To the Graduate Council:

I am submitting herewith a thesis written by Charles Raymond Bolt entitled "A Simplified Approach for Determining Stresses on Highway Bridges." 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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A SIMPLIFIED APPROACH FOR DETERMINING STRESSES ON HIGHWAY BRIDGES

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee

Charles Raymond Bolt

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ABSTRACT

The purpose of this study is to develop a simplified method for determining the stress cycles in highway bridge girders due to present day traffic. The simplified method generated is applicable to general I-beam stringer deck bridge configurations.

Two interstate highway bridges located in Knoxville, Tennessee were instrumented, and stress data were collected with the use of a Raytheon 703 computer. The Raytheon computer, along with a magnetic tape unit, was housed in a mobile trailer. Stress data and truck traffic characteristics were obtained and compiled into stress and weight histograms respectively. The actual data were compared to the theoretical analysis, which was determined by the use of finite element methods and Monte Carlo Simulations. From the results of the comparison, a simplified method for determining stresses on the bridges was generated. This method was modified and may be used for more general bridge configurations.

The similarities in the results were found to be very favorable. Further research was suggested to find a method to distribute the stresses to each individual girder. Once the individual girder stresses are known, along with truck traffic characteristics, it becomes possible to approximately predict the fatigue life of the bridge.

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CHAPTER I

INTRODUCTION

1. General

The Federal Highway Administration has financially aided research and development of design and analysis methods for highway bridges with respect to fatigue for the past seven years. The emphasis of stress history research programs has been toward developing a method for determining stresses which a highway bridge might be expected to experience due to present day traffic. The consensus of stress history studies to date directly relates the increase of truck traffic to the increase of highway bridge fatigue failures. Present day design regarding fatigue strength is based on calculating the number of cycles of stress ranges that occur. Hence, research programs have developed methods for obtaining data that will ultimately be used in generating a fatigue design procedure based on stress ranges induced by present day truck traffic.

With the use of minicomputers and magnetic tape data collection processes, instrumentation and testing of bridges has become a very prominent method within the realms of stress history researchers. The idea was to permanently record the measured strain experienced by a bridge girder onto

magnetic tape, and simultaneously register truck traffic characteristics. The recorded strains are read off the tape and reduced to corresponding stresses by use of an appropriate computer program. The collected data are reduced to stress histograms and truck traffic characteristics (i.e., gross weight histograms, lane position data, truck type data, etc.). The actual results are compared to theoretical analyses with the hopes of furthering the knowledge of bridge design techniques with respect to fatigue limits.

2. Objective

The objective of this study was to develop a simplified method for determining the maximum and "typical" stresses experienced by a highway bridge of known cross section. Minor importance was applied to acquiring a means by which the truck traffic characteristics might be closely approximated. It was desired to be able to apply the generated method to present day fatigue design procedures without the use of computers so as to make possible reasonable fatigue limit analyses in a moderate amount of time. The technique used for solving this problem was to determine a procedure for relating measured stresses to calculated stresses by means of a model of the tested bridge loaded with the actual truck traffic characteristics. Thus, a calculated-to-measured stress correlation could be predicted. The fatigue limit could then be approximated with the known truck traffic distribution.

3. Procedure

The initial step was to correlate the measured to calculated stresses due to actual truck loadings on a full scale tested bridge. Two, three lane interstate steel girder bridges in Knoxville, Tennessee were instrumented with strain gages to obtain measured strains. A control load truck test vehicle was used for the purpose of comparison. Finite element models of each bridge (one for the eastbound bridge and one for the westbound bridge) were developed. Each model was loaded with a simulated representation of the control load truck used in recording measured stresses. Theoretical moments were calculated using the ICES STRUDL II computer stiffness analysis procedure, and corresponding stresses were formulated from the computer results. Relatively close comparison of the actual measured stress to the calculated stress caused by the test vehicle satisfactorily completed the initial step.

Secondly, a method for analytically describing the actual stresses was generated by following the statical methods and bridge cross sectional properties used in the computer analysis. A previous investigator involved in this stress history research program, Mr. Cooper Patrick, developed the general procedure and necessary correlations used in generating the simplified method eventually derived.[1*]

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*Superscript numbers in parenthesis refer to those entries in the List of References.

The procedure resulted in an equation involving the truck traffic characteristics that directly influence highway bridge stresses.

On the completion of the first step (i.e., having a 15 percent or less error when comparing the actual to theoretical stresses), the assumption was made that the computer analysis was a valid method for determining the stresses on the actual field tested highway bridge. Using the results of the computer analysis, a similar equation to that of the simplified procedure generated in step two was developed. Placing each equation into a Monte Carlo Simulation, along with identical truck traffic characteristics (which describe the actual truck traffic on the bridge tested), stress histograms were produced. Favorable comparison of the stress histograms directly influenced the next assumption that the simplified procedure developed adequately described the actual stresses measured.

The third step was to determine the validity of the simplified method developed by using the eastbound bridge and comparing the computer analysis to the simplified analysis. The westbound bridge was used in developing all previous analytical procedures. Further modification of the simplified method could then be applied until a systematic analysis of most bridge configurations could be done.

Finally it became necessary to determine a way to predict the truck traffic type and volume for any highway bridge.

The possibility of classifying trucks so each type would have known characteristics regarding the variables described in the generated equation produced by the simplified method (gross vehicle weight, axle spacing, and percent distribution of middle to rear axle weights with their respective standard deviations) was attempted. This would reduce the necessity of collecting volumes of data in order to estimate variables of each truck type. The only variables remaining would be the amount of truck traffic and a breakdown of the major truck types. With all the variables outlined in the above procedure, it is possible to predict the fatigue life of highway bridges.

CHAPTER II

RELATED RESEARCH

1. Review

Stress history of highway bridge research programs have been in progress for the past seven years. These programs have been carried on in several states throughout the nation, and funded through the assistance of the Federal Highway Administration (FHWA) with guidance by committees of the American Society of Civil Engineers (ASCE) and the Transportation Research Board (TRB). In order to correlate data collected by stress history programs in various states, a general description for truck type classification, shown in Figure 1, was developed and used by all of the stress history researchers.

The overall objective of the research that has been and is being conducted is to ultimately produce a method for highway bridge designers to check fatigue life due to present day traffic. The following will describe research relating to this objective done in two states, along with a report on progress made within the research program at the University of Tennessee of which this study is a part.

