1358

STUDY OF MICHIGAN'S LINK PLATE AND PIN ASSEMBLIES

MDOT

Michigan Department of Transportation

CONSTRUCTION AND TECHNOLOGY DIVISION

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Technical Report Documentation Page

1. Report No. Research Report R-1358	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Study of Michigan's Link Plate and Pin Assemblies		5. Report Date April 1998
7. Author(s) David Juntunen		6. Performing Organization Code
9. Performing Organization Name and Address Michigan Department of Transportation Construction and Technology Division P.O. Box 30049 Lansing, MI 48909		8. Performing Org Report No. R-1358
12. Sponsoring Agency Name and Address Michigan Department of Transportation Construction and Technology Division P.O. Box 30049 Lansing, MI 48909		10. Work Unit No. (TRAIS) 11. Contract/Grant No.
15. Supplementary Notes		13. Type of Report & Period Covered Final Report 14. Sponsoring Agency Code 93TI-1683

16. Abstract

This report takes a comprehensive look at the condition of Michigan's link plate and pin assemblies. The condition rating of the population was reviewed, live load stresses were measured, fatigue evaluations were done, and problems that can be expected with link plate and pin assemblies were documented. Michigan has approximately 1,100 bridges with link plate and pin assemblies. Fifteen percent of these bridges are rated in *poor* or *serious* condition, and 20 to 30 percent of these structures have had the original link plates and pins replaced. Problems we have found with this detail are; corrosion, cracking, and beam ends pushing against each other at the link plates. Michigan has twelve bridges with link plate and pin assemblies on non-redundant spans. All of these bridges have had the link plates and pins replaced, and all twelve bridges are inspected on a yearly schedule. A live load study of link plates has revealed in-plane as well as out-of-plane bending often occurs in link plates, and the number of load cycles per truck in the link plate can vary from 2 to 10 cycles per truck. When designing link plates, the design code chosen will greatly affect the required size of the link plate. A *Michigan Fatigue Truck* was developed for analyzing fatigue details in Michigan.

17. Key Words Link Plates, Steel Bridges, Pin and Hangers, Fatigue		18. Distribution Statement No restrictions. This document is available to the public through the Michigan Department of Transportation.	
19. Security Classification (report) Unclassified	20. Security Classification (Page) Unclassified	21. No of Pages	22. Price

Form DOT F 1700.7 (8-72)

MICHIGAN DEPARTMENT OF TRANSPORTATION
MDOT

Study of Michigan's Link Plate and Pin Assemblies

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Research Project 93 TI-1683
Research Report No. R-1358

Lansing, April 1998

ACKNOWLEDGEMENTS

Many people participated in this project. While space and memory will not allow a complete list of everyone's involvement, the following people and work groups should be mentioned; Roger Till for editing and guidance, Douglas Needham, Larry Pearson, Chris Davis, Nick Lefke, Michael Isola, engineers on the engineering development program (EDPs), Co-op students and the Instrumentations and Data Systems Unit.

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INTRODUCTION

In March 1993 Dr. John Fisher of Lehigh University proposed that the American Association of State Highway and Transportation Officials (AASHTO) specify the net section across the pin hole of link plates be designed for Fatigue Category E. Although Dr. Fisher has found that link plates designed by AASHTO provisions have generally been satisfactory, he felt test data from Nishimura[1] warranted setting this lower bound for carbon steels and alloy steels until further research could be completed to justify modification of this requirement. The fatigue requirement was eventually adopted in the AASHTO LRFD Bridge Design Specifications, 1994 Edition[2] and the AASHTO Standard Specifications for Highway Bridges 1996 Edition[3]. (Note - the two codes are referred to in this report as AASHTO LRFD, and AASHTO Bridge Specification respectively).

Michigan has a large inventory of bridges having link plate and pin assemblies. Many of these bridges have been in service for 20 to 30 years. If the fatigue category for this detail is indeed low, it would be prudent to evaluate the condition of Michigan's link plates and to study the actual live loads they experience. With this purpose, the Structural Research Unit instrumented several of Michigan's link plate bridges to determine the stress range resulting from truck loads. Concurrently, the Structural Research Unit evaluated various problems associated with link plate and pin assemblies.

This report takes a comprehensive look at the condition of Michigan's link plate and pin assemblies. The condition rating of the population was reviewed, live load stresses were measured, fatigue evaluations were done, and problems that can be expected with link plate and pin assemblies were documented.

INVENTORY OF MICHIGAN'S BRIDGES

Michigan has approximately 1,100 bridges with pin and hanger details. Figure 1 shows a histogram of the condition rating of link plates and superstructures, along with the definition of each rating. There are 170 bridges, or 15 percent with link plate and pin assemblies rated in *poor* or *serious* condition, and 54 percent of these structures are rated in *good* condition or better. We can also see the condition of the link plates are almost always rated the same as the superstructure. Figure 2 shows the number of bridges with link plate and pin assemblies built in a given decade. Link plate and pin assemblies are replaced when a bridge is scheduled for work such as painting or deck joint replacements, or when a problem associated with the link plate and pin assembly is discovered. The decision to update the detail is usually made by a bridge scoping engineer or designer. By our estimates, approximately 20 to 30 percent of Michigan's structures with link plates have had the original link plate and pin assemblies replaced with new ones.

Special consideration is given to Michigan's twelve bridges with link plate assemblies on non-redundant spans; i.e., only two beams provide primary support for the span or beam spacing is wide enough that if one beam fails the structure could become unstable. The bridges are given detailed inspections annually, and in the mid 1980's the department replaced the link plates on all of these bridges. The non-redundant bridges are as follows:

US-12 over St. Joseph River (Structure Number - B01 of 11101)

The average daily truck traffic is estimated to be 760 trucks per day (two-way traffic). The link plates and pins were replaced in 1988.

I-96 over Grand River and Billwood Highway (Structure Numbers - B01 and B02 of 23151)

The average daily truck traffic is estimated to be 5,100 trucks per day (two-way traffic). The link plates and pins were replaced in 1987.

M-20 over CSX Railroad and Titabawassee River (Structure Number - R01 of 56021)

The average daily truck traffic is estimated to be 350 trucks per day (two-way traffic). The link plates and pins were replaced in 1985.

US-10 over Sanford Lake (Structure Number - B04 of 56044)

The average daily truck traffic is estimated to be 400 trucks per day (two-way traffic). The link plates and pins were replaced in 1985.

I-75 over Conrail Railroad and Raisin River (Structure Number - R03 of 58151)

The average daily truck traffic is estimated to be 10,900 trucks per day (two-way traffic). The link plates and pins were replaced in 1987.

M-37 over Muskegon River (Structure Number - B01 of 62031)

The average daily truck traffic is estimated to be 300 trucks per day (two-way traffic). The link plates and pins were replaced in 1986.

US-31 over Pentwater River (Structure Number - B01 of 64012)

The average daily truck traffic is estimated to be 900 trucks per day (two-way traffic). The link plates and pins were replaced in 1987.

M-29 over Black River (Structure Number - B03 of 77111)

The average daily truck traffic is estimated to be 430 trucks per day (two-way traffic). The link plates and pins were replaced in 1988.

M-14 and US-23 over Conrail RR and Huron River (Structure Number - R01 of 81075)
Average daily truck traffic is estimated to be 5,000 trucks per day (two-way traffic). The link plates and pins were replaced in 1990.

I-94 over GTW Railroad and Russel Road (Structure Number - R01 of 82024)

The average daily truck traffic is estimated to be 7,000 trucks per day (two-way traffic). The link plates and pins were replaced in 1985.

M-37 over Pine River (Structure Number - B01 of 83011)

The average daily truck traffic is estimated to be 220 trucks per day (two-way traffic). The link plates and pins were replaced in 1987.

PROBLEMS ASSOCIATED WITH LINK PLATE AND PIN ASSEMBLIES

As pointed out by Dr. Fisher, *few eyebars and pin plates have cracked from fatigue loading*, however, by reviewing literature and our own experience here in Michigan we have found several problems associated with this detail.

PROBLEMS DOCUMENTED IN OTHER STATES

Point Pleasant Bridge (US 35 over the Ohio River), West Virginia[4] - A 445 m (1,460 ft) eyebar suspension bridge collapsed suddenly in 1967. An eyebar is similar to a link plate, except the section between the pin holes is reduced. An extensive study of the collapse revealed that it was the result of stress corrosion cracking, where the eyebar material was found to be stress corrosion sensitive. The cracks first developed in an area of high local hardness. There was no evidence of fatigue crack growth at the point of failure.

Interstate Route 95 over the Mianus River, Connecticut - The structure was a non-redundant bridge with a suspended span supported by link plate and pin assemblies. In June 1983 one of the suspended spans suddenly collapsed. The investigation into the cause of the collapse revealed three primary causes; one, a build-up of rust on the back of the link plates forced them away from the girder webs; two, there was high design bearing stresses on the link plate and the pin; and three, there was out of plane deformations in the heavily skewed bridge[5]. The failure of the link plate at this bridge was not the result of fatigue.

Illinois Route 157 over St.Clair River[6]**.** A district employee of the Illinois Department of Transportation discovered three link plates were fractured on a bridge causing the beams to drop 13 to 19 mm below the deck. The bridge had multiple beams supporting a reinforced concrete deck with seven spans crossing at various angles. One span was suspended by link plates and pins. An investigation revealed that the cause of the fractured link plates was the result of repeated high in-plane bending, which resulted after the pin connection froze due to corrosion. High bending stresses occurred at the edge of the corrosion *welded* region due to thermal expansion of the bridge during the summer.

PROBLEMS DISCOVERED ON MICHIGAN'S LINK PLATE AND PIN ASSEMBLIES

Fractured Link Plate. Michigan has at least one known experience where a link plate has fractured. This occurred on the structure carrying M-46 over the Tittabawasse River, 6.4 km west of Saginaw (Structure Number - B02 of 73062). The bridge has seven spans with 14 steel beams supporting a reinforced concrete deck with a longitudinal open joint running along the center line of the bridge. There are three suspended spans supported by link plate and pin assemblies. A department bridge inspector discovered the fractured link plate, shown in Figure 3 during a routine inspection of the bridge. The link plate was broken near the top of the bottom pin hole. Since the structure was scheduled for link plate and pin replacement, the link plate was repaired temporarily with a welded plate crossing the fracture. No further analysis was done to determine the cause of the fracture, although the bridge inspector observed that the expansion joint at the link plate had closed causing the diagonal portion of the beam webs across the link plate to push against each other.

Corroded Link Plates. This is the most common problem associated with link plate and pin assemblies. Expansion joints above the detail leak water contaminated with road salts on to the link plates causing them to corrode. Similar to the Mianus River incident, Michigan has experienced corrosion build-up between the link plate and the beam web. On the bridge carrying I-75 over the Huron River (Structure Number - B04 of 58152), a department bridge inspector discovered this problem on several of the link plates. As shown in Figure 4, the corrosion product caused the link plate to move outward causing the pin end cap to *cup*. We have also seen that corrosion products can cause fixity against rotation. On I-75 over Toledo/Dix Avenue, discussed later in this report, we measured in-plane bending in the link plates as a result of corrosion products.

Beam Ends in Contact. A repeated problem Michigan has recently encountered with link plate and pin assemblies is when the expansion gap between the beams closes. Figure 5 shows a typical closed expansion joint. When this happens, the beam ends, sometimes, will make contact along the diagonal portion of the beam web. As the bridge wants to expand during warm weather, the beam ends will push against each other. The resulting force will be directed into the link plates across the diagonal portion of the beam webs.

In an effort to measure the stress increase, we mounted vibrating wire strain gages on the link plates of the bridge carrying I-196 over the Grand River in Grand Rapids, Michigan (Structure Number B02 of 41027). In the link plate where the beam ends were in contact, strain was measured for one year to see if dead load stress varied with seasonal changes in temperature. We discovered the stress increase can be substantial. Figure 6 shows the results of the strain measurements referenced to temperature for a link plate where the beam ends were pushing against each other. From the strain data we estimate the stress in the link plate increased about 2.3 MPa per degree Celsius. For a 60^0 C temperature change the stress increased approximately 140 Mpa. We then mounted vibrating wire strain gages on a similar link plate where the beam ends were in contact. When the link plate was removed from the bridge we measured the change in the gages to find a difference of 230 MPa, which is a substantial amount of additional dead load stress in the link plate.

As several bridges with the beam ends in contact were inspected, signs of high stress became evident. If a large compressive force is present in the beam web across the beam ends the web may bulge as shown in Figure 7, or the beam web may buckle causing a bend in the bottom flange as shown in Figure 8. On the structure carrying I-275 over M-153 near Detroit, Michigan (Structure Number - S08 of 82292), the buckled beam web caused the link plate to became disengaged from the pin, shown in Figure 9. The beam web had buckled slightly at the first web stiffener/diaphragm connection plate causing misalignment of the beam webs beneath the link plates. Figure 10 shows how out-of-plane bending resulted in the link plates which in turn caused one of the link plates to *walk* off the pin, shearing the cotter pins in the process. On this bridge, we found several link plates and pin assemblies had varying degrees of beam web misalignment and link plate movement. We also found the beams adjacent to the problem areas had closed expansion joints with the beam ends touching.

LOADING PATTERNS IN LINK PLATES

During this study, we observed three types of load conditions in link plates; axial loading, in-plane (strong axis) bending, and out-of plane (weak axis) bending. The following sections describes each.

Axial Loading

Axial loading is a load passing concentrically through the body of the link plate as shown in Figure 11. This is the type of load that is assumed when a link plate is designed. Because stress concentrations occur at the pin holes, the net section of the link plate across the pin hole is the critical section of the link plate for carrying load. For static strength design, the AASHTO Bridge Specification requires that the net section be sized 40 percent larger than otherwise required to account for the stress concentration.

For the live load study we needed to understand the nature of the stress flow through the link plates. Strain gages were placed at the midsection and at the pin hole of a reduced size link plate as shown in Figure 12. Table 1 shows the results of a tension test. Notice the difference between the values of the gage placed at the pin hole and the gages at the mid section decrease as greater load is applied to the link plate. The changing values likely indicate the change in stress flow as the bearing area of the link plate on the pin increases as load is applied; i.e., - as more load is applied, the link plate deforms to match the shape of the pin along the bearing area. To provide a visual representation of the stress concentration, we applied a photoelastic coating to the link plate. In Figure 13 we see the stress concentration does not always appear ideally at the side of the pin hole even in a controlled lab test.

In-Plane Bending

As discussed above, a known problem with link plate and pin details is corrosion. Over time corrosion product causes resistance to rotation of the link plate about each pin. Corrosion product build-up between the link plate and beam web also causes fixity. As a result, in-plane bending (illustrated in Figure 14) occurs in the link plate when trucks drive across the bridge causing midspan deflection and beam end rotation, and when the bridge wants to expand or contract from temperature. Bellenoit[7] measured this response by mounting strain gages on the link plates of two structures and measuring strain as truck loads crossed the bridge. He concluded that *hanger plates of suspended span bridge girders are subject to bending as well as axial forces.*

As discussed earlier, this was the cause for the fracture in the Illinois Route 157 over the St. Clair River. When in-plane bending occurs the stress flow in the link plate changes considerably. On the Illinois bridge, the link plate fractured approximately 40 mm above the pin hole.

Out-of-Plane Bending

Weak axis bending, shown in Figure 15, can be caused several ways; by vehicles impacting an expansion joint that is not perpendicular (at an angle) to the traffic flow, thermal forces on skewed and horizontally curved bridges, differential deflection of the adjacent beams, radial force exerted by a vehicle on a horizontal curve, and misalignment of the beam web (as on Structure number S08 of 82292, M-153 over I-275). During our study, we could easily see out-of-plane bending on the link plates on the bridge carrying I-75 over Toledo/Dix as trucks crossed the bridge. This is discussed further in the results section for this structure.

LIVE LOAD STRESSES IN LINK PLATE AND PIN ASSEMBLIES

The variables that affect the stress magnitudes in a link plate are truck weights, impact factor, lateral distribution of the truck loads, length of the span, redundancy of beams, angle of crossing, and corrosion in the link plate. Additional variables that affect the fatigue life of the link plate are; average daily truck traffic (ADTT); number of traffic lanes; percentage of trucks in a given lane; and number of stress occurrences per truck passage.

During this project, we studied live load strains in the link plates on three bridges. All the bridges had high ADTT's. The first bridge was in good condition with new link plates, the second bridge had non-redundant spans, and the third bridge had very old, corroded link plates. A fatigue analysis was done for each bridge and live load strains were measured. The measured values were compared to allowable values as per the AASHTO codes.

Methodology

The AASHTO Guide Specifications for Fatigue Evaluation of Existing Steel Bridges, 1990[8] (referred to in this report as *AASHTO Fatigue Guide*) provides guidance for analyzing bridges for fatigue.

Two important variables in a fatigue study are the number of load occurrences and the magnitude of stress range for each occurrence. For the passage of each truck, stress is applied to the link plate. Since each truck has a different weight and axle configuration, and considering that vehicles will be traveling at various speeds and in specific traffic lanes, the resulting stress to the link plate will also vary. The summation of these stresses is called the *variable amplitude stress range*. To relate these values to laboratory tests done at a constant amplitude, Miner's Rule is used as shown in the following equation and referenced in the AASHTO Fatigue Guide, Section 2.1:

$$S_r = [\Sigma f_i S_{ri}^3]^{\frac{1}{3}}$$

S_r = Constant amplitude stress range, or effective stress
f_i = Fraction of stress range within an interval
S_{ri} = Mid-width of an interval

As pointed out by Nowak[9], the accuracy of the effective stress is dependant on the stress ranges included in the calculation. This quickly became evident to us during this study. To account for this, we choose several cut-off points for the lower bound stress range when calculating the effective stress, and the resulting value was matched with a corresponding number of load occurrences.

For each bridge, the following data was collected:

1. Electronic resistance strain gages were attached to the link plates to collect strain data as normal live load traffic crossed over the bridge. The strain data was collected on digital audio tape, and then later processed at 100 Hz (cycles per second) so it could be analyzed by computer. Stress occurrences were counted using the Rainflow Method[10]. Stress histograms were compiled for number of occurrences at a given stress. The maximum stress occurrences along with the time were collected.

2. The number of trucks passing over the bridge and the respective lane the vehicle was traveling in was recorded using video tape. The video tape recorded the time, type of truck, and what lane the vehicle was traveling in. The total number of vehicles was counted so we could calculate the percentage of truck traffic. The number of axles for each truck was collected.

3. Histogram tables for the stress occurrences were created and the effective stress (S_r) was computed and compared with the allowable stress for Category E as shown in the *stress range versus number of cycles* graph from AASHTO LRFD, Figure C6.6.1.2.5-1.

4. Stress verses time plots were created for the three truck occurrences producing the highest stress in the link plates. The plots are useful for determining the number of stress cycles per truck passage and comparing magnitudes of the strain data from each of the gages placed on the link plate. A review of how each gage reacts to the load can tell us what type of loading patterns are in the link plates such as axial loading, strong axis bending, or weak axis bending.

We wanted to compare the measured effective stress in the link plates to the calculated effective stress from equations in the AASHTO codes. Using the AASHTO Fatigue Guide, we calculated the effective stress at the net section of the link plate across the pin hole. Since it is customary not to include stress concentrations from defects or local geometry, we did not include the localized stress concentration near the edge of the pin hole when calculating the effective stress.

Since, in Michigan, heavier trucks are allowed on the roadways than any other state, the weight of the fatigue truck used in the analysis was adjusted to be more representative of the truck loads on bridges in Michigan. Section 2.2 of the AASHTO Fatigue Guide was used along with weigh in motion data from Nowak[9] to calculate the effective gross weight of trucks on Michigan's roadways. The data included 14,698 trucks, which were weighed as they crossed eight different bridges in southern Michigan. The resulting effective gross weight of these trucks was 320 kN. The weight was distributed to three axles in a similar configuration to the truck shown in the AASHTO Fatigue Guide as shown in Figure 16. Coincidently the trucks configuration and weight matches that of the AASHTO HS-20 truck with a 9.1 m spacing between the second and rear axle.

Using the *Michigan Fatigue Truck,* we calculated the effective stress range at the net section of the link plates for each of the bridges in the study. Section 3 of the AASHTO Fatigue Guide was used to determine the link plate's *finite life*. We calculated the *mean life,* which is defined by the AASHTO Fatigue Guide as *the best possible estimate* of the details fatigue life. The *safe life* was also calculated, which *provides a probability of 97.7 percent that fatigue cracks will not occur for redundant members, and a 99.9 percent probability for non-redundant members.*

Finally, the two design codes, AASHTO Bridge Design Specifications, and AASHTO LRFD were used to calculate the size of the link plates to meet fatigue requirements. This was compared to the size of the existing link plates, which were designed based on tensile strength.

In the following sections a description of each bridge chosen for testing is given. The test methods are discussed and results are shown for each bridge.

I-96 over M-52 South of Weberville (Structure Number S02 of 33085)

The bridge carries two lanes of west bound I-96 traffic. The structure has three spans, the center span being supported by link plate and pin assemblies, shown in Figure 17. The composite reinforced concrete deck is supported by seven steel beams. The beams are on a 53 degree angle (counter clockwise) to the piers and abutments. Expansion is allowed at the link plate and pin assembly at the east end of the suspended span and the west assembly is fixed for expansion by a splice plate bolted to the top flanges of the beams. The bridge was built in 1962, and the link plate and pin assemblies were replaced in 1992.

In 1995, the Average Daily Traffic (ADT) over the bridge was 36,000 vehicles per day (two-way traffic), and the Average Daily Truck Traffic (ADTT) was 5,900 (two-way traffic). Since the link plates were new and expected to last for some time, we needed to determine the estimated traffic growth in the future. Traffic forecasters estimated truck traffic will grow by 4.8 percent per year for the next 20 years. Although, according to NCHRP Report 299, *Fatigue Evaluation Procedures for Steel Bridges*, the maximum trucks per day in a given traffic lane is limited to a practical amount in the range of 2,000 to 3,000 trucks per day; i.e., as traffic increases vehicles will find alternate routes. For our fatigue analysis we used 2,700 trucks per day in the outside lane.

We placed strain gages on one of the link plates at both the east end and the west end of the suspended span. The link plate chosen was directly below the outside traffic lane. During our test we found 92 percent of all trucks traveled in this lane. The gage placement for the west end of the span (fixed end) is shown in Figure 18. Gage 1 was placed at the center of the link plate to read vertical strain across the nominal section of the link plate. Gage 2 was placed at the net section across the pin hole to read vertical strain, 25 mm from the pin hole. Gage 3 was placed 25 mm below the bottom of the pin hole to record horizontal strain. The gage placement on the east end of the span was similar to the west end with the following exception; Gages 2 and 3 were placed directly adjacent to the pin hole instead of one inch offset, shown in Figure 19.

Results - West End of Span 2 (Fixed End)

We monitored strain in the link plates on June 27, 1994, from 11:10 a.m. to 1:06 p.m., for a total time of 1 hour and 56 minutes. In this time 218 trucks crossed the bridge. Figures 20 through 22 show stress versus time plots for each gage producing the highest stress in the link plates. As expected, Gage 2 clearly shows the maximum stress. Its values are approximately 2.7 times Gage 1. Comparing this to the values measured in the lab test of a link plate, shown in Table 1, we see the difference here is less. This is because Gage 2 was not placed directly next to the pin, therefore its strain values do not show the maximum stress concentration.

Figure 23 shows a histogram of the stress occurrences greater than 10.3 MPa for Gage 2. We see the distribution of occurrences is similar to past studies. The maximum value was 65.5 MPa.

Table 2 shows the number of cycles for each load range and Miner's effective stress (S_r) for Gage 2. With a cutoff stress of 6.9 MPa the number of stress cycles is 419. Dividing by the number of trucks we get approximately two stress cycles per truck. The effective stress, S_r, for Gage 2 is 21.2 MPa. When doing a fatigue analysis, the effective stress at the net section of the link plate is calculated neglecting the stress concentration at the pin hole. Therefore, to compare the effective stress based on measured strain values to the effective stress based on calculated loads we needed to determine the stress in the link plate at the net section of pin hole neglecting the stress concentration. To do this, we adjusted the Gage 2 effective stress as shown in the following equation:

$$S_{r(netsection)} = S_{r(Gage2)} \frac{D_{(net/gross)}}{D_{(Gage2/Gage1)}}$$

Where:

$S_{r(netsection)}$ = estimated effective stress at the net section of the link plate, neglecting the stress concentration from the pin hole.

