Development of a Strain Monitoring System for Bridges: Analysis of Diaphragm-to-Stringer Connection Failures in Composite Bridges

Final Report

June 30, 1992

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JHRAC 92-210 Project 87-4

This research was sponsored by the Joint Highway Research Advisory Council (JHRAC) of the University of Connecticut and the Connecticut Department of Transportation and was carried out in the Civil Engineering Department of the University of Connecticut.

Technical Report Documentation Page

1. Report No.
JHR 92-210

2. Government Accession No.

3. Recipient's Catalog No.

4. Title and Subtitle
Development of a Strain Monitoring System for Bridges: Analysis of Diaphragm-to-Stringer Connection Failures in Composite Bridges

5. Report Date
June 30, 1992

6. Performing Organization Code
JHR 92-210

8. Performing Organization Report No.

7. Author(s)
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10. Work Unit No. (TRAIS)

11. Contract or Grant No.

12. Sponsoring Agency Name and Address
Connecticut Department of Transportation
280 West Street
Rocky Hill, CT 06067-0207

13. Type of Report and Period Covered
Final Report

14. Sponsoring Agency Code

15. Supplementary Notes
None

16. Abstract

This report describes ongoing research at The University of Connecticut treating the behavior of diaphragms in composite steel plate girder bridges. This has included the collection of extensive field data, finite element studies to correlate the field strains with the load carrying capabilities, and evaluations of the fatigue histories.

The Connecticut Department of Transportation has reported problems in connections between diaphragms and connected beams. After 25 to 30 years, cracks typically occur in the angles used for the connections. The live load in the floorbeam results in rotations at the beam ends. This induces bending in the angle at the supporting girder. Secondary stresses occur and cracks initiate at the top of the angle. The cracks follow the line of connectors, usually rivets.

Field data collected during vehicular loading has been used in a comparison with finite element analyses to define the stress-strain behavior in the connections. This behavior is dependent on the overall bridge deformations. This is influenced by the stiffnesses of the diaphragms as compared to the stiffness of the girders and deck. The influence of the diaphragms on the overall bridge behavior is examined.

17. Key Words
Bridges, Diaphragms, Fatigue, Connections, Finite Element Analysis, Field Measurements

18. Distribution Statement
No Restrictions

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	29	

Form DOT F 1700.7 (8-72)

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ACKNOWLEDGEMENT

The study of the diaphragms was possible because of the development of a portable, computer based strain monitoring system. The assistance of Mr. Michael P. Culmo (Engineer, Connecticut Department of Transportation Bridge Design Unit) has been invaluable throughout this project.

CHAPTER 1
INTRODUCTION

Much has been written in recent years about the infrastructure crisis in the United States. The country is currently faced with large scale aging and deterioration of this infrastructure. An area of major concern has been the nations deteriorating highway bridge system. Of the 500,000 bridges on the U.S. road system approximately 231,000 are classified as structurally deficient. The cost of repairing the substandard bridges has been estimated at $53 billion.

Much of the U.S. Interstate system was built in the late 1950's and is reaching a critical point in its lifespan. Components of the system that are not now on the structurally deficient list will need major renovations within the next two decades. The cost of these renovations caused by natural aging will be competing directly with the cost of repairing the bridges that are structurally deficient.

In the bridge system many problems are beginning to surface that the original designers were unable to predict. Some of these problems include: traffic volumes that far exceed the design capacity of the bridges; the continued increase in allowable truck weights; premature deterioration of concrete decks and asphalt wearing surfaces due to excessive use of road de-icing salts; corrosion of steel framing members due to excessive use of road de-icing salts; serviceability problems related to excessive vibrations and fatigue cracking of critical member details.

In the fall of 1988 the Civil Engineering Department at the University of Connecticut was approached by the Connecticut Department of Transportation (ConnDOT) Bridge Design unit with a proposal. ConnDOT was beginning to see an alarming number of cracking problems with the framing members in composite steel-concrete slab bridges. ConnDOT engineers were able to hypothesize the cause of these cracking problems but they were searching for some way to verify their theories. They had developed a standard repair scheme for the problem but these repairs were numerous and expensive. Before the maintenance crews implemented the repairs the ConnDOT engineers wanted to be sure that the same type of cracking problem did not develop again in the future. They needed some type of field monitoring and testing system which could gather the data necessary to correlate their analytical work.

ConnDOT's proposal involved the design, purchase and development of a portable strain monitoring system. The system would have to be simple to setup and use so that it could be put into action quickly if a problem was suspected on a particular bridge. The proposal also involved testing the prototype system on a typical bridge that was plagued by the cracking problem. The test would include not only field work with the system but analytical work, probably along the lines of finite element computer modeling.

The bridge that the ConnDOT Bridge Design engineers proposed for study was a bridge on Interstate 95 in Bridgeport which serves as an overpass of Bostwick Avenue and Cherry Street (designated as bridge #101 by ConnDOT).

The structure carries three lanes of traffic in each direction through a major metropolitan area of the state. The bridge is in the 30 to 35 year old bracket, again typical of much of the U.S. Interstate system. Comprised of multiple simple spans in the 75 to 100 foot range, the superstructures are steel plate girder stringers which run parallel to the direction of traffic and support composite (the deck and the stringers are mechanically connected with shear connectors so they behave as an integral unit) reinforced concrete decks. Steel diaphragms run perpendicular to the stringers at the supports, midspan and quarter points. Figures 1.1, 1.2 and 1.3 show cross-sections of the bridge span at typical diaphragm locations.

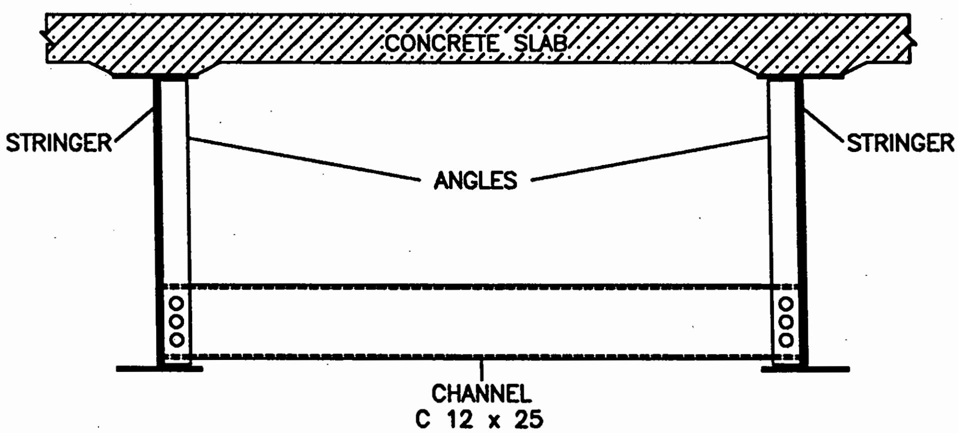

FIGURE 1.1: TYPICAL CHANNEL SECTION DIAPHRAGM AT MIDSPAN & QUARTER POINTS

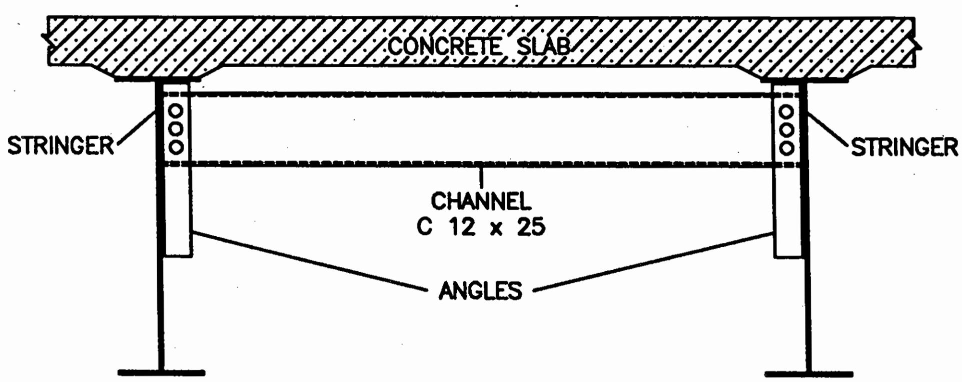

FIGURE 1.2: TYPICAL CHANNEL SECTION DIAPHRAGM AT SUPPORTS

Two types of diaphragms are used: channel sections (Figures 1.1 and 1.2) and K-frames built up from a channel section and two angles (Figure 1.3). At the supports, channel section diaphragms are connected to the stringers near the compression flange of the stringers. At the quarter points and at midspan the channel sections are connected to the stringers near the tension flange of the stringers. At the quarter point and midspan cross-sections of the bridge, channel section diaphragms and K-frame diaphragms are alternated as you proceed transversely across the bridge from one edge beam to the other (Figure 1.4). The K-frames are built up by connecting two angles to the midpoint of the channel and then running these angles diagonally to a point near the compression flange of the stringer. All diaphragm-to-stringer connections are made using clip angles and gusset plates where necessary. The shop fabricated connections (all components of the plate girders) use rivets and the field fabricated connections (all components of the diaphragms) use bolts.