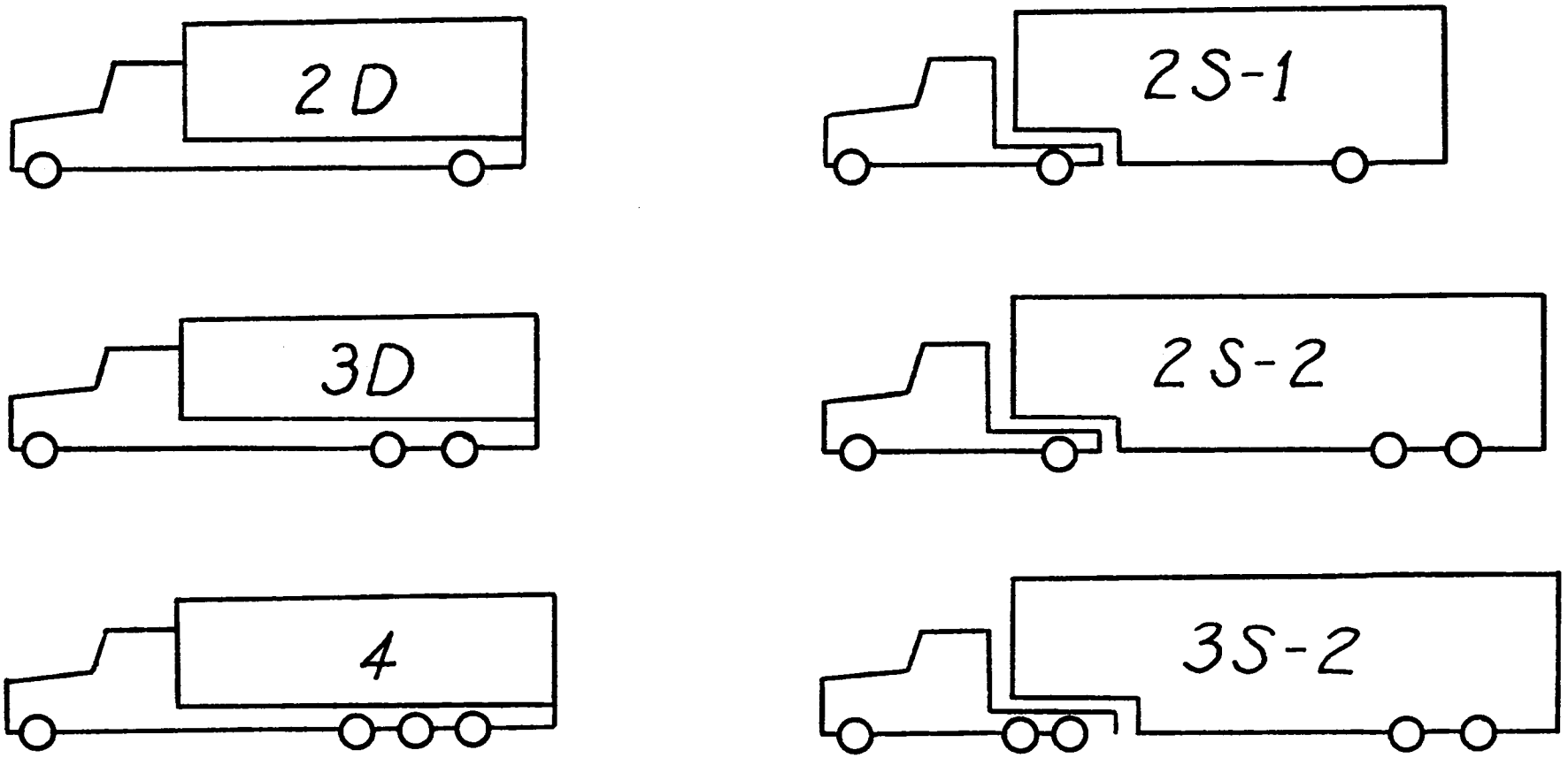


Figure 1. Truck Type Classification.

Illinois

The stress history research done in the state of Illinois was conducted by the Department of Civil Engineering at the University of Illinois, Urbana-Champaign, as a part of the Illinois Cooperative Highway Research Program.[3] The program was done in cooperation with the Illinois Department of Transportation and the FHWA. The research emphasized a probabilistic approach to generating a method for determining the stresses induced by present day truck traffic on a highway bridge.

Three bridges, two three-span continuous structures, and one two-span continuous structure, were investigated. A comprehensive, computer type, data collection process was utilized. A flow diagram simplifying the procedure used throughout the research done is shown in Figure 2. With the reduction of the pertinent data acquired in the field tests, an approach to the problem of simplifying stress calculations was made.

The main idea used was to produce functions of each random variable involved within the realm of the problem. This approach should compare favorably to the actual measurements in order to be valid. With the necessary correlation of the field data obtained, a simplified mathematical model was proposed which would be used to determine strains induced into the main load carrying members. The equation was developed into the following:

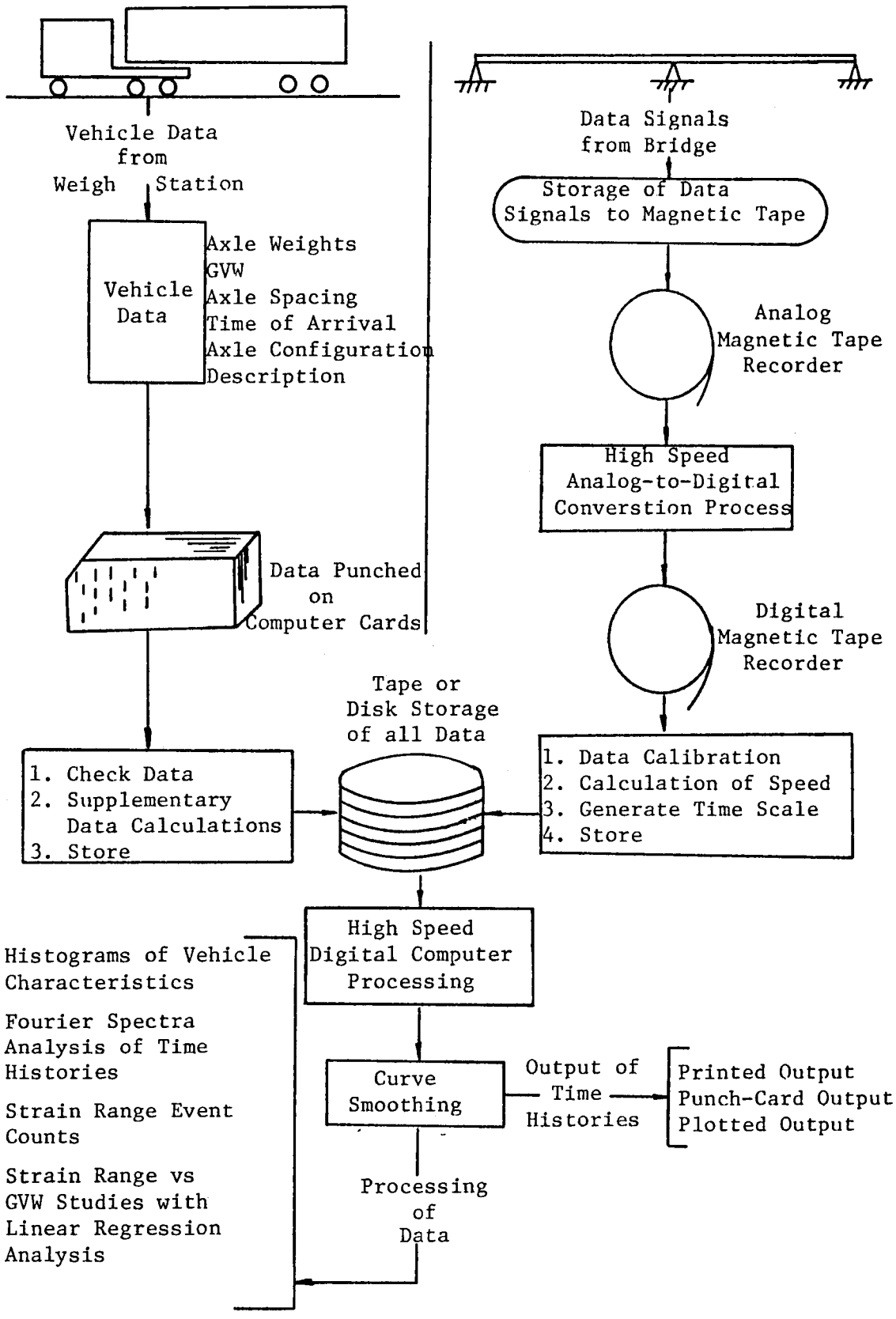


Figure 2. Flow Diagram for Data Acquisition and Reduction.

$$\varepsilon = K_1 \, (GVW) \, \theta_x \, \theta_y \, (1.0 + C_v + K_2 \, \alpha_v)$$

where the variables were defined as:

1) K_1 is a factor relating maximum strain to the gross load;
2) θ_x is a static influence function for a certain longitudinal location on the bridge;
3) θ_y is a static influence function for a certain transverse location on the bridge;
4) GVW is the gross vehicle weight;
5) C_v is a random variable accounting for the initial motion of the vehicle and roadway roughness;
6) K_2 relates the speed parameter to the level of impact induced in the bridge;
7) α_v is the speed parameter appearing explicitly in the analytical formulation of the problem; and
8) ε is the strain induced into the main load carrying member.