$S_{r(Gage2)}$ = effective stress at Gage 2.

$D_{(net/gross)}$ = Theoretical stress increase from gross section at center of link plate to net section of the link plate.

$D_{(Gage2/Gage1)}$ = stress increase from Gage 1 to Gage 2.

Using the above equation, the adjusted value is 16.7 MPa. If 2,700 trucks per day cross the bridge in a given lane, and the link plate receives two stress occurences per truck, in 75 years the link plate will experience approximately 148 million stress cycles. Figure 24 shows the adjusted value plotted on a Stress Range vs. Number of Cycles log-log graph along with the curve representing Fatigue Category E.

With this large amount of traffic, a portion of the heaviest truck population can possibly produce enough stress cycles to cause fatigue cracking. To account for this, we calculated S_r with various stress cut-off values and estimated the percentage of the heaviest truck traffic that would produce this number of stress cycles. The percentage of trucks that produces 10 million cycles and 50 million cycles was estimated to be 7 percent and 34 percent, respectively. In Tables 3 and 4, S_r is calculated for each of these values. For 10 million cycles S_r is 44.5 MPa, and for 50 million cycles S_r is 29.4 MPa. Again, since these values represent Gage 2, they were adjusted to estimate the effective stress at the net section of the link plate across the pin hole neglecting the localized stress concentration.

In Figure 24, the adjusted values are shown with the curve representing Fatigue Category E. All three points are slightly above the curve. This shows the effective stress for the link plate does not meet Fatigue Category E for a 75 year design life.

East End of Span 2 (Expansion End)

We monitored strain in the link plates on the expansion end of the span on September 25, 1994, from 11:18 a.m. to 1:10 p.m., for a total time of 1 hour and 52 minutes. In this time 246 trucks crossed the bridge. Figures 25 through 27 show stress versus time plots for each gage for the top three occurrences producing the highest stress in the link plates. Similar to the fixed end, Gage 2 clearly shows the maximum stress. The values for Gage 2 are approximately 3.8 times Gage 1. The greater difference as compared to the fixed end is because this time the gage was placed directly adjacent to the pin hole. Thus, it shows more of the stress concentration. Figure 28 shows a histogram of the stress occurrences greater than 10.3 MPa for Gage 2. The maximum stress occurrence was 58.6 MPa.

Table 5 shows the number of cycles for each load range and the effective stress for Gage 2. With a cut-off stress of 6.9 MPa the number of stress cycles is 419. Dividing by the number of trucks we get approximately two stress cycles per truck. The effective stress for Gage 2 is 17.8 MPa. Again, this value was adjusted to estimate the effective stress at the net section across the pin hole. The adjusted value is 8.8 MPa, which represents 148 million stress cycles in 75 years. Figure 29 shows the adjusted values plotted on a Stress Range vs. Number of Cycles log-log graph, along with the curve representing Fatigue Category E.

Similar to the fixed end, we calculated S_r with various stress cut-off points and estimated the percentage of the heaviest truck traffic that would produce this number of stress cycles. Again, the percentage of trucks that produces 10 million cycles and 50 million cycles was estimated to be 7 percent and 34 percent, respectively. In Tables 6 and 7, S_r is calculated for each of their values. For 10 million cycles S_r is 37.8 MPa, and for 50 million cycles S_r is 24.5 MPa. The values were adjusted to estimate the effective stress at the net section of the link plate across the pin hole. In Figure 29, the adjusted values are shown on the curve representing Fatigue Category E. All three points are below the curve for Fatigue Category E, which shows, when using these measured values, the link plate meets the fatigue category given a 75 year design life.

Fatigue Analysis and Finite Life

Using the AASHTO Fatigue Guide with the *Michigan Fatigue Truck*, we calculated the effective stress for the link plate to be 18.9 MPa. This can be compared to the measured effective stress, 16.7 MPa for the fixed end, and 8.8 MPa for the expansion end. We see the calculated effective stress is greater than both measured values, thus the calculated finite life is conservative. When using the calculated effective stress (18.9 MPa), the *mean life* for the link plate is 53 years and the *safe life* is 11 years. Since the finite life is calculated using the cube root of S_r, if S_r is doubled the finite life decreases by a factor of eight. If we use S_r equal to 16.7 MPa, as measured for the fixed end, the *mean life* of the link plate is increased to 75 years, which corresponds well to what is shown in Figure 24. When S_r equals 16.7 MPa the *safe life* becomes 15 years.

When sizing the link plates on this bridge for fatigue using the AASHTO BRIDGE SPECIFICATIONS and an HS25 truck, the existing link plates would need to be 60 percent larger.

When sizing the same link plates for fatigue using the 1994 AASHTO LRFD and the *Michigan Fatigue Truck*, the existing link plates would need to be 158 percent larger. The controlling variables resulting in the dramatically increased size is the larger truck weight, lateral distribution factor, impact factor, and the lowered allowable fatigue stress; i.e., - in the AASHTO LRFD, when the number of fatigue cycles are very large the allowable fatigue stress is reduced down to one half of the constant amplitude fatigue threshold.

M-14/US-23 over the Huron River and Conrail Railroad (Structure Number R01 of 81075)

The second bridge chosen for testing carries M-14/US-23 over Conrail Railroad and the Huron River north of Ann Arbor. The bridge has three lanes of traffic in both the north bound and south bound directions. The structure has eleven spans, six having multiple steel stringers supporting the reinforced concrete deck. The remaining five spans have only two primary support beams, which classify the bridge as non-redundant. Spans 8 and 10 are supported by link plate and pin assemblies, shown in Figure 30. Span 10, which is one of the non-redundant suspended spans was chosen for testing. It has two primary support beams, 20.1 m long, that are perpendicular to the substructure. In 1990, the link plates were replaced during painting of the bridge.

In 1995, the average daily traffic (ADT) over the bridge was 59,200 vehicles per day (two-way traffic), and the average daily truck traffic (ADTT) was 7,380 trucks per day (two-way traffic). In Figure 31 the ADT and the ADTT is plotted from 1970 to 1992. In this time the ADT increased at an average rate of 1,600 vehicles/day per year, and the ADTT increased by an average rate of 90 trucks/day/year. The planning division predicts the future ADT will increase by 730 vehicles/day/ year, and the ADTT will increase by 73 trucks/day/year.

The right most lane is a ramp lane for traffic exiting M-14/US-23. The middle lane is the normal slow traffic lane and the left most lane is the fast traffic lane. During our study we observed 8 percent of the trucks traveled in the ramp lane, 67 percent traveled in the middle lane, and 26 percent traveled in the fast lane. Given this percentage of trucks in each lane, for our analysis, we estimated 333 trucks per day in the ramp lane, 2,000 trucks per day in the slow lane, and 1,108 trucks per day in the fast lane.

Strain measurements were taken on the south west fascia beam of the south bound roadway, which puts the link plate beneath the ramp lane. The link plate was instrumented with two strain gages. Gage 1 was placed on the center of the link plate and Gage 2 was placed at the pin hole as shown in Figure 32.

Results - M-14/US-23 over the Huron River and Conrail Railroad

We monitored strain in the link plates on October 29, 1994, from 11:50 a.m. to 1:49 p.m., for a total time of 1 hour and 59 minutes. In this time, 280 trucks crossed the bridge. Unfortunately the data for Gage 1 was not amplified correctly, and therefore was lost. The data for Gage 2 was collected successfully. Figures 33 and 34 show stress versus time plots for Gage 2 for the four occurrences producing the highest stress in the link plates. We can see after the truck passes, the link plate experiences a cyclic stress at a frequency of 2.5 to 3 Hz. For large loads, the amplitude of the unloaded stress cycles can be as high as 27.6 MPa.

Figure 35 shows a histogram of the stress occurrences greater than 10.3 MPa for Gage 2. The maximum stress occurrence was 127.5 MPa.

The number of cycles for each load range is shown in Table 8, along with the resulting effective stress of 18.5 MPa for Gage 2. With a cut-off stress of 6.9 MPa the number of stress cycles is 9,023. This amounts to 32 stress cycles per truck. At 32 cycles per truck, in 75 years, the link plate experiences over 1,750 million stress cycles. For our analysis, we raised the cut-off point to 17.2 MPa. The total number of cycles reduced to 1,976 which is approximately 7 cycles per truck, which will produce approximately 380 million stress cycles in 75 years. Reviewing the stress versus time plots, and counting the number of cycles produced by a truck load, this cut-off point seems more appropriate for calculating the effective stress. As shown in Table 9, the effective stress for Gage 2 now is 33.0 MPa. Gage 2 was placed right next to the in hole, therefore its values include the maximum stress concentration from the hole. Because we did not have data from the center of the link plate (Gage 1), we could not directly compare the strain readings of Gage 2 to strain at the center of the link plate. Therefore, to adjust the Gage 2 data to represent strain at the net section of the link plate without the stress concentration, we divided the Gage 2 effective stress by 2.28 (2.28 is the estimated stress concentration factor for the geometry of this link plate). With this adjustment, the effective stress at the net section is 14.5 MPa. We raised the cut-off point one more time to 27.6 MPa and the total number of stress cycles then totals 597, which is approximately 2 cycles per truck. At 2 cycles per truck the link plate experiences approximately 110 million stress cycles in 75 years. Table 10 shows that S_r for Gage 2 becomes 45.8 MPa. Adjusting for the stress concentration we get 20.2 MPa. Figure 36 shows the adjusted S_r values plotted on a *stress range vs. number of cycles* graph, along with the curve representing Fatigue Category E. The plotted points are above the fatigue curve, therefore the measured effective stress does not meet Fatigue Category E given a 75 year design life.

Fatigue Analysis and Finite Life

Since the span had only two beams providing support, calculating the effective stress was a little more involved. Trucks traveling in the center lane distribute their load more equally to the two support beams, while trucks traveling in the outside lanes distribute more of their load to the beam closest to the lane. By analysis using the *Michigan Fatigue Truck*, the effective stress at the net section of the pin hole equals 14.5 MPa from traffic traveling in the outside lanes, while the effective stress equals 8.8 MPa for traffic in the center lane. The calculated effective stress (14.5 MPa) equaled the measured effective stress when the stress cut-off point equaled 17.2 MPa resulting in seven stress cycles per truck. Using these values we calculated the link plates have a *mean life* equal to 58 years and a *safe life* equal to 5 years.

When sizing the link plates on this bridge for fatigue using the AASHTO Bridge Specifications and an HS25 truck, the existing link plates would need to be 32 percent larger. When sizing the same link plates for fatigue using the AASHTO LRFD and the *Michigan Fatigue Truck*, the existing link plates are adequate for a design life of 75 years. Notice, when using the AASHTO Fatigue Guide the *mean life* is 58 years, but according to the AASHTO LRFD a design life of 75 years is achieved. This is because the AASHTO Fatigue Guide sets a *finite life* for any effective stresses above 11 MPa, whereas the AASHTO LRFD allows *infinite life* for effective stress below 15.5 MPa.

<u>I-75 over Toledo/Dix Bridge (Structure Number - S21 of 82191)</u>

The third and final bridge chosen for testing was I-75 over Toledo/Dix Avenue south of Detroit. It is a 10 span structure with 8 steel beams supporting a reinforced concrete deck. The bridge carries three lanes of traffic in each direction. Five of the spans are supported by link plates and pin assemblies. Span 8, which was chosen for testing, has a skew of 37 degrees to the substructure and a span length of 20.1 m. The bridge is on a 12 degree horizontal curve. This bridge, built in 1963, has its original link plates and pins, shown in Figure 37, which are corroded and most likely frozen in place.

In 1995, the structure had an ADTT of 9,600 trucks per day (two way traffic). We collected ADT and ADTT information for I-75 over the bridge from data going back to 1972. A linear regression line for the ADT (Figure 38) shows an average increase of 800 vehicle/day/year. Truck data was only available from 1983 to 1995. In that time, the ADTT remained nearly constant at about 9,600 trucks per day. During our study we determined 42 percent of the trucks traveled in the right lane (slow lane), 56 percent of the trucks traveled in the center lane, and 2 percent of the trucks traveled in the left lane (fast lane). For our study we estimated that 2,800 trucks per day travel in the middle lane.

Two link plates were instrumented with strain gages. The gages were placed as shown in Figures 39 and 40 on beams 5 and 6 of span 8. Figure 39 shows the placement of Gages 1 and 2 on beam 6. Gage 1 was placed on the center section of the link plate near the edge, and Gage 2 was placed on the link plate's net section across the pin hole. On all of the link plates, heavy corrosion existed between the beam web and the link plate, so in an attempt to measure strong axis bending, we placed Gages 3, 4, and 5 on the link plate over the diagonal gap of the beam web as shown in Figure 40.

Results - I-75 over Toledo/Dix

We monitored strain on December 19, 1994, from 1:11 p.m. to 1:34 p.m., for a total time of 23 minutes. In that time 132 trucks passed over the bridge. Figures 41 through 46 show stress versus time plots for the three live load occurrences producing the higest stress in the link plates. Because of the corrosion buildup between the link plate and beam web, we found stress at the link plate's net section is greatly reduced. Gage 2 actually showed lower stress than Gage 1, and Gage 3 clearly showed the greatest strain response. We believe the primary stress being shown in Gage 3 is from out-of-plane bending. We could see the link plate move laterally as trucks crossed the bridge, and in-plane bending would show greater stress response in Gage 5. From Gage 3, we can see the strain pattern produced by the passage of each truck is a sinusoidal wave form with the frequency of each cycle being between 3 and 4 cycles per second. Ten to fifteen stress cycles per truck passage are produced. Figure 47 shows a histogram of the stress occurrences greater than 6.9 MPa for Gage 3. The maximum stress occurrence was 72.4 MPa. For the other Strain Gages (1, 2, 4, and 5) the cyclic stress is not as evident. This is likely because the out-of-plane bending shown in Gage 3 does not occur in the other locations. Since the stress flow in the link plate has changed considerably from what is typically expected; i.e., strain across the net section of the link plate is greatly reduced, we did not estimate the effective stress from the strain data.

When sizing the link plates on this bridge for fatigue using the AASHTO Bridge Specifications and an HS25 truck, the existing link plates would need to be 28 percent larger.

When sizing the same link plates for fatigue using the AASHTO LRFD and the *Michigan Fatigue Truck*, the existing link plates need to be 150 percent larger. Again, the controlling variables resulting in the dramatically increased size is the lateral distribution factor, truck weight, impact factor, and the lowered allowable fatigue stress.

CONCLUSIONS

In this report we reviewed the condition of Michigan's bridges with link plate and pin assembles. One hundred seventy of these bridges, or 15 percent, are rated in poor condition. Link plate and pin assemblies are replaced when a bridge is scheduled for work such as painting or deck joint replacements, when the link plates are heavily corroded, or when a problem associated with the link plate and pin assembly is discovered. The decision to update the detail is usually made by a bridge scoping engineer or designer. By our estimates, approximately 20 to 30 percent of Michigan's structures with link plates have had the original link plate and pin assemblies replaced with new ones.

Michigan has 12 non-redundant bridges with link plate and pin assemblies. The bridges are given detailed inspections annually. In the mid 1980's the Department replaced the link plates on all of these bridges.

During this study, we found several problems that can occur with link plate and pin assemblies. Some of these are as follows:

1. **Link Plate Fracture.** Cracking can be caused by unintended loading such as in-plane bending. Michigan has one known fracture of a link plate. In this case, the link plate had

heavy corrosion and the beam ends were pushing against one another at the expansion joint between the link plates.

2. **Corroded Link Plates.** Since link plates are always located directly below expansion joints that eventually leak, corrosion is a common problem. Corrosion product can *build-up* between the link plate and the beam web. On the Mianus River bridge, located in Connecticut, the build-up of corrosion product slowly moved the link plate towards the end of the pin and eventually off the pin. Corrosion can also change the stress flow through the link plate. Corrosion on the bearing surface of the pin and link plate causes resistance to rotation, and corrosion build-up between the surface of the link plate and the beam web can also provide resistance to rotation of the link plate. In-plane bending was concluded to be the cause of a fractured link plate in Illinois.

3. **Beam Ends in Contact.** Bridge inspectors have found several bridges where the expansion gap between the beam end closed at the link plates. When the superstructure wanted to expand during warm weather, the beam ends pushed together causing distress in the beams and additional stress in the link plates. On one bridge we measured 2.3 MPa increased stress per degree Celsius, and we found the beam webs bulged from the compressive loads. On another bridge the beam web buckled causing the bottom flange to bend, and the beam webs to be out-of-plumb and misaligned to one another. This produced out-of-plane bending in the link plates causing one of the link plates to move and eventually disengage from the pin.

Although link plates are designed for idealized axial loads, it is known that more complex load conditions exist. Link plates provide little resistance to lateral movement of the suspended span. On one bridge, we observed the suspended spans swayed laterally causing out-of-plane bending in the link plate as trucks crossed the bridge. The resulting stress pattern created in the link plate had little damping, and produced ten to fifteen stress cycles per truck passage.

We measured the live load response of three bridges, two bridges having multiple beams; i.e,.- redundant spans, and one bridge with a non-redundant span. One of the multiple beam bridges was in good condition with new link plates. The other, was very old with corroded link plates. All three bridges had high average daily truck traffic (ADTT). We measured strain in the link plates for a very short period of time in relation to the life of the bridge, as data was taken for only a few hundred trucks of the millions of vehicles that cross the bridge in its life time. However, even this modest amount of data yielded useful and interesting information.

For the multiple beam bridge (S02 of 33085) with new link plates we measured stress in the link plates on two occasions. The measured effective stress (S_r) at the net section across the pin hole equaled 16.7 MPa (West end) on one day and 8.8 MPa (East end) on another day. The difference in values for the two days is noticeably different indicating a variation of traffic from day to day and our sample population was likely too small. However, comparing these values to the calculated effective stress S_r (18.9 MPa), we can see the measured values are less, thus indicating the *finite life* of the link plates are greater than what we calculated and our calculations are likely conservative.

The link plates on this bridge responded to load nearly as expected. We measured two stress cycles per truck from both sets of data. Using the higher measured effective stress to estimate the *finite life*, the *mean life* equals 75 years and the *safe life* equals 15 years.

For the redundant span bridge (S21 of 82191) with heavy corrosion, the stress flow in the link plates was not ideal axial loading as designed for. We found stress in the link plate at the net section across the pin hole was greatly reduced because of the corrosion, but stress along the outside edge of the link plate near the diagonal portion of the beam web was higher indicating either in-plane bending or out-of-plane bending. Since we could see the link plate move laterally each time a truck crossed over the bridge, we expect the stress pattern in this gage was produced by out-of-plane bending.

For the non-redundant bridge (R01 of 81075), the measured effective stress (S_r) at the net section across the pin hole was 14.5 MPa with seven stress cycles per truck. Using this effective stress, the link plates on this structure have a *mean life* equal to 58 years and a *safe life* equal to 5 years.

At first look, the *safe life* can be startling because of its low value. Therefore it is important to understand how to interpret this value. The *safe life provides a probability of 97.7 percent that fatigue cracks will not occur for redundant members, and a 99.9 percent probability for non-redundant members*. Dr. Fred Moses, lead author of NCHRP Report 299, *Fatigue Evaluation Procedures for Steel Bridges*[11], provides insight on how to interpret *safe life*, *The reason for giving both safe life and mean life is to show rating engineers that an inadequate safe life does not mean a crack is about to occur. On the other hand, an adequate safe life means cracking is very unlikely. Providing both values (safe life and mean life) helps to focus inspection resources.*[12]

When doing a fatigue analysis, the *mean life* should be used for determining the *finite life* of the detail. This is corroborated by AASHTO LRFD, where the equations used to design fatigue sensitive details are comparable to the steel is specified[13]. In Michigan, all link plates, on redundant as well as non-redundant spans, are specified with fracture toughness requirements.

When designing link plates for fatigue, the design code chosen will greatly affect the resulting size of the link plate. Three variables influencing the design are truck weight, the lateral distribution factor, and the impact factor, which were developed for tensile capacity design. In this study, we found our analysis using the AASHTO Fatigue Guide better reflected the measured values.

One assumption made through this entire report is the fatigue life of a link plate is Fatigue Category E, but there is minimal data to support this, and there is little or no history of fatigue cracks on typical link plate and pin bridge details. The test data from Nishimura[1] was for eyebars. Although the stress flow in an eyebar is concentrated around the pin hole similar to link plates, the difference in geometry may produce a different stress concentration.

RECOMMENDATIONS

Inspection Recommendations

Bridge inspectors and scoping engineers should be aware of the problems we have experienced with link plate and pin assemblies. The following are inspection recommendations:

Redundant Spans Supported by Link Plates

Fractured or Cracked Link Plates. Look for cracking in the link plate near the pin hole. If a crack is suspected, it should be confirmed by ultrasonic inspection. If a crack is confirmed, the beam should be supported and the fractured link plate should be replaced immediately. The link plate should be investigated to determine the cause of the fracture; i.e., - fatigue, over stress, bending, section loss, corrosion product. The remaining link plates should be inspected for cracking or signs of distress. Most likely, if one of the link plates has fractured the remaining link plates will require replacement.

Corroded Link Plates. This is the most common problem associated with link plate and pin assemblies. Heavily corroded link plates and pins should be replaced. Deck expansion joints that are leaking water onto the link plates should also be replaced.

Beam Ends in Contact. When the beam ends are in contact, the link plates and the beam ends should be inspected for distress. Types of distress that are known to occur are, the link plate is cracked or fractured, the beam web may bulge or buckle near the link plate and pin assembly, out-of-plane bending of the link plate, and general movement of the link plate and/or pin (link plate *walking* off the pin). Besides inspecting the link plates and the immediate vicinity nearby, the inspector should also look at other portions of the bridge for signs of distress that could be related to problems with the link plates. Two things to check are, tilting of the rocker bearings on adjacent piers and the beams pulling out of the abutments. These two events often accompany, or are the cause, of the closed expansion joints. This may be a sign that the expansion joint above the link plates may close in the future. If greater than 50 percent of the beam ends make contact and three or more of the beams in contact are adjacent, or if there are signs of distress to the link plates, pins, or the beams, corrective action is needed. The type of corrective action is dependant on the individual site conditions and the amount of distress evident in the link plates, pins, and beams. Some possibilities are:

> **Replace the pin and hanger assembly.** This will remove any built-up stress in the link plate. A larger link plate (50 percent greater section than normally required) should be used for replacement to resist any additional stress resulting from the beam ends making contact. Most often the link plate will be made thicker with an appropriately longer pin. **Cut relief joint in the approach pavement (if concrete).** Do this when pavement growth is suspected. **Provide fixity at the link plates and move the expansion joint elsewhere.** Many times a suspended span will have a *fixed* and *expansion* pin and hanger assembly. The

fixed end will have a splice plate connecting the top flange of the beams. This splice plate can be removed from the original fixed end and a new splice plate attached to the original expansion end. Expansion can also be moved to the abutments. Designers need to determine how this changes the expansion characteristics of the bridge and design accordingly. Both of these methods require work on the bridge deck. **Provide a gap between the beam webs that are touching (Trim the Beam Web).** When the distress from the beam ends pushing against each other is severe a gap should be provided between the beam webs that are touching. To do this, special procedures are required. Possible methods are: thermal cutting, realign the spans, or provide longer replacement link plates and adjust the deck height accordingly. Prior to doing any of these methods, precautions should be taken to prevent the spans from returning to the position where the beam ends are touching. Methods to prevent this are: cut relief joints in the approach pavement, plumb or replace rocker bearings, and provide greater restraint at the *fixed* ends.

Non-Redundant Spans Supported by Link Plates

The Maintenance Division should continue to do a detailed inspection of all of Michigan's non-redundant bridges with link plates annually. Link plates on non-redundant spans should be maintained in excellent condition. Any sign of distress to the link plates or the beams near the link plates should be addressed immediately.