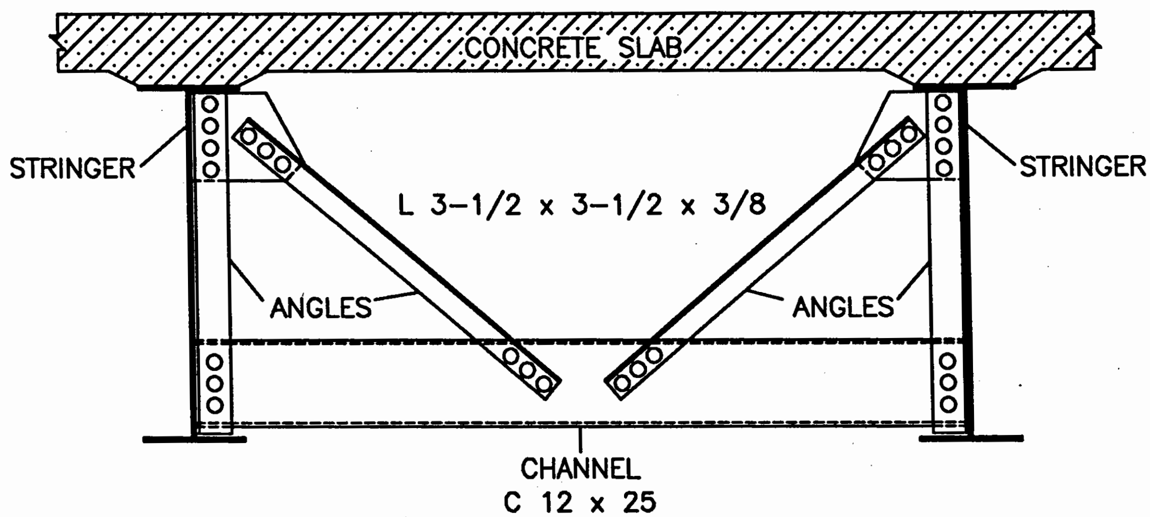

FIGURE 1.3: TYPICAL K-FRAME DIAPHRAGM AT SUPPORTS

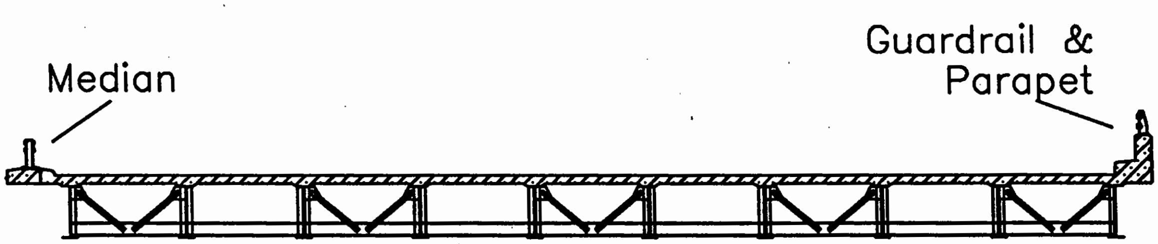

FIGURE 1.4: TYPICAL SPAN CROSS-SECTION SHOWING DIAPHRAGM ARRANGEMENT

The problem with this particular bridge is in the angles that connect the diaphragms to the webs of the stringers. A chronic cracking problem has developed with these angles on a large number of the diaphragms. The cracks do not appear to be linked to the type of diaphragm used. The only obvious pattern is that a large concentration of the cracks occur in the diaphragms near midspan where the differential deflection of adjacent stringers is the greatest. This differential deflection would require the diaphragm-to-stringer connection to resist stresses much larger than it was designed for. The cracks all occur on the angle leg that is connected to the stringer web. The cracks appear to initially form on the back side of the angle between the rivet holes. The cracks continue to grow vertically following the rivet hole line and horizontally through the thickness of the angle leg until the crack passes completely through the leg.

As mentioned earlier ConnDOT engineers have proposed a repair for this problem. Their solution would replace the damaged connection angles with a tee-section. One leg of the tee would be bolted using the rivet holes from the removed angle. New holes would be needed in the stringer web for the other tee leg. This tee-section repair has already been implemented in several locations on the subject bridge where the cracks in the original angles were deemed excessive (greater than 6 inches in length).

The study that ConnDOT proposed had three main goals:

1. Development of a strain monitoring system suitable for field testing of steel superstructure bridges.

2. Through field testing and finite element models determine the cause of the cracking in the connection angles.

3. Also through field testing and finite element models determine if the tee-section repair scheme proposed by the ConnDOT engineers is adequate to solve the problem.

In addition to answering these questions for the bridge engineers the project also addressed the overall behavior of the bridge superstructure. The following questions were also answered:

4. How the actual bridge behavior (deck-stringer-diaphragm relationship) differs from the behavior that was assumed at the time of design and construction.

5. If the diaphragms were largely a construction aid, what would be the effect of removal of the diaphragms rather than repair?

CHAPTER 2
BACKGROUND

Traffic Data For The Subject Bridge

Information concerning traffic flow and vehicle composition was necessary to begin any detailed study of the bridge in question. The following data was supplied by Mr. William Duff of the Connecticut Department Of Transportation - Planning, Inventory and Data Division.

As mentioned earlier, one of the causes of this bridge's problems is the large increase in traffic (both volume and allowable loads) that it has experienced since its construction. After its construction this section of I-95 quickly became the main traffic artery connecting New York City and Boston. In addition to connecting these two cities it also passes through or near the metropolitan areas of Bridgeport, New Haven, New London and Providence. This section of the interstate has seen a large increase in traffic in the last three decades as shown in Table 2.1.

1965		1985		1990	
ADT*	HS-20	ADT	HS-20	ADT	HS-20
37,500	2,250	57,500	3,450	61,000	3,975

* AVERAGE DAILY TRAFFIC

TABLE 2.1: TRAFFIC DATA FOR INTERSTATE 95 BRIDGE #101

Bridge #101 opened for traffic around the spring of 1959. The earliest traffic count data that the Planning, Inventory and Data division have on record is for 1965. As can be seen in the table bridge #101 had an ADT (average daily traffic) volume of 37,500 vehicles in 1965. On the basis of a manual site survey of vehicle types conducted near bridge #101 ConnDOT estimates that 6% of the ADT consisted of 5 axle trucks (HS 20 type).

This section of Interstate 95, like most highways in the United States, experienced a steady increase in volume as time passed. When it was originally constructed, I-95 through Connecticut was a toll road. Because of the high cost of the tolls, many heavy trucks traveling between New York and Boston avoided I-95. The less direct but more economical route of I-684, I-84, I-90 (also a toll road but cheaper and for a shorter distance that I-95) saw a large share of the heavy commercial traffic. In 1986 the tolls were removed from all of Connecticut's highways. Once the tolls were removed a larger portion of commercial traffic started using I-95. Table 2.1 shows that from 1965 to 1985 (just prior to toll removal) the section of I-95 near bridge #101 experienced an ADT increase of 53% to 57,500 with 6% of this increase consisting of 5 axle trucks. In the years immediately following toll removal the ADT increased by another 3500 vehicles per day. The significance of this is that 15% of the increase consisted of 5 axle trucks (according to ConnDOT estimates) whereas in the past only 6% of the increase could be attributed to 5 axle trucks.

Figure 2.1 shows the rate of HS-20 traffic growth for bridge #101. Using the data from Table 2.1, ~~an equation was formulated projecting the daily traffic~~ through the year 2000. Assuming the same exponential rate of growth that this section of I-95 has experienced in its first 31 years, the number of HS-20 trucks per day in the year 2000 will be approximately 4900.

Using the equation mentioned above an estimate can also be made for the number of HS-20 cycles that bridge #101 has experienced since it opened for traffic in 1959. Integrating the equation once with respect to time and evaluating between the years 1959 and 1991 yields an estimate of 33.5 million cycles.

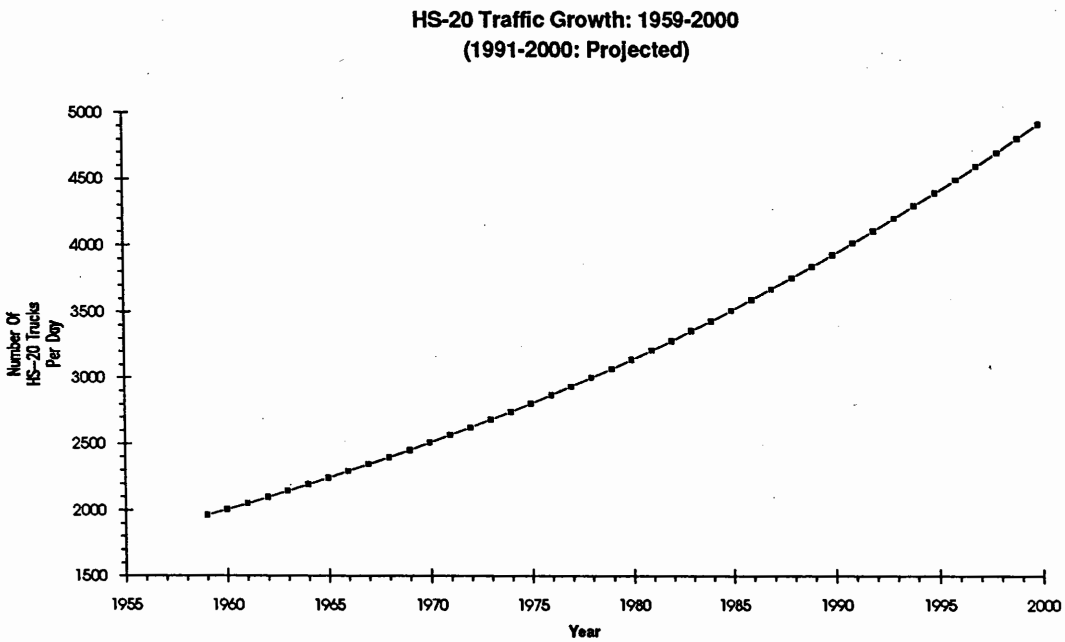

FIGURE 2.1: TRAFFIC PROJECTIONS FOR THE YEAR 2000

The approximation is crude but it is sufficiently accurate to make the following point. AASHTO's (The American Association of State Highway and Transportation Officials) Standard Specifications for Highway Bridges (ref #24) includes a section on fatigue design (section 10.3). In this section allowable stress ranges are given for the design of various components based on an estimate of the maximum number of cycles that the component will see during its service life. The largest number of cycles given in Table 10.3.1A is 2,000,000. The bridge designer may make the assumption that AASHTO is implying most bridges will experience in the vicinity of 2,000,000+ cycles and that the design code is based on this magnitude of cycles. What design category should a structure like bridge #101, which has seen approximately 33,500,000 cycles, be placed in? Is this portion of the AASHTO code adequate for today's high volume bridges?