All of the variables involving truck traffic characteristics have data presently available in order to determine their values. However, other variables within the derived equation must be based on engineering judgment and analytical experience. For example, K_2 and C_v must be evaluated on these basis, with the help of statistical analyses of their means and coefficients of variation.

This simplified mathematical model can be further reduced by using one variable for all of the parameters. The expression which was linearized using a one-term Taylor expansion to incorporate the variance and mean values is:

$$S_{\varepsilon}^2 = \Sigma \left\{ \left(\frac{\partial q}{\partial x_i} \right)^2 \Big|_{x_i = \bar{x}_i} \times S_{x_i}^2 \right\}$$

where S_{ε}^2 is the variance for strain induced into the main load carrying girder. The partial derivative is evaluated at a value of $x_i = \bar{x}_i$ where $\bar{x}_i$ are the respective mean values of the variables x_i; $S_{x_i}^2$ represents the variance of the variables x_i. The summation sign represents the sum of all the variables indicated in the first equation being subjected to the partial derivative of the equation above. The significance of this approximation of variance is that even with the imperfect data, one can still make rational estimates of the variables and obtain an acceptable answer.

The main objective of the research was to discuss the problems associated with the prediction of girder stresses and discuss ways in which research results could be interpreted in order to make effective use of the collected data. As far as producing a simplified method for determining the stresses was concerned, the research done within Illinois was not directly related. In essence, the program discussed a possible solution to follow in order to reach the main goal of the stress history researchers.

Ohio

The research performed within the state of Ohio was conducted by the Case Western Reserve University Department of Solid Mechanics, Structures and Mechanical Design.[4] The major impetus within the study was primarily directed towards development and application of a sophisticated bridge measurement system. It appeared only logical to develop a refined method for collecting pertinent field data since actual field testing was considered the key to generating a method for outlining a design criteria for fatigue limit analysis. Once a method for collecting and reducing data obtained from field tests was developed and applied, the next procedure was to determine a refined analysis method for fatigue using the collected data.

Ten bridges in Ohio were chosen and instrumented in order to represent a wide range of bridge designs, locations, and truck volumes. Heavy volumes of truck traffic were recorded (up to 250 trucks per hour) over periods of 12 to 24 hours each. For each truck, the measurement system developed recorded truck type, headway (spacing between successive trucks), lane, and velocity. The data was acquired with the use of field personnel using botton boxes to control an analog tape recorder which recorded pertinent truck traffic data.

The final process developed by the Case Western research program was one which facilitated button boxes, analog tape

recorders, multiplexers, an analog-digital converter, and a computer for reducing the collected data into relative stress and truck traffic histograms. The flow charts depicting a simple schematic diagram of the data collection process used is shown in Figures 3 and 4. A brief summary of the process indicated that the strain gages were activated by a truck crossing the bridge. The signals from the gages were amplified and recorded onto magnetic tape. Simultaneously, field personnel punched an appropriate button relating the truck type, headway, and lane position of the truck. By the use of photocells (a light beam triggered by the vehicle breaking the path of the beam), the data was recorded on the tape corresponding to the stresses induced by the particular truck. The tapes of recorded data were then reduced with the use of an analog digital converter, and a computer in order to print out the relative information.

Once the bridge measurement system was developed and put into use, the next step was to correlate the collected data in order to determine a fatigue-based solution for the design of bridge girders. An equation for determining the maximum stress range was developed and presented at the Specialty Conference on Metal Bridges conducted by the American Society of Civil Engineers in November 1974. A brief description of the equation, and a synopsis of the report was discussed by Mr. Cooper Patrick in a thesis presented for an M.S. degree at the University of Tennessee[1] and will not be further discussed within this study.

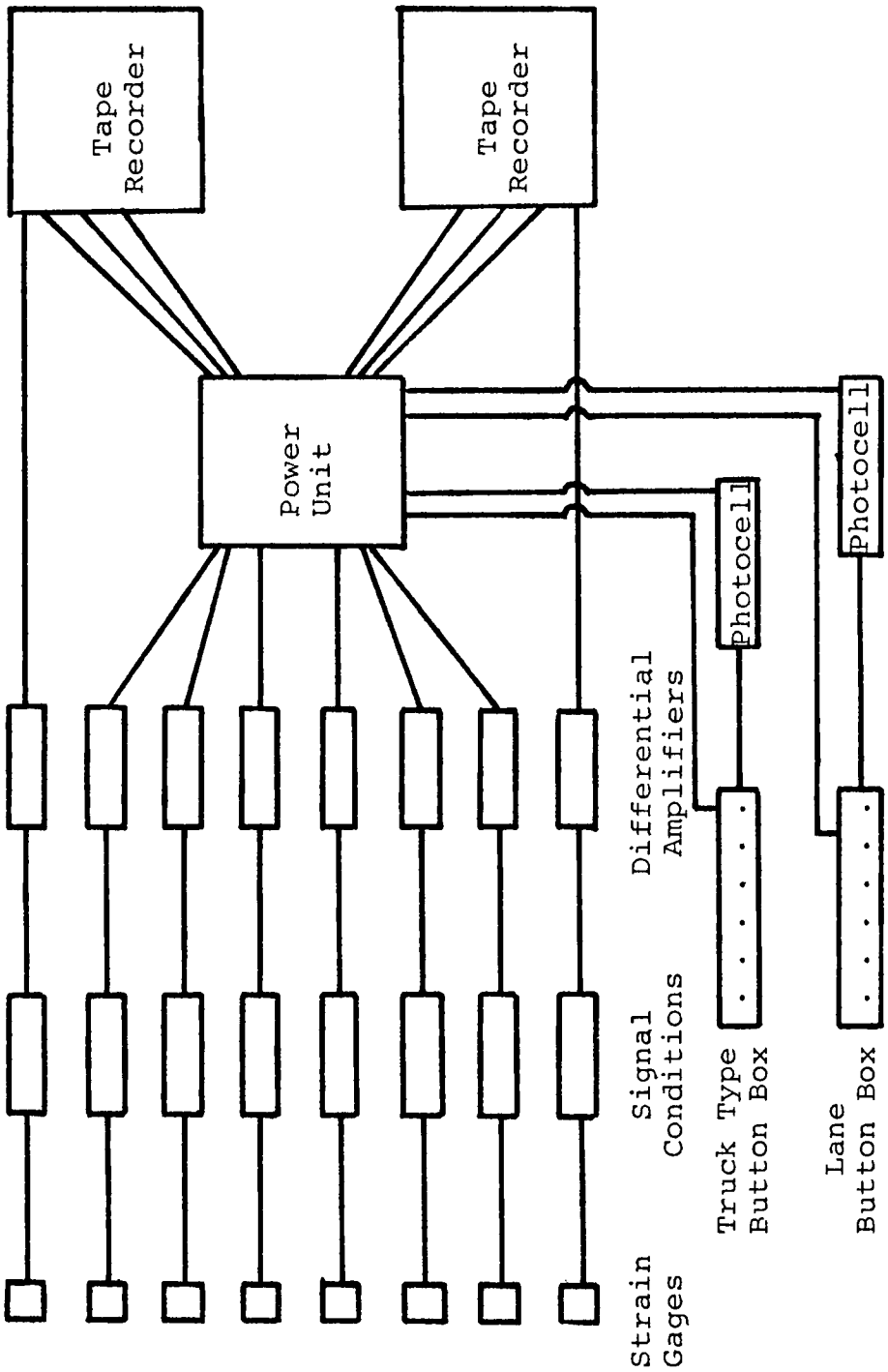


Figure 3. Field Equipment Schematic Diagram.

