized due to the post-tensioning of the slabs. On the first four joints at the south end of the bridge DuPont 829-915 "Imron" was used which is the same sealant evaulated in the laboratory tests on joint sealants. For best results, this material requires an approximately square cross section, necessitating the use of the previously described polychoroprene back-up rod near the top of the joint. The "Imron" was applied with a caulking gun and then tooled into the joint with a soapy finger. The sealant used on the remaining joints had essentially the same composition, but was in liquid form and was poured into the joint. Since it was a liquid, it seeped past the beaded strip and the joint had to be filled several times. One

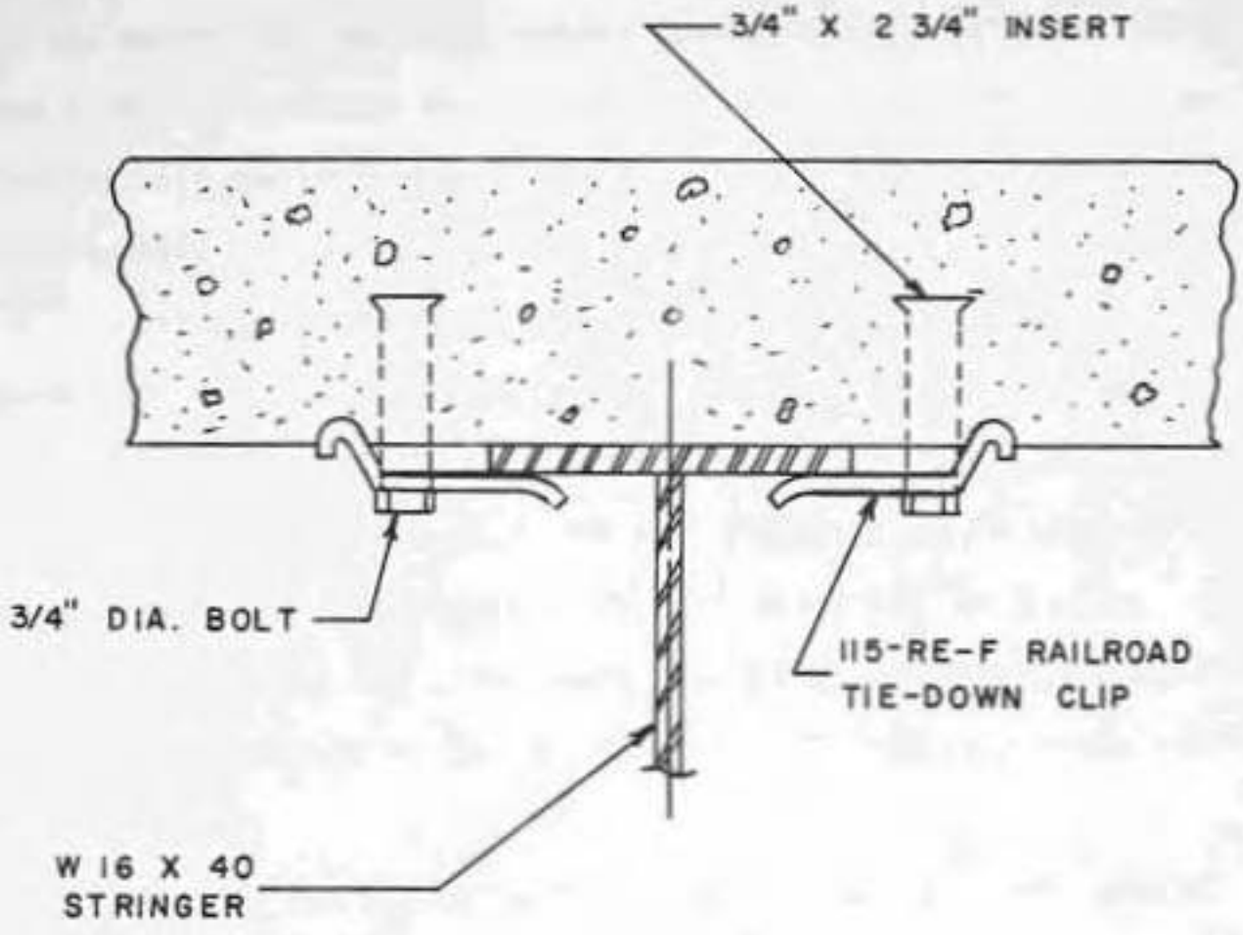


FIGURE NO. 5

SECTION OF A TYPICAL SLAB TIE-DOWN CONNECTION

BLOOMINGTON AND KNIGHTSTOWN BRIDGE

problem which arose with the liquid sealant was that air became entrapped in the sealant and expanded, pushing some of the sealant out of the joint and forming a permanent bubble above the surface of the slabs. Under traffic this bubble wore away, leaving a void in the seal which subsequently led to water seepage through the joint.

In areas where the slabs did not bear directly on the stringers, epoxy mortar was used to fill these spaces. This tuck pointing was accomplished using a small mason's trowel to force the mortar between the bottom of the slab and the top flange of the stringer. This was only successful where the space between the top flange and slab bottom was 1/4" or greater. If the space was less than 1/4", the laborer had difficulty getting enough epoxy mortar into the space to form a uniform bearing surface over the entire flange surface.

The bridge was opened to traffic on July 16, 1970. The contract specified that the bridge would be closed for thirty days but due to the extra work of raising the stringer elevations and a delay in the delivery of the slabs, the bridge was opened fifteen days behind schedule. Table #1 shows the time spent on the various construction operations.

TABLE 1

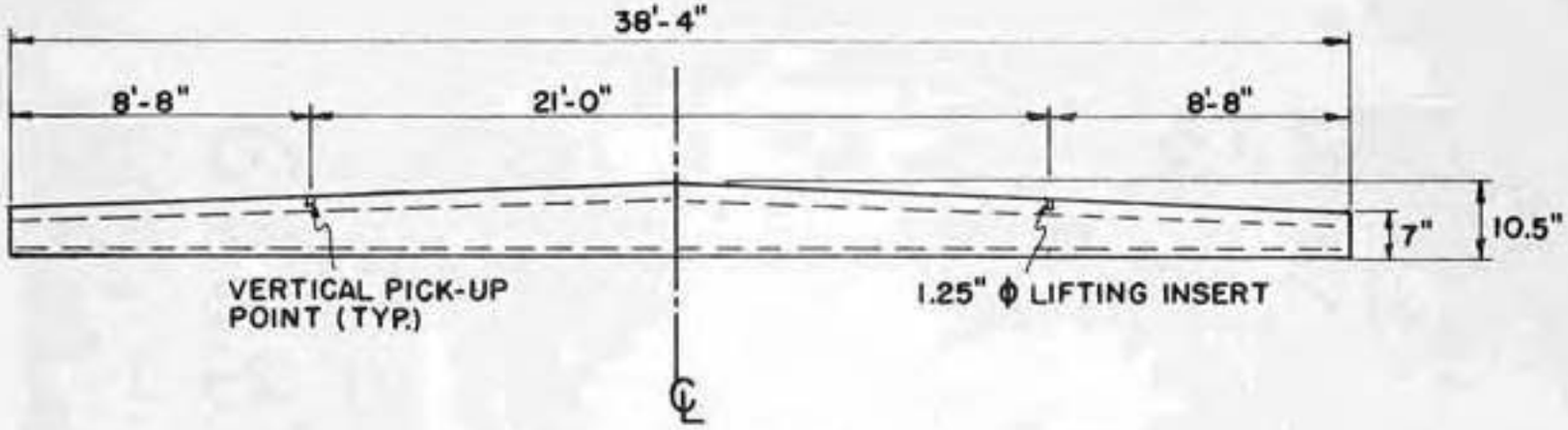
TIME SPENT FOR VARIOUS OPERATIONS - BLOOMINGTON BRIDGE

Operation	Time Spent
Removal of old deck and the north and south approaches	5 days
Sandblasting stringers, floor beams - applying epoxy coating	7 days
Raising outside stringers, tack welding plate to stringers and floor beams	3 1/2 days
Placing precast slabs and post tensioning	2 days
Bolting slabs to stringers	5 days
Joint preparation and applying joint sealant	5 days
Epoxy mortar tuck pointing between the stringers and floor slabs	3 days
Expansion joint installation	2 days
Forming and pouring mudwalls	4 days
Forming and pouring approaches - guardrail installation	5 days
No work due to delivery delays	4 days
No work due to weather conditions	1 day

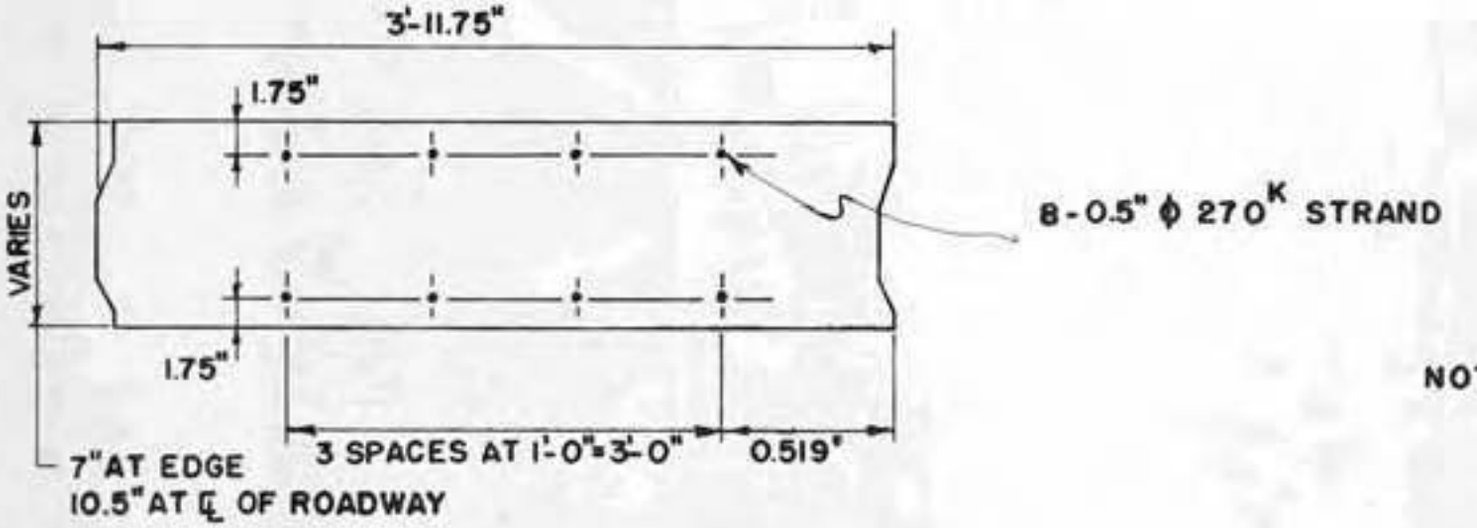
Knightstown Bridge

The second experimental field installation is on Indiana State Road 140 over the Big Blue River just south of Knightstown. The structure is a three span continuous steel beam bridge having spans of 70'-70'-60'. The deck consists of 52 individual slabs, 38'-4" long and 3'-10 13/16" wide, positioned on steel beams spaced at six feet on center. The slabs have a built in crown with the thickness of each slab varying from seven inches at each end to ten and one-half inches at the center. Figure #6 shows an elevation and cross section of a typical slab. The keyway dimensions are identical with those used on the Bloomington bridge. The neoprene sheet between each slab is 1/16" x 6", bonded to the male keyways over the full length of each slab. As in the Bloomington installation a flexible polychloroprene back-up rod was used at a distance of 3/16" below the top of each slab. The slab was anchored to the beams with 115-RE-F railroad clips and 3/4" diameter bolts. In this case the offset spacer on the clip did not have to be burned off as at Bloomington since the flanges of the floor beams were of sufficient thickness that the clip could be drawn up against the lower side of the upper flange. A typical slab contained eight 1/2" diameter 270 kip pretensioning strands with an initial pull of 28,900 lbs. The top strands were harped at the center to follow the crown of the slabs and thus maintain a uniform compressive stress across the slab section. The concrete strength at 28 days was specified at 5000 psi and the surface was broom finished. The Knightstown slabs were also manufactured by Construction Products Corporation of Lafayette, Indiana.

The construction of the bridge was started in the early spring of 1970. After completion of piers and erection of the steel beams, the first 15 precast slabs were placed on June 24 and on June 25 the remaining 37 slabs



ELEVATION



NOTE: SEE FIGURE NO. 3 FOR KEYWAY DIMENSIONS

SECTION

FIGURE NO. - 6 TYPICAL ELEVATION AND CROSS SECTION OF THE KNIGHTSTOWN BRIDGE SLABS

Figure #7. Precast Slab Placement at the Knightstown Bridge

were set and post-tensioned. Figure #7 shows the slab placement operation. A problem which arose during the post-tensioning operation was the appearance of eight 2'-0" long cracks which ran in the longitudinal direction of the bridge. This was due to irregularities in the top portions of the joints which caused a poor fit between adjacent units. The post-tensioning operation caused a bending stress in the slabs which fit poorly, resulting in the cracks. Due to the fact that water could seep down into these cracks and rust the pretensioning cables, a seal coat of epoxy was applied to the bridge. Integral curbs were poured in place on the bridge deck and the bridge was opened to traffic on August 6, 1970.

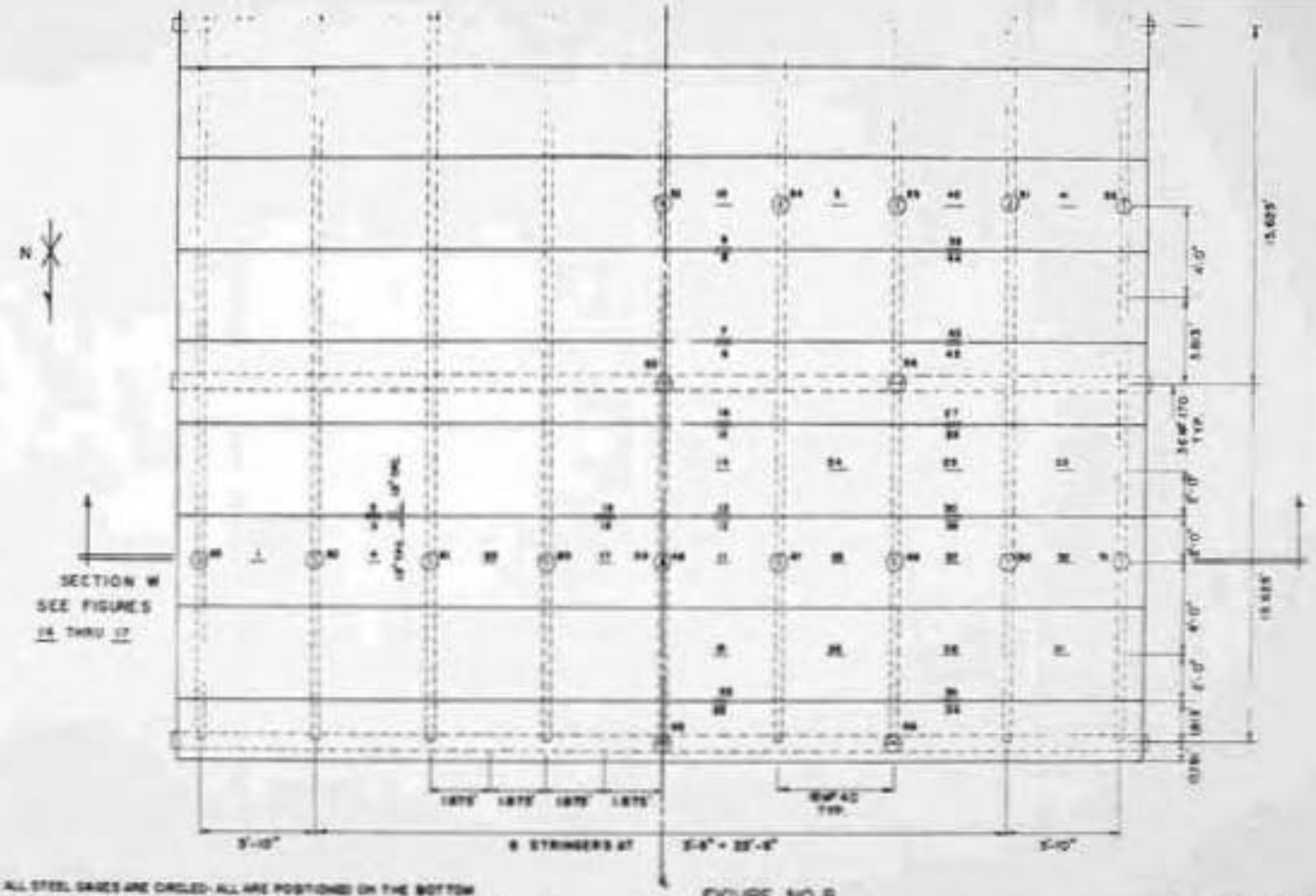
FIELD TESTING

In order to interpret the results of periodic static load tests, strain gages were mounted on both the precast slabs and the supporting steel structure at Bloomington and on the precast slabs at Knightstown. Load tests have been periodically conducted on both structures since construction was completed. Figure #8 shows the Bloomington gage locations and Figure #9 shows the Knightstown gage locations. In all cases the strain gages were positioned so as to attempt to record maximum strain under working loads.

On the Bloomington bridge there were a total of eighteen gages on the steel stringers and floor beams and forty-five gages on the underside of the precast slabs. The strain gages mounted on the steel were Microdot type SG189-6 which had a gage factor of 1.80 ± 3%, a gage length of 1", and a nominal resistance of 120 ohms. The concrete gages were Micro-Measurements foil gages with a two inch gage length, a resistance of 120 ohms ± 0.2%, and a gage factor of 2.11 ± 0.5%.

The lead wires from both the steel and concrete gages were protected by EMT thinwall conduit which was installed between each gage and a central watertight junction box. Inside the junction box, the wires from each gage were soldered to a 52 pin female Amphenol connector with each connector accomodating ten strain gages. During a load test a 52 pin male Amphenol connector was attached to the female connector in the junction box and the various strains were recorded using a data acquisition system.

At the Knightstown bridge the forty concrete strain gages were identical to those used on the Bloomington Bridge. All gages are wired to four 52 pin female Amphenol connections in the central junction box as shown in Figure #10.



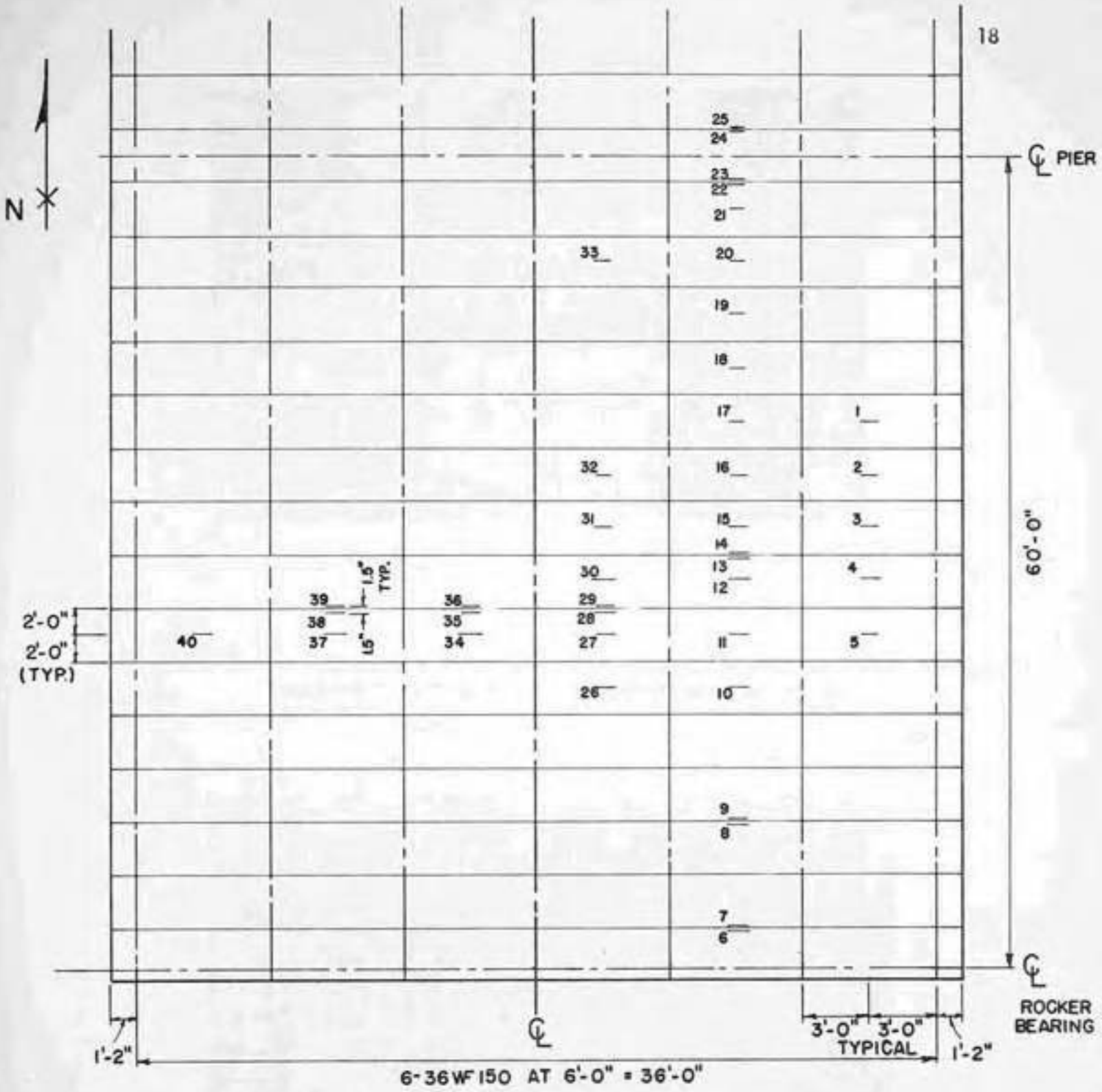
NOTES: ALL STEEL GAGES ARE CIRCLED - ALL ARE POSITIONED ON THE BOTTOM OF THE BOTTOM FLANGE - GAGE ORIENTATION AS SHOWN. ALL CONCRETE GAGES ARE POSITIONED ON THE BOTTOM OF THE SLABS AS SHOWN. GAGE ORIENTATION AS SHOWN

FIGURE NO. 5

PLAN OF THE STRAIN-GAGE LOCATIONS

BLOOMINGTON BRIDGE

SCALE: HOR. 1"=5'
VER. 1"=4'



NOTE: ALL GAGES ARE POSITIONED AND ORIENTED ON THE BOTTOM OF THE SLABS AS SHOWN.