The estimate of cycles made above is the number of cycles that the main members have seen. The problem areas for bridge #101 are in the diaphragm to stringer connections. Testing performed on both highway and railroad bridges indicate that diaphragms experience a larger number of stress cycles than the main members (reference #9).

Literature Review

A literature review was conducted in the areas of fatigue cracking, finite element analysis of bridges and diaphragm behavior.

1. Many bridges have experienced fatigue cracking of a different nature but still related to diaphragms. The webs of the main stringers have experienced cracking due to out of plane displacements caused by the diaphragms. The actual connections between the diaphragms and the stringers were not damaged.

   John W. Fisher of Lehigh University is the leading researcher in the area of fatigue cracking of bridge components. Most of his work has been in the area of cracking of welds or cracking of the "web gap" region of girder webs (see Figure 2.2). This "web gap" cracking is obviously different than the connection cracking experienced by bridge #101. Nevertheless he has made some design recommendations concerning cracking of connection angles similar to the problem with bridge #101 (see reference #7 and chapter 5 of this report).

2. Much work has been conducted examining the scope and limitations of the finite element method as it pertains to bridge analysis.

   Jategaonkar et al. (reference #18) have published an extensive work containing recommendations for finite element analysis of bridge superstructures. Their methods and suggestions were closely adhered to during the analytical portion of this study.

3. Development of simplified analysis techniques for composite concrete slab - steel stringer bridges that can be applied by the design office engineer.

   A large quantity of work has been done in the area of simplified analysis techniques. This work is responsible for the format of the present day AASHTO and Ontario Highway Bridge Design codes. The main problem with these codes is the high degree of conservatism that has resulted from over simplification of the three dimensional behavior of a bridge superstructure in order to make the analysis economical. Many studies have shown that the codes do not accurately predict the lateral load distribution for many bridge types. Over the last ~~decade many researchers have proposed modifications to the~~ design codes in order to solve this problem.

References in these areas are listed in the bibliography. A complete discussion of previous research can be found in reference 19.

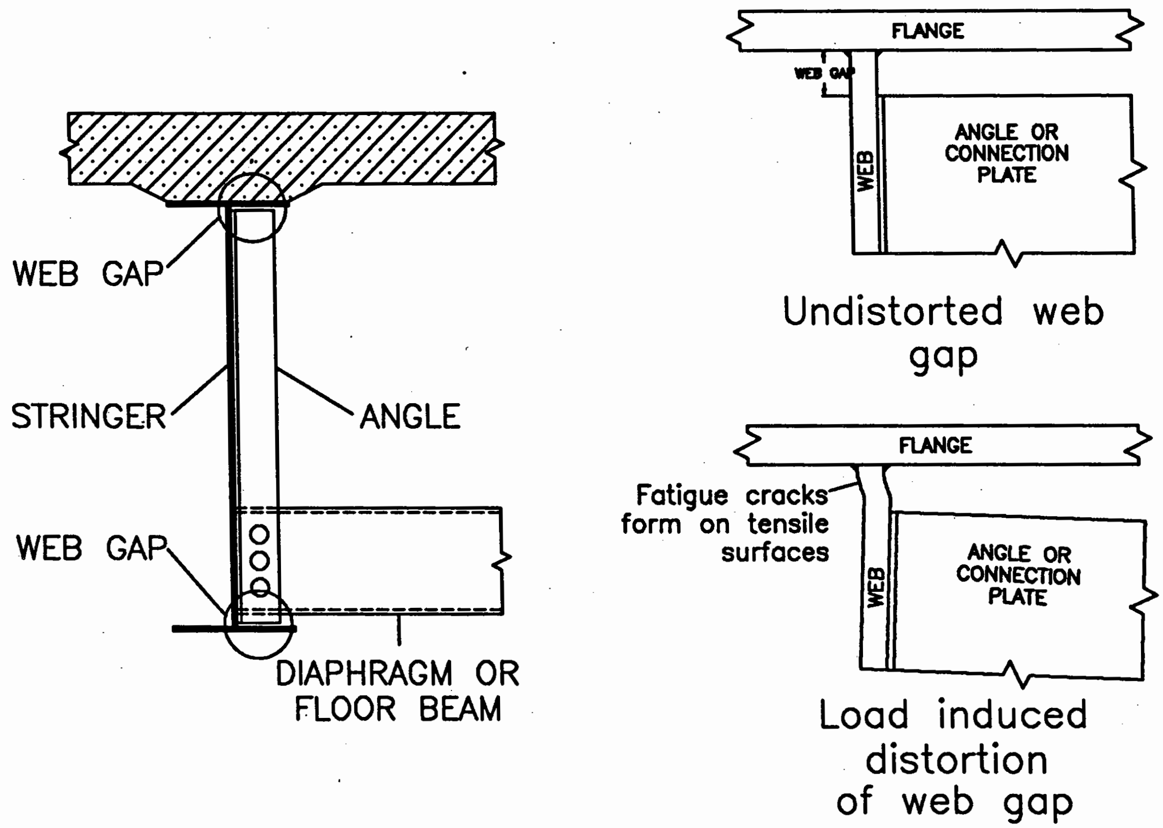

FIGURE 2.2: FATIGUE CRACKING DUE TO "WEB GAP" DISTORTION

CHAPTER 3
SYSTEM DEVELOPMENT

Hardware Requirements

During the initial phase of the project several finite element computer models of the bridge and its various components were developed. In the development phase of any computer model several key assumptions need to be made (concrete compressive strength, degree of connection fixity, etc...), especially with models of this complexity. Some method of verifying these assumptions was necessary. This was the main purpose of the experimental study.

The ultimate goal of the experimental portion of the project was to obtain stress and strain data under actual traffic loading conditions. Today, the accepted technique for obtaining this type of data is to use electrical resistance strain gages. Before any field work could commence, the data acquisition hardware and software systems that would excite the strain gages had to be assembled.

The experimental portion of the project consisted of the following steps:

1. Determination of the type of data required. How many channels (strain gages) are necessary and how often does each channel need to be read?

2. What type of data transmission scheme is needed (digital or analogue)?

3. Does the data acquisition system need to be computer controlled? If so how much automatic data reduction needs to be done in the field? What type of software is needed?

4. What type of power source should be used? AC or DC?

5. What type of attachment method should be used for the strain gages? Adhesives or spot welding?

Potential Error Sources

As with any form of electronic measuring devices, several sources of induced errors are present with electrical resistance strain gages. Steps can be taken during attachment of the gages and with the method of data acquisition to minimize these errors. Some common errors are:

1. Damage to the gage during installation from soldering iron heat: the lead wires are, in most cases, soldered to the gages. The small size of most gages and the proximity of the lead wire tabs ~~to the gage itself make this a difficult task under field conditions.~~

2. Electrical noise and interference: this includes thermally induced voltages caused by thermocouple effects at the junction of different metals (commonly at connection points) and magnetically induced voltages.

3. Temperature effects: the expansion and contraction of the gage materials with fluctuations in temperature can cause false resistance changes in the gage.

The hardware choices that were made were largely controlled by these three factors.

Data Acquisition Systems

The design and implementation of the data acquisition system were the most difficult parts of the experimental study. The three problems listed in the previous section had to be addressed while still maintaining a reasonable budget for the project. In this section each of the items listed above will be discussed and an explanation given for the chosen solution.

The method of attaching the strain gages turned out to be more critical than was first assumed. The conditions under which field work on bridges is done are often not the most optimal. First, the access to a bridge's superstructure is usually, if not always, difficult. Work of this type must be conducted from ladders, staging, hydraulic lift platforms, cherry pickers or, in rare instances, from permanent inspection catwalks under the bridge. None of these work surfaces provide the type of environment needed to do small scale, delicate work such as attaching strain gages. Second, a bridge is constantly in some state of motion. Normal traffic patterns create vibrational motion of the superstructure that makes attaching gages and lead wires extremely difficult. The only time a bridge is at rest is when there is no traffic on its deck. This rarely occurred on the high volume bridges on Connecticut's Interstate system that are normally of interest for field monitoring.

For these reasons spot welding was chosen over liquid adhesives for strain gage attachment. Spot welding equipment for strain gages is compact, portable and has a self-contained rechargeable DC power source. Liquid adhesives, which typically have several components that need to be applied separately, would be too unwieldy under the difficult field conditions that were expected. Also, most liquid adhesives must be used at ambient temperatures above freezing in order to cure properly. Spot welding equipment is expensive but, fortunately for this project, the Connecticut Department of Transportation had purchased a system for a pervious study.

The issue of fluctuating readings due to changes in temperature were easily handled using a standard technique for strain gages. An unstrained piece of material (the same type of material as what is being tested) with a "dummy" strain gage attached is placed near the active gage. Both gages experience the same temperature fluctuations and the Wheatstone bridge circuit can use the readings from the "dummy" gage to compensate the active gage.