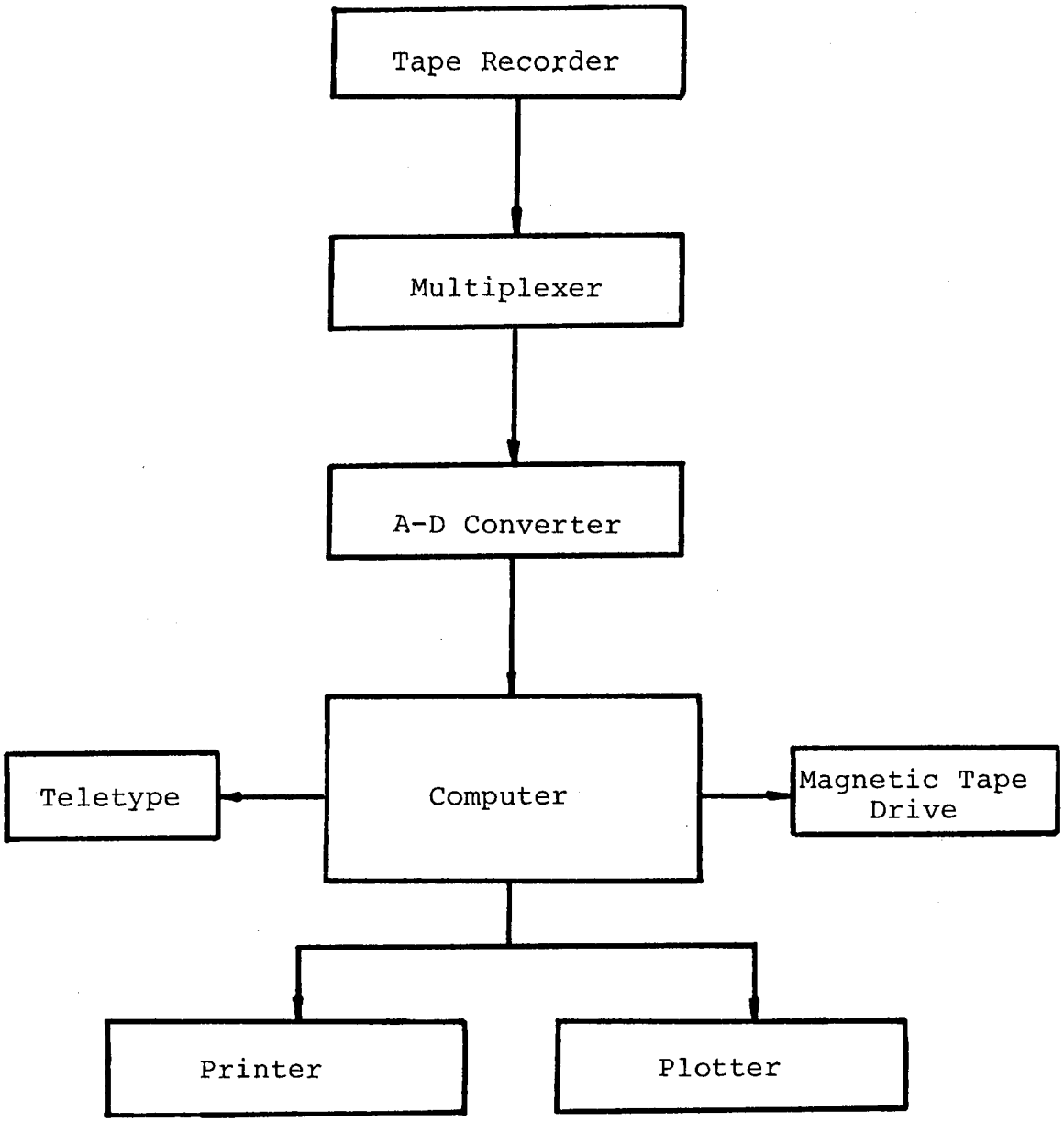


Figure 4. Processing Equipment Schematic Diagram.

In conclusion, the method for the acquisition of data developed by the Case Western Research Program proved to be exceptional. The derived fatigue-based solution using the collected data showed that measured stresses due to present truck traffic were considerably lower than the design values. The results pointed to the need to separate fatigue design loadings from standard design loadings for bridge girders.

Tennessee

The research conducted in Tennessee was performed by the Department of Civil Engineering at the University of Tennessee under the sponsorship of the Tennessee Department of Transportation, Bureau of Highways in cooperation with the FHWA. The research was directed in two paths which were, (1) predicting the fatigue life of bridge girders using actual field measurements of several tested bridges, and (2) relating field data with analytical data in an effort to establish an adequate method for bridge designers to use with respect to fatigue limit. The first path was discussed in detail within reports emanating from studies directed by the University of Tennessee.[1,2,5,6,7] The main body of the research done within this study was directed towards the second avenue previously mentioned.

The original procedure outlined in earlier reports for determining a means for obtaining a simplified method to predict the stresses on highway bridge girders was rewritten into the following:

1. Obtain truck traffic characteristics.
2. Compute the calculated stress on the bridge based on information from step 1. Use Monte Carlo technique to predict stress histogram for a large number of trucks.
3. Obtain stress histogram by measuring actual stresses on the bridge.
4. Correlate the measured values (step 3) with the calculated values (step 2).
5. Develop a method to estimate future traffic characteristics.
6. Determine the volume of each truck type on the bridge from information in step 5.
7. Develop a general analysis to calculate stresses expected due to loads defined by steps 5 and 6.
8. Apply necessary factors of safety.

Earlier research within the program at Tennessee has generated enough data to complete steps 1 and 3 and partially complete steps 2 and 4. The program is presently engaged in completing the remaining steps and, in essence, is the main emphasis of this study.

CHAPTER III

FIELD TESTING PROCEDURES

1. Brief Description of Bridges

In accordance with the eight steps previously outlined in Chapter II, two U. S. interstate bridges were chosen in Knoxville, Tennessee for field testing. The bridges are a part of the I-40 highway system and are located in an urban area of Knoxville. One bridge accommodates the traffic traveling westbound and the other the eastbound. Both bridges are three-span, simple-supported, seven steel girder construction with concrete decks.

The research considered in this study was focused primarily on the westbound bridge. The eastbound bridge was used as a check for procedures developed based on westbound bridge calculations. The similarity of both bridges made it possible to easily compare results of one bridge to the other. The major difference between the two was the skew in the eastbound bridge which was approximated in the theoretical analysis included in this study. The average daily traffic (ADT) and breakdown of truck types correlated closely also. Therefore, with the westbound bridge being easier to work with, the decision was made to use it for the basis of the majority of the research conducted, and to use the eastbound bridge as a check.

