
**Research Report
KTC 94-10**

**Summary of Stress Evaluations
of Welded Steel Bridges
on Coal-Haul Routes**

by

Theodore Hopwood, II
Transportation Research Engineer

Keith J. Hogan
Former Graduate Research Assistant

and

Christopher M. Oberst
Transportation Research Technologist

Kentucky Transportation Center
College of Engineering
University of Kentucky
Lexington, Kentucky

in cooperation with
Transportation Cabinet
Commonwealth of Kentucky

and

Federal Highway Administration
U.S. Department of Transportation

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the views or policies of the University of Kentucky, the Kentucky Transportation Cabinet, nor the Federal Highway Administration. This report does not constitute a standard, specification, or regulation. The inclusion of manufacturer names and trade names are for identification purposes and are not to be considered as endorsements.

May 1994



Commonwealth of Kentucky
Transportation Cabinet
Frankfort, Kentucky 40622

Fred N. Mudge
Secretary of Transportation

Paul E. Patton
Governor

Mr. Paul E. Toussaint
Division Administrator
Federal Highway Administration
330 West Broadway
Frankfort, Kentucky 40601

Dear Mr. Toussaint:

SUBJECT: IMPLEMENTATION STATEMENT
Research Study KYHPR 91-139
"Stress Evaluation of Welded Steel Bridges on Coal Haul Routes"

Under this study, Kentucky Transportation Center (KTC) researcher performed comprehensive field live stress measurements on 15 welded steel bridges on extended weight coal haul routes and on one interstate bridge (for comparative purposes). Those measurements were intended to determine whether coal truck traffic posed significant risks to those structures by fatigue cracking. Additionally, this data was acquired to inform KYTC designers of the actual service stress experienced by those structures.

The researcher's acquired data and subsequent analyses indicate, as expected, that service load stresses are relatively low. Also, for the members tested, fatigue does not seem to be a problem with relatively low volumes of overweight vehicles. As pointed out in the report, very few locations tested in this study were in areas most susceptible to fatigue problems. In fact, nine sites had no category E or higher details.

The test procedures employed in this study will be used when needed to evaluate other welded steel bridges with potential problems. The findings provided in the report fulfill the stated objectives of the study.

Sincerely,

A handwritten signature in cursive script, reading "J. M. Yowell".

J. M. Yowell, P.E.
State Highway Engineer

c: D. E. Smith
R. K. Sutherland

1. Report No. KTC-94-10		2. Government Accession No.		3. Recipient's Catalog No.	
4. Title and Subtitle Summary of Stress Evaluations of Welded Steel Bridges on Coal-Haul Routes				5. Report Date May 1994	
				6. Performing Organization Code	
7. Author(s) T. Hopwood, II; K.J. Hogan; and C. M. Oberst				8. Performing Organization Report No. KTC-94-10	
9. Performing Organization Name and Address Kentucky Transportation Center College of Engineering University of Kentucky Lexington, KY 40506-0281				10. Work Unit No. (TRAIS)	
				11. Contract or Grant No. KYHPR-91-139	
				13. Type of Report and Period Covered Final	
12. Sponsoring Agency Name and Address Kentucky Transportation Cabinet State Office Building Frankfort, KY 40622				14. Sponsoring Agency Code 4D1A1111	
15. Supplementary Notes Prepared in Cooperation with US Department of Transportation, Federal Highway Administration Study Title: Stress Evaluation of Welded Steel Bridges on Coal-Haul Routes					
16. Abstract Stress analyses were performed on continuous girder welded steel bridges on extended weights coal-haul routes. The tests were intended to determine whether extended weight coal trucks pose fatigue problems to those bridges. Measurements were performed by strain gaging selected bridges subject to high coal transport tonnages. Stress measurements were conducted on fatigue-prone weld details or test sites where high tensile stresses were anticipated. Test sites on the bridges were instrumented with strain gages. Strains induced by routine traffic including coal trucks were monitored for periods of one to two weeks. Unattended monitoring of the variable amplitude strain data was performed using rainflow counting. Eighteen successful tests were performed on 15 coal-haul route bridges and one interstate bridge. The derived strain data are provided as stress histograms. Fatigue analyses were performed by expressing the stress histogram data as single-value equivalent stresses. The accumulated number of stress cycles was estimated using 3 different assumptions based upon variations in traffic. Accumulated stress cycles were determined over the current age of each weld detail and a projected service life of 75 years. Susceptibility to fatigue was determined by superimposing the equivalent resolved stresses and total number of cycles as accumulated damage on AASHTO fatigue design curves for the applicable structural details. The fatigue analyses indicate that none of the test bridges with fatigue-prone weld details is susceptible to fatigue cracking either at their current age or over their project 75-year service lives. While coal trucks may induce high live stresses on those bridges, the number of those stress applications was not sufficient to pose fatigue problems. The equivalent resolved stresses measured on the interstate bridge were similar in magnitude to those measured on coal-haul routes. However, the number of stress cycles was greater for the interstate bridge than most of the coal-haul route bridges.					
17. Key Words Bridges Coal-haul routes Fatigue Strain gaging			18. Distribution Statement Unlimited, with approval of the Kentucky Transportation Cabinet		
19. Security Classif. (of this report) Unclassified		20. Security Classif. (of this page) Unclassified		21. No. of Pages 57	22. Price

EXECUTIVE SUMMARY

Kentucky statutes provide for the legal operation of overweight coal trucks on specific roads designated as extended weight coal-haul routes. Permitted coal trucks using those routes may weigh up to 120,000 lbs. Those trucks may increase the susceptibility of welded steel bridges on those routes to fatigue cracking. Fatigue cracks may disable a bridge or, possibly, cause catastrophic failure. Therefore, the impact of permitted overweight coal trucks on welded steel bridges is of concern to Kentucky Transportation Cabinet (KyTC) officials.

Welded steel bridges may be prone to fatigue cracking due to live stresses induced by routine traffic. The propensity for fatigue cracking depends upon the magnitude of live stresses and the accumulated number of repetitive stress applications (i.e. cycles). Fatigue cracks may be caused by fabrication flaws. More commonly, they result from the nucleation of cracks at weld details that act as stress risers. When the service stresses and the total number of stress applications experienced by a particular weld detail are known, the susceptibility to fatigue cracking can be determined.

In this study, welded steel bridges on extended weight coal-haul routes with high tonnages of coal-truck traffic were identified by KyTC officials. Drawings and coal data were reviewed to identify fatigue-prone weld details and determine structural members that were subject to high live loads. Fatigue-prone weld details or high stress areas of bridge members were strain gaged. Thereafter, variable-amplitude live stresses induced by routine traffic including coal trucks were monitored over periods typically from one to two weeks using battery-powered signal conditioner/data logging instruments. After the test period elapsed, the instruments and data were retrieved.

In this study, 20 welded steel bridges were to be monitored. All of those bridges were continuous welded plate girder or I-beam structures. The test structures included 19 bridges on extended weight coal-haul routes and one interstate bridge. Due to several factors, field data were obtained from only 16 bridges including the interstate structure. Two of those bridges were tested twice to accommodate for changes in coal-truck traffic and to determine the effect of weld-detail locations along girders.

Data obtained from the field monitoring were represented as stress histograms. The histograms represent the distribution of variable-amplitude live stresses in terms of preselected stress ranges and frequencies of occurrence. Those data may be used to validate design assumptions or, as was the focus of this study, to perform fatigue analyses.

Fatigue analyses were performed by resolving the stress histograms into single-valued equivalent stresses that produce equivalent fatigue damage to the variable-amplitude stresses for the number of cycles measured. The total number of equivalent stress cycles applied over the current age of a bridge and its projected 75-year service life were estimated by assuming: 1) a 5 percent cyclic growth rate over the present age of the bridge, 2) a constant cyclic rate over the present age of the bridge and 3) a variable cyclic

rate over the present age of the bridge based upon regression analysis of traffic (ADTs).

~~The single-valued equivalent stress and total number of accumulated stress cycles were projected as damage on the American Association of State Highway Transportation Officials (AASHTO) fatigue design curves for the applicable weld details monitored on the test bridges. Typically, fatigue-prone welded details on the test bridges were AASHTO Category E details. Data points projected for all the bridge test locations lie below fatigue limits provided in AASHTO fatigue design curves. That includes the 75-year projected service lives.~~

The analyses indicate that fatigue cracking due to extended weight coal-truck traffic is not a potential problem for the particular welded bridges that were monitored. Since a significant number of bridges were tested, that probably applies to most continuous girder (or I-beam) welded steel bridges on the extended weight coal-haul routes.

That finding is due to the fact that those bridges are not subject to high coal-truck traffic volumes, despite the occasional heavy coal-haul truck stresses they sustain. The single-valued equivalent stresses measured on the interstate bridge compares to those measures on extended weight coal-haul bridges. However, the frequency of stress applications on the interstate bridge was higher than those measured on most extended weight coal-haul bridges.

On the bridges tested, only AASHTO fatigue category E (or higher) details warranted consideration for fatigue problems. For the continuous steel girder bridges, it is likely that "arms-length" or nondestructive inspections could be concentrated on such details when searching for fatigue cracks.

The focus of this study was limited to the potential for fatigue cracking on bridges. The study was not intended to address broad issues related to the effects of the extended weight coal-haul system including deterioration of bridge decks.

Field strain gaging techniques employed in this study have proven useful to address and resolve a specific concern of KyTC officials. They may be employed to address concerns related to fatigue cracking in welded steel bridges.

METRIC CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO METRIC UNITS					APPROXIMATE CONVERSIONS FROM METRIC UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol	Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH					LENGTH				
in.	inches	25.40000	millimetres	mm	mm	millimetres	0.03937	inches	in.
ft	feet	0.30480	metres	m	m	metres	3.28084	feet	ft
yd	yards	0.91440	metres	m	m	metres	1.09361	yards	yd
mi	miles	1.60934	kilometres	km	km	kilometres	0.62137	miles	mi
AREA					AREA				
in. ²	square inches	645.16000	millimetres squared	mm ²	mm ²	millimetres squared	0.00155	square inches	in. ²
ft ²	square feet	0.09290	metres squared	m ²	m ²	metres squared	10.76392	square feet	ft ²
yd ²	square yards	0.83613	metres squared	m ²	m ²	metres squared	1.19599	square yards	yd ²
ac	acres	0.40469	hectares	ha	ha	hectares	2.47103	acres	ac
mi ²	square miles	2.58999	kilometres squared	km ²	km ²	kilometres squared	0.38610	square miles	mi ²
FORCE					FORCE				
kip	pound-force	4.44822	kilonewton	kN	kN	kilonewton	0.22481	pound-force	kip
VOLUME					VOLUME				
fl oz	fluid ounces	29.57353	millilitres	ml	ml	millilitres	0.03381	fluid ounces	fl oz
gal.	gallons	3.78541	litres	l	l	litres	0.26417	gallons	gal.
ft ³	cubic feet	0.02832	metres cubed	m ³	m ³	metres cubed	35.31448	cubic feet	ft ³
yd ³	cubic yards	0.76455	metres cubed	m ³	m ³	metres cubed	1.30795	cubic yards	yd ³
PRESSURE					PRESSURE				
psi	pound-force per square inch	6.89476	kilopascal	kPa	kPa	kilopascal	0.14504	pound-force per square inch	psi
MASS					MASS				
oz	ounces	28.34952	grams	g	g	grams	0.03527	ounces	oz
lb	pounds	0.45359	kilograms	kg	kg	kilograms	2.20462	pounds	lb
T	short tons (2000 lb)	0.90718	megagrams	Mg	Mg	megagrams	1.10231	short tons (2000 lb)	T
TEMPERATURE (exact)					TEMPERATURE (exact)				
°F	Fahrenheit temperature	(°F-32)/1.8	Celsius temperature	°C	°C	Celsius temperature	(1.8C) + 32	Fahrenheit temperature	°F

TABLE OF CONTENTS

Title Page	i
Implementation Statement	ii
Technical Report Documentation Page	iii
Executive Summary	iv
Metric (SI) Conversion Factors	vi
Table of Contents	vii
Introduction	1
Background	1
Welding and Fatigue of Steel Bridges	1
Initiation of Study	2
Preliminary Work	2
Field Instrumentation	4
Data Analyses and Results	6
Stress Histograms	6
Fatigue Analysis	7
Cracks in Structural Members	8
Conclusions	10
Bibliography	11
Tables	12
Figures	27
APPENDIX A - Stress Histograms	30
APPENDIX B - Accumulated Damage Superimposed on Fatigue	
Design Curves for Test Sites with Category E Details	49

INTRODUCTION

Background

KRS statute 177.977 provides for the legal operation of coal trucks in excess of normal legal maximum weight limits on roads designated as extended weight coal-haul routes. The maximum allowable weights for extended weight permits range from 90,000 to 120,000 lbs, with a 5-percent tolerance.

The extended weights of coal trucks may subject bridges to significantly higher live-load stresses than sustained by bridges on normal routes. While those stresses may be within the design structural capacity of those bridges, they may impart cumulative damage that is not readily apparent. That damage may be manifested as fatigue cracking in welded steel bridges due to high service stresses. Fatigue cracks may disable a bridge or possibly cause structural collapse. Consequently, their occurrence is of major concern to Kentucky Transportation Cabinet (KyTC) officials.

The need existed to measure service stresses on welded steel bridges on coal-haul routes and to provide fatigue evaluations based on those stress measurements to determine whether structural members on those bridges were prone to fatigue cracking.

Welding and Fatigue of Steel Bridges

Welded fabrication used on modern steel bridges presents several factors impacting the potential for and severity of fatigue crack problems. First, welding processes may impart small flaws in structural members that are not detected during fabrication inspection. Such flaws may eventually nucleate into growing fatigue cracks. Second, weldments may act as geometric stress risers and promote the fatigue crack nucleation process. Third, welded structural members are typically monolithic and fatigue may cause them to fracture.

The first factor cannot be anticipated from a crack initiation standpoint. However, such cracks must be detected by maintenance inspections. Bridge inspection procedures employed by the KyTC have proven effective in detecting fatigue cracks prior to the onset of severe problems. The second factor is probably the most common encountered in bridges and may be anticipated if the stress history of a structural member is known. That allows KyTC officials to adopt a proactive posture in addressing potential fatigue cracking situations. The third factor emphasizes the potentially catastrophic consequences of fatigue cracking if undetected prior to complete fracture of a structural member. That is the primary reason that detection of such cracks is a major focus and concern of KyTC officials.

Initiation and growth of fatigue cracks in welded steel bridges are primarily affected by the magnitude and the frequency of the live loads applied by vehicles using the structures -- primarily truck traffic. Live loads impart alternating stresses in structural members. The range of those cyclic stresses (the magnitude from the minimum stress to the maximum stress) and the rate of live-load application (as it impacts the total

number of stress cycles imposed on a structural member) are primary variables affecting both the initiation and growth of fatigue cracks. Welded structural details which are stress raisers interact with those alternating stresses to promote fatigue crack initiation. Once fatigue cracks initiate, they grow away from those details. The subsequent fatigue crack growth is affected by magnitude and frequency of the stress cycling.

The subject of fatigue cracking of steel bridges, test procedures and equipment used for conducting field tests, and data analyses procedures, are discussed in more detail in an earlier report (1).

Initiation of Study

Kentucky Transportation Center (KTC) personnel prepared a research study work plan to address the concerns of KyTC officials relative to potential fatigue problems on welded steel bridges on extended weight coal-haul routes. The study objectives were:

- 1) To identify and select steel bridges on coal-haul routes that have fatigue-prone welded structural details.
- 2) To strain gage fatigue-prone structural details and monitor them to determine the frequency and magnitude of cyclic stresses.
- 3) To apply analytical procedures to the strain-gage data to deduce whether those welded structural details are prone to fatigue cracking.
- 4) To provide recommendations for follow-up actions by KyTC officials to deal with potential fatigue problems identified by the analyses.
- 5) To determine if extended weight limits on coal-haul routes provide a higher potential of fatigue cracking on welded steel bridges than exists on similar bridges not on coal-haul routes.

The study work plan was approved by KyTC and Federal Highway Administration (FHWA) officials in 1990.

PRELIMINARY WORK

The initial effort was to identify coal-haul routes that carried significant extended weight truck traffic. The KyTC Division of Planning maintains a database containing the county number, route number, beginning and ending mile points, and the tonnage of coal hauled in both the cardinal and non-cardinal directions of extended weight coal-haul routes.

Extended weight coal-haul routes in coal-producing and coal-impact counties are classified and compiled in the annual **Directory of Official Coal Haul Highway System** published by KyTC (2). The extended weight coal-haul routes selected for this study consist of state maintained roads that carry coal tonnages in excess of 50,000 tons

per year. Review of data for those routes indicates that coal tonnages transported sometimes varies significantly between succeeding years. That is apparently based on economic conditions and other factors. KyTC Division of Planning officials provided KTC personnel with detailed data files on the weight of coal transported in 1989 and 1990 for all extended weight coal-haul routes.

Information in the directory assisted KTC researchers in identifying welded steel bridges that might be prone to fatigue damage based upon annual tonnages of coal transported on those routes. The data files allowed determination of which direction on a route sustained the highest traffic from loaded coal trucks (i.e. the heavy-haul direction). That allowed KTC researchers to identify which bridge members to instrument.

The first two bridges tested during the study were selected by KyTC Study Advisory Committee members. Thereafter, bridges incorporated in the study were based upon recommendations of KyTC district personnel. The Study Advisory Committee contacted personnel in districts with routes having annual coal-hauling tonnages exceeding 50,000 tons requesting that the District Engineers identify specific bridges on those routes as candidates for stress measurements. The Study Advisory Committee reviewed the list and identified 17 bridges for inclusion in the study. Additionally, the Study Advisory Committee recommended that an interstate bridge be included in the program for comparative purposes. An interstate bridge (I64-I75) in Fayette County was selected due to its high traffic volume.

Bridges tested during this study including routes, locations and structural descriptions are listed in Table 1. All bridges selected were multi-span, continuous deck girder (or I-beam) structures. Several were two-girder (i.e. structurally non-redundant) bridges. Those bridges were of special concern since they might collapse if fatigue fractured a girder.

The Study Advisory Committee provided design drawings of the test bridges. KTC researchers reviewed those drawings to identify fatigue-prone weld details (i.e. those details that were significant stress raisers). Weld details corresponding to American Association of State Highway Transportation Officials (AASHTO) Category D or higher were sought.

On bridges containing fatigue-prone details, AASHTO Category E fatigue details were commonly encountered. Those were horizontal stiffener and gusset plate terminations welded to girder webs (Figure 1). Nine test bridges did not contain fatigue-prone weld details. By the mid-to-late 1970s, the fatigue performance of weld details was widely known. Many designers were avoiding the use of the more fatigue-prone types.

Test bridges containing fatigue-prone weld details sometimes possessed many of those details throughout the structure. The test instruments used during this study could only monitor four discrete connections. To achieve the most significant results with that limitation, reviews of coal-haul data and design drawings were necessary to limit tests to specific weld details considered to be subject to the greatest live-load tensile stresses.

Typically, weld details on girders supporting the travel lane in the heavy-haul directions were considered for monitoring. Those selected were commonly located on the lower portions of girders at midspan of the longest spans or on negative moment areas over piers adjacent to the longest spans. Locations for strain measurements for bridges tested in the study are listed in Table 2.

If no fatigue-prone details were present, test locations typically selected were where the greatest live-load tensile stresses were anticipated. On several bridges, strain gages were placed on flange transition welds. Fatigue analyses were not performed for those locations.

FIELD INSTRUMENTATION

Strain (i.e. stress) measurements were conducted by attaching foil strain gages to test sites on a structure. The strain gages were wired to signal conditioners/data logger instruments. Those instruments measured changes in strain induced by all routine traffic over the bridges including coal trucks.

The signal conditioners/data loggers provide an electronic carrier signal that is modulated by changes in strain gage resistance. Those changes in resistance are proportional to changes in strain in bridge members at locations where the gages are attached. The instruments digitize those analog strain gage signals in real time. The digitized data are then preprocessed "on the fly" by the instrument's microprocessors and the derived data are subsequently stored in non-volatile memory for future retrieval.

Preprocessing of variable amplitude strain data is performed using a standard procedure termed "rainflow counting." Basically, that procedure entails decomposition of the complex strain cycles into half cycles and counting the number of those that fall into preselected limits (i.e. bandwidths). The number of counts is converted to full cycles of constant amplitude and stored in "bins" that represent incremental increases in magnitudes of the constant-bandwidth strains.

Two signal conditioner/data loggers were employed for most of the bridge tests. Each instrument contained two independent channels for data acquisition and recording.

The signal conditioner/data loggers were battery-powered to provide for operation in remote locations. Those instruments were programmed by a PC to perform unattended strain gage monitoring (Figure 2). They were stored in tool boxes with batteries and placed on bridge piers during monitoring. After monitoring for a preselected period, typically 1-2 weeks, the instruments automatically shut down retaining the stored data. Thereafter, KTC researchers returned to the bridges, uploaded the stored data into a PC and retrieved the units for use elsewhere.