The next consideration was the type and quantity of data required. How many channels are needed and how often should each channel be read?

These questions were answered by the economics. What is a reasonable number of channels? Ideally strain data would be desired from several structural components at different locations. For example, if a connection subject to fatigue cracking is being studied, strain data from the members framing into that connection as well as strain data from the connection itself would be necessary. Based on this need and finances a decision was made to purchase an eight channel system. But the speed of the system also had a direct bearing on cost. If purely static load tests under controlled traffic conditions were to be done, an extremely simple system could be employed with each channel being read and recorded manually. Performing static load tests under controlled traffic conditions would mean closing the road and using a Department of Transportation truck of known weight. Closing the road was not a feasible solution for the high volume interstate system bridges of interest.

In a test like the one described above all channels would need to be read under the same load conditions for the data to be meaningful. If a static load test was not possible it appeared that manually reading the channels was out of the question. If the tests were to be performed under normal traffic conditions and all channels had to be read under the same load conditions, all channels would have to be read simultaneously. These conditions began to point to some type of computer controlled system. Often a computer is coupled with the Wheatstone bridge circuit and the signal conditioning equipment that is necessary to read strain gages. The computer can greatly simplify the use of the circuit and enhance the accuracy of the measurements. It eliminates the need to manually balance the bridge and, for multi-channel systems, handles the channel switching and data storage. A computer controlled system cannot read more than one channel at a time but the technology was available to read all channels fast enough so that, for the project's purpose, they appear to be read simultaneously. Subsequent investigation into this type of system revealed that a system with the desired speed was prohibitively expensive at the time the system was developed. A system that was slightly slower than optimal was selected due to economic restrictions. Recent advances in portable computers and data acquisition equipment could provide a much faster system at less cost.

The use of a computer controlled system eliminated several problems, as well as providing new opportunities. Manual recording of data can result in errors. Mistakes can be made when recording data on paper and mistakes can be made when keying the recorded data into a computer for analysis. The computer controlled system solved this problem by recording all data on magnetic media (floppy disks). This media could be transported from one computer system to another without the possibility of human induced error.

~~The computer control system also presented the opportunity to do~~ automatic data reduction in the field. Programs could be written or purchased to perform the desired manipulations automatically, immediately after the data was recorded. Because this project was in its early stages it was not readily apparent what type of data manipulation would be needed. Would the information be more useful to the user as strain data or stress data? Would graphical output or tabular format be more useful? Would any adjustments be needed for environmental factors (strain gage zero drift, temperature compensation, etc...)? Due to these uncertainties this option was put aside but the flexibility of the computer based system allows these changes to be implemented at any time.

Digital Or Analogue?

Once the decision had been made to purchase a computer controlled system another dilemma arose. Should this system be an analogue system or a digital system? A brief discussion of the advantages and disadvantages of each system is necessary.

Electrical resistance strain gages are analogue devices. They produce changes in electrical voltages which are in a continuous form. An analogue signal can be transmitted at very high speeds but is susceptible to electrical noise and interference which can create false readings. A digital signal, on the other hand, is composed of discrete values or impulses that are arranged in coded patterns. The technology is available to transmit these coded patterns over long distances with little chance of outside electrical distortion.

The noise reduction abilities of a digital system made it the obvious choice for this application. The standard serial communication protocol for this type of system has transmission speeds that are significantly slower than an analogue system but they were adequate for most bridge monitoring purposes. Figure 3.1 shows a schematic of the system.

As mentioned before strain gages produce an analogue signal. The hardware included an analogue-to-digital (A-D) converter that was located as close to the active strain gage as possible. This A-D converter would take the analogue strain gage signal and produce a digital (RS-485) signal that was transmitted to the vicinity of the portable computer. Before the computer received the RS-485 signal it was further converted to an RS-232 signal which is the standard serial communications protocol for todays' PCs.

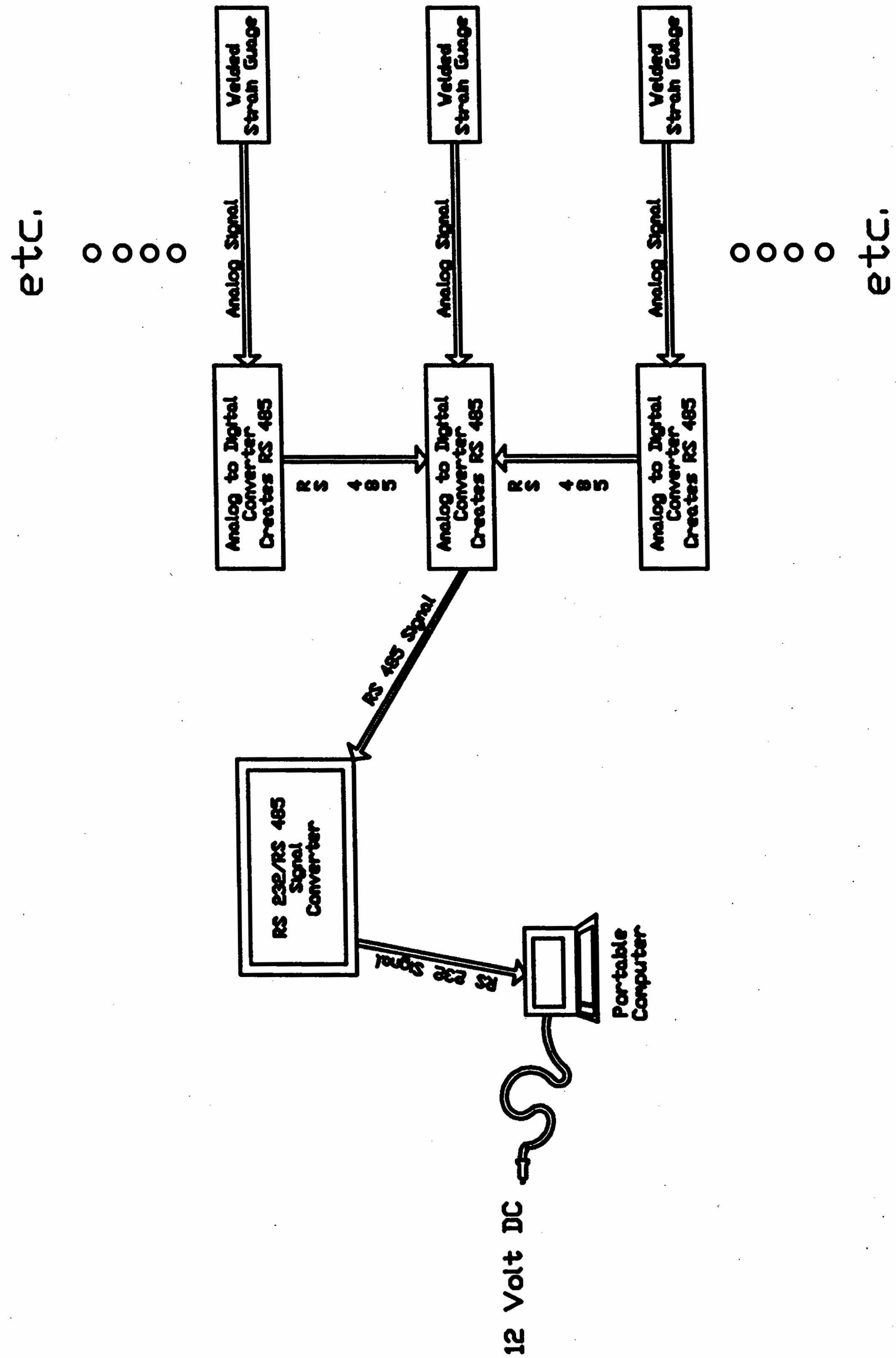

FIGURE 3.1: DATA ACQUISITION SYSTEM SCHEMATIC

Software Development

Any computer based data acquisition system requires some type of controlling software. The hardware that was purchased included proprietary software but it was found to be inadequate for the projects purpose. Commercial software packages were investigated but they were found to include many complicated features that were unnecessary for the project. The initial costs of these packages are high and most have a steep learning curve.

The control requirements for the hardware were relatively simple. The analogue-to-digital conversion modules are serial communications modules that receive and transmit ASCII commands. A program was written using one of the new generation executable Basic languages that allows the user to communicate with the modules through the keyboard. The program was written to be as user friendly as possible and to perform routine data gathering operations as transparently as possible. The user is required to input the number of channels, the frequency of the readings and the serial communications parameters (serial port, baud rate, parity, etc...). Once the program has this information it controls all monitoring and data storage operations until the user interrupts it.

Field Work & Data Reduction

As mentioned in Chapter 1 the subject bridge of this study is located on Interstate 95 in Bridgeport, Connecticut and serves as an overpass for the intersection of Bostwick Avenue and Cherry Street. Fortunately, a portion of the southbound lane of the bridge is over an abandoned parking lot and this section was found to be suitable for the tests (Figure 3.2). In this region, the superstructure is approximately 16 feet above grade. Even though this bridge does not have any permanent inspection catwalks the abandoned parking lot and the bridge's low elevation allowed easy access to the superstructure using ladders, staging and a hydraulic scissor lift platform.

The field monitoring system was used to examine both the original connection configuration and the proposed repair (tee-section) that had already been installed in several locations. Three individual test sessions were conducted. The sessions consisted of:

1. Full instrumentation of a K-frame type diaphragm, the two adjacent stringers and the connection between diaphragm and stringer. Gages were placed on the bottom flange of both stringers, on the diaphragm channel top flange and web, on the diaphragm angle and on the connection angle in two locations.

