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I am submitting herewith a thesis written by Perry Cooper Patrick entitled "Comparison of Measured and Theoretical Stresses in a Simple Span Bridge." I recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

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COMPARISON OF MEASURED AND THEORETICAL STRESSES IN A SIMPLE SPAN BRIDGE

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee

Perry Cooper Patrick

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ABSTRACT

The purpose of this study was to compare the measured and theoretical stresses in a simple-span highway bridge and to work toward establishing a method to predict the number of stress fatigue cycles in a bridge girder.

Two highway bridges, part of the interstate system, were selected and instrumented for the collection of data. A Raytheon 703 computer with a magnetic tape unit, housed in a trailer, was used for the collection of stress data which was stored on magnetic tapes. Data concerning the classification of trucks was taken at the bridge site and at an official weigh station. Several controlled load tests, using a truck of known weight and axle spacing, were also conducted. The field stress data was separated into truck types, and histograms of the girder stresses were formed for each type. A finite element model of the bridge was used to formulate equations to predict the theoretical stress caused by a truck on the bridge. A Monte Carlo simulation using random numbers to select weights and spacings from their respective distributions was used to generate theoretical stress histograms for 3S2 and 2D trucks. The histograms of the measured and theoretical girder stresses were then compared for the two truck types.

It was found that the two stress histograms for 3S2 trucks did not compare favorably. The measured and

actual stress histograms for 2D trucks were very similar. Because of the wide variation between theoretical stresses and the stresses from the controlled load tests, no attempt was made to draw any conclusions from the comparison of the stress histograms. Further research was suggested to obtain a one truck-one stress-one weight relationship in order to get a better comparison between measured and theoretical stress distributions. Once these distributions compare favorably, a method is available to predict the number of stress fatigue cycles in bridge girders.

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CHAPTER I

INTRODUCTION AND OBJECTIVE

1. INTRODUCTION

The Federal Highway Administration (FHWA) has funded and promoted research for the past six years on stress and loading histories of highway bridges in an attempt to better define the magnitude and number of fatigue cycles that bridge girders are undergoing. With fatigue being pinpointed as possibly a major factor in the catastrophic collapse of the Silver Bridge between Ohio and West Virginia in December, 1967, the FHWA decided to implement the program of research in order to more accurately predict the life expectancy of highway bridges. Increasingly heavy and more frequent truck loads were singled out as a prime contributor in the fatigue failure of the Silver Bridge and other highway bridge fatigue failures. The effect of a large number of load applications causing low stresses obviously cannot be ignored in the fatigue analysis of girders.

Because of dynamic effects and the complexity of the structures, actual field testing of bridges was considered the only way to give an accurate description of stresses in the structure. Since large numbers of stresses and the corresponding truck types and weights needed to be recorded, several studies involved in testing, including this study,

utilized a computerized data collection system to record and reduce the stress data. Early results from most of the studies gave no indication that fatigue should be a factor in the design of the girders. For example, earlier research in this study produced girder life expectancies due to fatigue of around four hundred years.[1*] Impetus was added to the stress history studies, however, when fatigue cracks were discovered in the girders of the Yellow Mill Pond Bridge on the Connecticut Turnpike in late 1970. Several other bridges with fatigue cracks have been discovered, among which are the Lehigh River and Lehigh Canal Bridges on U.S. Route 22 in Pennsylvania and the Allegheny River Bridge on the Pennsylvania Turnpike.[2] The stress history studies are continuing in an effort to possibly explain the above mentioned failures and to further our knowledge in this area so that future bridge designs will be designed adequately for fatigue.

2. OBJECTIVE

The "heart" of an analysis for fatigue failure of a bridge girder is to predict the number of load cycles that a girder will undergo. A bridge can be properly instrumented and tested, and the resulting stress distribution can be used to predict the life of the girders if assumptions are made

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*Superscript numbers refer to similarly numbered entries in the List of References.

about the corresponding truck traffic. To apply this to design, however, a relationship is desired to link the results from the tested bridge to a bridge that most probably has a completely different cross-section and different traffic characteristics. One way to solve this problem is to relate the measured stresses to the theoretical stresses from an analysis of the tested bridge, using the weight data for the traffic distribution of the bridge loads. Thus, a theoretical-to-actual stress relationship that can be applied to other bridge cross-sections is developed. The object of this study was to pursue this method of determining the magnitude of stresses a bridge will experience for a given traffic distribution.

3. SCOPE

In order to determine the theoretical-to-actual stress relationship, a three-lane steel girder bridge in Knoxville, Tennessee, was instrumented with strain gages and tested under actual truck loadings. A finite element model of the bridge was then formulated and "loaded" with trucks of varying gross weights, axle weight distributions, and axle spacings.

Two types of actual stress data were taken. Individual data consisted of recording the stresses on magnetic tape for a given truck, with truck type and lane position being recorded by hand. This allows a one truck type - one stress range correlation, which lends itself more to the purposes of this study than does the other type, continuous sampling.

The individual stress data were reduced and recorded on a teletype printout, from which they were then separated according to truck type and lane position.

The theoretical model of the bridge was a finite element model formulated in a manner compatible with the ICES STRUDL computer program. Using this model, the axle spacings and axle weight distributions were varied for trucks in various transverse positions on the bridge. These results were then used to formulate equations which describe the stress at the longitudinal centerline of the bridge in terms of the weight and spacing characteristics of a truck.

Since the weights were not available for the particular population of trucks included in the field testing, a one truck - one actual stress relationship was not possible for most of the data. A one truck weight - one stress relationship was obtained from a special test that was performed in which a truck of known weight and spacing characteristics was driven over the bridge at varying speeds in each of the three traffic lanes. Previously recorded truck weights from a state weigh station ten miles from the bridge site were assumed for the weight distribution of the trucks on the bridge. To convert these weights to theoretical stresses, a Monte-Carlo type simulation was used in which a truck weight, axle weight distribution, and axle spacing were randomly selected from arrays with these values tabulated in them. Once these values relative to the load were obtained, they

could be converted to stresses, and a theoretical-to-actual stress relationship could be formed that would be applicable to bridges having different cross-sections.

CHAPTER II

REVIEW OF RELATED RESEARCH

1. REVIEW

The stress history program has been actively promoted by the Federal Highway Administration since 1969. Research is guided by committees of the American Society of Civil Engineers (ASCE) and the Transportation Research Board (TRB). Field testing of bridges is considered the key to defining fatigue damage occurring to girders in highway bridges. In order to correlate data from the several stress history studies being promoted, a standard classification of trucks, as shown in Figure 1, was developed.

Several states are presently engaged in stress history studies. Presented herein is a review of the work of four of those states, including a review of previous work on this study at Tennessee.

Ohio

The research in Ohio was performed by the Department of Solid Mechanics, Structures and Mechanical Design of Case Western Reserve University.[3] In this study, ten girder-slab bridges were chosen on a statistical sampling basis which represented the bridges in Ohio with regard to design details, configuration, truck traffic volume, and other pertinent design

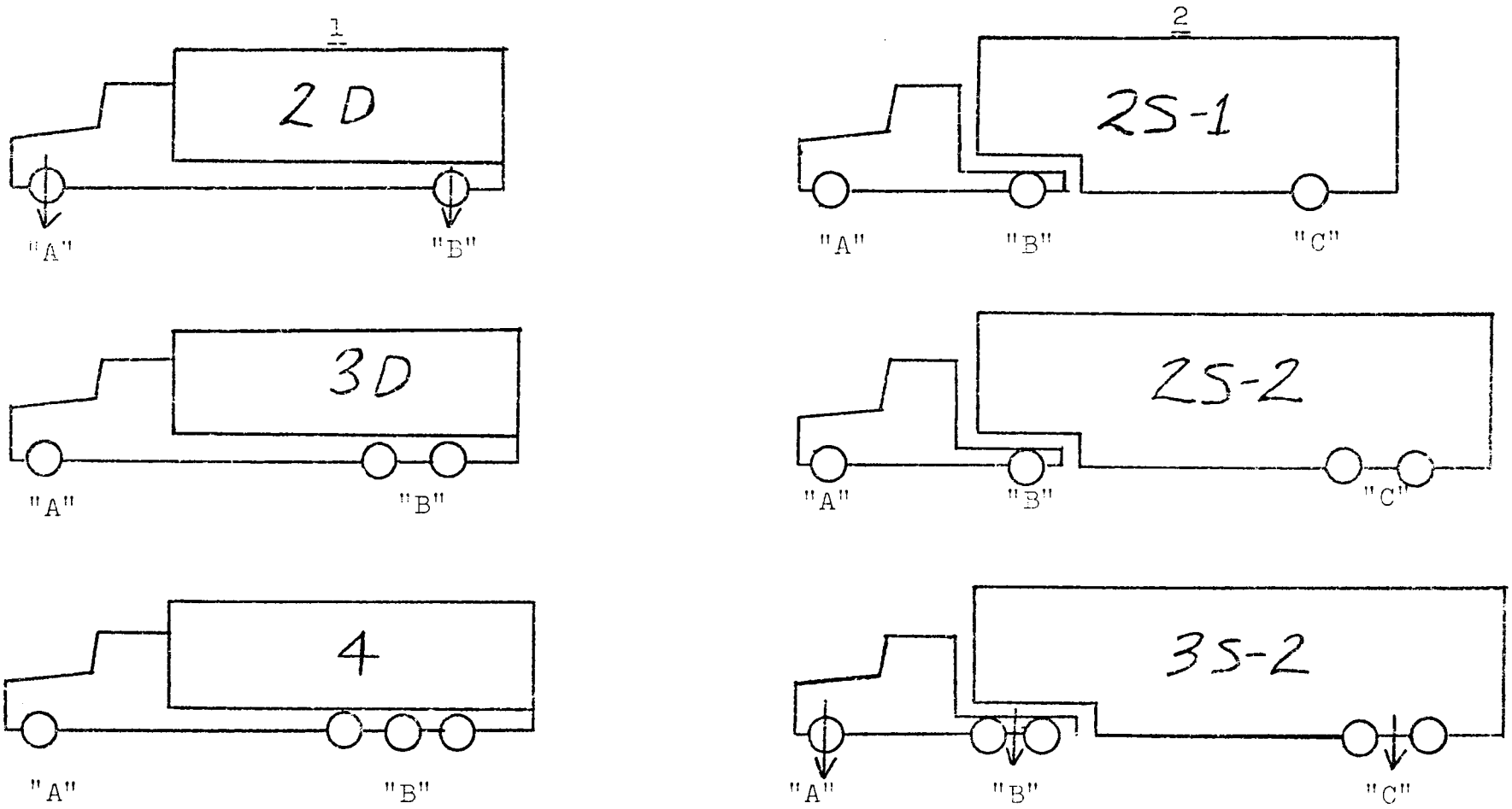


Figure 1. Axle Type Designations.

considerations. Over 20,000 truck passages were processed, with strain range, spacing between successive trucks (headway), and lane location being recorded for each passage. These field data were used to give approximate values for variables that appear in equations that are discussed below.

This report presents a fatigue-based solution for the design of bridge girders. Using the Miner linear damage theory, an equation is derived that will predict the allowable stress range of a girder given the total number of load cycles the girder will undergo and information on truck weight distribution. The equation is as follows:

$$S_r \text{ allow} = \frac{S_2 \times 10^6}{K\ N_m} \left(\frac{2 \times 10^6}{V}\right)^{1/3} L^{-1/3}$$

where

S_r allow = the allowable stress range used to proportion the girder

$S_2 \times 10^6$ = the base metal stress range for two million cycles (50% confidence level)

K = a stress concentration factor for the specified weld detail

N_m = safety factor to correspond to acceptable level of risk of premature damage

V = total number of load cycles

$$L = \sum_i \left(\frac{W_i}{W_d}\right)^3 f(W_i)$$

where

W_i = truck weight corresponding to stress range, S_i

W_d = design vehicle truck weight

$f(W_i)$ = frequency of occurrence of weight, W_i

Values for the variables "V" and "L" can be found from traffic studies and loadometer studies of truck weights from the proposed bridge site. The variable, $S_2 \times 10^6$, can be found from an S-N fatigue life curve for the girder metal. Values of K are suggested in the report for such stress raisers as coverplate termination (3.5), welded flange splices (1.6), and welded beams and girders (1.4). These values are based on a value of 1.0 for rolled sections. Values for other stress raising attachments are referred to extensive laboratory tests performed at Lehigh University.[4] The study gives no specific values for N_m, but suggests that this variable can be used to keep future designs in line with current designs, which are known to be generally safe.

Once the allowable stress on the girder is known, the section modulus must be checked. An equation to do this is presented below:

$$S_{girder} = \frac{M_{girder}}{S} = M_r \, g^* \, h^* \, I^* \, N_1$$

where

S_{girder} = the design stress at the point being checked

M_{girder} = the bending moment range at this location on the girder

S = section modulus

M_r = the bending moment range due to the design vehicle on the total bridge cross-section

g^* = girder distribution factor for one lane road

h^* = a magnification quantity to represent the effect on loading of short headway spacings

I^* = impact factor

N_1 = a load safety factor to account for increase in truck weights and any anaylsis errors

The results from the field testing mentioned previously on the ten girder-slab bridges were used to obtain values for some of the variables listed above. A value for g^* of S/12 is suggested for girder-slab bridges, where S is the spacing in feet between girders. Values for h^* are still under study, but a figure between 1.2 - 1.4 should be expected. No field information was available on impact, I^*, but the AASHTO value of $50/(L + 125)$, where L is the span length in feet, was considered adequate. The load safety factor, N_1, should have a value of about 1.5, although more research into the legal and economic aspects of this needs to be done.

Results of the field testing in Ohio showed that measured stresses due to random truck traffic are considerably below the fatigue design allowable values. It concluded that fatigue design loadings of bridge girders should be separated from ultimate design loadings, since fatigue loadings are an averaging process of millions of cycles, whereas the ultimate design loading uses an extremely rare truck that will occur only a few times during the life of the structure.

Illinois

This research was performed by the Department of Civil Engineering in the Engineering Experiment Station, University of Illinois at Urbana-Champaign as part of the Illinois Cooperative Highway Research Program.[5] The work is being done with the cooperation of the Illinois Department of Transportation and the FHWA. This report addresses the stress history problem as one of calculating the mean stress and the related stress variance that is expected on a bridge. It considers the determination of girder stress for any given truck as a function of four variables, three of which have a distribution function associated with them. The equation itself is a simple straightforward one and is as follows:

$$\sigma = KG(x\,y)\,GVW(1 + D)$$

where

σ = stress at the point of interest

K = structural coefficient which is a function of the location at which the stress is computed and of the geometry and type of bridge

$G(x_1y)$ = static influence surface for the stress component being studied, a function of the lateral and longitudinal position of the truck on the bridge

GVW = Gross Vehicle Weight

D = Dynamic amplification of response

The variable K is a function of bridge type and geometry. For a single load on a simply supported beam,

K would be equal to (L/4)(C/I) for a stress at midspan. For a girder-slab bridge, the variable $G(x_1y)$ may be simplified to a single parameter describing the proportion of the total moment at the cross-section of interest that is distributed to the girder being considered. A variable degree of composite action causes the percentage of moment carried by a particular beam to be a random variable. Recent studies[6] have verified this variable composite action, and a Gaussian, or normal, distribution, with the mean value as that predicted by static theory and a coefficient of variation of 0.15 or less, describes the randomness of the parameter. Although not reported in this study, research conducted in Tennessee[7] showed no indication of a variable degree of composite action for bridges tested up to their ultimate capacity. This was true even for a bridge that was designed to act non-compositely. Obviously more research needs to be done to determine how to handle the distribution of moments to the girders.

The term D is similar to the usual dynamic impact factor. Instead of using a simple formula for it, however, this study expresses it as a random variable which considers several contributing factors such as roadway roughness, vehicle suspension conditions, and initial vehicle oscillation. It is suggested that a suitable normal distribution and its mean and variance may be found for D from researching literature on analytical results of the problem, field

measurements, or a computer simulation. Gross vehicle weight is treated as having a linear effect on stress, which is a somewhat intuitive conclusion but has also been proven by field measurements. A distribution for gross vehicle weight may be found from loadometer studies or from state weighing stations. Truck weight histograms usually exhibit a strong bimodal shape, which is partially due to the fact that five axle vehicles, 3S2's, dominate truck traffic and are usually either fully loaded (70,000 lbs) or lightly loaded (30,000 lbs). Figure 2a shows a truck weight histogram from a study in Illinois.[8]

Once all the variables in the equation above are defined with a suitable distribution, the mean stress and its variance can be calculated. The equation above may be rewritten as:

$$\sigma = g(X_i)$$

where

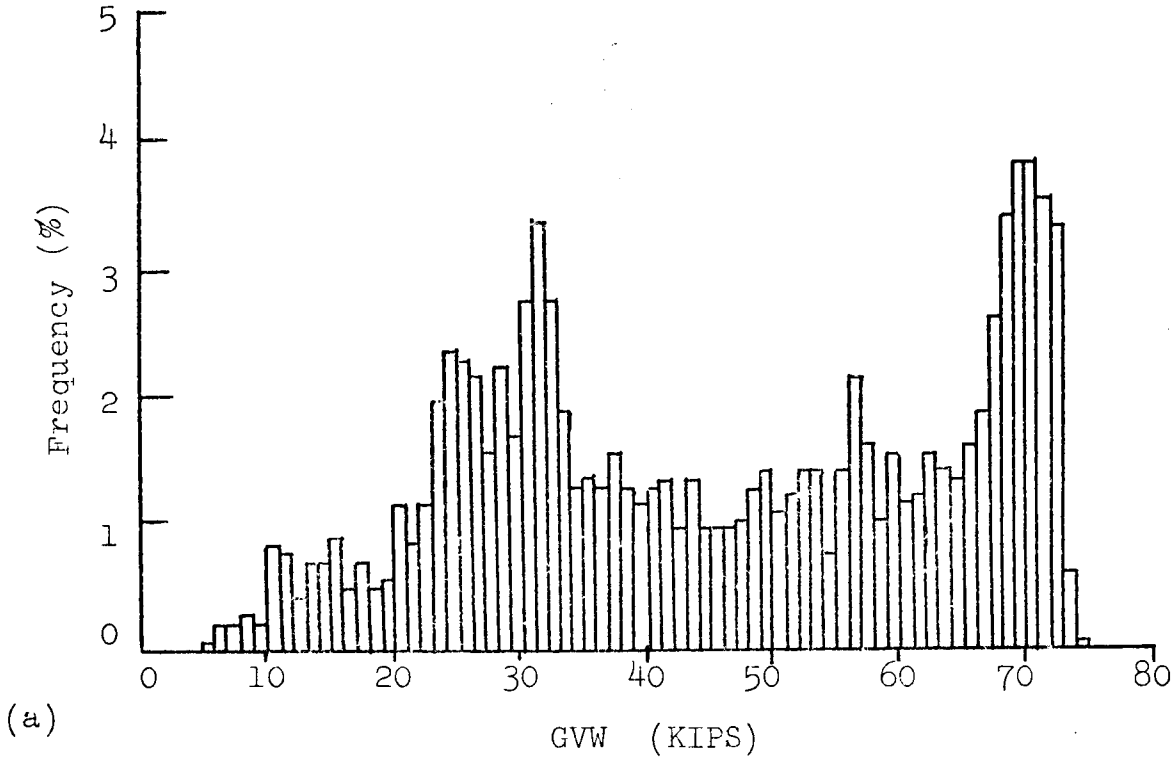
X_i = various parameters of the stress equation, such as D, GVW, etc.