A KyTC truck-mounted snoopers was used to access remote bridge locations for strain gaging (Figure 3). Prior to monitoring routine traffic, the snoopers truck was driven over the bridge in the heavy-haul test direction and the resulting strains were obtained.

Those were used to ascertain that magnitudes of strains subsequently measured in the tests of routine traffic were reasonable in relation to strains that could be anticipated from coal-truck loads.

Early in the test program, problems were experienced with the signal conditioner/data loggers which did not function consistently. Eventually, the problem was identified and remedied by the manufacturer. Thereafter, the instruments functioned properly for the balance of the study.

Unfortunately, nearly six months of test data were lost due to that problem and the bridge test schedule was impacted unfavorably. In an attempt to complete the field work on time, the decision was made to reduce the field monitoring period from two weeks to one week. In cases where several test bridges were in close proximity, the number of test sites per structure was reduced to two and both bridges were tested concurrently using a single signal conditioner/data logger.

Several other operational problems arose near the end of the field testing.

One signal conditioner/data logger installed on the KY 3451 bridge over the Cumberland River in Harlan County was stolen during testing and never recovered. It could not be replaced in time for use in the study and subsequent inspections were limited to one 2-channel unit. Due to the potential for further vandalism, the decision was made not to re-test that bridge.

A second problem occurred during a test on the Daniel Boone Parkway bridge over KY 80 in Clay County. Strain gages were installed on the lower flange of girder 1 at midspan and on the web of girder 2 at midspan approximately 6 inches above the lower flange. Both test sites were sited under the traffic lane portion of the deck in the heavy-haul direction. At the time of gage installation, the lower flanges of both girders were found to be deformed due to previous impacts with over-height trucks using KY 80.

KTC researchers hoped that bridge impacts would not occur during the week-long monitoring period. When KTC researchers returned to retrieve the data and instruments, they found that both girders had been struck by traffic at or near the test sites. The gage on girder 1 was destroyed. The data from the strain gage mounted on girder 2 indicated that it was also rendered inoperable. Due to the likelihood that the beams would be impacted during another test, the decision was made not to re-test that bridge.

Sixteen bridges were tested successfully during the study. Due to the early problems experienced with the signal conditioner/data loggers, the timetable to complete all the desired tests became very tight. In the remaining duration of the study, the KyTC snooper truck became unavailable for a significant period. That coupled with the loss of one test instrument prevented testing of the remaining two coal-haul route bridges. However, the interstate bridge was monitored as the last structure in the test program. Several coal-haul bridges were tested twice providing a total of 18 successful field tests.

The resulting data obtained from the field tests support the conclusions of KTC researchers.

DATA ANALYSES AND RESULTS

Stress Histograms

The strain data retrieved from the signal conditioner/data loggers may be converted to stresses by multiplying by the modulus of elasticity for steel (29×10^6 psi). The resulting data can be represented as a stress histogram. The vertical axis of the histogram indicates the number of times (counts) that a particular stress range was measured on the bridge. The horizontal axis represents stress ranges preselected for counting. In this study, the stress ranges selected varied by 0.5 ksi increments.

The stress histogram represents the spectrum of variable amplitude stresses caused by normal traffic loads. The histograms provide the distribution of live stresses (i.e. the number and magnitudes of the live stresses). Those data may be reviewed to determine the suitability of live-load stress design assumptions. Where applicable, these data can be readily manipulated for fatigue analyses. Stress histograms from the field tests are provided in Appendix A.

The snoop truck drive-over stresses were used to provide guidance as to what stress ranges could be considered reasonable. After several tests, it was determined that a live stress of about 15 ksi would represent the highest value that could be anticipated for a heavily loaded coal truck. Therefore, a live stress range with a midwidth of 15.25 ksi was employed as the maximum limiting value for all data. Live stresses above that level were not considered.

At most test sites, few, if any cycles were recorded at stresses above 10 ksi. Some of those stresses may have resulted from permit-type overloads related to transport of extremely heavy coal handling equipment. Such loads were occasionally observed during the field work. It is also possible that some of the high amplitude stresses may have been false indications generated by random electrical noise. Due to the infrequent occurrence of high-amplitude noise-related signals, such events would not significantly impact the resulting analyses.

As is common practice in variable amplitude stress cycle counting, low amplitude stresses such as those generated by automobile traffic are not incorporated in subsequent analyses. For these tests, stresses below 0.5 ksi were not considered.

In reviewing some test data, it is likely that electrical noise may have contributed some "false calls" related to stress cycle measurements. Various measures were employed to limit such events. However, any false stress cycles generated by noise did not suppress valid data. Their possible inclusion in subsequent analyses did not impact the primary study conclusions.

Fatigue Analyses

To determine the potential for fatigue problems, the derived stress histogram data must be converted into a single-valued constant-amplitude stress range equivalent in impact (damage) to all the variable-amplitude live-load stresses measured. That can be accomplished using the Miner equivalent stress equation:

$$S_{re(Miner)} = (\sum P_i * S_{ri}^3)^{1/3}$$

where

$S_{re(Miner)}$ = the equivalent stress range

P_i = the proportion of stress cycles for S_{ri} , and

S_{ri} = the preselected stress range or midwidth of the i^{th} interval.

The resolved single-valued stress range, $S_{re(Miner)}$, represents an equivalent constant-amplitude stress for the complete data set that will result in the same amount of fatigue damage when applied in place of the variable-amplitude stress range spectrum as indicated by the stress histogram.

The resulting $S_{re(Miner)}$ stress and total number of cycles measured over the monitoring period can be used for fatigue analyses. Fatigue analyses can be employed to determine whether accumulated fatigue damage poses a failure risk to a structure (or structural member). If that is not the case, the structure is in a "safe-life" condition.

The accumulated damage may be determined by assuming that the $S_{re(Miner)}$ and number of cycles measured by field monitoring are representative of conditions over the life of a bridge. The accumulated number of stress cycles may be calculated by multiplying the age of a bridge by the frequency of cyclic stressing as determined by the field data and monitoring period. The accumulated damage to the structural member can be determined by superimposing the $S_{re(Miner)}$ stress and the accumulated cycles on the appropriate AASHTO fatigue design curve for the weld detail monitored.

In the KTC analyses, $S_{re(Miner)}$ values derived from the field data were considered invariant. More uncertainty existed relative to the number of stress cycles experienced over extended time periods such as the current life of a bridge. In the fatigue analyses, damage points were determined using three methods to measure long-term accumulation of stress cycles. Those methods included: 1) a 5 percent cyclic growth rate over the present age of the bridge, 2) a constant cyclic rate over the present age of the bridge and 3) a variable cyclic rate over the present age of the bridge based upon regression analysis of traffic (ADTs). The worst-case method for each bridge was also projected over an anticipated 75-year service life. AASHTO fatigue design curves showing fatigue analyses for bridge locations with applicable weld details (i.e. Category E) are provided in Appendix B. Listings of the test locations, weld details, $S_{re(Miner)}$ stresses, annual number of stress cycles (based on test data) and percent of fatigue lives (based upon worst case

analyses) at projected structure service lives of 75 years are provided in Table 2. Analyses of lower AASHTO fatigue categories proved insignificant. That may be observed in Table 2 by the low percentages of fatigue life projected to be expended over the 75-year service life of category C details on the Daniel Boone Parkway bridge over Big Creek and KY 80 in Perry Co (B-81).

Accumulated damage points lying below the fatigue limit for a specific weld detail typically indicate that a structure is in a safe-life condition at the anticipated level of accumulated damage. That applies in this study because test locations used in this study are the "worst-case" examples where the highest live-load tensile stresses would occur. If accumulated damage was above that limit, the potential would exist for fatigue cracking.

Fatigue data analyses indicated that all of the fatigue-prone weld connections monitored during this study were in the safe-life condition, not only over the current age of the bridges, but also over the projected 75-year service lives. Since the average age of the bridges tested is about 21 years, that indicates crack initiation from fatigue-prone weld details is unlikely.

While $S_{re(Miner)}$ stresses were above the fatigue limits at several test sites, the number of traffic-induced stress cycles estimated to have been sustained or projected over the anticipated life of the bridges were (or will be) insufficient to pose fatigue problems. The stresses measured on the interstate bridge and the coal-haul bridges differ primarily in the number of annual stress cycles. The $S_{re(Miner)}$ values of 1.510 ksi and 1.379 ksi from the I64-I75 bridge in Fayette County lie within the range of similar values encountered on the coal-haul route bridges. However, the annual number of stress cycles experienced by the interstate bridge exceed most of those experienced by the coal-haul route bridges.

Cracks in Structural Members

Three structures tested were observed to have existing cracks. Those were the KY 80 bridge over the Russell Fork of the Big Sandy River in Pike Co., the US 23 and 460 bridge over Levisa Fork in Johnson Co. and the KY 15 bridge over Carr Fork in Knott Co.

The KY 80 bridge is a two-girder structure that had experienced out-of-plane cracking in the girder webs. The cracks emanated from corners between the webs, upper flanges and floorbeam attachment gussets welded to the webs. Most cracks ran horizontally along the flange to web fillet welds. Vertical cracks also ran downward along the gusset to web welds. That type of cracking is usually not serious since crack growth is very stable and may become autoextinguishing once a certain amount of crack growth is obtained. Some cracked areas had been previously retrofitted by placing check holes ahead of the crack tips. Not all of those locations had cracked.

The instruments were not capable of measuring stresses related to out-of-plane cracking. However, the vertical cracks were of interest. It was decided to orient the strain gages

horizontally along the girder web adjacent to the gusset attaching the floorbeam to measure the tensile component of forces that might initiate or drive those cracks (Figure 4).

Strain gages were originally positioned in the westbound lane indicated to be the heavy-haul direction by the 1990 coal-haul data. During strain gage installation, loaded coal trucks, commonly the single-unit three axle types were travelling in the eastbound direction. Only empty trucks were observed travelling westbound. Based upon that observation, the decision was made to repeat the tests in mirror locations in the eastbound direction. The $S_{re(Miner)}$ and annual number of stress cycles for eight test sites are provided in Table 2. Higher $S_{re(Miner)}$ values and significantly higher numbers of stress cycles were measured in the eastbound direction.

The US 23 and 460 bridge was also a two-girder structure that had experienced cracking in a gusset plate that connected lateral x-bracing to the lower flange of haunched girders over the piers. The branched cracking pattern in the gusset indicated that complex stresses were involved.

An identical uncracked gusset plate on the opposite girder at the other pier supporting a haunched portion of the continuous girder span was instrumented with two gages mounted along the bottom face of the gusset plates, one aligned longitudinally with and the other transverse to, the axis of the lateral x-bracing. The signal conditioner/data logger wired to those gages was programmed to record "peak-valley" data to determine whether the forces acting on the gusset were primarily tensile or compressive. Only tensile strains were detected. Conventional data processing was also used for the gages. Subsequent inspection of the stress histogram data from the longitudinal gage did not reveal abnormal stresses though stresses in excess of 10 ksi were measured by the transverse gage. However, the longitudinal gage measured many more stress cycles.

The KY 15 bridge was observed to have a transverse crack in the fillet weld connecting the lower flange to the web in the suspended span. The crack was about 1-foot long. It was located about 5 feet from the end of the suspended span between two vertical stiffeners. The lower flange was instrumented with two gages placed on the lower flange near the web. One was oriented longitudinally with the other transverse to the girder. Ensuing data indicated only tensile stresses. The effect of gage orientation in terms of magnitudes and frequency of live stresses was similar to that for gages installed on the gusset plate of the US 23 and 460 bridge.

The cracking observed on the three test bridges is either due to unusual structural responses to loading or, in the case of the KY 15 bridge, possibly to a fabrication defect. The types of cracking observed were not the typical Mode I fatigue cracks that were the subject of this study. While stress data were obtained for those situations, it is presented for review by KyTC officials. It is of interest that all three bridges were two-girder structures with floorbeams and stringers.

CONCLUSIONS

The field data and subsequent data analyses indicate that fatigue cracking due to extended-weight coal trucks is not a potential concern for the bridges that were monitored. Since several number of bridges were tested, it is likely that this finding is applicable to a majority of the continuous girder (or I-beam) welded steel bridges on extended weight coal-haul routes. A primary reason for that finding is that those bridges are not subject to extremely high coal-truck traffic volumes, despite the occasional heavy coal-haul loads they sustain. That may be discerned by comparing equivalent stresses and number of cycles per year between the extended weight coal-haul route and the interstate bridges.

Since conventional fatigue problems were not indicated on the test bridges, fracture mechanics/nondestructive testing analyses were not required. It was observed that only AASHTO fatigue category E (or higher) details warranted consideration for fatigue problems. For the continuous girder welded steel bridges, "arms-length" or nondestructive inspections could be concentrated on such details when searching for fatigue cracks.

The focus of this study was limited to the potential for fatigue cracking on bridges. The study was not intended to address other issues related to the effects of the extended weight coal-haul system. While this study did not address other types of bridge deterioration that might be related to extended weight coal-truck traffic, it was observed that four of the bridges tested had noticeable deck deterioration. That deterioration was either extensive transverse cracking or patching of deck spalling. Other test bridges appeared to have been previously overlaid masking possible earlier damage by extended weight coal-truck traffic. The effect of extended weight coal-truck traffic may merit further and more detailed examination.

During this study, field strain gaging techniques were employed for measurement of traffic-induced live stresses. They have proven useful to address and, hopefully, resolve concerns of KyTC officials. Strain gaging procedures employed during this study are useful for addressing many structural problems where fatigue cracking may be a problem. In the future, it is anticipated that those techniques will be utilized on a much more widespread basis by KyTC and other state highway agencies.

BIBLIOGRAPHY

1. Hopwood, T.; Hogan, K.J.; and Oberst, C.M., "Stress Evaluation of Welded Steel Bridges on Coal-Haul Routes," Kentucky Transportation Center, Report No. KTC-93-14, May 1993.
2. **Kentucky's Official Coal Haul Highway System**, Division of Planning, Department of Highways, Kentucky Transportation Cabinet, Frankfort, KY, August 1990, p.122.

Table 1. Summary of Test Bridges -- Locations and Descriptions

Bridge Location	Year Completed	Bridge Description	Type of Steel	Roadway Width	Girders/Spacing
KY 11 over Licking River, Bath-Fleming Co. (B-70)	1977	132'-145'-132' Continuous Welded Plate Girder Spans, 2° Curve, 9°30'Skew	ASTM A 36	50'	6-72" Girders @ 9'-4" w/ 3'-5" Cantilever (Non-composite Action)
Daniel Boone Parkway over L&N R.R. and Goose Creek, Clay Co. (B-73)	1969	140'-140' Continuous Welded Plate Girder Spans, & 62'-3", 54'-0" and 92' I-Beam Spans	ASTM A 36	44'	6-78" Girders @ 8'-2" w/ 3'-4" Cantilever (Non-composite Action)
Hazard By-Pass over L&N R.R., Perry Co. (B-105)	1982	146'-146' Continuous Welded Plate Girder Spans, 6° Curve, 43°30' Skew	ASTM A 36	88'	10-78" Girders @ 9'-4" w/ 3'-8" Cantilever (Non-composite Action)
Daniel Boone Parkway over Big Creek and KY 80, Perry Co. (B-87)	1972	120'-170'-170'-120' Continuous Welded Plate Girder Spans, 35° Skew	ASTM A 36	50'	6-72" Girders @ 9'-4" w/ 3'-5" Cantilever (Non-composite Action)
Daniel Boone Parkway over North Fork of Kentucky River, KY 80 and L&N R.R., Perry Co. (B-89)	1973	70'-100'-185'-185'-100' Continuous Welded Plate Girder Spans, 2° Curve, 0° Skew	ASTM A 36 A 441	32'	4-84" Girders @ 9'-4" w/ variable Cantilever (3'-9" max.) (Non-composite Action)
KY 15 over North Fork of Kentucky River and KY 80, Perry Co. (B-55)	1965	140'-178'-140' Continuous Welded Plate Girder Spans, 30° Skew	ASTM A 373	57' (Including a 3'-0" Curb)	8-84" Girders @ 9'-0" w/ 4' Cantilever (Non-composite Action)

Table 1. Summary of Test Bridges (Cont.)

Bridge Location	Year Completed	Bridge Description	Type of Steel	Roadway Width	Girders/Spacing
Daniel Boone Parkway over Red Bird River, Perry Co. (B-82)	1974	58'-84'-109'-109'-109'-84'-58' Continuous Welded Plate Girder Spans, 45° Skew	ASTM A 36	32'	5-48" Girders @ 7'-6" w/ 2'-9" Cantilever (Non-composite Action)
US 119 over Poor Fork of Cumberland River, Harlan Co. (B-116)	1974	75'-110'-75' Continuous Welded Plate Girder Spans, 45° Skew	ASTM A 36	44'	6-52" Girders @ 8'-3" w/ 3'-1½" Cantilever (Non-composite Action)
US 421 over Clover Fork and L&N R.R., Harlan Co. (B-131)	1974	92'-131'-137'-120'-80' Continuous Welded Plate Girder Spans, 40° Skew	ASTM A 36	2-29'-9" (Plus a 16' Median)	Each Roadway is Supported by 5 Haunched Girders (4'-6" Min. Depth) @ 8'-3" w/ 2'-9" Cantilever at the Medians and w/ 3'-9" Cantilever at the Plinths (Non-composite Action)
US 23 and 460 over Levisa Fork, Johnson Co. (B-41)	1962	135'-225'-135' Continuous Welded Plate Girder Spans, 50' & 87' I-Beam Spans	ASTM A 373	30'	Roadway is Supported by 2 Haunched Girders (8' Min. Depth) @ 28' with 3 I-Beam Stringers @ 7' w/ 4'-6" Cantilever (Non-composite Action)
Hazard By-Pass over KY 451, L&N R.R. and Messer Branch, Perry Co. (B-104)	1982	80'-115'-150'-115'-80' Continuous Welded Plate Girder Spans, 4° Curve	ASTM A 36	88'	10-5' Girders @ 9'-4" w/ 3'-9" Cantilever (Non-composite Action)

Table 1. Summary of Test Bridges (Cont.)

Bridge Location	Year Completed	Bridge Description	Type of Steel	Roadway Width	Girders/Spacing
US 421 over L&N R.R. and Clover Fork, Harlan Co. (B-132)	1974	60'-80'-80'-80' Continuous Welded Plate Girder Spans, 80' Simple Span, 100'-100'-140'-140' Continuous Welded Plate Girder Spans, 0° & 26° Skew	ASTM A 36	2-24' (Plus a 16' Median)	5-5' Girders @ 7'-7", 1-5' Center Girder @ 6'-2" on both sides, 5-5' Girders @ 7'-7" w/ 3' Cantilever (Non-composite Action)
KY 15 over Carr Fork, Knott Co. (B-56)	1966	195'-235'-195' Spans with 2-128' Welded Plate Girder Cantilever Spans and 1-169' Welded Plate Girder Plate Girder Suspended Span, 1° Curve	ASTM A 36	31'-6"	2-12' Girders @ 28' w/ 3 I-Beam Stringers @ 7' w/ Variable Cantilever (Non-composite Action)
US 119 over Poor Fork of Cumberland River, Harlan Co. (B-117)	1974	90'-110'-90' Welded Plate Girder Continuous Spans, 50° Skew	ASTM A 36	44'	6-52" Girders @ 8'-3" w/ 3'-1" Cantilever (Non-composite Action)
KY 80 over Russell Fork of Big Sandy River and CSX R.R., Pike Co. (B-137)	1968	150'-215'-150' Welded Plate Girder Continuous Spans, 115'-115' Welded Plate Girder Simple Spans	AASHTO M-183	30'	2-7' Girders @ 28'-4" w/ 3 I-Beam Stringers @ 7'-1" w/ 3'-2" Cantilever (Non-composite Action)
I64-I75 over US 27, L&N R.R. and N. Limestone St., Fayette Co. (B-83)	1964, 1978 (Widened)	67'-94'-94'-64' Continuous I-Beam Spans, 50'-70'-50' Continuous I-Beam Spans, 4° Skew	ASTM A 36	2-36' (Plus a 36' Median)	2 Roadways Each Supported by 9-36" I-Beams @ Var. Spacings 5'-9" to 8', w/ Variable Cantilever (2' Max.) (Composite Action)

Table 2. Summary of Bridge Test Results -- Test Locations, Results and Percent of Fatigue Life

Bridge Location	Test Duration	Strain Gage Locations and (Gage Nos.)	Fatigue Details Monitored (AASHTO)	Test Results (Equivalent Stresses and No. of Cycles)	% Fatigue Life (75 yrs)
KY 11 over Licking River, Fleming-Bath Co. (B-70)	Two Weeks	Span 1 (132'); Bottom Face of Lower Flange of Girder 3 - 53' from Bearing on Pier 1 (1); Span 2 (145'); Bottom Face of Upper Flange of Girder 2 - 1' from Bearing on Pier 1 (2); Bottom Face of Upper Flange of Girder 3 - 1' from Bearing on Pier 1 (3); Bottom Face of Lower Flange of Girder 3 at Midspan (4)		(1) Sre = 1.493 ksi N = 24,336 cycles/yr (2) Sre = 1.256 ksi N = 624 cycles/yr (3) Sre = 2.071 ksi N = 260 cycles/yr (4) Sre = 1.707 ksi N = 122,980 cycles/yr	

Table 2. Summary of Bridge Test Results (Cont.)