2. Full instrumentation of a channel type diaphragm, the two adjacent stringers and the connection between diaphragm and

stringer. Gages were placed on the bottom flange of both stringers, on the diaphragm channel top flange and web and on the connection angle in two locations.

3. Instrumentation of a tee section repair. Two gages were placed on a tee in a K-frame type diaphragm and two gages were placed on a tee in a channel type diaphragm.

The raw data, which is stored in ASCII format by the data acquisition system, was transferred to a spreadsheet for analysis. The maximum strains for each configuration were extracted from the data and this information was used to correlate the finite element models that will be discussed in the next chapter. For a more detailed look at this procedure see reference 19.

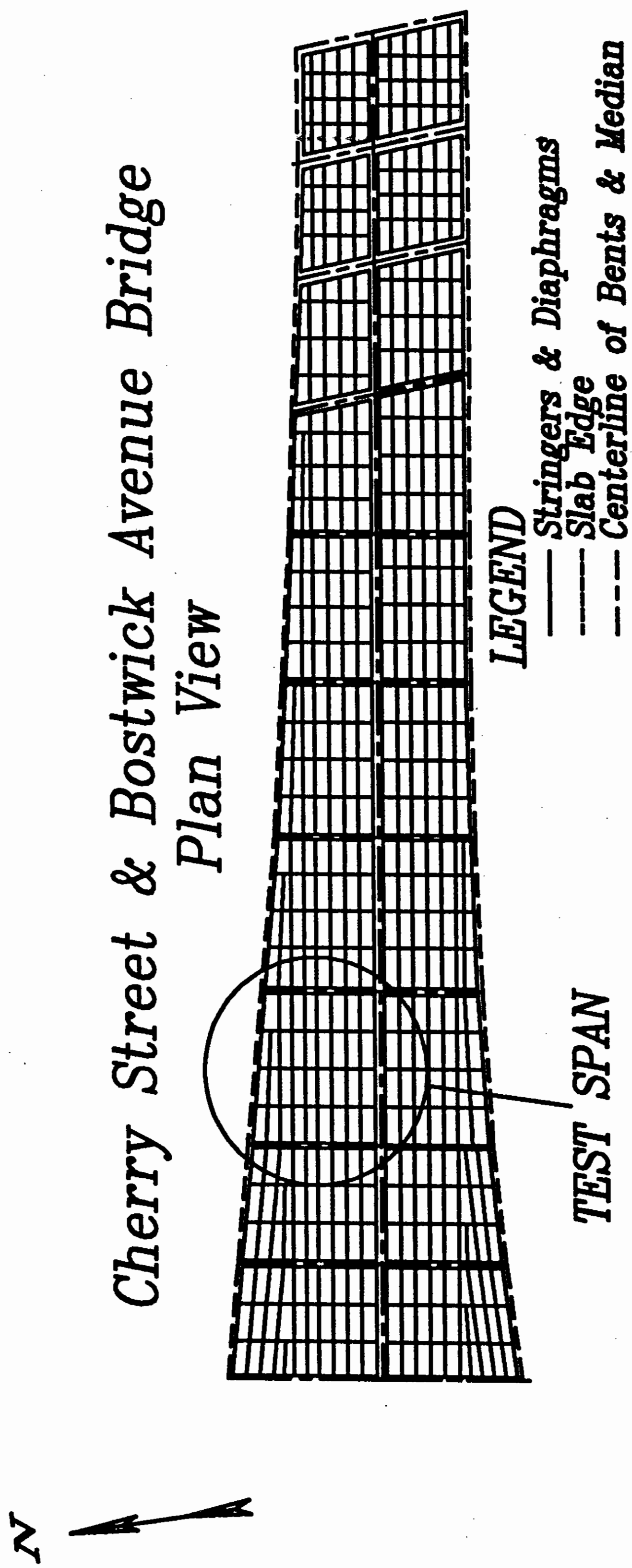

FIGURE 3.2: BRIDGE #101 PLAN VIEW SHOWING TEST SPAN

CHAPTER 4
ANALYTICAL STUDY

Introduction

Several finite element models of the bridge and its various components were constructed during the course of the study. The results of the field work were used to correlate and "fine tune" the finite element models. The analytical study is reported in more detail in reference 19.

Global Bridge Model

The initial model was a global model of a typical simple span from the southbound lanes of bridge #101 (Figure 3.2). The purpose of this model was two-fold: 1) to examine the overall behavior of the system under simulated traffic conditions, and 2) to determine the magnitude of the forces that the stringers and diaphragms induced on the connections in question.

The preliminary work on this bridge indicated that the structure may not be behaving in the way that the design engineers anticipated. In a steel stringer - composite concrete slab bridge, any diaphragms that were present were thought to be performing the following services:

1) During erection the diaphragms provided stability to the stringers before the deck slab was cast.

2) While the deck was being cast the diaphragms braced the compression flange of the stringers against buckling. After the concrete was cured the composite deck would provide any needed lateral bracing for the stringers.

3) It was thought that the diaphragms would distribute wind loads from the exterior stringers to the interior stringers. This would be true before the deck was cast but after the deck was in place it would provide all of the wind load transfer necessary. From the viewpoint of lateral loading the deck would act like a deep beam or a plate subjected to edge loading.

Results

The data from the strain gage monitoring of bridge #101 were used to correlate the finite element model. Available design information on the bridge is minimal so several assumptions had to be made during model development.

These included items such as concrete strength, degree of connection fixity, etc... Minor adjustments were made to these parameters based on the strain gage data until the model behaved similarly to what the field data indicated. Once the model was correlated to the field data it was used to test five different diaphragm configurations:

1) All diaphragms in place (the actual situation).

2) All diaphragms removed - the model consisted of just the ten stringers and the deck.

3) Midspan and support diaphragms only - the quarter point diaphragms were removed.

4) Quarter point and support diaphragms only - the midspan diaphragms were removed.

5) All angle members were removed from the K-frame (see Figure 1.3) style diaphragms. All diaphragms were made up of channel sections only (see Figure 1.1).

For each of these diaphragm configurations four items were examined:

1) The stringer deflections at midspan.

2) The stringer bottom flange stresses at midspan.

3) The deck stresses in the longitudinal (parallel to traffic) direction.

4) The deck stresses in the transverse (perpendicular to traffic) direction.

Two load cases were used: (see Figure 4.1) a moving HS 20-44 load on stringer E (approximately the middle of the right travel lane) and a moving HS 20-44 load on stringer I (approximately under the top of the exit ramp that begins on this span). These are felt to be the loading patterns that induce the largest forces in the diaphragms.

Figure 4.2 shows the maximum deflections of the stringers at bridge midspan (approximately 48 feet from each support) for a moving load on stringer E. Figure 4.3 shows the corresponding deflections for a load on stringer I. Both figures indicate that the type, quantity and location of the diaphragms has little effect on the midspan deflections. This would indicate that the deck slab is providing most of the lateral load distribution to adjacent stringers.

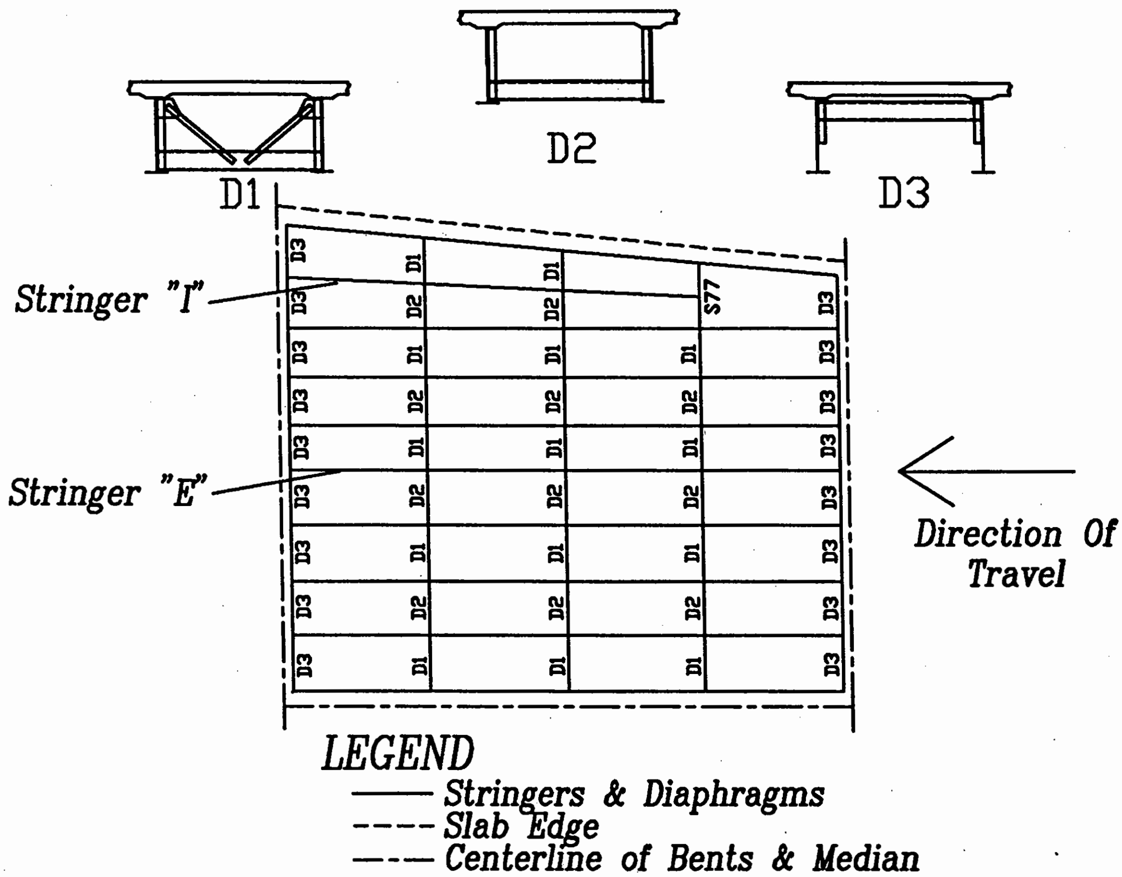

FIGURE 4.1: PLAN VIEW OF SPAN SHOWING LOADING.