2. Instrumentation, Collection and Reduction

Previous reports discussing in detail the methods used for instrumenting the bridges, along with data collection and reduction procedures, have already been written.[1] Therefore, only a brief description of these methods will be discussed to familiarize the reader with them. The main idea is to give the reader a knowledge of the source used for the basis of the methods developed in this study.

Each bridge was tested in the same manner outlined in the diagram shown in Figure 5. In short, each bridge girder was instrumented with a strain gage at its centerline which was activated by a switch box. A switch was flipped by an observer starting a magnetic tape which recorded the voltage changes resulting from a change of resistance of the strain gages. Simultaneously, the observer would manually record the truck type and lane position. At a later time, the magnetic tapes were read, and the digitized voltage changes were input to a computer program. The program changed the voltage variations into corresponding stress ranges which were printed on a teletype. Hence, for each truck passage the maximum stress range, truck type, and lane position was recorded. From this data, histograms of the stresses were established. This procedure produced the completion of the third step of the eight step outline in Chapter II.

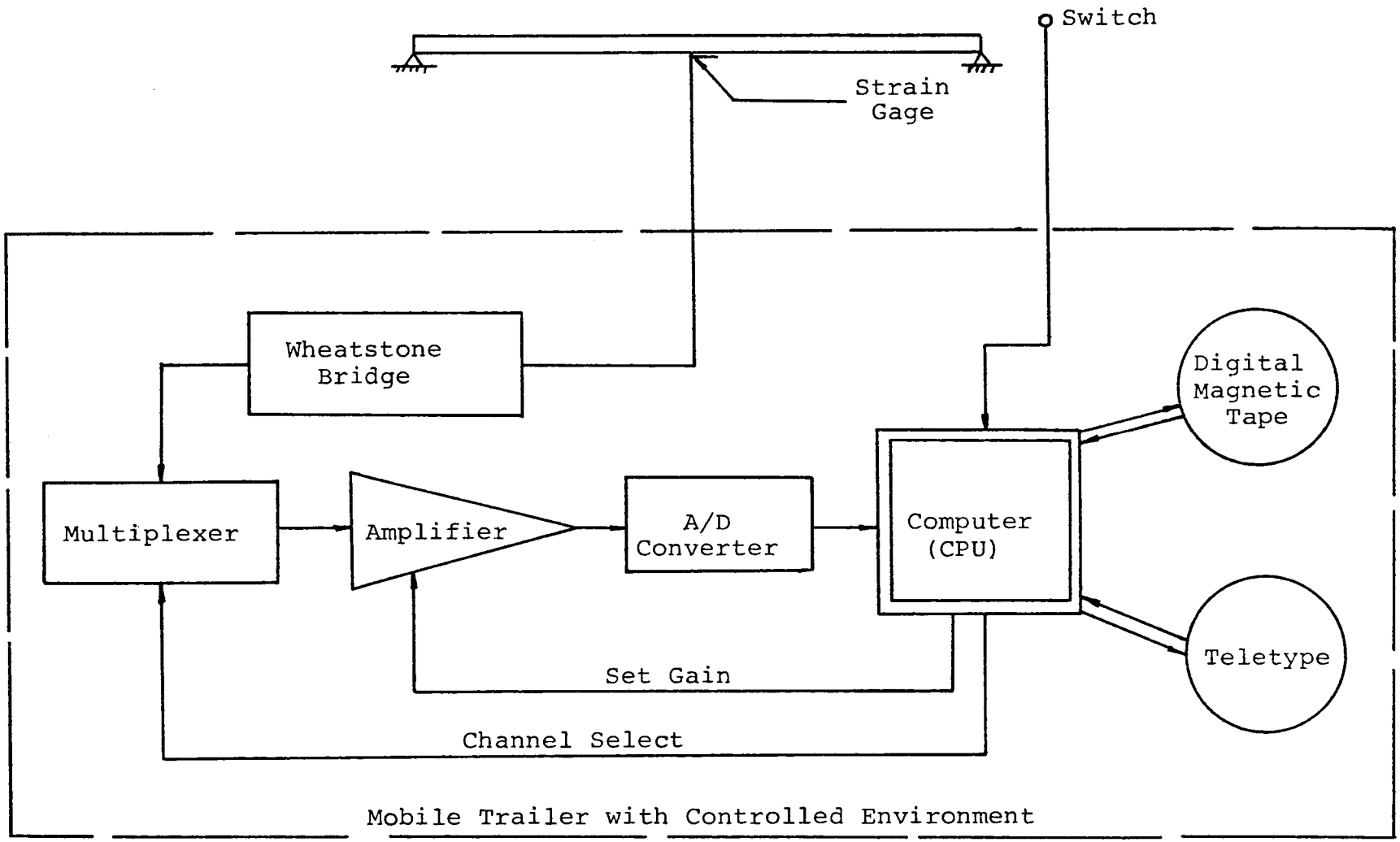


Figure 5. Schematic of Instrumentation and Processing

A weigh station located approximately ten miles from the tested bridges was used in order to collect pertinent weight data. Since there were several exits available for the trucks to take between the weigh station and the tested bridges, it was obvious that a direct comparison of the weight histograms with the stress histograms earlier recorded was meaningless. The primary objective of weight data collection was to be able to approximate the individual truck types by use of the means and standard deviations for the truck type variables (axle spacing, gross weight, percent weight distribution for middle to rear axles). In this manner, a detailed simulation of the truck traffic was made. A chart of the actual truck traffic regarding types and lane positions was earlier recorded as shown in Table I. Since the field testing was completed, the research was directed towards finding solutions for the remainder of the outlined procedure of Chapter II.

TABLE I

NUMBER OF TRUCKS SAMPLED BY TYPE AND LANE

Bridge	Truck Type	Lane Position		
		Left	Middle	Right
Westbound	3S2	689	660	429
	2D	359	507	447
	2S2	41	49	47
	2S1	204	226	134
	3	149	201	194
Eastbound	3S2	491	401	166
	2D	192	350	200
	2S2	17	21	16
	2S1	94	120	39
	3	75	194	55

CHAPTER IV

ANALYSIS OF FIELD DATA

1. Truck Traffic Characteristics

With the data collected in the field, in conjunction with the truck traffic information obtained at the nearby weigh station, a general description of the truck traffic characteristics was developed. The ADT for each bridge was 46,128 vehicles. Approximately 11 percent of the ADT were trucks. The chart shown in Table I gives a detailed analysis of the main truck types traveling over the tested bridges. The information gathered at the weigh station was recorded in order to find a good basis of describing each truck type. For each truck the type, axle spacing, and axle weight was recorded. Hence, each truck was described according to these variables. In Table II the conclusion of the analysis of the data collected at the weigh station is shown.