Bridge Location	Test Duration	Strain Gage Locations and (Gage Nos.)	Fatigue Details Monitored (AASHTO)	Test Results (Equivalent Stresses and No. of Cycles)	% Fatigue Life (75 yrs)
Daniel Boone Parkway over L&N R.R. and Goose Creek, Clay Co. (B-73)	Two Weeks	Span 2 (140'); Lower Portion of Web of Girder 2 near Midspan adjacent to Gusset (1); Bottom Face of Top Flange of Girder 2 - 2' from Bearing on Pier 1 (2); Bottom Face of Lower Flange of Girder 2 - 56' from Bearing on Pier 1 (3); Bottom Face of Upper Flange of Girder 2 Adjacent to Transition Weld - 38' from Bearing on Pier 1 (4)	Category E (1)	(1) Sre = 1.503 ksi N = 8,268 cycles/yr (2) Sre = 1.336 ksi N = 31,304 cycles/yr (3) Sre = 1.578 ksi N = 3,016 cycles/yr (4) Sre = No Data N = No Data	(1) = 0.21

Table 2. Summary of Bridge Test Results (Cont.)

Bridge Location	Test Duration	Strain Gage Locations and (Gage Nos.)	Fatigue Details Monitored (AASHTO)	Test Results (Equivalent Stresses and No. of Cycles)	% Fatigue Life (75 yrs)
Hazard By-Pass over L&N R.R., Perry Co. (B-105)	One Day	Span 2 (146'); Bottom Face of Upper Flange at Transition Weld on Girder 7 - 9' from Bearing on Pier 1 (1); Bottom Face of Upper Flange at Transition Weld on Girder 8 - 9' from Bearing on Pier 1 (2); Web below Transition Weld on Girder 8 - 9' from Bearing on Pier 1 (3); Bottom Face of Upper Flange at Transition Weld on Girder 9 - 9' from Bearing on Pier 1 (4)		(1) Sre = 1.803 ksi N = 3,650 cycles/yr (2) Sre = No Data N = No Data (3) Sre = 0.780 ksi N = 10,585 cycles/yr (4) Sre = 1.065 ksi N = 12,410 cycles/yr	

Table 2. Summary of Bridge Test Results (Cont.)

Bridge Location	Test Duration	Strain Gage Locations and (Gage Nos.)	Fatigue Details Monitored (AASHTO)	Test Results (Equivalent Stresses and No. of Cycles)	% Fatigue Life (75 yrs)
Daniel Boone Parkway over Big Creek and KY 80, Perry Co. (B-87)	One Week	Span 3 (169'); Web of Girder 1 near Upper Horizontal Stiffener over Pier 3 (1); Web of Girder 1 near Upper Horizontal Stiffener at Splice Plate - 30' from Pier 3 (2); Web of Girder 1 near Upper Horizontal Stiffener at Splice Plate - 30' from Pier 2 (3); Web of Girder 1 near Upper Horizontal Stiffener over Pier 4	Category E	(1) Sre = 4.196 ksi N = 10,972 cycles/yr (2) Sre = 6.771 ksi N = 2,340 cycles/yr (3) Sre = 3.783 ksi N = 45,812 cycles/yr (4) Sre = 4.628 ksi N = 208 cycles/yr	(1) = 19.33 (2) = 17.52 (3) = 62.37 (4) = 0.48
Daniel Boone Parkway over North Fork of Kentucky River, KY 80 and L&N R.R., Perry Co. (B-89)	One Week	Span 3 (185'); Web of Girder 2 near Upper Flange over Pier (1); Webs of Girders 1-3 near Lower Flange (2-4) at Midspan	Category C	(1) Sre = 1.078 ksi N = 14,820 cycles/yr (2) Sre = 1.852 ksi N = 147,004 cycles/yr (3) Sre = 1.664 ksi N = 599,300 cycles/yr (4) Sre = 1.466 ksi N = 303,628 cycles/yr	(1) = 0.47 (2) = 4.62 (3) = 18.82 (4) = 9.53

Table 2. Summary of Bridge Test Results (Cont.)

Bridge Location	Test Duration	Strain Gage Locations and (Gage Nos.)	Fatigue Details Monitored (AASHTO)	Test Results (Equivalent Stresses and No. of Cycles)	% Fatigue Life (75 yrs)
KY 15 over North Fork of Kentucky River and KY 80, Perry Co. (B-55)	One Week	Span 2 (178'); Web of Girder 7 at horizontal stiffener termination - 40' from Pier (1); Gusset Welded to Lower Flange of Girder 7 - 93' from Bearing on Pier 1 (2); Web of Girder 8 at horizontal stiffener termination - 40' from Pier (3); Gusset Welded to Lower Flange of Girder 8 - 79' from Bearing on Pier 1 (4)	Category E	(1) Sre = 3.774 ksi N = 12,428 cycles/yr (2) Sre = 2.872 ksi N = 58,604 cycles/yr (3) Sre = 2.011 ksi N = 80,756 cycles/yr (4) Sre = 2.104 ksi N = 287,092 cycles/yr	(1) = 12.01 (2) = 24.93 (3) = 10.10 (4) = 46.97

Table 2. Summary of Bridge Test Results (Cont.)

Bridge Location	Test Duration	Strain Gage Locations and (Gage Nos.)	Fatigue Details Monitored (AASHTO)	Test Results (Equivalent Stresses and No. of Cycles)	% Fatigue Life (75 yrs)
Ky 15 over North Fork of Kentucky River and KY 80, Perry C.. (B-55)	One Week	Span 2 (178'); Gusset Welded to Lower Flange of Girder 7 - 66' from Bearing on Pier 1 (1); Gusset Welded to Lower Flange of Girder 8 - 43' from Bearing on Pier 1 (2); Web of Girder 7 at horizontal stiffener termination - 40' from Pier 1 (3); Web of Girder 7 at horizontal stiffener termination - 40' from Pier 2 (4)	Category E	(1) Sre = 2.462 ksi N = 61,000 cycles/yr (2) Sre = 3.218 ksi N = 68,328 cycles/yr (3) Sre = 1.513 ksi N = 93,808 cycles/yr (4) Sre = 1.001 ksi N = 24,856 cycles/yr	(1) = 15.29 (2) = 41.52 (3) = 4.69 (4) = 0.53
Daniel Boone Parkway over Red Bird River, Perry Co. (B-82)	One Week	Span 3 (109'); Bottom Face of Lower Flange at Midspan of Girder 1 (1); Bottom Face of Lower Flange at Midspan of Girder 2 (2)		(1) Sre = 2.112 ksi N = 202,956 cycles/yr (2) Sre = 2.264 ksi N = 58,136 cycles/yr	

Table 2. Summary of Bridge Test Results (Cont.)

Bridge Location	Test Duration	Strain Gage Locations and (Gage Nos.)	Fatigue Details Monitored (AASHTO)	Test Results (Equivalent Stresses and No. of Cycles)	% Fatigue Life (75 yrs)
US 119 over Poor Fork of Cumberland River, Harlan Co. (B-116)	One Week	Span 2 (110'); Bottom Face of Lower Flange at Midspan of Girder 1 (1); Bottom Face of Lower Flange at Midspan of Girder 2 (2); Bottom Face of Lower Flange at Transition Butt Weld 33' from Bearing on Pier 1 (3); Bottom Face of Lower Flange at Midspan of Girder 3 (4)		(1) Sre = 0.846 ksi N = 11,192,948 cycles/yr (2) Sre = 2.441 ksi N = 52,936 cycles/yr (3) Sre = 1.637 ksi N = 13,549,900 cycles/yr (4) Sre = 1.128 ksi N = 2,223,520 cycles/yr	
US 421 over Clover Fork and L&N R.R., Harlan Co. (B-131)	One Week	Span 3 (137'); Lower Portion of Web of Girder 1 adjacent to Gusset Plate - 64' from Bearing on Pier 2 (1); Lower Portion of Web of Girder 2 adjacent to Gusset Plate -74' from Bearing on Pier (2)	Category E	(1) Sre = 1.319 ksi N = 948,064 cycles/yr (2) Sre = 1.378 ksi N = 402,584 cycles/yr	(1) = 42.60 (2) = 21.11

Table 2. Summary of Bridge Test Results (Cont.)

Bridge Location	Test Duration	Strain Gage Locations and (Gage Nos.)	Fatigue Details Monitored (AASHTO)	Test Results (Equivalent Stresses and No. of Cycles)	% Fatigue Life (75 yrs)
US 23 and 460 over Levisa Fork, Johnson Co. (B-41)	One Week	Span 4 (225'); Lower Portion of Web of Girder 1 adjacent to Horizontal Stiffener 36' from Bearing at Pier 3 (1); Web of Girder 1 at X-Bracing Gusset - 28' from Bearing at Pier 3 (2); X-Bracing Gusset at Lower Flange of Girder 1 at Bearing of Pier 3 - Longitudinal (3); X-Bracing Gusset at Lower Flange of Girder 1 at Bearing of Pier 3 - Transverse (4)	Category E (1) & (2)	(1) Sre = No Data N = No Data (2) Sre = 0.750 ksi N = 1,820 cycles/yr (3) See Stress Histogram (Appendix A) (4) See Stress Histogram (Appendix A)	(2) = 0.03

Table 2. Summary of Bridge Test Results (Cont.)

Bridge Location	Test Duration	Strain Gage Locations and (Gage Nos.)	Fatigue Details Monitored (AASHTO)	Test Results (Equivalent Stresses and No. of Cycles)	% Fatigue Life (75 yrs)
Hazard By-Pass over KY 451, L&N R.R. and Messer Branch, Perry Co. (B-104)	One Week	Span 2 (115'); Bottom Face of Lower Flange on Girder 7 - 52' from Bearing on Pier 1 (1); Bottom Face of Lower Flange on Girder 8 - 52' from Bearing on Pier 1 (2); Bottom Face of Lower Flange on Girder 9 - 52' from Bearing on Pier 1 (3); Bottom Face of Lower Flange on Girder 9 - 52' from Bearing on Pier 1 (4) Girder		(1) Sre = 0.852 ksi N = 4,246,320 cycles/yr (2) Sre = 1.691 ksi N = 46,332 cycles/yr (3) Sre = 1.467 ksi N = 993,928 cycles/yr (4) Sre = 1.016 ksi N = 20,176 cycles/yr	
US 421 over L&N R.R. and Clover Fork, Harlan Co. (B-132)	One Week	Span 8 (140'); Lower Portion of Web of Girder 1 adjacent to Gusset Plate - 70' from Bearing on Pier 7 (1); Lower Portion of Web of Girder 8 adjacent to Gusset Plate - 70' from Bearing on Pier 7 (2)	Category E	(1) Sre = 2.073 ksi N = 24,648 cycles/yr (2) Sre = 1.784 ksi N = 11,700 cycles/yr	(1) = 4.43 (2) = 1.47

Table 2. Summary of Bridge Test Results (Cont.)

Bridge Location	Test Duration	Strain Gage Locations and (Gage Nos.)	Fatigue Details Monitored (AASHTO)	Test Results (Equivalent Stresses and No. of Cycles)	% Fatigue Life (75 yrs)
KY 15 over Carr Fork, Knott Co. (B-56)	One Week	Span 2 (235'); Upper Face of Lower Flange of Girder 1 - 5' from End of Suspended Span, Longitudinal (1); Upper Face of Lower Flange of Girder 1 - 5' from End of Suspended Span, Transverse (2); Web of Girder 1 near Upper Flange - 30' from Bearing on Pier 2 (3); Span 3 (195'); Web of Girder 1 adjacent to Upper Horizontal Stiffener - 24' from Bearing on Pier 2 (4)	Category E (4)	(1) Sre = 2.415 ksi N = 25,532 cycles/yr (2) Sre = 2.966 ksi N = 832 cycles/yr (3) Sre = 4.026 ksi N = 10,816 cycles/yr (4) Sre = 0.786 ksi N = 473,772 cycles/yr	(4) = 3.36

Table 2. Summary of Bridge Test Results (Cont.)

Bridge Location	Test Duration	Strain Gage Locations and (Gage Nos.)	Fatigue Details Monitored (AASHTO)	Test Results (Equivalent Stresses and No. of Cycles)	% Fatigue Life (75 yrs)
US 119 over Poor Fork of Cumberland River, Harlan Co. (B-117)	One Week	Span 2 (110'); Bottom Face of Lower Flange of Girder 1 at Midspan (1); Bottom Face of Lower Flange of Girder 2 at Midspan (2); Bottom Face of Lower Flange of Girder 2 at Transition Weld - 40' from Bearing on Pier 1 (4)		(1) Sre = 2.634 ksi N = 334,308 cycles/yr (2) Sre = 0.897 ksi N = 4,228,900 cycles/yr (3) Sre = 1.122 ksi N = 6,208,436 cycles/yr (4) Sre = 2.329 ksi N = 135,876 cycles/yr	
KY 80 over Russell Fork of Big Sandy River and CSX R.R., Pike Co. (B-137)	One Week	Span 2 (215'); Web of Girder 1 Adjacent to East Face of Floorbeam 16 - 43' from Pier 2 (1); Web of Girder 1 Adjacent to West Face of Floorbeam 17 - 22' from Pier 2 (2); Span 3 (150'); Web of Girder 1 Adjacent to East Face of Floorbeam 19 - 22' from Pier 2 (3); Web of Girder 1 Adjacent to West Face of Floorbeam 20- 22' from Pier 2 (4)		(1) Sre = 1.229 ksi N = 48,880 cycles/yr (2) Sre = No Data N = No Data (3) Sre = 1.348 ksi N = 63,648 cycles/yr (4) Sre = 1.097 ksi N = 23,816 cycles/yr	

Table 2. Summary of Bridge Test Results (Cont.)

Bridge Location	Test Duration	Strain Gage Locations and (Gage Nos.)	Fatigue Details Monitored (AASHTO)	Test Results (Equivalent Stresses and No. of Cycles)	% Fatigue Life (75 yrs)
Ky 80 over Russell Fork of Big Sandy River and CSX R.R., Pike Co. (B-137)	One Week	Span 2 (215'); Web of Girder 2 Adjacent to East Face of Floorbeam 16 - 43' from Pier 2 (1); Web of Girder 2 Adjacent to West Face of Floorbeam 17 - 22' from Pier 2 (2); Span 3 (150'); Web of Girder 2 Adjacent to East Face of Floorbeam 19 - 22' from Pier 2 (3); Web of Girder 2 Adjacent to West Face of Floorbeam 20- 22' from Pier 2 (4)		(1) Sre = 1.614 ksi N = 944,648 cycles/yr (2) Sre = 1.328 ksi N = 1,171,820 cycles/yr (3) Sre = 1.046 ksi N = 394,888 cycles/yr (4) Sre = 2.423 ksi N = 359,112 cycles/yr	
I64-I75 over US 27, L&N R.R. and N. Limestone St., Fayette Co. (B-83)	One Week	Span 2 (97'); Bottom Face of Lower Flange of Girder 3 at Midspan (3); Bottom Face of Lower Flange of Girder 4 at Midspan (4)		(1) Sre = 1,510 ksi N = 1,363,584 cycles/yr (2) Sre = 1.379 ksi N = 1,518,140	

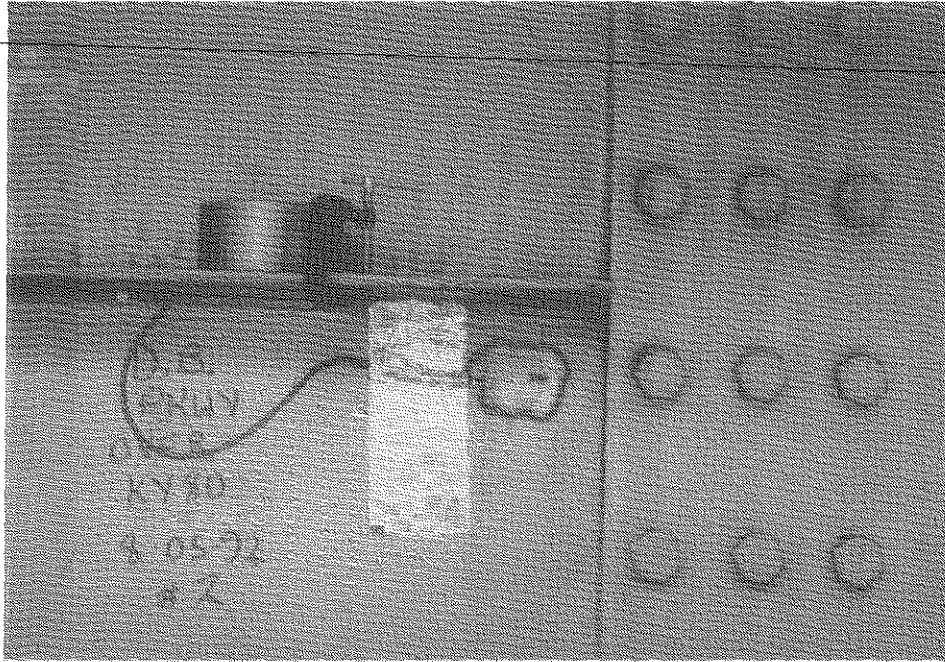


Figure 1. Strain Gage Installed Adjacent to Horizontal Stiffener on Daniel Boone Parkway Bridge over Big Creek and KY 80, Perry Co. (B-87).

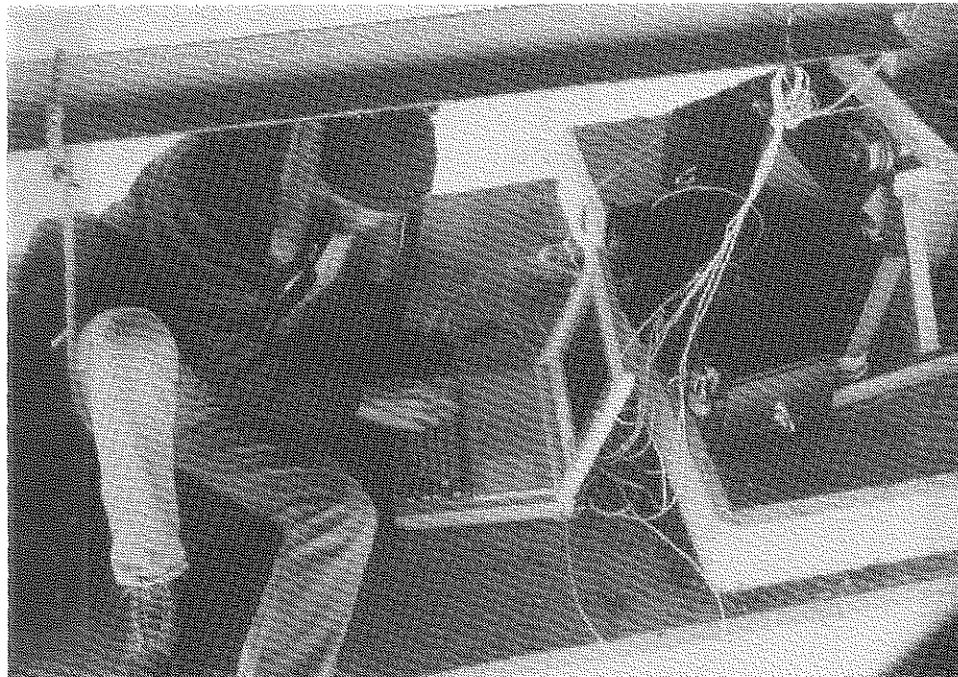


Figure 2. Technician Situated on Bridge Pier Programming Signal Conditioner/Data Logger with a Portable PC to Perform Unattended Strain Monitoring.