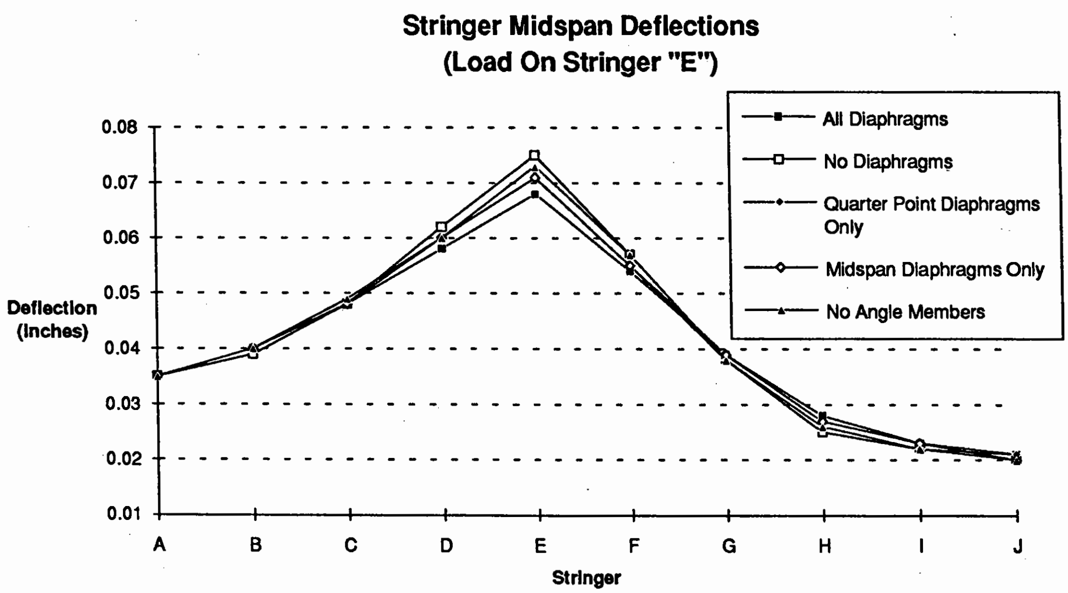

FIGURE 4.2: STRINGER MIDSPAN DEFLECTIONS (LOAD ON STRINGER "E")

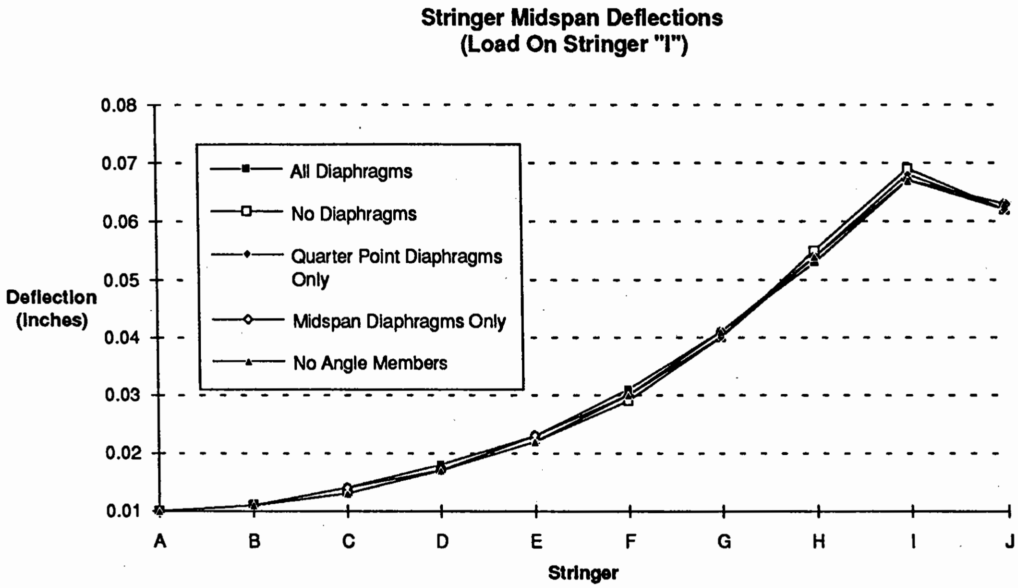

FIGURE 4.3: STRINGER MIDSPAN DEFLECTIONS (LOAD ON STRINGER "I")

While running the tests the prime concern was the effect that removal (or a change in configuration) of the diaphragms would have on the stringer deflections. A secondary (but no less critical) concern would be the change in stress levels for the structural members. The concept of removing the diaphragms would be senseless if it would overstress some other component.

As mentioned above the stress levels were monitored in the stringers and the slab during the finite element model tests. Figures 4.4 through 4.13 show the stringer and deck stresses for all diaphragm configurations. The figures show that the increase in stresses due to a change in diaphragm configuration is negligible in all situations and locations.

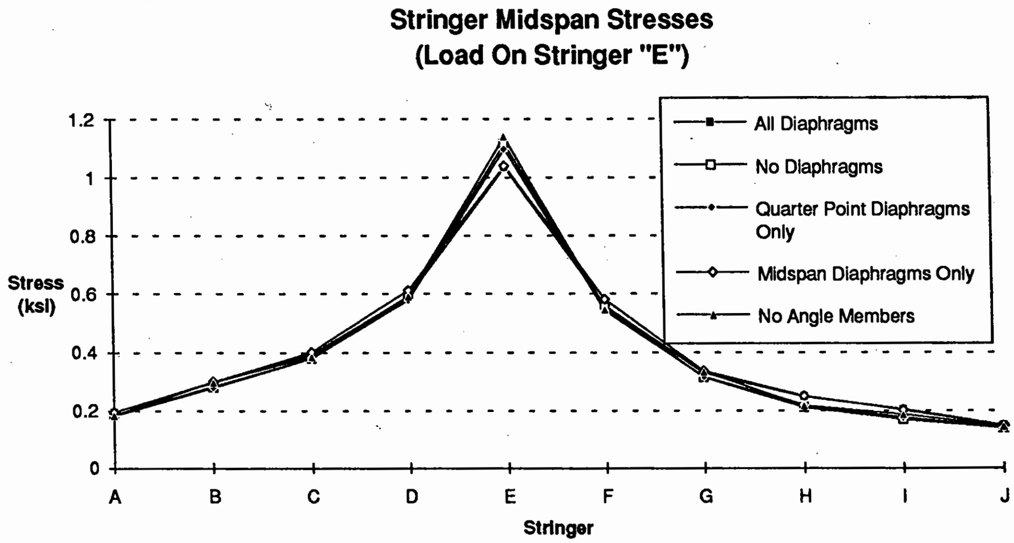

FIGURE 4.4: STRINGER MIDSPAN STRESSES (LOAD ON STRINGER "E")

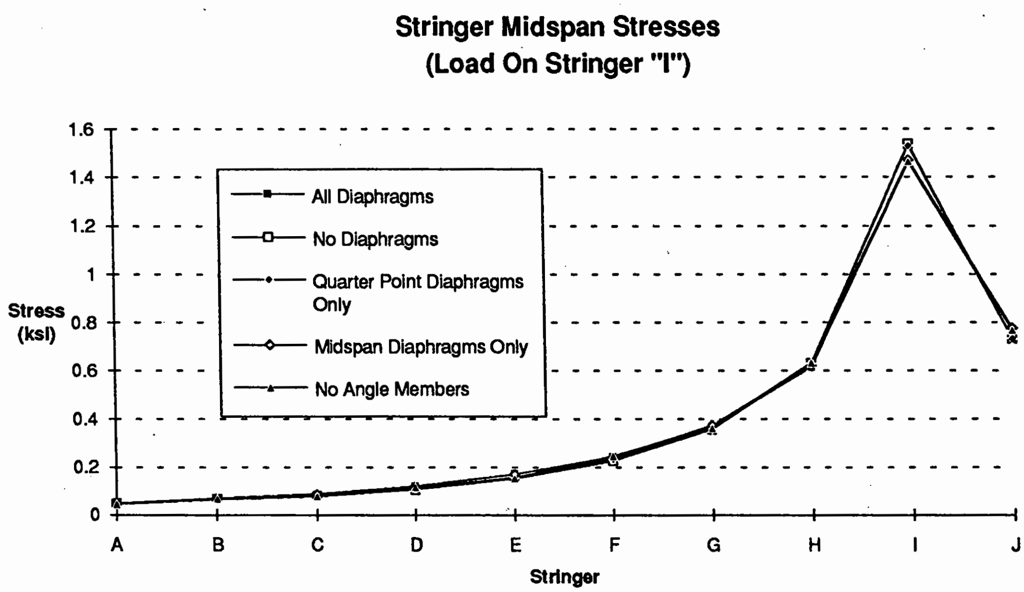

FIGURE 4.5: STRINGER MIDSPAN STRESSES (LOAD ON STRINGER "I")