FIGURE NO. 9

PLAN OF THE STRAIN-GAGE LOCATIONS

KNIGHTSTOWN BRIDGE

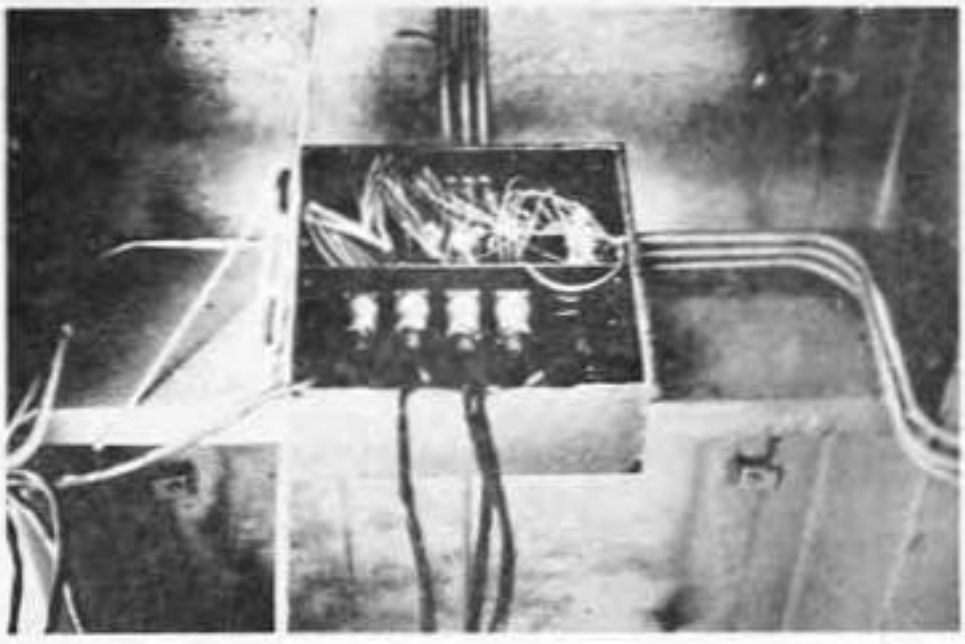


Figure #10. Junction Box at Knightstown Bridge Showing the Strain Gage Hook-Up from the 4-52 Pin Connectors to the Data Acquisition System

Figure #11. Data Acquisition System with the Switching Unit on the Left and the Strain Recorder on the Right

The equipment used to record strain readings during each load test was manufactured by Matrix Corporation of Indianapolis, Indiana and is shown in Figure #11. This data acquisition systemwill record the strains from one hundred gages in a matter of seconds. Before a test was made, the acquisition system was connected to the strain gages on the bridge using cables, with one end of the cable connected to the female Amphenol connectors in the junction box on the bridge (See Figure #10) and the other end plugged into the acquisition system (See Figure #11). Each cable contained wiring for ten strain gages.

Static Load Testing

In order to conduct a load test, an Indiana State Highway Commission tandem dump truck (See Figure #12) was loaded with material, which was usually sand, and both the front axle and tandem axle weights were recorded. The strain gages were attached to the data acquisition system, the bridge was closed to traffic and strain gage zero and calibration readings were recorded with no vehicles on the bridge. Then the test truck was positioned on the bridge and a record was made of the strains from all gages. The truck was then directed to a new position and the process repeated with each truck position being selected so as to attempt to give a maximum strain in as many strain gages as possible. Figure #13 shows the test truck wheel dimensions while Figures #14 through #17 show the truck positions used for the load test at the Bloomington bridge. Figure #18 shows the truck positions for the static load tests on the Knightstown bridge.

A static load test was performed on the Bloomington structure on June 1, 1970, prior to removal of the original bridge deck. The purpose of the test was to record steel strains with the original deck in place for later comparison with steel strains obtained after reconstruction. Fourteen (of the final nineteen) steel strain gages were fastened to the stringers and floor beams. The test truck was positioned over the points as shown in

Figure #12. Typical Test Truck for the Static Load Tests

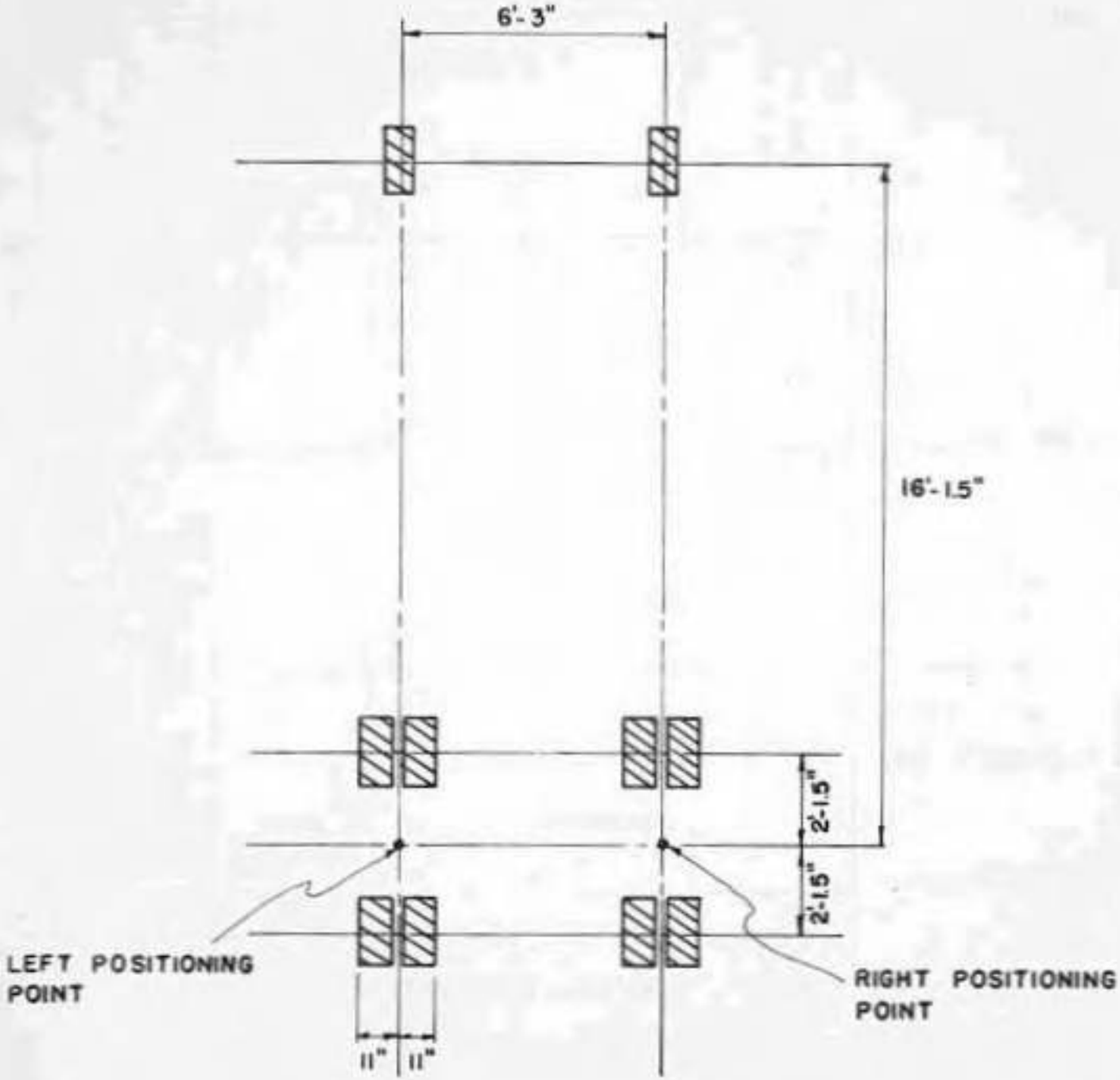
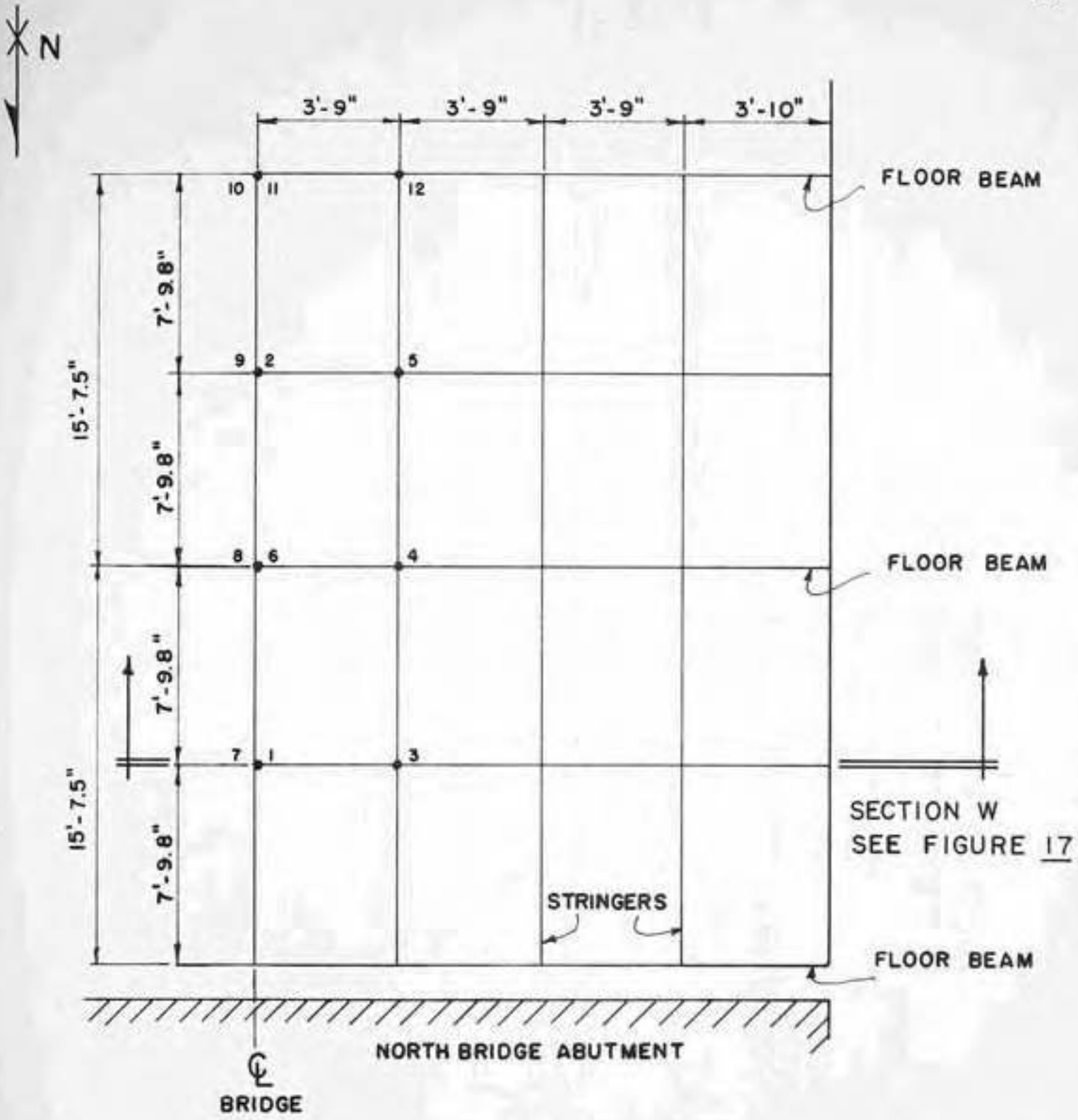


FIGURE NO. 13

WHEEL SPACING DIMENSIONS FOR THE TYPICAL TEST TRUCK



POSITIONS: 1,2,3,4,5,6,11,12 - TRUCK FACING SOUTH WITH LEFT SET OF TANDEMS LOCATED SYMMETRICALLY OVER THE POSITION.

7,8,9,10 - TRUCK FACING SOUTH WITH RIGHT SET OF TANDEMS LOCATED SYMMETRICALLY OVER THE POSITION.

NOTE: SEE FIGURE NO. 13 FOR THE WHEEL SPACING DIMENSIONS OF THE TEST TRUCK.