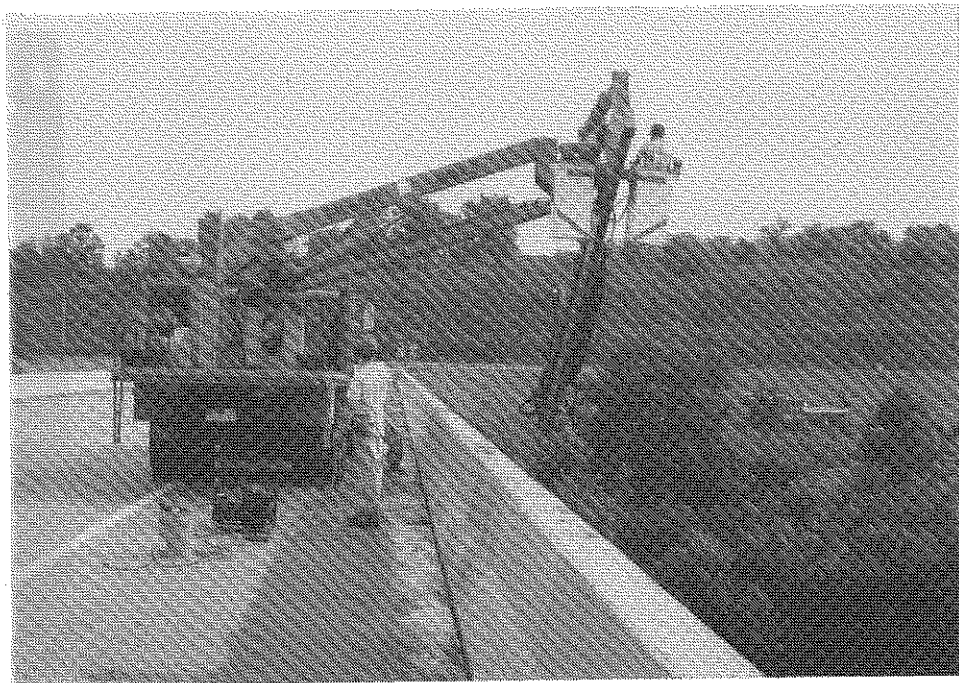


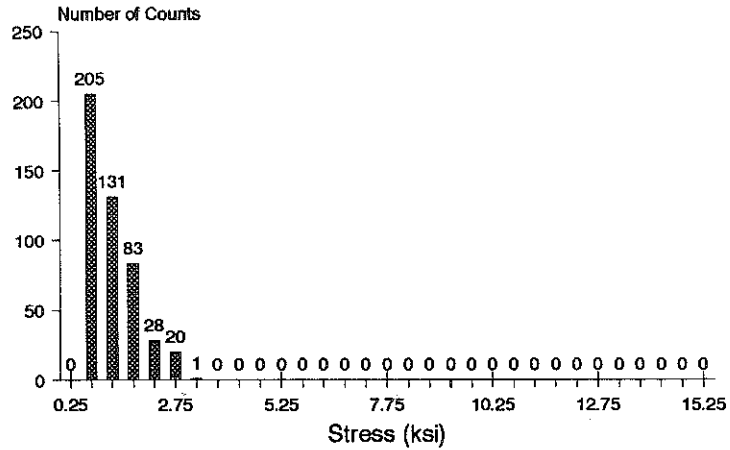
Figure 3. Snoop Truck Used to Access Remote Bridge Locations for Strain Gaging.



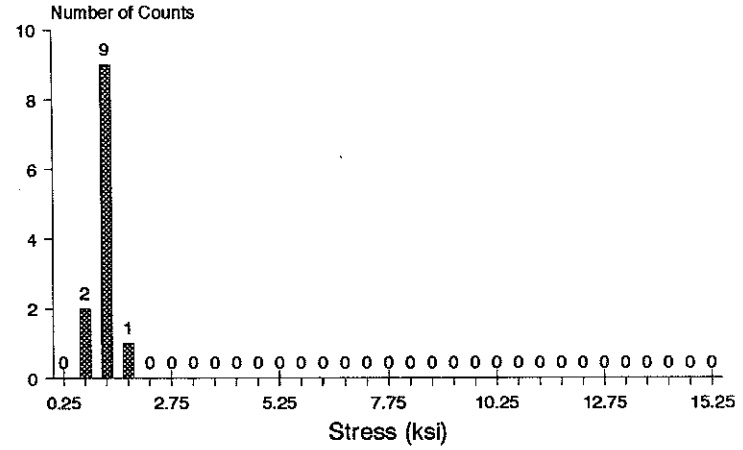
Figure 4. Strain Gage Located Adjacent to Floorbeam on KY 80 over Russell Fork of Big Sandy River and CSX R.R., Pike Co. (B-137).

APPENDIX A
(Stress Histograms for Bridge Test Sites)

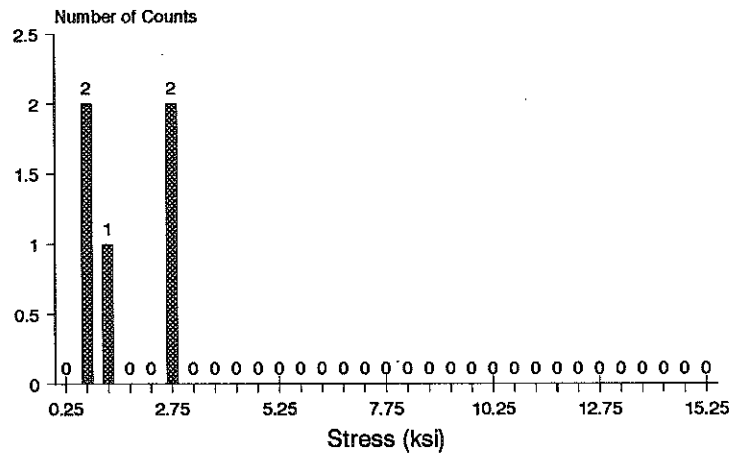
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

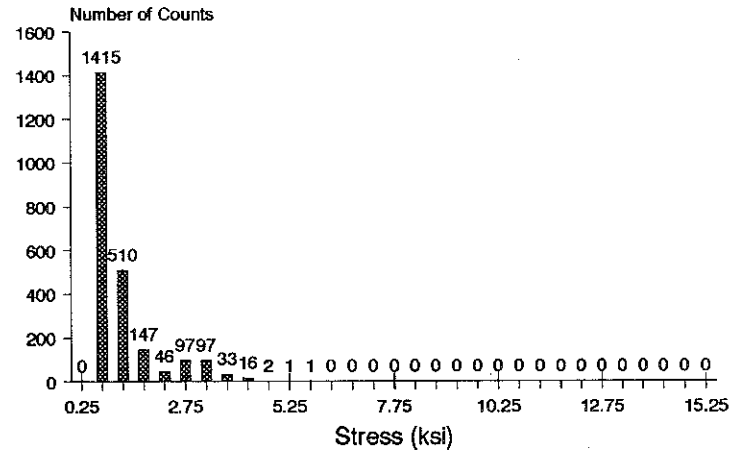
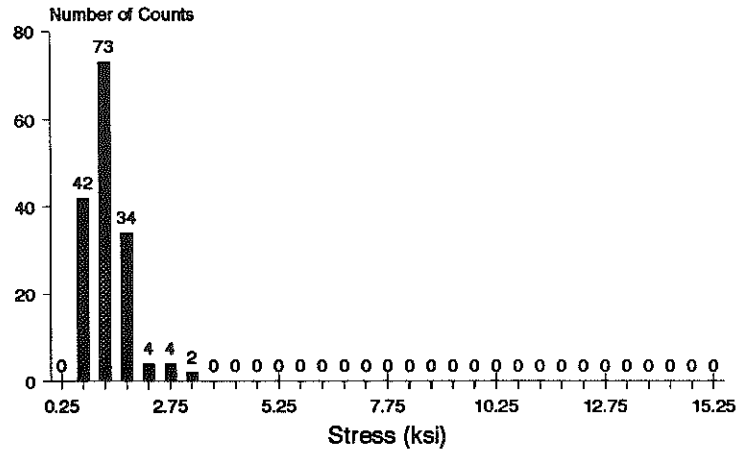
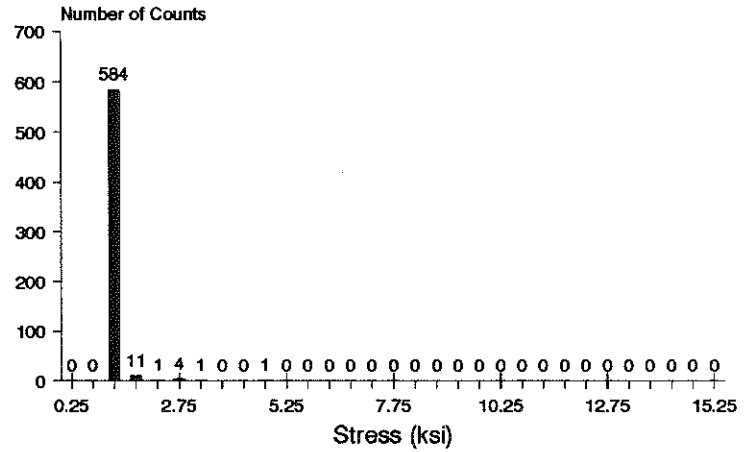


Figure 1. Stress Histograms for the KY 11 Bridge over Licking River, Bath-Fleming Co. (B-70).

Strain Gage #1



Strain Gage #2



Strain Gage #3

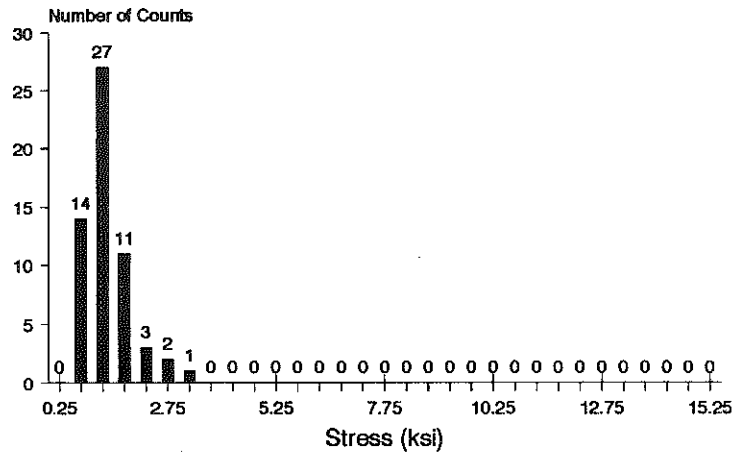
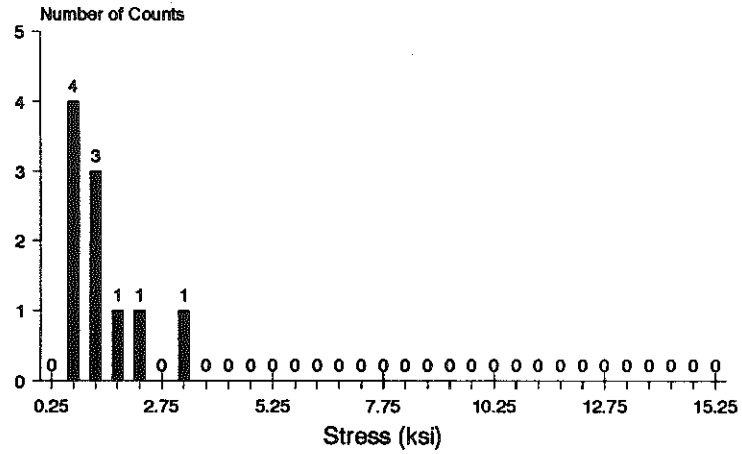


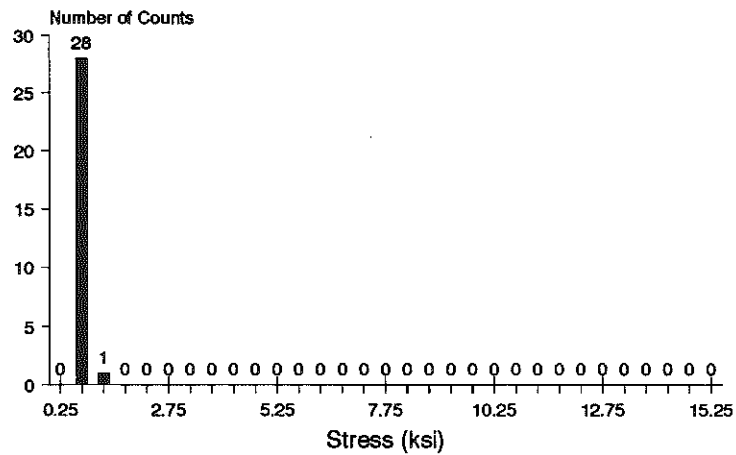
Figure 2. Stress Histograms for the Daniel Boone Parkway Bridge over L&N R.R. and Goose Creek, Clay Co. (B-73).

Strain Gage #1



33

Strain Gage #3



Strain Gage #4

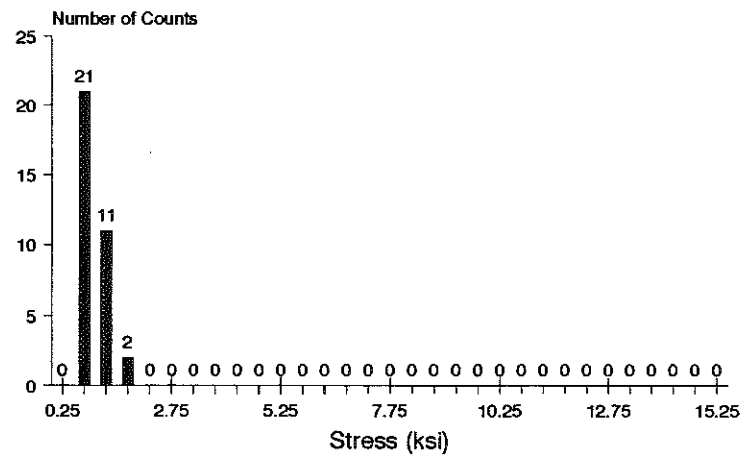
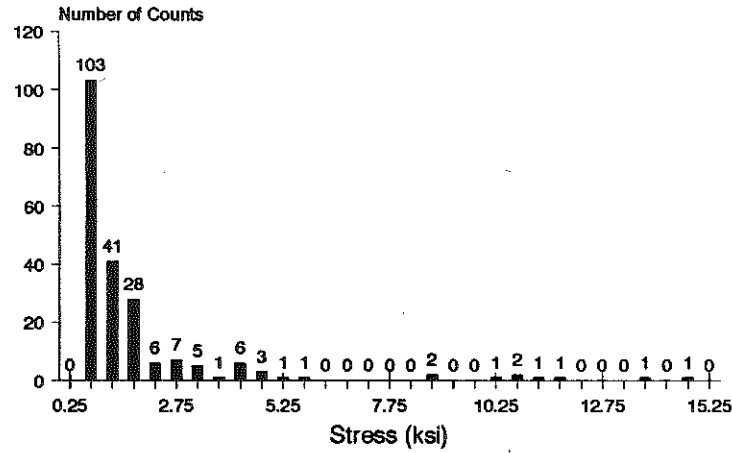
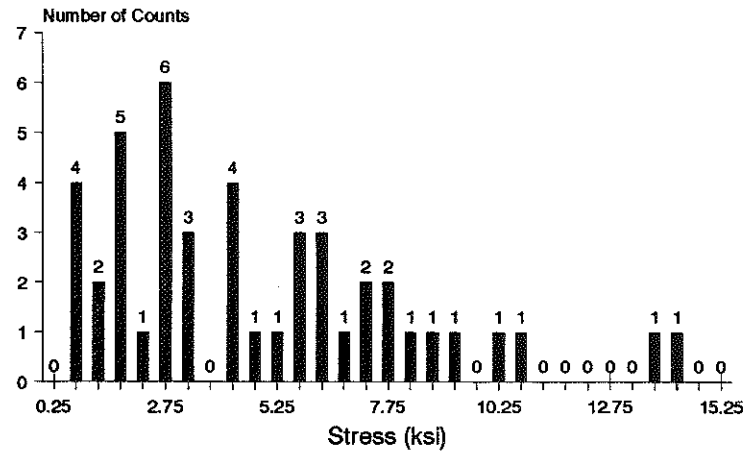


Figure 3 Stress Histograms for the Hazard By-Pass Bridge over L&N R.R., Perry Co. (B-105).

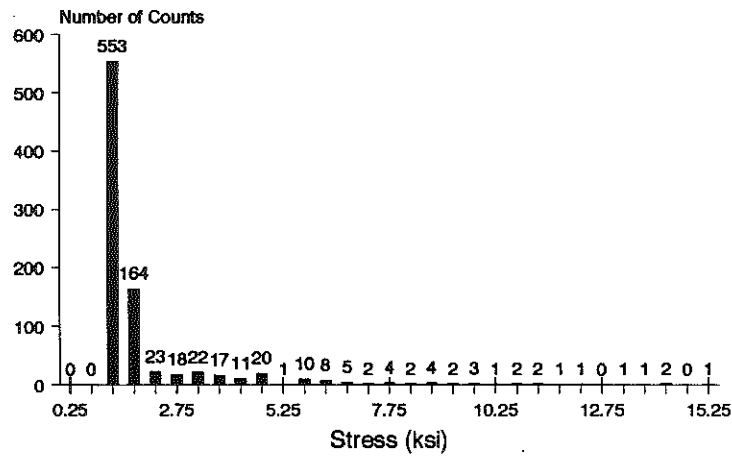
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

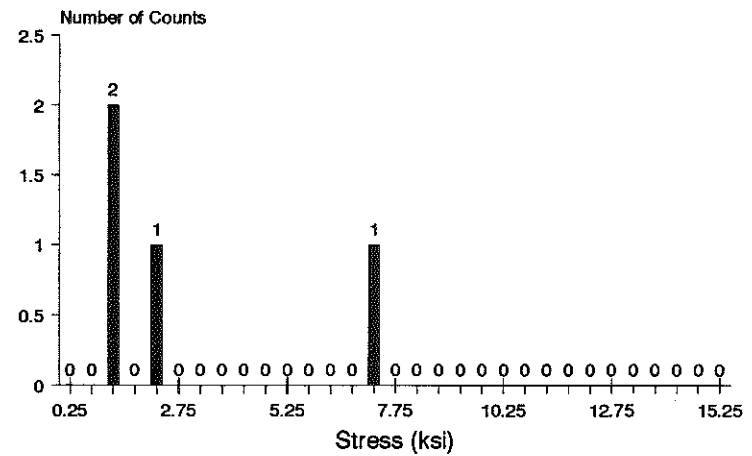
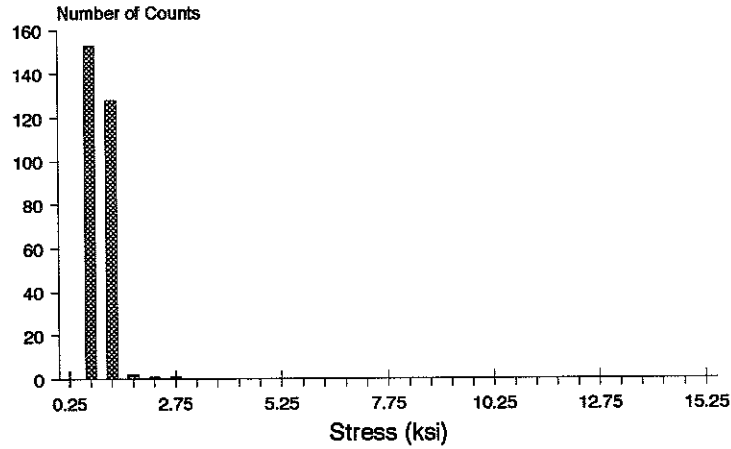
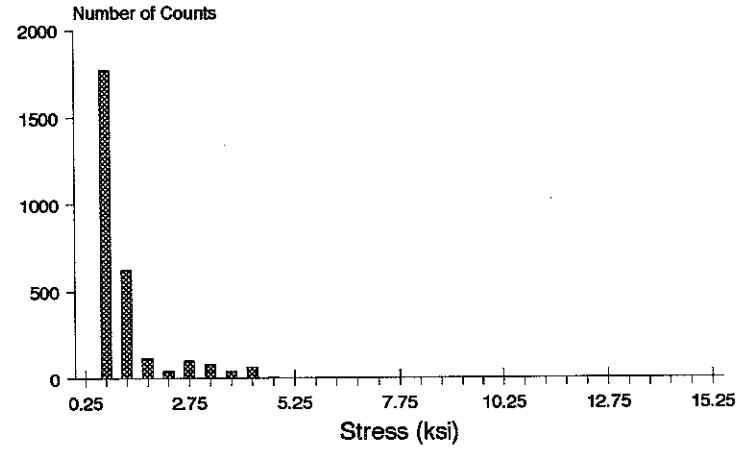


Figure 4. Stress Histograms for the Daniel Boone Parkway Bridge over Big Creek and KY 80, Perry Co. (B-87).