Using standard techniques of uncertainty analysis, equations for the mean stress and the associated variance can be rewritten as follows:

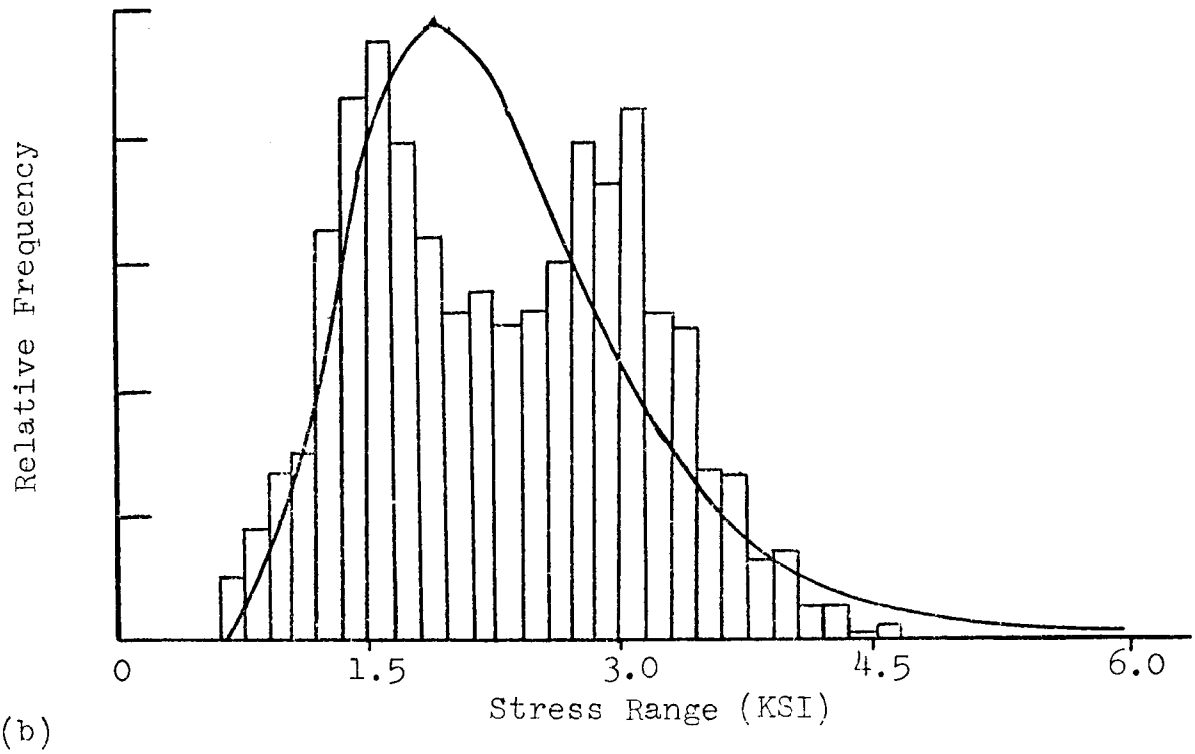
$$\bar{\sigma} = g(\bar{X}_i)$$

$$V_\sigma = \sum_{\text{all } i} \left(\frac{\partial g^2}{\partial X_i}\right) V_{X_i}$$

where



(a)



(b)

Figure 2. Histograms of Gross Vehicle Weight and Stress Range in a Bridge Girder.

$\overline{X}_i$ = mean value of parameter, X_i

$V_{xi} = S_{xi}^2$, variance of the parameter, X_i

A probability density function will also need to be specified to describe the stress histogram. A beta (β) distribution is suggested for this purpose.

A stress histogram such as that shown in Figure 2b can be expected for a girder near the path of travel of the trucks. The smoothed-out curve is typical of the stresses in a girder more distant from the path of the trucks. The histogram of stresses in Figure 2b is similar to that for weights in Figure 2a. Studies have shown that the coefficients of variation for stresses and weights to be approximately equal, which follows from the fact that gross vehicle weight is the dominant factor in inducing stresses.

The work done in this report is supported by field studies being conducted at the University of Illinois at Urbana-Champaign. However, more research needs to be done in defining the distributions representing the various factors causing stresses.

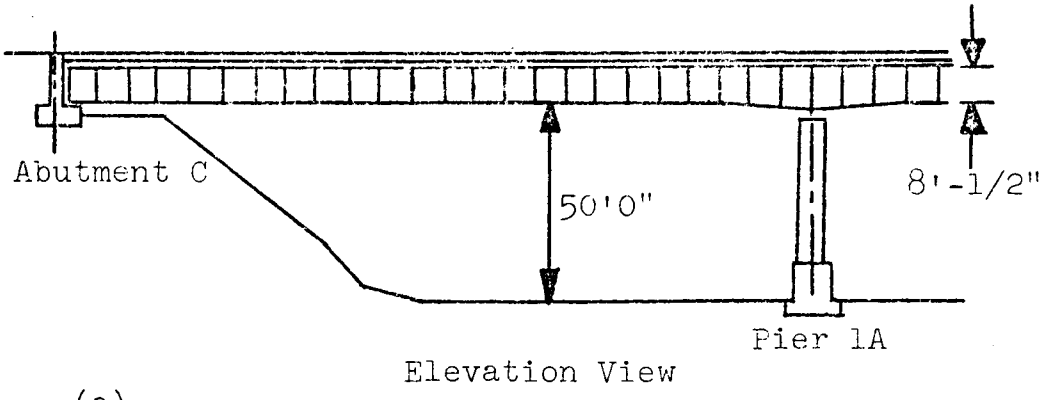
Pennsylvania

This investigation[2] is part of a research project being conducted at Fritz Engineering Laboratory, Lehigh University, under the sponsorship of the Pennsylvania Department of Transportation and the FHWA. Inspection of the Lehigh Canal Bridge by the Pennsylvania Department of

Transportation in 1972 revealed cracks in tie plates riveted to the longitudinal girders that covered the three spans of the bridge. A pilot program of field testing was undertaken to explore the fatigue behavior of bridge details under low stress, high cycle loading, and to correlate the stress history of a bridge with laboratory fatigue data.

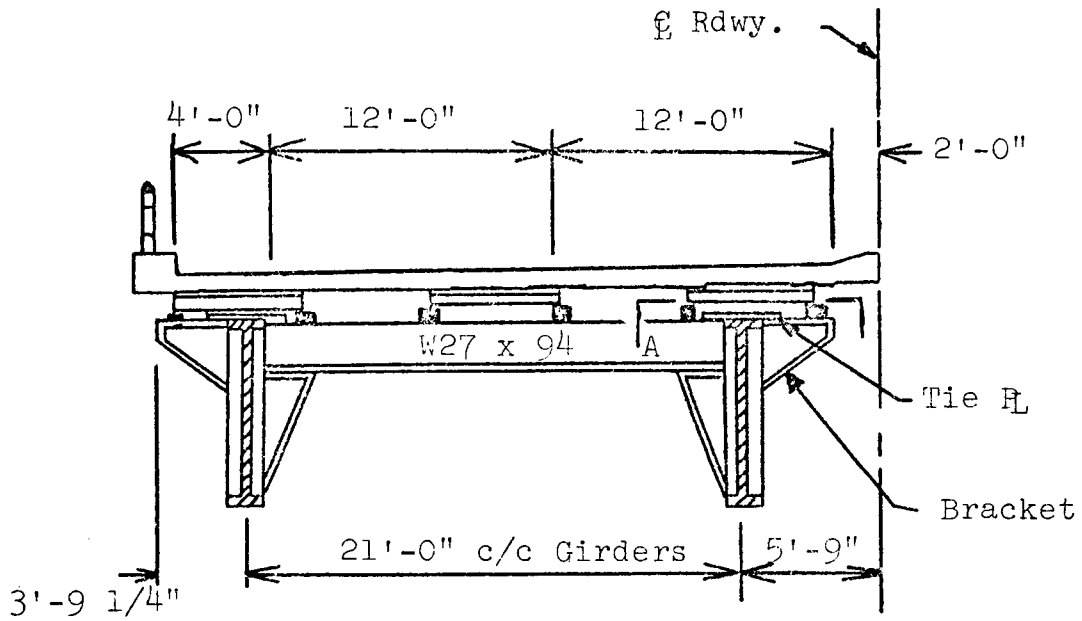
Figure 3a shows an elevation view of the span that was tested. Figure 3b shows a cross section of the bridge with the location of the tie plates. Electrical resistance gages were attached to tie plates, the main longitudinal girders, and to a stringer beam. The FHWA automatic data acquisition system and an analog trace recorder were used to collect the stress data. This allowed both a histogram of stress ranges for all trucks and a stress trace for each truck to be recorded. Visual traffic counts were also taken during testing to correlate the stresses with traffic on the bridge.

Stresses in the longitudinal girders were similar to those found in longitudinal girders in other studies, with live load stresses rarely exceeding 8 ksi. Stresses in the tie plates were much higher, however, with values over 18 ksi being recorded for some tie plates. The stress distributions for the tie plates indicated that the stresses were caused by horizontal and not vertical bending of the plates. The pattern of stress variation for the plates was the same regardless of the type of truck, varying only in magnitude. Analysis of these facts showed that a longitudinal



Elevation View

(a)



Cross-Section View

(b)

Figure 3. Elevation View and Cross-Section View of Lehigh Canal Bridge.

displacement of the top flange of the main girders would cause horizontal bending of the tie plates. Since the displacement of a point on the flange is proportional to the slope of the girder at that point, the influence line for the slope of the girder should be similar to the influence line for the strain in the tie plates. This is shown to be true in Figure 4, where the strain history for two tie plates is compared to their respective influence lines for girder slopes.

The cracks in the tie plates originated at short tack welds used to connect the top flange of the brackets to the tie plates prior to them being riveted together. Laboratory tests on beams with short attachments have been performed, and Figure 5 shows the results for the 95 percent confidence limits. These were run under constant stress range cycles, though, while the loading cycles on the bridge varied according to truck weights. The total number of truck loads for the bridge since its opening in 1953 was estimated, and the truck weights were assumed constant over the years. The root-mean-square (RMS) method was used to arrive at a constant stress range for the tie plate. This figure had to be adjusted to account for the distance between the gages and the cracks. A plot of the RMS values versus the number of cycles is shown in Figure 5. Also shown in Figure 5 is the value of stress range obtained by combining Miner's rule with the relationship provided by constant data.[4] It can be seen from this graph that the fatigue life of these tie plates can be

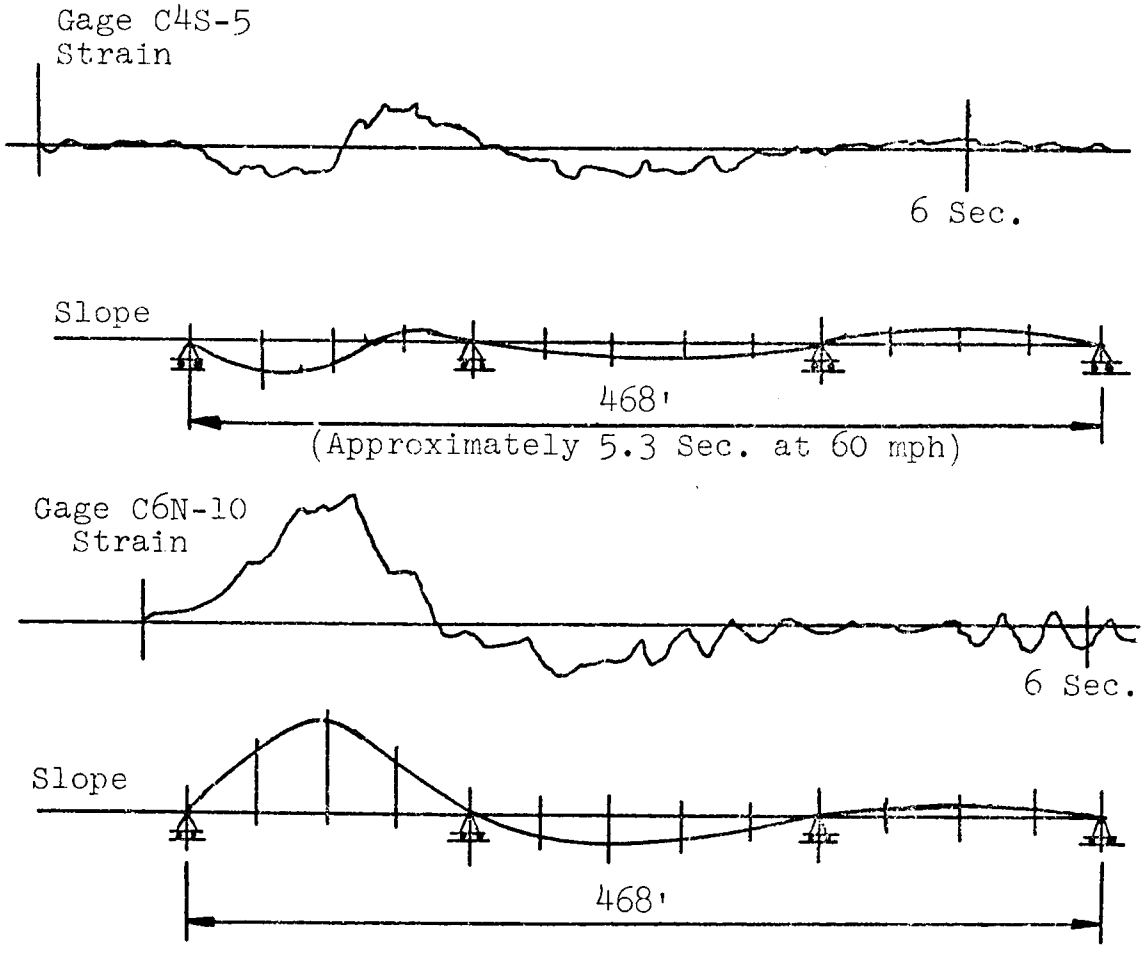


Figure 4. Comparison of Measured Strain History in Tie Plates With Influence Lines for Girder Slope.

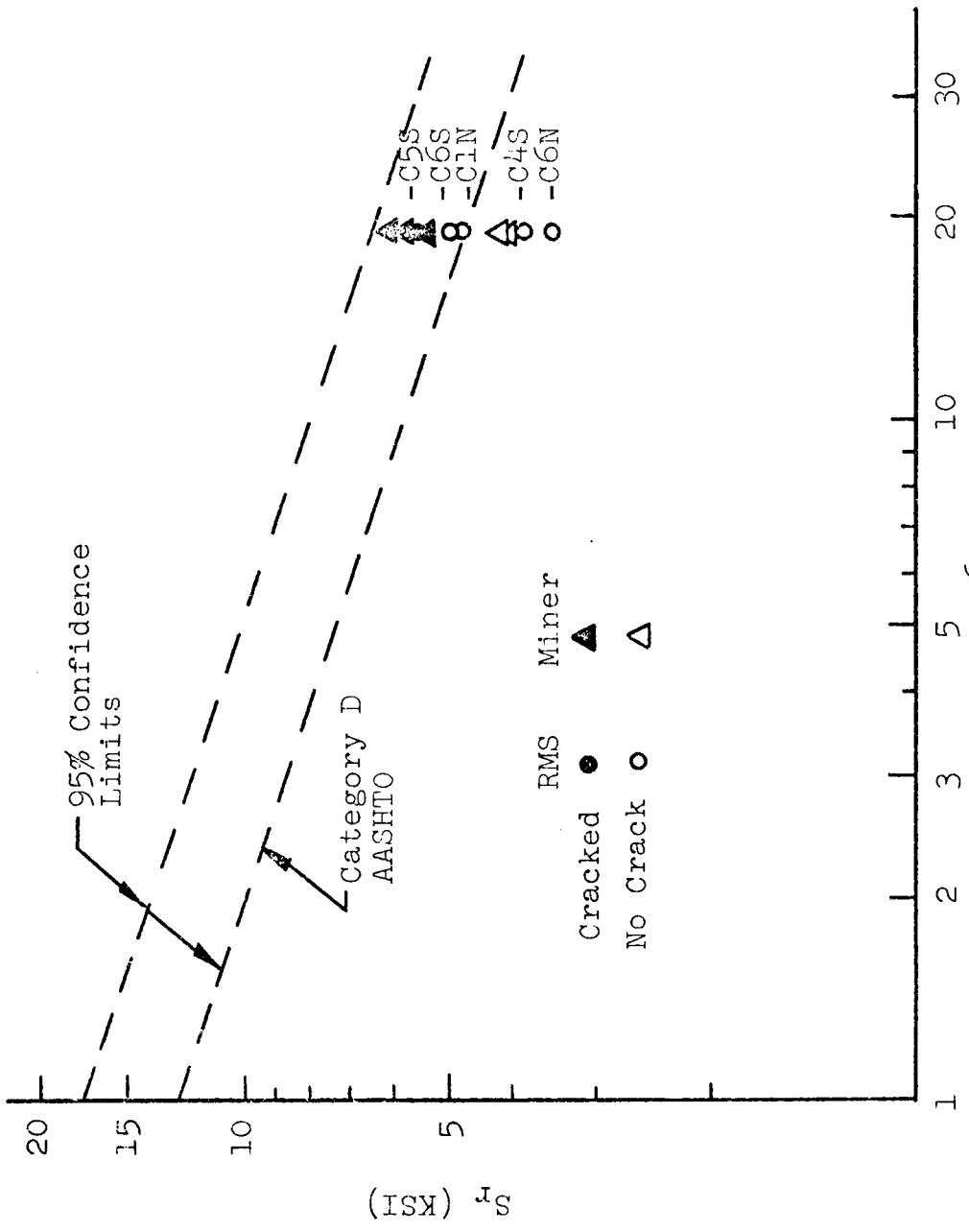


Figure 5. Comparison of Estimated Equivalent Stress Range With Laboratory Test Data.

predicted fairly accurately. Testing of the Lehigh Canal Bridge is continuing in an effort to learn more about the stress range-life relationship, which the study of this bridge provides.