FIGURE NO. 14

TEST TRUCK LOADING POSITIONS

BLOOMINGTON BRIDGE

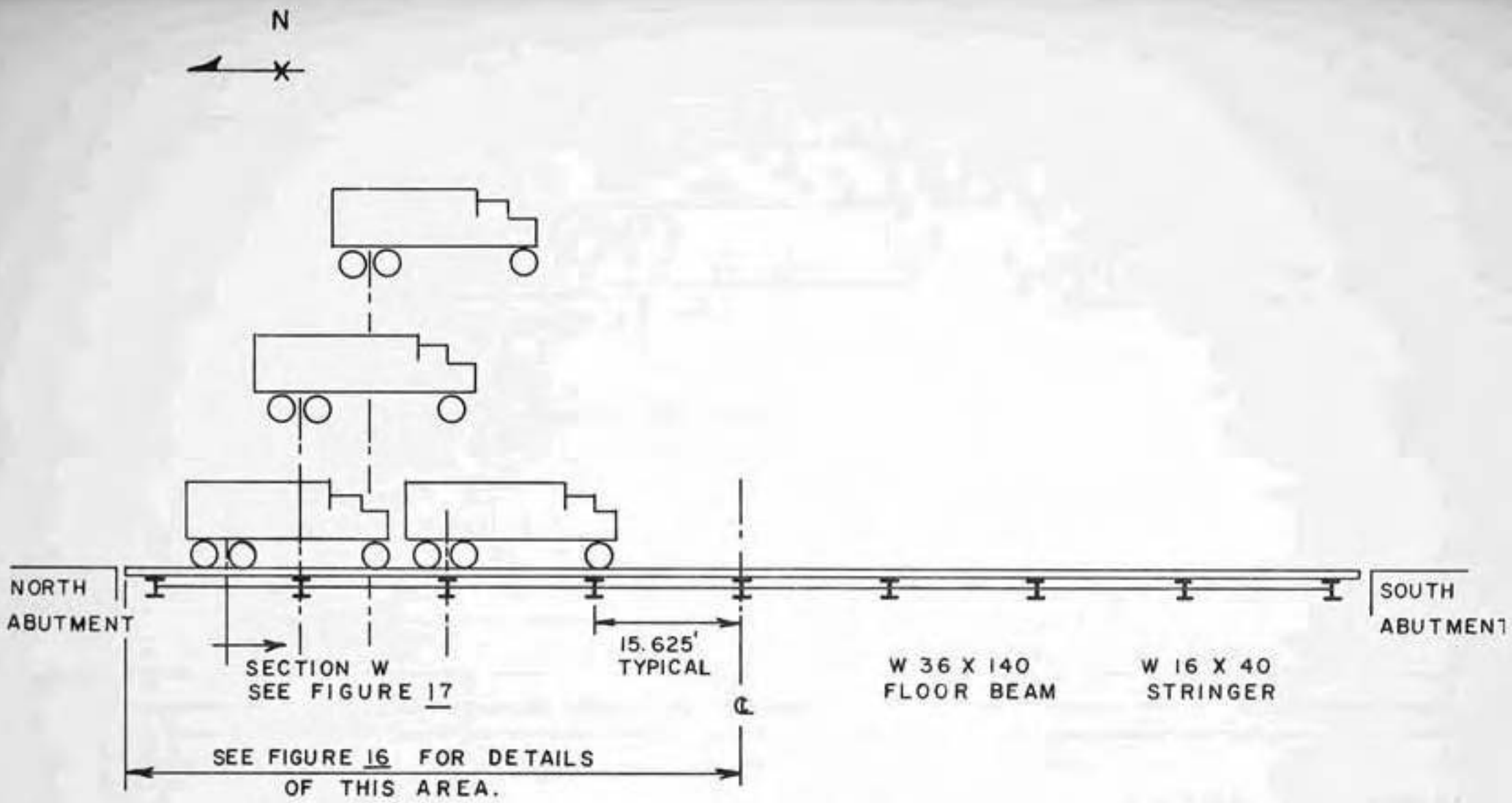


FIGURE NO.15

ELEVATION OF THE TEST
TRUCK POSITIONS

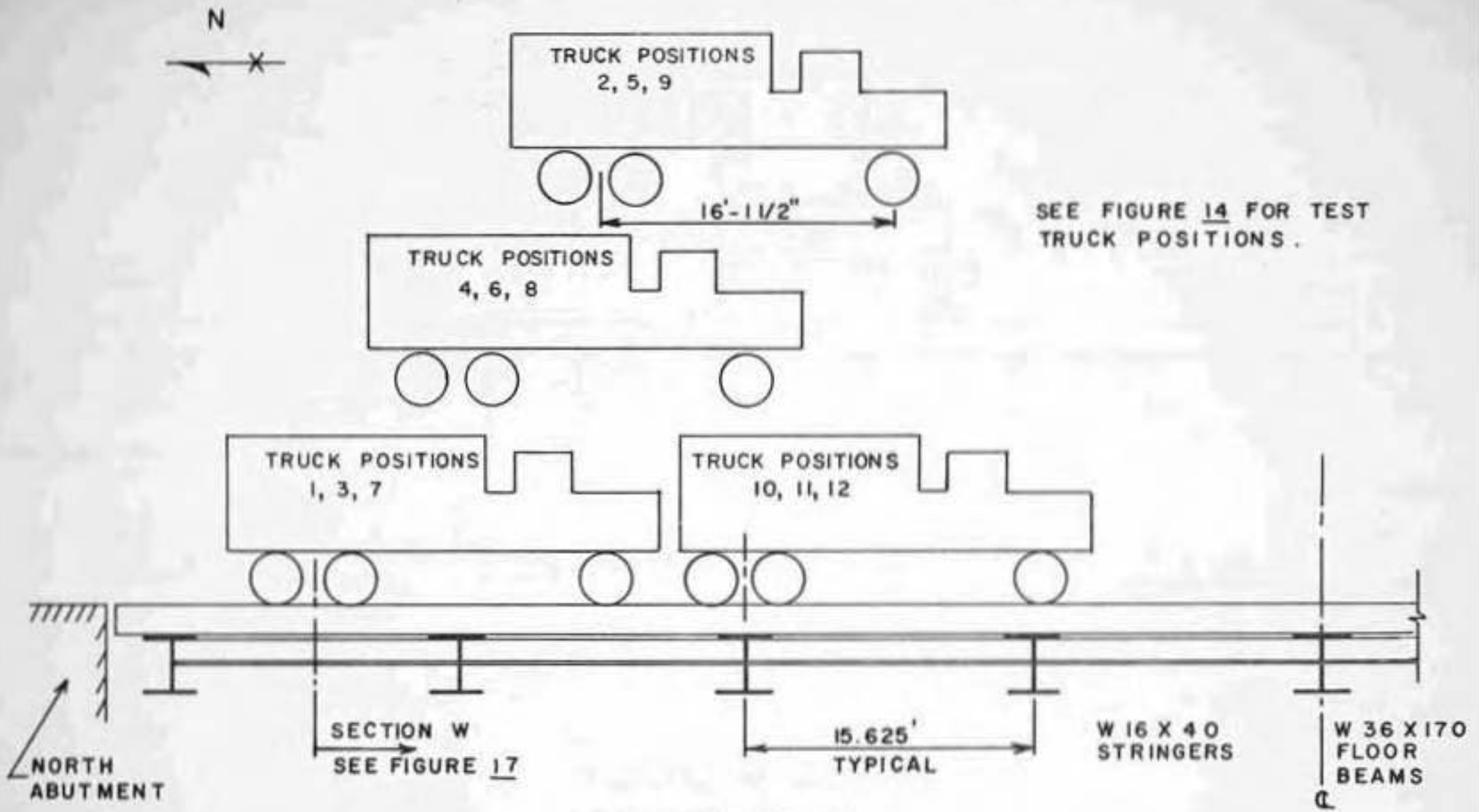


FIGURE NO. 16

DETAILED ELEVATION OF THE TEST TRUCK POSITIONS

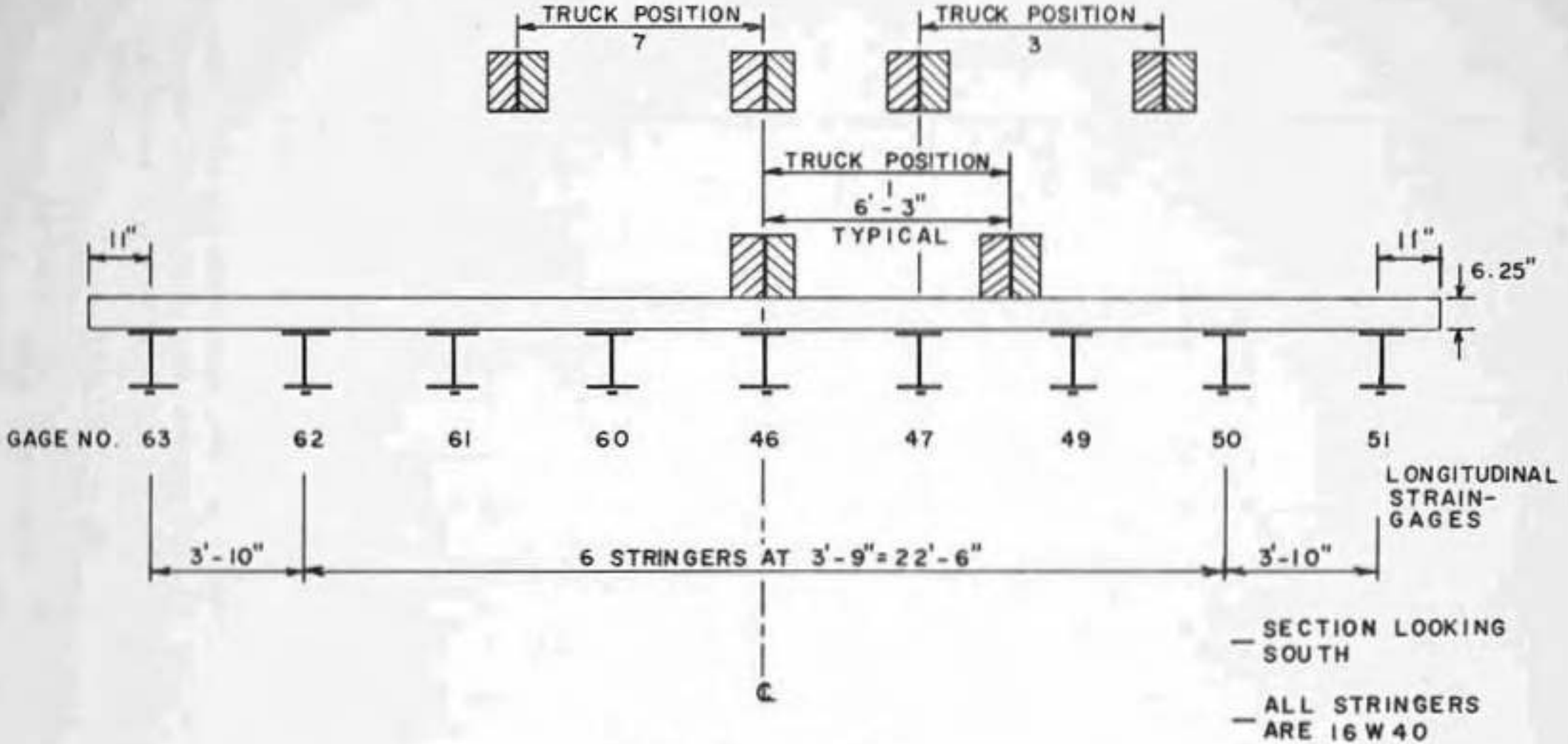
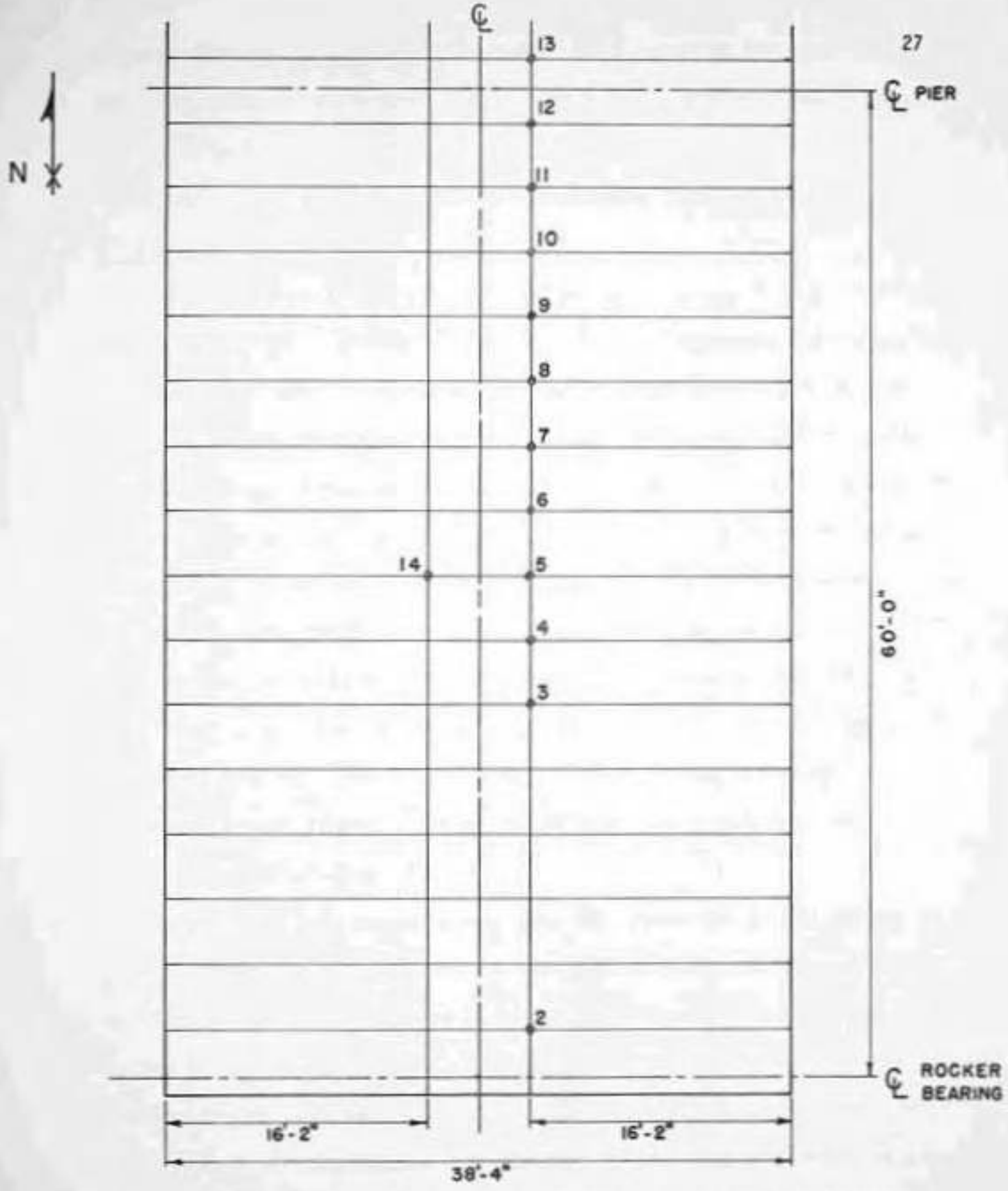


FIGURE NO. 17

SECTION W



NOTE: -TEST TRUCK IS IN NORTH-BOUND LANE FACING NORTH WITH THE LEFT SET OF TANDEMS LOCATED SYMMETRICAL OVER THE POSITION POINT.

-SEE FIGURE NO.13 FOR THE WHEEL SPACING DIMENSIONS OF THE TEST TRUCK.

FIGURE NO. 18

TEST TRUCK LOADING POSITIONS

KNIGHTSTOWN BRIDGE

Figures #14 through #17 and the strains were recorded for each truck position. The stresses were obtained by multiplying measured strains by a steel elastic modulus of 29 x 10^6 psi. These stresses and the test truck weight are shown in Table A1 in Appendix A.

Static Load Test Results

The main purpose for conducting load tests on the bridges was to determine if there was any significant change in stresses over a period of time. For example, a significant decrease in stress in a particular stringer may be indicative of a loss of bearing which could be caused by the tuck pointed epoxy mortar (See page 10) working out from between the stringer flange and bottom of the slab. In this case a corresponding increase in the adjacent stringer stresses would also be observed. Since the Bloomington bridge had a much higher traffic volume than the Knightstown bridge the initial tests at Bloomington were more frequent. The test interval was approximately one month intervals for Bloomington and two month intervals for Knightstown. However, these tests had to be scheduled around adverse weather conditions such as a heavy snow or high water under the bridges.

After each test the measured data were converted to the stress at each gage for every truck position by a computer program. A representative portion of this data is shown in Tables A2, A3 and A4 in Appendix A. Tables A2 and A3 include the stresses from eight of the sixty-three gages at Bloomington and Table A4 contains the results from six of the forty strain gages at Knightstown. If the measurement from a particular gage during a load test was obviously wrong, the stress for that gage was not included in the tables. The tabulated stresses in these tables are put into graphical form in Figures C1 through C14 in Appendix C for the purpose of showing the variation of stress with time for a particular gage. The plot for each gage is a linear least squares fit of the

tabulated stresses. Figures C1 through C8 show the graphs for the Bloomington bridge gages and Figures C9 through C14 show the results for the Knightstown gages.

It can be seen from the graphs that the stress variation over time for all the gages is relatively small. This is a good indication that the precast slabs haven't cracked and that the initial slab bearing conditions have remained the same. It should also be noted that the slope for the major portion of the gages is slightly decreasing which might be due to fatigue of the strain gages. Since the Bloomington strain gages were installed four years ago, they have experienced more than ten million load cycles which would make strain gage fatigue a distinct possibility. This fatigue would cause a slow increase in the resistance of the gage which would cause the gage to "see" less strain even though the actual strain remained the same. Obviously this reduction in strain output would manifest itself as a gradual reduction in the indicated stress over a long period of time.

During each load test there were many variables which could have affected the recorded strain. Some of these were variations in the test truck position (Figures #14 through #17), outside air temperatures, humidity, and the temperature changes within the data acquisition system. The influence of test truck positioning on the recorded strain was the only variable which could be easily and accurately checked. Therefore, two complete, consecutively run load tests (Tests 4 and 4A shown on Table A2) were conducted on the morning of January 14, 1971. During both tests the outside air temperature, humidity, and the air temperature inside the acquisition system remained reasonably constant. It can be seen from the test results shown on Table A2 that most of the stresses differ between the two tests by approximately 10%. Thus the "spotting" of the truck over the truck positions can cause a major portion of fluctuation of the stress over a period of time for a particular gage.

PRECAST SLAB LOAD DISTRIBUTION CHARACTERISTICS

In the following section the load distribution characteristics of the precast slabs will be discussed, utilizing the stress data collected during the load tests at the Bloomington bridge. The reasons for doing this are twofold: (1) To compare the actual load distribution characteristics to the load distribution factor (Table 1.3.1 (B) of the Highway Bridge Specs) given in the current 1973 edition of the Standard Specifications for Highway Bridges. (2) To see if there are any changes in the load distribution over a period of time.

For clarity in presentation, only one load position on the bridge will be studied. This will be a transverse section located at the north end of the Bloomington Bridge midway between the first and second floor beams (7'-10" from the north bridge support). At this transverse section, denoted "Section W",there is one longitudinal strain gage mounted symmetrically on the bottom flange of each of the nine stringers. "Section W" is also located where the maximum bending moment is expected to occur in the stringers. Refer to Figures #14 through #17 for the location of "Section W".

Located on section W are nine longitudinal strain gages, numbers 51, 50, 49, 47, 46, 60, 61, 62, and 63 and these are shown on Figure #17. In order to present the load distribution at Section W for any truck position, the stresses from each of the nine steel strain gages were averaged from three separate load tests. This average stress is plotted to a vertical scale directly above its corresponding stringer on the horizontal axis with the average stress points above the nine springers being connected by straight lines. To indicate the effects of various truck positions on the distribution of load at Section W, three different truck postion load distributions are superimposed on each other. Figure #19 shows the load distribution for truck positions #1, #3, and #7. The vertical axis is cali-

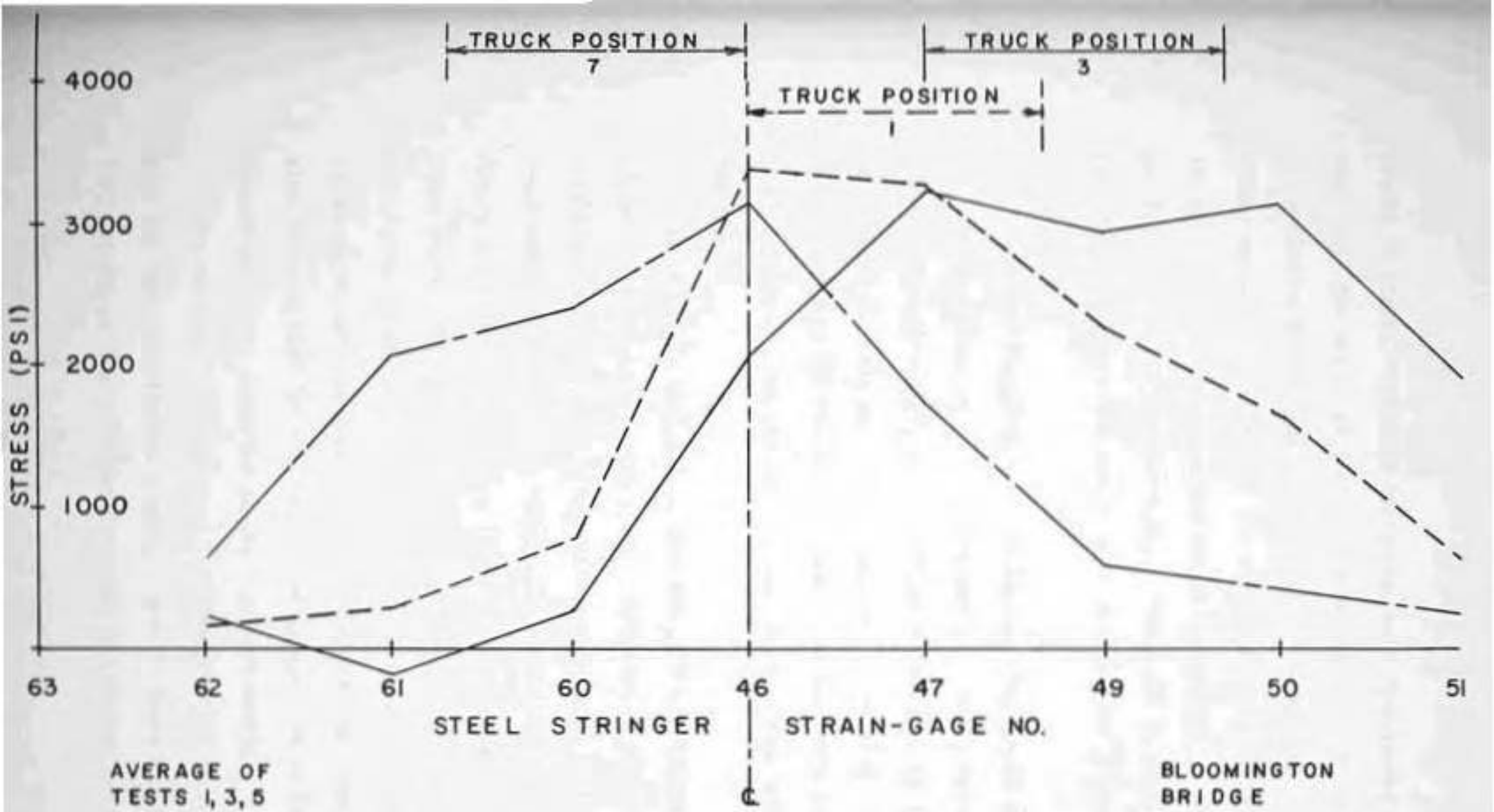


FIGURE NO. 19

DISTRIBUTION OF STRESSES AT SECTION W
FOR TRUCK POSITIONS 1, 3, AND 7.

brated in stress, while the horizontal axis is a scaled transverse cross section of the bridge at Section W with the stringers being located by their corresponding strain gage number. The stress plotted above each gage number was obtained by taking the average of the appropriate stresses from Tests 1, 3, 5, which were conducted on November 12, 1970, November 17, 1970, and February 18, 1971 respectively. Figure #20 is identical to Figure #19 except three other truck positions were used, namely truck positions #4, #6, and #8.

Figure #19 indicates that the maximum stresses or moments occur in the stringers directly under the tandem axle, which was to be expected. (For truck positions #1, #3, and #7 the tandem axle of the test vehicle is centered directly over Section W. (See Figure #16). Figure #20 shows that as the test vehicle is moved away from the centerline of the stringers (i.e. Section W), the stresses at Section W decrease which also was to be expected.

Figure #21 is included to show the magnitude of the load transmitted across the floor beam by the slabs. Referring to Figure #16 it can be seen that truck position #3 is midway between the first two floor beams while truck position #5 is midway between the second and third floor beams. Figure #21 indicates that very little moment is transferred across the floor beams. This would justify disregarding the front wheel loads in the calculation of the moment under the tandem axle, provided the front and tandem axles were not located between the same two floor beams. This would also indicate that for the purposes of design, the stringers could be assumed as simply supported at their support points.

The calculated longitudinal load distribution factor will be compared with the load distribution equation given in Table 1.3.1 (B) of AASHO'S 1973 edition of Standard Specifications for Highway Bridges. A distribution

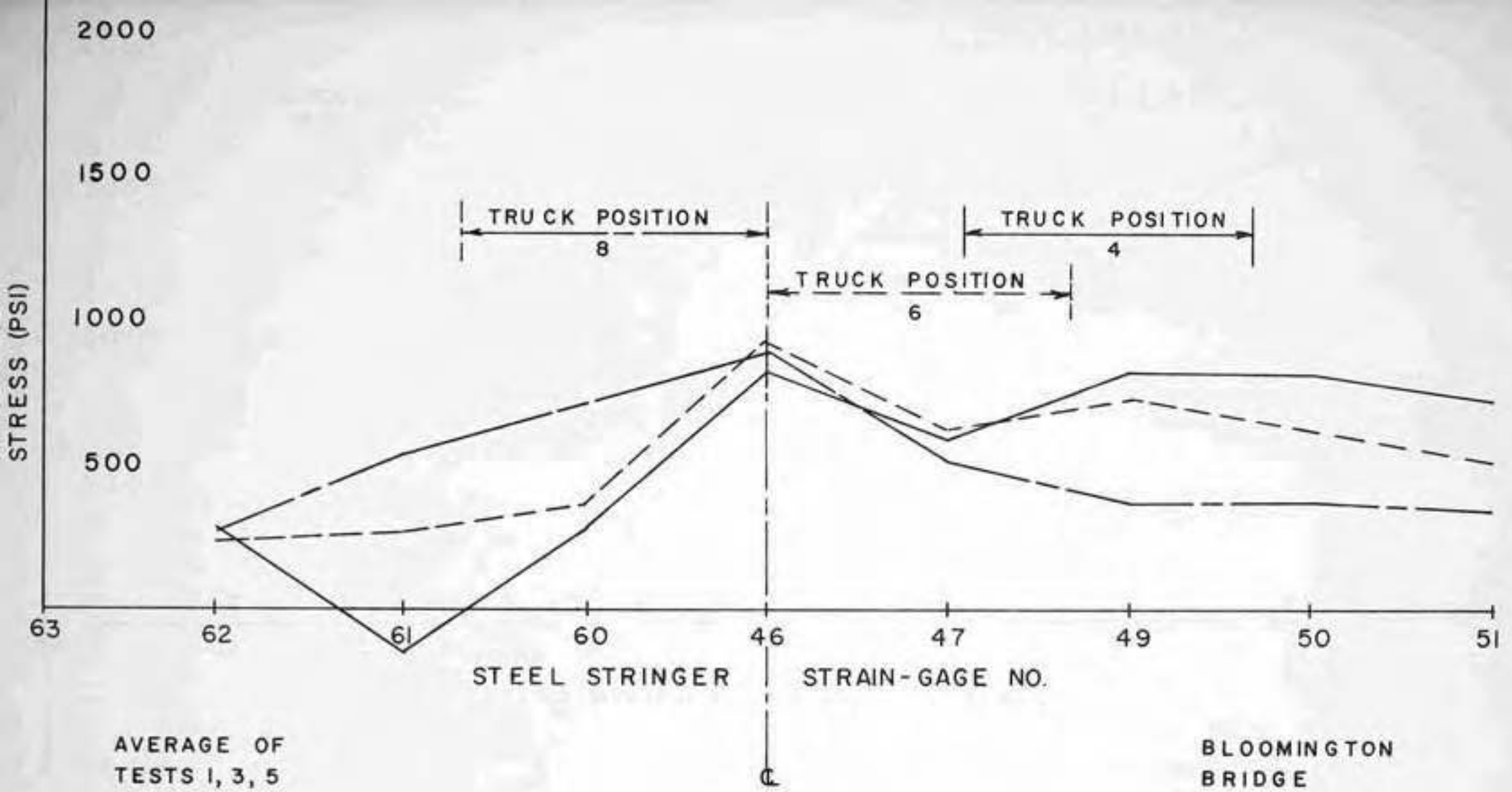


FIGURE NO. 20

DISTRIBUTION OF STRESSES AT SECTION W FOR TRUCK POSITIONS 4, 6, AND 8.