In order to establish the validity of the data analysis, the results of the analysis were compared to a study which had gathered the same information for tests done in several other states.[8] Tables III and IV show the results of that study, and since they compare well with the analysis performed in this study, it provided sufficient basis for establishing the validity of Table II. Also included in the research of

TABLE II

CHARACTERISTICS OF EACH TRUCK TYPE

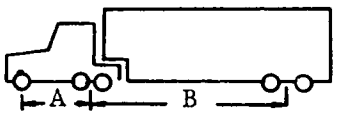
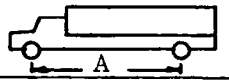
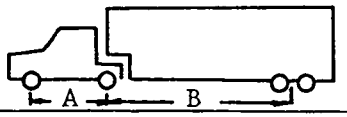
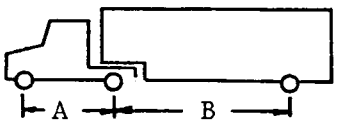
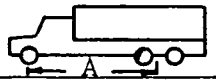
Truck Type	Configuration	Axle Spacing and Standard Deviation (feet)	Gross Weight and Standard Deviation (kips)	Percent Weight to Middle Axle and Standard Deviation
3S2		A = 12.75 ± 1.0 (constant) B = 30.75 ± 3.3	50.1 ± 16.9 Fix front axle weight = 8.75	52.6 ± 6.6
2D		A = 15.0 ± 2.25	14.5 ± 5.9	37.0 ± 6.5
2S2		A = 11.10 ± 0.9 (constant) B = 28.5 ± 0.9	35.2 ± 10.6 Fix front axle weight = 8.5	48.6 ± 6.4
2S1		A = 11.0 ± 1.0 (constant) B = 29.5 ± 3.5	29.7 ± 6.7 Fix front axle weight = 8.0	52.6 ± 7.5
3		A = 17.25 ± 2.4	26.8 ± 9.5	38.0 ± 6.0

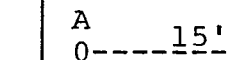
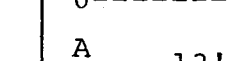
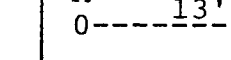
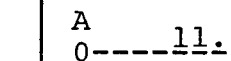
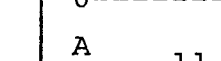
TABLE III

VEHICLE GROSS WEIGHTS BY TRUCK TYPE

Ref. / T.T.	Va.		Mich.		Md. 1		Md. 2		Conn.		Average		Range
	Mean	S	Mean	S	Mean	S	Mean	S	Mean	S	Mean	S	
2D	13.1	5.51	15.0	5.0	14.7	6.32	13.0	-	19.7	-	14.3	5.6	$3 \leq GW \leq 25.5$
3	22.4	9.91	-	-	32.4	12.89	48.0	-	38.4	-	35.3	11.4	$12 \leq GW \leq 58.1$
2S1	29.7	15.93	36.6	9.7	31.4	9.67	29.8	-	54.7	-	36.1	11.8	$12 \leq GW \leq 59.7$
2S2	38.5	9.86	37.0	13.0	43.2	15.19	38.0	-	45.8	-	40.5	12.7	$15 \leq GW \leq 65.9$
3S2	54.9	13.96	48.7	13.7	56.6	19.79	53.0	-	48.2	-	52.3	17.8	$21 \leq GW \leq 83.9$

TABLE IV

VEHICLE AXLE SPACING BY TYPE

Ref. / T.T.	A-B Va.	A-B Md.	A-B Conn.	B-C Va.	B-C Md.	B-C Conn.	Average A-B	Average B-C	
2D	14'	16'	15.7'	-	-	-	15.2	-	
3	14'	18'	19.1'	-	-	-	17.0	-	
2S1	11.0	12'	11.8'	29.0	28.0	23.9	11.6	27	
2S2	11.0	12'	12.1'	27.0	28.0	28.7	11.7	27.9	
3S2	12.0	12'	11.1'	30.0	30.0	33.3	11.7	30.1	

the aforementioned study was a chart, shown in Table V, which depicts the breakdown of truck traffic according to truck and bridge location.

By defining the volume of trucks according to type, and lane position, along with characteristics regarding the average truck type, the truck traffic characteristics were adequately described. Tables II and V present a method for determining a close simulation of the truck traffic on a highway bridge.

2. Actual Stress Distributions

The emphasis of the research conducted within this study was directed towards comparing actual stresses caused by a single truck, with its known characteristics (i.e., axle weights, axle spacings, percent distribution of axle weight from middle to rear, type, speed, and lane position), to the theoretical stresses caused by the simulation of the same truck. Hence, the field tests involved the collection of data corresponding to the passage of a 3S2 type truck, which was previously recorded with respect to its characteristics. In this manner, it was possible to obtain a one truck-one stress relationship. This, in essence, is the basis by which all stress correlations should be made in order to validate a direct comparison of theoretical to actual analyses.

Utilizing the collection and reduction process described earlier, a control truck test vehicle was driven over the

TABLE V

PERCENT DISTRIBUTION OF TRUCK TYPES

Location / Truck Type	Metropolitan	Urban	Rural
3S2	25.0	44.0	41.0
2D	35.0	13.0	21.0
2S2	11.0	30.0	25.0
2S1	6.0	10.0	7.0
3	23.0	3.0	6.0

bridge several times varying the speed and lane position. The truck variables remained constant except for the vibrations of the truck. Several tests were performed at speeds of 55, 30 and 5 miles per hour with various lane positions. The results of the high speed (55 mph and 30 mph) tests are shown in Tables VI, VII, and VIII. Inherent with the high speed tests is the fact that the impact, truck vibrations, and traffic varied for each test run. The general indication from the comparisons above for a truck in each lane shows that lane position is not a significant variable with respect to the sum of the stresses. A definite conclusion could not be reached from the test results at high speeds.

In an effort to eliminate as many of the conflicting variables (impact, vibrations, traffic, etc.) as possible, six test runs were conducted at crawl speeds of 7 miles per hour or less. To reduce the danger of accidents on the interstate bridge, the tests were run at 1:00 a.m. when traffic was light enough to be completely halted during the tests. Only one test was recorded with any traffic on the bridge except the test truck. Two cars passed over the bridge during the first test run, but there was no evidence indicating any effect on the outcome. Because of the extreme weight difference between the control truck and the few cars present on the bridge, the cars had little effect on the girder stresses. Table IX gives the results of the crawl tests in each lane. With the exception of the fifth

TABLE VI

MEASURED STRESSES FOR CONTROL LOAD

IN LEFT LANE (KSI)

Trial	I	I	I	I	I	I	I	Σ	Speed (mph)
1	1.594	2.477	1.930	0.842	0.279	0.156	0.183	7.461	60
2	1.628	2.615	2.014	0.760	0.314	0.176	0.206	7.713	52
3	1.261	1.973	1.930	0.994	0.337	0.166	0.206	6.867	45
4	1.158	2.156	1.878	0.936	0.337	0.185	0.137	6.787	32
5	0.665	1.812	2.255	1.415	0.465	0.195	0.229	7.036	60

TABLE VII

MEASURED STRESSES FOR CONTROL LOAD

IN RIGHT LANE (KSI)

Trial	I	I	I	I	I	I	I	Σ	Speed (mph)
1	0.206	0.263	0.335	0.912	1.606	2.075	1.261	6.658	60
2	0.217	0.275	0.293	0.877	1.513	2.368	1.628	7.171	53
3	0.263	0.172	0.430	0.947	1.793	2.300	1.376	7.281	50
4	0.183	0.137	0.304	0.807	1.595	2.584	1.778	7.388	43
5	0.137	0.172	0.430	1.029	1.525	2.026	1.468	6.787	43
6	0.126	0.091	0.262	0.713	1.536	2.437	1.342	6.507	32
7	0.114	0.126	0.356	0.784	1.490	2.476	1.399	6.745	20
8	0.114	0.114	0.314	0.725	1.455	2.437	1.491	6.650	27
9	0.137	0.080	0.314	0.795	1.443	2.310	1.376	6.455	27

TABLE VIII

MEASURED STRESSES FOR CONTROL LOAD

IN MIDDLE LANE (KSI)