Tennessee

The research in Tennessee was performed under the sponsorship of the Tennessee Department of Transportation, Bureau of Highways, in cooperation with the U.S. Department of Transportation, FHWA. Two different aspects of the stress history problem were studied. The first area researched was the prediction of the fatigue life of bridge girders using actual field measurements of the bridges in question.[1] The other area of study was directed toward relating field data to analytical data in an effort to provide the bridge designer with a workable method to use in designing bridges relative to their fatigue strength.[9, 10]

Six bridges at three different locations on I-40 in Tennessee were selected to instrument and collect strain data.[1] Two of the bridges tested were steel girder-concrete slab designs, with the remaining four being reinforced concrete deck girder bridges. The stress ranges measured in the tests of these bridges were not very high, with few stresses above 5 ksi. Stresses at the ends of cover plates on the steel girder bridges were higher than the stresses measured at midspan, though. The concrete bridges produced stress ranges generally higher than those on the steel bridges.

However, the magnitude and number of stresses in any of the girders of the six bridges appeared to have a minimal effect on the life expectancy of the girders. Life expectancies of around 400 years were calculated for most of the girders, with the lowest life estimation being 298 years. Other stress history studies in Alabama, Minnesota, Maryland, and Virginia corroborate these findings of low stresses and high life expectancies for bridge girders.

Two studies were conducted to relate measured stress to theoretical stress in an effort to be able to use field testing results to predict the stresses in bridges with different cross sections. The first of these studies[9] used the strain data from the six bridges mentioned previously, and correlated these data with simplified analyses of the bridges. The AASHTO HS 20 loading with one truck in each traffic lane was used in computing the maximum moment at midspan for each bridge. A static analysis was used in calculating the moment for the simply supported steel girder bridge. The ICES STRUDL II program was used to calculate the midspan moments for the continuous concrete bridges because haunches required the use of varying moments of inertia in order to adequately model the bridges. The moments were then uniformly distributed to each girder, which was considered a reasonable assumption for the case with a truck in each lane. A stress was then calculated using the moment of inertia of the entire bridge cross-section at midspan, assuming a cracked section for the concrete bridges.

Two stresses for each bridge were calculated: the maximum expected stress, caused by loading all lanes with trucks, and a "typical" stress due to a truck in only one lane, and taken to be half of the maximum expected stress. These calculated stresses were then compared to the stresses actually measured in the bridge. Figure 6 shows this comparison for the steel girder bridge. From this figure it is seen that the maximum predicted stress range gives a good estimation of the largest stress that probably would occur. The typical predicted stress range also supplies a good average of the stresses that would occur on the bridge. The results from the two concrete bridges are similar to the results for the steel bridge, and lend support to this method as a step in the right direction in predicting the maximum stress a bridge would expect a significant number of times during its life.

The second study[10] concerning the relationship of theoretical-to-actual stresses was concerned mainly with presenting a logical methodology to relate weight data to measured stress data to theoretical stress data in an effort to predict girder stresses in a highway bridge. As a starting point, it is assumed that a multi-girder bridge exists for which facilities are available to collect stress data, truck traffic data, and truck weight data. Listed below are the seven steps necessary to predict girder stresses:

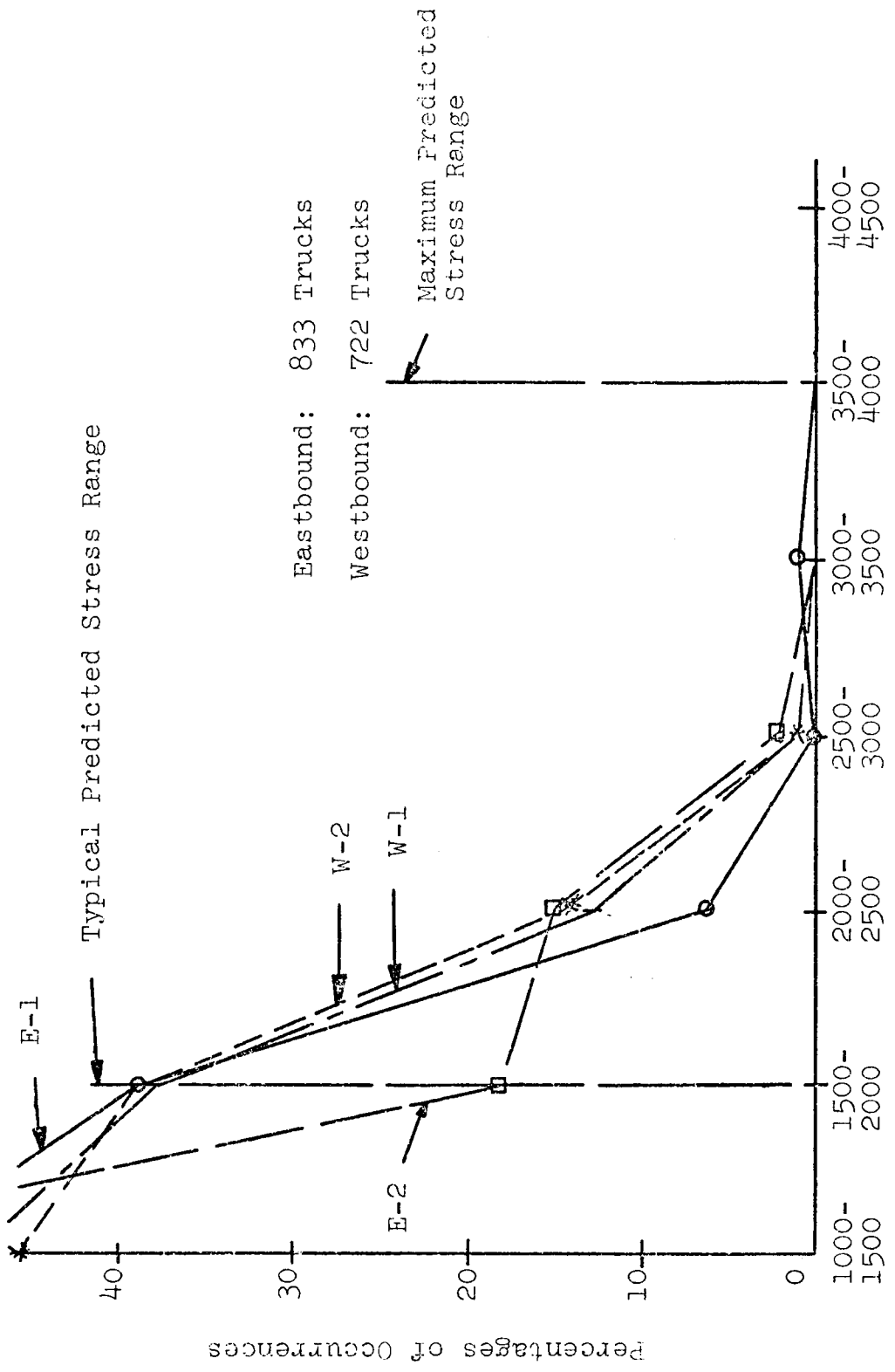


Figure 6. Plot of Percentage of Occurrences Vs. Stress Range - Bridge Site 1 (Both Bridges).

1. Obtain a stress histogram for each girder in the bridge for each major truck type traveling in each traffic lane.
2. Obtain weight histograms for each major truck type.
3. Correlate the stress and weight histograms obtained in steps 1 and 2.
4. Obtain the percentage of each major truck type traveling in each traffic lane.
5. For a known distribution of truck types on the bridge, predict the stress histograms for each girder and compare the results with the field data.
6. Relate measured stress in a girder due to a truck of known type and weight in a particular traffic lane to the theoretical stress in the girder under similar conditions.
7. Finally, for a known distribution of truck types, the stress histogram can be predicted for each girder on the basis of the calculated stress in the girder.

There is obviously much work required to complete each of these steps. Steps 3 and 6 are key steps in this method because they entail finding relationships between two different variables. Step 3 requires a knowledge of statistical methodology in order to relate stress to weight, while step 6 indicates that an analytical method to calculate stresses be found that is both easy to use and reasonably

accurate. Some work on step 3 was done in this study. Both weight and stress data were collected in an effort to correlate the two, but no direct correlation between stress range and weight was developed. A positive correlation between the two seems possible, however. The work being done presently at Tennessee, and in fact the material presented in this thesis, is directed toward the completion of the seven steps which make up the method presented above.

CHAPTER III

DESCRIPTION OF BRIDGE AND TESTING METHODS

1. BRIDGE DESCRIPTION

For this study, two bridges on Interstates 40 and 75 in Knoxville were selected. Both bridges were designed with rolled steel girders and a concrete deck roadway over three simple spans. Each bridge had seven girders per span, with the center span being approximately 62 feet long. Data were collected only on the middle span of the three span bridges, with the strain gages being located at the longitudinal centerline of the middle span. Although the point of maximum moment on the bridge is dependent on the number of axles and the axle spacing of each truck and although it will usually not be at the centerline, stress data taken at the span centerline will be less than 5 percent off of the maximum moment in the bridge.

The westbound bridge carried an estimated average daily traffic (ADT) volume of 46,128 vehicles, of which approximately 11 percent was truck traffic. 3S2 trucks made up 41 percent and 2D trucks constituted 39 percent of the truck traffic. The eastbound bridge had an ADT volume of 43,032 vehicles. Of this volume, approximately 11 percent was truck traffic, with 43 percent of these

being 3S2 trucks and 35 percent being 2D trucks. An official state weigh station is located approximately ten miles from the bridge on I-40, but a one truck weight - one stress range correlation was not considered feasible for this study because of the numerous turnoffs between the bridge and the weigh station. These turnoffs significantly changed the truck type distribution at the weigh station compared to that at the bridge, which can be seen from the fact that there was twice the percentage of 3S2's at the weigh station as there was at the bridge. It was assumed, however, that the truck weight distribution at the bridge and at the weigh station were the same although it will be shown later that this assumption was not entirely correct. Weight distributions that were collected earlier in the project at the same weigh station were used. Gross weights for 1634 3S2 trucks were available, while 443 2D truck gross weights were used. These data were stored on a computer so that weight histograms with varying intervals could be obtained readily.

The westbound bridge was selected for this study. One of the main reasons for this was that traffic on the eastbound bridge would begin to back up past the bridge starting about three o'clock every evening. Data collection would have to stop during these times because trucks could not pass over the bridge in the three and one-half seconds that the data collection program allotted for each truck.

Another reason for this was that the westbound bridge exhibited less skew than the eastbound bridge, and thus it was easier to model. Some testing was done on the eastbound bridge, however, and further studies will be made which will utilize these stress data.

Instrumentation

As mentioned before, strain gages were located at the middle of the beams in the center spans of both the bridges. Other gages were located at the ends of cover plates on the beams, but these were not sampled in this study. The gages sampled were connected to wires which were hooked into amplifier units housed in an office trailer which was moved to the bridge site. Housed in the trailer with the amplifier units was a Raytheon 703 computer which was used to do all the data collection and reduction. The amplifier units relayed the electrical signals to an analog-to-digital converter located in the trailer, which digitized the signals at a rate of 240 samples/second/gage. The computer then recorded either the strains or a histogram of strain ranges on magnetic tapes which were stored for further reduction at a later time. A remote sense switch box was used so that the computer's sense switches could be operated by a person sitting on the bridge. A schematic diagram of the strain recording system is shown in Figure 7.

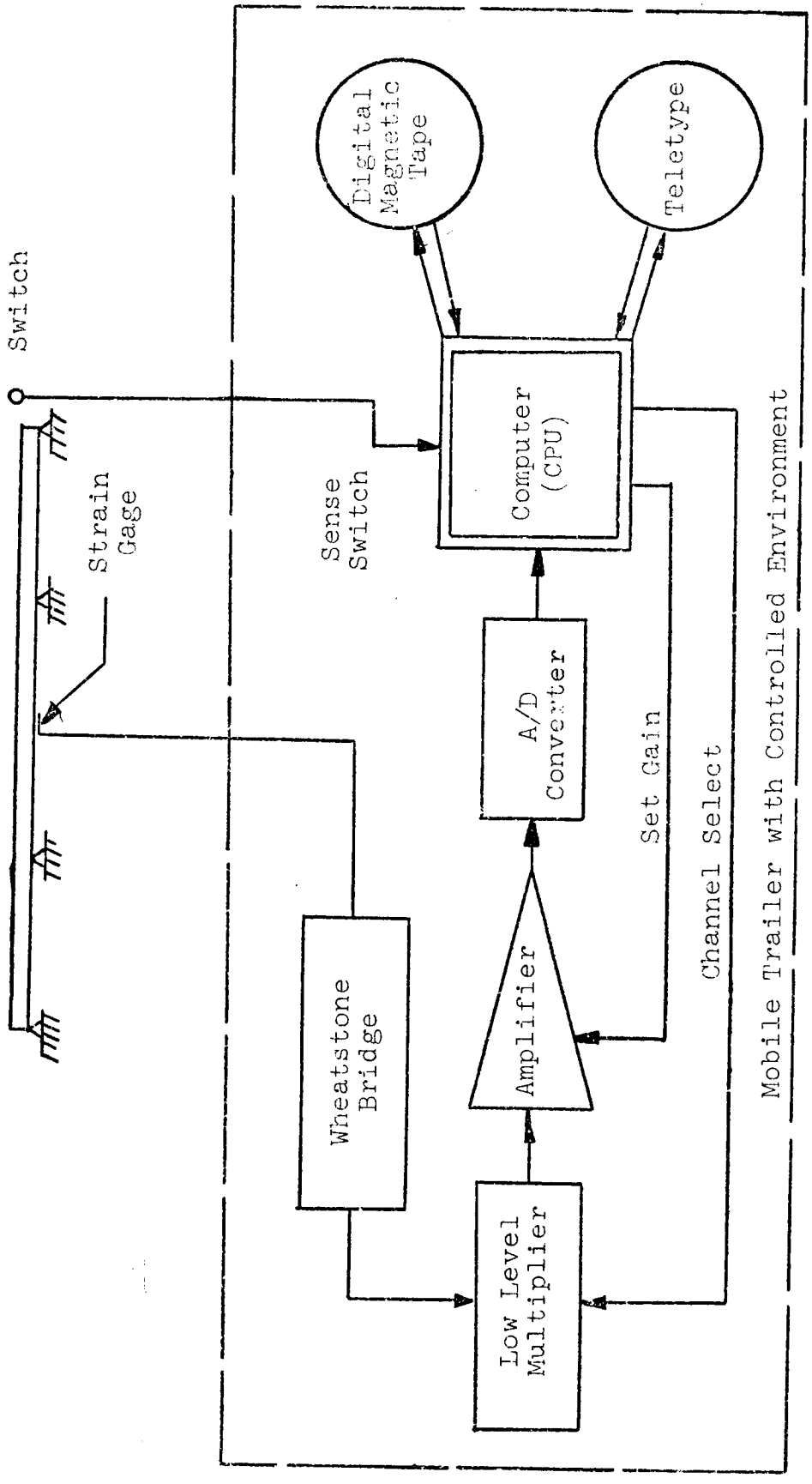


Figure 7. Schematic of Instrumentation.

2. DATA COLLECTION

Two different sampling methods were used in field testing: continuous sampling and individual sampling. Both types required a person on the bridge to operate the sense switch box and to record the truck types and the lanes they traveled in. A description is given below of only the individual sampling method because this study utilized only the strain data recorded in this type of sampling.

The individual sampling program was written in FORTRAN and consisted of recording strains on magnetic tape for each individual truck passage. This required an observer on the bridge to flip a sense switch right as the truck entered onto the bridge. The computer would then sample and write strains on tape for 3.5 seconds, recording 800 strains for each gage. The observer would then list on paper the truck type and the lane it was traveling in. This provided a one truck type/lane, one stress relationship.

Before sampling started for each test, identifying and other necessary information was written on the magnetic tape. When the bridge was clear of traffic, initial and calibration values were written on the tape. The difference between the initial and calibration values provided a conversion factor for the numbers on the tape, which are actually voltage changes but have previously been and will henceforth be referred to for simplicity as strains, to actual strains. These initial and calibration numbers will generally vary

with the gage and corresponding amplifier unit. After these numbers were written on tape, identifying information such as test number, time, and date were written. Then truck sampling was started, with approximately 320 trucks being recorded on each tape. These tapes were then stored for reduction at a later time.

One advantage of this type of sampling is that the strain trace for each truck is permanently stored, and a reduction program may be written at a later time to reduce the data. This allows as many or as few strain ranges as desired to be printed out (see Figure 8). A histogram such as that recorded by continuous sampling may even be formed. A disadvantage to this type sampling, however, is the number of tapes required to collect a large sample of trucks.