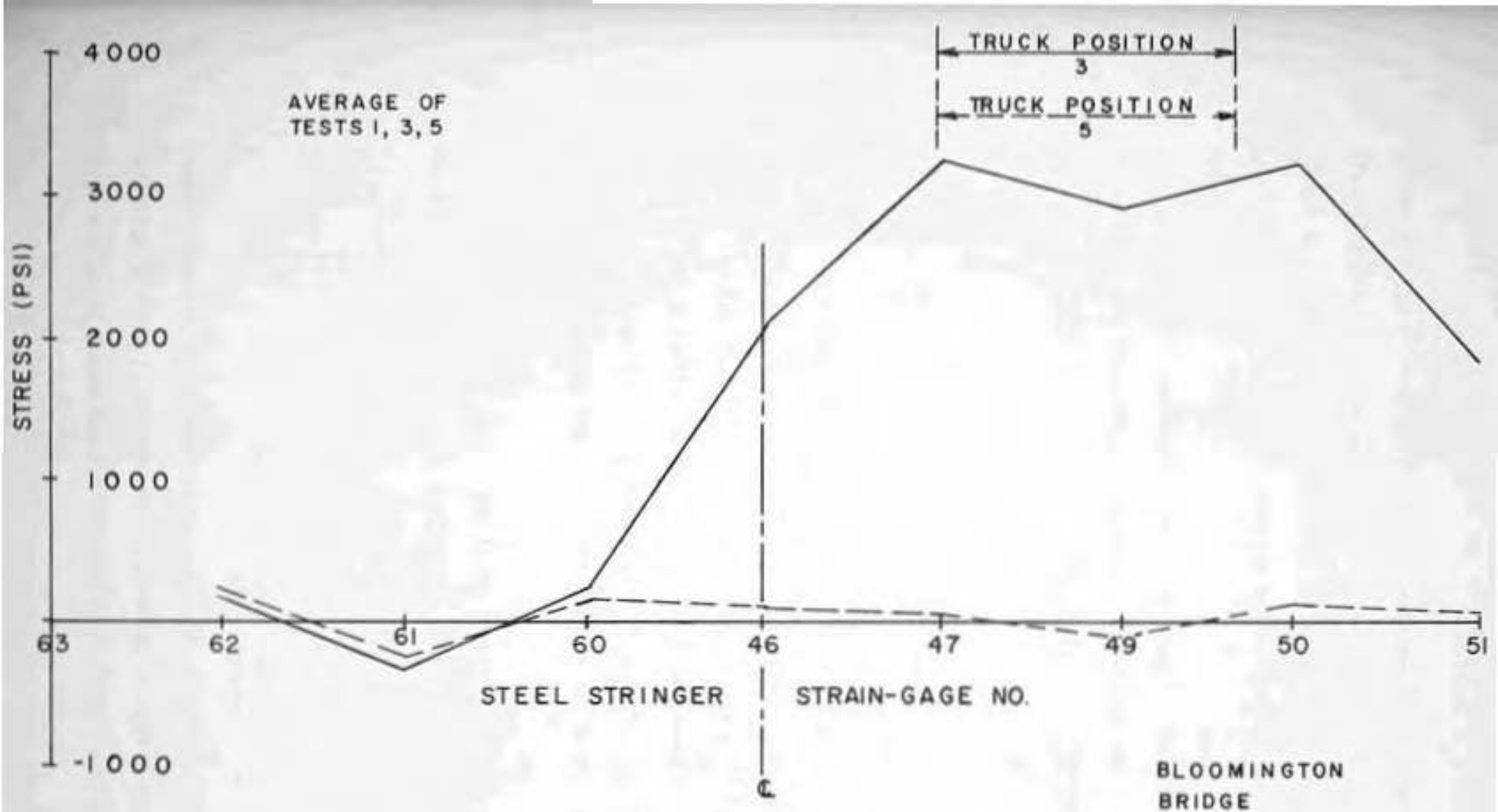


FIGURE NO. 21

DISTRIBUTION OF STRESSES AT SECTION W
FOR TRUCK POSITIONS 3 AND 5.

factor will be calculated for Truck Position #3 using strain gage #47, Truck Position #1 using gage #46 and one for Truck Position #7 also using strain gage #46. See Figures #16 and #17 for the truck location with respect to the stringer. The detailed calculations may be found in Appendix B.

Assuming full composite action the experimental load distribution factors for the three truck positions are as follows:

Truck Position	Strain Gage	Experimental Dist. Factor
#3	#47	0.62
#7	#46	0.57
#1	#46	0.66
	Average experimental load distribution factor =	0.62
	AASHO distribution factor =	0.68

The AASHO distribution factor agrees quite well with the average experimental value and none of the experimental factors exceed the AASHO factor. In assuming full composite action between slab and stringer, which is not actually the case, a maximum experimental distribution factor was obtained. Since the maximum factor was less than the AASHO design specification, the interior stringers are sufficient to carry the design loads.

If it is assumed that there is no composite action between the deck and the stringers the experimental load distribution factors are

Truck Position	Strain Gage	Computed Dist. Factor
#3	#47	.38
#7	#46	.34
#1	#46	.40
	Average Computed load distribution factor =	.37

Comparing the results considering complete composite action and no composite action, it is evident that through friction there is a large degree of composite action between the deck and the stringers.

In a previous section the variation of stress over a period of time for a particular gage was examined. Figures #22 and #23 indicate the change in the stress (load) distribution at Section W over a period of approximately two years. The stresses were averaged from Tests #1, 3, 5, and Tests #21, 22, 24 (See Tables B1, B2 and B3). The average of Tests #21, 22, 24 is plotted as a solid line. As was concluded for the stress variations over time for the individual strain gages, the load distribution has remained virtually unchanged over the two year time period.

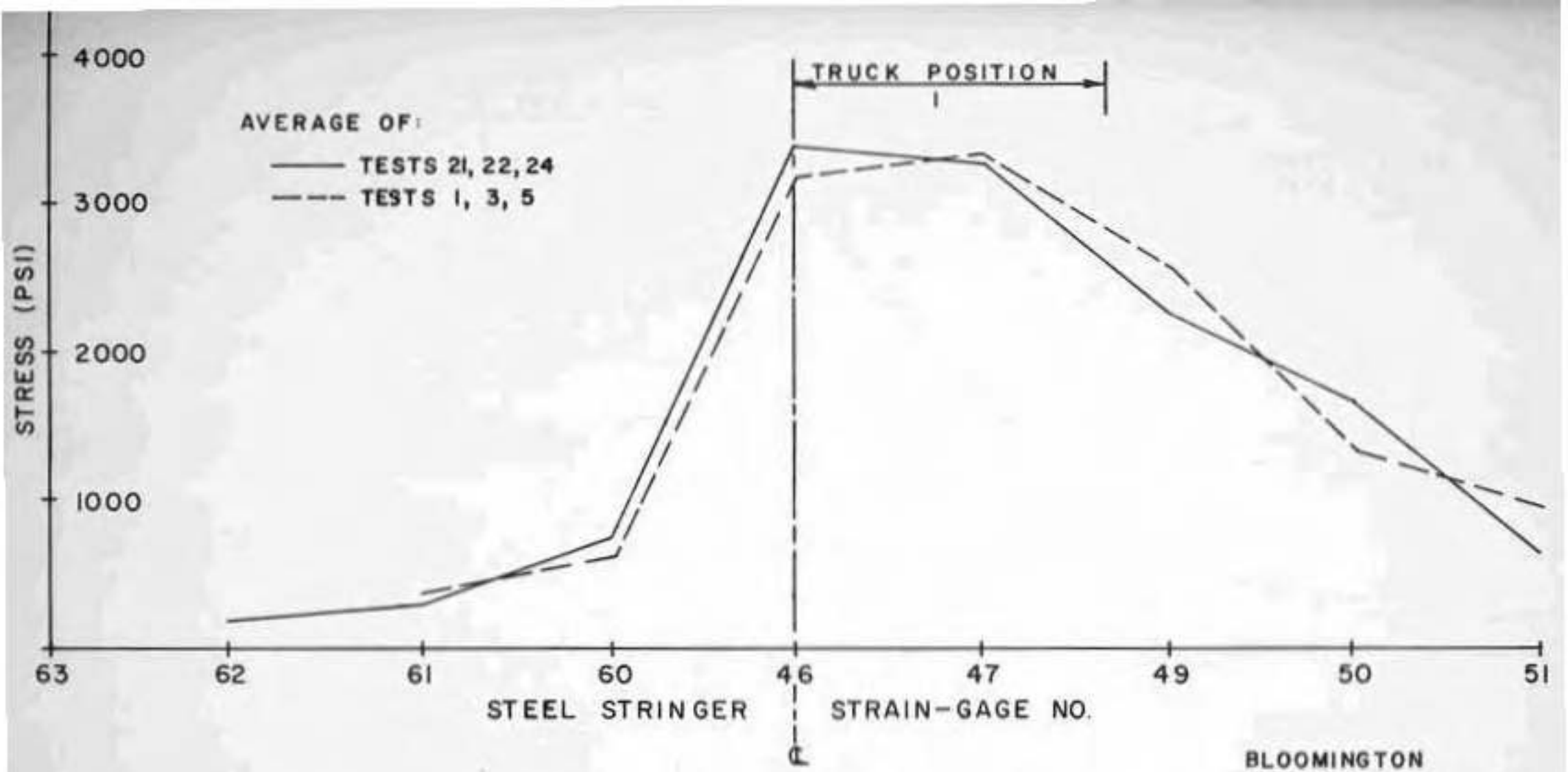


FIGURE NO. 22

TIME EFFECT ON THE DISTRIBUTION OF
STRESS FOR TRUCK POSITION I AT SECTION W

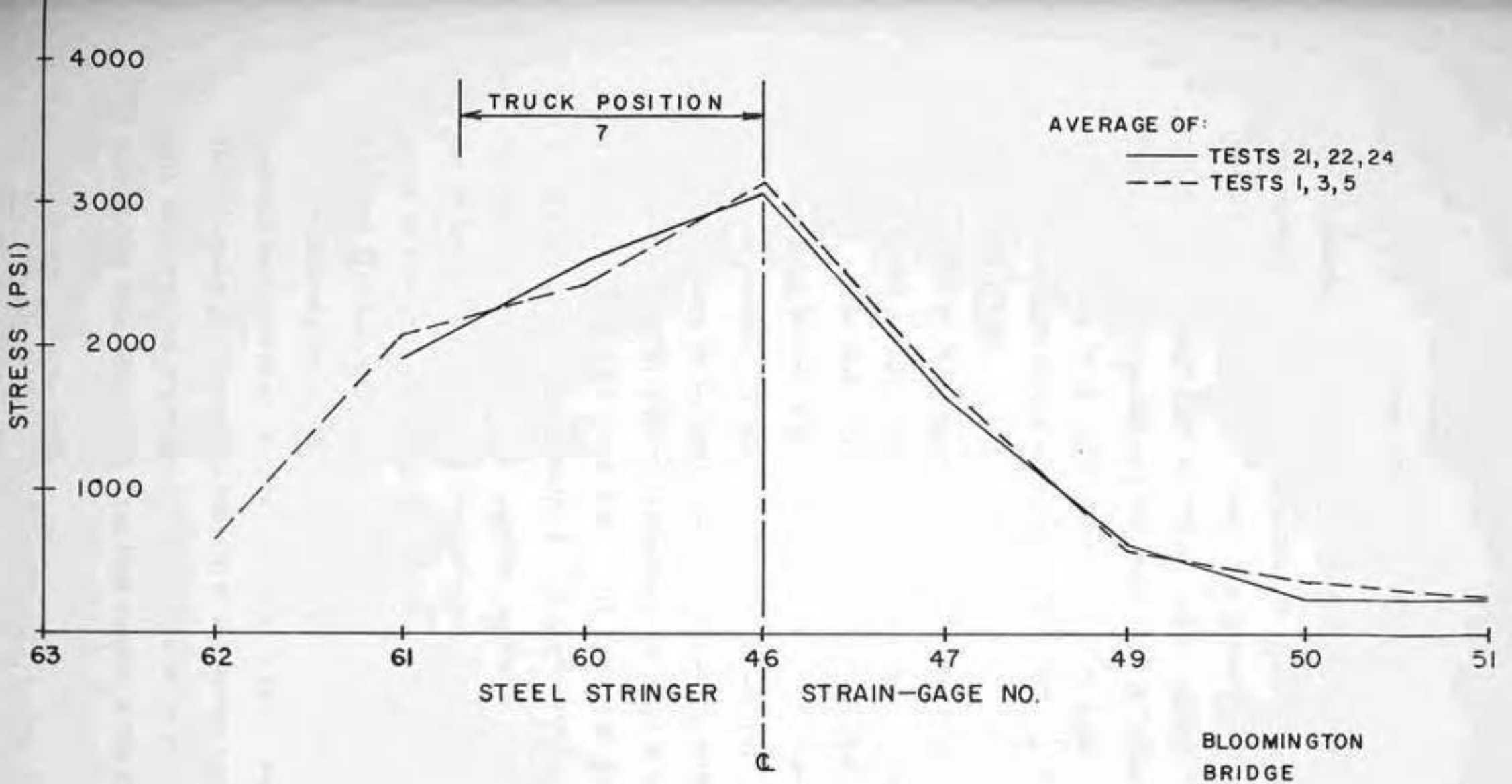


FIGURE NO. 23

TIME EFFECTS ON THE DISTRIBUTION OF STRESSES FOR TRUCK POSITION 7 AT SECTION W

BRIDGE PHYSICAL CONDITIONS

A thorough inspection of the Bloomington and Knightstown bridges was conducted in June, 1974 to ascertain the physical condition of the precast, prestressed decks. This examination included the mapping of all distressed areas such as spalled joints, cracks in the slabs, and areas where the neoprene seal had worked out of the joint. This report will concentrate on the condition of the Bloomington deck since this deck has seen the larger volume of traffic (ADT = 9000 VPD, 17% trucks).

Bloomington Bridge

The deck at Bloomington has been in service for five years and for the most part has performed very well. Figure #24 shows the present condition of the Bloomington deck and Figure #25 shows a typical wearing surface joint in the south-bound lane. Most of the problems that have arisen can be traced to poor construction procedures and are not due to the concept used in the design of the precast prestressed deck.

In the order of their presentation, the problem areas which will be discussed are: (1) cracked or spalled joints on the wearing surface, (2) wearing surface joints which had to have the joint sealant replaced, (3) areas on the underside of the deck where the neoprene seal has worked out of the joint and (4) distressed areas on the underside of the deck which include hairline cracks in the slabs, cracked joints, and one broken tie-down clip and bolt.

Immediately after the bridge was open to traffic in 1970, the north approach was resurfaced in order to remove a dip in the pavement. During the resurfacing, a steel wheeled roller was driven a number of times onto both the north and south bound lanes at the north end of the bridge. The high stress from the applied line load cracked a few of the top portions

Figure #24. Bloomington Bridge Deck, September, 1974

Figure #25. Bloomington Bridge - Typical Wearing Surface Joint in the Southbound Lane. See Figure #27 for Location.

Figure #26. Bloomington Bridge - Repaired Joint in the Southbound Lane. See Figure #27 for Location.

of the female joints and after a couple of weeks, traffic pressure worked out the cracked portions of the joint. These joints were repaired by cleaning the joint, forming the joint with Masonite and then filling the broken joint with quick-setting epoxy mortar. The epoxy mortar was allowed to cure and DuPont "Imron" was then used to reseal the joint. One-way traffic was maintained on the bridge during the entire repair operation. One of these repaired joints in the southbound lane can be seen in Figure #26 and its location is shown on Figure #27. All the joints which were repaired at this time are performing satisfactorily.

Figure #27 is a map of all the joints which have cracked or spalled since the bridge was opened to traffic. All of the distressed areas lay in the traffic lanes which tends to indicate that traffic wear is the major cause of the joint cracking or spalling. The two main reasons why the traffic is causing the cracked joints are variations in slab elevations and the use of snow removal equipment. The elevation between slabs vary as much as 1/4" and under repeated wheel load applications, the raised lip of the high slab will either ravel or crack. Usually if the raised edge is the female side of the joint, the female edge will crack and if the raised edge is the male portion it will ravel. The only solution to this problem would be making sure that the slabs were leveled during erection. The other major cause of the cracked joints is the impact load imposed on the slab joints by the blades from the snow removal trucks. Two solutions to this problem might be: (1) using more de-icing chemicals and not using the snow plow blade on the bridge or (2) in the future use a different joint configuration which would eliminate the weak edge inherent in the female portion of the joint.

Closely related to the problem of the cracked joints is failure of the joint sealant. If a joint spalls, ravels or cracks the joint sealant

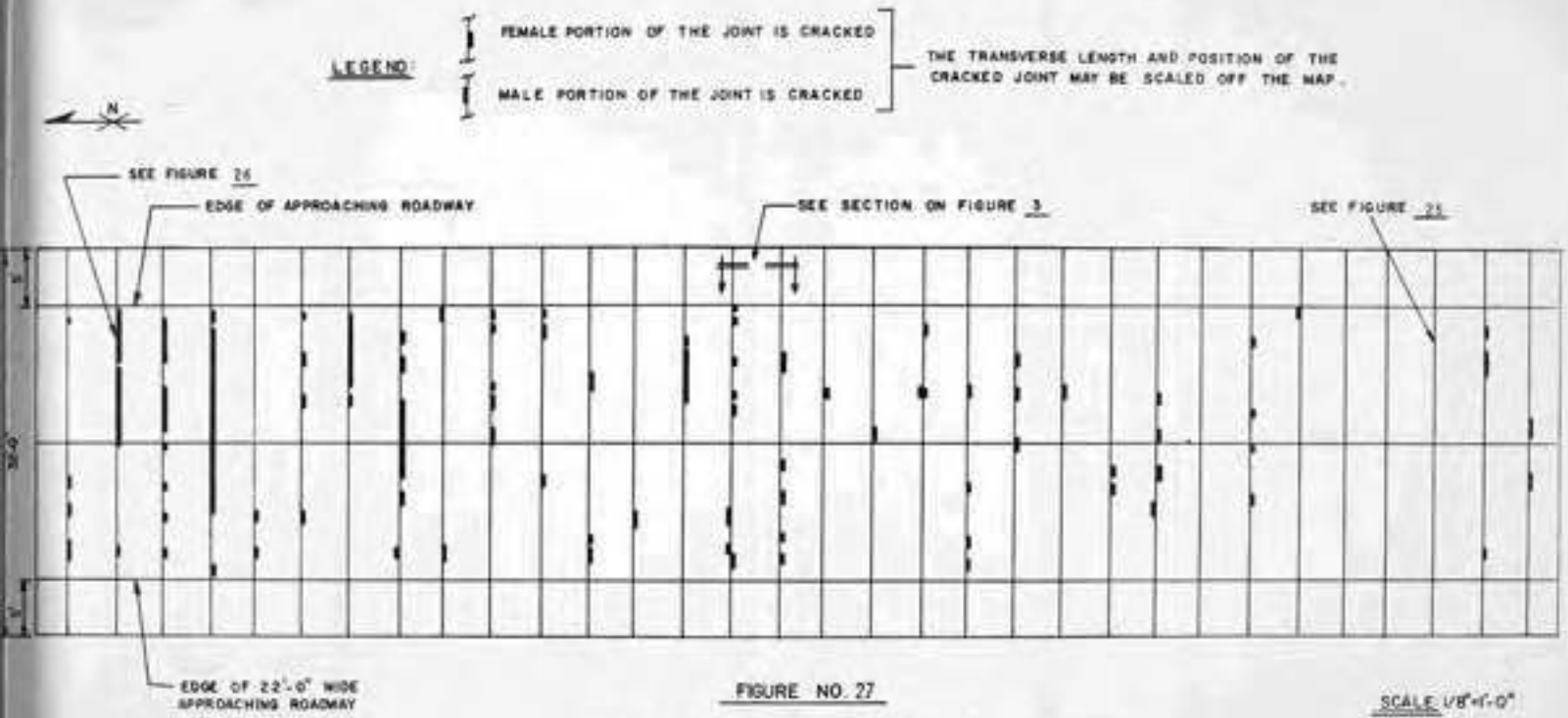


FIGURE NO. 27

SCALE 1/8"=1'-0"

PLAN VIEW OF BLOOMINGTON BRIDGE DECK

MAP OF SPALLED OR CRACKED JOINTS

will usually work out of the joint under traffic loads. Figure #28 is a map of the areas where the joint sealant has been replaced and shows which areas originally were sealed with the liquid polyurethane sealant and which joints were sealed with DuPont "Imron". In addition to the reasons stated above, the joint sealant also had to be replaced for the following reasons: (1) joint material itself, (2) application techniques which were mentioned earlier and (3) gravel punching the sealant into the joint.

During the inspection of the precast deck in June, 1974 it was noted that nearly all of the joints leaked during a rain. The exact position of the leaks could not be located since the water entering the joint can travel along the inside of the joint before appearing at the bottom side. These leaks were mainlycaused by the liquid polyurethane sealant which was initially poured into the majority of the joints. As was described earlier, air became entrapped in the liquid sealant as it was being poured into the joint. The entrapped air formed bubbles which expanded when heated and pushed the sealant above the surface of the deck. These bubbles were subsequently worn away by the traffic, leaving honeycombed voids which would readily pass water. Since this condition occurred in the majority of the joints, it was decided at the time that it would not be feasible to replace all of this sealant.

One positive aspect of the liquid sealant is that it resists the punching action of gravel better than the DuPont "Imron". The DuPont "Imron" was installed in the first four joints at the southern end of the bridge using a caulking gun and then tooling the joint with a soap finger. DuPont's requirement is that the depth of the sealant be equal to the width of the joint. This requirement was not strictly adhered to and the bond subsequently failed between the "Imron" and the sides of the

LEGEND: AREAS WHERE THE ORIGINAL JOINT MATERIAL WAS REMOVED. THE TRANSVERSE LENGTH AND POSITION OF THE REPLACED SEALANT MAY BE SCALED FROM THE MAP.

NOTE: ALL JOINT SEALANT REPAIRED WITH DUPONT "IMRON".

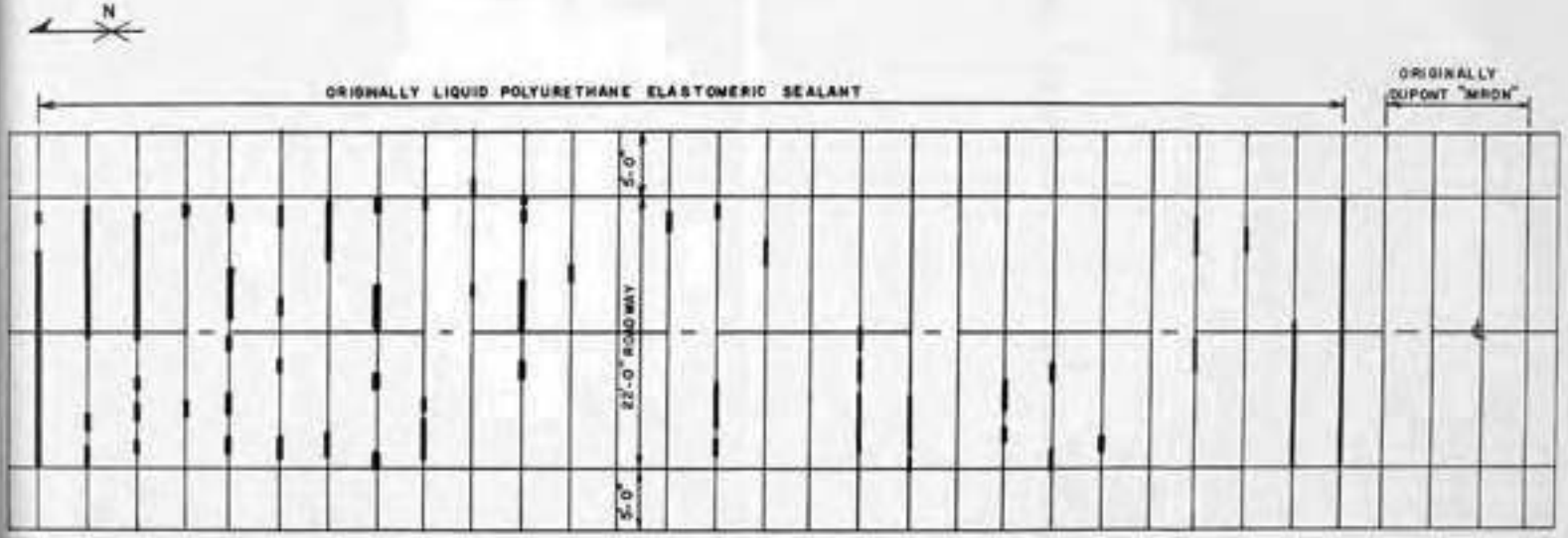


FIGURE NO. 28

PLAN VIEW OF THE BLOOMINGTON BRIDGE DECK MAP OF THE AREAS WHERE THE JOINT SEALANT HAS BEEN REPLACED

SCALE: 1/8"=1'-0"

joint. The sealant was then punched down into the joint which allowed gravel to collect and allowed the passage of water through the joint. In areas where DuPont's shape requirement was met and the bond between the sealant and the joint sides appeared to be satisfactory, the punching action of the gravel in the traffic lanes broke the bond and pushed the sealant and the beaded backup strip down into the joint.