Design and Analysis Recommendations

Because of the lack of data justifying the fatigue requirement and absence of fatigue damage on the many bridges that by calculation should be experiencing fatigue cracks, further research needs to be done to show that Fatigue Category E is appropriate for link plates on highway bridges. This work should be done at the University level. Until further research can justify a higher fatigue catagory, designers should detail link plates for Fatigue Category E as specified in the AASHTO Bridge Specifications or the AASHTO LRFD. When designing as per AASHTO LRFD the *Michigan Fatigue Truck* may be used to calculate live load. When analyzing existing link plates, the 1990 Fatigue Guide should be used with the *Michigan Fatigue Truck.* When designing or analyzing link plates for fatigue, if the span has multiple beams, the angle of crossing is greater than 80 degrees, and the deck is not on a horizontal curve, 2 stress cycles per truck can be used when estimating load cycles, otherwise ten cycles per truck should be used.

TABLES

	Load (kN)					
	17.8	26.7	35.6	44.5	53.4	62.3
Gage A Stress (MPa)	1.540	2.280	3.010	3.800	4.550	5.360
Gage B Stress (MPa)	1.760	2.600	3.430	4.320	5.170	6.070
Gage C Stress (MPa)	6.340	8.950	11.460	14.000	16.340	18.710
Difference Gage A to C	4.12	3.93	3.81	3.68	3.59	3.49
Difference Gage B to C	3.60	3.44	3.34	3.24	3.16	3.08

Table 1 - Tenson test of link plate showing stress concentration at the pin hole.

Equivalent Sri Range (MPa)	Midpoint Sri (MPa)	n(i)	Sri^3 (MPa)	Alpha	(Sri^3)*(alpha) (MPa)
6.89-10.34	8.62	154	640	0.368	235.28
10.34-13.78	12.07	79	1757	0.189	331.20
13.78-17.24	15.51	76	3733	0.181	677.18
17.24-20.67	18.96	38	6816	0.091	618.19
20.67-24.13	22.41	17	11251	0.041	456.50
24.13-27.56	25.86	12	17284	0.029	495.01
27.56-31.03	29.30	16	25161	0.038	960.79
31.03-34.45	32.75	9	35127	0.021	754.51
34.45-37.92	36.20	3	47428	0.007	339.58
37.92-41.34	39.64	3	62310	0.007	446.14
41.34-44.82	43.09	3	80020	0.007	572.93
44.82-48.23	46.54	0	100802	0.000	0.00
48.23-51.68	49.99	2	124902	0.005	596.19
51.68-55.12	53.43	3	152567	0.007	1092.37
55.12-58.57	56.88	2	184043	0.005	878.49
58.57-62.01	60.33	1	219574	0.002	524.04
62.01-65.46	63.78	1	259407	0.002	619.11
65.46-68.9	67.22	0	303788	0.000	0.00
Summation		419	1636611.7	1	9597.52
Constant Amplitude Effective Stress		*Sr miner*		21.25 MPa	

Table 2.
Constant Amplitude Equivalent Stress.
Equivalent to 148 million stress cycles (in 75 years).
I-96 over M-52 South of Webberville.
Gage 2, Fixed End.

Equivalent Sri Range (MPa)	Midpoint Sri (MPa)	n(i)	Sri^3 (MPa)	Alpha	(Sri^3)*(alpha) (MPa)
27.56-31.03	29.30	1	25161	0.036	898.60
31.03-34.45	32.75	9	35127	0.321	11290.75
34.45-37.92	36.20	3	47428	0.107	5081.58
37.92-41.34	39.64	3	62310	0.107	6676.12
41.34-44.82	43.09	3	80020	0.107	8573.55
44.82-48.23	46.54	0	100802	0.000	0.00
48.23-51.68	49.99	2	124903	0.071	8921.61
51.68-55.12	53.43	3	152568	0.107	16346.54
55.12-58.57	56.88	2	184043	0.071	13145.93
58.57-62.01	60.33	1	219574	0.036	7841.94
62.01-65.46	63.78	1	259408	0.036	9264.56
65.46-68.9	67.22	0	303788	0.000	0.00
Summation		28	1595132	1	88041.18
Constant Amplitude Effective Stress		***Sr miner***		44.49 MPa	

Table 3.
Constant Amplitude Equivalent Stress.
Based on 10 million stress cycles.
I-96 over M-52 South of Webberville.
Gage 2, Fixed End.

Equivalent Sri (MPa)	Midpoint Sri (MPa)	n(i)	Sri^3 (MPa)	Alpha	(Sri^3)*(alpha) (MPa)
13.79-17.24	15.5132	32	3733	0.225	841.33
17.24-20.68	18.96058	38	6816	0.268	1824.11
20.68-24.13	22.40796	17	11251	0.120	1347.00
24.13-27.58	25.85534	12	17284	0.085	1460.64
27.58-31.03	29.30272	16	25161	0.113	2835.01
31.03-34.47	32.7501	9	35127	0.063	2226.34
34.47-37.92	36.19747	3	47428	0.021	1002.00
37.92-41.37	39.64485	3	62310	0.021	1316.42
41.37-44.82	43.09223	3	80020	0.021	1690.56
44.82-48.26	46.53961	0	100802	0.000	0.00
48.26-51.71	49.98699	2	124902	0.014	1759.19
51.71-55.16	53.43437	3	152567	0.021	3223.26
55.16-58.61	56.88175	2	184043	0.014	2592.15
58.61-62.05	60.32912	1	219574	0.007	1546.30
62.05-65.50	63.7765	1	259407	0.007	1826.81
65.50-68.95	67.22388	0	303788	0.000	0.00
Summation		142	1634215	1.000	25491.11
Constant Amplitude Effective Stress					
		Sr miner		29.43 MPa	

Table 4.
Constant Amplitude Equivalent Stress.
Based on 50 million stress cycles.
I-96 over M-52 South of Webberville.
Gage 2, Fixed End.

Equivalent Sri Range (MPa)	Midpoint Sri (MPa)	n(i)	Sri^3 (MPa)	Alpha	(Sri^3)*(alpha) (MPa)
6.89-10.34	8.62	244	641	0.444	284.15
10.34-13.79	12.07	134	1758	0.244	428.41
13.79-17.24	15.51	88	3733	0.160	597.34
17.24-20.68	18.96	34	6816	0.062	421.38
20.68-24.13	22.41	17	11251	0.031	347.77
24.13-27.58	25.86	12	17284	0.022	377.11
27.58-31.03	29.30	3	25161	0.005	137.24
31.03-34.47	32.75	4	35127	0.007	255.47
34.47-37.92	36.20	0	47428	0.000	0.00
37.92-41.37	39.64	3	62310	0.005	339.87
41.37-44.82	43.09	2	80020	0.004	290.98
44.82-48.26	46.54	3	100802	0.005	549.83
48.26-51.71	49.99	2	124902	0.004	454.19
51.71-55.16	53.43	3	152567	0.005	832.19
55.16-58.61	56.88	1	184043	0.002	334.62
58.61-62.05	60.33	0	219574	0.000	0.00
62.05-65.50	63.78	0	259407	0.000	0.00
65.50-68.95	67.22	0	303788	0.000	0.00
Summation		550	0	1.000	5650.56
Constant Amplitude Effective Stress					
		Sr miner		17.81 MPa	

Table 5.
Constant Amplitude Equivalent Stress.
Equivalent to 148 million stress cycles (in 75 years).
I-96 over M-52 South of Webberville.
Gage 2, Expansion End.

Equivalent Sri Range (MPa)	Midpoint Sri (MPa)	n(i)	Sri^3 (MPa)	Alpha	(Sri^3)*(alpha) (MPa)
20.68-24.13	22.41	4	11251	0.108	1216.37
24.13-27.58	25.86	12	17284	0.324	5605.70
27.58-31.03	29.30	3	25161	0.081	2040.06
31.03-34.47	32.75	4	35127	0.108	3797.48
34.47-37.92	36.20	0	47428	0.000	0.00
37.92-41.37	39.64	3	62310	0.081	5052.19
41.37-44.82	43.09	2	80020	0.054	4325.39
44.82-48.26	46.54	3	100802	0.081	8173.12
48.26-51.71	49.99	2	124902	0.054	6751.48
51.71-55.16	53.43	3	152567	0.081	12370.34
55.16-58.61	56.88	1	184043	0.027	4974.13
58.61-62.05	60.33	0	219574	0.000	0.00
62.05-65.50	63.78	0	259407	0.000	0.00
65.50-68.95	67.22	0	303788	0.000	0.00
Summation		37	0	1.000	54306.27
Constant Amplitude Effective Stress		***Sr miner***		37.87 MPa	

Table 6.
Constant Amplitude Equivalent Stress.
Based on 10 million stress cycles.
I-96 over M-52 South of Webberville.
Gage 2, Expansion End.

Equivalent Sri Range (MPa)	Midpoint Sri (MPa)	n(i)	Sri^3 (MPa)	Alpha	(Sri^3)*(alpha) (MPa)
10.34-13.79	12.07	13	1758	0.070	123.56
13.79-17.24	15.51	88	3733	0.476	1775.89
17.24-20.68	18.96	34	6816	0.184	1252.74
20.68-24.13	22.41	17	11251	0.092	1033.91
24.13-27.58	25.86	12	17284	0.065	1121.14
27.58-31.03	29.30	3	25161	0.016	408.01
31.03-34.47	32.75	4	35127	0.022	759.50
34.47-37.92	36.20	0	47428	0.000	0.00
37.92-41.37	39.64	3	62310	0.016	1010.44
41.37-44.82	43.09	2	80020	0.011	865.08
44.82-48.26	46.54	3	100802	0.016	1634.62
48.26-51.71	49.99	2	124902	0.011	1350.30
51.71-55.16	53.43	3	152567	0.016	2474.07
55.16-58.61	56.88	1	184043	0.005	994.83
58.61-62.05	60.33	0	219574	0.000	0.00
62.05-65.50	63.78	0	259407	0.000	0.00
65.50-68.95	67.22	0	303788	0.000	0.00
Summation		185	0	1.000	14804.09
Constant Amplitude Effective Stress					
		Sr miner		24.55 MPa	

Table 7.
Constant Amplitude Equivalent Stress.
Based on 50 million stress cycles.
I-96 over M-52 South of Webberville.
Gage 2, Expansion End.

Equivalent Sri Range (MPa)	Midpoint Sri (MPa)	n(i)	Sri^3 (MPa)	Alpha	(Sri^3)*(alpha) (MPa)
6.89-10.34	8.62	4011	641	0.445	284.72
10.34-13.79	12.07	1878	1758	0.208	365.99
13.79-17.24	15.51	1120	3733	0.124	463.42
17.24-20.68	18.96	750	6816	0.083	566.59
20.68-24.13	22.41	428	11251	0.047	533.70
24.13-27.58	25.86	260	17284	0.029	498.05
27.58-31.03	29.30	193	25161	0.021	538.18
31.03-34.47	32.75	114	35127	0.013	443.80
34.47-37.92	36.20	76	47428	0.008	399.48
37.92-41.37	39.64	48	62310	0.005	331.47
41.37-44.82	43.09	36	80020	0.004	319.26
44.82-48.26	46.54	27	100802	0.003	301.63
48.26-51.71	49.99	23	124902	0.003	318.38
51.71-55.16	53.43	12	152567	0.001	202.90
55.16-58.61	56.88	12	184043	0.001	244.76
58.61-62.05	60.33	8	219574	0.001	194.68
62.05-65.50	63.78	7	259407	0.001	201.25
65.50-68.95	67.22	6	303788	0.001	202.01
68.95-72.39	70.67	6	352963	0.001	234.71
72.39-75.84	74.12	8	407177	0.001	361.01
75.84-79.29	77.57	4	466676	0.000	206.88
79.29-82.74	81.01	4	531705	0.000	235.71
82.74-86.18	84.46	1	602512	0.000	66.78
86.18-89.63	87.91	1	679341	0.000	75.29
89.63-93.08	91.36	1	762439	0.000	84.50
93.08-96.53	94.80	2	852051	0.000	188.86
96.53-99.97	98.25	1	948423	0.000	105.11
99.97-103.42	101.70	1	1051801	0.000	116.57
103.42-106.87	105.15	2	1162430	0.000	257.66
106.87-110.32	108.59	1	1280558	0.000	141.92
110.32-113.76	112.04	1	1406428	0.000	155.87
113.76-117.21	115.49	0	1540288	0.000	0.00
117.21-120.66	118.93	1	1682383	0.000	186.45
120.66-124.11	122.38	0	1832958	0.000	0.00
124.11-127.55	125.83	0	1992260	0.000	0.00
127.55-131.00	129.28	1	2160535	0.000	239.45
Summation		9023	1636614	1	6410.30
Constant Amplitude Effective Stress		***Sr miner***		18.58 MPa	

Table 8.
Constant Amplitude Equivalent Stress.
Equivalent to 1,750 million cycles (in 75 years).
M-14/US-23 over Conrail/Huron River.
Gage 2.

Equivalent Sri Range (MPa)	Midpoint Sri (MPa)	n(i)	Sri^3 (MPa)	Alpha	(Sri^3)*(alpha) (MPa)
17.24-20.68	18.96	691	6816	0.350	2383.67
20.68-24.13	22.41	428	11251	0.217	2437.05
24.13-27.58	25.86	260	17284	0.132	2274.24
27.58-31.03	29.30	193	25161	0.098	2457.50
31.03-34.47	32.75	114	35127	0.058	2026.54
34.47-37.92	36.20	76	47428	0.038	1824.15
37.92-41.37	39.64	48	62310	0.024	1513.61
41.37-44.82	43.09	36	80020	0.018	1457.85
44.82-48.26	46.54	27	100802	0.014	1377.35
48.26-51.71	49.99	23	124902	0.012	1453.82
51.71-55.16	53.43	12	152567	0.006	926.52
55.16-58.61	56.88	12	184043	0.006	1117.67
58.61-62.05	60.33	8	219574	0.004	888.96
62.05-65.50	63.78	7	259407	0.004	918.95
65.50-68.95	67.22	6	303788	0.003	922.43
68.95-72.39	70.67	6	352963	0.003	1071.75
72.39-75.84	74.12	8	407177	0.004	1648.49
75.84-79.29	77.57	4	466676	0.002	944.69
79.29-82.74	81.01	4	531705	0.002	1076.33
82.74-86.18	84.46	1	602512	0.001	304.91
86.18-89.63	87.91	1	679341	0.001	343.80
89.63-93.08	91.36	1	762439	0.001	385.85
93.08-96.53	94.80	2	852051	0.001	862.40
96.53-99.97	98.25	1	948423	0.001	479.97
99.97-103.42	101.70	1	1051801	0.001	532.29
103.42-106.87	105.15	2	1162430	0.001	1176.55
106.87-110.32	108.59	1	1280558	0.001	648.06
110.32-113.76	112.04	1	1406428	0.001	711.76
113.76-117.21	115.49	0	1540288	0.000	0.00
117.21-120.66	118.93	1	1682383	0.001	851.41
120.66-124.11	122.38	0	1832958	0.000	0.00
124.11-127.55	125.83	0	1992260	0.000	0.00
127.55-131.00	129.28	1	2160535	0.001	1093.39
Summation		1976	21343409	1	36111.97
Constant Amplitude Effective Stress		*Sr miner*		33.05 MPa	

Table 9.
Constant Amplitude Equivalent Stress.
Equivalent to 380 million stress cycles (in 75 years).
M-14/US-23 over Conrail/Huron River.
Gage 2.

Equivalent Sri Range (MPa)	Midpoint Sri (MPa)	n(i)	Sri^3 (MPa)	Alpha	(Sri^3)*(alpha) (MPa)
27.58-31.03	29.30	193	25161	0.323	8134.05
31.03-34.47	32.75	114	35127	0.191	6707.62
34.47-37.92	36.20	76	47428	0.127	6037.74
37.92-41.37	39.64	48	62310	0.080	5009.88
41.37-44.82	43.09	36	80020	0.060	4825.31
44.82-48.26	46.54	27	100802	0.045	4558.87
48.26-51.71	49.99	23	124902	0.039	4811.99
51.71-55.16	53.43	12	152567	0.020	3066.68
55.16-58.61	56.88	12	184043	0.020	3699.35
58.61-62.05	60.33	8	219574	0.013	2942.37
62.05-65.50	63.78	7	259407	0.012	3041.63
65.50-68.95	67.22	6	303788	0.010	3053.15
68.95-72.39	70.67	6	352963	0.010	3547.37
72.39-75.84	74.12	8	407177	0.013	5456.30
75.84-79.29	77.57	4	466676	0.007	3126.80
79.29-82.74	81.01	4	531705	0.007	3562.52
82.74-86.18	84.46	1	602512	0.002	1009.23
86.18-89.63	87.91	1	679341	0.002	1137.93
89.63-93.08	91.36	1	762439	0.002	1277.12
93.08-96.53	94.80	2	852051	0.003	2854.44
96.53-99.97	98.25	1	948423	0.002	1588.65
99.97-103.42	101.70	1	1051801	0.002	1761.81
103.42-106.87	105.15	2	1162430	0.003	3894.24
106.87-110.32	108.59	1	1280558	0.002	2144.99
110.32-113.76	112.04	1	1406428	0.002	2355.83
113.76-117.21	115.49	0	1540288	0.000	0.00
117.21-120.66	118.93	1	1682383	0.002	2818.06
120.66-124.11	122.38	0	1832958	0.000	0.00
124.11-127.55	125.83	0	1992260	0.000	0.00
127.55-131.00	129.28	1	2160535	0.002	3618.99
Summation		597	21308057	1	96042.89
Constant Amplitude Effective Stress		***Sr miner***		45.80 MPa	

Table 10.
Constant Amplitude Equivalent Stress.
Equivalent to 110 million stress cycles (in 75 years).
M-14/US-23 over Conrail/Huron River.
Gage 2.

FIGURES

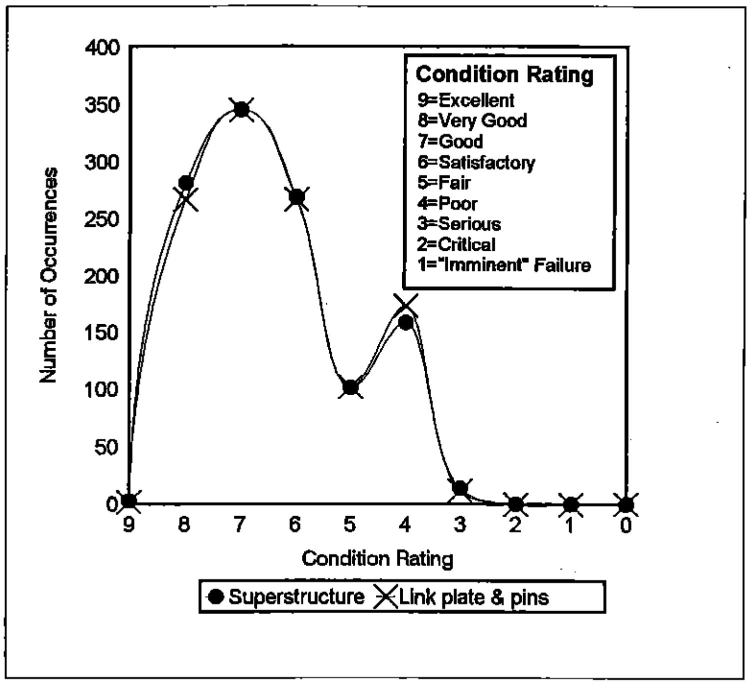


Figure 1 - Condition Rating of Michigan's Bridges with link plates.

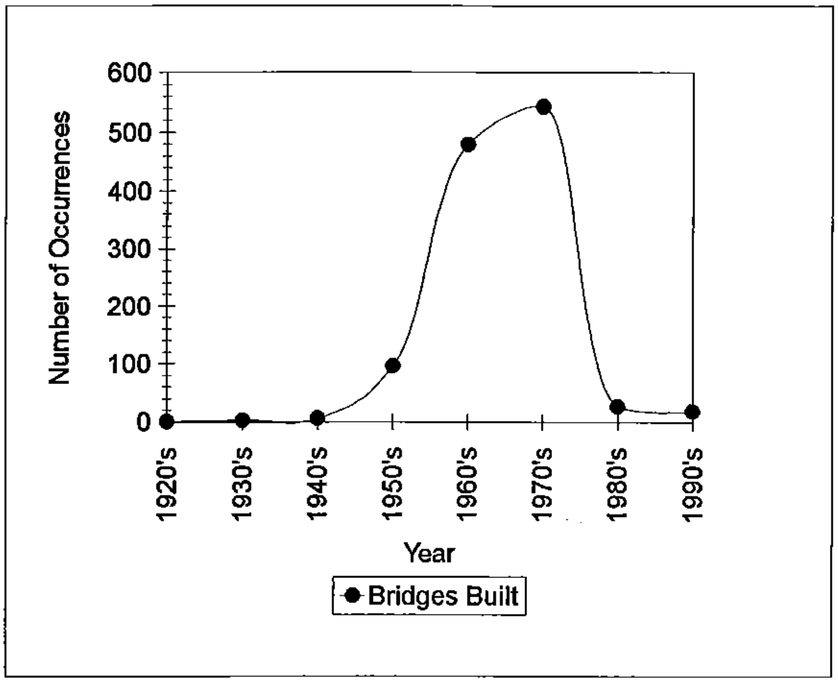


Figure 2 - Michigan's Link Plate Bridges

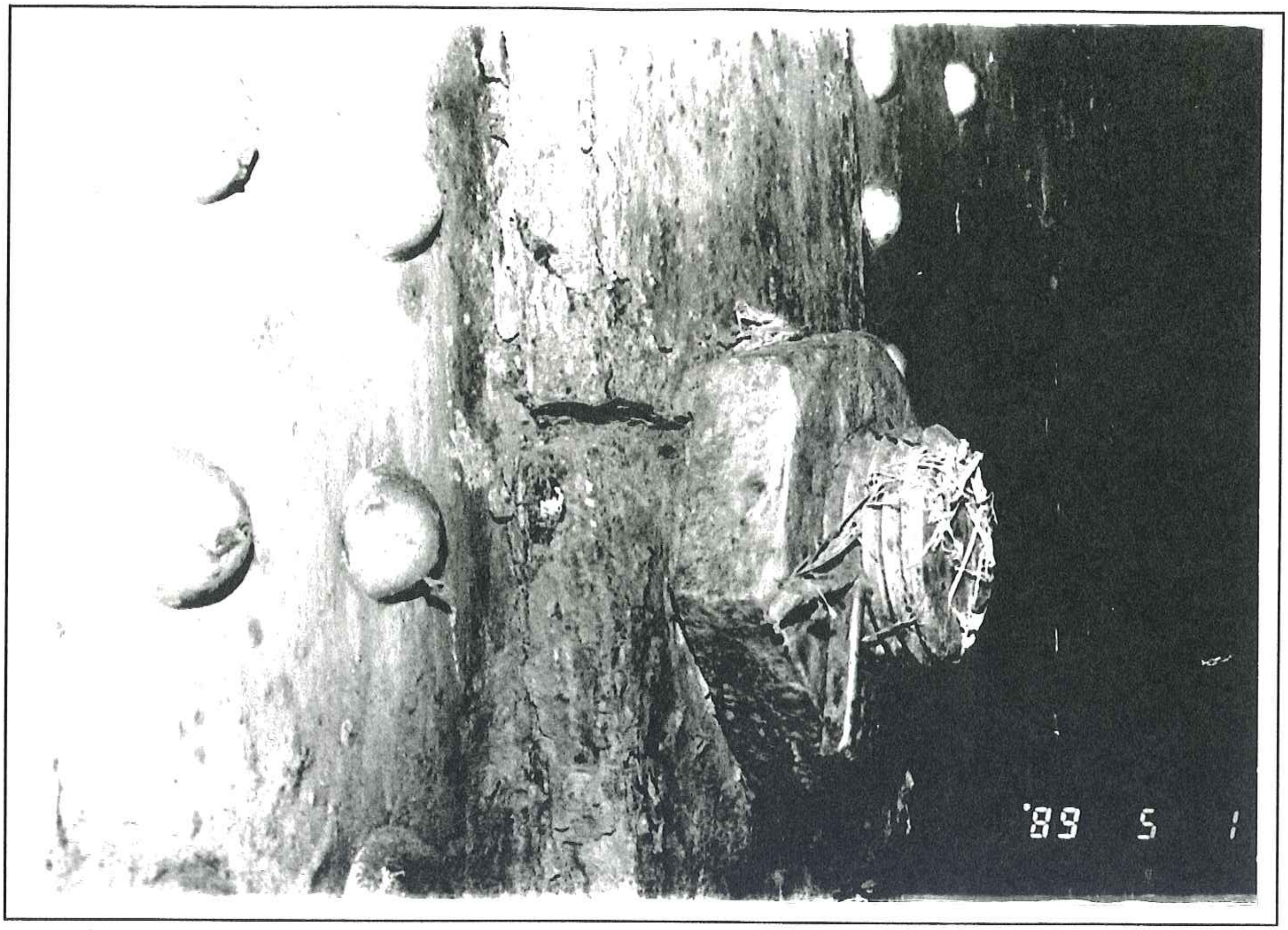


FIGURE 3
M-46 over Tittabawassee river (Structure Number B02 of 73062) Showing 1/4 in. wide crack in link plate at bottom pin. Span 3 east bound at center longitudinal joint.