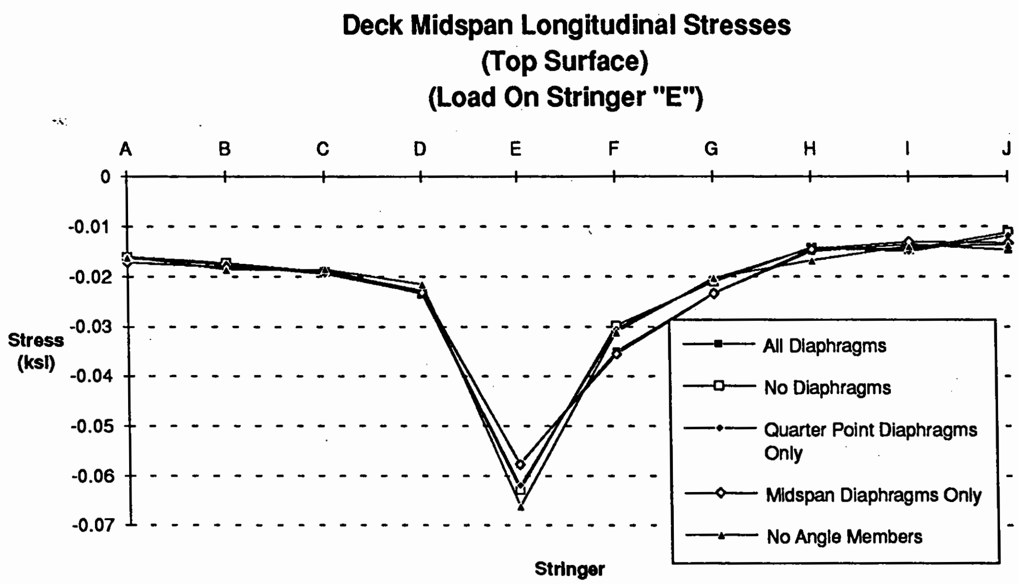

FIGURE 4.6: LONGITUDINAL DECK STRESSES: TOP SURFACE (LOAD ON STRINGER "E")

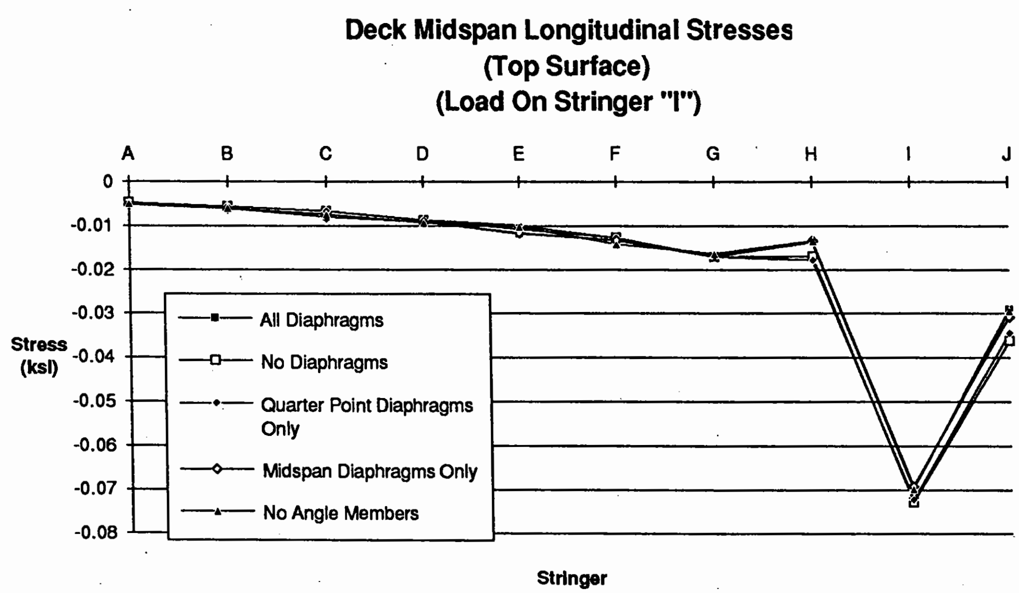

FIGURE 4.7: LONGITUDINAL DECK STRESSES: TOP SURFACE (LOAD ON STRINGER "I")

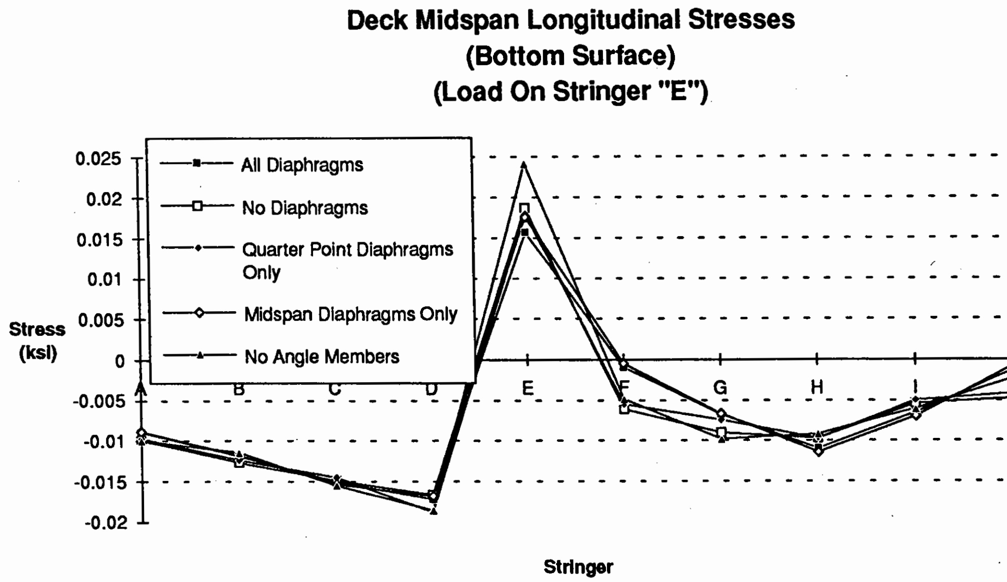

FIGURE 4.8: LONGITUDINAL DECK STRESSES: BOTTOM SURFACE (LOAD ON STRINGER "E")

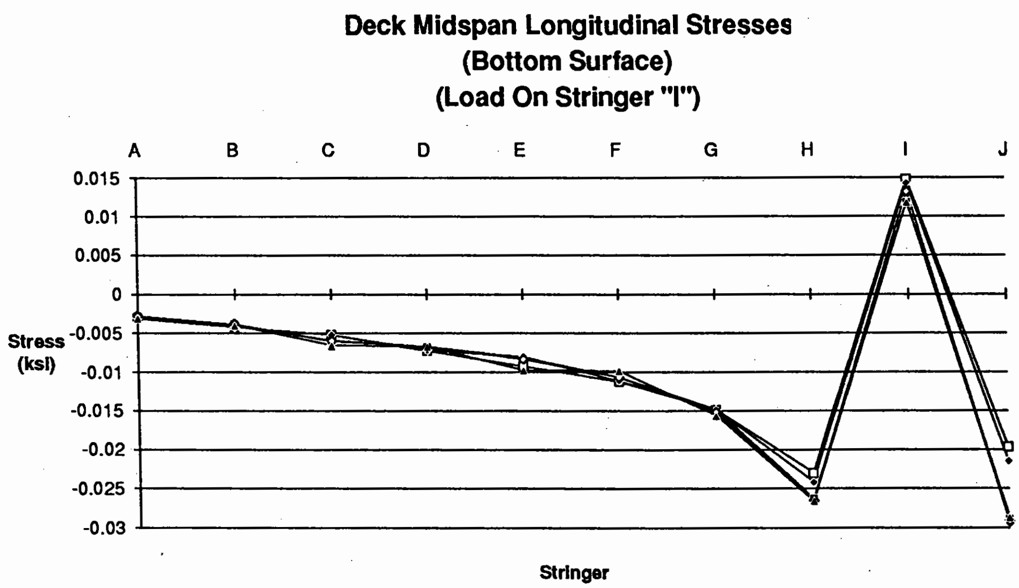

FIGURE 4.9: LONGITUDINAL DECK STRESSES: BOTTOM SURFACE (LOAD ON STRINGER "I")

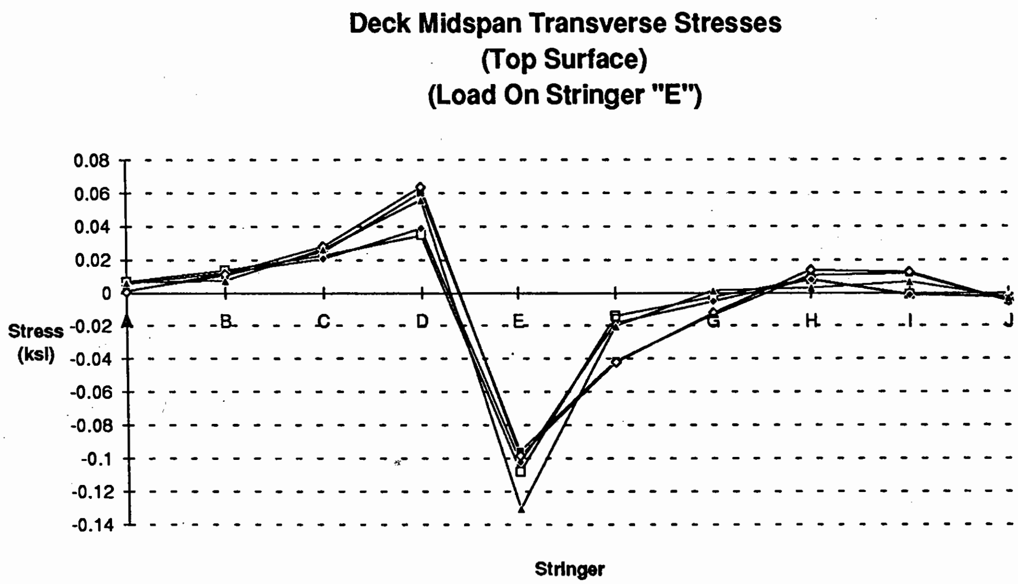

FIGURE 4.10: TRANSVERSE DECK STRESSES: TOP SURFACE (LOAD ON STRINGER "E")