Since all of the sealant repaired and shown on Figure #28 was replaced in late 1970, it is approximately four and one-half years old. DuPont "Imron" was used in all of the repair work and to date has performed satisfactorily, with the exception of minor gravel penetration. A repaired joint can be seen in Figure #26.

Figure #29 shows the areas under the deck where the neoprene sheet has worked down out of the joints. The neoprene sheet was used to minimize stress concentrations between the joints due to minor surface irregularities. The compressive stress in the neoprene was relatively high in the areas where it has worked out of the joints. This high stress was due to a major irregularity in the surface of the joint which was in turn caused by warping of the forms where the slabs were poured. In order to relieve this high stress, the neoprene works out the joint under the repeated traffic load.

Figure #30 is a map of the other distressed areas on the underside of the slabs and Table 2 describes the condition of each of the numbered areas on the map. The probable cause for the broken clips and bolt in area ①was the initial lack of bearing between the slab and the stringer. This lack of bearing was noticed after the bridge was opened to traffic and shims were immediately driven between the slab stringerand precast slab, but the traffic load kept working the shims back out of the void (See Figure#31).

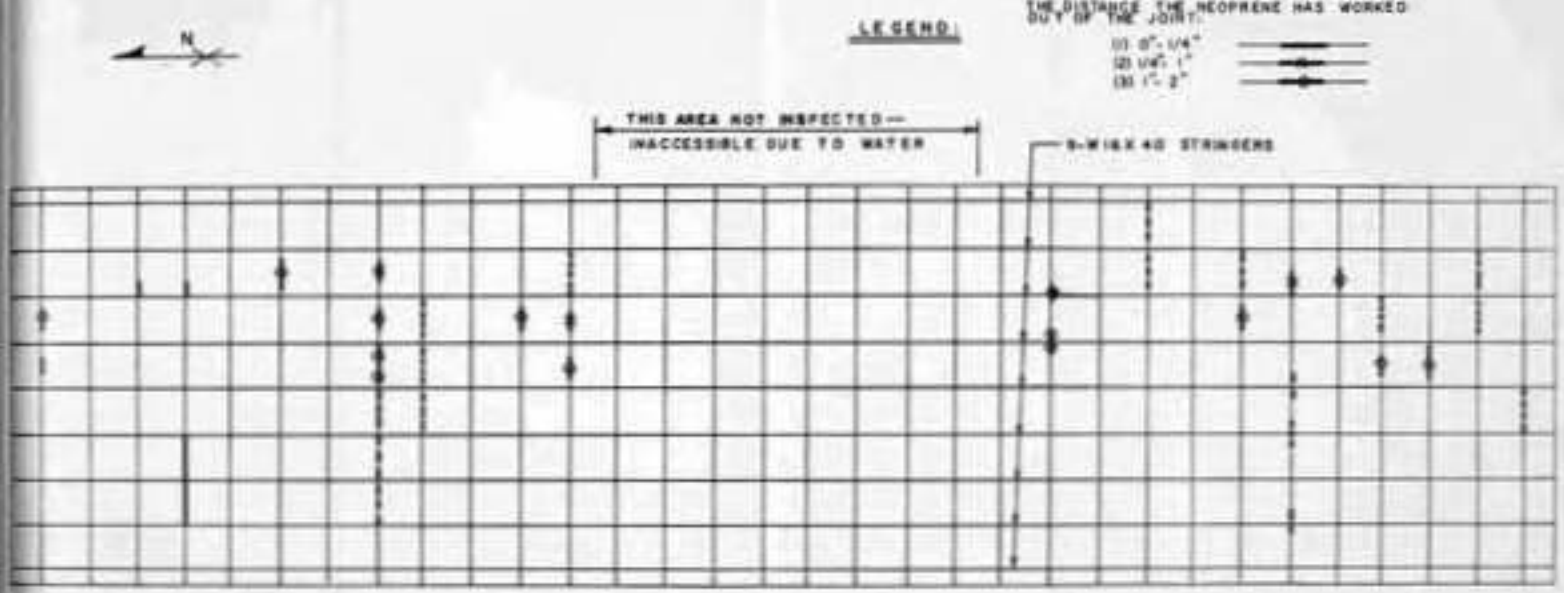


FIGURE NO. 20

SCALE 1/8"-1'-0"

PLAN VIEW OF THE BLOOMINGTON BRIDGE DECK
MAP OF THE BOTTOM SIDE OF THE JOINTS WHERE
THE NEOPRENE SHEET HAS WORKED OUT OF THE JOINT

LEGEND:
CRACKED FEMALE PORTION OF JOINT
CRACKED MALE PORTION OF JOINT
— HAIRLINE SURFACE CRACK

NOTE: SEE TABLE 7 FOR DESCRIPTION OF (1) THRU (15)

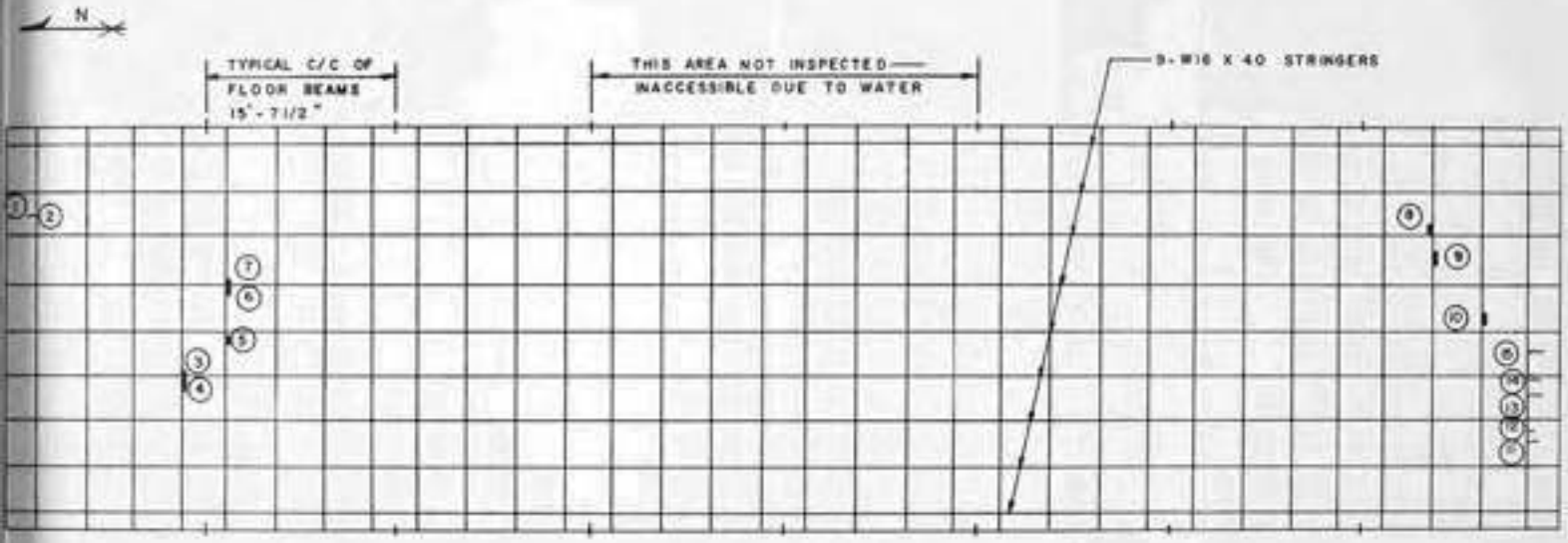


FIGURE NO. 30

SCALE: 1/8"=1'-0"

PLAN VIEW OF THE BLOOMINGTON BRIDGE DECK MAP OF THE DISTRESSED AREAS ON THE UNDERSIDE OF THE BRIDGE DECK

Table 2
Description of the Distressed Areas Shown on Figure #30

Area		
①	One 3/4" diameter bolt broken, one tie-down clip missing, and one tie-down clip broken	See Figure #31
②	8" longitudinal surface crack	See Figure #32
⑤	Typical of Areas ③ thru ⑦ - Cracked female joints at stringer bearing point	See Figure #33
⑧	Spalled male portion of the joint 2"x2"x1/4" deep	See Figure #34
⑨	Cracked female joint, 8" long	
⑩	1"x1" hole - entire bottom portion of the female joint has fallen out	
⑪ thru ⑮	6", 1", 12", 7" and 12" longitudinal surface cracks respectively	

The resulting excessive flexing of the slab under repeated traffic loads probably caused the bolt and clips to fail in fatigue and caused the 8" longitudinal crack (Area ②) shown in Figure #32.

The cracked female joints (Areas ③ through ⑦ and Area ⑨) were caused by uneven bearing between the slabs and the steel stringers. Figure #33 shows Area ⑤ as a typical example. Area ⑧, shown in Figure #34, is a minor spall and Area ⑩ is a patched female joint which failed. The longitudinal surface cracks (Areas ⑪ through ⑮) in the southern slab are due to improper storage conditions before the slab was installed. When the slab was stored in the precaster's yard, the blocking was improperly positioned, causing the slab to crack in several places.

Another problem which arose after the bridge was open to traffic was that some of the bolts clamping the slabs to the steel beams worked loose and had to be re-tightened. During construction a torque of 50 ft. lbs. was applied to each bolt, which was the torque applied to the bolts in the laboratory experiments. Since 50 ft. lbs. of torque was not sufficient, three quarters of all the bolts were re-tightened using a torque of 125 ft. lbs. after making sure that each bolt was greased. None of the bolts have since worked loose.

Figure #31. Area ① - One 3/4" diameter bolt broken, one tie-down clip missing; and one tie-down clip broken (See Figure #30 and Table 2).

Figure #32. Area ② - 8" longitudinal surface crack (See Figure #30 and Table 2)

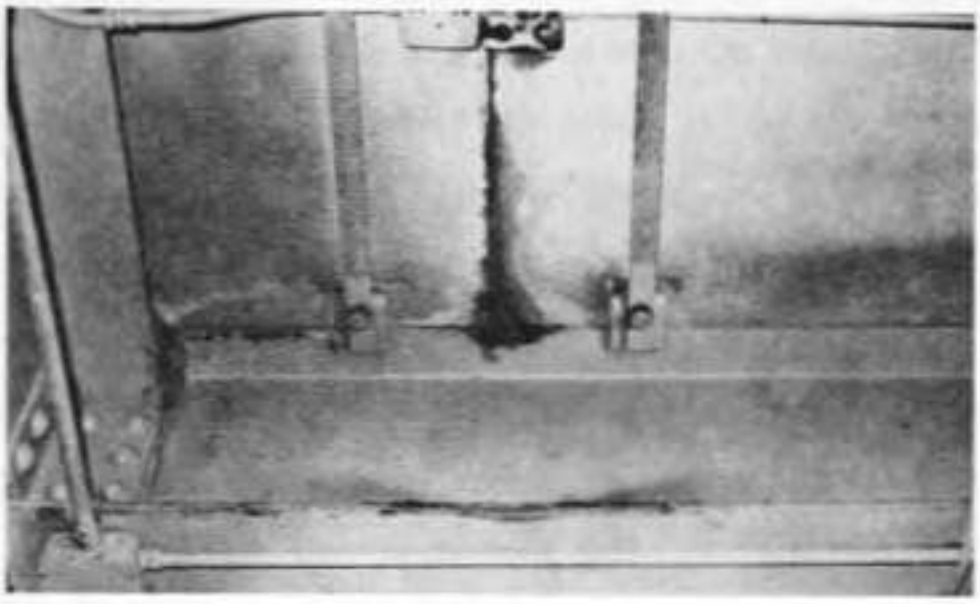


Figure #33. Area⑤- (Typical of Areas 3 thru 7) Cracked female joint at the stringer bearing point. (See Figure #30 and Table 2).

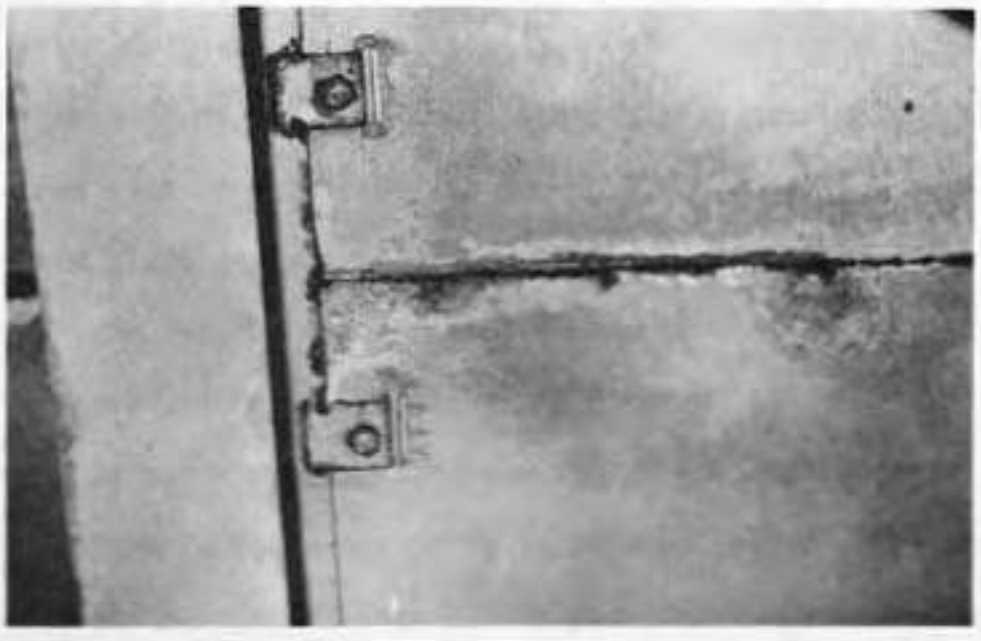


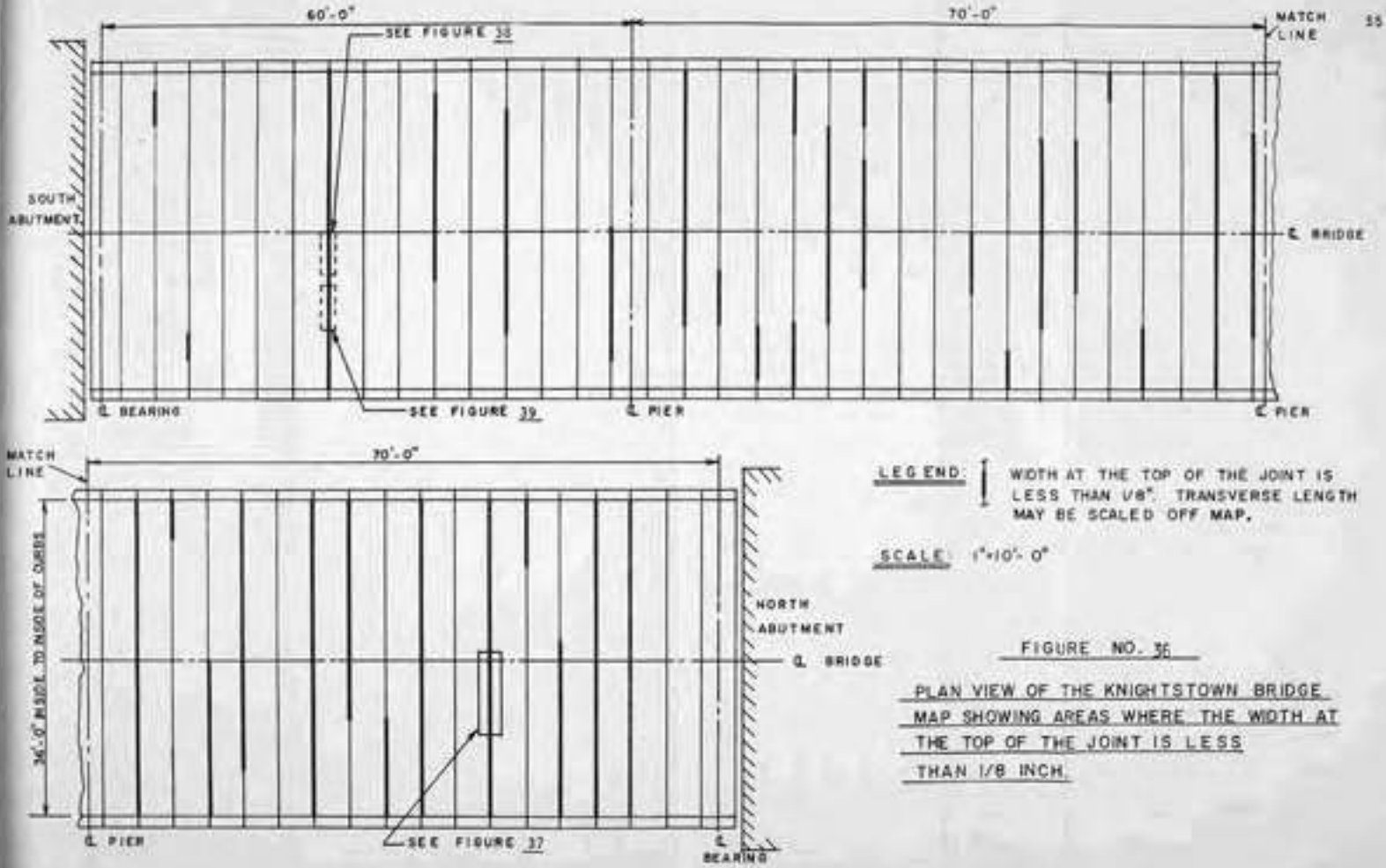
Figure #34. Area⑧- Spalled male portion of the joint--2"x2"x1/4" deep (See Figure #30 and Table 2).

Knightstown Bridge

Figure #35 shows the present (June, 1974) condition of the Knightstown deck. The main problem encountered with the Knightstown Bridge was due to irregularities in the width of the joints at the top of the slab. After the slabs were post-tensioned together, there were numerous locations where the indicated 1/4" clearance at the top of the joints did not exist. Figure #36 is a map of the areas where the joint widths are less than 1/8", and approximately half of the defective joints shown are completely closed. There were no immediate effects except the joints leaked water when it rained due to the fact that no sealant could be installed in the joint. However, a few months after the bridge was opened, the concrete in the vicinity of the closed joints began to spall and this spalling has continued to the present time. In some areas the spall has been large enough to require an asphalt patch. Figure #37 shows a closed joint where a portion of the joint has been progressively failing on the surface of the slab. The underside of this joint appears normal. The cause of the joint irregularities was irregularity in the forms used to cast these slabs. These forms had additional steel strips tack-welded to the tops of the main sideforms in order to form the crown in the section. These tack-welded steel strips warped and caused a bad fit of the slabs in the field. In the future, the sides of the forms used to cast these slabs should be parallel and as straight and stiff as possible.

The only other problem observed at Knightstown was that concrete patches applied in the precaster's yard deteriorated after about one year. The photographs in Figures #38 and 39 show patched female joints on the underside of the slab where the patch has deteriorated or has fallen out of the joint. Also there are only a few places where the neoprene has started to work out of the joint.

Figure #35. Knightstown Bridge Deck, September, 1974



LEGEND: WIDTH AT THE TOP OF THE JOINT IS LESS THAN 1/8". TRANSVERSE LENGTH MAY BE SCALED OFF MAP.

SCALE: 1"=10'-0"

FIGURE NO. 36

PLAN VIEW OF THE KNIGHTSTOWN BRIDGE MAP SHOWING AREAS WHERE THE WIDTH AT THE TOP OF THE JOINT IS LESS THAN 1/8 INCH.

Figure 37. Knightstown Bridge - Progressive Failure of the Male Portion of the Joint. See Figure #36 for Location.

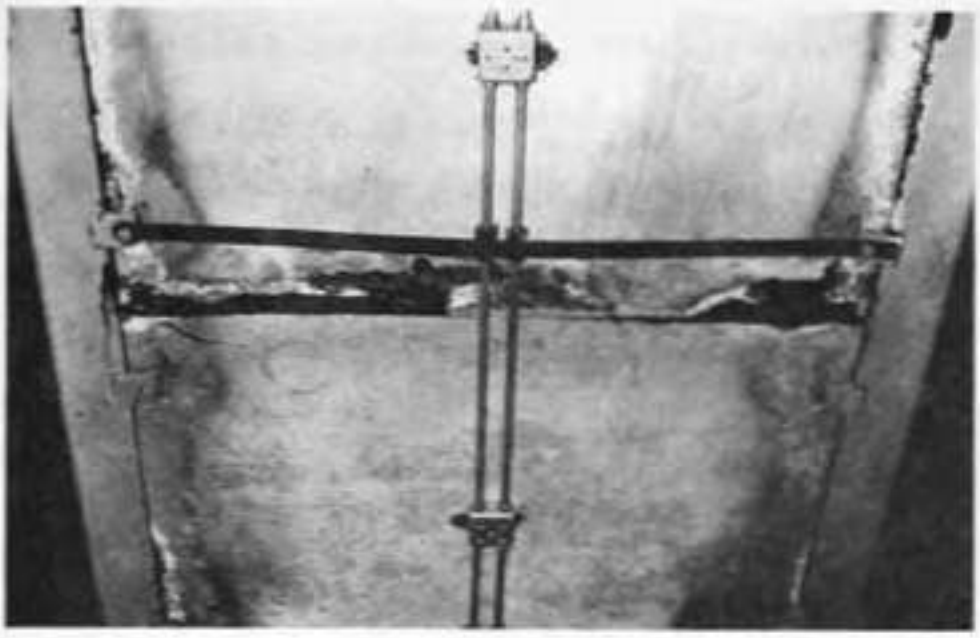


Figure #38. Knightstown Bridge - A Patch Applied in the Precaster's Yard Which Has Failed. See Figure #36 for Location.