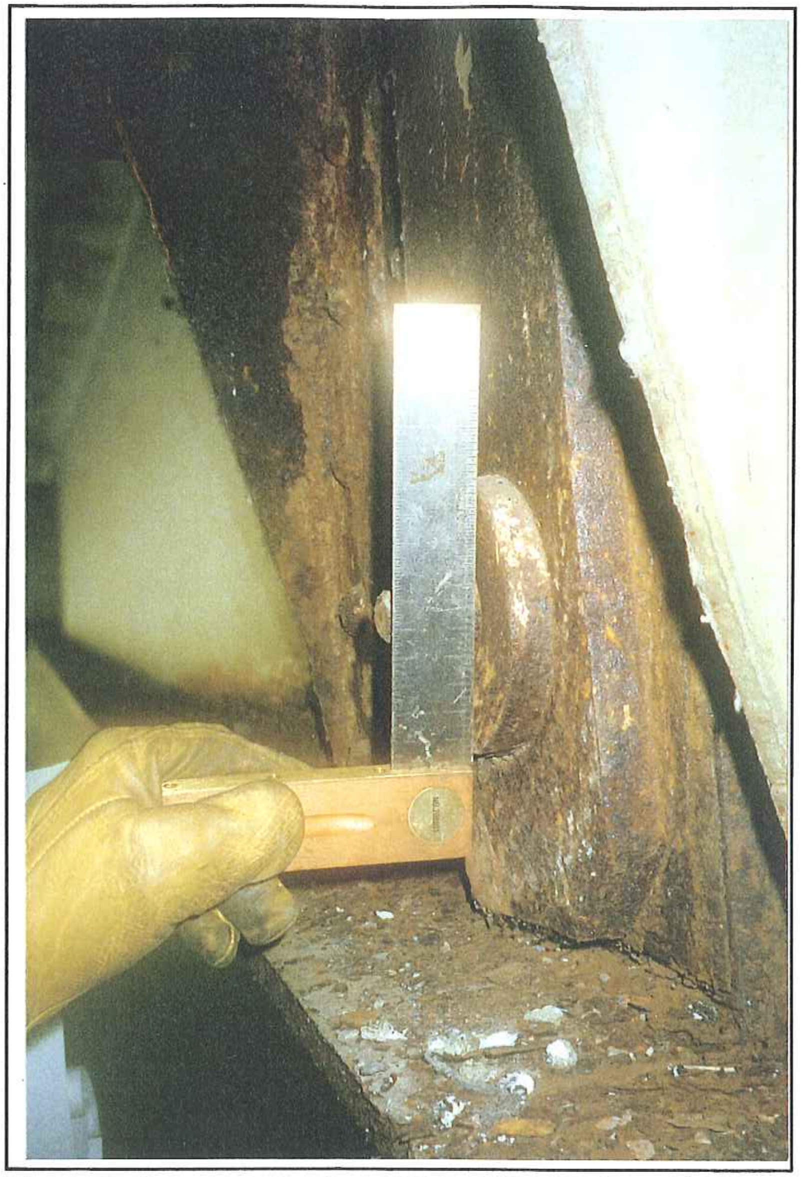


Figure 4. Structure Number - B04 of 58152. Corrosion build-up between the link plate and beam web is causing the link plate to move to the end of the pin, resulting in "cupping" of the pin cover plate.

FIGURE 5 - Expansion joint has closed causing the beam ends to push against each other. (Structure Number B02 of 41027).

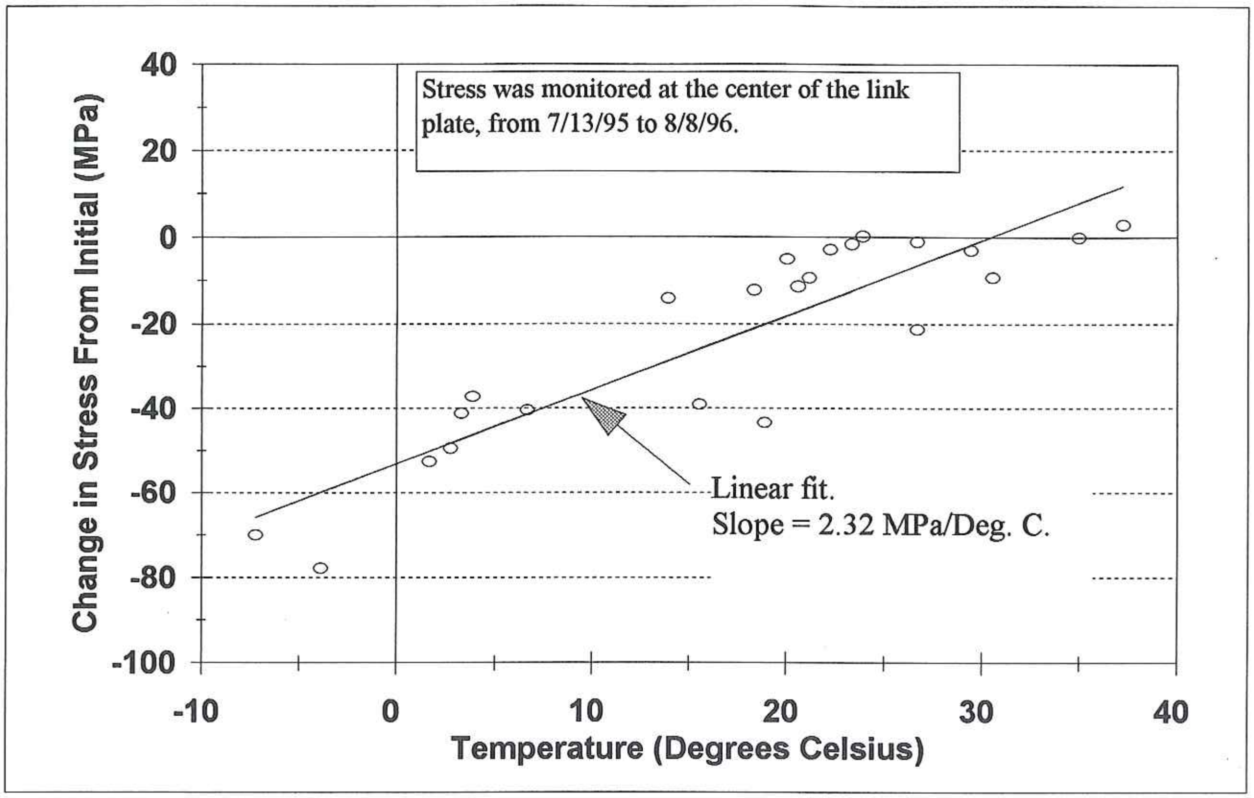


Figure 6 - Stress change vs. Temperature.
Beam A. Span 7. Structure B02 of 41027
I-196 over the Grand River, Grand Rapids.

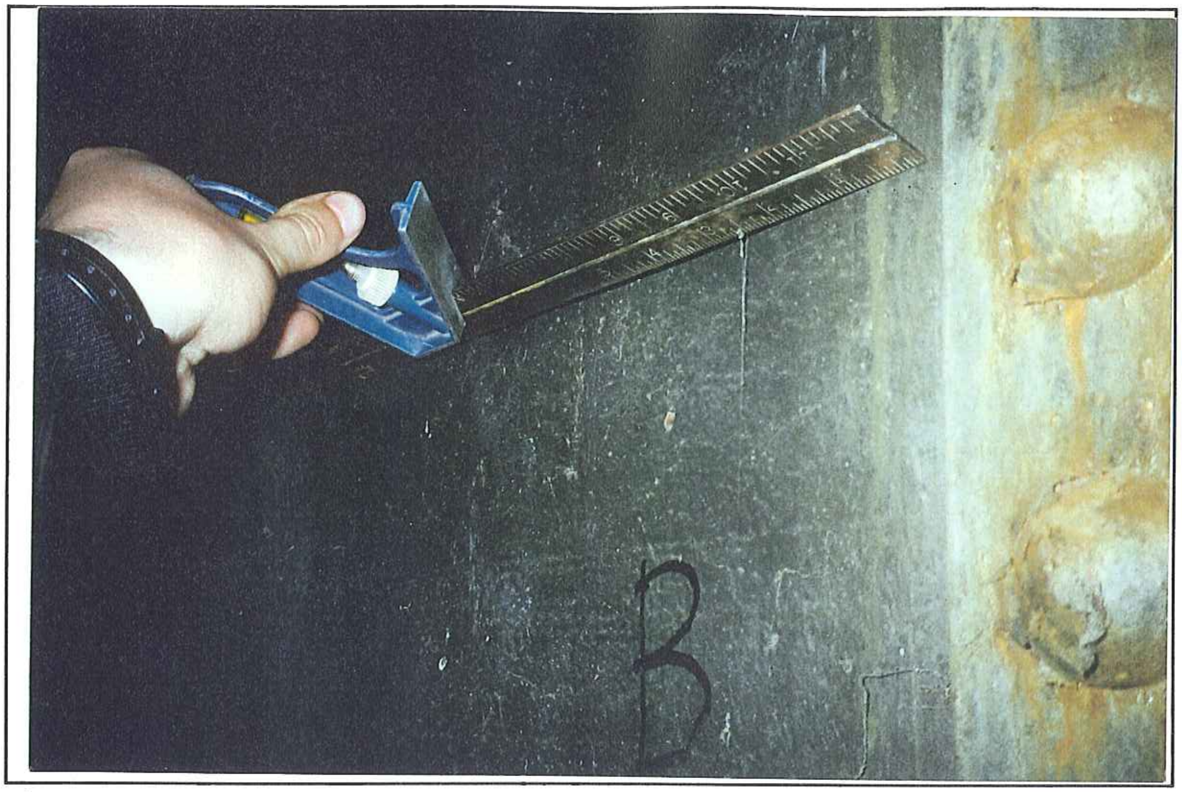


Figure 7. Concave side of bulged beam web resulting from beam ens pushing against each other. (Structure Number - B02 of 41027)

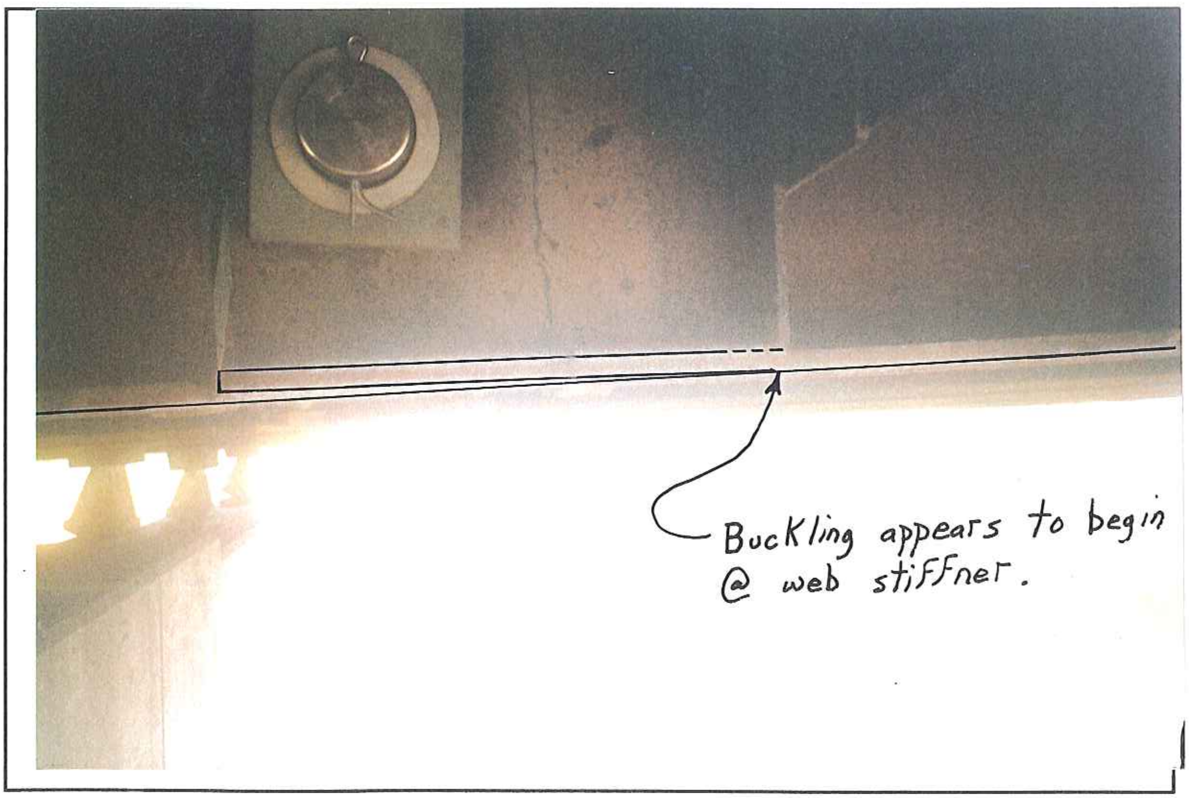


Figure 8. Buckling of the suspended span beam web. (Structure Number S15 of 82292)

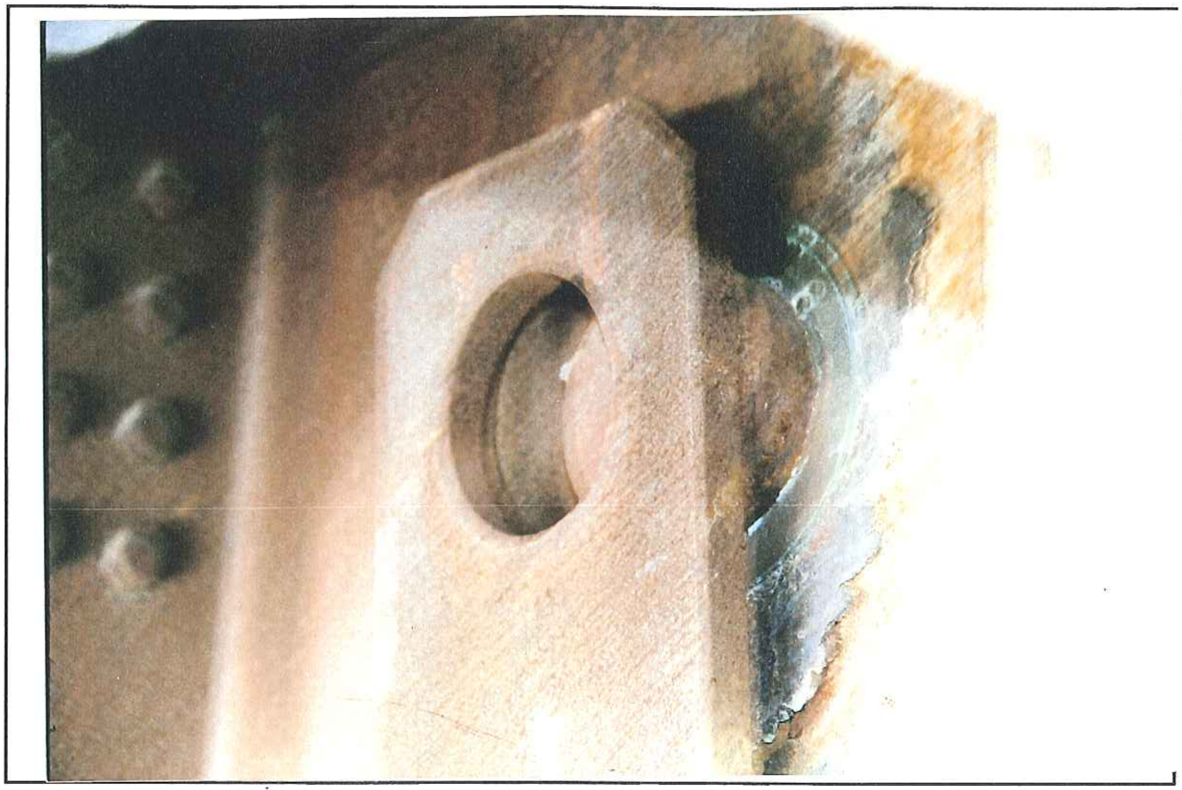


Figure 9. Link plate disengaged from pin. (Structure Number S15 of 82292)

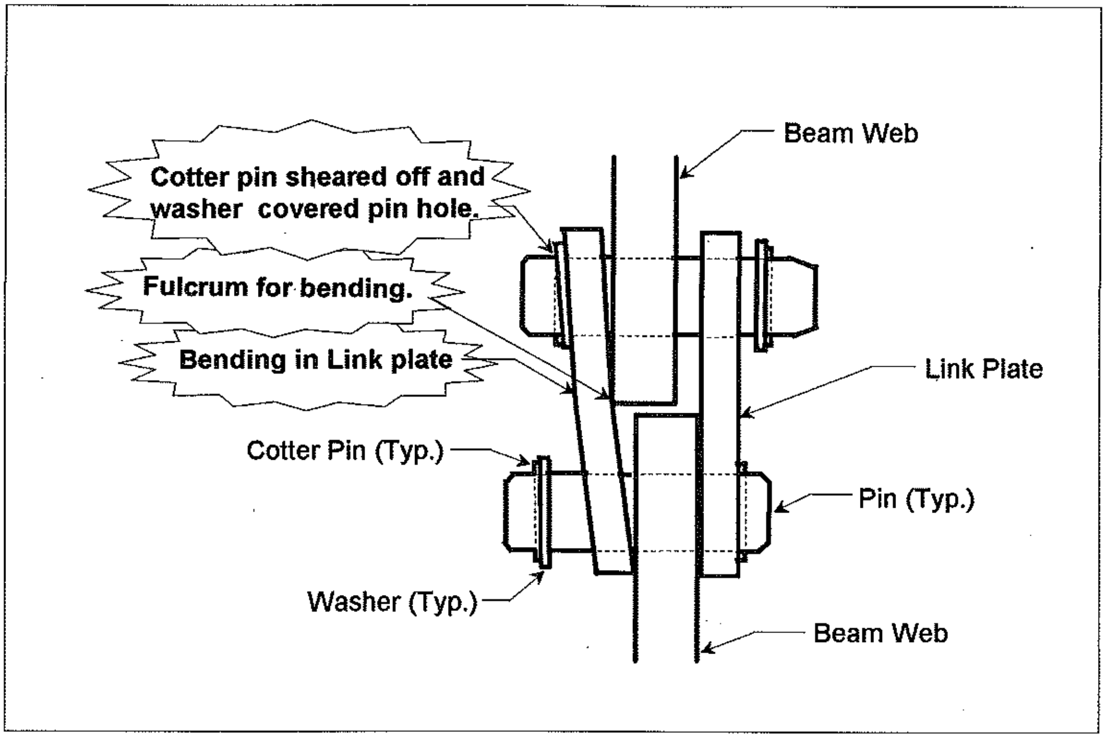


Figure 10 - Offset beam webs cause out-of-plane bending in the Link Plate.

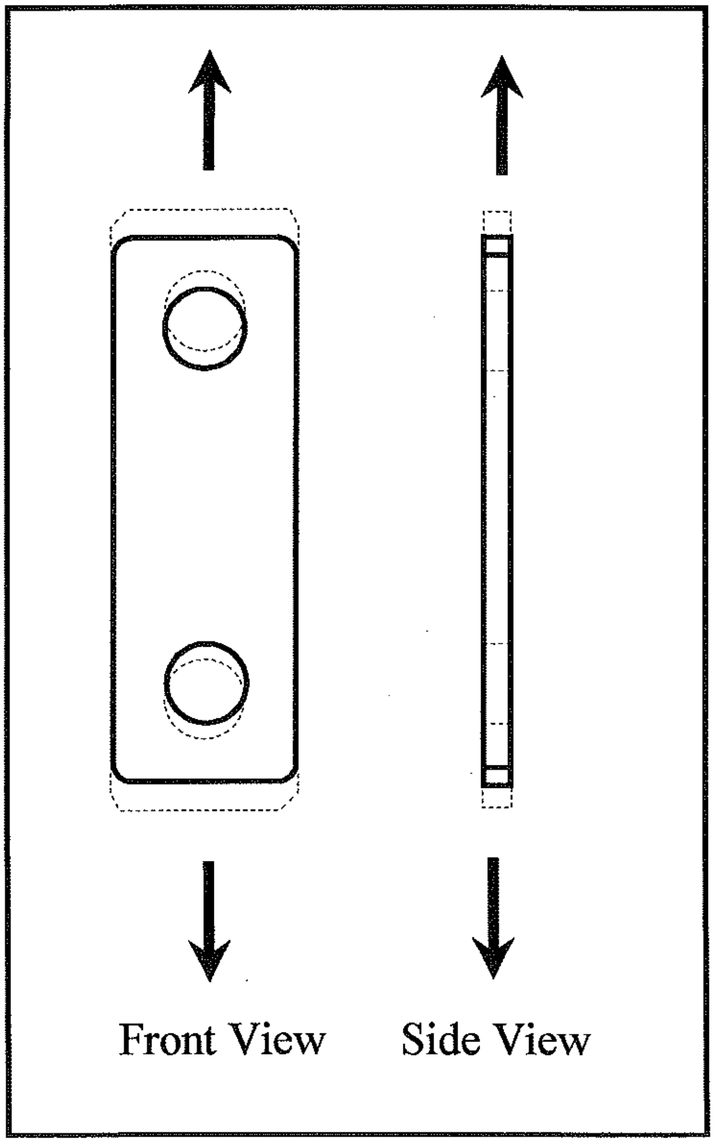


Figure 11 - Axial loading of link plate.

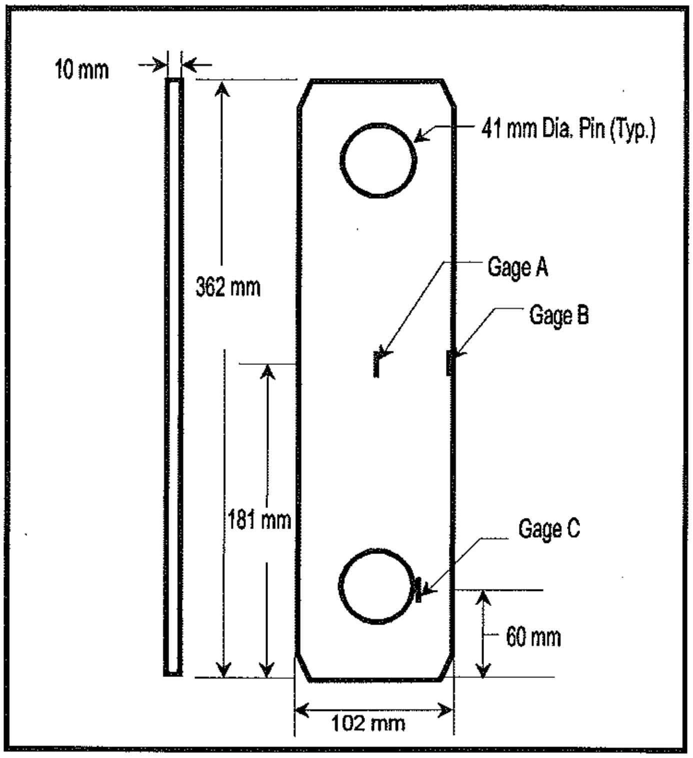


Figure 12 - Reduced Size Link Plate for Lab Test to Determine Stress Concentration at Pin Hole.