3. DATA REDUCTION

A FORTRAN program was written to reduce the individual sampling data. The program would read the strain data for a given truck from the tape, and the largest strain range (S_r in Figure 8) for each girder would be computed. Calibration numbers and a modulus of elasticity were used to convert to stress ranges. The printout for a given truck would show the truck number and the maximum stress for each girder, with the stresses being printed from left to right on the paper to represent the girders as they appeared from left to right on the bridge, looking in the direction of traffic. The sum of the stresses for all seven girders was

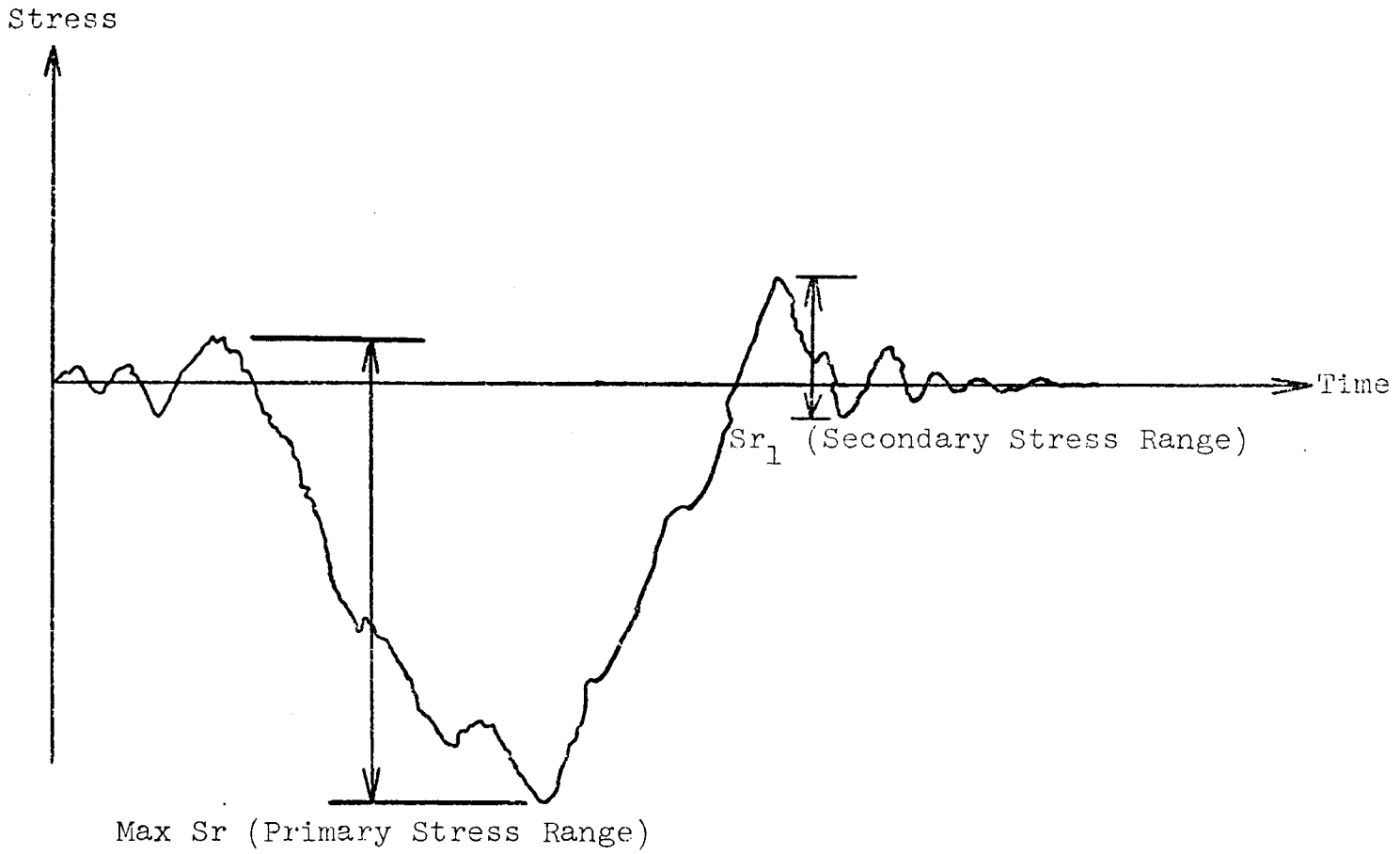


Figure 8. Stress Trace Indicating Maximum and Secondary Stress Ranges Per Truck Passage.

also calculated and printed. The data from each test were reduced twice and these results were compared to insure that no computer or tape reading errors were present. One of the checked outputs was then taken and matched with the corresponding truck field data. Each truck number on the printout was assigned a truck type by matching the corresponding number on the field data. This truck type was then written on the computer printout beside the stresses for that truck. Any inconsistencies in the data were then resolved, which consisted of eliminating those trucks where the lane position was obviously miscopied. A typical test reduction is shown in Table I. The figure at the top of each stress column represents the number of the amplifier unit for each girder. Once every test had been reduced in this manner, a histogram of both individual girder stresses and summation of stresses for a particular truck type could be formed. These histograms were also further broken down into truck type by lane categories, with a histogram representing the left, middle, and right lanes for each truck type.

TABLE I

SAMPLE OF INDIVIDUAL TEST REDUCTION

TEST 177

	7	3	2	1	0	4	9		
1	68	34	31	46	58	48	68	353	clear
2	2764	12010	2927	10835	4901	450	1353	-30296	scratch
3	149	240	178	386	919	2016	1319	5207	352-R
4	91	91	157	351	838	1634	998	4160	352-R
5	412	848	734	643	186	107	195	3125	252-L
6	539	1193	1384	666	256	146	81	4275	352-L
7	562	1170	860	304	197	264	160	3517	352-L
8	68	45	73	117	256	342	172	1073	2D-R
9	126	229	293	526	954	1536	940	4604	352-R
10	114	252	241	397	267	107	68	1446	252-M
11	780	1617	1395	608	174	254	137	4965	352-L
12	217	217	430	1135	931	489	263	3682	252-M
13	412	286	734	1240	663	332	344	4011	352-M
14	286	160	472	889	605	283	229	2924	352-M
15	103	229	283	327	617	910	527	2996	352-R
16	860	1514	1101	561	174	225	195	4630	352-L
17	298	630	1468	2480	1397	724	286	7283	352-M
18	998	1812	1468	702	232	195	217	5624	352-L

CHAPTER IV

ANALYSIS OF WEIGHT AND STRESS DATA

The two most numerous truck types, 3S2's and 2D's, were chosen to give a good approximation of the type and number of loads a bridge would undergo. These two truck types represent an axle configuration from each of the major truck types, as shown in Figure 1, page 7. Given below is a description of the weight distributions and truck characteristics for 3S2's and 2D's, along with a breakdown of their respective actual stress histograms.

1. WEIGHT DISTRIBUTIONS AND TRUCK CHARACTERISTICS

In order to better define the loads on the bridge, it was necessary to specify the truck characteristics by parameters other than a single weight distribution for all truck types. Below is a list of the factors that can significantly effect the girder stresses in a bridge, noting that only those which our truck data afforded are included:

1. Gross vehicle weight
2. Distribution of weight to axles
3. Axle spacings

By separating truck data into the various truck types, and further dividing the data according to the three categories listed, a good picture of the loads on the bridge is formed.

These data formed the load input into the mathematical model of the bridge. A description of these characteristics for 3S2's and 2D's is given below.

Type 3S2 Trucks

Table II gives the gross weight distribution of 3S2 trucks, which is also shown as a graph in Figure 9. Two peaks are present, the first at 30,000 pounds and the second at about 72,000 pounds, which is near the maximum legal weight in Tennessee of 73,280 pounds. The majority of the trucks appear to be either very lightly loaded, or heavily loaded and approaching the legal limit. This is in line with what studies in other states have shown.[5] It should be mentioned, however, that these weights are from the official weigh station already described, and their distribution is not necessarily the weight distribution seen on the bridge. These weight data are considered to be a representative sample of the 3S2 trucks at the weigh stations, though, because of the large sample size of 1634 trucks.

The distribution of the weight to the axles can effect the girder stresses significantly. This is illustrated by the fact that a 60,000 pound 3S2 truck with 62 percent of the weight on the middle axle will cause the same moment on the total bridge cross-section as a 72,000 pound truck with 50 percent of the weight on the middle axle. The weight distribution was separated into two parts, with the front axle weight being handled separately from the middle and

TABLE II

GROSS VEHICLE WEIGHT DISTRIBUTION
OF 3S2 TRUCKS

Weight (Kips)	Percent
<15.06	.12
15-21	.12
21-27	5.20
27-33	20.99
33-39	6.92
39-45	5.94
45-51	7.10
51-57	8.57
57-63	11.63
63-69	13.83
69-75	18.85
75-81	.49
> 81	.24

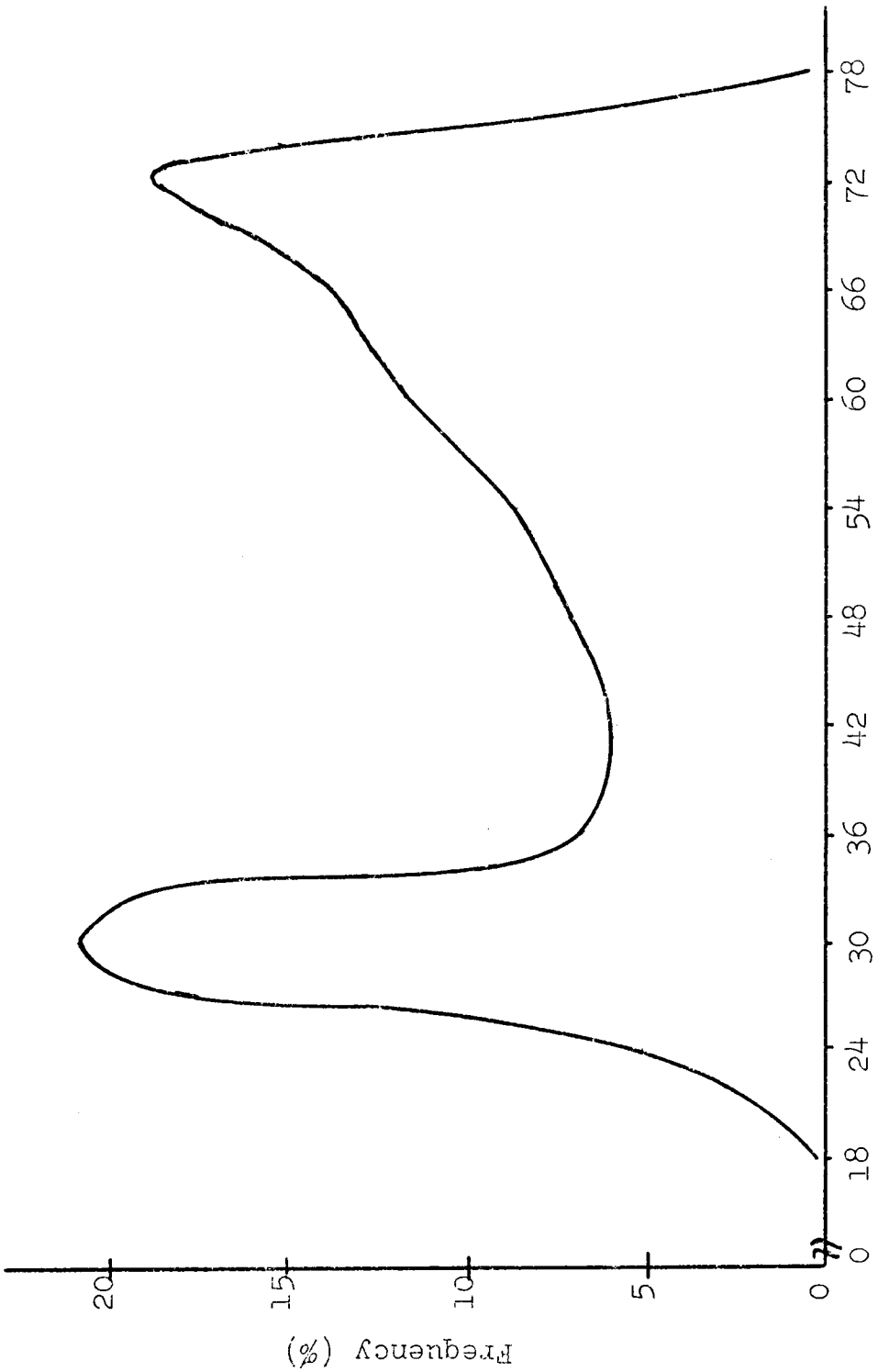


Figure 9. Gross Weight Distribution of 3S2 Trucks.

rear axle weights. The front, middle, and rear axles are shown in Figure 1, page 7, as "A", "B", and "C", respectively. For each of the weight intervals listed in Table II, a separate weight distribution was calculated for the front axle. These weight intervals were considered small enough to adequately represent the change in the front axle weights as the gross weight of the vehicle changed. The front axle weights generally increased with increasing gross weight, and ranged from an average of 7.75 kips for the 21 to 27 kip interval to 9.25 kips for the 69 to 75 kip range.

The middle and rear axle weights of a 3S2 truck must be handled in a way such that each axle weight distribution is properly represented without distorting the total picture of the weight distribution to the axles. For example, a middle axle weight of 30,000 pounds may be selected from a weight distribution for middle axles, and a rear axle weight of 10,000 pounds may be selected in the same way. Although there may be separate axles with these weights, it is possible that no individual truck has its weight distributed in this ratio. In an attempt to describe the true picture, the means and standard deviations of the middle and rear axle weights were calculated to see if possibly one of the axle weights was essentially constant while the other axle weight varied. This was not the case, however, because the middle axle had a mean weight of 22.2 kips with a standard deviation of 7.6 kips, and the back axle had a mean weight of 20.9 kips with a standard deviation of 8.9 kips.[12]

To accurately describe the middle and rear axle weights, a distribution of axle weight ratios was calculated for each 6 kip interval from 15 to 81 kips. This ratio represented the percent of the gross vehicle weight, minus the front axle weight, that was on the middle axle. This ratio, when divided into the intervals mentioned, gave an accurate description of the individual axles without misrepresenting the total truck weight picture. In general, as the gross vehicle weight increased, the load shifted from having more weight on the middle axle to an even weight distribution to the two axles. This is illustrated in Figures 10a, 10b, and 10c, for three weights representing a light, a medium, and a heavy truck, respectively.

In addition to the gross vehicle weight and axle weight distribution, the front and rear axle spacings also effect the stresses caused by a truck. Axle spacing data were taken for 340 trucks of all types at the weigh station mentioned previously to see how the spacings varied. Of these 340 trucks, 204 were of the 3S2 type. The front axle spacing for 3S2's was, on the average, 12.84 feet with a standard deviation of 1.10 feet. The back axle spacing averaged 30.19 feet with a standard deviation of 3.75 feet. After using the mathematical model of the bridge to see the effect on stresses of the small distribution in front axle spacings, it was decided to neglect this spacing and hold it constant at 12.84 feet, the mean value. The back axle

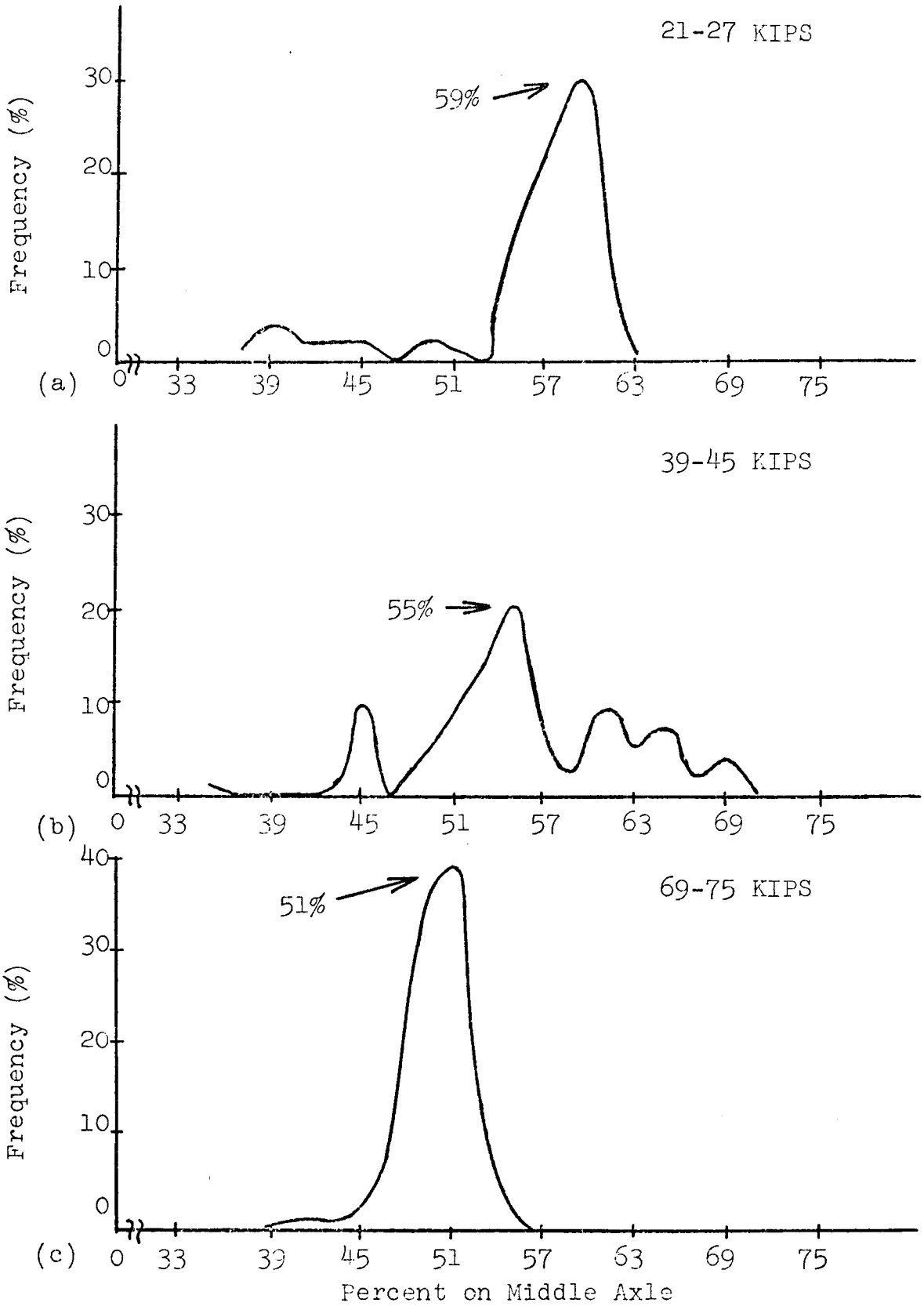


Figure 10. Percentage of Weight on Middle Axle - 3S2 Trucks

spacing had a larger distribution, however, and its effect on the stresses could not be ignored. An attempt was made to relate this spacing to gross vehicle weight, thinking that the back axle was being shifted according to the load for easier truck handling. A back axle spacing distribution was computed for each of the 6 kip weight intervals as before. It was not clear from these distributions whether this weight-spacing relationship existed, though. A multiple linear regression analysis was then used to try and find this relationship. The equation below was formulated for predicting the back axle spacing:

$$Y = 29.454 + .00012996\ X_1 - .00086281\ X_2$$

where

Y = axle spacing

X_1 = middle (or drive) axle weight

X_2 = rear (or trailer) axle weight

This equation only explains .88 percent of the variation in rear axle spacings, however. With 0 percent being the worst and 100 percent being the best possible confidence levels, it was concluded that there was no correlation between vehicle weights and rear axle spacings. A histogram of rear axle spacings was therefore formulated to represent all 3S2 truck weights.