Trial	I	I	I	I	I	I	I	Σ	Speed (mph)
1	0.195	0.711	1.542	2.387	1.269	0.763	0.146	7.016	56
2	0.424	0.642	1.689	2.714	1.315	0.548	0.401	7.733	56
3	0.298	0.630	1.468	2.480	1.397	0.724	0.286	7.283	50
4	0.424	0.711	1.668	2.153	1.187	0.577	0.321	7.041	43
5	0.217	0.722	1.416	2.586	1.595	0.792	0.160	7.488	32
6	0.195	0.722	1.437	2.574	1.338	0.655	0.091	7.012	27

TABLE IX

MEASURED STRESSES FOR CONTROL LOAD

AT CRAWL SPEEDS (KSI)

Lane Position		I	I	I	I	I	I	I	Σ
Right	1)	0.103	0.091	0.251	0.666	1.408	2.398	1.560	6.477
	2)	0.114	0.240	0.283	0.678	1.257	2.339	1.537	6.448
Middle	3)	0.103	0.493	1.290	2.293	1.397	0.636	0.160	6.372
	4)	0.103	0.481	1.248	2.281	1.467	0.694	0.160	6.434
Left	5)	1.158	2.145	2.035	0.982	0.314	0.117	0.103	6.854
	6)	1.032	2.030	1.972	0.947	0.302	0.088	0.103	6.474

run, the total stresses clearly indicate that the lane position is irrelevant when summing the stresses across the transverse section of the bridge for a given truck. Even the fifth run does not vary enough (7 percent maximum) to justify regarding the conclusion as false.

The theoretical analysis also indicated that the total stress on the bridge cross section was constant regardless of lane position. A simulated truck loading was placed at several positions transversely across the bridge, and the results were essentially identical. There was only a 7 percent maximum deviation. Hence, the variability of lane position was eliminated, and emphasis was placed on determining the stress induced on the total bridge cross section instead of the individual girders.

In retrospect to the outline in Chapter II, steps 1, 3, and 5 have now been successfully completed. The steps concerning theoretical analysis, comparison of theoretical-to actual stresses, and development of a simplified method for determining stresses on a bridge cross section will be discussed in detail within the following chapters.

CHAPTER V

THEORETICAL ANALYSIS

1. Bridge Model Simulation

The tested bridges were theoretically simulated using plate bending finite elements and beam elements of the ICES STRUDL II program. Earlier defined procedures[1,9] for the westbound bridge were followed to develop a similar model for the eastbound bridge. The westbound model was developed using rectangular elements, whereas the eastbound model used triangular elements in order to account for the skew. Cross sectional properties necessary for the program model description were calculated according to the construction prints (i.e., moments of inertia and cross sectional thicknesses). The finite elements used were the plate bending type so the stiffness analysis used would produce moments correlating to the load input on the bridge model.

The westbound bridge is 62 feet long and 47.5 feet wide. A partial view of the model is shown in Figure 6. Since the eastbound bridge is tapered, the average width of the model was taken as the width at the centerline and is partially shown in Figure 7. The length of the eastbound bridge is 63 feet, and the width is approximately 54 feet. These models were developed so that they could be mathematically loaded

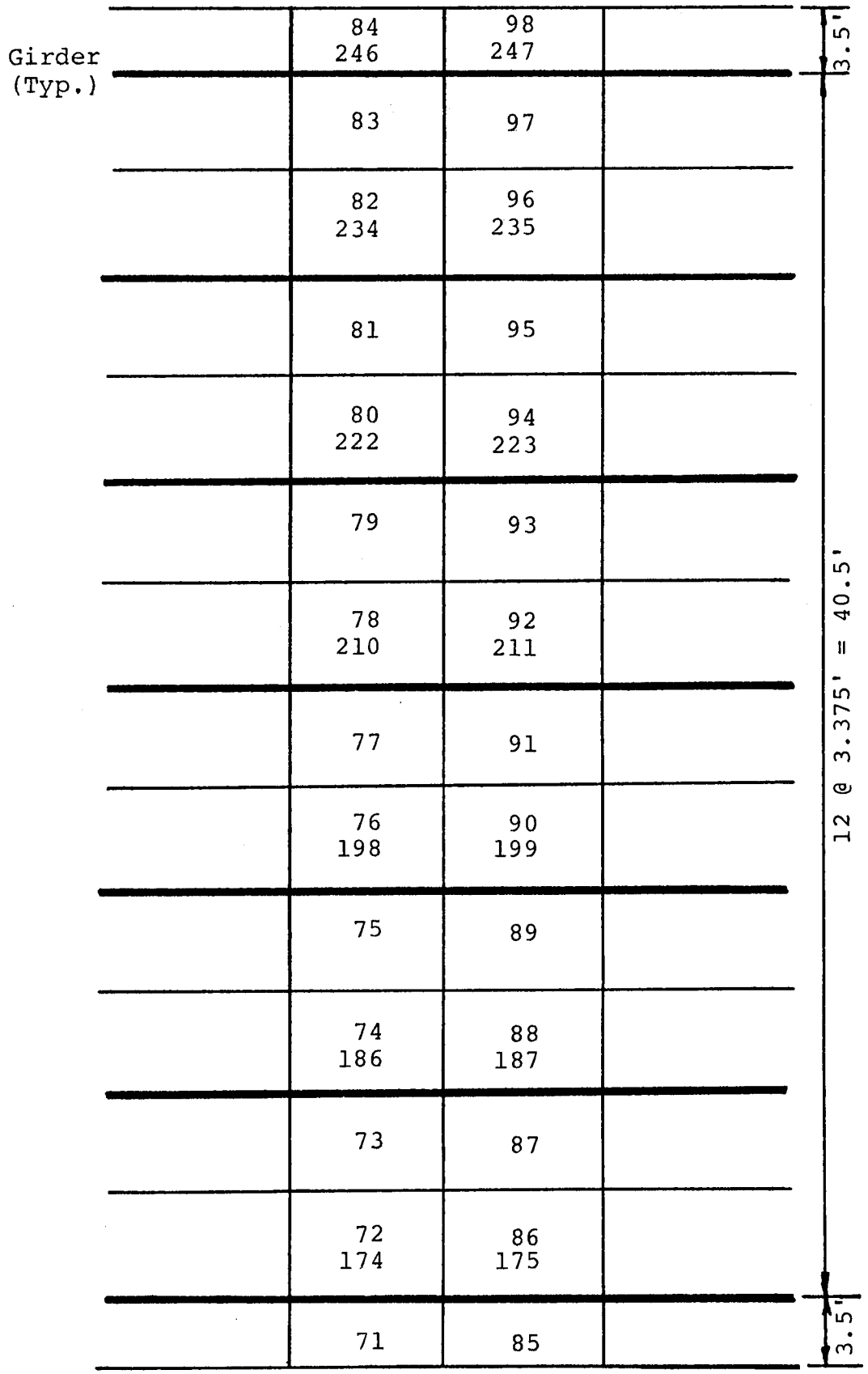


Figure 6. Finite Element Section of Westbound at Centerspan.