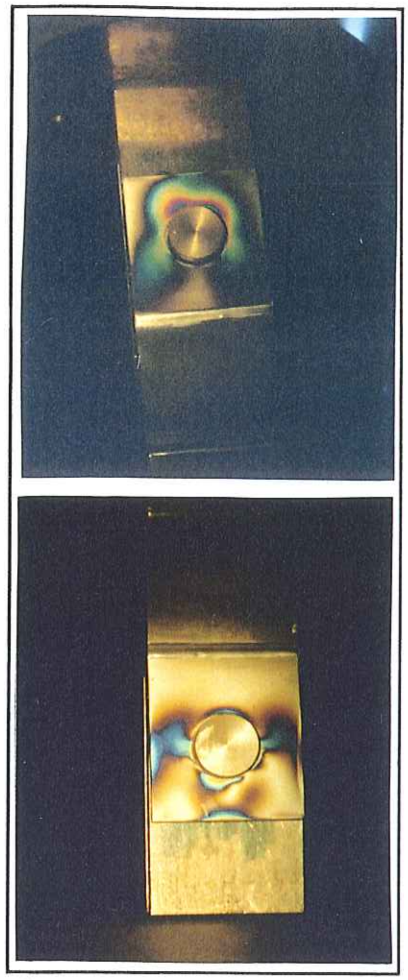


Figure 13.
Stress concentrations seen
with with Photolastic coating.

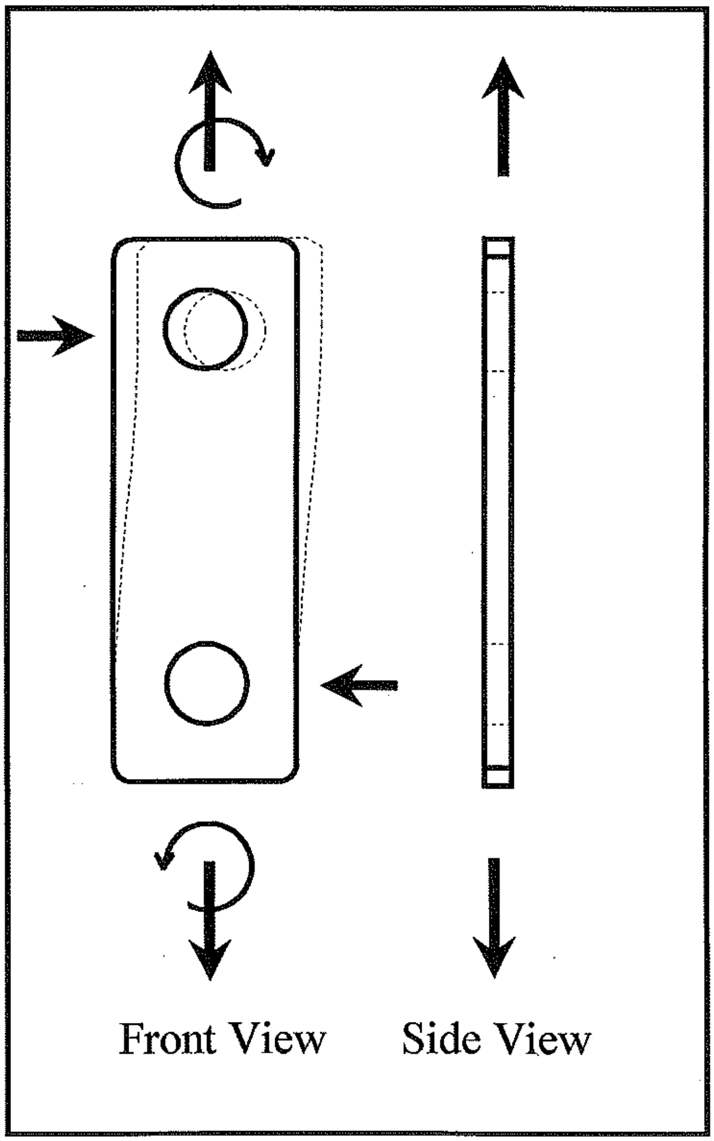


Figure 14 - In-plane bending of link plate.

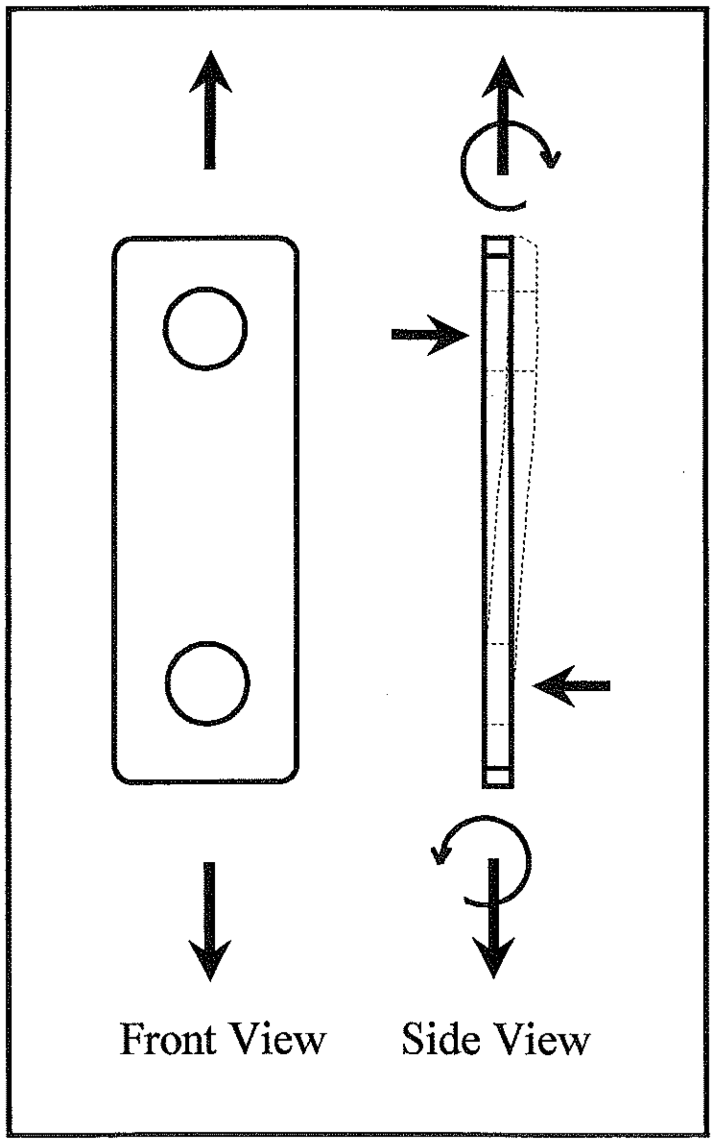


Figure 15 - Out-of-plane bending of link plate.

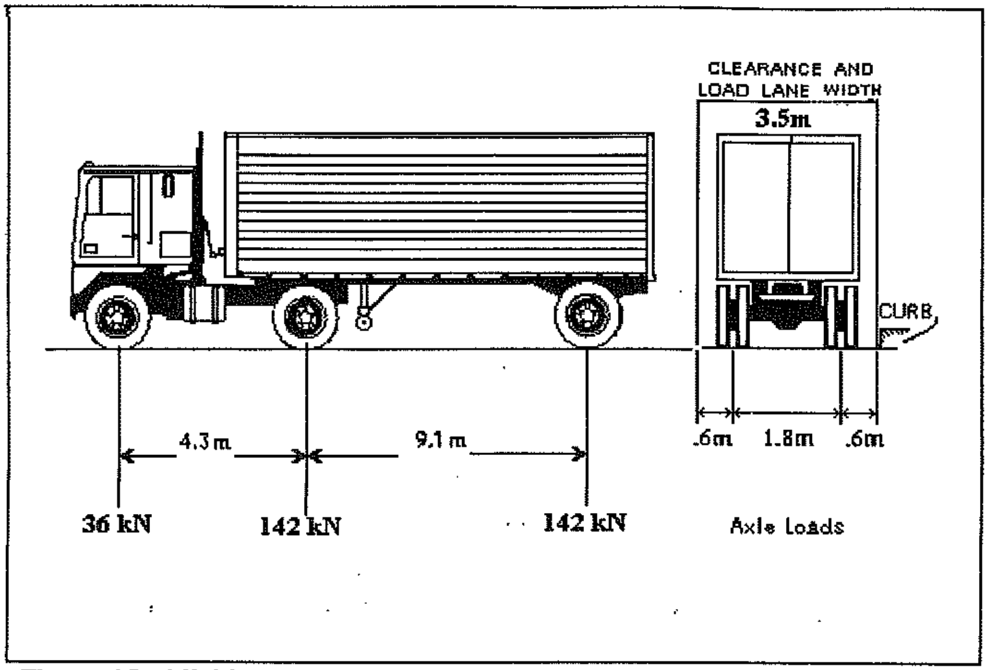


Figure 16 - Michigan Fatigue Truck

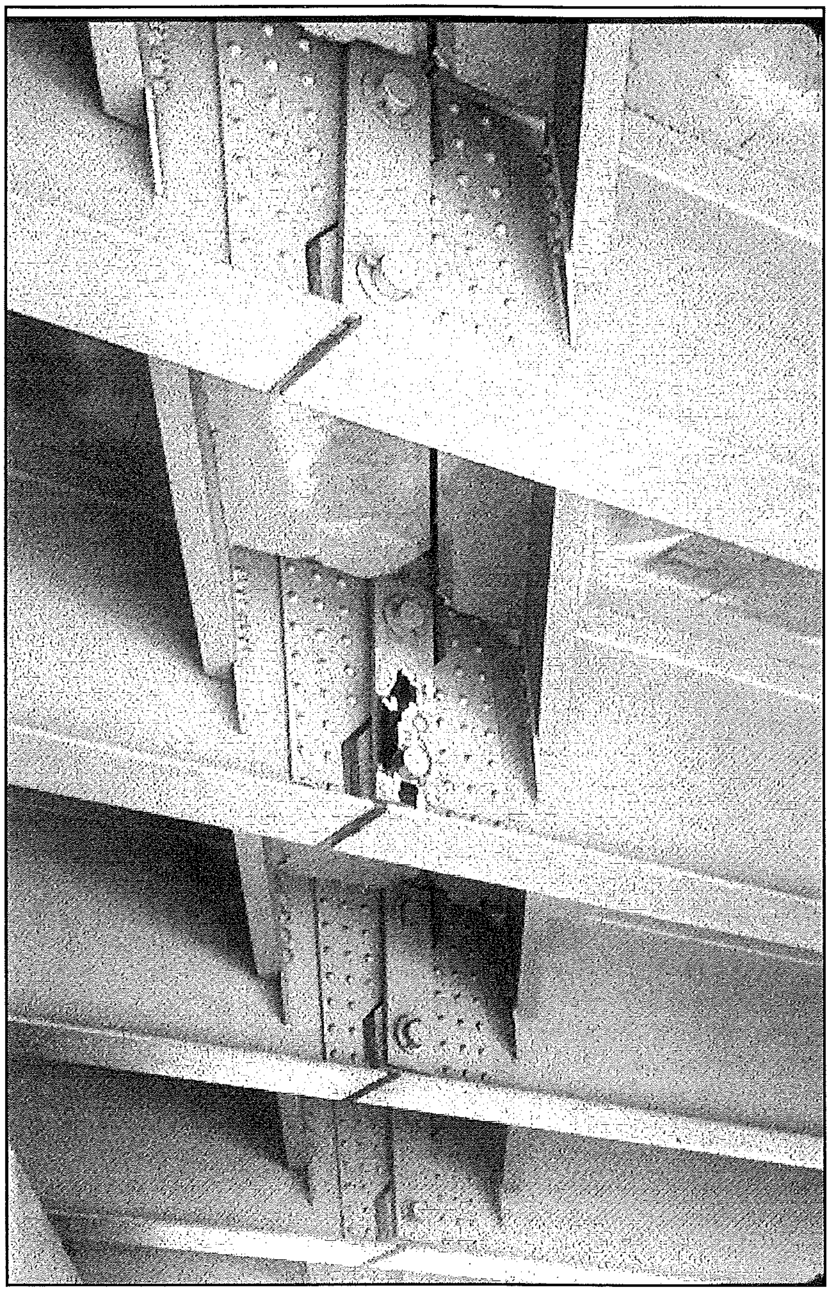


Figure 17 - Link Plates. I-96 over M-52. South of Webberville.

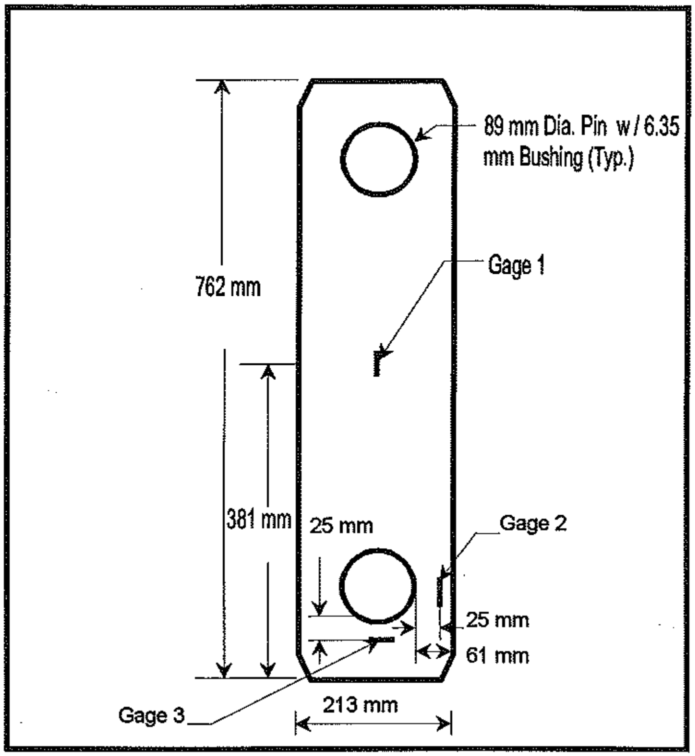


Figure 18. Span 2, Pier 1, Beam C. Fixed End Link Plate with Strain Gages.

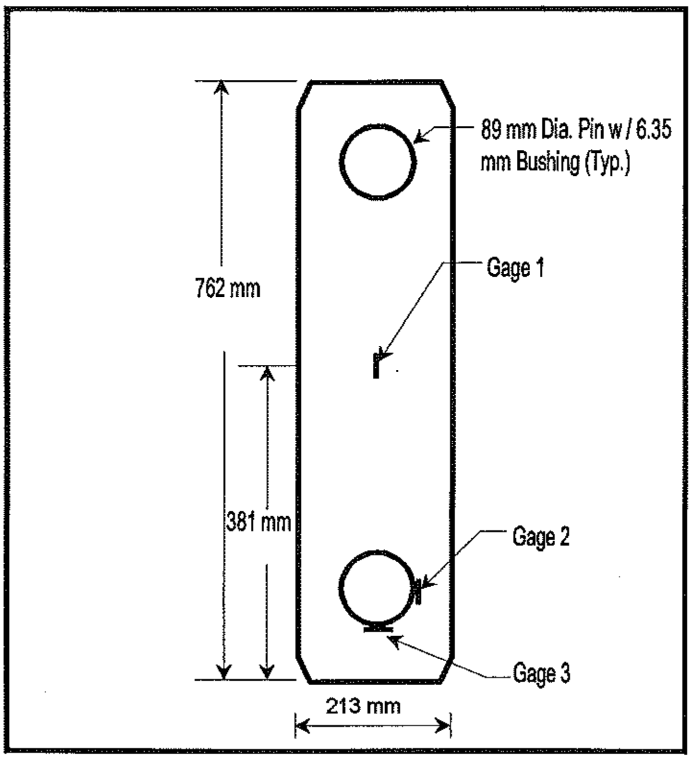


Figure 19. Span 2, Pier 2, Beam C.
Expansion End Link Plate Showing Strain Gages.

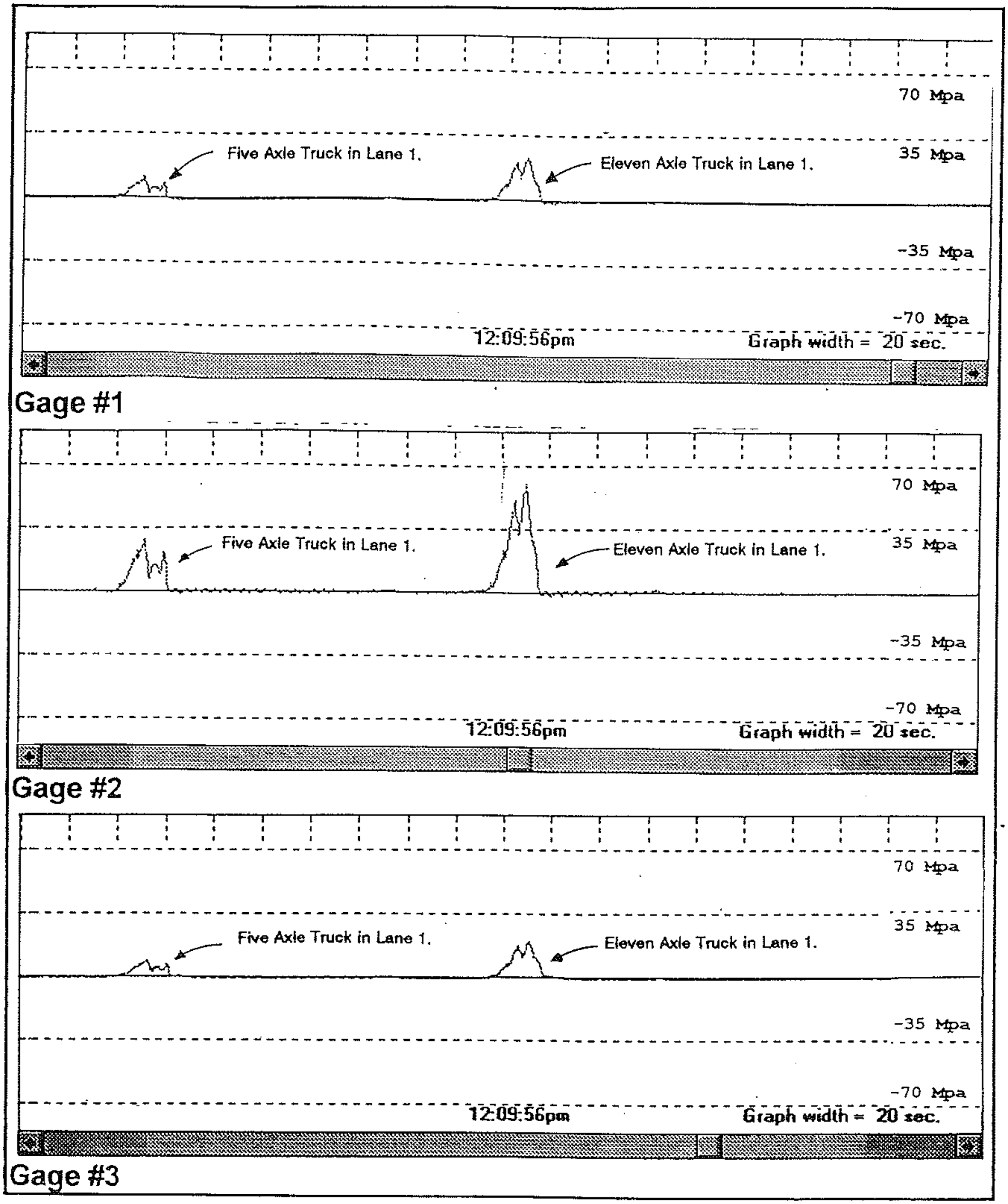


Figure 20 - Stress vs. Time Plot.
Structure Number S02 of 33085.
I-96 over M-52, South of Webberville.
Expansion End.

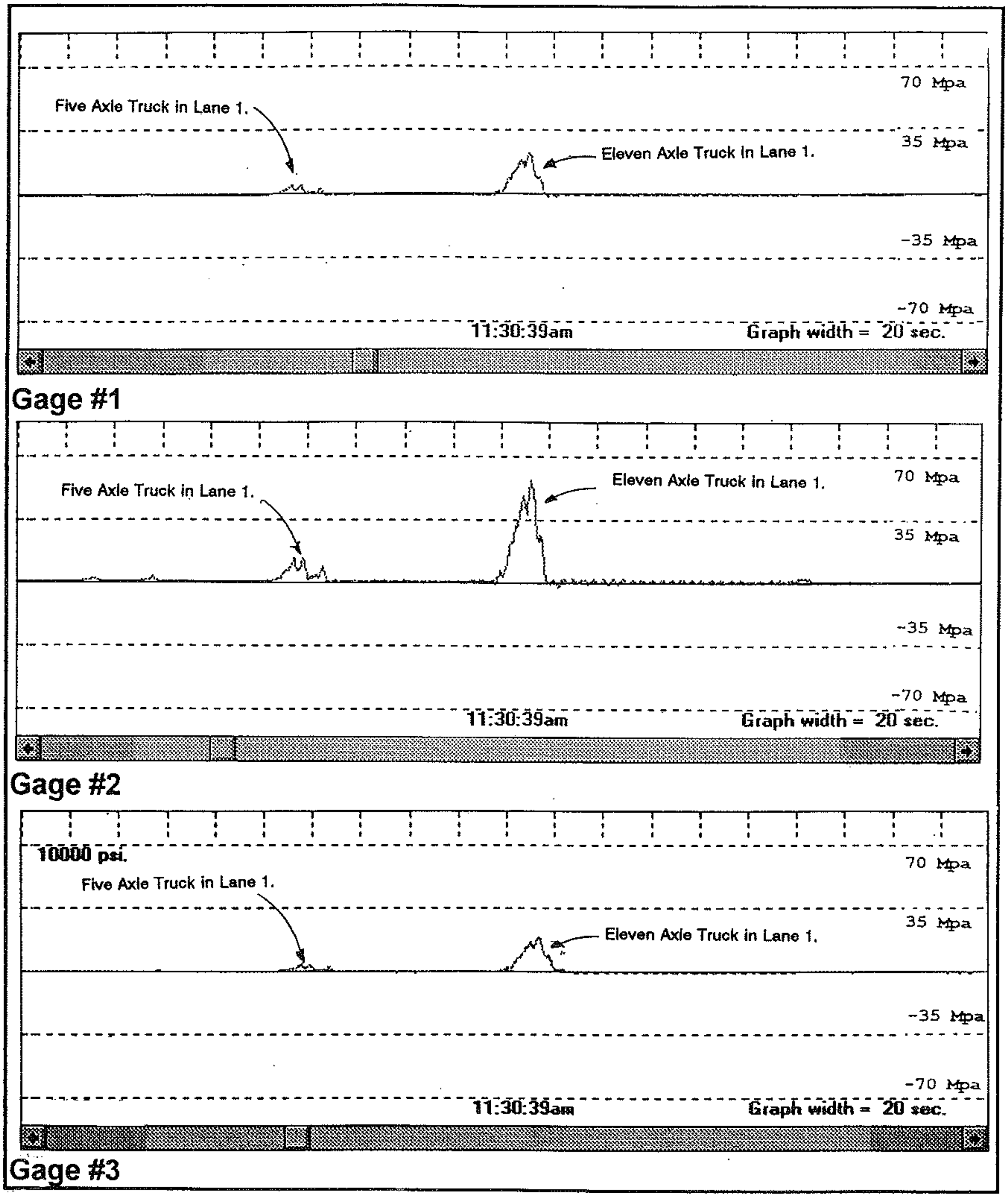


Figure 21 - Stress vs. Time Plot.
Structure Number S02 of 33085.
I-96 over M-52, South of Webberville.
Expansion End.