Type 2D Trucks

The gross weight distribution for 2D trucks showed no bimodal tendencies as did the 3S2's. Table III gives a histogram of the gross weights, which is also shown graphed in Figure 11. Although the sample size of 443 trucks was considered large enough to be representative of 2D's, it was only one-fourth the sample size of 3S2's at the weigh station. Considering that there were only 2 percent more 3S2 trucks than 2D's at the bridge site, some idea of the difference in truck population at the bridge site and at the weigh station is obtained.

As was the case with 3S2 trucks, both of the 2D axle weights need to be represented so that the total weight distribution of any individual 2D is not distorted. The mean front axle weight for 2D's was calculated to be 5.5 kips with a standard deviation of 1.7 kips. The average rear axle weight was 9.9 kips with a deviation of 4.4 kips.[11] The front and rear axles are shown in Figure 1, page 7, as "A" and "B" respectively. Although these standard deviations were numerically much smaller than they were for 3S2's, they formed approximately the same percentage of the gross vehicle weight as they did for 3S2's. Neither axle weight was being held approximately constant while the other axle weight varied. It was decided to represent the axle weights by forming a ratio similar to that formed for 3S2's. With only two axles to represent, the ratio was that of the front axle weight to the gross vehicle weight. A distribution of ratios was

TABLE III

GROSS WEIGHT HISTOGRAM OF 2D TRUCKS

Weight	Percent
<6.00	2.48
6.00- 9.00	14.67
9.00-12.00	22.12
12.00-15.00	20.77
15.00-18.00	14.22
18.00-21.00	9.48
21.00-24.00	9.93
24.00-27.00	3.61
27.00-30.00	.90
>30.00	1.81

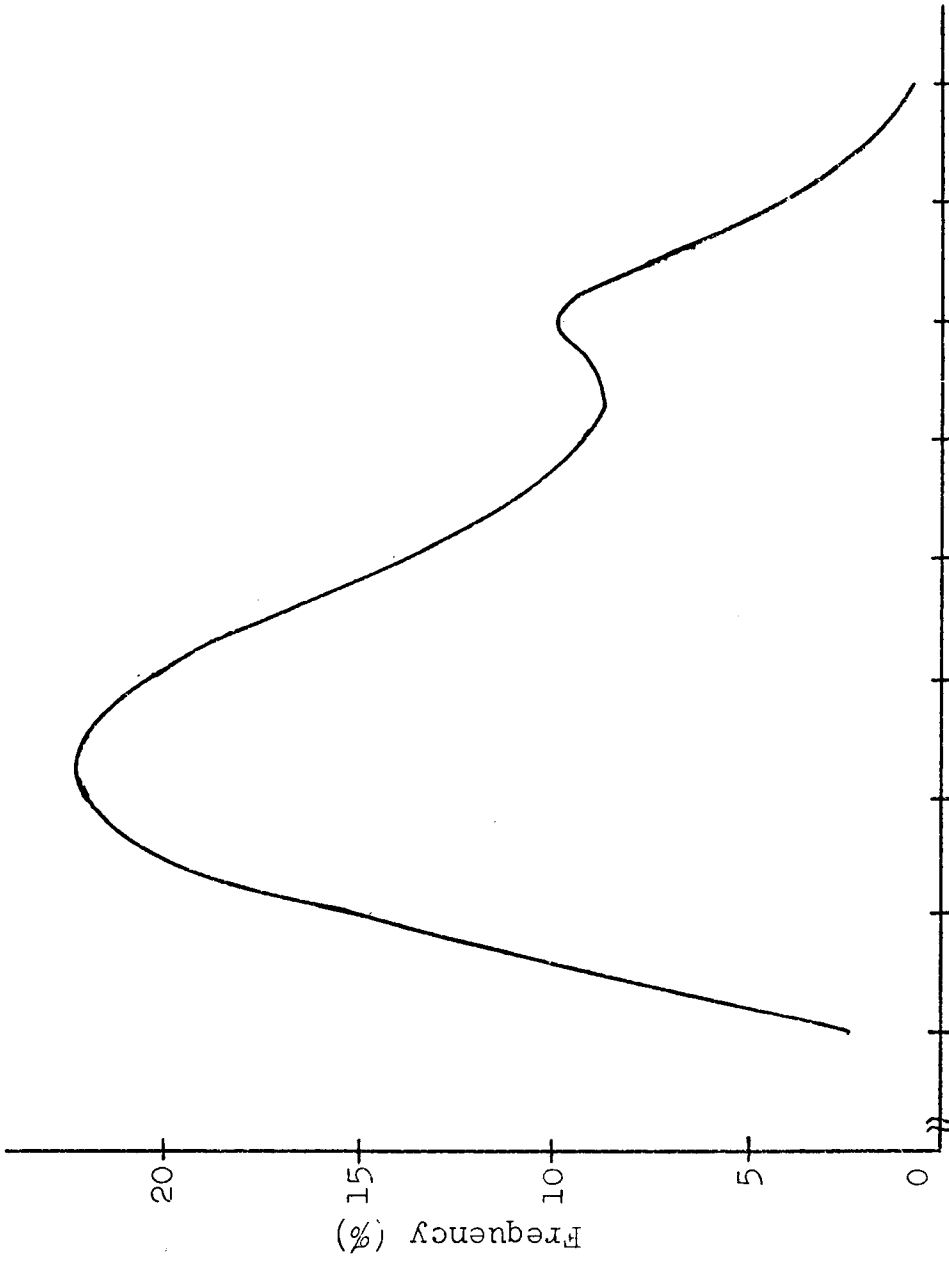


Figure 11. Gross Weight Distribution of 2D Trucks.

calculated for each 4 kip interval between 4 and 28 kips. Generally, as the gross weight of the truck increased, the weight shifted to the rear axle.

An axle spacing is also required to completely specify the loading caused by a 2D. For a sample size of 82 trucks, the mean axle spacing was computed to be 14.90 feet with a standard deviation of 2.46 feet. A distribution of spacings for each of the weight intervals in Table III, page 45, did not show a clear correlation between weight and spacing. As was done for 3S2's, a multiple linear regression analysis was used to see if this correlation existed. An equation was formulated to predict the spacings, given the front and rear axle weights. However, the equation explained only 25 percent of the variation in axle spacings, and it was concluded that there was no correlation between weight and spacing. A distribution of axle spacings was therefore calculated to represent all 2D's, without regard to weight.

2. MEASURED STRESSES - 3S2's AND 2D'S

Individual Girder Stresses

Stress histograms were assembled for each of the truck types shown in Figure 1, page 7, but only the 3S2 and 2D histograms will be reported herein. A histogram was formed for each of those two truck types for each of the seven girders of the bridge, with these being further separated into histograms representing trucks in the left, middle, and right lanes.

Table IV gives a histogram for each girder for 3S2's in each of the three lanes. The girder position and the approximate placement for the loads in these lanes are shown. No stresses below 0.5 ksi are included in the histograms because electrical noise in the system clouded the lower stress ranges. It can be seen that the girder nearest the load is the most highly stressed, with the stresses getting progressively lower as the distance from the load to the girder increases. The distributions shift from skewed to the right to skewed to the left as the load placement is moved from the left lane to the right lane. Note that few stresses occurred above 3 ksi. Table V shows the histograms for 2D's in the three separate lanes. These histograms exhibit the same general shape and skew characteristics as they did for 3S2's, except that the stress magnitudes are generally much lower. This is to be expected, however, with the weights of 2D's being much lower than the 3S2 weights. Few stresses occurred above 2.25 ksi for 2D's.

Summation of Girder Stresses

One of the key steps in predicting stress histograms for bridge girders is to be able to relate measured stresses to the appropriate truck weight distributions for a test bridge. Lane positions of the trucks caused problems in trying to relate any individual girder stress histograms to the weight distribution, however. In an attempt to circumvent the transverse position problem, the stresses

TABLE IV

STRESS RANGES FOR 3S2 TRUCKS BY LANE

	Stress Range (ksi)	Lane: Left			Middle		Right	
		7	6	5	4	3	2	1
381 Trucks in Left Lane	0.50-0.75	103	0	23	137	27	16	7
	0.75-1.00	104	44	100	79	3	1	
	1.00-1.25	102	94	73	11			
	1.25-1.50	32	56	52	1			
	1.50-1.75	16	42	44				
	1.75-2.00	6	42	47				
	2.00-2.25	1	36	29				
	2.25-2.50	2	31	4				
	2.50-2.75		18	0				
	2.75-3.00		5	1				
	3.00-3.25		0					
	3.25-3.50		0					
	3.50-3.75		0					
	3.75-4.00		1					
361 Trucks in Middle Lane	0.50-0.75	36	132	82	4	57	118	24
	0.75-1.00	2	25	90	44	69	23	
	1.00-1.25			94	48	54	0	
	1.25-1.50			50	38	36	0	
	1.50-1.75			16	32	72	1	
	1.75-2.00			4	40	34		
	2.00-2.25				65	14		
	2.25-2.50				38	5		
	2.50-2.75				17	2		
	2.75-3.00				3	0		
	3.00-3.25				3	0		
	3.25-3.50				2	1		

TABLE IV (Continued)

	Stress Range (ksi)	Lane						
		Left			Middle		Right	
		7	6	5	4	3	2	1
300 Trucks in Right Lane	0.50-0.75	0	6	20	95	22	4	75
	0.75-1.00				20	61	30	60
	1.00-1.25				7	32	53	47
	1.25-1.50				2	35	30	31
	1.50-1.75					37	20	7
	1.75-2.00					23	29	4
	2.00-2.25					12	38	1
	2.25-2.50					4	21	
	2.50-2.75					0	6	
	2.75-3.00					2	2	
	3.00-3.25					0	1	
	3.25-3.50					1	0	
	3.50-3.75						2	

TABLE V

STRESS RANGES FOR 2D TRUCKS BY LANE

	Stress Range (ksi)	Lane: Left		Middle		Right		
		7	6	5	4	3	2	1
271 Trucks in Left Lane	0.50-0.75	53	85	91	17	1	3	0
	0.75-1.00	24	65	55	3	2	1	
	1.00-1.25	8	30	17				
	1.25-1.50		19	16				
	1.50-1.75		13	6				
	1.75-2.00		8					
	2.00-2.25		3					
	2.25-2.50		1					
360 Trucks in Middle Lane	0.50-0.75	2	36	104	118	73	10	1
	0.75-1.00		1	49	48	55		
	1.00-1.25		1	34	34	37		
	1.25-1.50			10	28	27		
	1.50-1.75			2	26	8		
	1.75-2.00			0	27	3		
	2.00-2.25			1	16	1		
	2.25-2.50				5			
	2.50-3.00				1			
318 Trucks in Right Lane	0.50-0.75	0	3	1	12	90	138	43
	0.75-1.00					29	48	10
	1.00-1.25					13	23	2
	1.25-1.50					3	10	
	1.50-1.75					1	4	
	1.75-2.00						1	
	2.00-2.25						1	

occurring in each of the seven girders were summed for each truck sampled. These were then divided into truck types and a histogram formed for each.

To sum the girder stresses, several assumptions were made. The most obvious assumption was that transverse lane position did not affect the sum of the stresses. This is theoretically correct for a bridge with a completely uniform cross-section with regard to girder size and deck thickness. The bridge that the data were taken from had essentially a uniform cross-section except for the curbs adjacent to the left and right lanes. Using the mathematical model of the bridge and a representative 3S2 loading, the summation of stresses was computed for a number of transverse loadings on the bridge. The maximum difference in the stresses was only seven percent, and therefore this assumption was considered valid. Also inherent in summing the girder stresses is the assumption that the maximum stress ranges for all the girders occur at the same time. This assumption was necessary because our strain data were recorded with time. Therefore, if the strains are to be summed, they must be summed for the same instant of time to be meaningful. The data reduction program was modified to output where the maximum and minimum values that make up a stress range occurred in the strain trace. This program showed that these values always occur at the same time, and thus this assumption was also valid. Figure 12 shows the strain trace for three adjacent girders, and demonstrates that the maximum

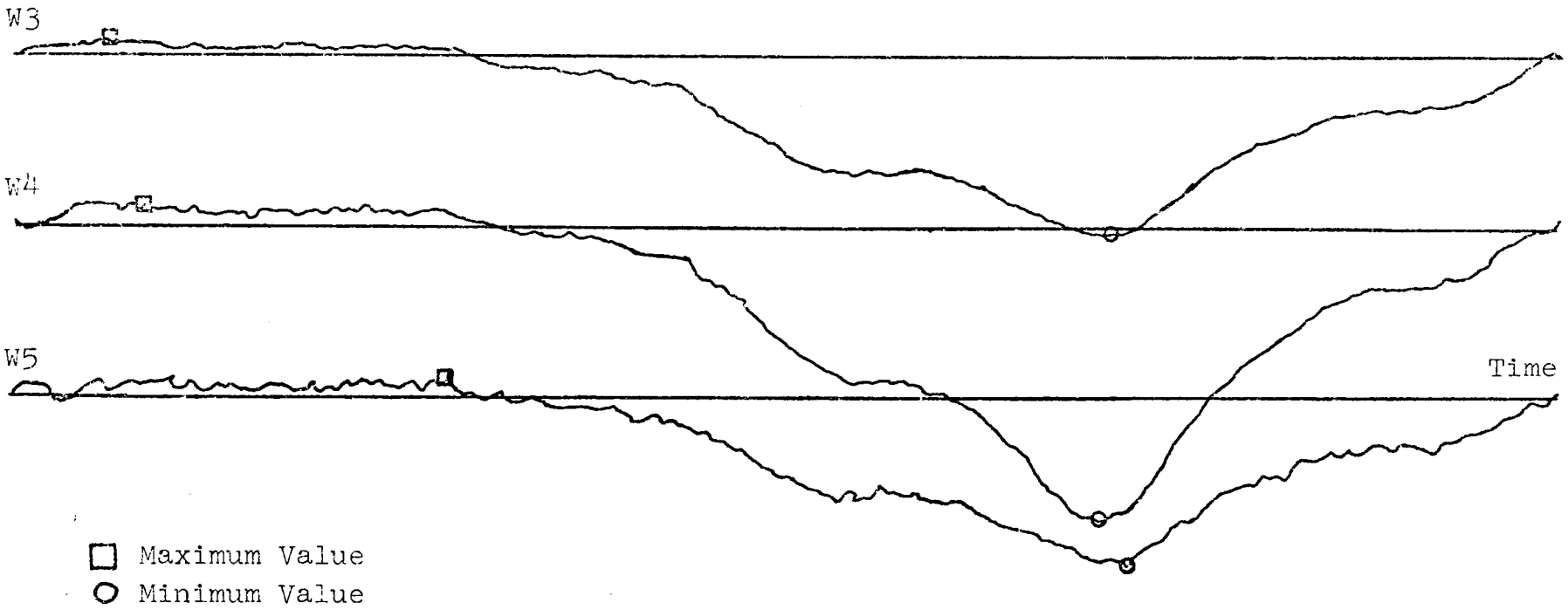


Figure 12. Strain Trace at Centerline for Three Adjacent Girders.

and minimum values do occur at the same general time period.

Having proved that summing the stresses for the seven girders was a valid technique, the stresses were summed for each truck, and histograms were constructed for each of the major truck types. Figures 13 and 14 show the graphs of summation of stress versus frequency of occurrence for 3S2 and 2D trucks, respectively. A more detailed explanation and analysis of these histograms with relation to their corresponding weight distributions will be given in Chapter VI.

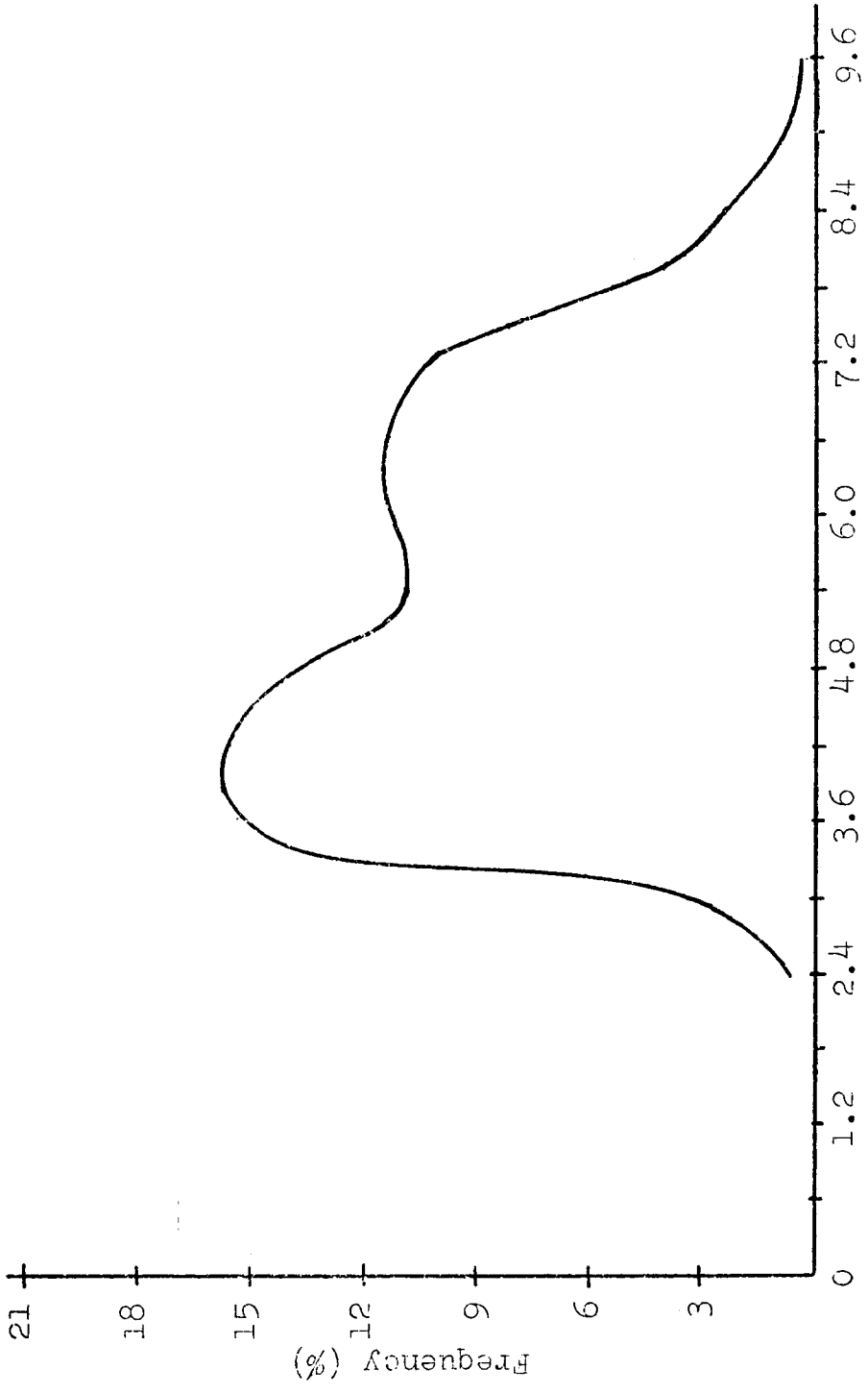


Figure 13. Summation of Actual Girder Stresses Versus Frequency - 3S2 Trucks.