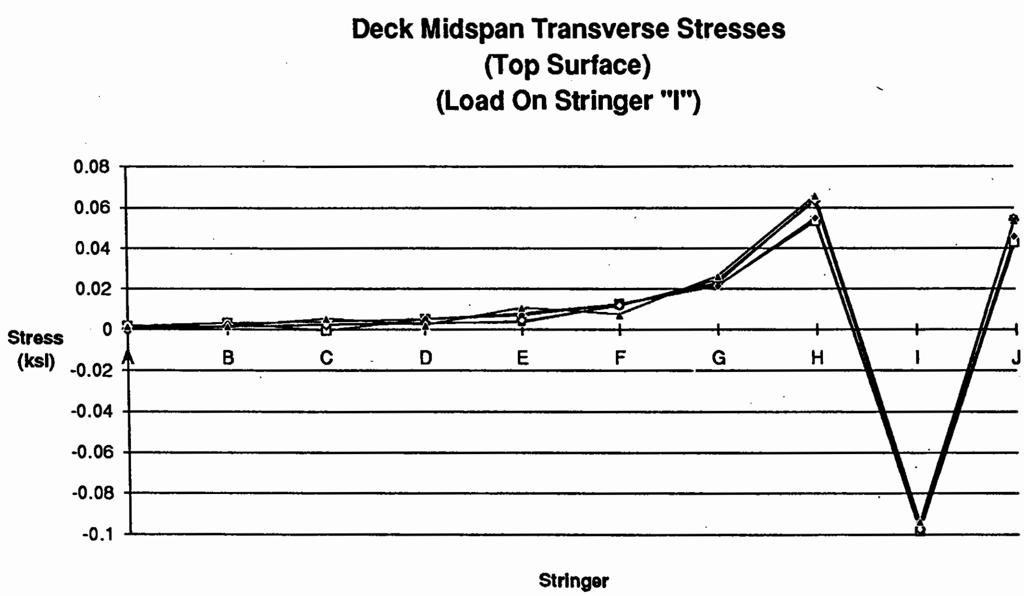

FIGURE 4.11: TRANSVERSE DECK STRESSES: TOP SURFACE (LOAD ON STRINGER "I")

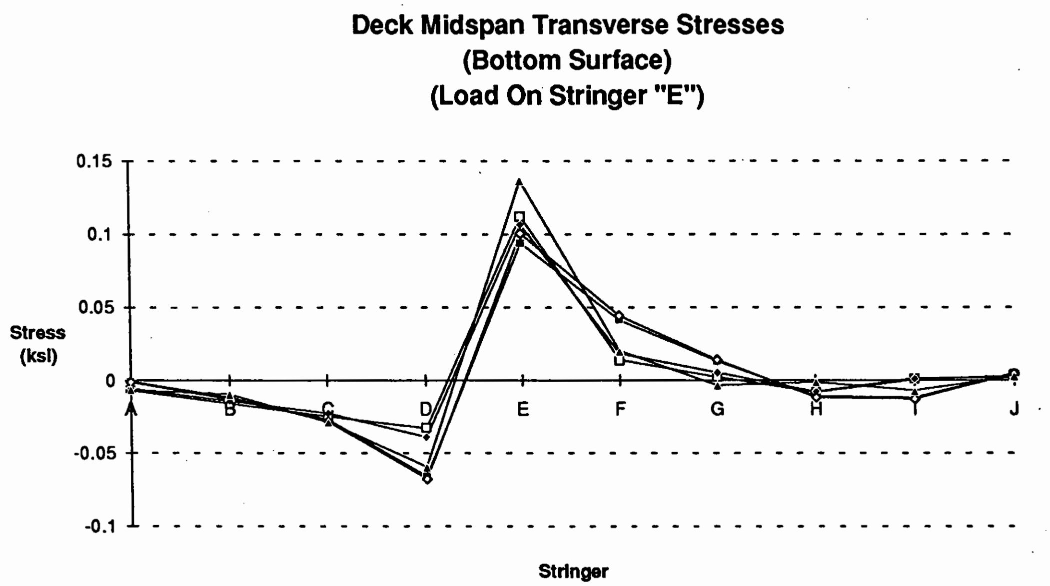

FIGURE 4.12: TRANSVERSE DECK STRESSES: BOTTOM SURFACE (LOAD ON STRINGER "E")

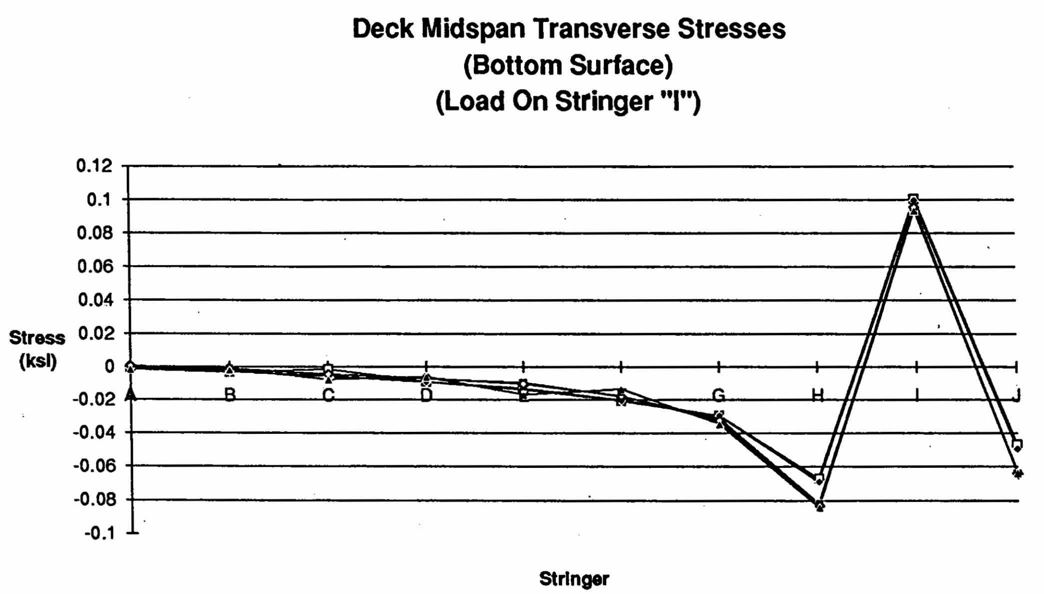

FIGURE 4.13: TRANSVERSE DECK STRESSES: BOTTOM SURFACE (LOAD ON STRINGER "I")

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

The main causes of bridge #101's problems appear to be inadequate fatigue design of connection details and an over simplification of the structural system. The oversimplification was necessary because of the design tools that were available at the time of design (mid - 1950's) and because of the three-dimensional behavior of the bridge. The complex three-dimensional behavior causes secondary stresses that are usually neglected in design practice.

The differential deflection between adjacent stringers under live load appears to be much larger than the original designers had anticipated. This deflection is causing greater than expected loadings in the diaphragms. The connections between the stringers and the diaphragms were not designed to transmit loads of this magnitude and still remain below their fatigue limit. Thus the connection angles are experiencing stresses that are above their fatigue limit, and therefore their fatigue life is limited. These stress levels combined with the large number of loading cycles that bridge #101 has experienced since its construction appear to be causing the fatigue cracking.

The test results discussed in the previous chapters indicate that the bridge behavior is different than the designers thought. The composite slab accounts for the majority (> 90%) of the lateral distribution of the live load. The diaphragm configuration (location, quantity and type) appears to have little influence on the load distribution characteristics.

The results indicate that the connection cracking situation will not present any long term performance problems for bridge #101. The diaphragms were necessary only during the construction phase for overall stability of the superstructure and to provide lateral bracing to the compression flange while the concrete deck was cast. Once the deck cured, it assumed all functions previously assigned to the diaphragms in addition to providing distribution of live and wind loads. Should the deck need replacement, the diaphragms will again provide these functions.

Replacing the cracked connection angles with the proposed tee section does not appear to be necessary. Many of the connection angles that are not cracked at midspan and quarter span will crack sometime in the future. Because of the behavior of the bridge these additional cracks will not be a problem.

As mentioned earlier some of the more severe cracked angles (cracks > 6" in length) have already been replaced with the tee section. Strain data that was gathered from these tee sections were compared to strain data from the original angle configuration under a similar loading. The stress levels in the tee sections are sufficiently low to give the tees infinite fatigue life. If ConnDOT chooses to follow through with the proposed replacements, the WT 5x22.5 sections appear to be adequate.

Future Research Possibilities

Future research in this area should include the following:

1) Testing of alternative diaphragm arrangements and types. The diaphragm configuration should provide the needed support during construction and not be susceptible to fatigue problems during its service life.

2) Examining the influence that the deck has on the behavior. What is the optimum deck thickness for both load distribution and economy?

3) Development of a system that eliminates the diaphragms completely. Is such a system feasible given today's construction techniques?

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