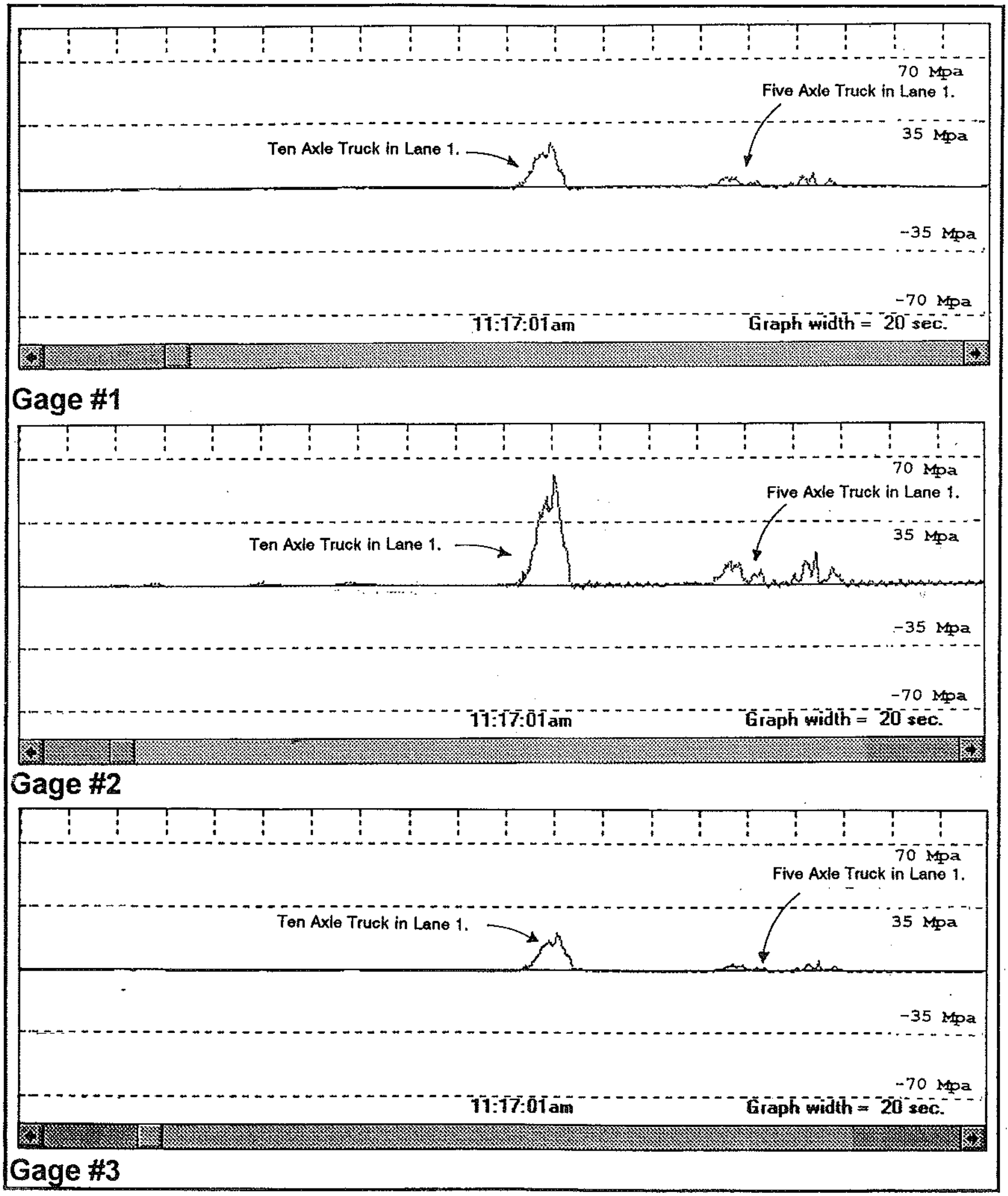


Figure 22 - Stress vs. Time Plot.
Structure Number S02 of 33085.
I-96 over M-52, South of Webberville.
Expansion End.

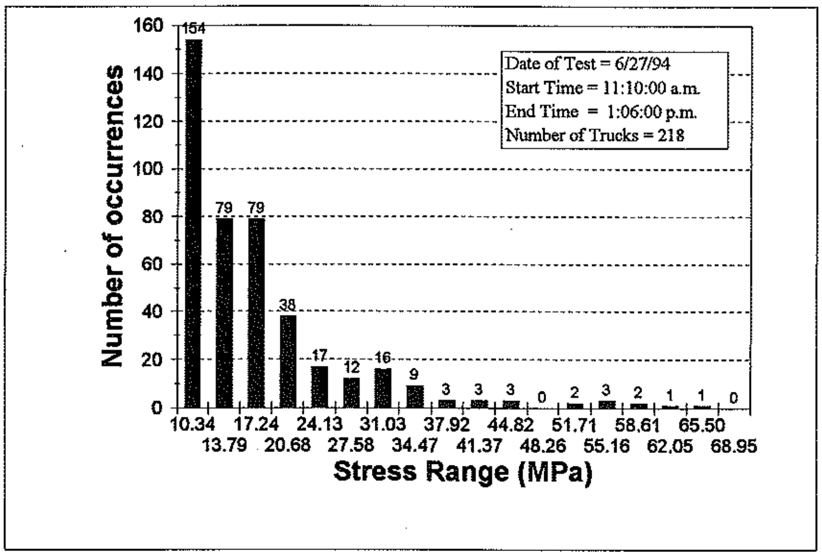


Figure 23 Histogram.
I-96 over M-52 South of Webberville.
Gage 5, Fixed End.

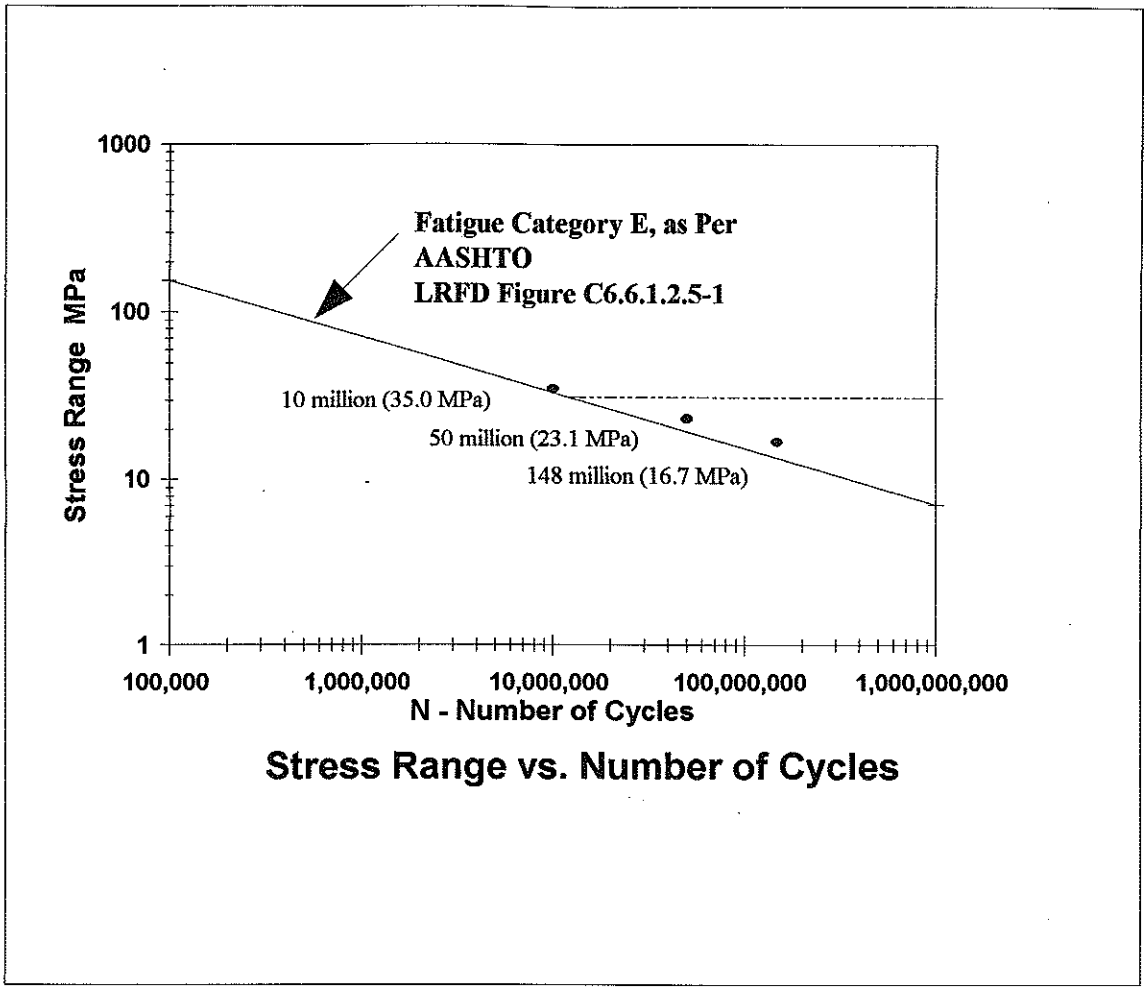


Figure 24

Estimated Constant Amplitude Equivalent Stress at Net Section of the Pin Hole

Based on 6/27/94 strain data using a 75 year design life.

South of Webberville. Fixed End.

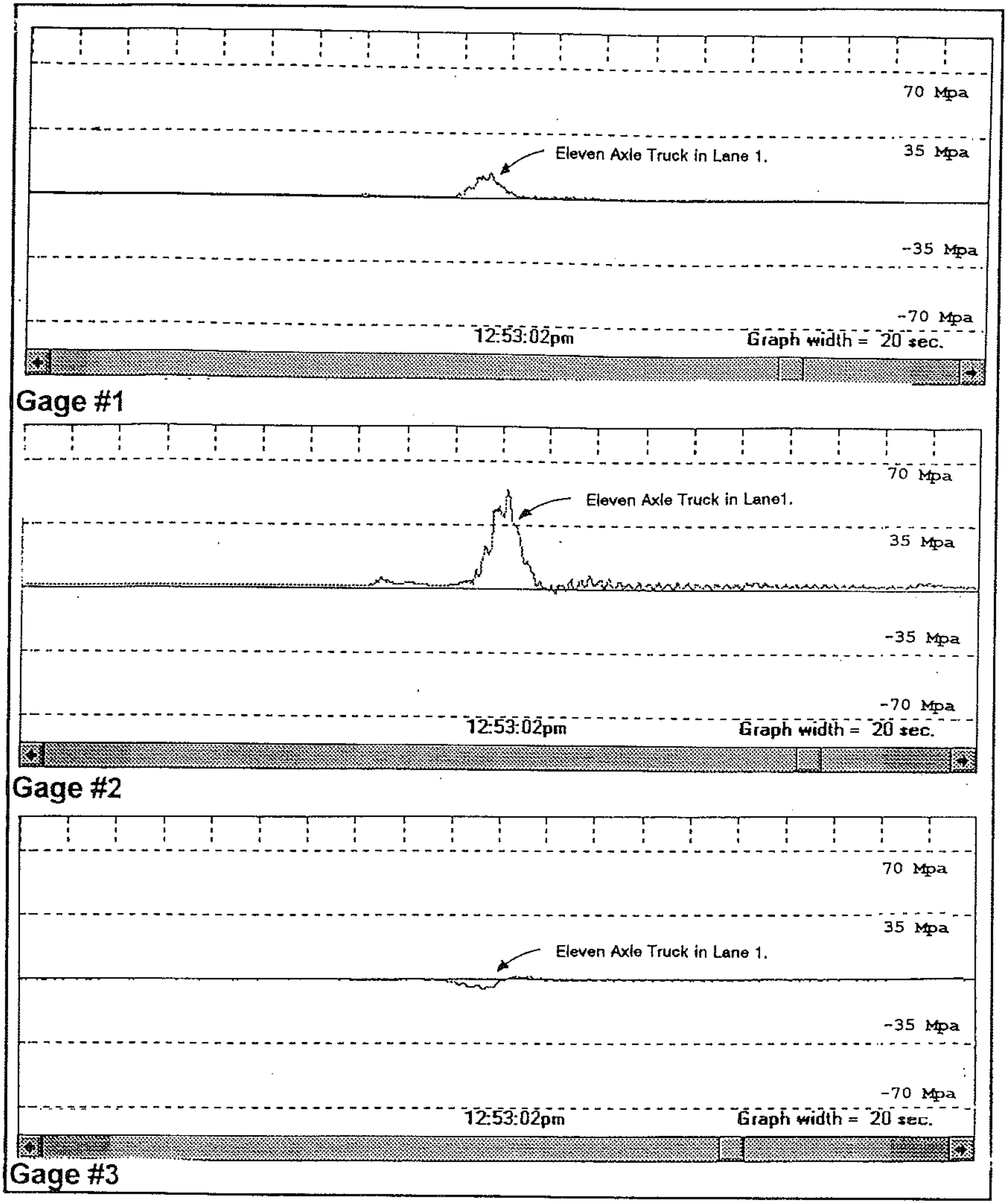


Figure 25 - Stress vs. Time Plot.
Structure Number S02 of 33085.
I-96 over M-52, South of Webberville.
Expansion End.

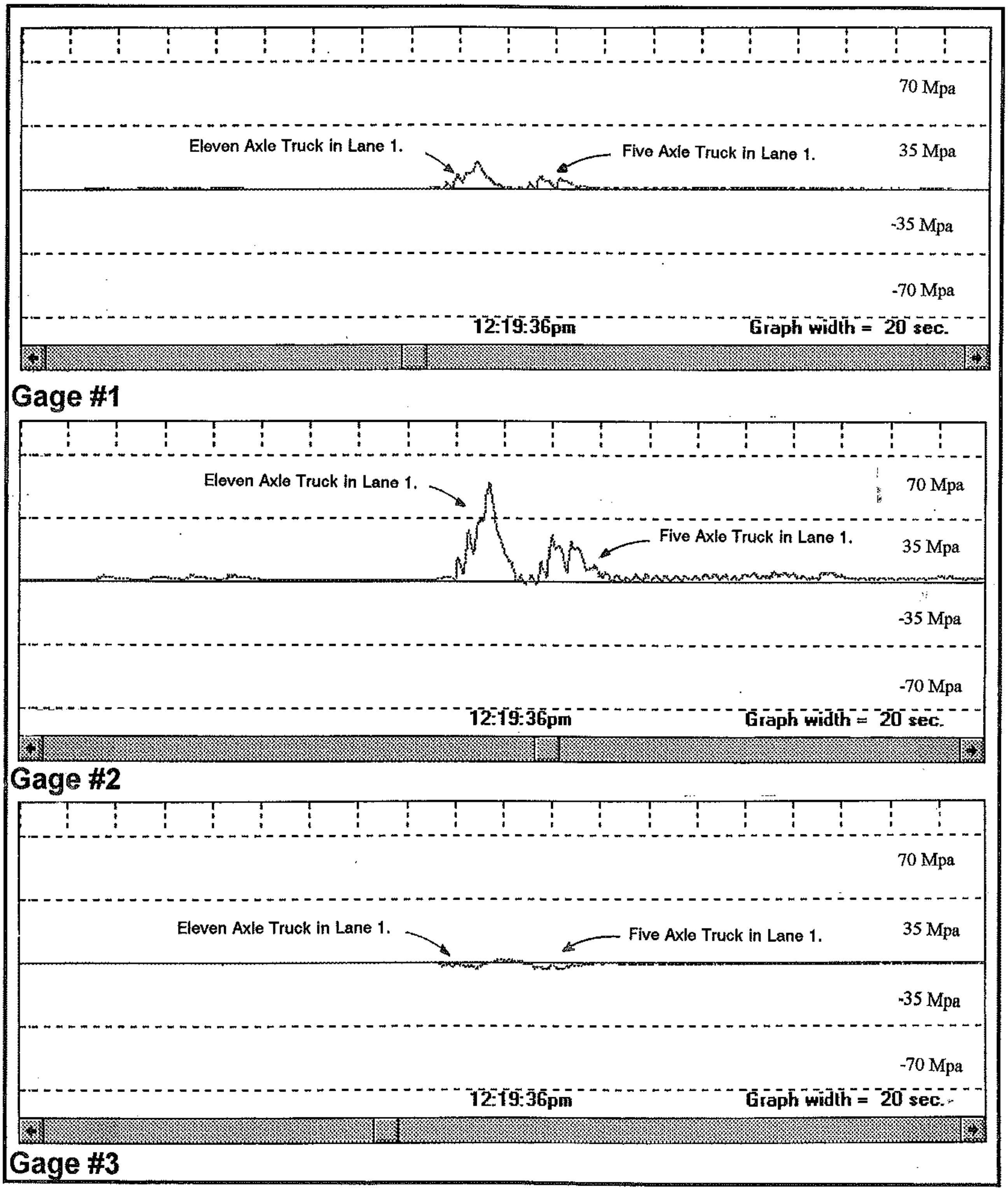


Figure 26 - Stress vs. Time Plot.
Structure Number S02 of 33085.
I-96 over M-52, South of Webberville.
Expansion End.

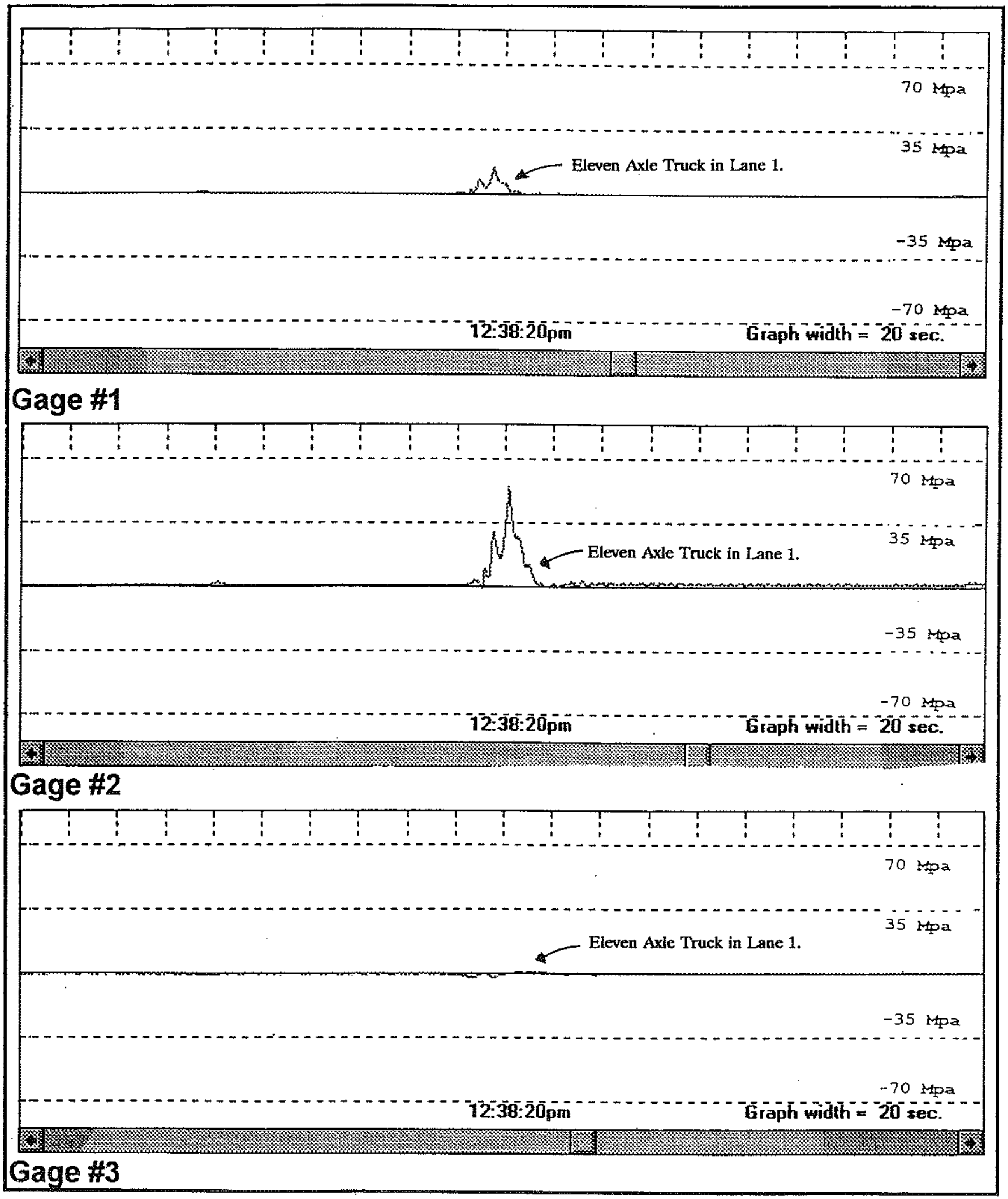


Figure 27 - Stress vs. Time Plot.
Structure Number S02 of 33085.
I-96 over M-52, South of Webberville.
Expansion End.

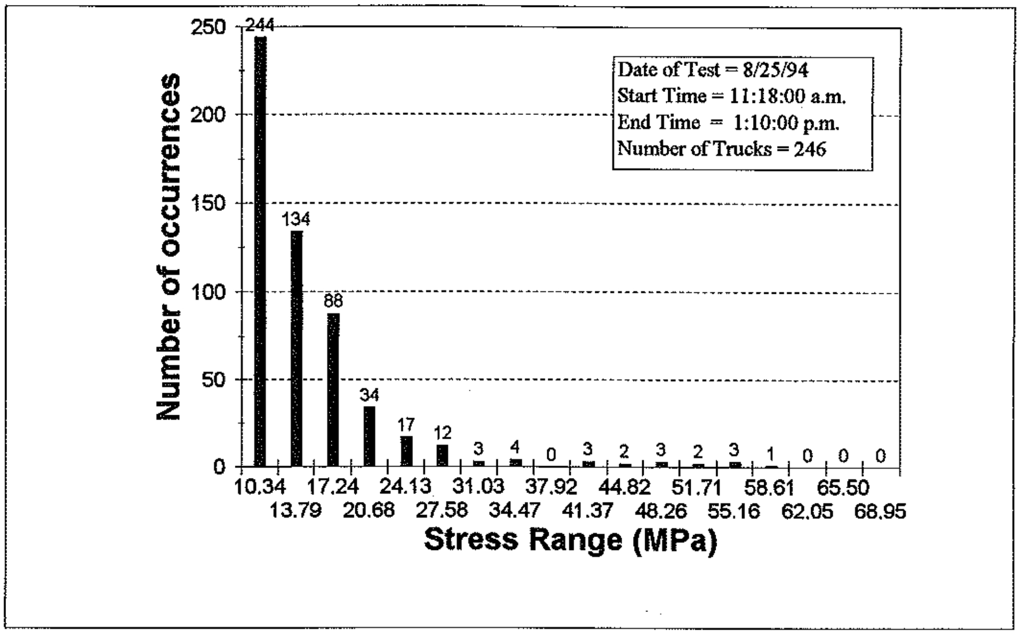


Figure 28 Histogram.
I-96 over M-52 South of Webberville.
Gage 5, Expansion End.

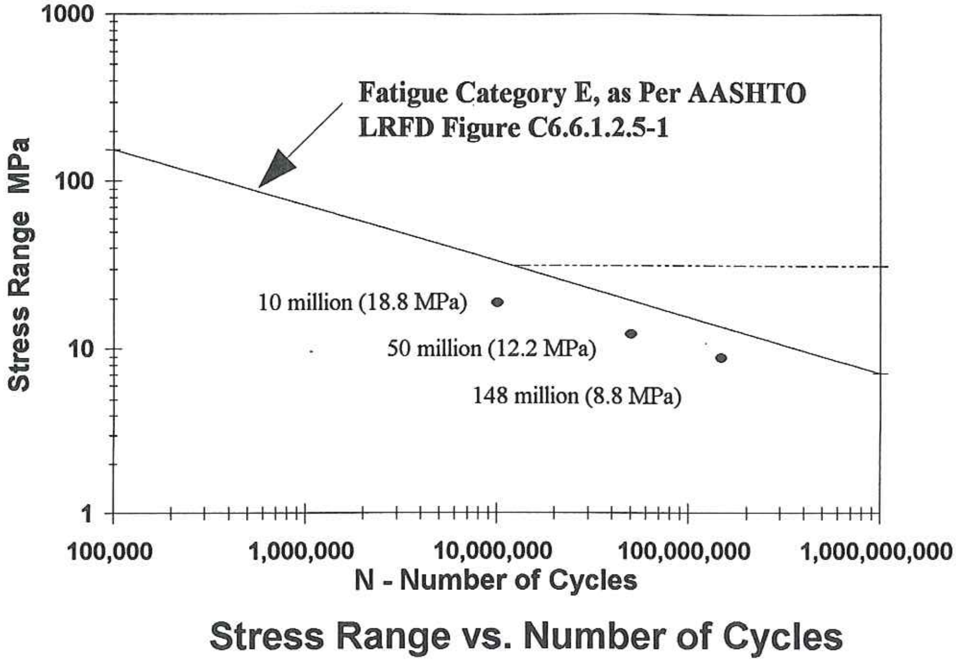


Figure 29
Estimated Constant Amplitude Equivalent Stress at Net Section of the Pin Hole Based on 8/25/94 strain data using a 75 year design life.
South of Webberville. Expansion End.