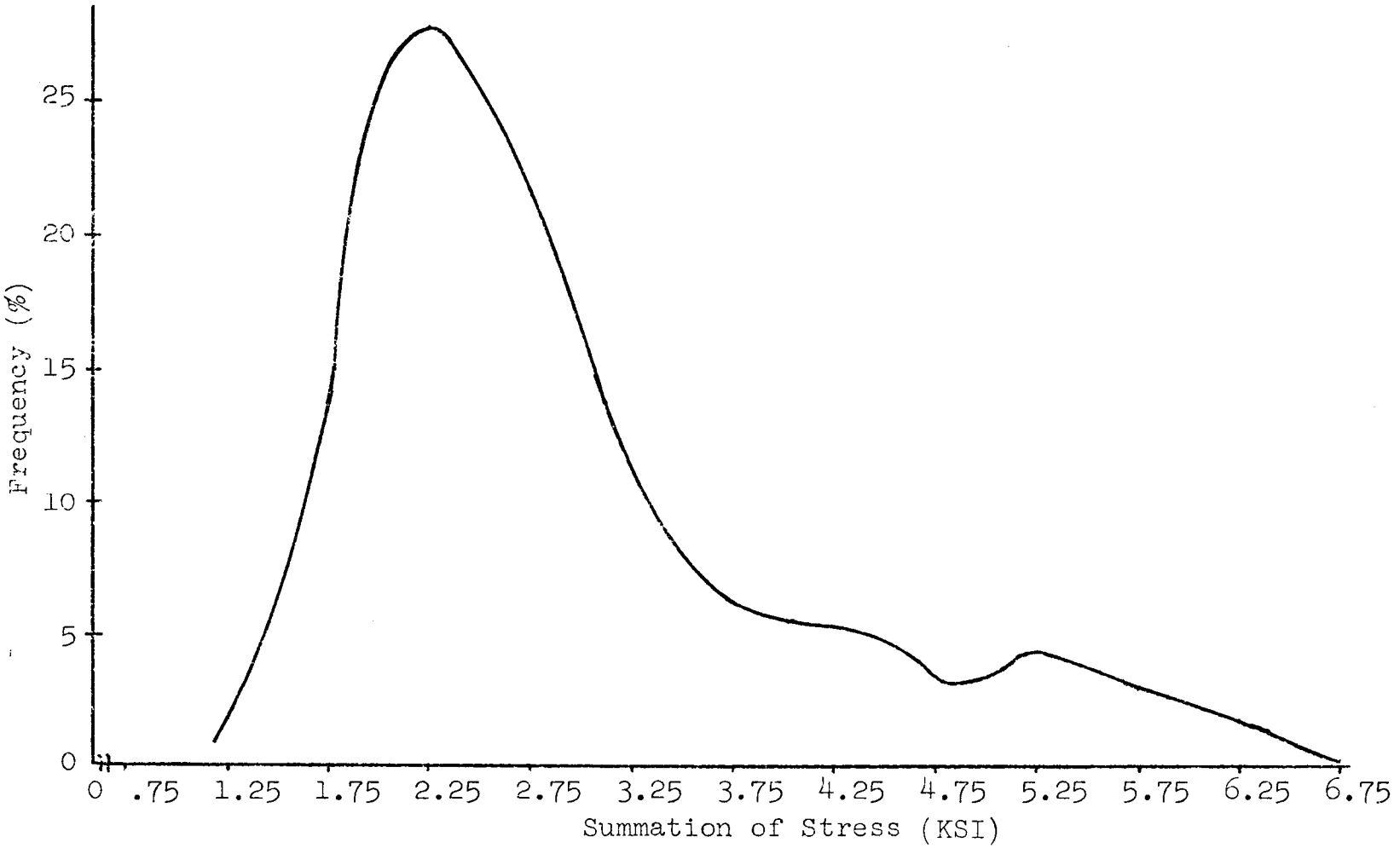


Figure 14. Summation of Actual Girder Stresses Versus Frequency - 2D Trucks.

CHAPTER V

THEORETICAL ANALYSIS OF BRIDGE

1. DESCRIPTION OF FINITE ELEMENT MODEL

A finite element model of the bridge using the ICES STRUDL computer program was used for the theoretical analysis of the bridge. A segment of the middle of the bridge model is shown in Figure 15. The concrete slab roadway was idealized using 168 plate bending elements, while the girders were divided into 84 separate beam elements. As shown in Figure 15, the 47.5 foot width of the bridge was divided into 14 elements, with two elements between each of the steel girders (shown as heavy black lines), except for the exterior girders. The 62 foot length of the bridge was divided into 12 elements, with the divisions being such that the cover plates on the girders ended on element boundaries. The seven steel girders were divided into 12 segments each to coincide with the element division.

The slab elements were analyzed as plate bending elements, in which only bending deformations and no in-plane deformations were considered. All of the slab elements were seven inches thick except for the exterior elements on the extreme left and right sides of the bridge. Curbs on both of these sides required a larger element thickness to account for the increased stiffness provided by the curbs.

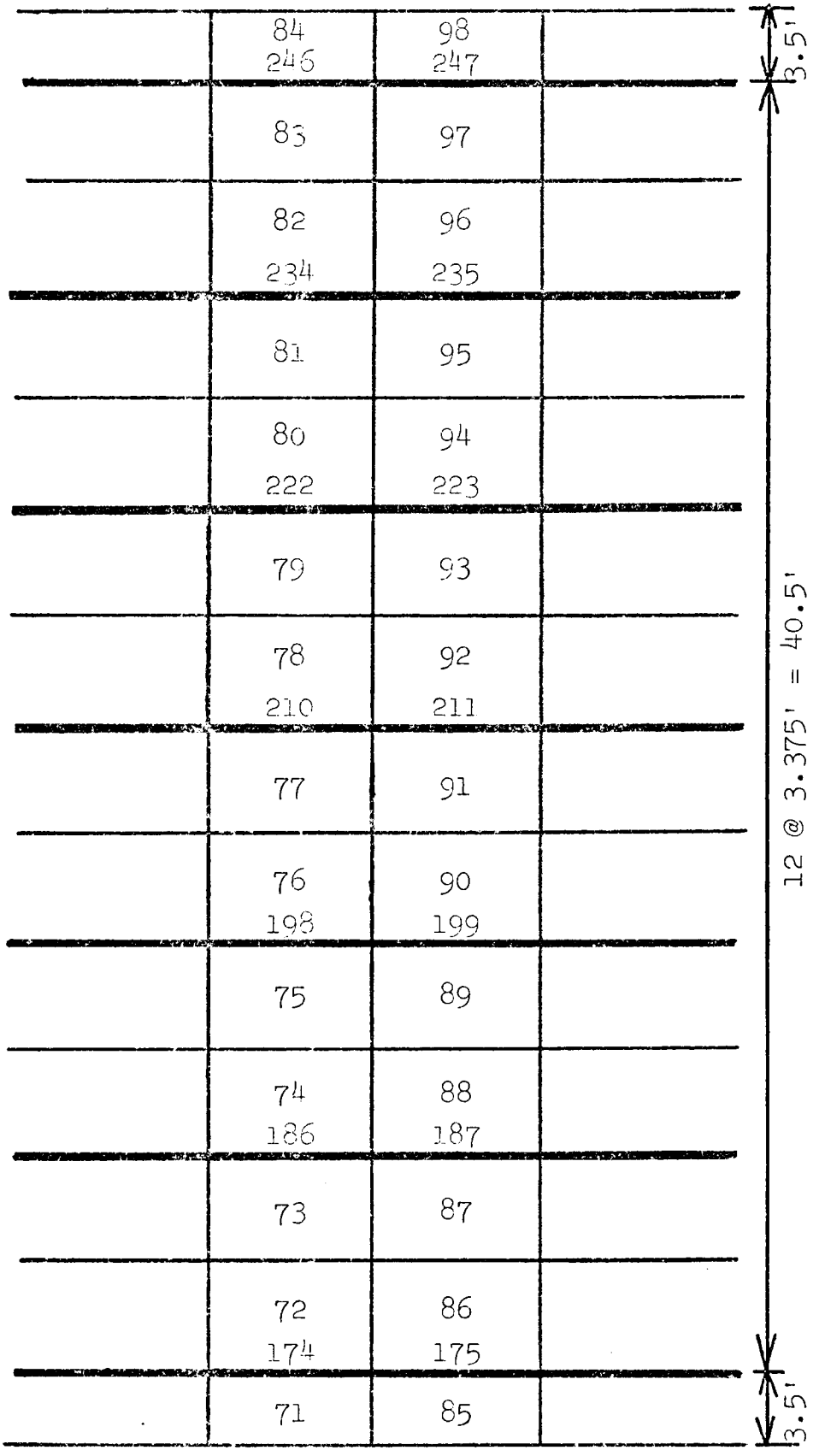


Figure 15. Section of Finite Element Model at Centerline.

A cross-section of the bridge at its centerline is shown in Figure 16. The thickness was calculated by computing the moment of inertia for the slab element alone, and transferring this moment to the centroid of the seven inch thick roadway. Knowing this moment of inertia, the thickness the curb elements needed to be to provide this moment of inertia was found from a simple inertia formula.[12] A modulus of elasticity of 4030 ksi was used for these elements.

The steel girders were analyzed as stringers supplying support to the slab elements. The STRUDL program calculated the moment of inertia for the slab about its centroidal axis only, and any additional properties had to be input by the user. This required that the additional moment of inertia supplied by the slab, due to the fact that the neutral axis of the bridge was below the slab's neutral axis, be added to the moment of inertia of the girders. The slab thus had to be transformed into an equivalent area of steel and the moment of inertia calculated. Since the cover plates covered only the middle 42 feet of the 62 feet long girders, two separate cross-sections had to be analyzed: one cross-section with cover plates and one without them. For each of these cross-sections, the moment of inertia for an exterior beam and an interior beam had to be calculated, since the exterior beams were adjacent on one side to the curb elements. A modulus of elasticity of 29,000 ksi was used for all steel elements.

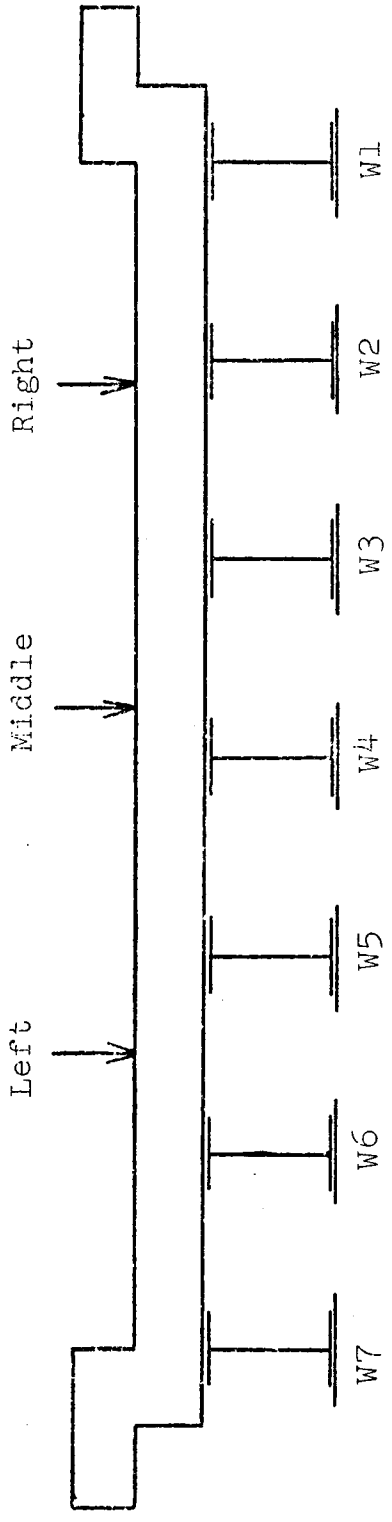


Figure 16. Cross-Section of Westbound Bridge at Centerline.

2. INPUT AND OUTPUT OF MATHEMATICAL MODEL

The effect on girder stresses of truck loadings on various positions on the bridge was desired. Since each transverse loading required many tedious calculations, a short computer program was written to generate these loads, given the weight and spacing, and output them on cards. These cards were then used as the load input for the STRUDL analysis program. The wheel loadings were idealized by using point loads for all axle configurations, including tandem axles. For example, the trailer wheel loadings for a 3S2 truck were represented by two loads, one for each of the four wheels on either side of the trailer. Figure 1, page 7, shows arrows for the load placements for 3S2 and 2D trucks. A standard width of six feet was assumed for all trucks.

Once the loads were input into the computer program and the program had been run, the results of the analysis had to be interpreted. Answers were in the form of both girder moments and element moments. To compare the answers with the actual data, however, the moments had to be transformed into stresses at the bottom of the steel girders. This was done by adding the moments for each slab element adjacent to a girder to the moment for that girder. This total moment was then converted to the stress at the desired point from the equation:

$$\text{Stress} = \frac{M\,y}{I}$$

where

M = Moment

I = Moment of inertia

y = Distance from centroid to point in question

Because of the many computer runs made to analyze the loads, a short computer program was written to automatically take the output from the STRUDL runs, which was put on disc, and calculate the stress in each girder.

3. STRESS EQUATIONS

3S2 Trucks

An equation was desired to describe the summation of the seven girder stresses for 3S2's as a function of gross vehicle weight, front axle weight, middle-to-rear axle weight ratio, and rear axle spacing. Since the summation of the stresses was assumed constant regardless of lane position, the analysis was made with the truck loading in the middle lane only. One computer run was made with a 20 kip load on the front axle only. The front axle spacing was set at 12.84 feet, the average spacing. This load was set 12.84 feet off the longitudinal centerline of the bridge, too, because the maximum moment at the centerline of the bridge occurred when the middle axle was on the centerline. Other runs were made with a 50 kip load divided between the middle and rear axles in varying ratios. One loading was run with a 31 percent distribution of load to the middle

axle, while another loading had 69 percent on the middle axle. Each of these ratios was analyzed with two different rear axle spacings.

Figure 17 shows that, by knowing the stresses caused by two axle weight ratios for two different spacings, an equation can be written describing the stress caused by any other combination of these variables. The straight lines can be drawn through the two points because a change in axle spacing causes a linear change in stress. The intercept point "A" is easily found, and the slope of any other axle weight distribution line is a linear variation between the 31 percent line and the 69 percent line. An equation was formulated which describes the summation of the seven girder stresses at the bridge centerline for any 3S2 truck with any combination of the four variables mentioned. The equation is shown below:

$$\text{Stress} = \left(\left\{9.8722 - \left(.22045 - \left[.003194 * (\text{PER} - 31)\right]\right) * \text{SPACE}\right\} * \left\{(\text{WT} - \text{FRWT})/50\right\}\right) + \left((\text{FRWT}/20) * 2.319\right)$$

where

PER = Percentage of weight on middle axle

SPACE= Rear axle spacing

WT = Gross vehicle weight

FRWT = Front axle weight

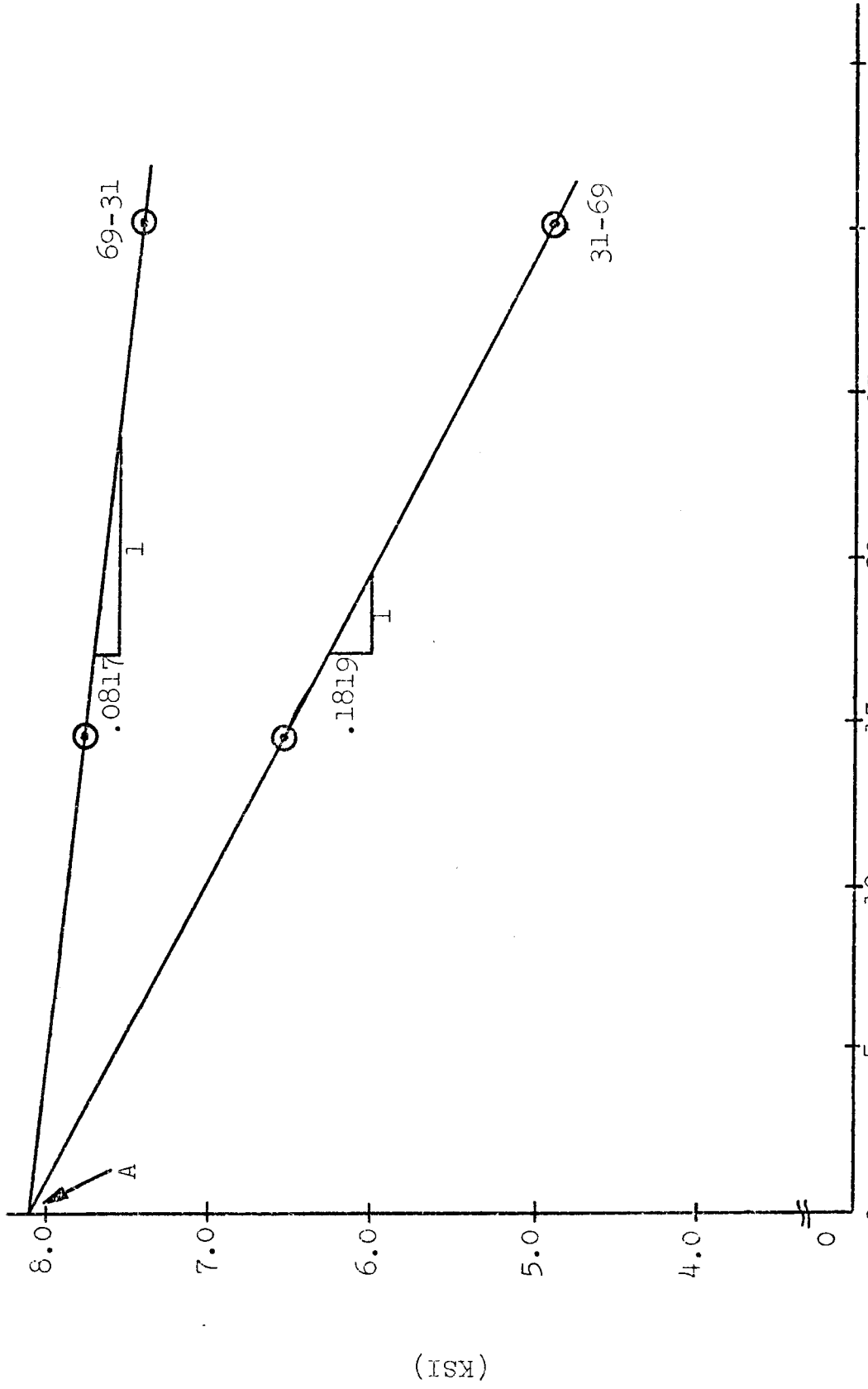


Figure 17. Summation of Stresses Versus Axle Spacing.