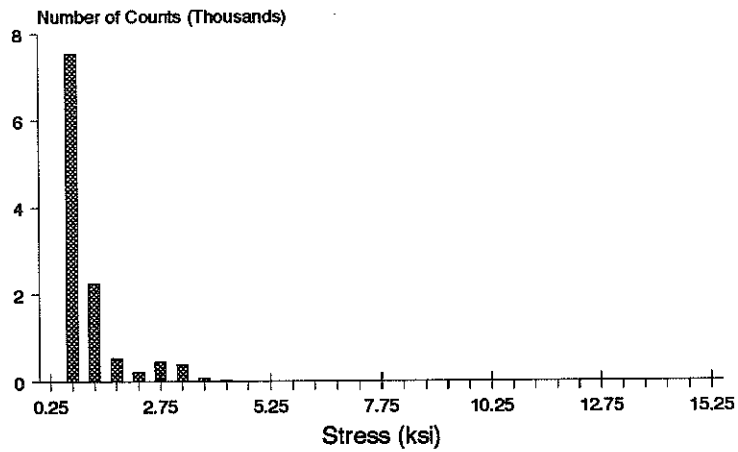
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #2

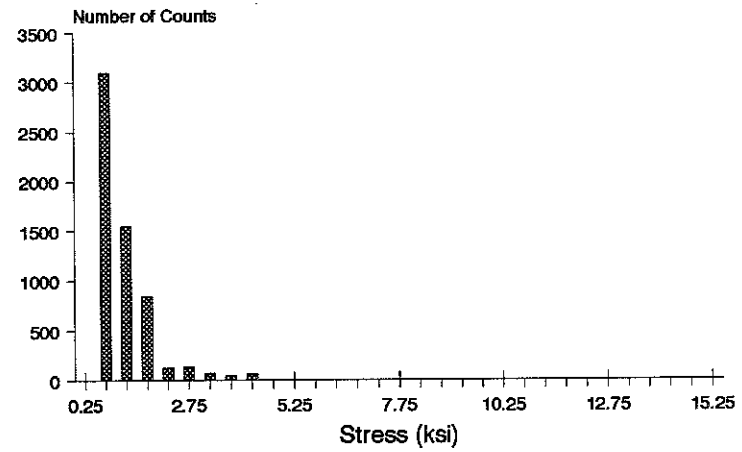
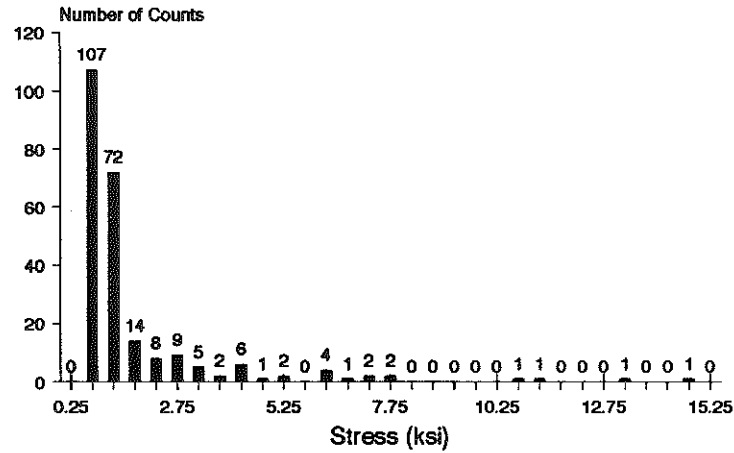
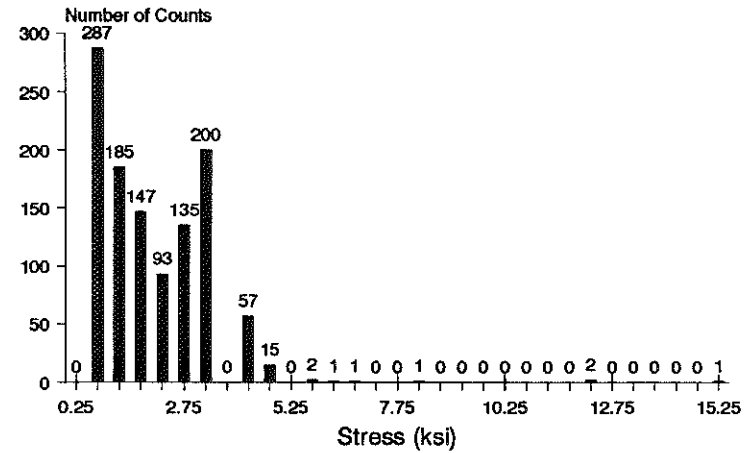


Figure 5. Stress Histograms for the Daniel Boone Parkway Bridge over North Fork of Kentucky River, KY 80 and L&N R.R., Perry Co. (B-89).

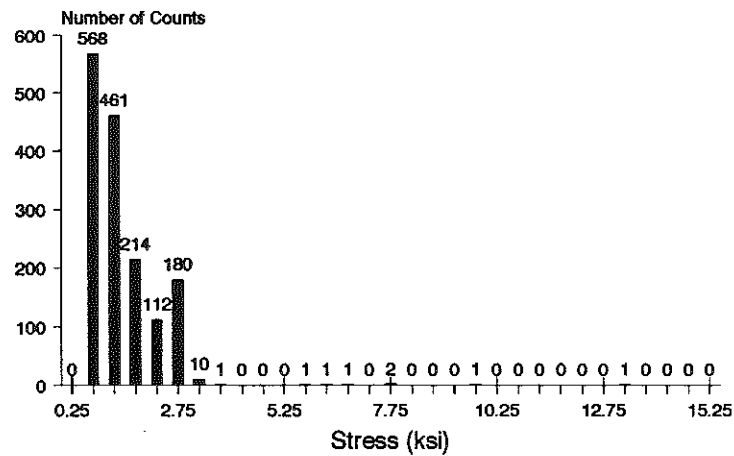
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

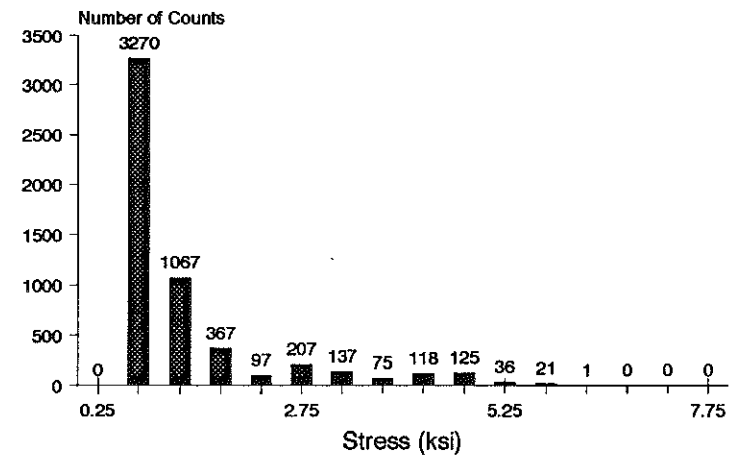
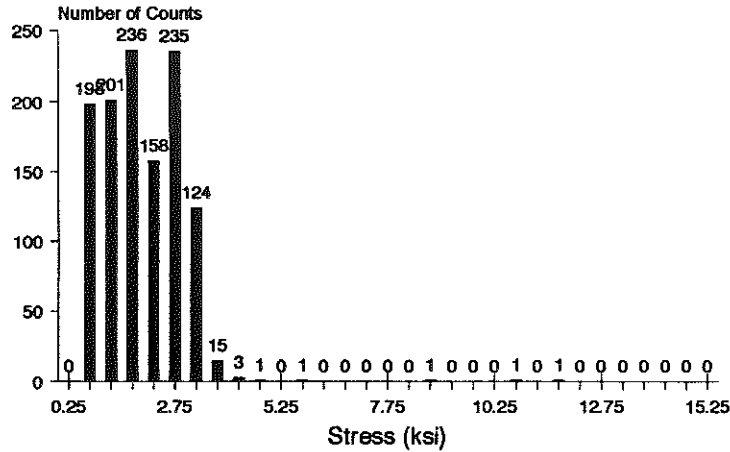
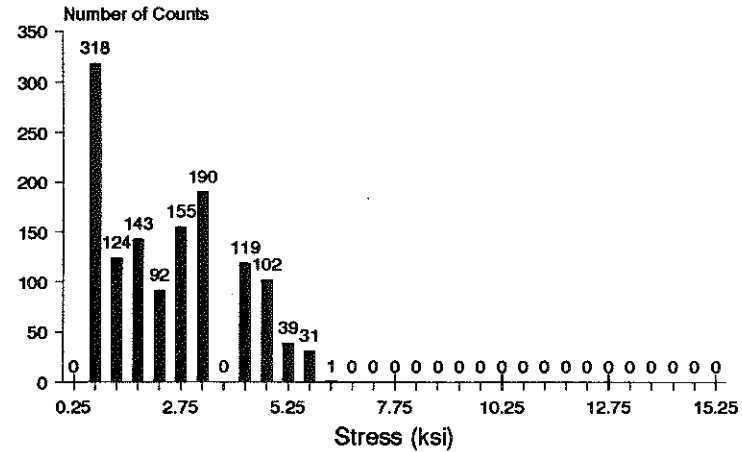


Figure 6. Stress Histograms for Test 1 on the KY 15 Bridge over North Fork of Kentucky River and KY 80, Perry Co. (B-55).

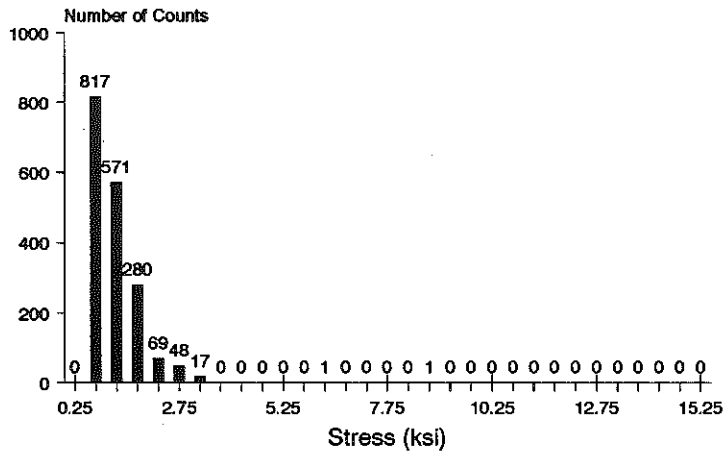
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

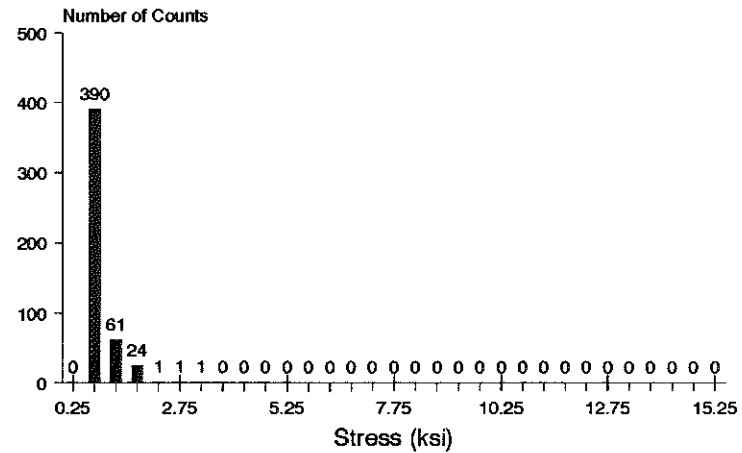
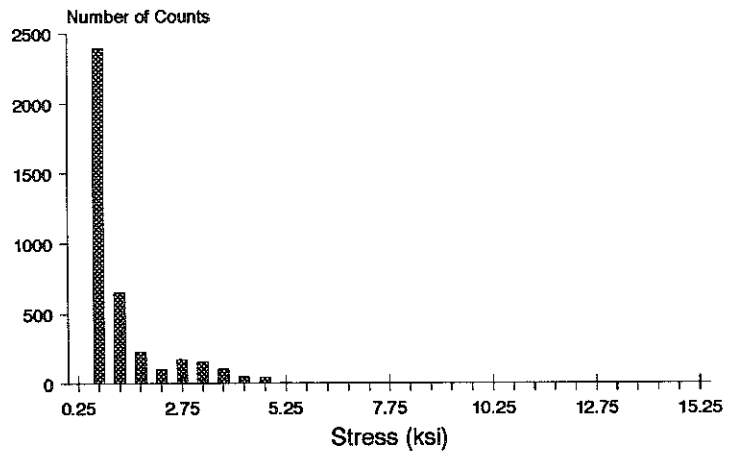


Figure 7. Stress Histograms for Test 2 on the KY 15 Bridge over North Fork of Kentucky River and KY 80, Perry Co. (B-55).

Strain Gage #1



Strain Gage #2

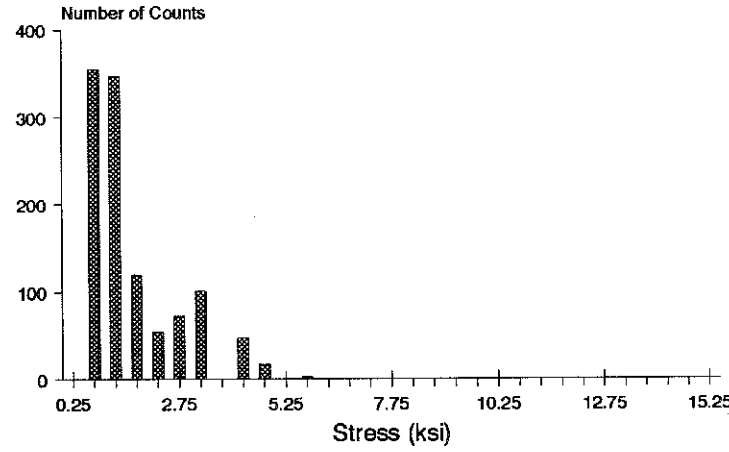
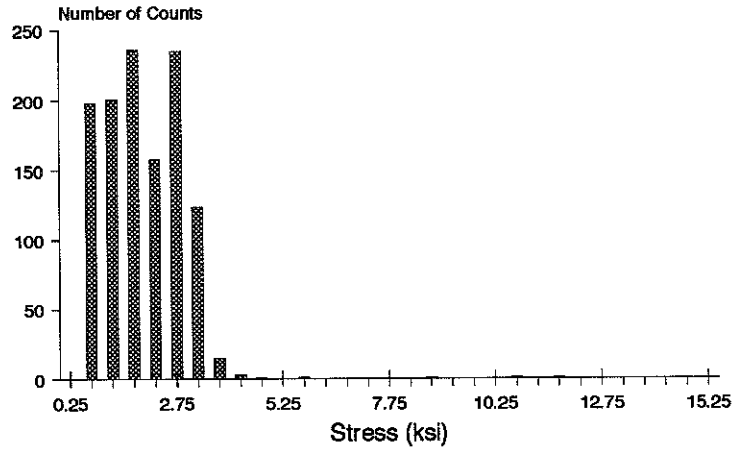
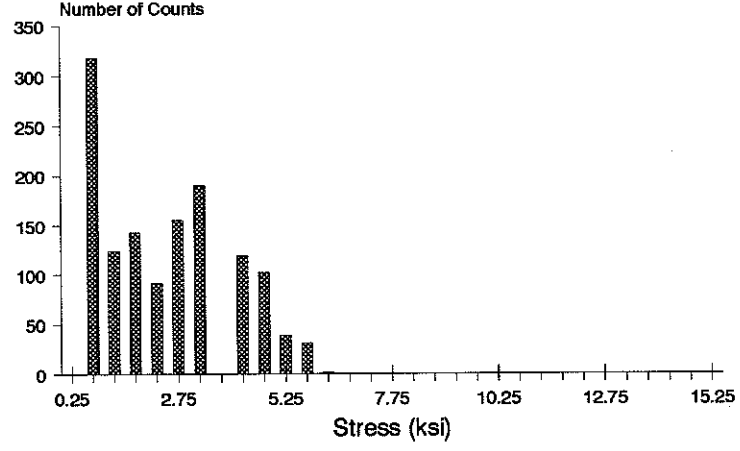


Figure 8. Stress Histograms for the Daniel Boone Parkway Bridge over Red Bird River, Perry Co. (B-82).

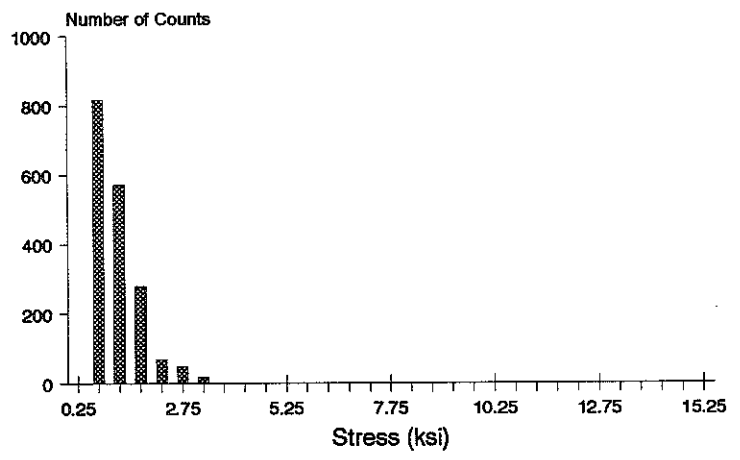
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

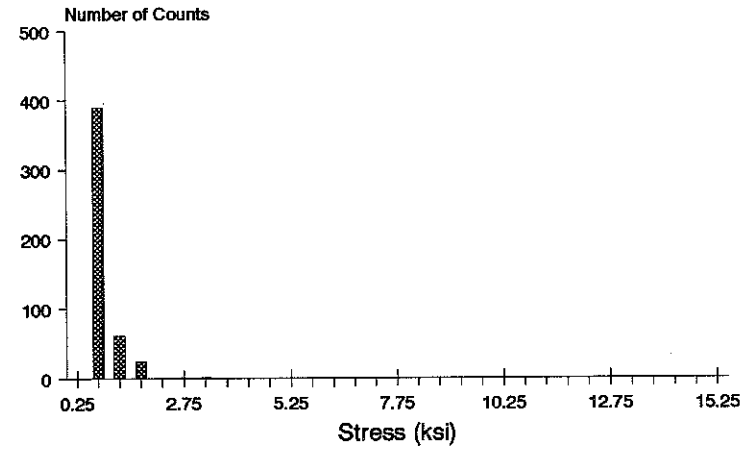


Figure 9. Stress Histograms for the US 119 Bridge over Poor Fork of Cumberland River, Harlan Co. (B-116).

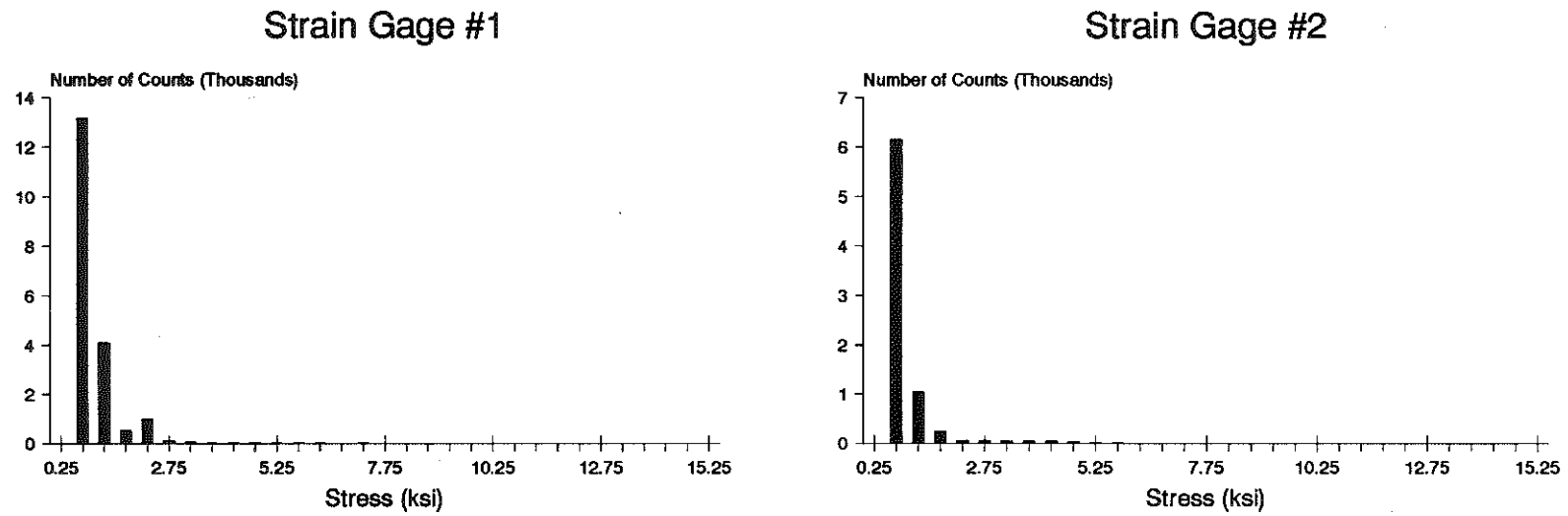
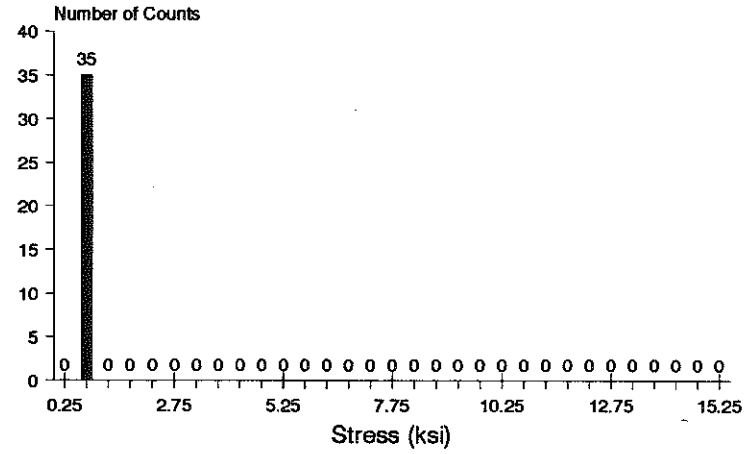
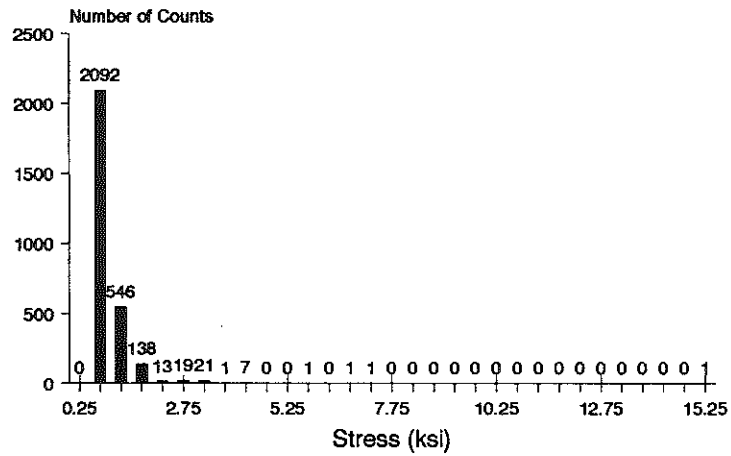


Figure 10. Stress Histograms for the US 421 Bridge over Clover Fork and L&N R.R., Harlan Co. (B-131).