Figure 30. Link Plate. M-14/US-23 over the Huron River and Conrail Railroad.
Note: Technician is preparing link plate for strain gage.

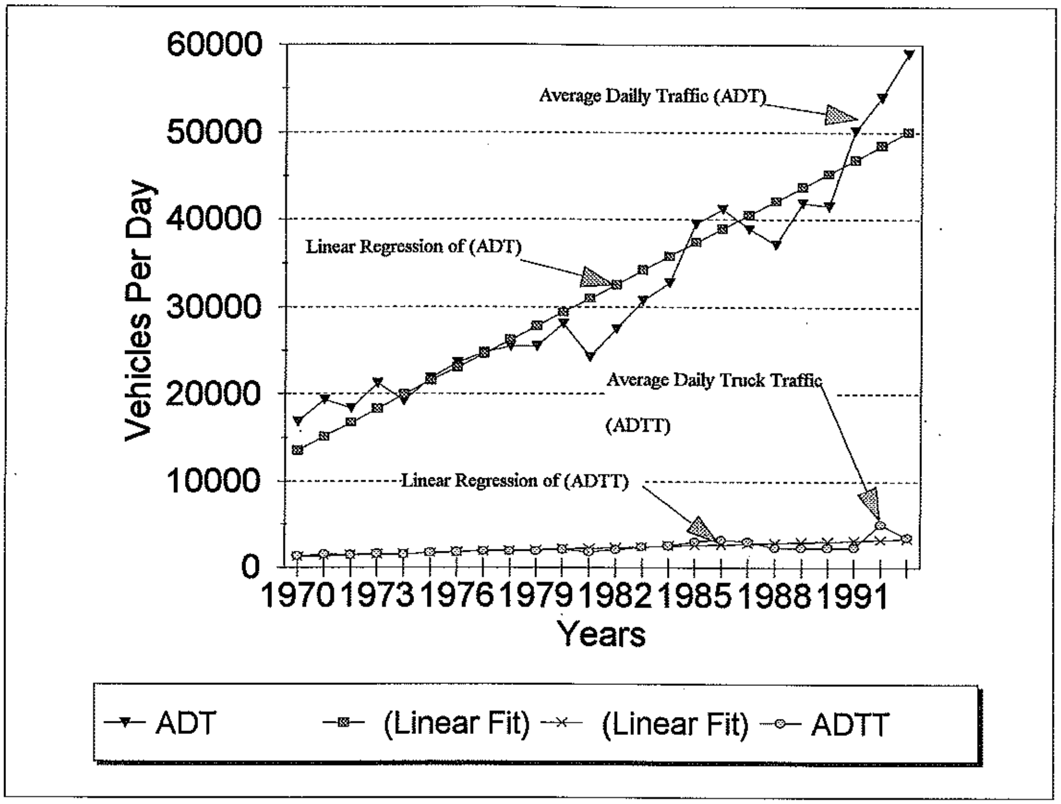


Figure 31 - ADT and ADTT for M-14/US-23 BR over Conrail/Huron River.

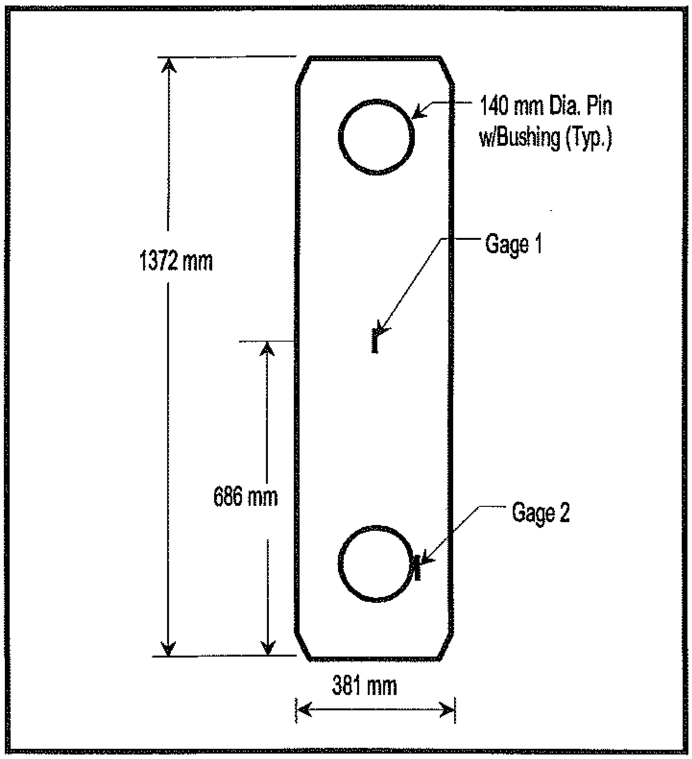
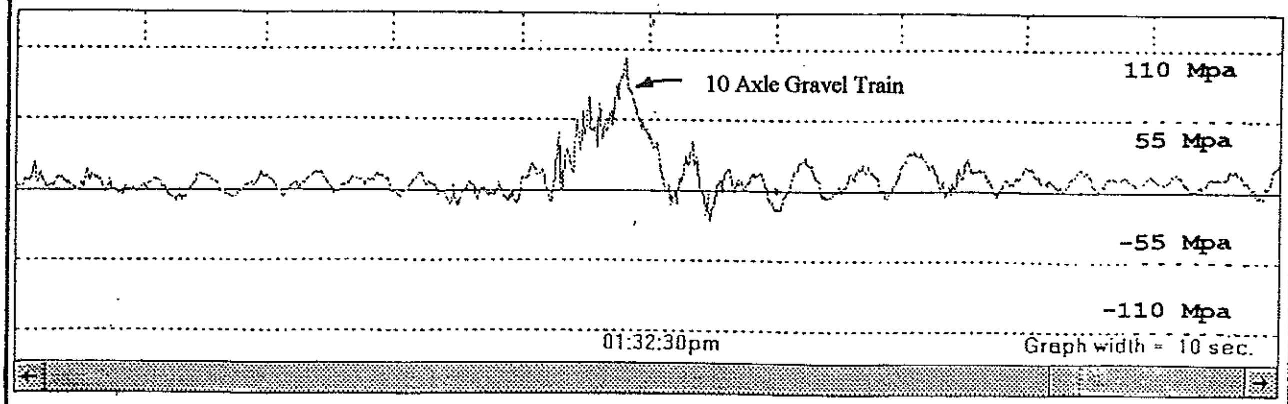
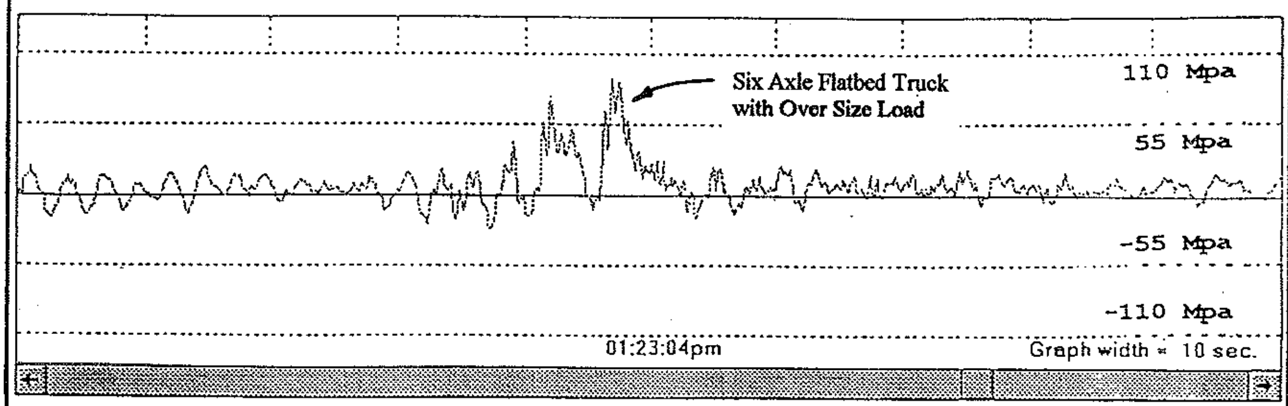


Figure 32. Span 10, Pier 10, West Fascia Beam.
Link Plate Showing Strain Gages.

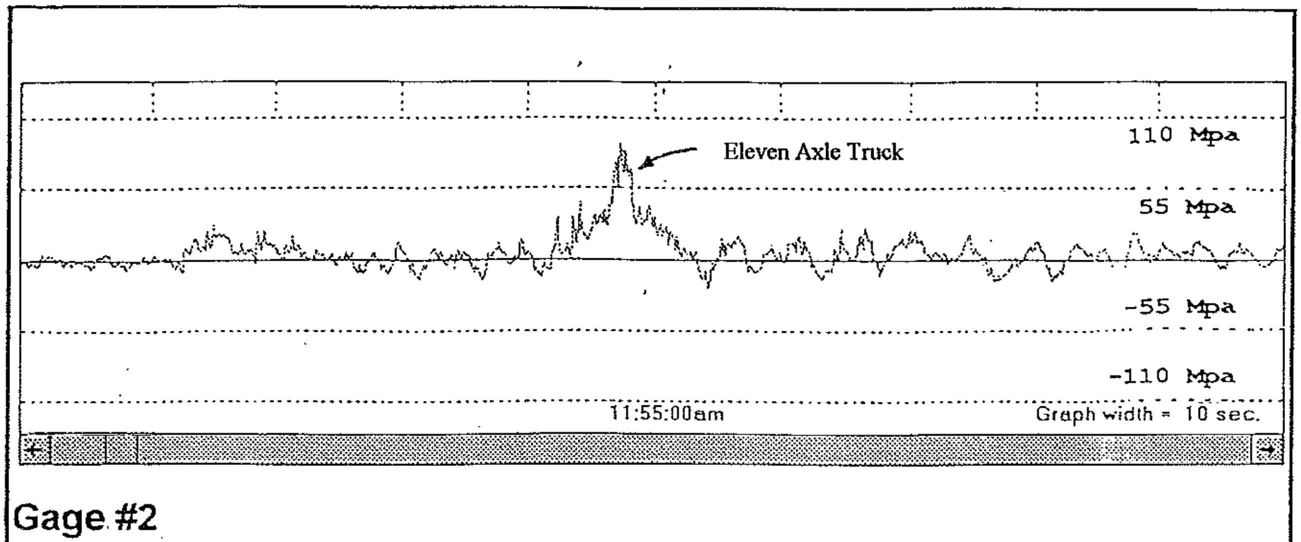


Gage #2



Gage #2

Figure 33 - Stress vs. Time Plot.
Structure Number R01 of 81075.
M-14/US-23 over Conrail/Huron River.
Expansion End.



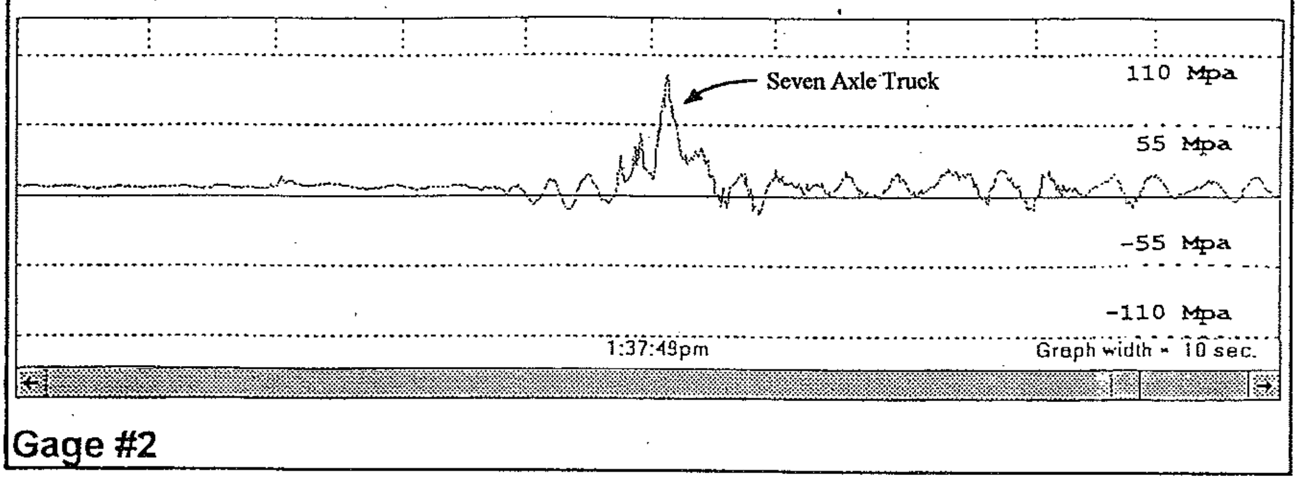


Figure 34 - Stress vs. Time Plot.
Structure Number R01 of 81075.
M-14/US-23 over Conrail/Huron River.
Expansion End.

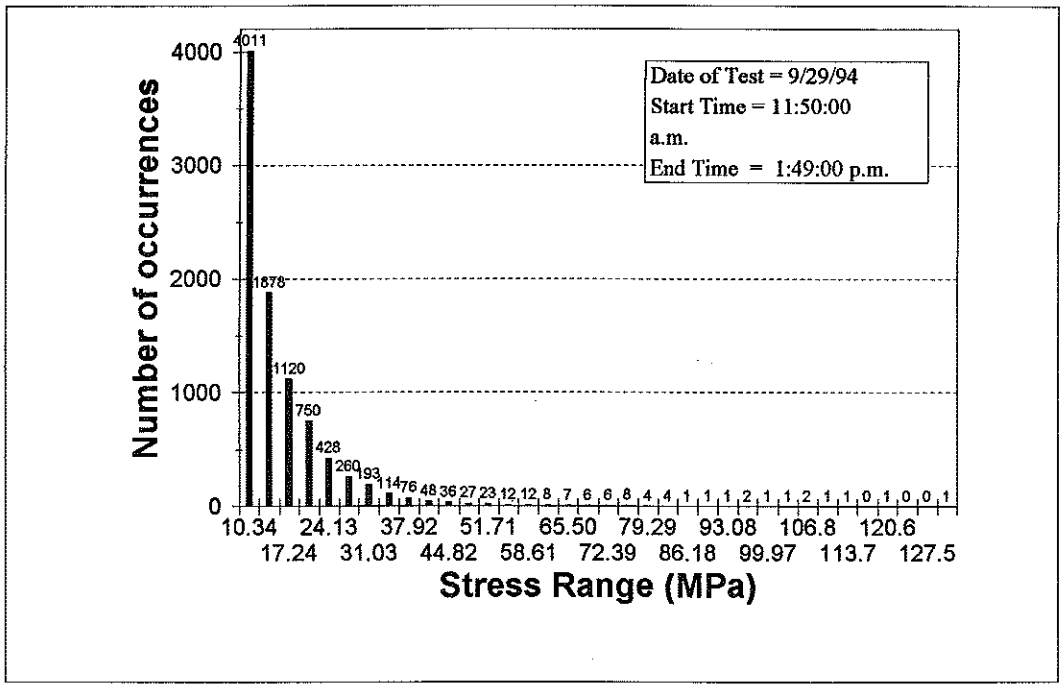


Figure 35 - Histogram.
M-14/US-23 over Conrail/Huron River.
Gage 2.

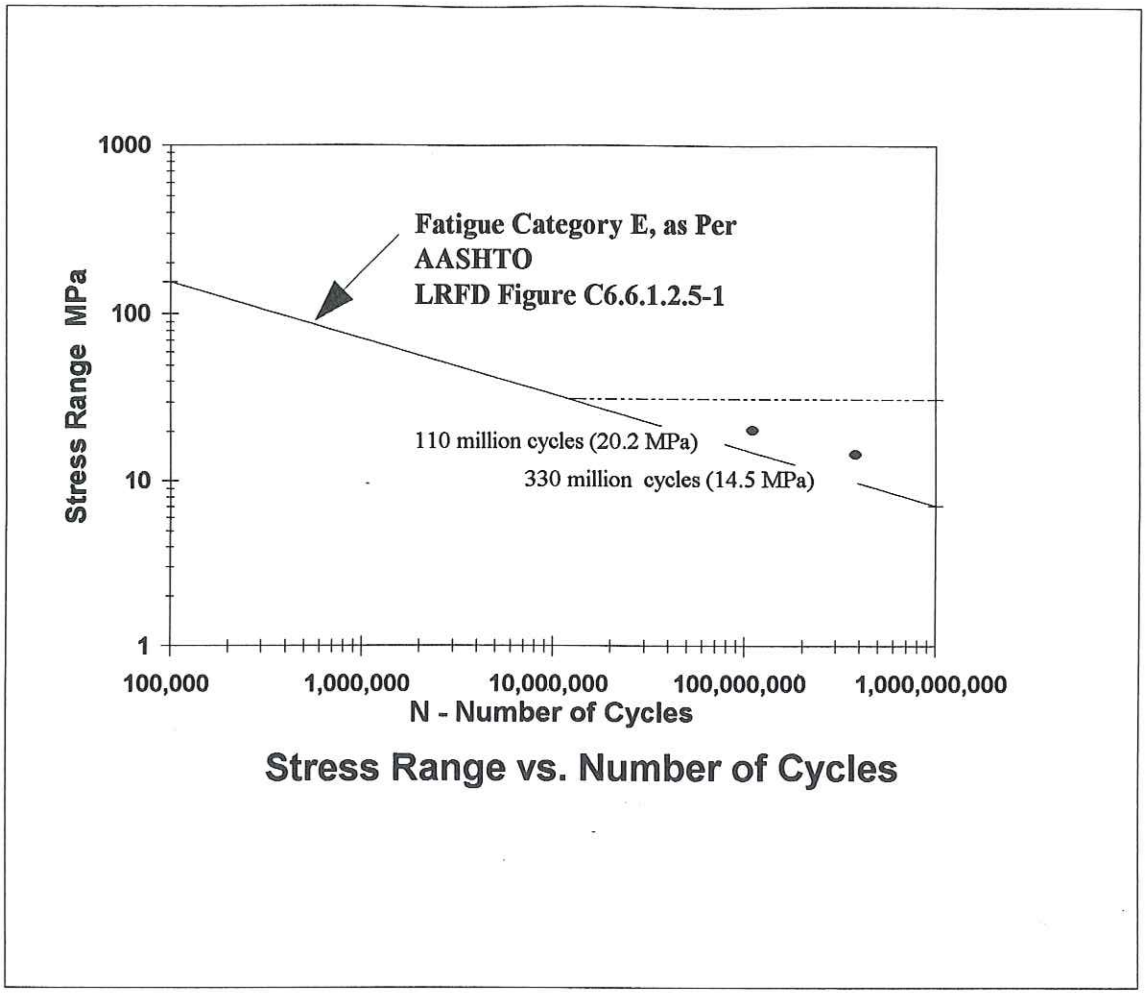


Figure 36

Estimated Constant Amplitude Equivalent Stress at Net Section of the Pin Hole.

Based on 10/29/94 strain data using a 75 year design life.

M-14/US-23 over Conrail Railroad and the Huron River.


Figure 37 - Link Plate. I-75 over Toledo/Dix Highway.

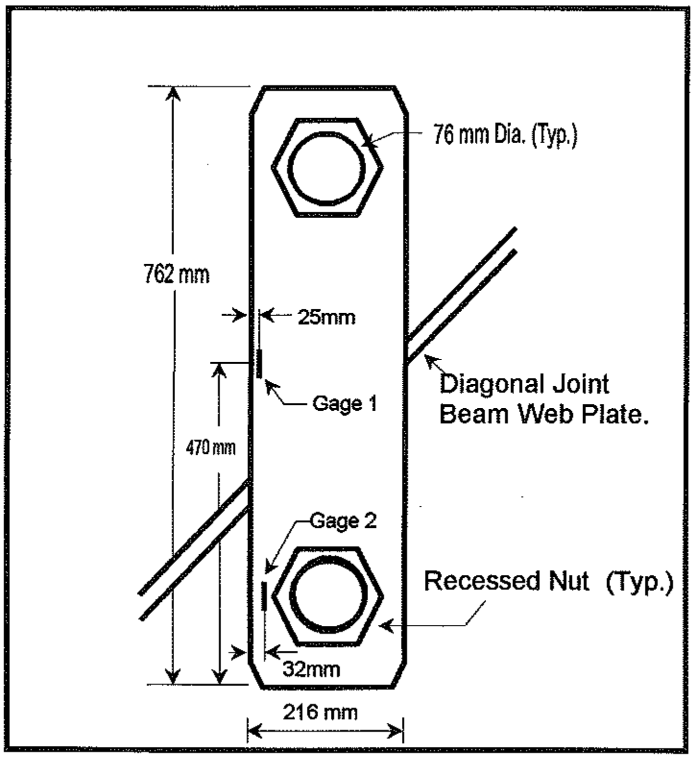


**Figure 39. I-75 over Toledo/Dix Avenue.
Beam 6, Span 8
Expansion End Link Plate with Strain Gages.**

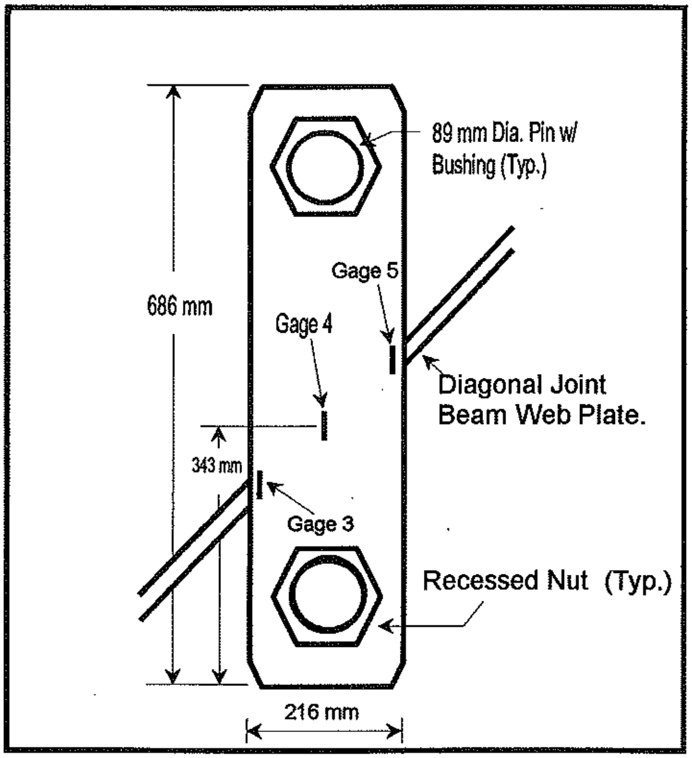


**Figure 40. I-75 over Toledo/Dix Avenue.
Beam 5, Span 8
Expansion End Link Plate with Strain Gages.**

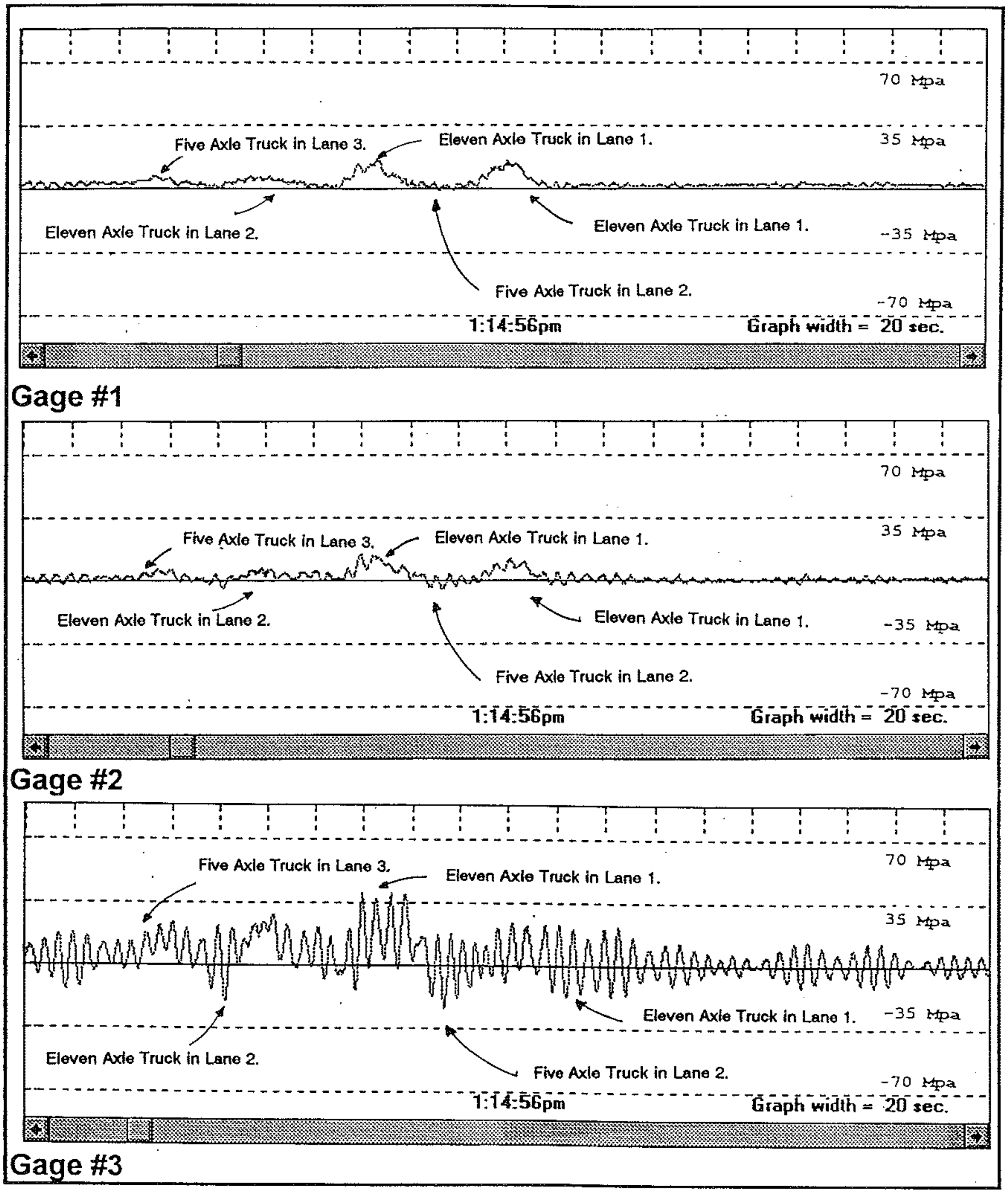


Figure 41 - Stress vs. Time Plot.
Structure Number S21 of 82191.
I-75 over Toledo/Dix Avenue.