The equation is divided into two parts, the first part describing the stress caused by the middle and rear axle weights and spacing, and the second part describing the stress caused by the front axle weight alone. The numbers 50 and 20 appearing respectively in the first and second parts of the equation are used to modify the stresses from the original computer runs to account for trucks with weights other than these two figures. This implies that the bridge is completely linear in its reaction to trucks of different weights. This is theoretically true, and actual testing of bridges has shown this to be true also.[7] Similar equations can be written for the stress in an individual girder for any transverse position of the truck loading. This requires that four computer runs be made, including runs with each of the axle spacings for each of the axle weight distribution ratios, for each transverse truck position desired. Because of the many computer runs involved, equations were derived for each of the seven girders in each of the three traffic lanes only.

2D Trucks

An equation was desired for 2D trucks that represented the summation of the seven girder stresses as a function of gross vehicle weight, axle weight distribution, and axle spacing. As with the 3S2 trucks, the equation for the summation of stress was derived from computer runs with trucks in the middle lane only. Since 2D's have only two axles,

the maximum stress in the girders at the middle of the bridge occurred when the heaviest axle was on the longitudinal centerline of the bridge. This required two equations to be written, one to describe each of the cases when either the front or the rear axle was on the middle of the bridge. The equation for when the front axle was at the longitudinal centerline was the same as the first part of the equation for the summation of stress for 3S2's. The equation for the summation of stress when the rear axle is at the centerline is as follows:

$$\text{Stress} = \left\{9.8760 - \left[.219 - (69 - \text{PER})(.00317105)\right] * \text{Space}\right\}\frac{GW}{50}$$

where the variables are as previously defined.

It should be noted that this equation is very similar to the first part of the equation for 3S2's. The differences in the two equations, except for the terms "(69 - PER)" and "(PER - 31)", are attributable to the bridge not being quite symmetric about the centerline. The change to the term "(69 - PER)" for the rear axle on the longitudinal centerline was required in order to describe the equation as a linear variation of the highest weight ratio on the front axle (see Figure 17, page 64). Equations were also written to describe the stress in the girders for a truck loading in each of the three traffic lanes.

CHAPTER VI

CORRELATION OF ACTUAL AND THEORETICAL DATA

1. CONTROLLED LOAD TEST

A special test was performed in which a 3S2 truck was rented and strain data were collected when only the rented truck was on the bridge. The truck was loaded with concrete blocks, and then each axle was weighed at two separate official weigh stations. The average gross vehicle weight from the two weighings was 74,265 pounds. The front axle weight averaged 8910 pounds, with 51 percent of the remaining weight being distributed to the middle axle and 49 percent to the rear axle. The front and rear axle spacings were measured to be 12 feet, 4 inches and 26 feet, 7 inches, respectively. Thus, with the loading and spacing characteristics of the truck being completely defined, a theoretical stress for each girder and a summation of stresses for all seven girders were calculated. These were compared with the actual stress data in order to gauge the accuracy of the theoretical stress equations in predicting stress.

Controlled truck runs, runs with only the rented truck on the bridge, were performed with the truck in each of the three traffic lanes for each of the speeds listed below:

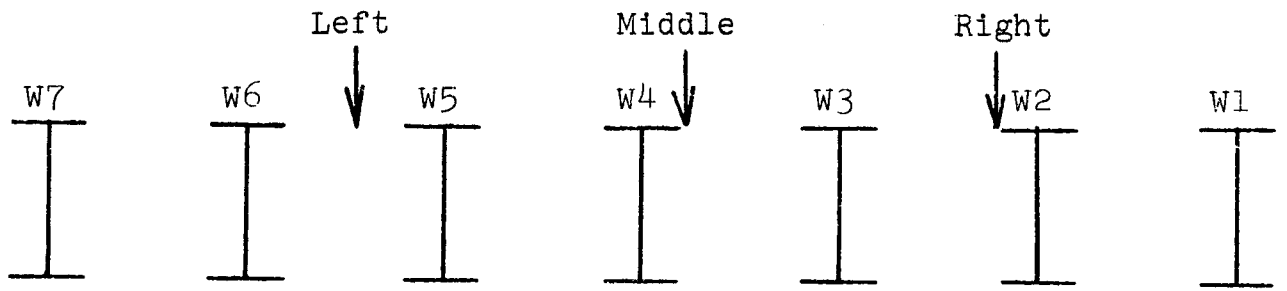
1. Crawl speed (7 miles per hour or less)
2. 30 mph
3. 55 mph

Because these special tests were run under normal traffic conditions, it was impossible to have only the truck on the bridge with no cars present. An exception to this was the crawl speed controlled runs. Because of the extremely slow speed and the attendant dangers of having the truck slow down to this speed under normal traffic conditions, these runs were made late at night with a policeman directing the traffic flow. Therefore, cars were on the bridge for only one of the six crawl speed runs. Because of the extreme weight difference between the rented truck and the few cars that might be present on the bridge, however, the presence of cars had little effect on the girder stresses.

Table VI shows the actual stress data from the crawl speed control runs. It should be noticed how closely the summations of the stresses were for all the runs regardless of lane position, except for run No. 5. No explanation is available for why the individual girder stresses on this run were consistently higher than they were for the other left lane controlled run, No. 6. Cars on the bridge were eliminated as a reason because run No. 1 was the only run with any cars on the bridge. If run No. 5 is discarded, however, the summation of the stresses vary by only a maximum of 2 percent. This fact lends credence to the assumption that the summation of the girder stresses is constant for a given truck regardless of lane position. It is also apparent that the girder nearest the load centerline is the most highly stressed girder. Girders G4 and G2, which

TABLE VI

CRAWL STRESS DATA (KSI)



Lane Position									Summation of Stress
Right	1)	103	91	251	666	1408	2398	1560	6477
	2)	114	240	283	678	1257	2339	1537	6448
Middle	3)	103	493	1290	2293	1397	636	160	6372
	4)	103	481	1248	2281	1467	694	160	6434
Left	5)	1158	2145	2035	982	314	117	103	6854
	6)	1032	2030	1972	947	302	88	103	6474

are situated almost directly under the load centerline for the middle and right lanes, are stressed a minimum of 38 percent more than either of the adjacent girders. Girders G6 and G5, where the load centerline is almost halfway between the girders, carry approximately the same stress.

Table VII shows a comparison of controlled runs at two different speeds for each of the three lanes. No definite conclusions are obvious from these data. In each of the three lanes, the summation of the girder stresses for the 55 mph runs is higher than for the 30 mph runs. This is not true for each of the individual girders, however, where the girder stress is in several cases higher for the 30 mph runs than for the 55 mph runs. Each of these runs, Nos. 1-6, is an averaged value from several runs from the same lane at the same general speed. Considerable scatter was observed for the summation of stress in all the runs, even between runs in the same lane at exactly the same speed. The summation of stress was even higher for some runs at lower speeds than it was for runs at faster speeds. This type of scatter is to be expected in considering the dynamic impact effect on stresses, though. In general, by comparing the 30 mph runs to the 55 mph runs, and in turn comparing these to the crawl run data in Table VI, page 69, it can be concluded that the dynamic impact effect increases as the speed of the truck increases. No attempt was made to formulate a dynamic impact factor based on speed or on any other variable.

TABLE VII

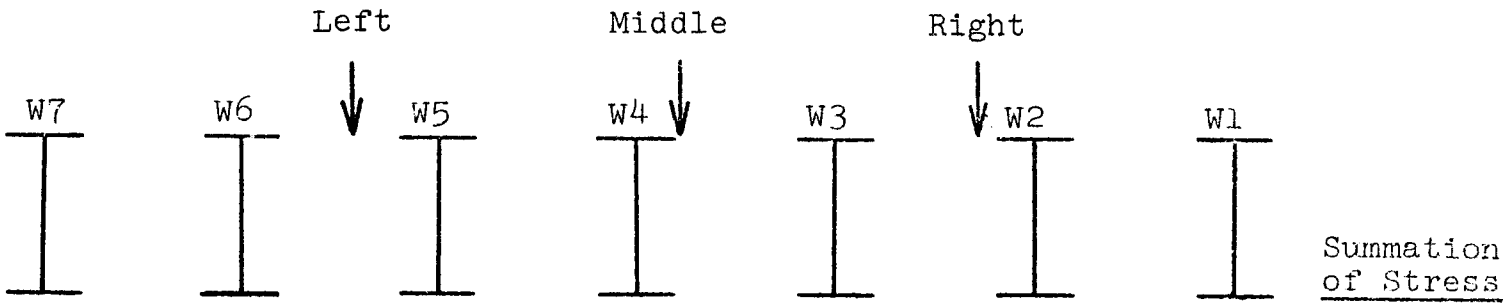
ROLLING LOAD STRESS DATA (KSI)

Lane Position		Speed (MPH)	W7	W6	W5	W4	W3	W2	W1	Summation of Stress
				Left ↓		Middle ↓		Right ↓		
Right	1)	55	229	237	353	912	1637	2248	1422	7037
	2)	30	123	103	312	754	1481	2415	1402	6589
Middle	3)	55	306	661	1566	2527	1327	678	279	7344
	4)	30	206	722	1427	2580	1467	724	126	7250
Left	5)	55	1611	2546	1972	801	297	166	195	7587
	6)	30	1158	2156	1878	936	337	185	137	6787

The controlled truck runs, in particular the crawl speed runs, presented an excellent opportunity to gauge the accuracy of the theoretical model in predicting girder stresses. The crawl runs were, in effect, static loads with little or no dynamic effects to scatter the stress data. This has been shown to be true by the extremely small amount of scatter observed in the crawl speed stress data (Table VI, page 69). Because the mathematical model of the bridge was a static model and not a dynamic model, the theoretical data should best compare with the crawl speed stress data. Table VIII shows a comparison between these two sets of data for both the individual girder stresses and the summation of stresses for the middle lane. The stresses in each of the girders are shown plotted versus position on the bridge in Figure 18. It can be seen that the theoretical stresses are much higher than are the actual stresses. Larger moments of inertia for the mathematical model elements would be required to lower the theoretical stresses. No explanation is available for the large difference between the theoretical and actual data. The bridge was modeled compositely, as it was constructed compositely, and this supplied the largest moment of inertia possible. Factors that could change the moment of inertia would be to consider the steel in the concrete and to model the skew in the bridge. These would have very little effect, however, since little steel was present in the concrete and the bridge was only slightly skewed. The modular ratio used to transform the concrete into an equivalent

TABLE VIII

COMPARISON OF THEORETICAL AND CRAWL RUN STRESS DATA (KSI)



	W7	W6	W5	W4	W3	W2	W1	Summation of Stress
Crawl Run Data	103	481	1248	2281	1467	694	160	6434
Theoretical Data	0	679	2105	3371	2454	846	67	9522

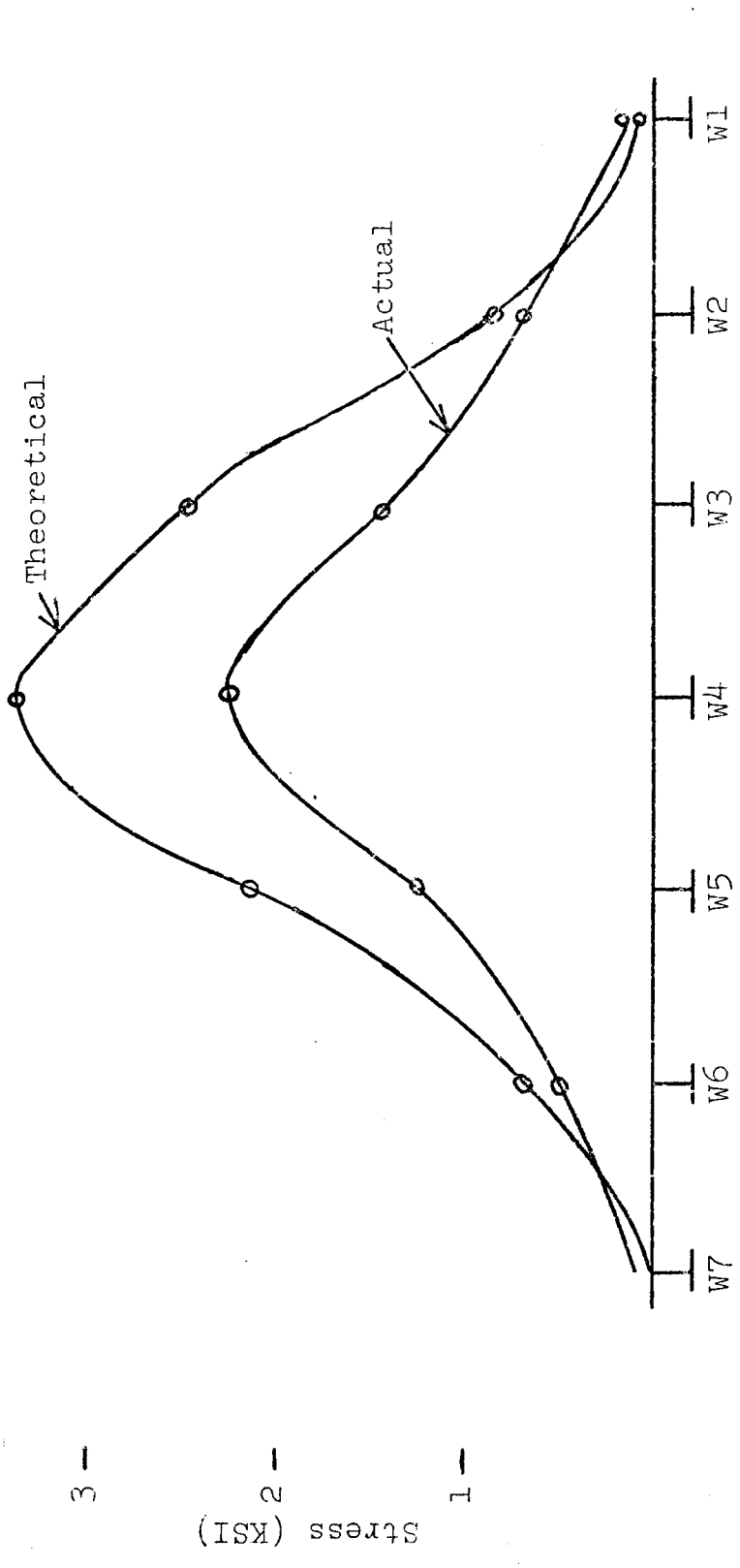


Figure 18. Theoretical Versus Actual Girder Stresses for 3S2 in Middle Lane

area of steel was assumed to be 7.2, to correspond with Young's Modulus of 29,000 ksi for steel and 4030 ksi for 5000 psi concrete. A change in this ratio to a smaller number would have the effect of increasing the moment of inertia, also.

2. MONTE CARLO SIMULATIONS - 3S2 AND 2D

Once the truck characteristics have been defined for 3S2's and 2D's, and equations in terms of these characteristics are available to calculate the summation of the girder stresses for both truck types, a correlation of theoretical to actual data is possible. The most feasible and accurate way to represent the truck characteristics is by the use of a Monte Carlo simulation. This method involves simply generating a random number between 0 and 1, going to the desired distribution (such as a distribution of gross weight) where the frequencies are tabulated in cumulative form between 0 and 1, and picking out the number from the table whose cumulative frequency matches the random number. These tabulated distributions and how they were formed were explained in Chapter IV. They are simply histograms in cumulative percentage form of gross weights, front axle weights, middle-to-rear axle weight ratios, and axle spacings. Interpolation between frequencies is used when no frequency exactly matches the random number. To simulate the truck characteristics for a 3S2, a random number would have to be generated and

a number picked from each of the cumulative distribution tables mentioned above. For 2D trucks, numbers would have to be selected from each of the 2D cumulative distribution tables mentioned above except for the front axle weight table. This type of simulation, if run for a large enough number of cycles, will give distributions of these individual variables that are very close to the exact distributions if all of the pertinent variables have been considered.