Strain Gage #2



Strain Gage #3



Strain Gage #4

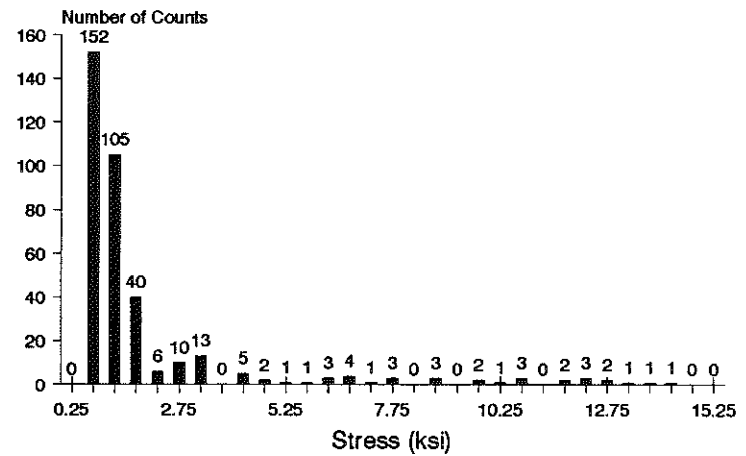
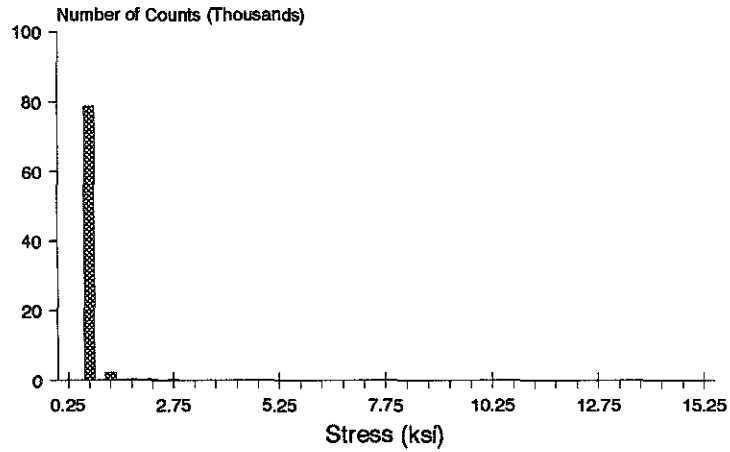
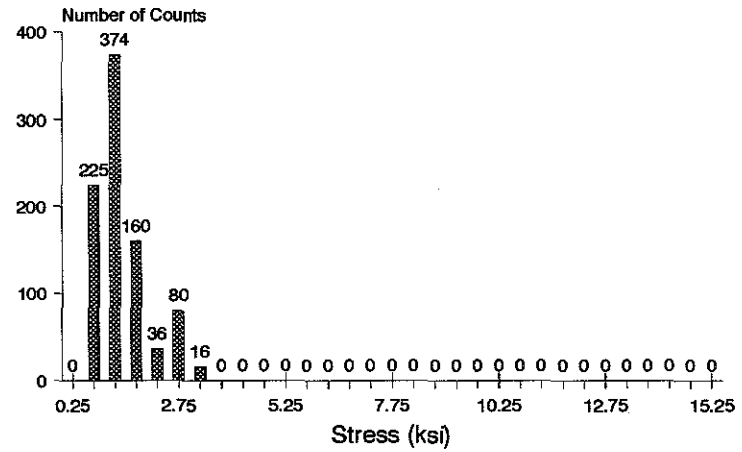


Figure 11. Stress Histograms for the US 23 and 460 Bridge over Levisa Fork. Johnson Co. (B-41).

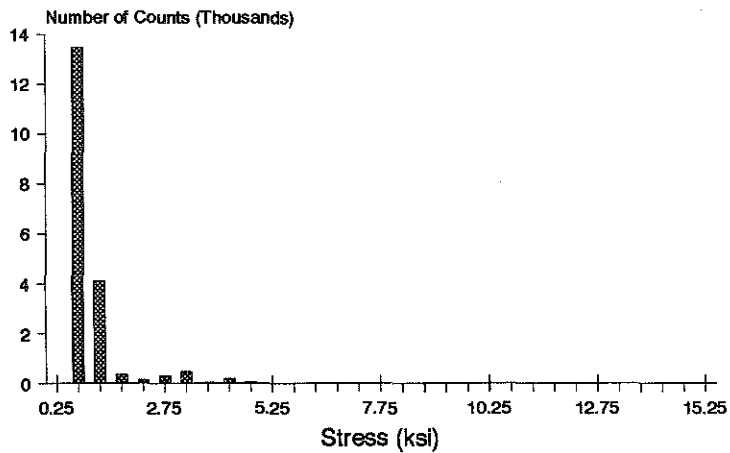
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

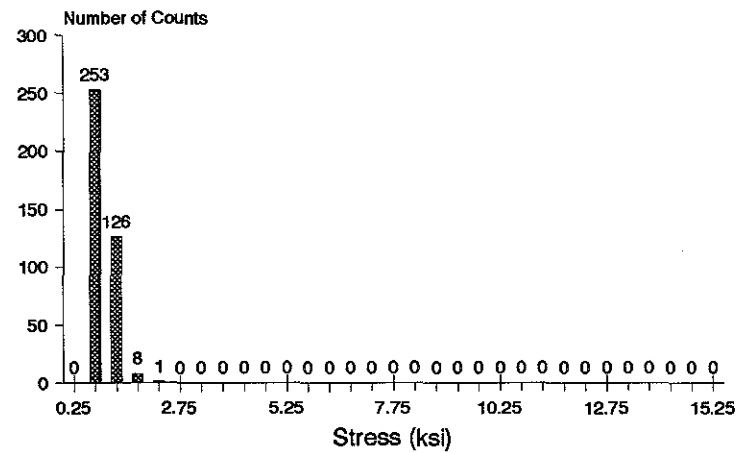
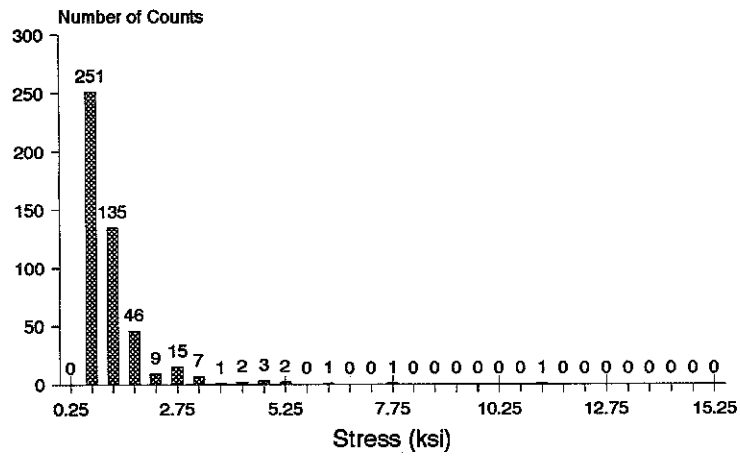


Figure 12. Stress Histograms for the Hazard By-Pass over KY 451, L&N R.R. and Messer Branch, Perry Co. (B-104).

Strain Gage #1



Strain Gage #2

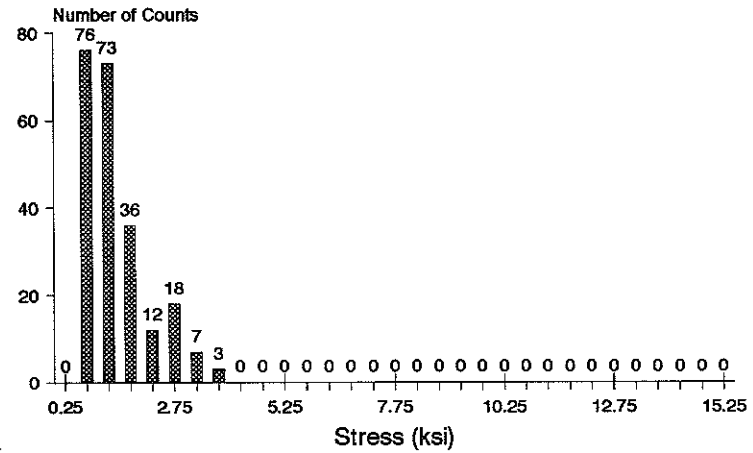
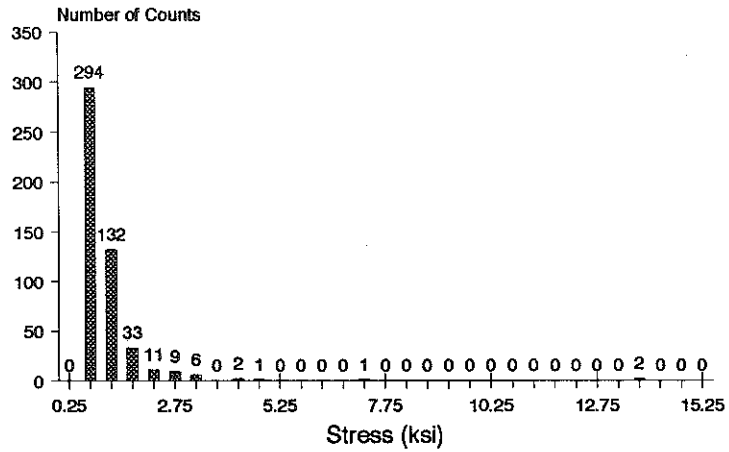
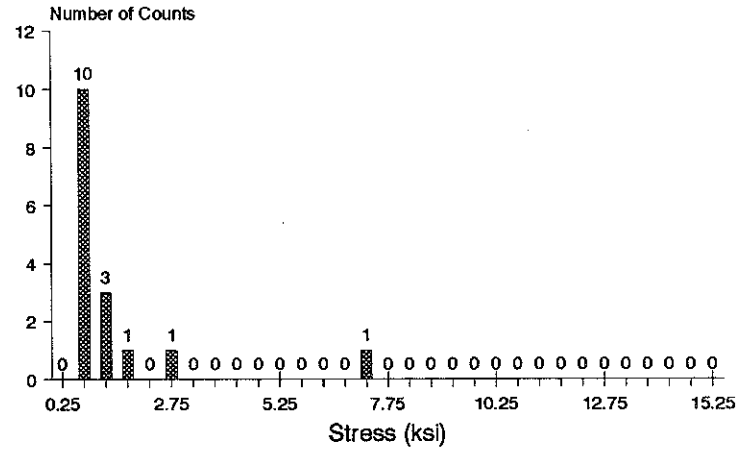


Figure 13. Stress Histograms for the US 421 Bridge over L&N R.R. and Clover Fork, Harlan Co., (B-132).

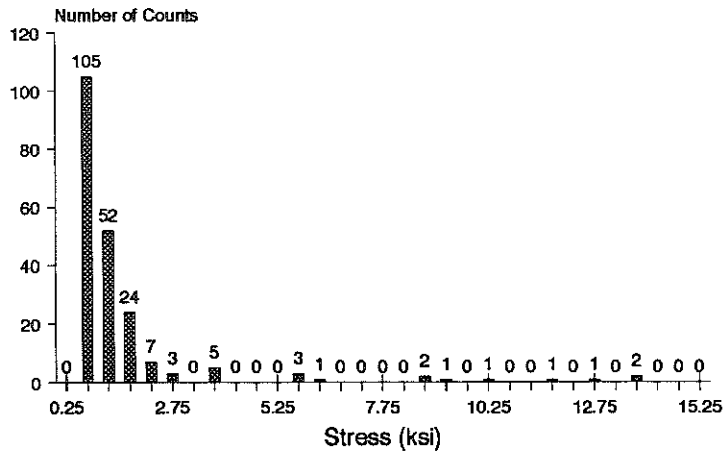
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

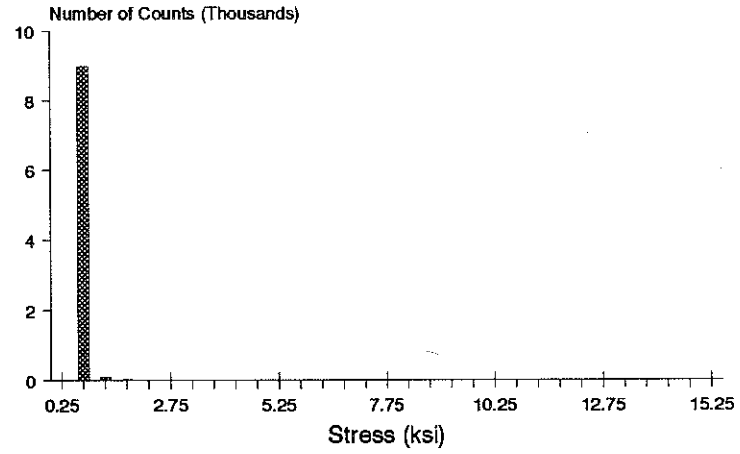
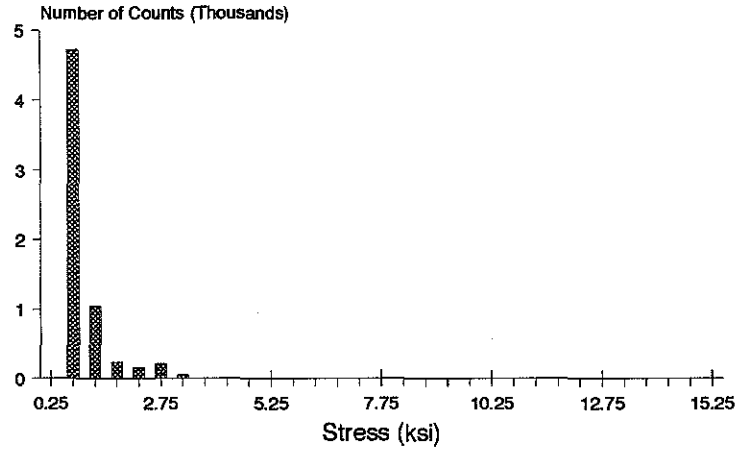
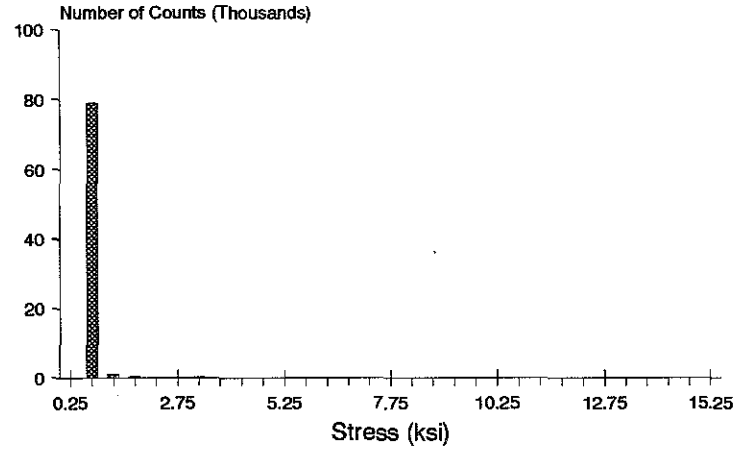


Figure 14. Stress Histograms for the KY 15 Bridge over Carr Fork , Knott Co. (B-56).

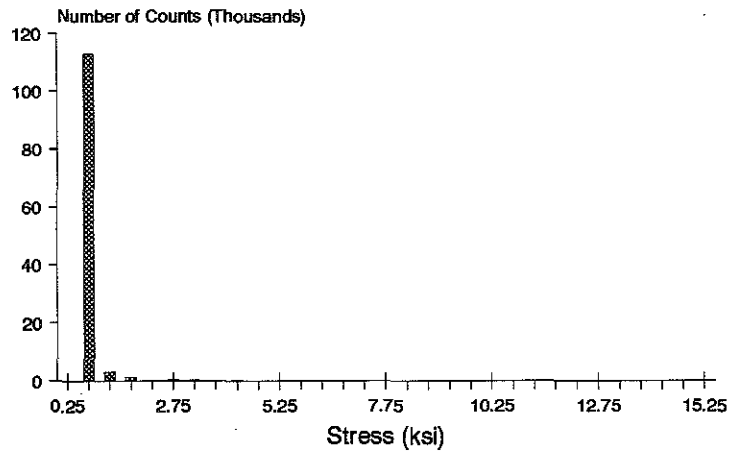
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

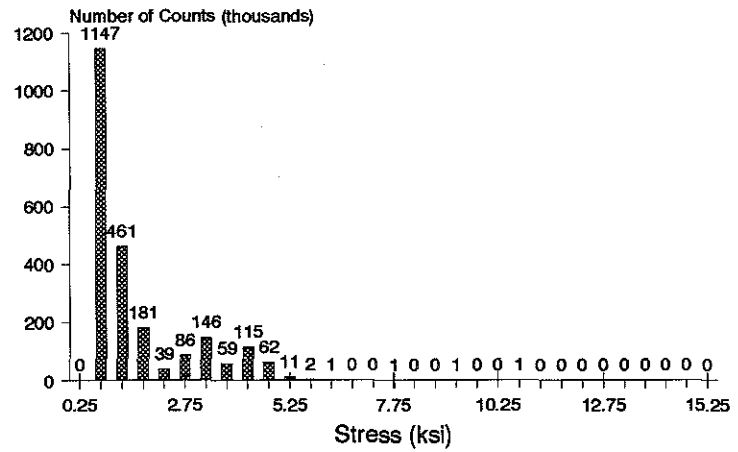
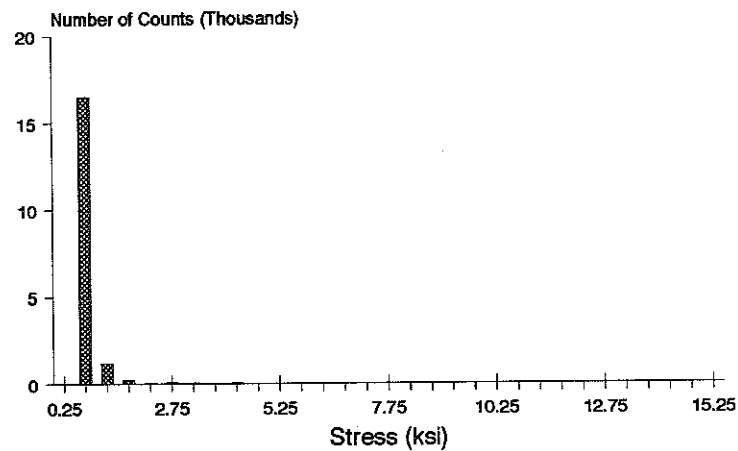
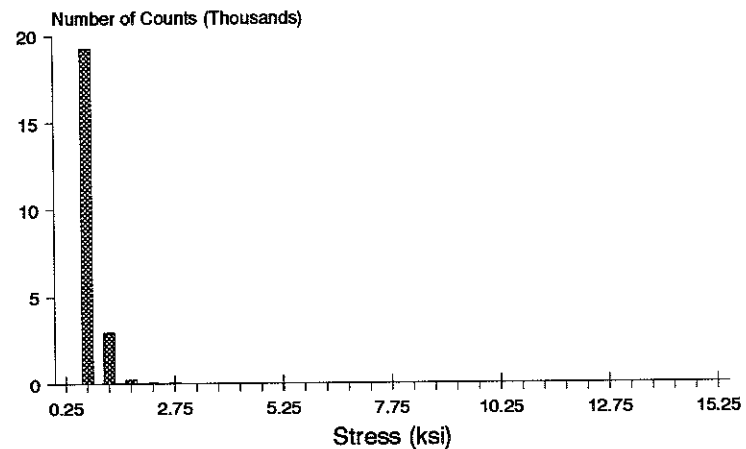


Figure 15. Stress Histograms for the US 119 Bridge over Poor Fork of Cumberland River, Harlan Co. (B-117).