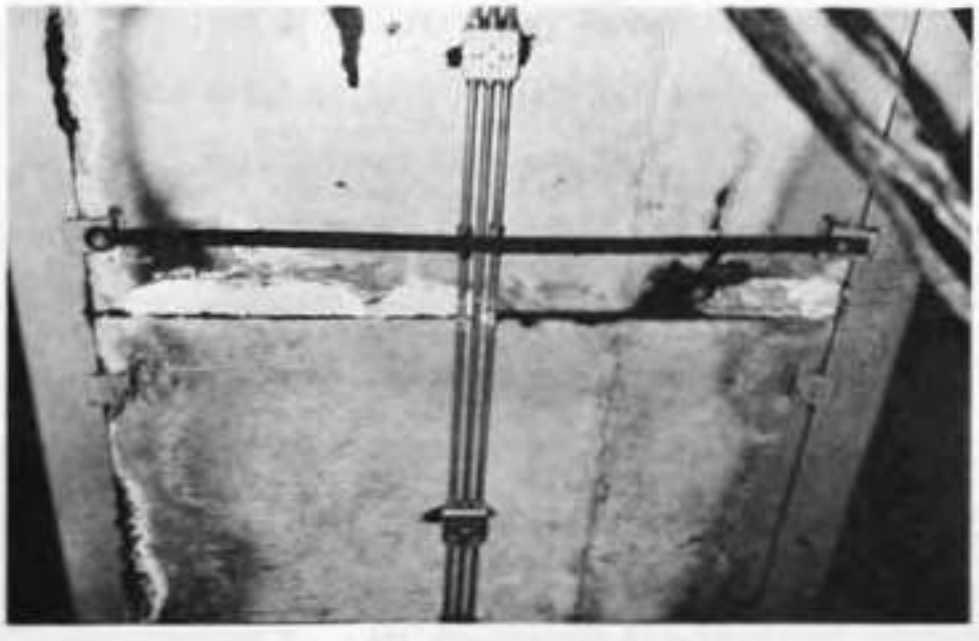


Figure #39. Knightstown Bridge - A Patch Applied in the Precaster's Yard Which Has Failed. See Figure #36 for Location.

CONCLUSIONS AND RECOMMENDATIONS

This investigation has shown that precast prestressed concrete bridge decks produce a durable deck which can be constructed in a minimum of time. The joints that were properly formed and sealed are functioning well. The surfaces of both bridges show no signs of distress even though they both have been through five winters. The neoprene sheet has worked out of the several portions of the joints due to faulty installation and the tie-down system has performed well after the initial bolt installations were corrected. Neither of the bridges have an overlay so the slabs and joints take the direct load of all the traffic. Overlays were not installed in order that the direct effects of the traffic on the bridges could be observed and monitored. In a metropolitan area where pedestrian and/or vehicular traffic would be present under the bridge, an asbestos-asphalt overlay might be required in order to insure against water leakage through the joints. It should also be noted that water leakage through the joints can eventually cause a problem with corrosion of the stringers supporting the deck. Some corrosion has been noted on both bridges due to leakage. An asbestos-asphalt membrane would prevent this problem.

There are two alternatives to enhance the serviceability of the Knightstown and Bloomington bridges. The first and the most drastic, is to close the bridge, detension the post-tensioning cables, then repair the damaged joints and replace the joint sealants. This would be an expensive and time consuming task. The second alternative is to apply an asbestos-asphalt membrane over the deck and then apply an asphalt overlay. The second alternative is the more reliable and would require less time to perform.

In the future it is recommended that the bridge deck be constructed with flat joints rather than the key joints used in this investigation and that an asbestos-asphalt membrane be placed over the deck followed by an overlay. This combination would alleviate the problems of joint distress and water leakage. In general it would enhance the overall durability and serviceability of the bridge. Several county bridges in Allen County in northern Indiana have been constructed using the concept of flat joints. The slabs were 8 ft. wide with flat joints and had weld plates at the joint for vertical alignment. An asbestos-asphalt membrane and asphalt overlay were placed on the bridge after the slabs had been secured to the bridge girders. To date these bridges have been performing satisfactorily.

APPENDIX A

STATIC LOAD TESTS

Table A1
Static Load Test Stresses (psi) With Original Deck in Place
Bloomington

Gage No.*	Test Truck Position**											
	1	2	3	4	5	6	7	8	9	10	11	12
45	1540	29	1189	290	87	348	1566	377	58	-145	0	0
46	3306	-406	1885	377	-232	609	3364	986	174			
47	3219	-145	2871	725	-145	841	1334	464	-87	-58	-58	-29
48	1247	-29	1363	261	58	290	783	232	0	58	29	0
49	2262	-58	2987	551	-58	464	348	217	0	-116	0	0
50	928	-174	2088	493	-145	290	261	261	116	58	203	-174
51	348	-29	1247	406	0	290	348	406	319			
52	1392	3654	754	580	2146	696	1276	609	3422	493	406	290
53	1972	1392	1363	1653	1015	2436	2146	2523	1566	377	290	174
54	551	2610	493	638	2436	696	145	145	1029	290	58	290
55	58	609	58	174	1044	174	116	145	203	0	58	0
56	1566	1160	1537	1885	1189	1885	1102	1276	870	319	290	290
57	667	1624	1305	609	3074	406	290	290	522	203	522	725
58	319	667	348	522	1044	522	145	319	348	406	203	493

*See Figure 8 for gage location.

**See Figure 14 for Test Truck Positions.

Stress units are psi, tension is positive.

Also see Test 1A on Table A2.

Table A2
Stresses (psi) from Selected Gages for the Static Load Tests
Bloomington Bridge

Test	Date	Weather	Temp. (°F)	Truck Wt. (lbs.)	Steel Gages				Concrete Gages			
					Gage 50	Gage 52	Gage 55	Gage 56	Gage 25	Gage 36	Gage 39	Gage 40
1A*	6-1-70	Cloudy	70	43,690	2088	754	58	1537				
1	11-12-70	Cloudy	40	44,900	3208	582	1107	2131	168	256	137	186
2	11-19-70	Cloudy	45	43,000	3222	554	905	2187	151	221	110	126
3	12-17-70	Cloudy	35	48,400	3317	536	928	2116	156	217	100	88
4	1-14-71	Cloudy	28	47,300	2936	800	1147	1808	139	188	111	129
4A	1-14-71	Cloudy	28	47,300	3010	704	1031	1971	157	193	123	120
5	2-18-71	Cloudy	40	48,500	3015	807	1058	2169	158	203	135	156
6	4-6-71	Clear	55	58,500	2974	827	1006	2154	257	439	143	
7	5-6-71	Rain	61	43,300	2921	669	1154	2034	171	297	139	121
8	6-3-71	Clear	84	46,500	3252	628	1108	2139	199	535	127	137
9	7-1-71	Clear	85	53,000	3206		331	1367	174	394	68	76
10	7-29-71	Clear	80	53,300	3126	271	781	2079	183	363	82	11
11	9-9-71	Clear	81	51,000	2659	165	436	1511	114	264	26	30
12	10-7-71	Clear	54	54,500	2949	448	809	2097	153	192	59	58
13	11-11-71	Clear	58	45,000	2813	322	1231	2429	160	212	70	72
14	12-9-71	Rain	39	55,200	2698	252	1294	1966	149	169	100	156
15	1-20-72	Rain	52	45,600		1420				339		
16	2-17-72	Cloudy	35	52,300		1610						

All stresses were corrected by proportioning the truck loadings using the truck loading for Test #1 as a base.
Stresses correspond to Truck Position #3.
*These stresses are from Table A1 which refers to the static load test with the original deck in place.

Table A3
Stresses (psi) from Selected Gages for the Static Load Tests (continued)
Bloomington Bridge

Test	Date	Weather	Temp. (°F)	Truck Wt. (lbs.)	Steel Gages				Concrete Gages			
					Gage 50	Gage 52	Gage 55	Gage 56	Gage 25	Gage 36	Gage 39	Gage 40
17	3-16-72	Cloudy	45	51,900			302	2089	159	198	81	115
18	4-20-72	Cloudy	60	60,900								
19	5-11-72	Clear	70	55,500								
20	6-6-72	Cloudy	83	53,000								
21	8-31-72	Clear	75	50,300	2941	435	623	2045	169	276	70	77
22	11-30-72	Cloudy	30	50,900		456	539	2117	128	164	85	95
23	1-15-73	Clear	34	46,200								
24	2-22-73	Cloudy	32	47,700		1449	1509	1988	138	148		63
25	4-19-73	Cloudy	63	51,400	3143	842			211	241	116	226
26	5-17-73	Clear	64	51,000	2913			2008	224		77	99
27	8-3-73	Clear	75	50,000		351			152	240	96	111
28	9-11-73	Clear	78	53,000		399		2214	131	238	48	48
29	10-18-73	Clear	56	46,600		541	331	2120	123	143	77	102
30	5-2-74	Rain	57	50,100		129	877		161	218	61	
31	6-5-74	Cloudy	65	49,200			1165	1880	127	281	69	35

All Stresses were corrected by proportioning the truck loadings using the truck loading for Test #1 as a base.
Stresses correspond to Truck Position #3.

Table A4
Stresses (psi) from Selected Gages for the Static Load Tests
Knightstown Bridge

Test	Date	Weather	Temp. (°F)	Truck Wt. (lbs.)	Stress* (psi) Gage 26	Gage 28	Gage 29	Gage 30	Gage 35	Gage 36
1	3-4-71	Clear	26	53,800	128	242	201	248	232	195
2	4-6-71	Clear	50	48,100	122	231	154	208	183	185
3	5-6-71	Rain	61	52,600	86	195	132	202	162	143
4	6-3-71	Clear	78	49,600	124	237	187	206	212	170
5	7-28-71	Rain	70	54,300	126	234	171	210	191	181
6	10-6-71	Clear	65	55,000	120	246	186	219	216	179
7	12-9-71	Rain	39	53,000	181	294	266	252	275	152
8	1-20-72	Rain	52	54,800	82	160	134	170	197	117
9	3-16-72	Cloudy	50	53,500	124	250	202		219	179
10	5-11-72	Clear	78	54,100	155	268	188	221	206	193
11	7-13-72	Clear	88	48,000	150	268	204	238	222	180
12	8-31-72	Clear	95	47,000	138	208	164	204	197	176
13	1-15-73	Clear	45	49,300						
14	4-19-73	Cloudy	70	48,000	131		211	259	205	174
15	8-3-73	Clear	75	50,500	108	269	175			194
16	10-18-73	Clear	62	50,800						
17	5-2-74	Rain	56	48,900	103	187	200	226	216	99
18	6-5-74	Rain	65	58,000	120	185	222	246		

*All stresses were corrected by proportioning the truck loadings using the truck loading for Test #1 as as base. Stresses correspond to Truck position #14.

APPENDIX B

LOAD DISTRIBUTION FACTORS

APPENDIX B

Load Distribution Factors

The calculated longitudinal load distribution factor will be compared with the load distribution equation given in Table 1.3.1 (B) of AASHO'S 1973 edition of Standard Specifications for Highway Bridges. A distribution factor will be calculated for Truck Position #3 using strain gage #47, Truck Position #1 using gage #46 and one for Truck Position #7 also using strain gage #46. See Figures #16 and #17 for the truck location with respect to the stringer.

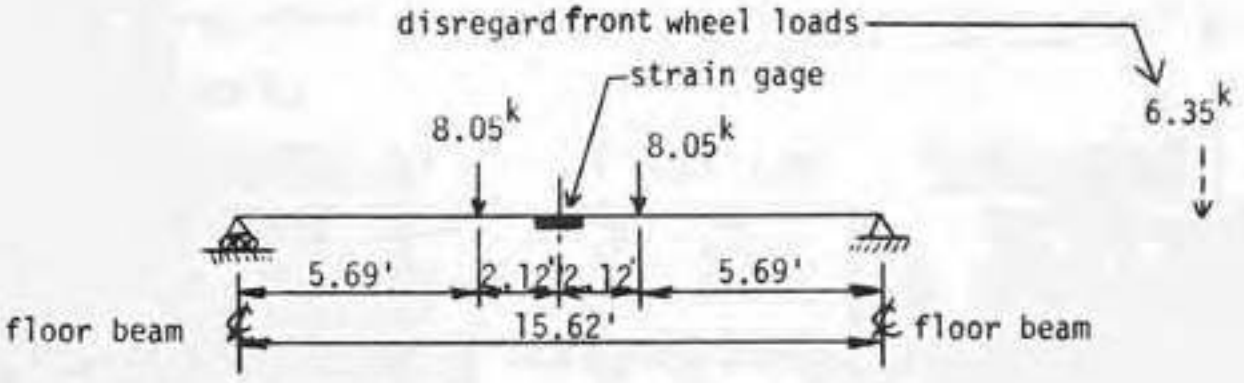
The moment resisted by the particular stringer in the three above mentioned cases is:

for Truck Positions #3, #7, #11,

truck weight from Test #1 = 44,900 lbs.

front axle = 12,700 lbs.

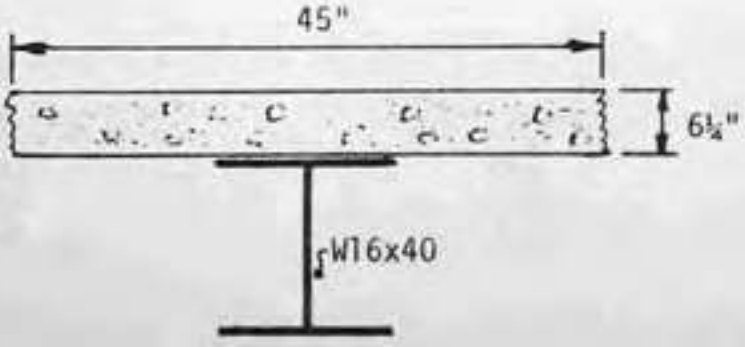
tandem axle = 32,200 lbs.



Moment @ strain gage = (8.05^k) (5.69')

$= 45.80^k$

The moment of inertia for an interior stringer assuming full composite action and using the AASHO specifications for the transformed effective slab width is:



Section properties of the W16 x 40

$I = 517\ in^4$
$A = 11.8\ in^2$
$S = 64.6\ in^3$
$d = 16"$

find the neutral axis of the transformed section:

$f'_c = 5000\ psi$ $\qquad n = 7$
$b = 45"$
$b/n = 6.43"$

$A_c = (6.43")\ (6.25") = 40.19\ in^2$
$Y_b = ((40.19\ in^2)\ (19.125") + (11.8\ in^2)\ (8.0"))/$
$51.99\ in^2 = 16.6\ in$

find transformed moment of inertia:

$I = 517\ in^4 + (11.8\ in^2)\ (16.6" - 8.0")^2$
$+ (6.43")\ (6.25")^3/12 + (40.19\ in^2)\ (19.125" - 16.6")^2$
$= 1776.7\ in^4$ (assuming full composite action)

The average stress for the particular strain gages can be found in Tables B1, B2, and B3.

Truck Position	Strain Gage	Average Stress
#3	#47	3194 psi
#7	#46	2935 psi
#1	#46	3402 psi

Calculate the moment resisted by the stringer using the average stress:

Moment = $\sigma I/c$

σ = average stress

$I/c = 1776.7\ in^4 / 16.6\ in = 107.0\ in^3$

Moment = $(107.0\ in^3)$ (average stress)

for Truck Position #3, Gage #47:

Actual moment carried by stringer = $(107.0\ in^3)\ (3194\ psi)$
$= 28.48'^k$

The actual moment resisted for all three cases is:

Truck Position	Strain Gage	Actual Moment Resisted
#3	#47	$28.48'^k$
#7	#46	$26.17'^k$
#1	#46	$30.33'^k$

The experimental longitudinal load distribution factor is the actual moment resisted by the stringer divided by the bending moment ($45.80'^{k}$) calculated from the truck loading. Thus for truck position #3, the distribution factor would be $28.48'^{k}/45.80'^{k} = 0.62$. The load distribution factor specified by AASHO is S/5.5, where S = 3'-9" giving 3.75/5.5 = 0.68 as a distribution factor for this problem. The average of the experimental load factor is:

Truck Position	Strain Gage	Experimental Dist.Factor
#3	#47	0.62
#7	#46	0.57
#1	#46	0.66
	Average experimental load distribution factor =	0.62
	AASHO distribution factor =	0.68

Let us now consider the case where there is no composite action between the deck and the stringer. In this case the moment of inertia for the section is the moment of inertia of the stringer ($I = 517$ in^4 and $S = 64.6$ in^3). Calculate the moment resisted by the stringer using the average stress, considering truck position #3:

Moment = σ I/c = σS

Actual moment carried by the stringer = (64.6 in^3) (3194 psi)
= $17.19'^{k}$

Truck Position	Strain Gage	Actual Moment Resisted
#3	#47	$17.19'^{k}$
#7	#46	$15.80'^{k}$
#1	#46	$18.31'^{k}$

Distribution factor for truck position #3:

17.19/45.8 = .38

The average load factor for all cases is:

Truck Position	Strain Gage	Computed Dist. Factor
#3	#47	.38
#7	#46	.34
#1	#46	.40

Average Computed load dist. factor = .37

Comparing the results considering complete composite action and no composite action, it is evident that through friction there is a large degree of composite action between the deck and the stringers.

Table B1
Stresses (psi) in the Steel Stringers at Section W for Truck Position #3

Test Number	Strain Gages								
	63	62	61	60	46	47	49	50	51
1		305	114	320	2059	3284	2965	3207	1507
2		161	-135	108	1975	3052	2944	3233	1618
3		115	-549	199	2064	3339	3037	3317	1642
4	B	121	-685	93	1879	2798	2770	2936	1523
5				260	2118	3172	2892	3015	2649
6	A	201	-734	233	2233	3175	2855	2974	2020
7	D	128		252	1952	3071	3296	2921	2805
8			56	166	2014	3197	3059	3269	2809
9			-262	185	2041	3254	3041	3206	2796
10	G		-84	213	2381	3513	3114	3126	2442
11	A		-365	232	1466	2586	3191	2660	1422
12	G			253	1769	2820	3329	2944	351
13			-135	262	1898	3041	3336	2807	699
14	E		-156	572	1694	2922	3008	2698	2663
15									
16									
17			-126	136	1909	3151	3471		1112
18									
19									
20									
21				280	1714	2930	3424	2941	1709
22			-284	129	1693	2855	3041		1109
23									
24			193	35	1739	2967	3247		1460
25			499	235	2262			3143	1952
26									
27			480	164	1788	3213	3236		2823
28			274		1583	2981	3282		2489
29			1142	80	1897	2998	3078		1394
30			-1727	189	1544	4882	3171		1760
31			-1053	23	1424	4269	3449		1212

Table B1(continued)

Average of Tests #1, 3 and 5

Strain Gages								
63	62	61	60	46	47	49	50	51
Bad Gage	210	-217	260	2080	3265	2964	3200	1932

Average of Tests #27, 29 and 30

Strain Gages								
63	62	61	60	46	47	49	50	51
Bad Gage		35	144	1743	3697	3161		1992

Table B2
Stresses (psi) in the Stringers at Section W for Truck Position #1

Test Number	Strain Gages								
	63	62	61	60	46	47	49	50	51
1		145	323	759	3348	3207	2351	1761	506
2		147	110	700	3462	3433	2383	1653	426
3		136	270	758	3502	3430	2372	1745	547
4		192	638	678	3157	3056	2167	1623	528
5	B	318	282	820	3493	3324	2178	1560	900
6	A	191	964	448	3588	3475	2183	1460	831
7	D	178	257	879	3441	3388	2668	1860	1452
8			313	549	3315	3625	2504	1729	640
9	G		-175	560	3288	3298	2206	1508	767
10			429	700	3638	3530	2370	1703	
11	A		404	856	3658	3546	2895	1581	212
12	G		520	605	3236	3251	2680	1453	270
13	E		428	302	3296	3211	2521	1361	429
14									
15									
16									
17			621	693	3325	3322	2550		694
18									
19									
20									
21			248	681	3179	3453	2714	1382	1467
22				666	3149	3191	2412		1002
23									
24			487	487	3042	3319	2649	1332	302
25			1262	315	5182	5050	3875	2856	1912
26									
27			394	500	3203	3391	2484		363
28			112	913	3152	3369	2549		666
29			528		3830	970			
30			370	719	3045	6376	2407		662
31			885	670	2730	6163	2572		

Table B2 (continued)

Average of Tests #1, 3 and 5

Strain Gages								
63	62	61	60	46	47	49	50	51
Bad Gage	200	291	779	3447	3320	2300	1688	651

Average of Tests #21, 22 and 24

Strain Gages								
63	62	61	60	46	47	49	50	51
Bad Gage		367	611	3123	3321	2591	1357	923