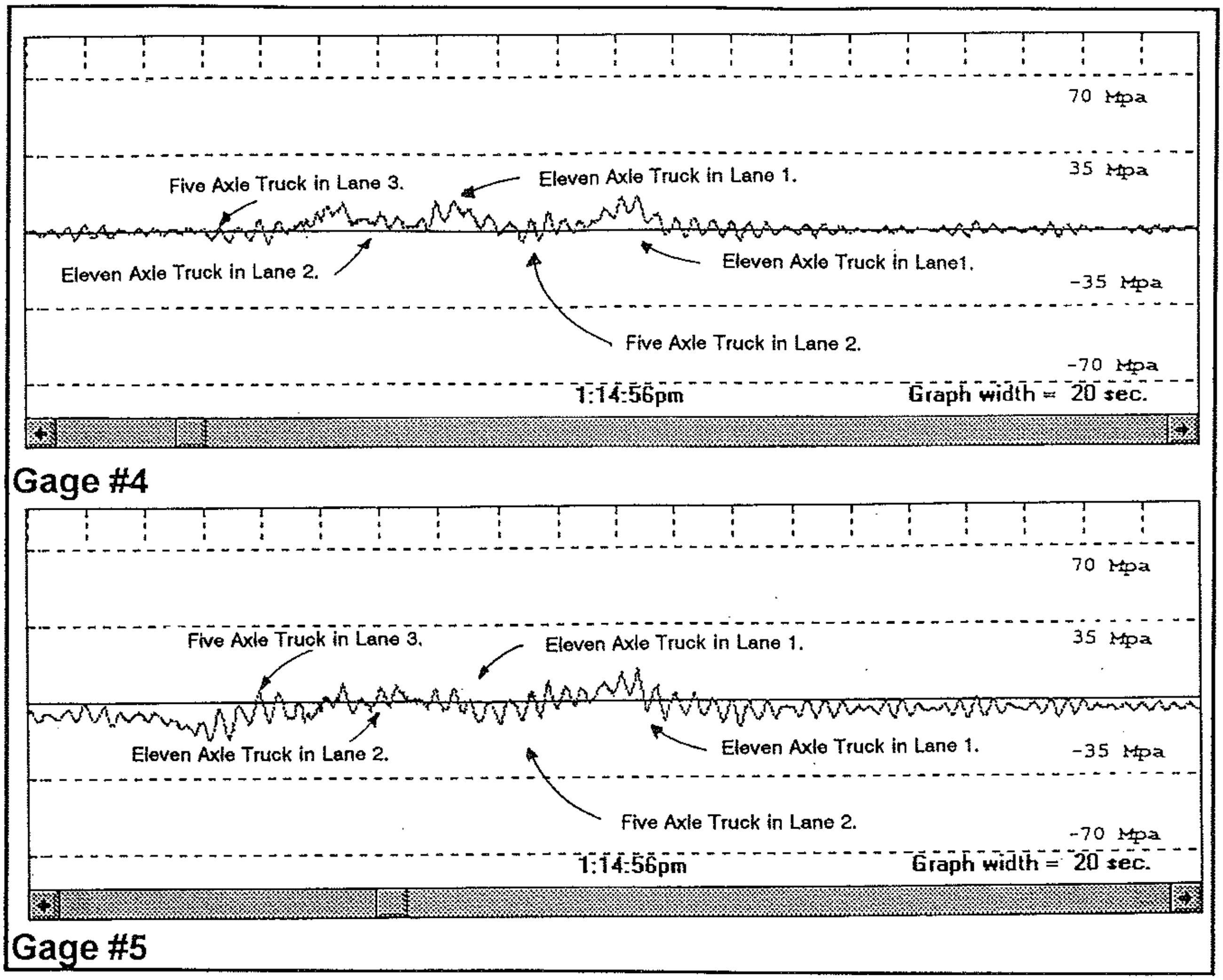


Figure 42 - Stress vs. Time Plot.
Structure Number S21 of 82191.
I-75 over Toledo/Dix Avenue.

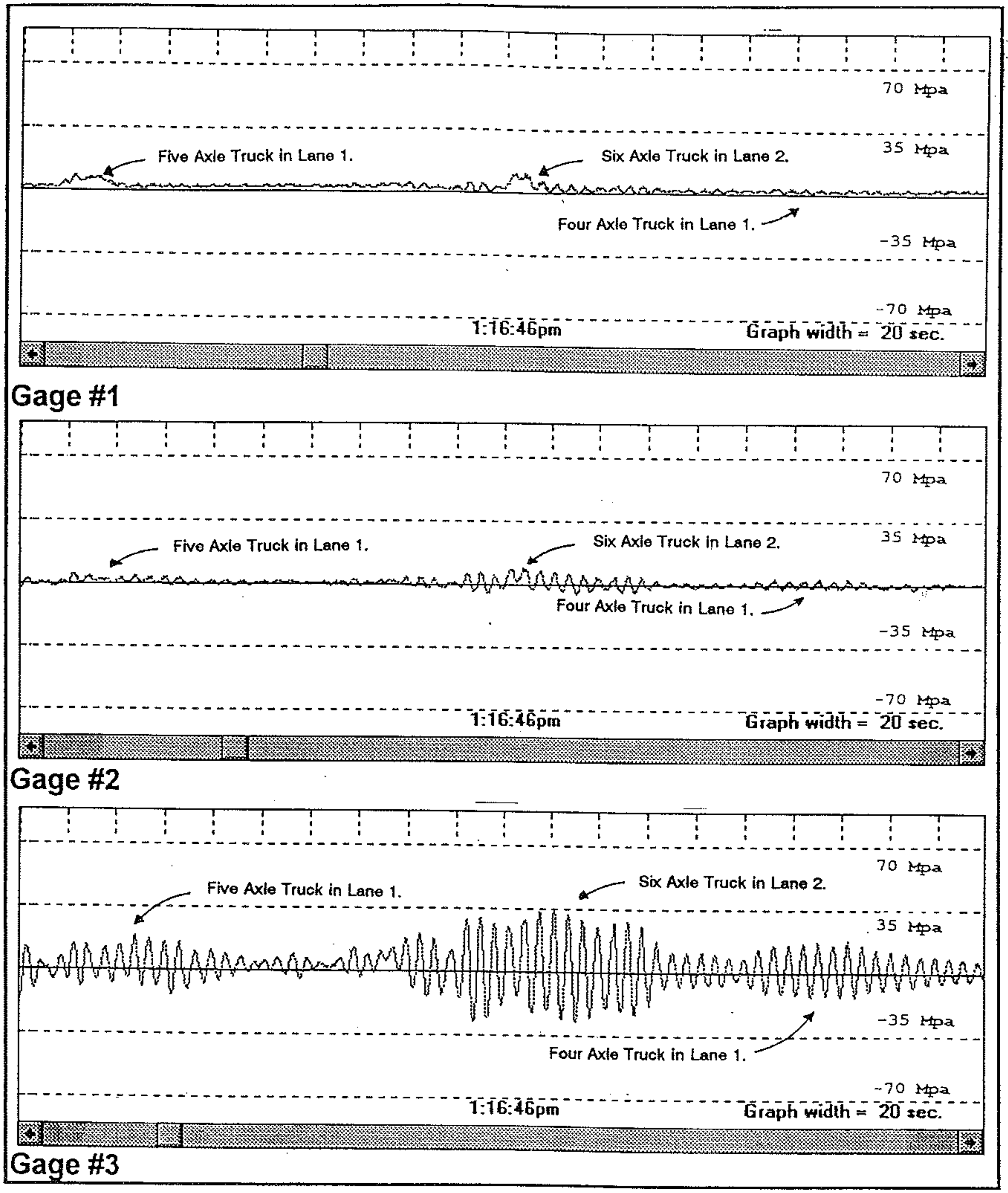


Figure 43 - Stress vs. Time Plot.
Structure Number S21 of 82191.
I-75 over Toledo/Dix Avenue.
Expansion End

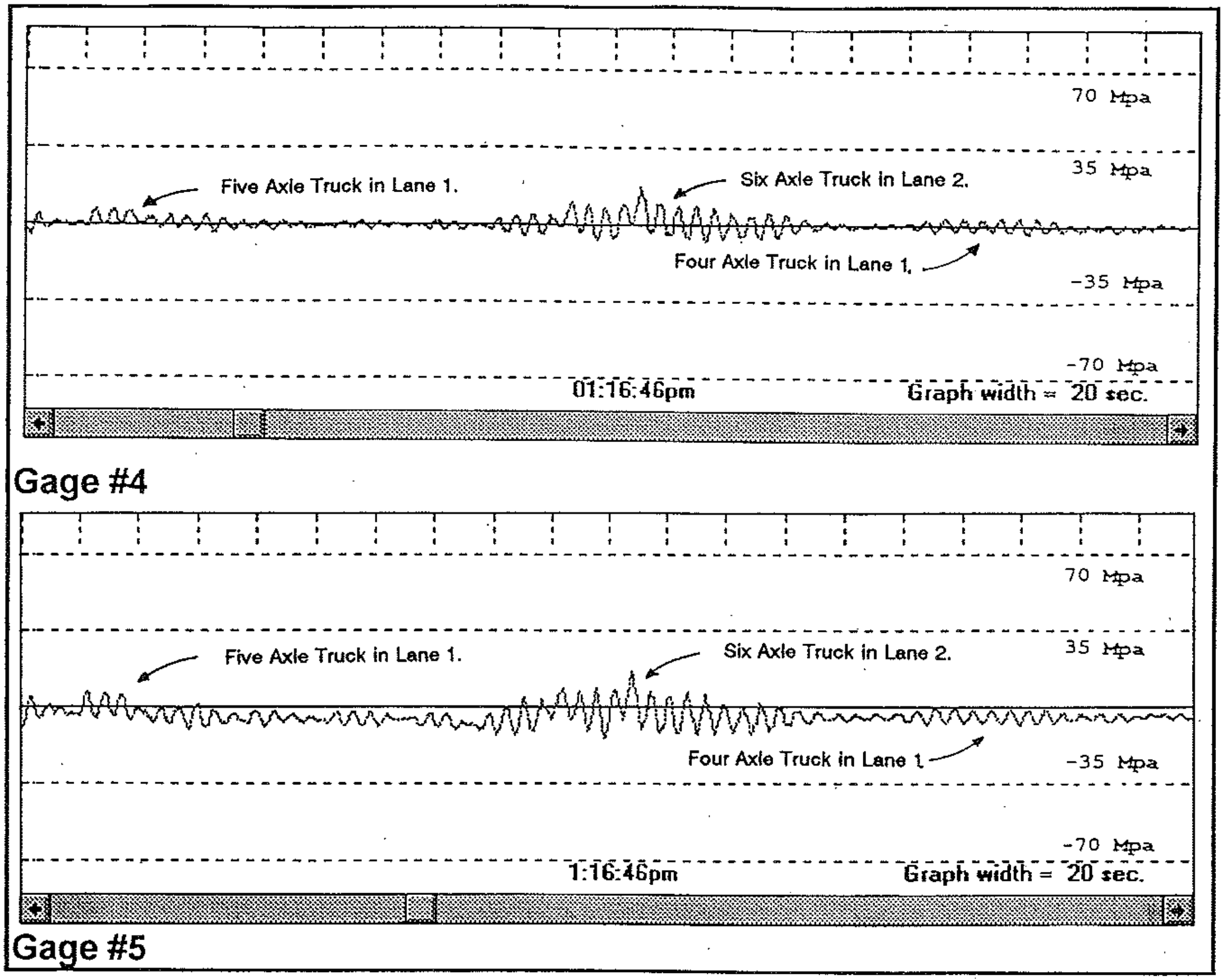


Figure 44 - Stress vs. Time Plot.
Structure Number S21 of 82191.
I-75 over Toledo/Dix Avenue.
Expansion End

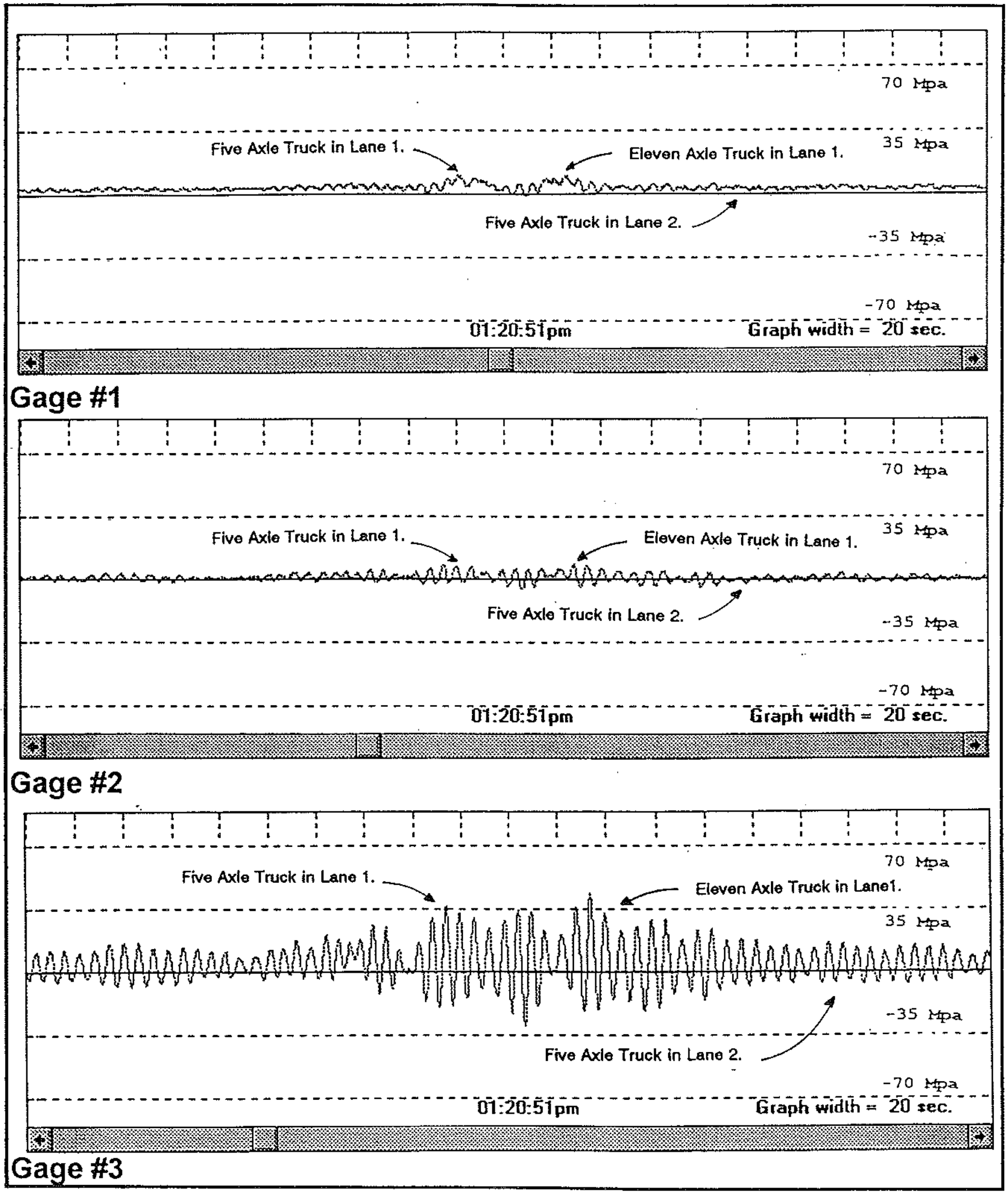
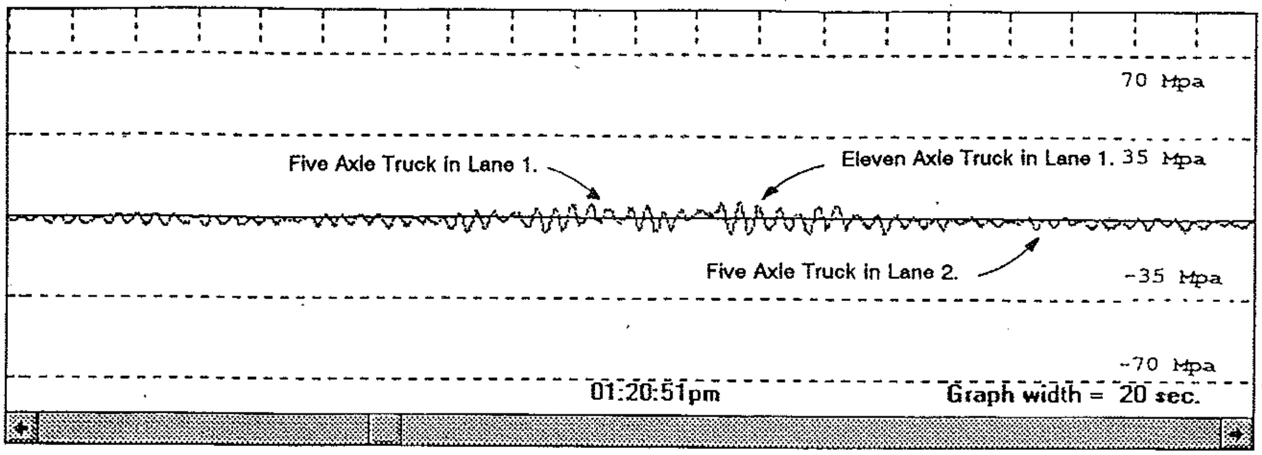
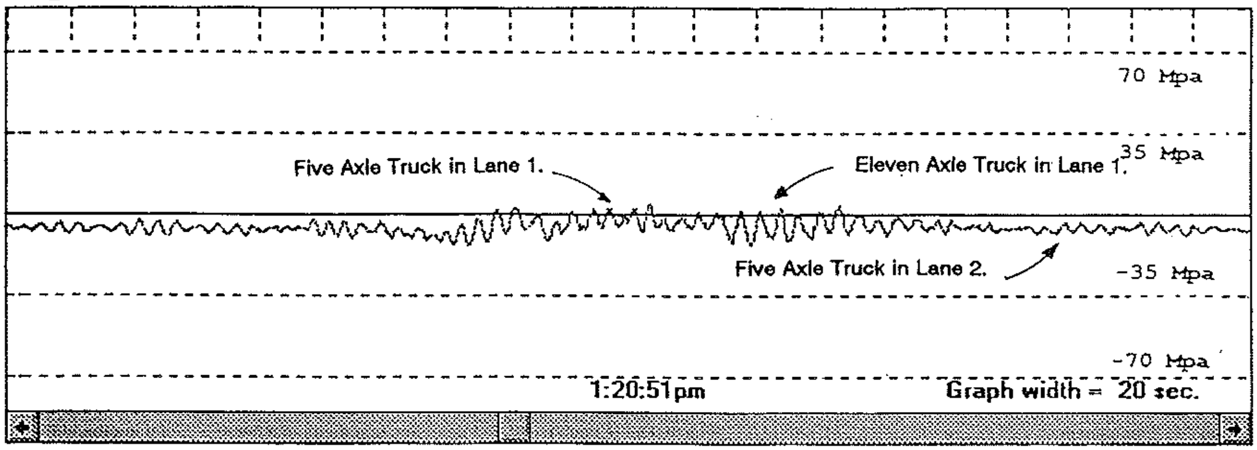


Figure 45 - Stress vs. Time Plot.
Structure Number S21 of 82191.
I-75 over Toledo/Dix Avenue.
Expansion End



Gage #4



Gage #5

Figure 46 - Stress vs. Time Plot.
Structure Number S21 of 82191.
I-75 over Toledo/Dix Avenue.
Expansion End

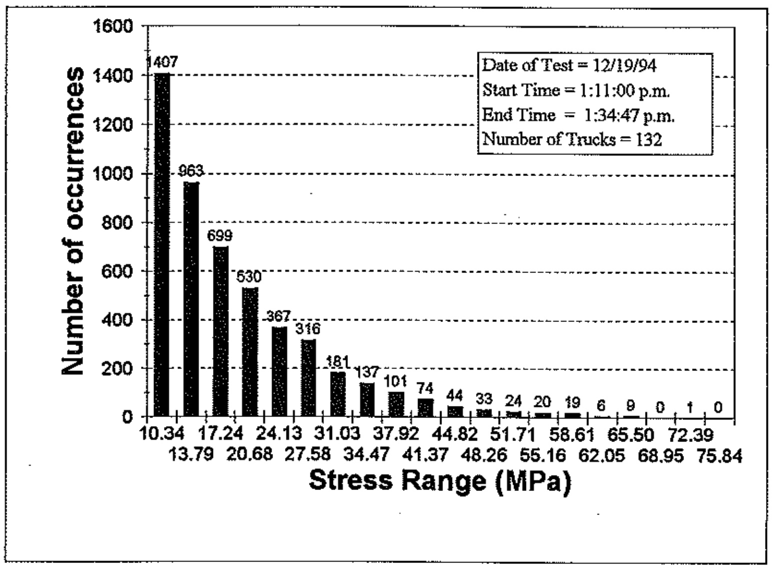


Figure 47 - Histogram.
I-75 over Toledo/Dix Avenue.
Gage 3.

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