3S2 Simulation

Figure 19 shows a flow chart of the Monte Carlo simulation for 3S2 trucks. A random number is generated to select a gross vehicle weight. Once the gross weight of the truck is known, a middle-to-rear axle weight ratio and a front axle weight can be selected from the tables for the proper weight intervals by the use of two more random numbers. Finally, a random number is generated to select a rear axle spacing, which has been shown to be independent of the weight of the truck. Knowing these four truck characteristics, the equation for the summation of stress, which is a function of these four variables, can be used to predict the stress that a particular truck would cause. This must be done for many trucks, however, to properly represent the distributions. A computer program was written, therefore, to perform all the stress calculations and to form a stress histogram, for any desired number of trucks. The theoretical stress distribution for 3S2 trucks settled

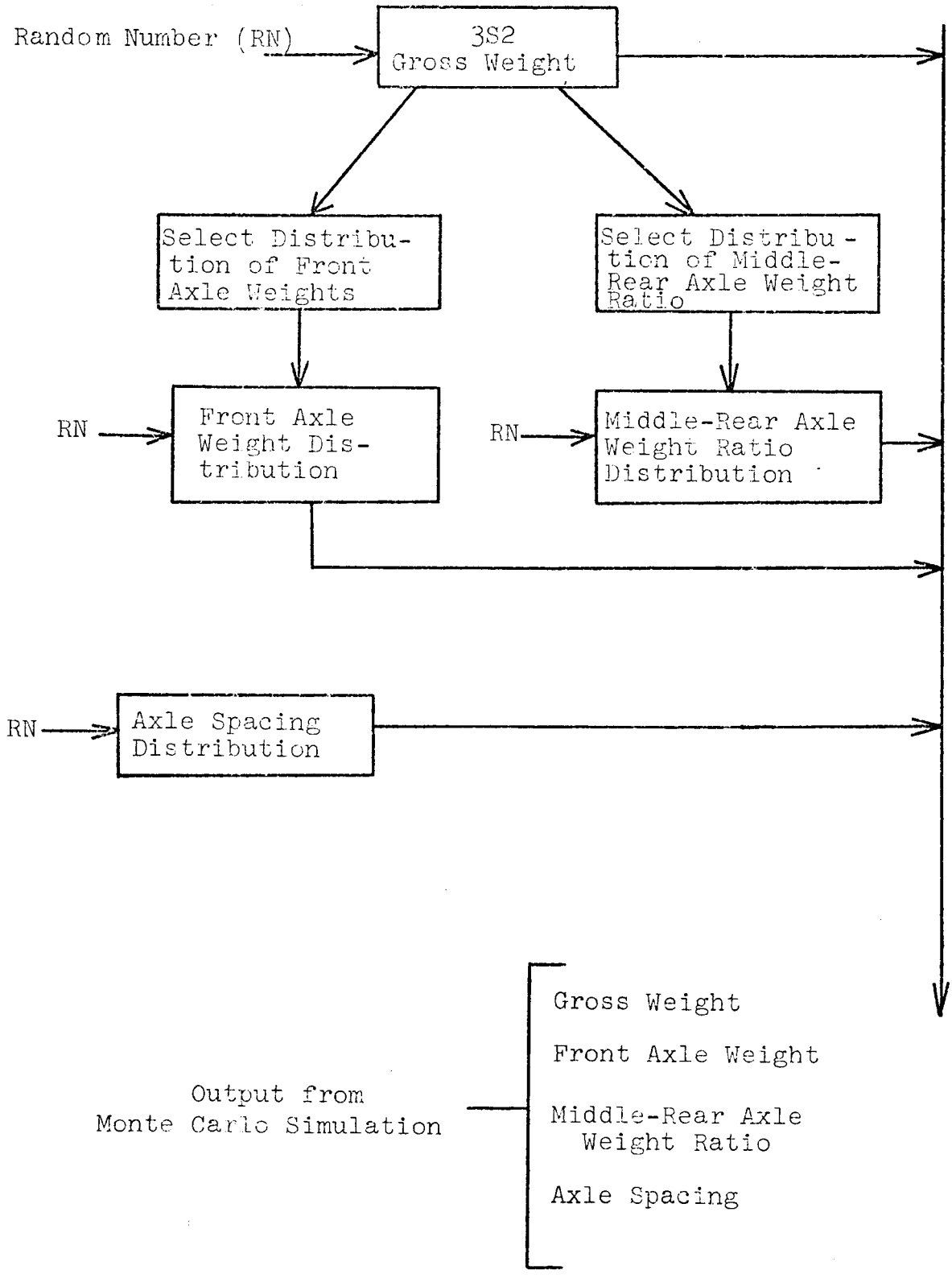


Figure 19. Flow Chart for 3S2 Trucks.

down at 2000 cycles. Figure 20 shows the stress distribution for the summation of the girder stresses that was generated by the Monte Carlo simulation, along with the actual stress distribution for 3S2's. Also shown on Figure 20 is a graph of a revised theoretical stress distribution with the AASHTO dynamic impact factor included. This revised theoretical stress was calculated in an attempt to bring the theoretical stresses in line with the measured stresses. To do this, the theoretical summation of the stresses for the controlled truck tests was divided by the summation of the stresses for a crawl speed control run in the middle lane (see Table VIII, page 73). This provided a factor of 1.47, which was then divided into each theoretical stress. To include dynamic impact effects, the AASHTO dynamic impact factor, calculated to be $50/(L + 125) = 1.27$, was then multiplied by the revised stress summation. However, the theoretical stress distribution still coincides better with the actual stress distribution than does the revised theoretical stress distribution. Since it is obvious from the crawl speed control runs that the theoretical summation of stresses is too high, other factors not considered must be effecting the revised theoretical stress distribution. One possible factor is that the correction coefficient from theoretical stress to revised theoretical stress could vary with different weight and length trucks. Another variable to be considered is the fact that the population of trucks at the bridge site and at the weigh station is possibly

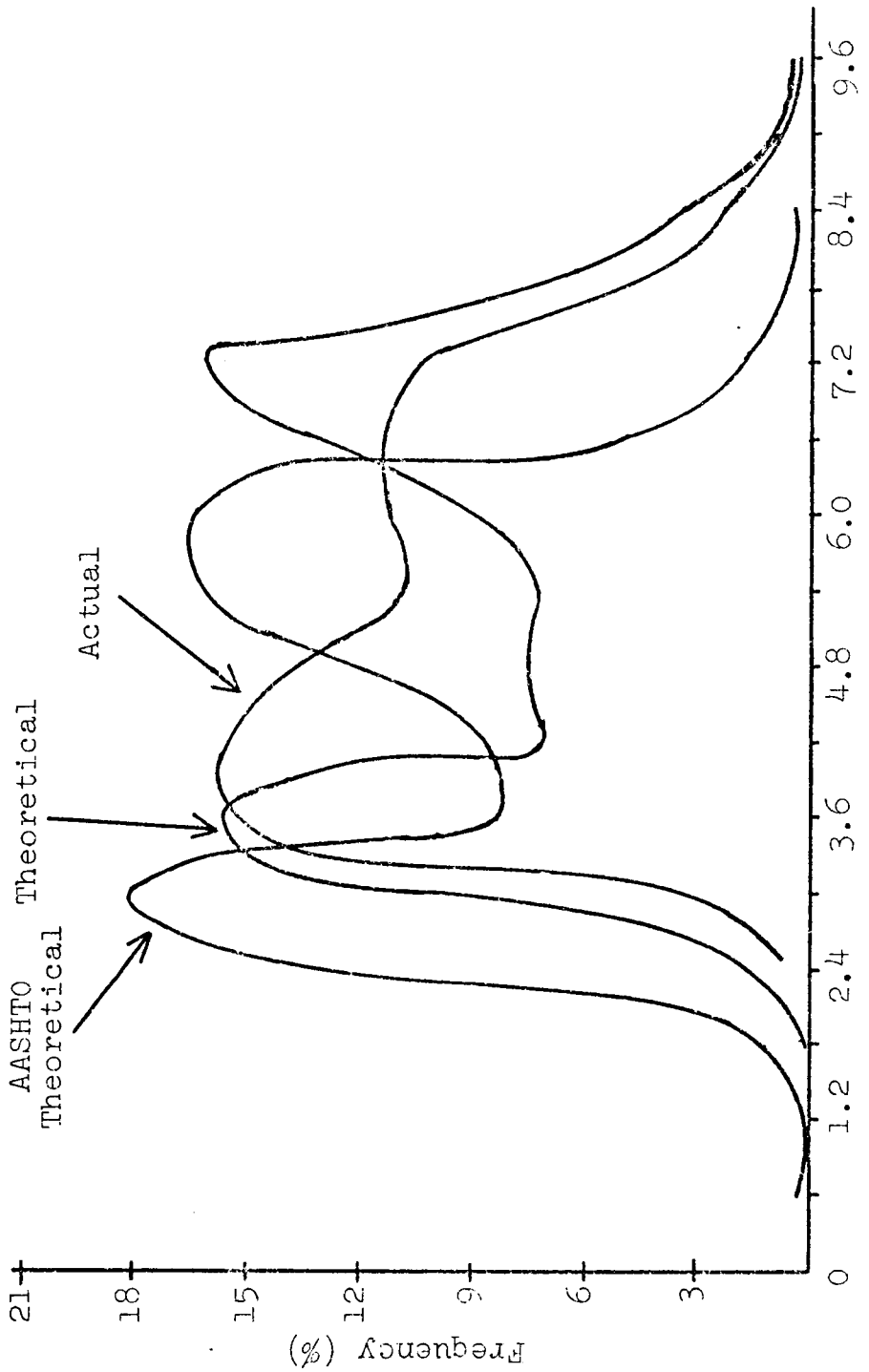


Figure 20. Comparison of Actual and Theoretical Summation of Stresses - 3S2 Trucks.

not the same. Local, lighter weight trucks probably cross the bridge but turn off before they get to the weigh station, which is ten miles from the bridge. Because of these variables and the need to define them more closely, no conclusions were reached on correlating theoretical-to-actual stresses for 3S2 trucks.

2D Simulation

A Monte carlo simulation very similar to that for 3S2 trucks can be run for 2D trucks. A random number to select the gross weight must first be generated. Knowing the gross weight, the proper weight interval table for front-to-rear axle weight ratios may be entered, and, using a random number, a ratio is selected. An axle spacing, which for 2D's has already been shown to be independent of gross weight, may be selected by using a third random number. By knowing these three variables, a stress may be calculated using the proper equation. For 2D trucks, two equations were derived to compute the stress. One equation was used if the front axle weighed more than the rear axle, and a second equation was used if the rear axle weighed more than the front axle. The proper equation must be selected in order to obtain the maximum stress for the bridge. This is easily done, however, because the front-to-rear axle weight ratio is known, and thus the correct equation can be used. Figure 21 shows a graph of the actual summation of stress data and a Monte Carlo distribution of the

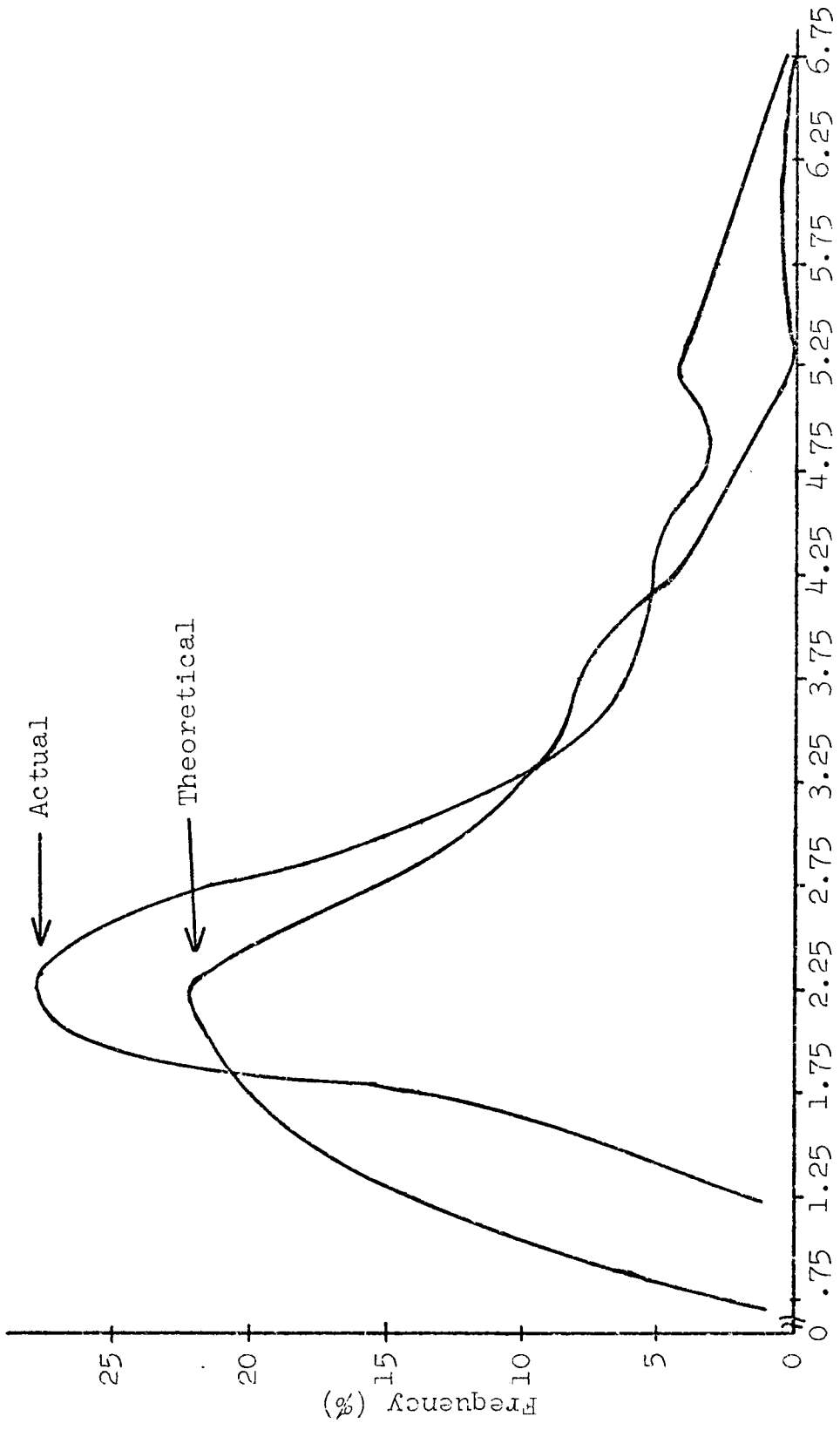


Figure 21. Comparison of Actual and Theoretical Summation of Stresses - 2D Trucks.

theoretical stress data. It can be seen that these two distributions have a better correlation than did the theoretical and actual stress distributions for 3S2 trucks. No revised theoretical stress was calculated for 2D trucks because no correction factor was available for 2D's to bring the theoretical stresses in line with the actual stresses. The correction factor for 3S2's was considered inappropriate because of the extreme weight and spacing difference between the 3S2 test truck and the average 2D truck. No conclusions were made on a theoretical-to-actual stress relationship for 2D trucks, either, because the same variables that needed to be analyzed for 3S2 trucks also need to be studied more closely for 2D's before any concrete relationships can be developed.

CHAPTER VII

SUMMARY AND CONCLUSION

I. SUMMARY

Two bridges in Knoxville, Tennessee, which carried eastbound and westbound traffic on Interstates I-40 and 75 were instrumented with strain gages. A computerized recording system utilizing magnetic tapes was used to record strain data for a large sample of individual trucks. A special test was also performed in which a 3S2 truck of known weight and length was driven over the bridge while the girder strains were being recorded. Strains were recorded at midspan of the bridges, with the gages located at the bottom of each of seven steel girders. Truck type and lane were manually recorded for each truck that was sampled.

The strain data that were recorded for the westbound bridge were reduced on a computer into maximum stress ranges for each girder caused by each individual truck. In addition to the maximum stress range for each girder, the summation of the seven girder stresses was also printed. These data were then separated into stresses caused by individual truck types in each of the three traffic lanes. Histograms were then formed for both individual girder stresses and summation of girder stresses for each major truck type traveling in each lane.

Weight data from a weigh station located on Interstate 40 were available from previous studies. Axle spacing data were obtained for all type trucks at the same weigh station. These data were separated into parts representing the two most numerous truck types, 3S2 and 2D trucks. Gross weight histograms were assembled for each truck type. For 3S2 trucks, front axle weight histograms and middle-to-rear axle weight ratio histograms were formed for each 6 kip weight interval over the effective range of weights. For 2D trucks, a histogram of the front-to-rear axle weight ratios was formed for each 4 kip interval over the effective weight range. Rear axle spacing histograms were assembled for both 3S2's and 2D's.

A finite element model of the westbound bridge was developed to study the theoretical stresses that should occur in the bridge girders. An equation was developed for 3S2 trucks from computer runs of this model to describe the summation of girder stresses for an individual truck as a function of gross weight, front axle weight, middle-to-rear axle weight ratio, and rear axle spacing. Equations were also formulated to predict the stresses in individual girders as a function of these variables. A similar equation for the summation of the stress was developed for 2D trucks as a function of gross weight, front-to-rear axle weight ratio, and axle spacing.

A Monte Carlo simulation of the various truck characteristics was used to represent the variables appearing

in the summation of stress equations for 3S2 and 2D trucks. The theoretical stress distributions obtained from the Monte Carlo simulations for these two truck types were compared to their respective actual stress distributions. The distributions for 3S2 trucks showed some similar characteristics, but, in general, a good correlation was not exhibited. The actual and theoretical stress distributions for 2D trucks showed very similar characteristics in the shape of the summation of stress versus frequency curve. No attempt was made to make definite conclusions on a theoretical-to-actual stress relationship that would be applicable to other bridges because of various factors apparently having a significant effect on both distributions that need further investigation.

II. CONCLUSION

Significant work was done in this study on steps 3, 5, and 6 of the seven steps (page 23) necessary to predict girder stresses. Although no definite conclusions could be made from comparing the theoretical stress distributions to the actual stress distributions for 3S2 and 2D trucks, similar shapes were exhibited by both distributions for both truck types. The stress distributions for 2D trucks were especially similar. The method that was used to compare actual stresses to theoretical stresses in bridge girders shows promise as an intermediate step in predicting the number of loading cycles a bridge will undergo, given the truck type and weight distribution for a bridge.

III. SUGGESTIONS FOR FURTHER RESEARCH

Additional field testing needs to be performed with emphasis on a one truck - one stress - one truck weight relationship. This could be accomplished by sampling the strains for a particular truck as it crosses the bridge, recording the truck's license plate number, and weighing the truck and measuring the axle spacings if it arrives at the weigh station. The stress data could then be separated into histograms representing stress data for all trucks of a particular type, and stress data for only those trucks arriving at the weigh station. The latter histogram, if a large enough sample of trucks is obtained, should compare more closely to the stress histogram from the Monte Carlo simulation of the stress data than did the stress histogram for all trucks of a particular type. By separating the stress data into these two parts, any uncertainties in the comparison of actual and theoretical stresses histograms caused by differences in the truck population at the bridge site and at the weigh station are eliminated.

Work needs to be done to find the cause of the differences between the theoretical and actual stresses in the bridge girders. Changes in the mathematical model could then be made in an attempt to lower the theoretical stresses so that they compare more closely with the actual stresses. If all reasonable changes have been made to the model and a close theoretical and actual stress comparison has not

been obtained, controlled load tests could then be used to "calibrate" the theoretical stresses to bring them in line with the actual stresses. A ratio of theoretical-to-actual stresses for a "crawl" controlled load test could be used to adjust all theoretical stresses. This was attempted in Chapter VI for 3S2 trucks, but differences in truck population at the bridge and the weigh station made the results of any comparison uncertain. Controlled load tests for 2D trucks should also be run to see if the stress adjustment factor varies from that of 3S2 trucks.

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VITA

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