Table B3
Stresses (psi) in the Stringers at Section W for
Truck Position #7

Test Number	Strain Gages								
	63	62	61	60	46	47	49	50	51
1		472	2812	1621	3268	1781	608	382	101
2		630	1708	2866	3370	1768	613	388	227
3		667	1647	2962	3318	1849	653	402	176
4	B	598	1534	2535	2993	1648	610	351	240
5		773	1896	2838	2978	1614	567	404	478
6	A	630	2208	2921	2989	1474	433	402	617
7	D	464	1727	2863	3239	1935	928	620	519
8			1578	2458	3092	1698	652	435	407
9	G		1713	2532	2987	1602	579	347	307
10	A		2131	2919	3027	1306	378	250	1707
11			2150	2935	3162	1716	783	474	612
12	G		2224	2670	2929	1483	623	300	485
13	E		2032	2952	2990	1470	603	391	-156
14			1328	1655	1475	996	411	392	799
15									
16									
17			2193	2688	2949	1508	540		160
18									
19									
20									
21			2195	2461	3123	1640	606	226	290
22			1466	2516	3098	1733	717		
23									
24			2052	2793	2927	1527	568	273	238
25			1716	2284	2620	1082			
26									
27							458		336
28			2367	2416	2800	1644	578	650	1071
29			2026	1349	2725	1401	569		
30			2767	2539	2675	2793	519		398
31			1661	2685	2786	1617	563		729

Table B3 (continued)

Average of Tests #1, 3, and 5

Strain Gages								
63	62	61	60	46	47	49	50	51
Bad Gage	637	2118	2473	3188	1748	609	396	251

Average of Tests #21, 22 and 24

Strain Gages								
63	62	61	60	46	47	49	50	51
Bad Gage		1904	2590	3049	1633	630	250	264