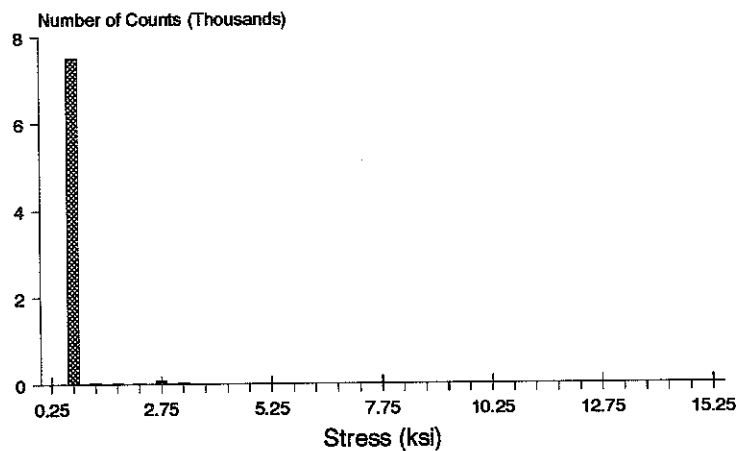
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

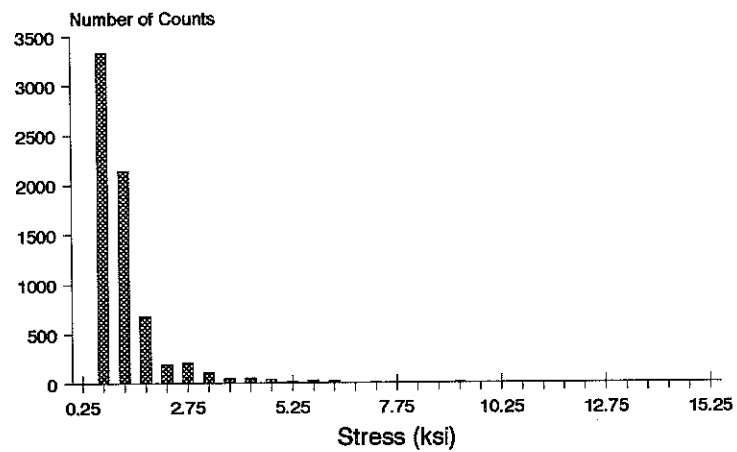
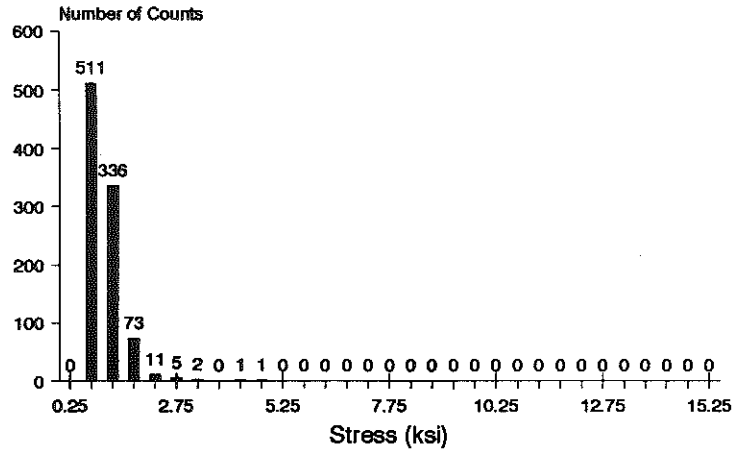


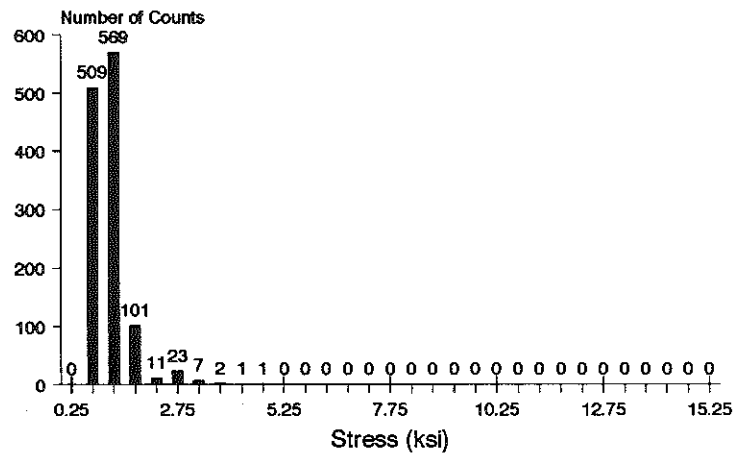
Figure 16. Stress Histograms for the Eastbound Test on KY 80 Bridge over Russell Fork of Big Sandy River and CSX R.R., Pike Co. (B-137).

Strain Gage #1



47

Strain Gage #3



Strain Gage #4

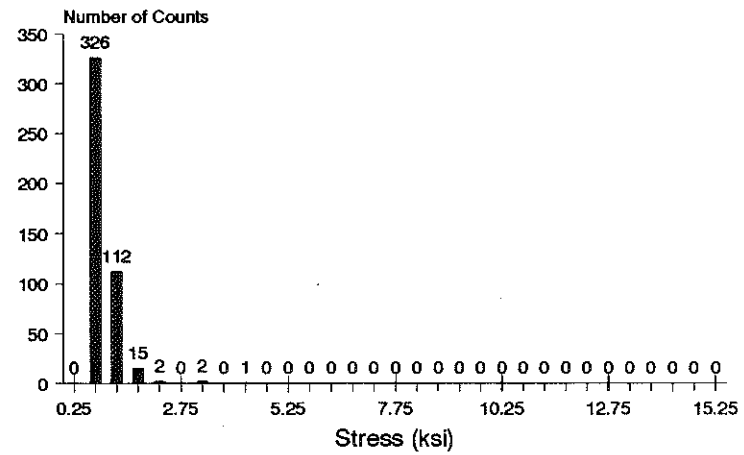


Figure 17. Stress Histograms for the Eastbound Test on KY 80 Bridge over Russell Fork of Big Sandy River and CSX R.R., Pike Co. (B-137).

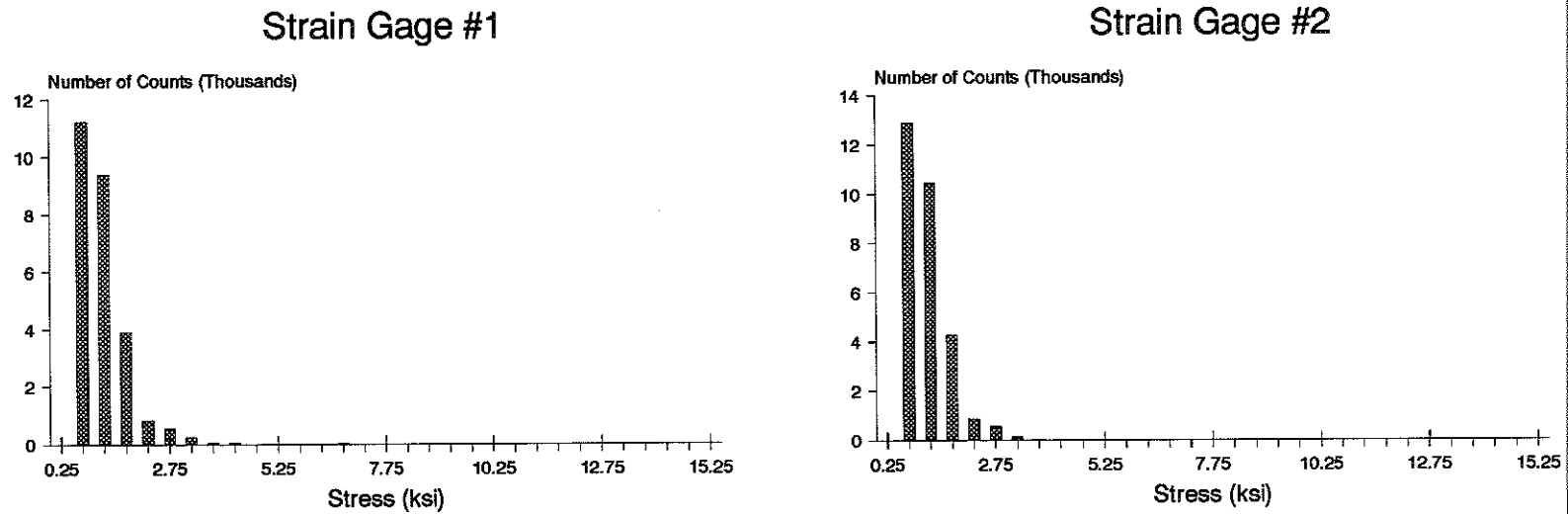


Figure 18. Stress Histogram for the I64-I75 Bridge over US 27, L&N R.R. and S. Limestone St., Fayette Co. (B-83)

APPENDIX B
(Accumulated Damage Superimposed on AASHTO Fatigue
Design Curves for Test Sites with Category E Details)

Strain Gage #1

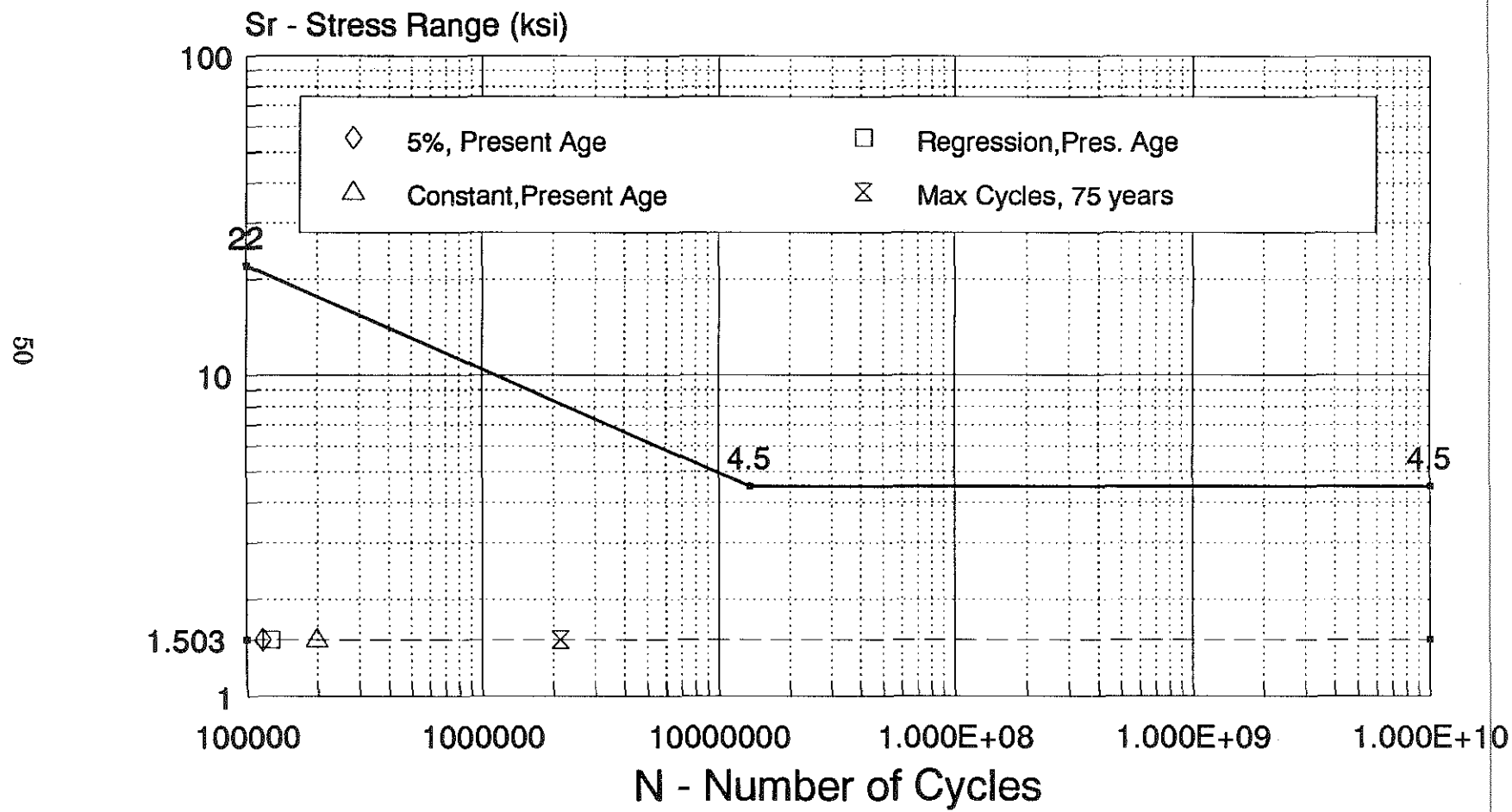
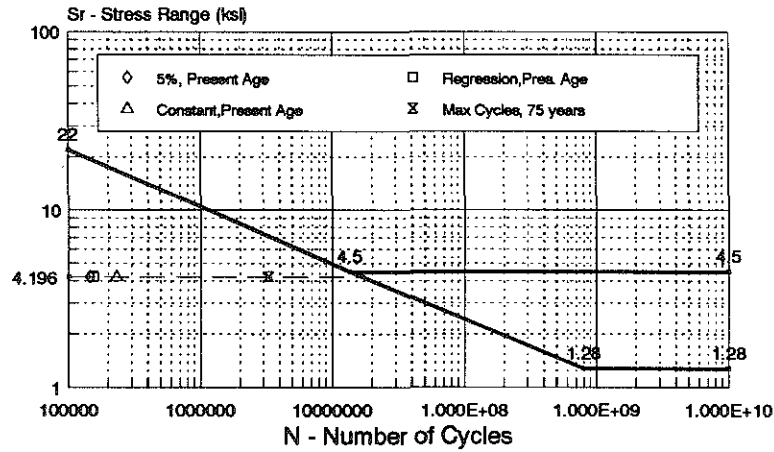
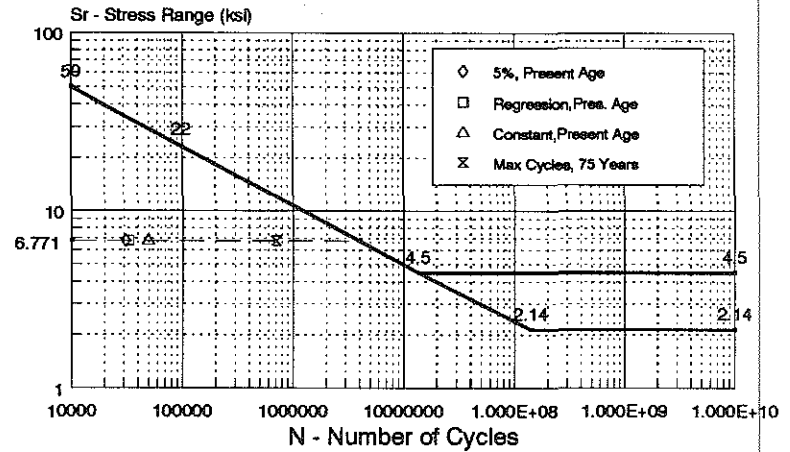


Figure 19. AASHTO Category E Fatigue Design Curve with Data for the Daniel Boone Parkway Bridge over L&N R.R., and Goose Creek, Clay Co. (B-73)

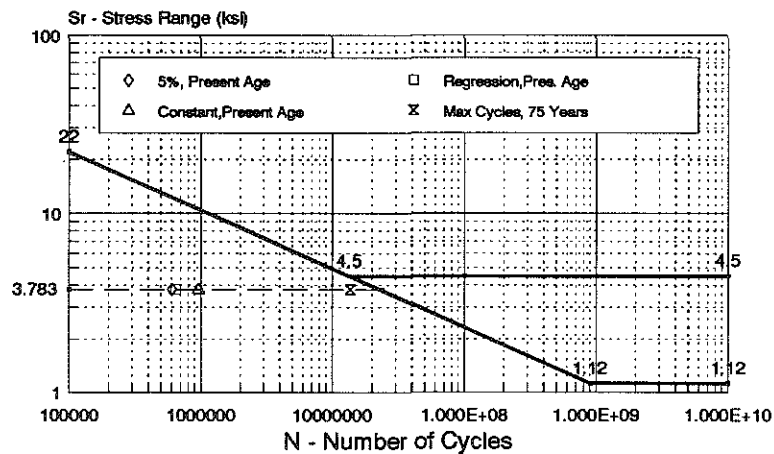
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

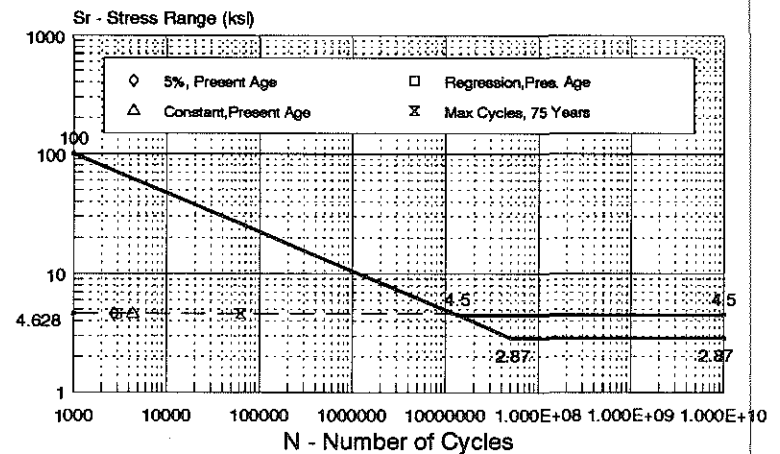
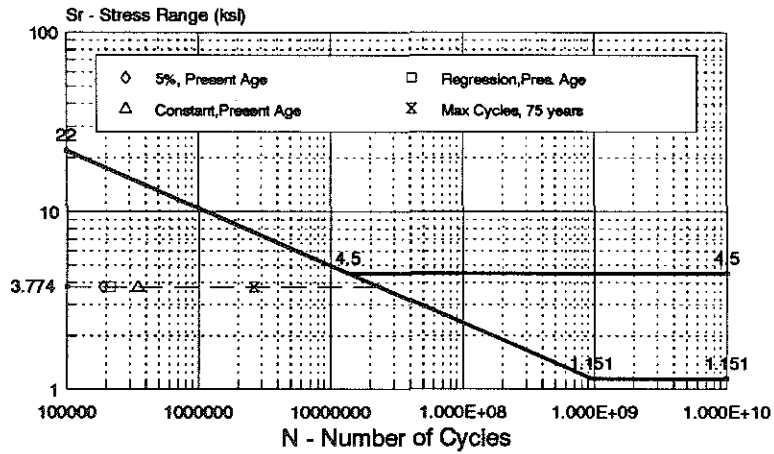
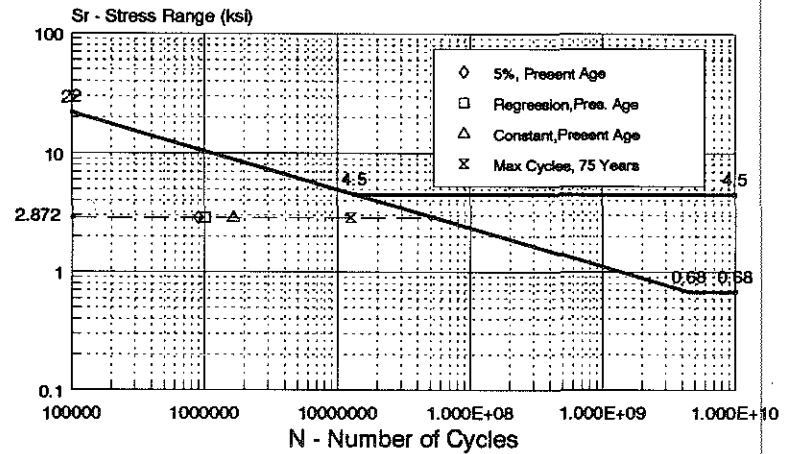


Figure 20. AASHTO Category E Fatigue Design Curve with Data for the Hazard Bypass Bridge over L&N R.R., Perry Co. (B-87).

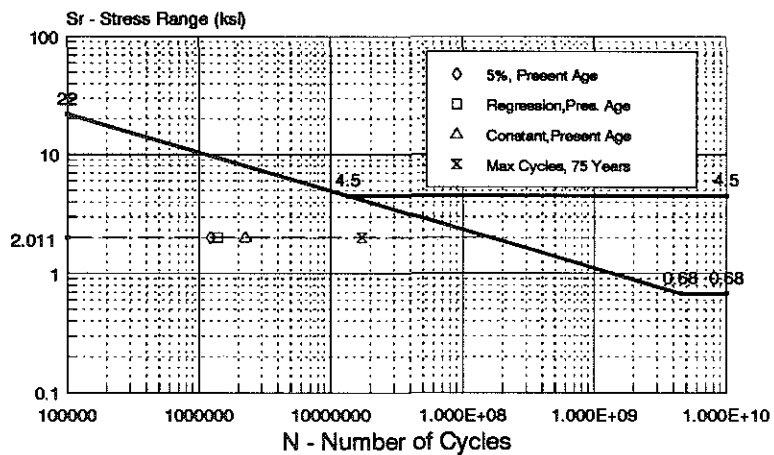
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

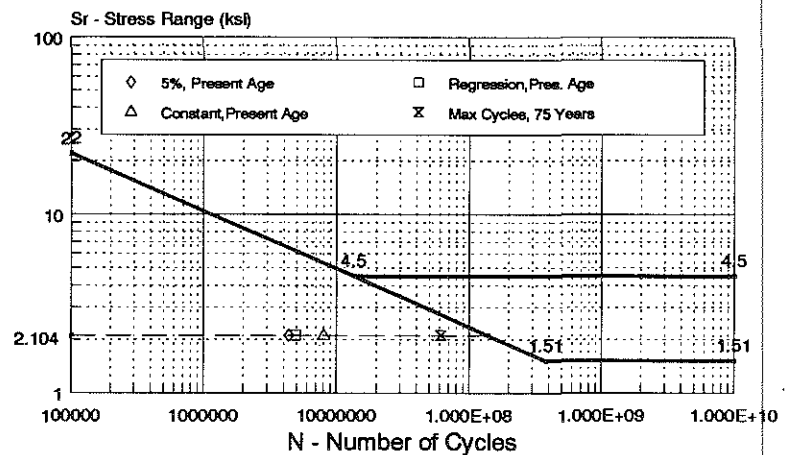
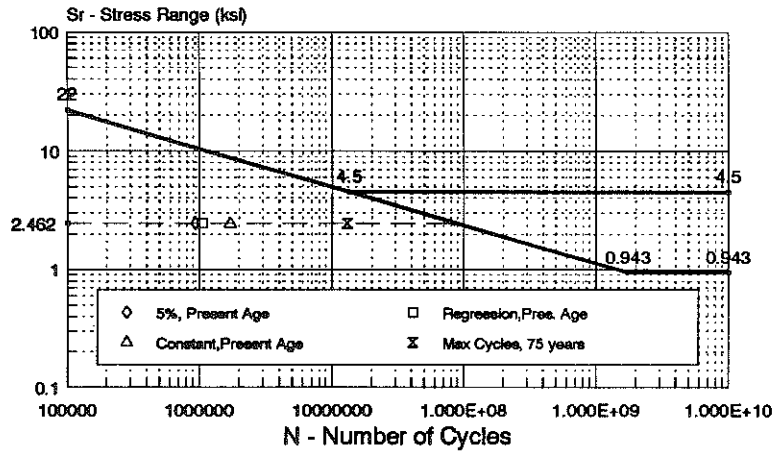
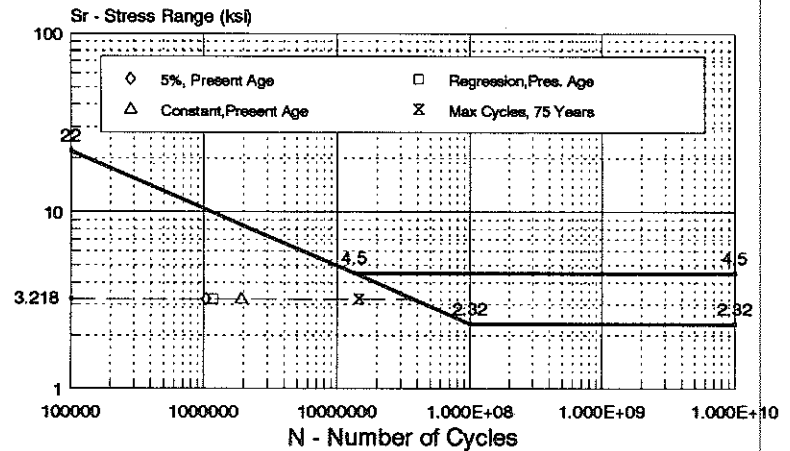


Figure 21. AASHTO Category E Fatigue Design Curve with Data for Test 1 on the KY 15 Bridge over North Fork of Kentucky River and KY 80, Perry Co. (B-55).

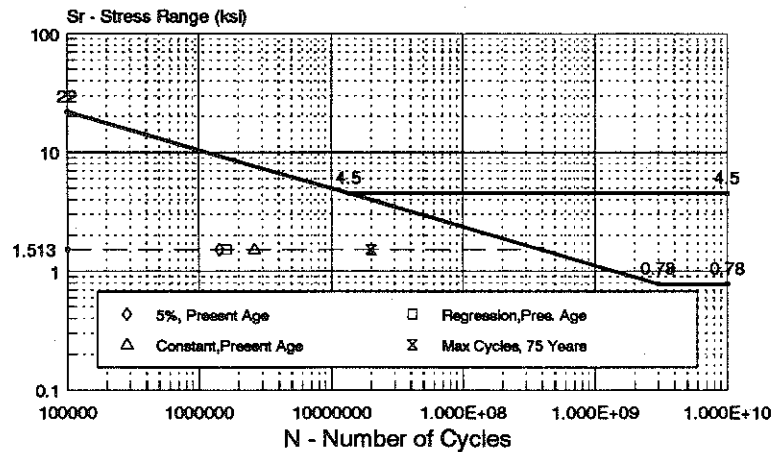
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

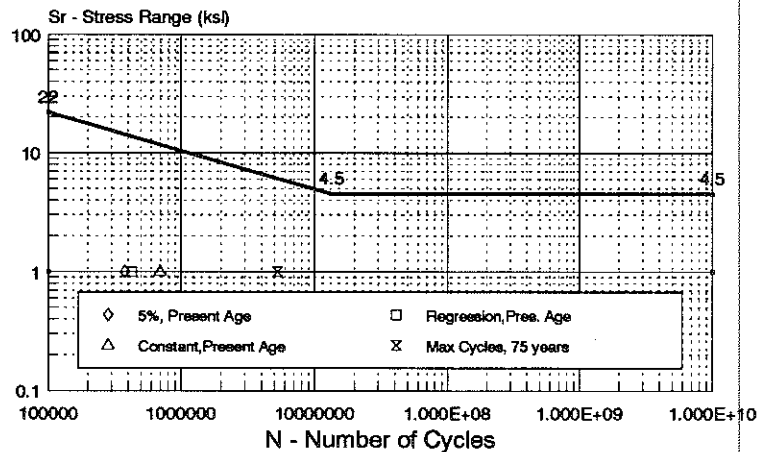
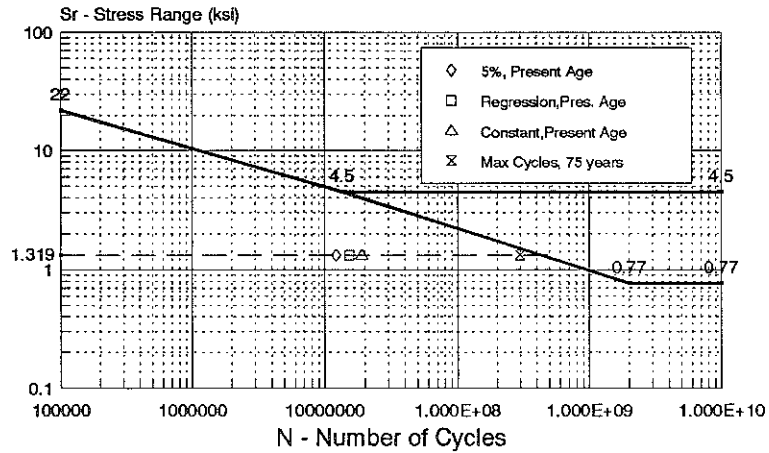


Figure 22. AASHTO Category E Fatigue Design Curve with Data for Test 2 on the KY 15 Bridge over North Fork of Kentucky River and KY 80, Perry Co. (B-55).

Strain Gage #1



Strain Gage #2

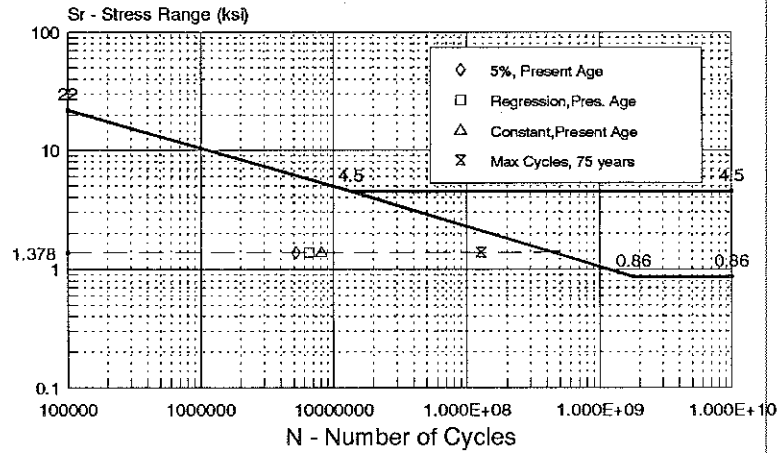
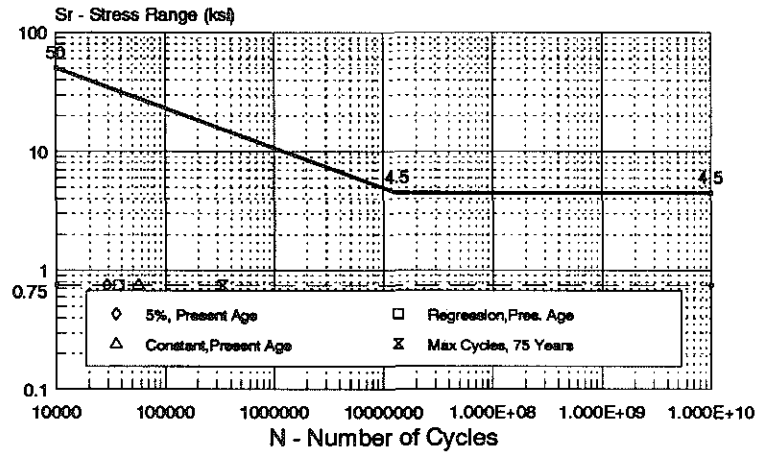
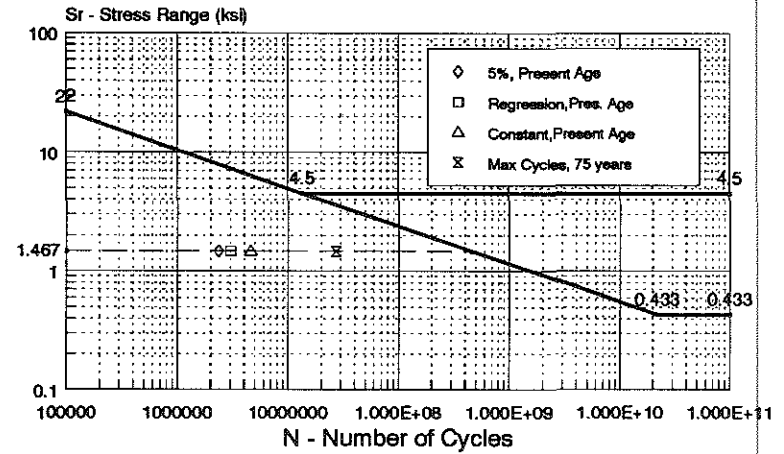


Figure 23. AASHTO Category E Fatigue Design Curve with Data for the US 421 Bridge over Clover Fork and L&N R.R., Harlan Co. (B-131).

Strain Gage #2



Strain Gage #3



Strain Gage #4

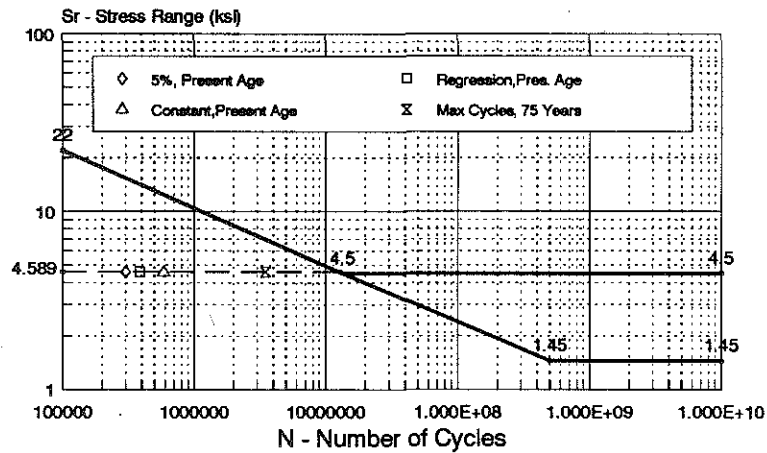
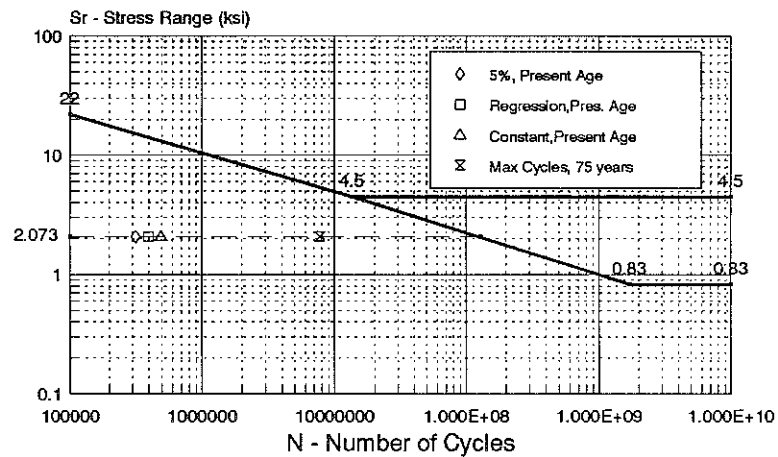


Figure 24. AASHTO Category E Fatigue Design Curve with Data for the US 23 & 460 Bridge over Levisa Fork, Johnson Co. (B-41).

Strain Gage #1



Strain Gage #2

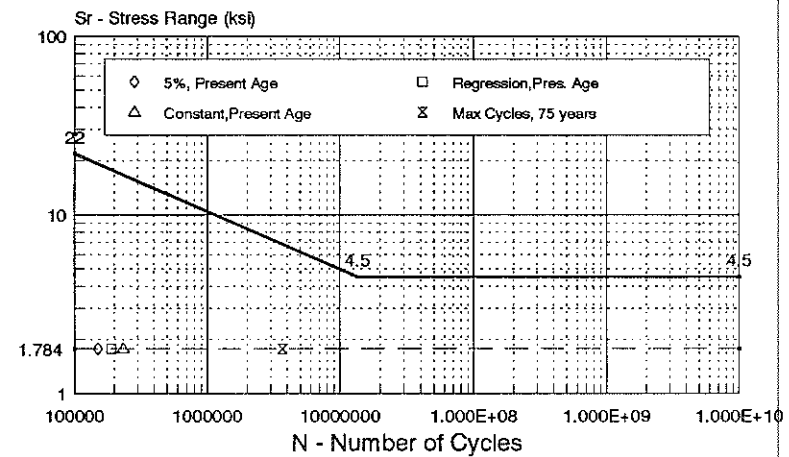
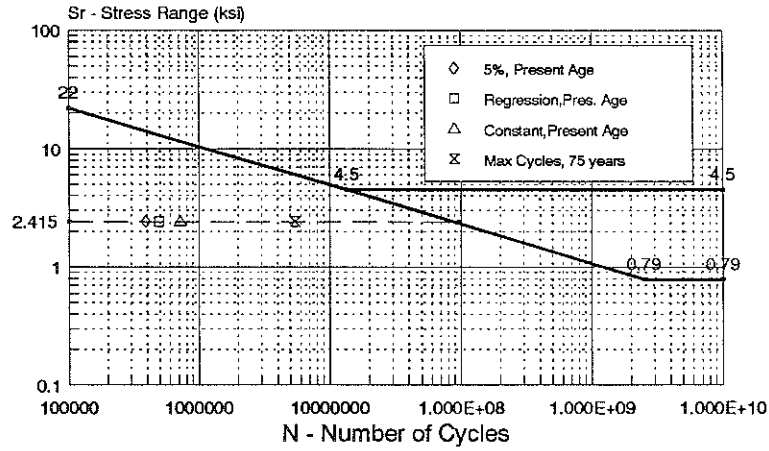
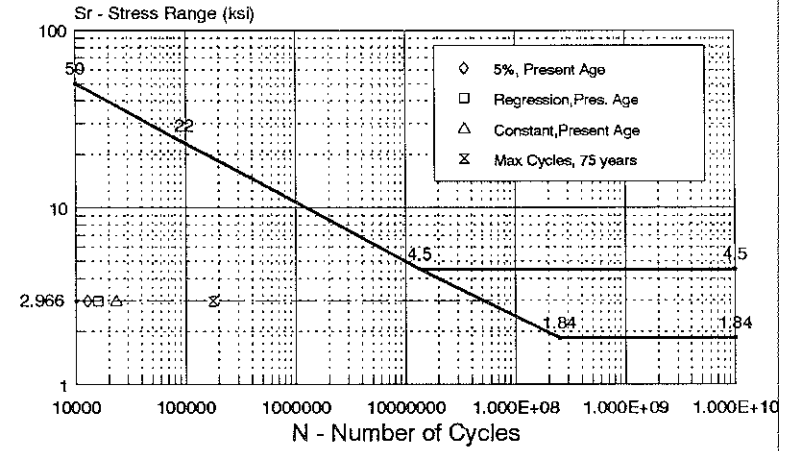


Figure 25. AASHTO Category E Fatigue Design Curve with Data for the US 421 Bridge over L&N R.R. and Clover Fork, Harlan Co. (B-132).

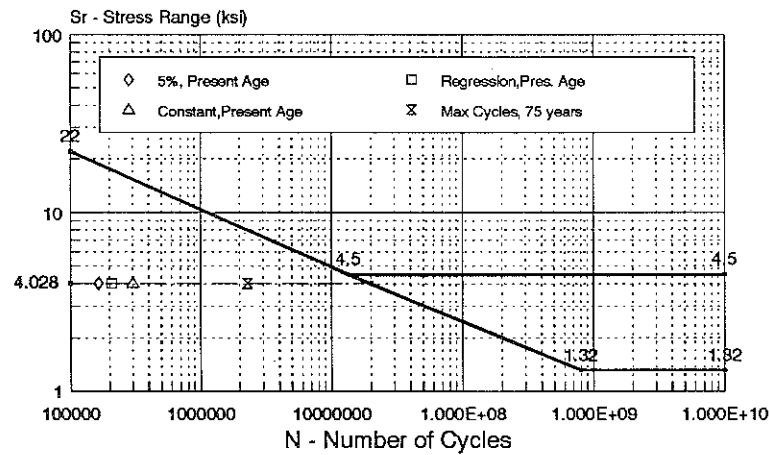
Strain Gage #1



Strain Gage #2



Strain Gage #3



Strain Gage #4

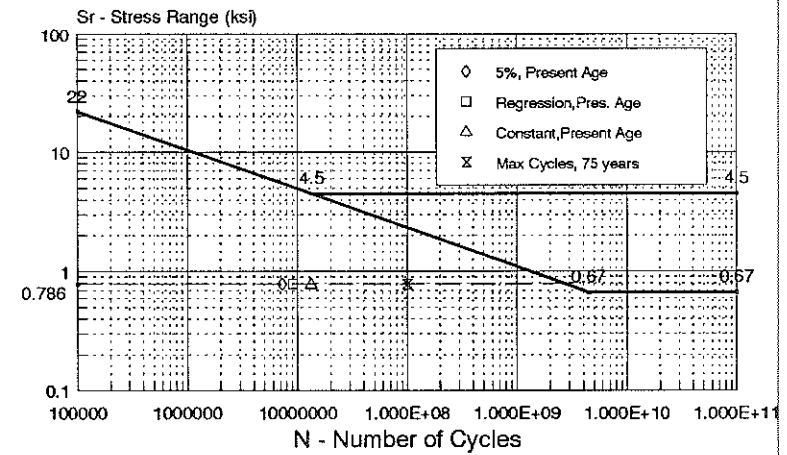


Figure 26. AASHTO Category E Fatigue Design Curve with Data for the KY 15 Bridge over Carr Fork, Knott Co. (B-56).