APPENDIX C

GRAPHS OF STRESS VS. TIME

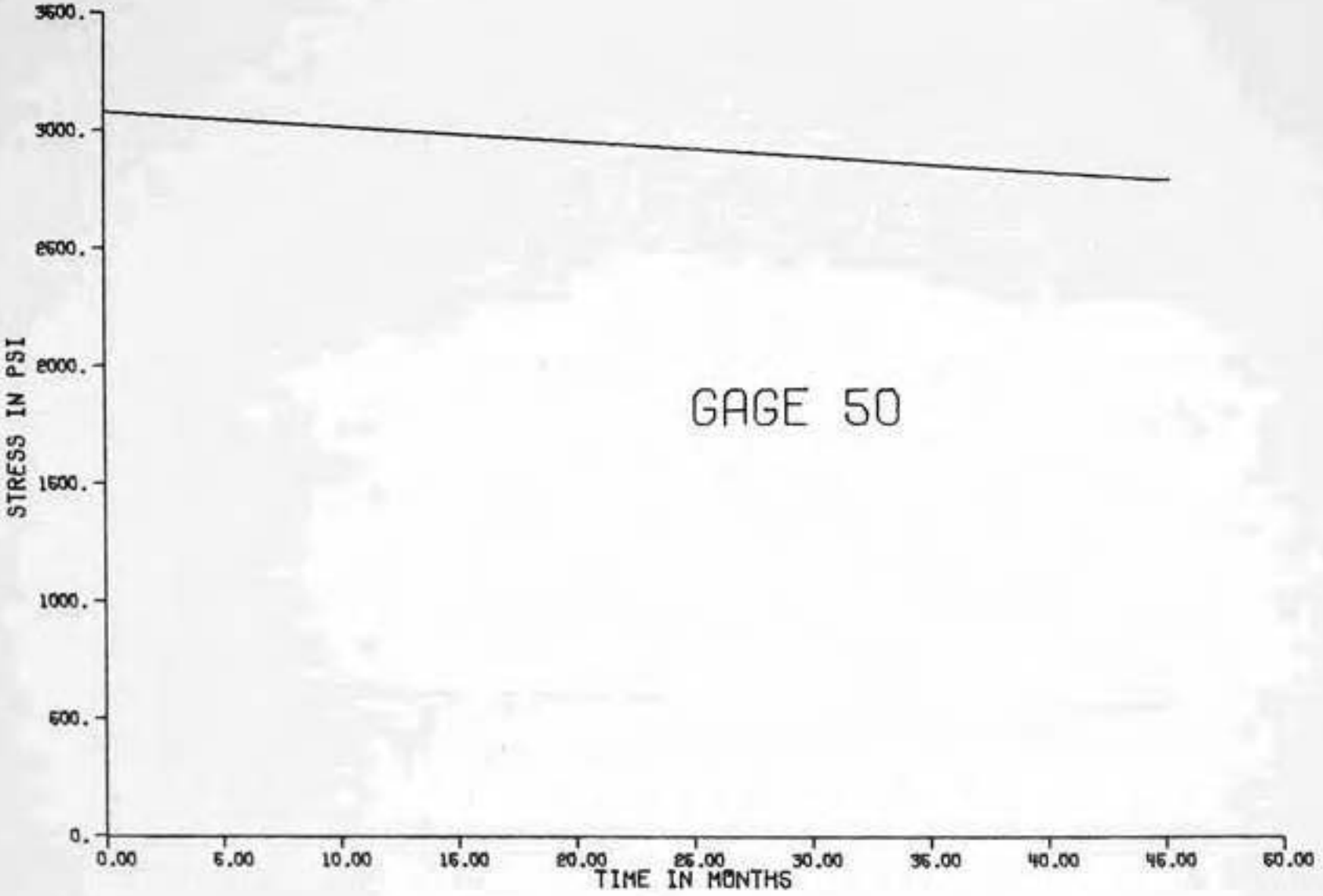


FIGURE C1 STRESS VS. TIME FOR GAGE 50
TRUCK POSITION 3
BLOOMINGTON BRIDGE

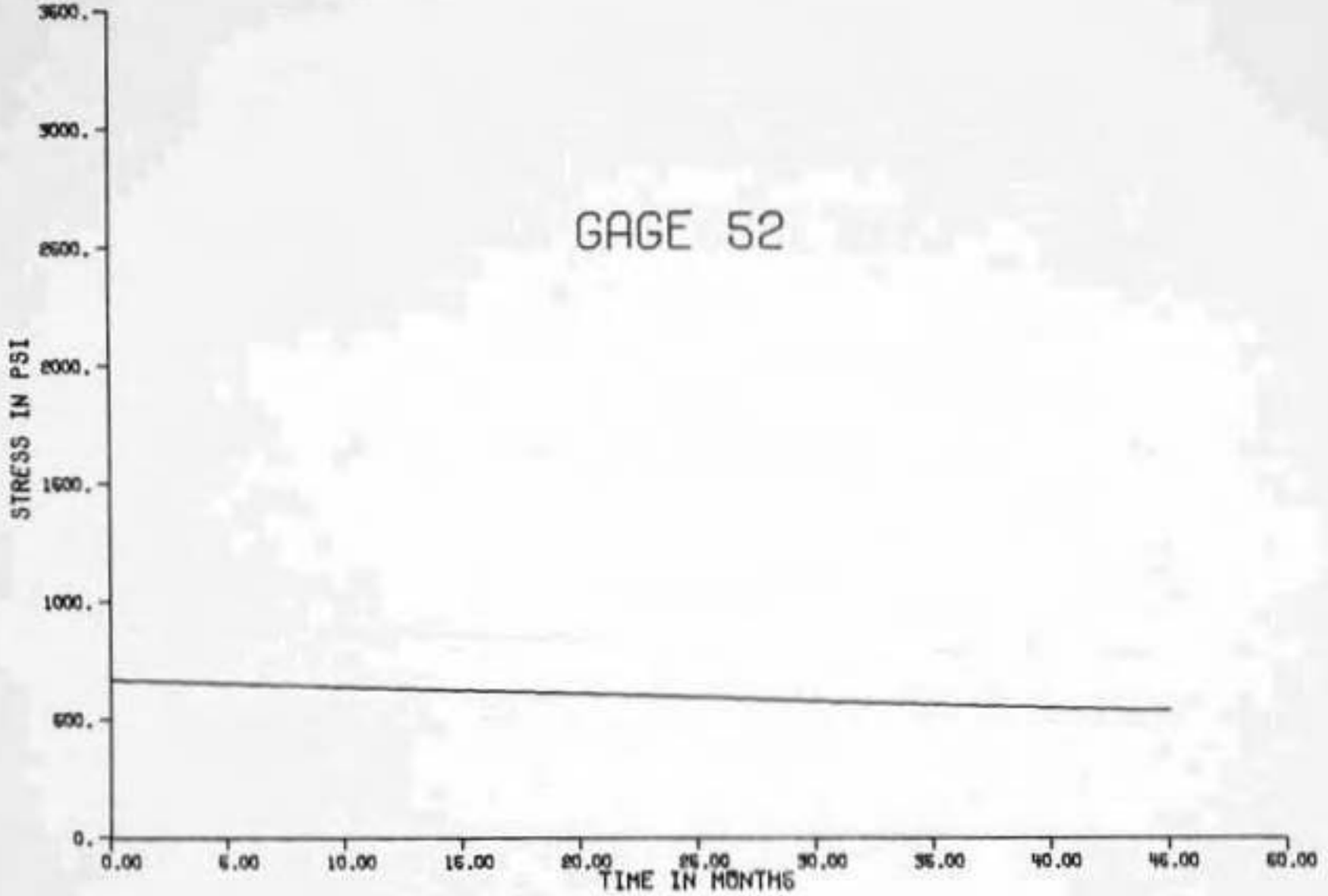


FIGURE C2 STRESS VS. TIME FOR GAGE 52
TRUCK POSITION 3
BLOOMINGTON BRIDGE

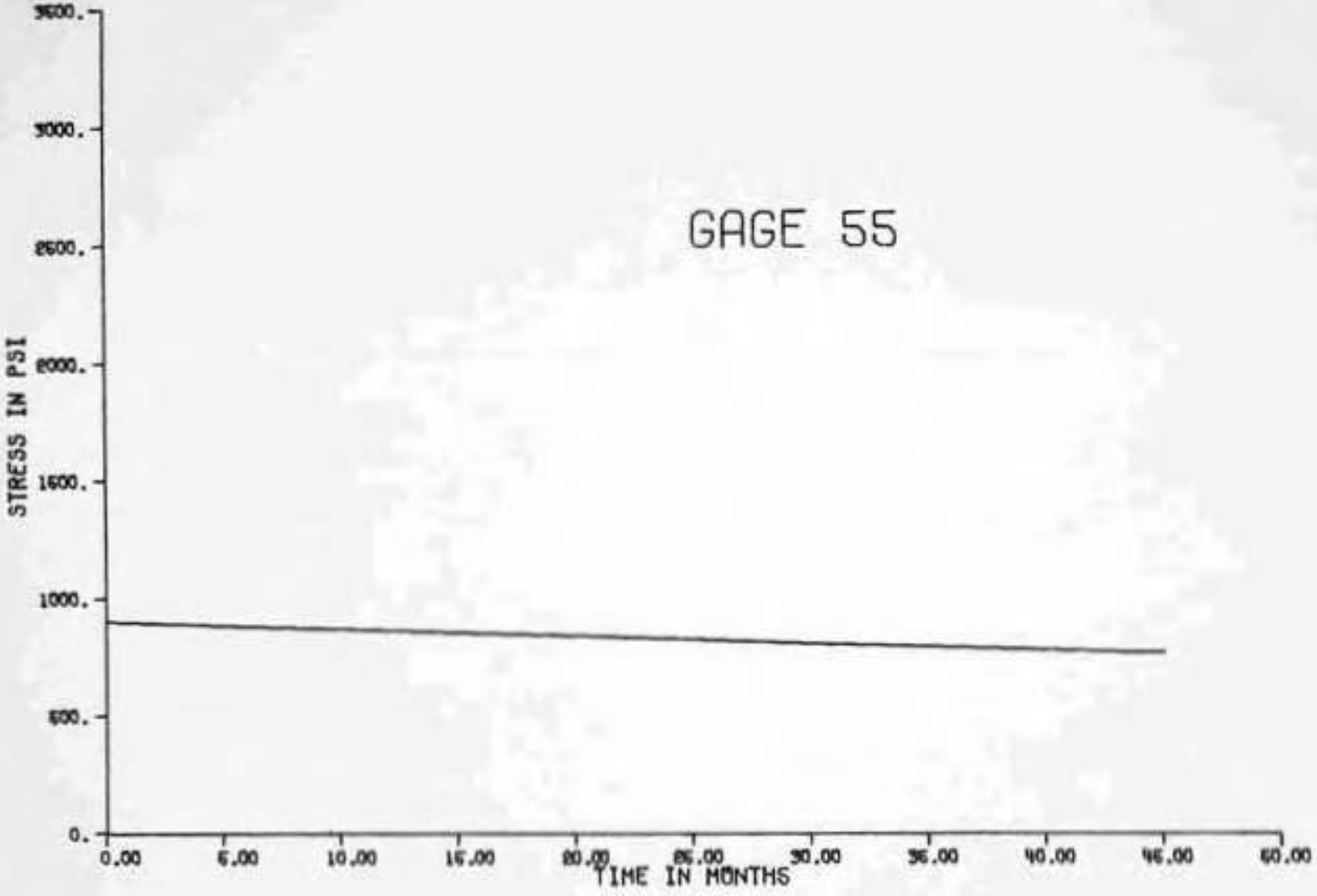


FIGURE C3 STRESS VS. TIME FOR GAGE 55
TRUCK POSITION 3
BLOOMINGTON BRIDGE

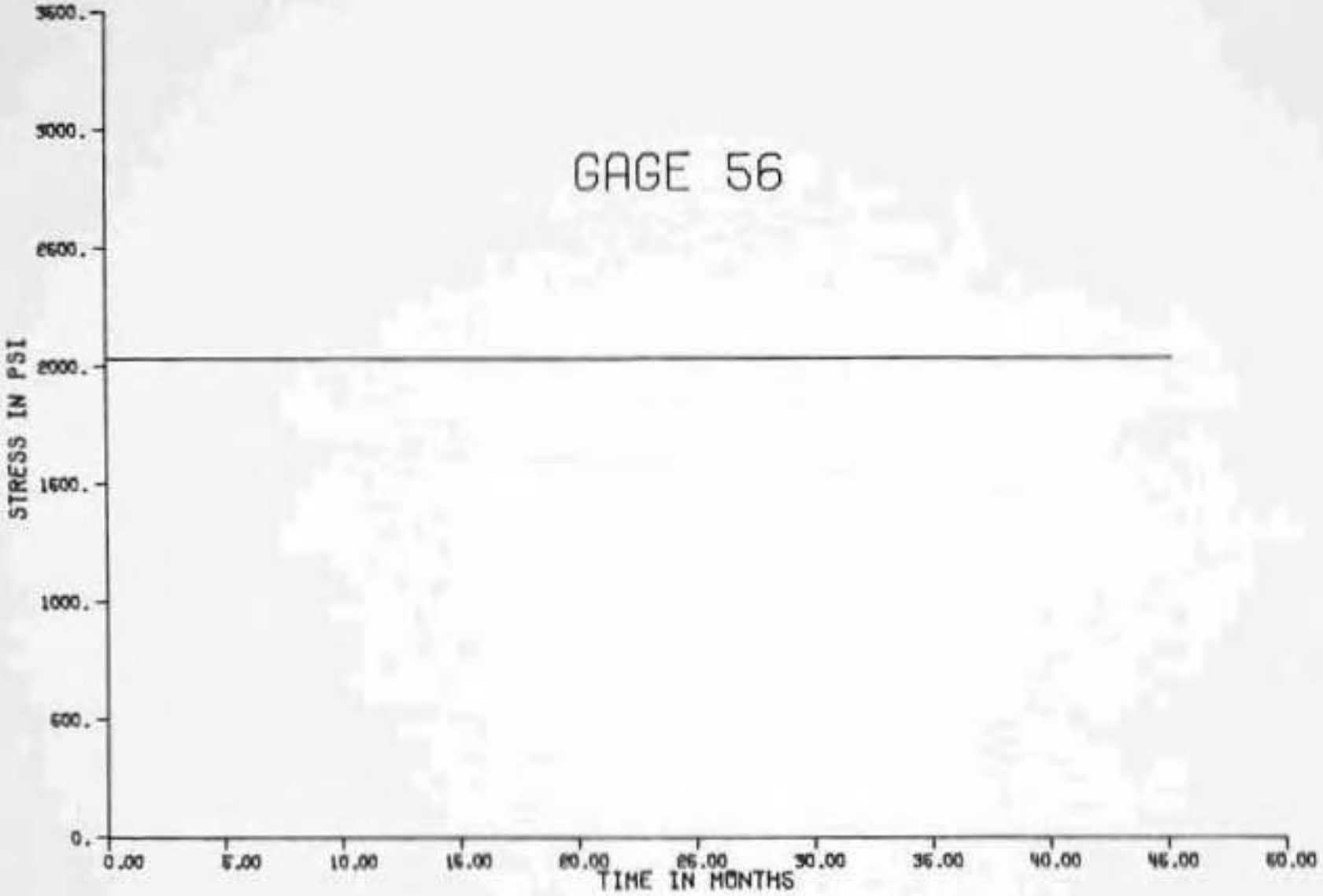


FIGURE C4 STRESS VS. TIME FOR GAGE 56
TRUCK POSITION 3
BLOOMINGTON BRIDGE

FIGURE C5 STRESS VS. TIME FOR GAGE 25
TRUCK POSITION 3
BLOOMINGTON BRIDGE

FIGURE C6 STRESS VS. TIME FOR GAGE 36
TRUCK POSITION 3
BLOOMINGTON BRIDGE

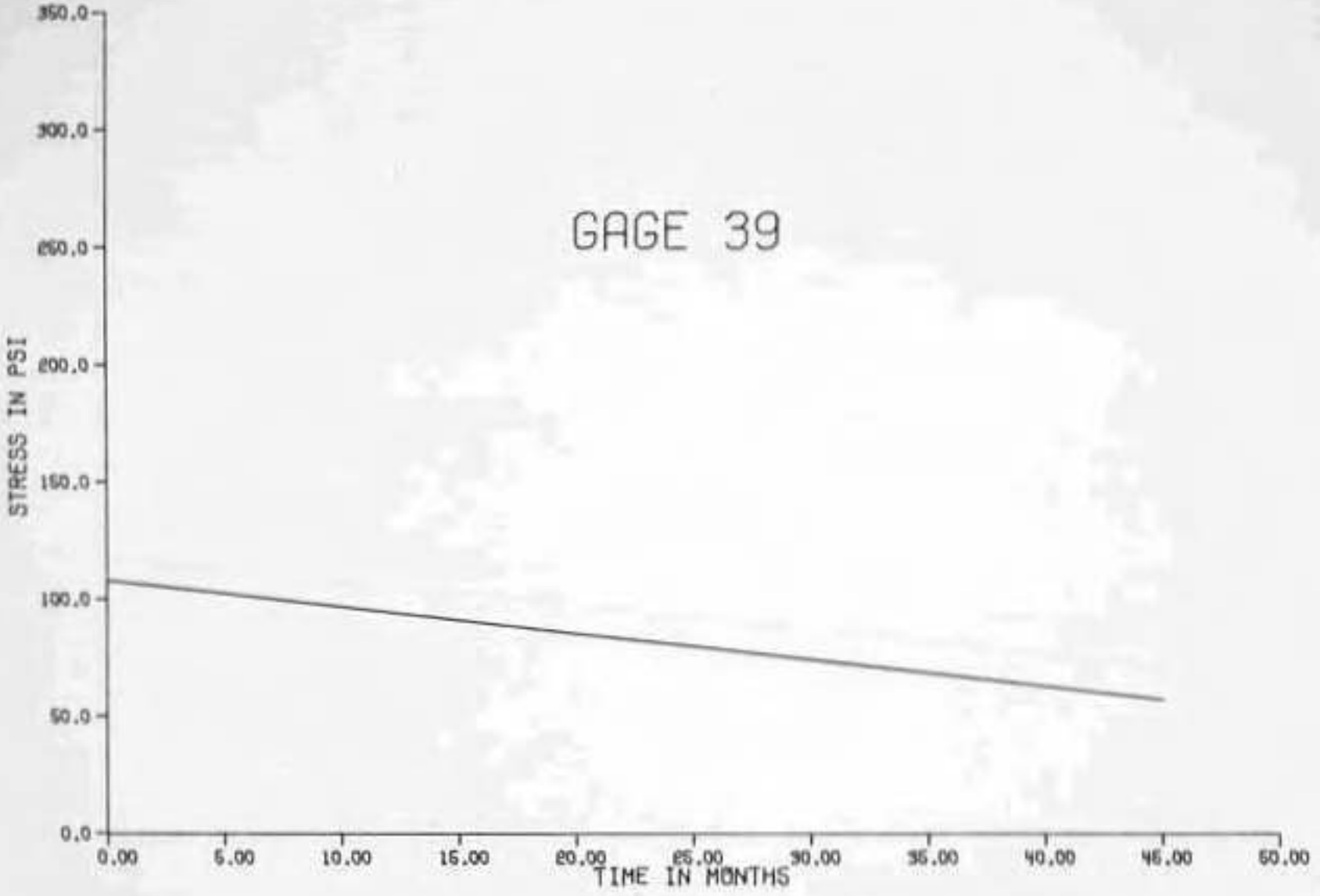


FIGURE C7 STRESS VS. TIME FOR GAGE 39
TRUCK POSITION 3
BLOOMINGTON BRIDGE

FIGURE C8 STRESS VS. TIME FOR GAGE 40
TRUCK POSITION 3
BLOOMINGTON BRIDGE

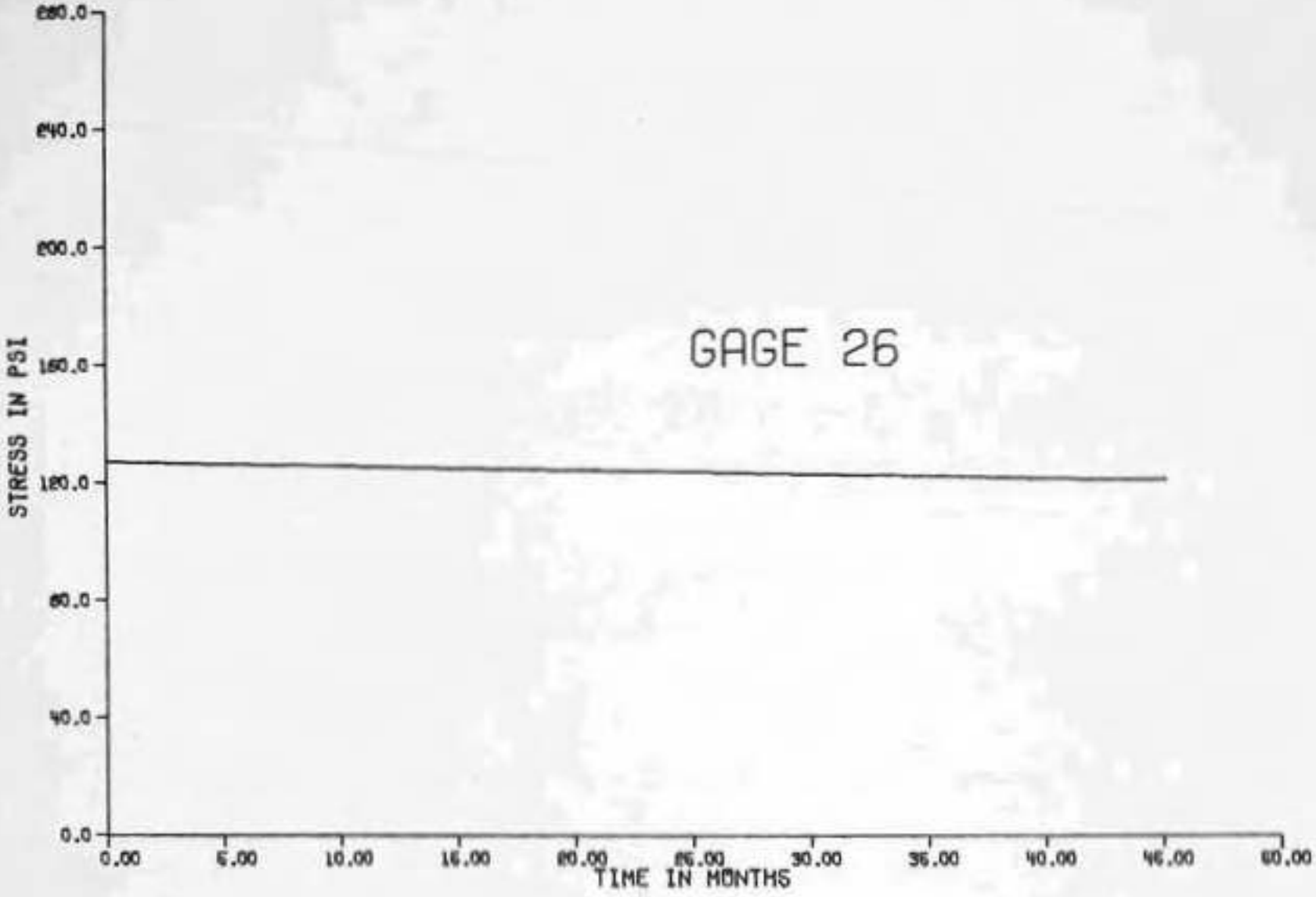


FIGURE C9 STRESS VS. TIME FOR GAGE 26
TRUCK POSITION 14
KNIGHTSTOWN BRIDGE

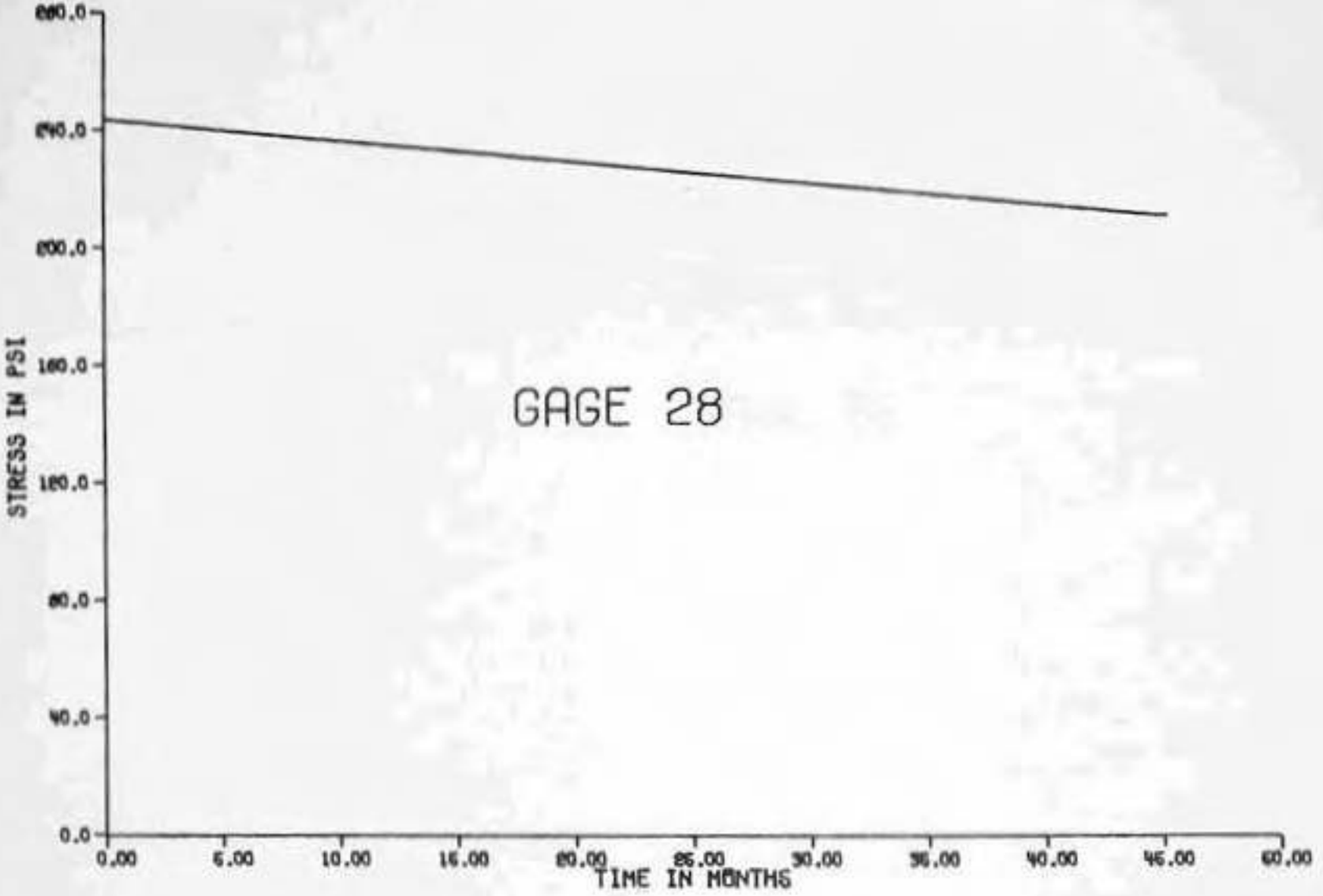


FIGURE C10 STRESS VS. TIME FOR GAGE 28
TRUCK POSITION 14
KNIGHTSTOWN BRIDGE

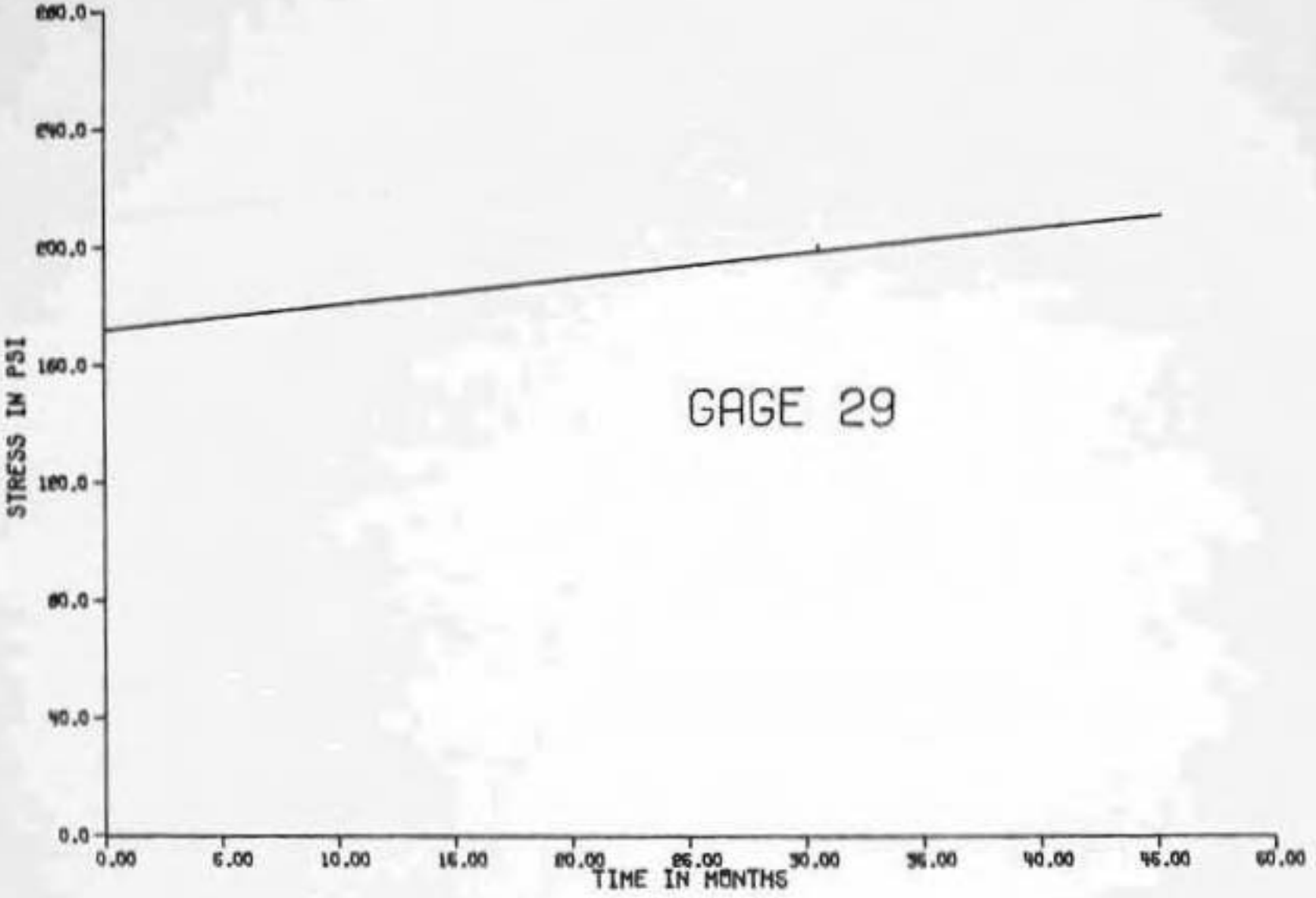


FIGURE C11 STRESS VS. TIME FOR GAGE 29
TRUCK POSITION 14
KNIGHTSTOWN BRIDGE

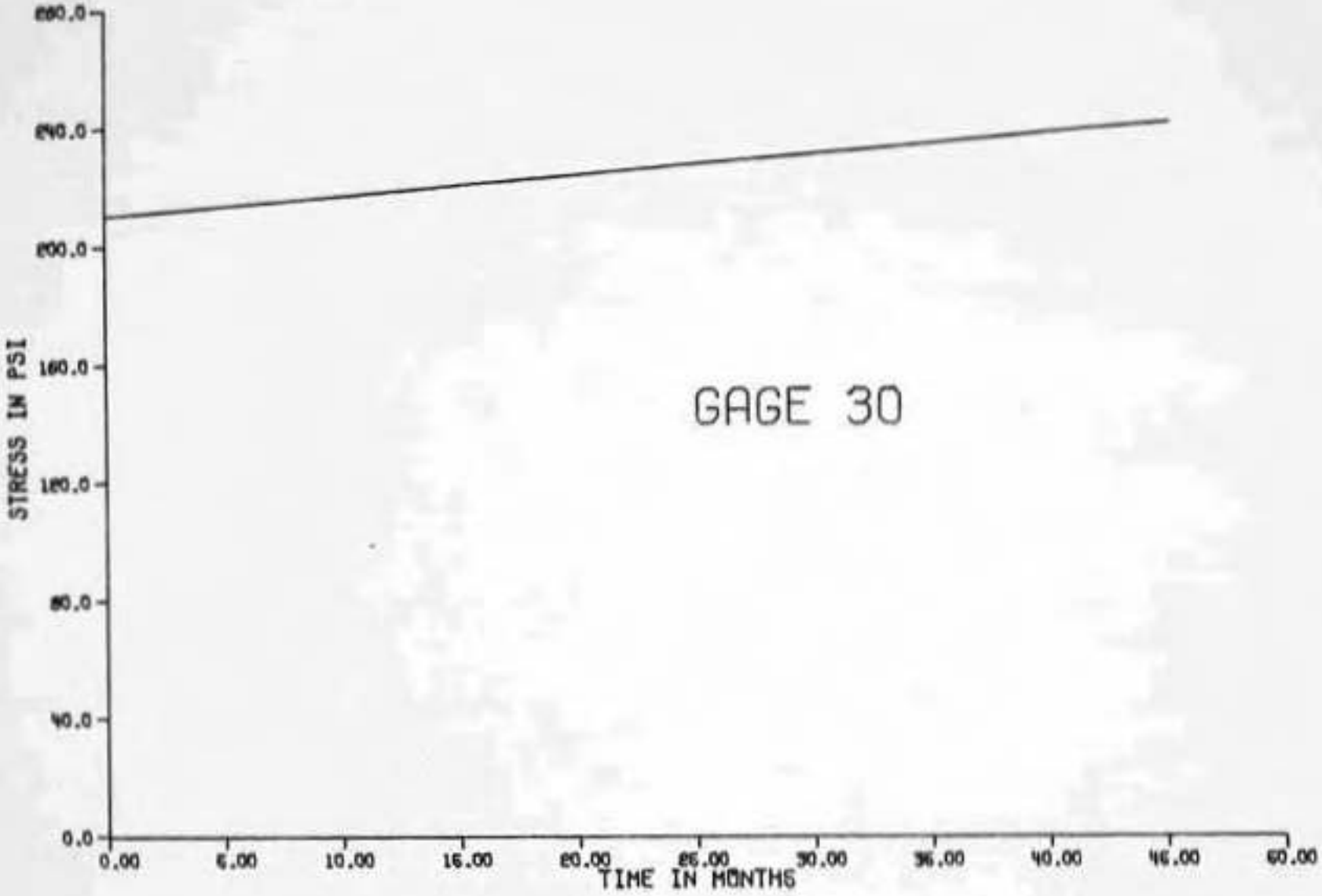


FIGURE C12 STRESS VS. TIME FOR GAGE 30
TRUCK POSITION 14
KNIGHTSTOWN BRIDGE

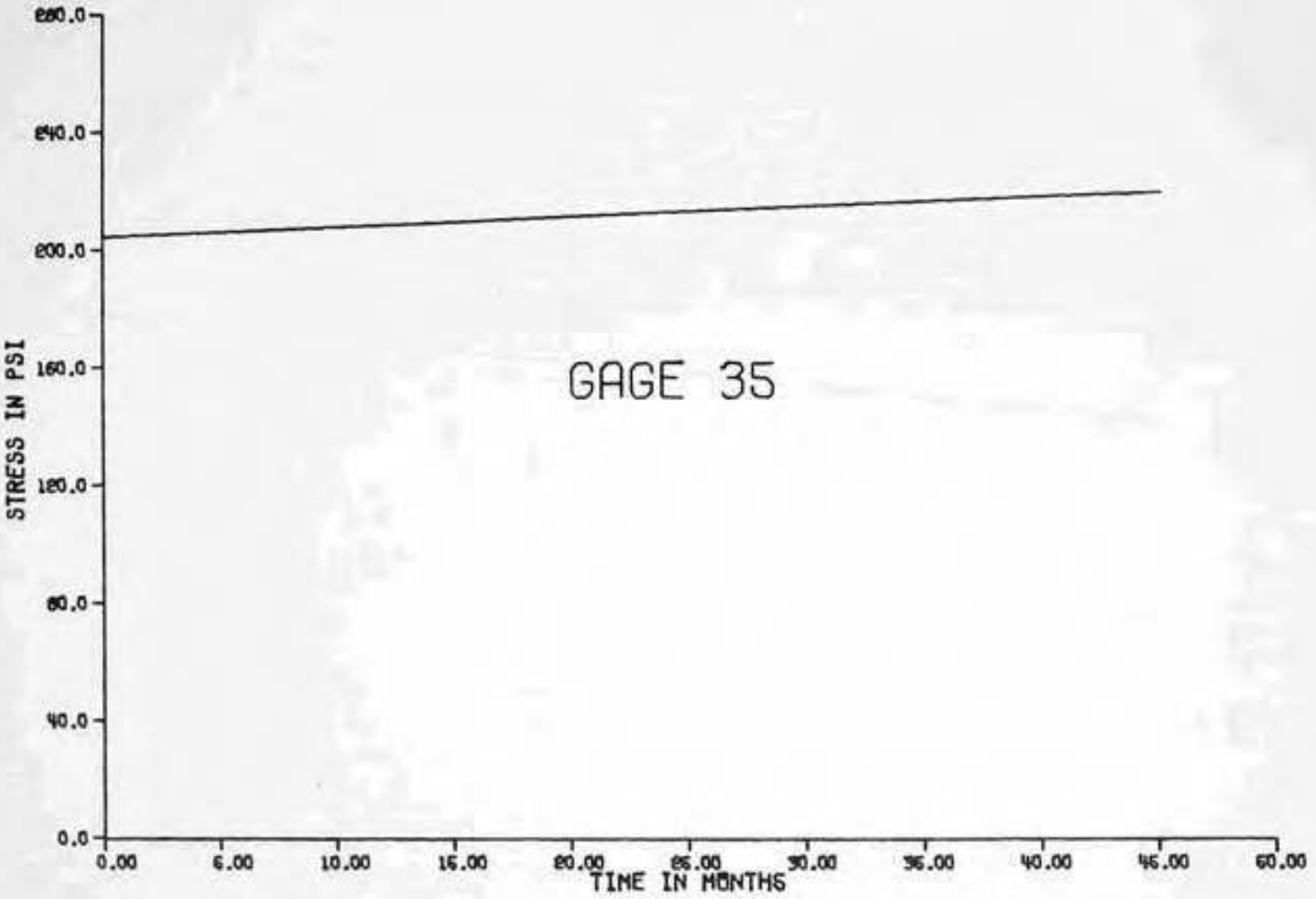


FIGURE C13 STRESS VS. TIME FOR GAGE 35
TRUCK POSITION 14
KNIGHTSTOWN BRIDGE

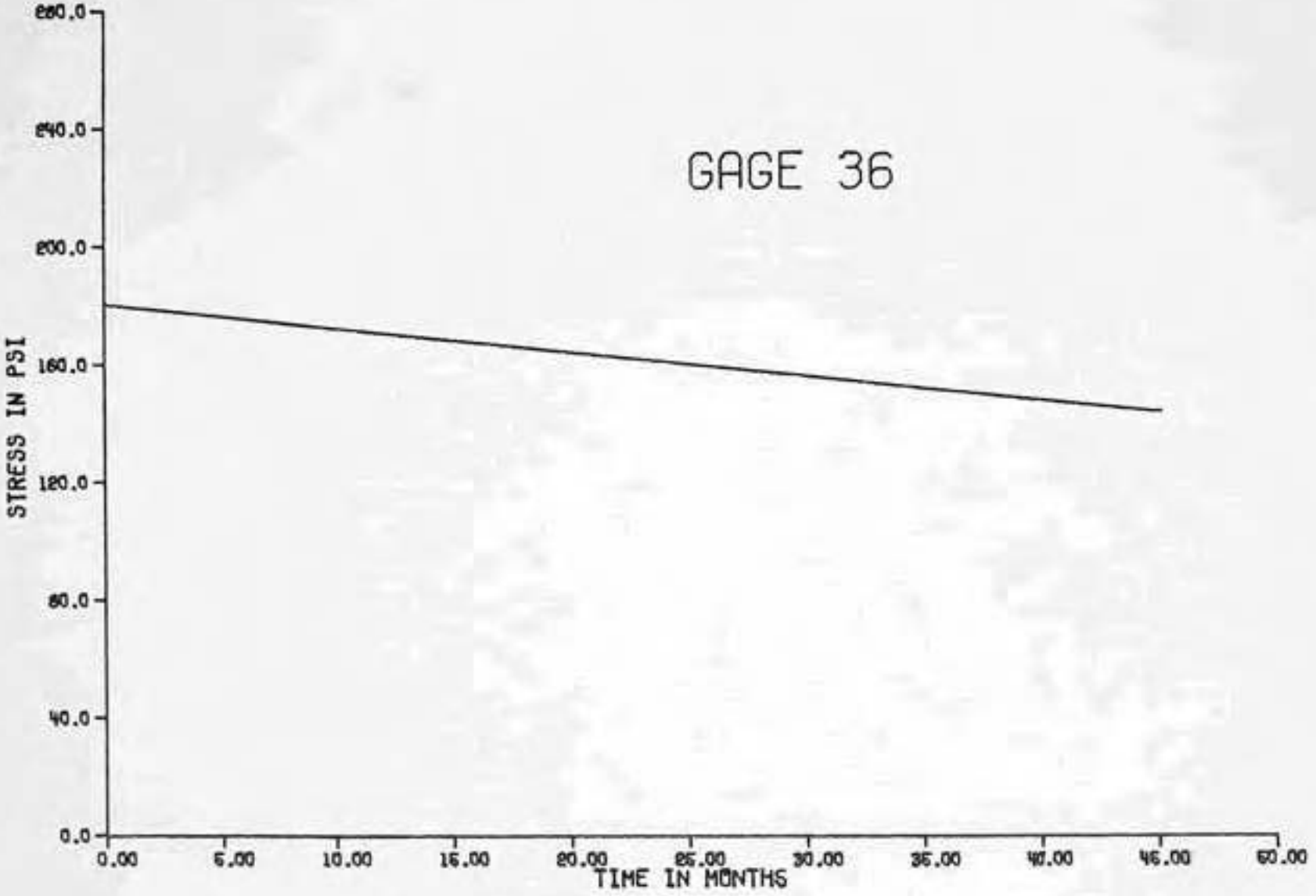


FIGURE C14 STRESS VS. TIME FOR GAGE 36
TRUCK POSITION 14
KNIGHTSTOWN BRIDGE