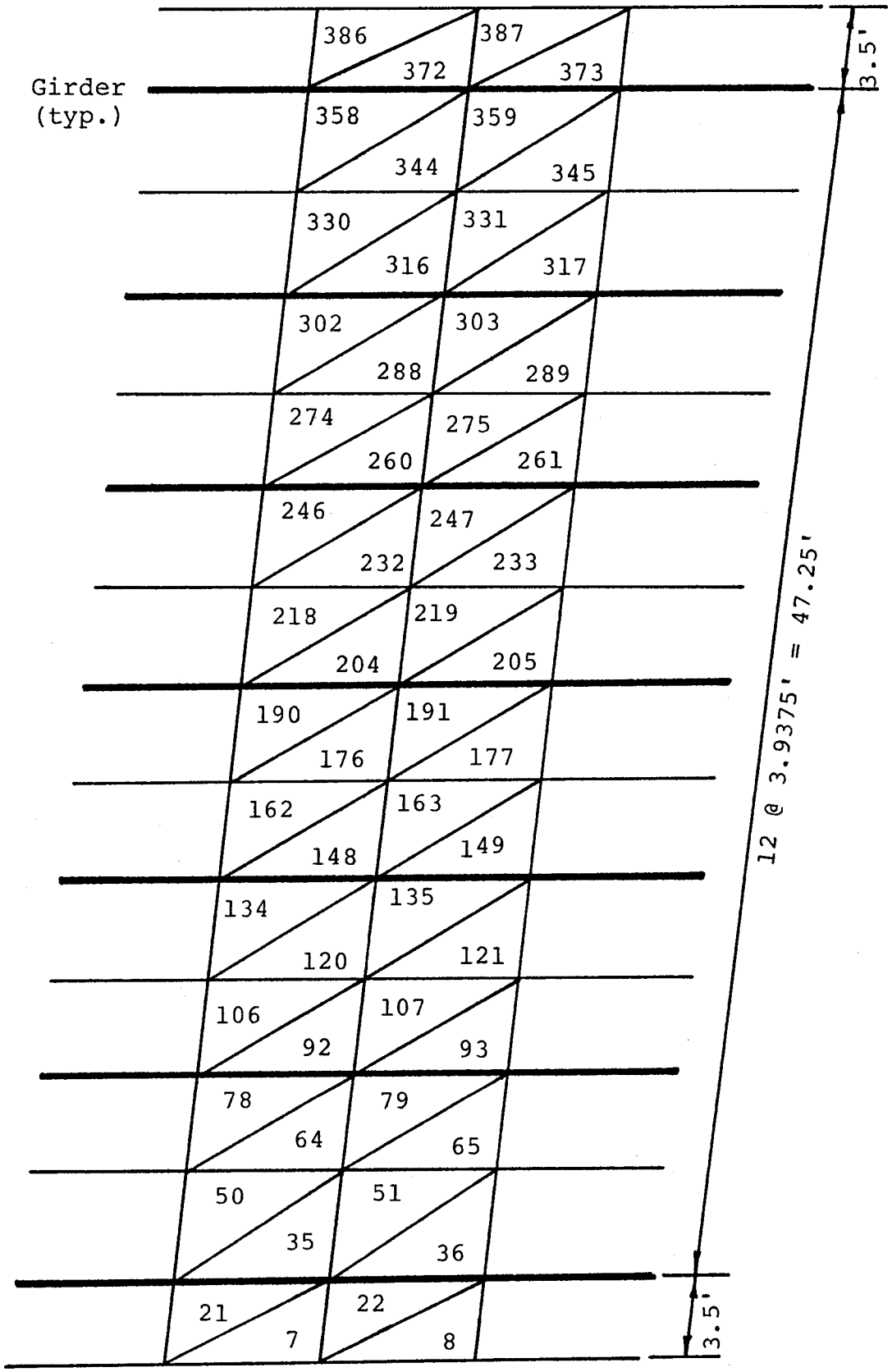


Figure 7. Finite Element Section of Eastbound at Centerspan.

with the control load truck vehicle used in actual field measurements. Figure 8 shows the axle spacings and loads of the control truck vehicle. The computer output consisted of static slab and girder moments. By superimposing the slab and girder moments, the stress for each girder, and hence the total stress on the entire bridge cross section was calculated.

2. Refinement of Theoretical Analysis

The original theoretical analysis[1] simulated the control test truck vehicle by placing concentrated loads directly under the centerline of each axle. In the case of the 3S2 test truck used, there were six concentrated loads. This loading was placed on the bridge model so the middle concentrated load was on the centerline of the bridge span (this produced the maximum moments, hence, the maximum stresses at the centerline of the span). The concentrated loads were distributed to the element nodes. The joint loads were placed in the computer program and resulting stresses were calculated. The original analysis results were too high when compared to the actual stresses. It was concluded that appropriate changes must be made in order to have a more accurate theoretical analysis.

The refinements made on the original procedure were (1) spreading the loads due to the test vehicle to each individual axle as shown in Figure 9, and (2) introducing the handrailing

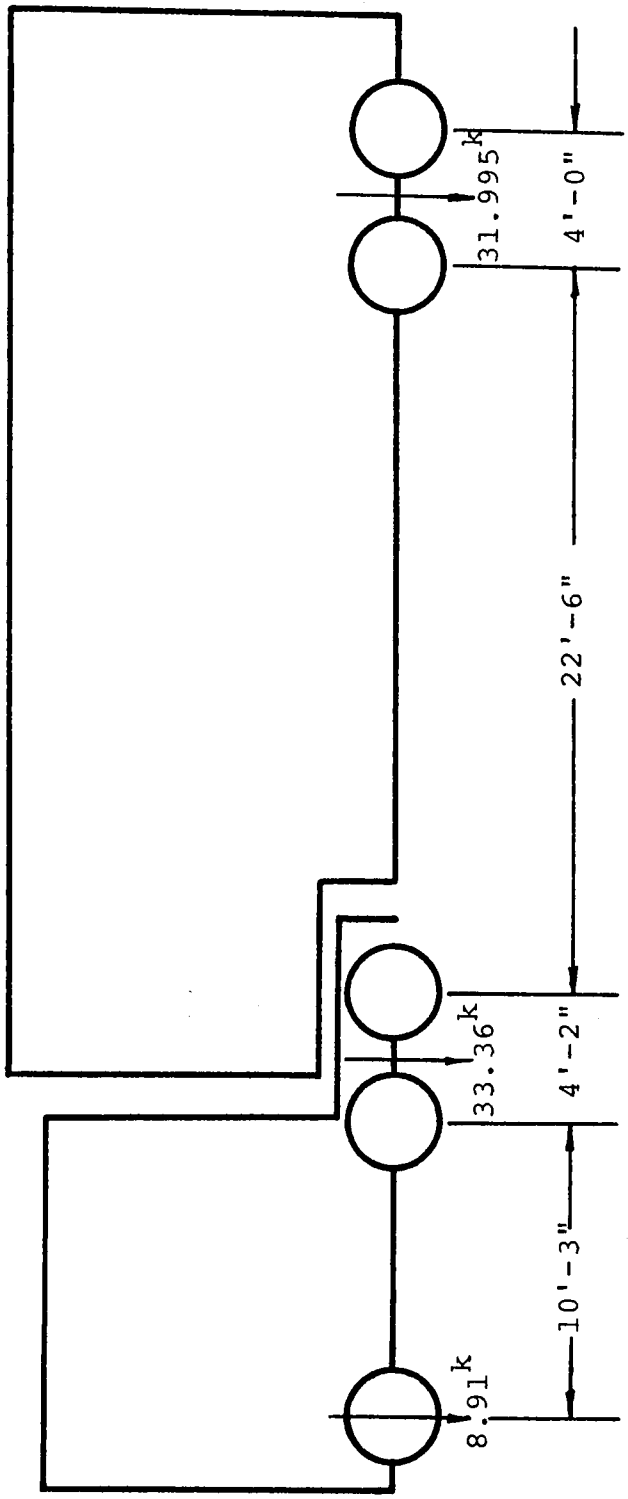


Figure 8. Control Test Vehicle Axle Weights and Spacings.

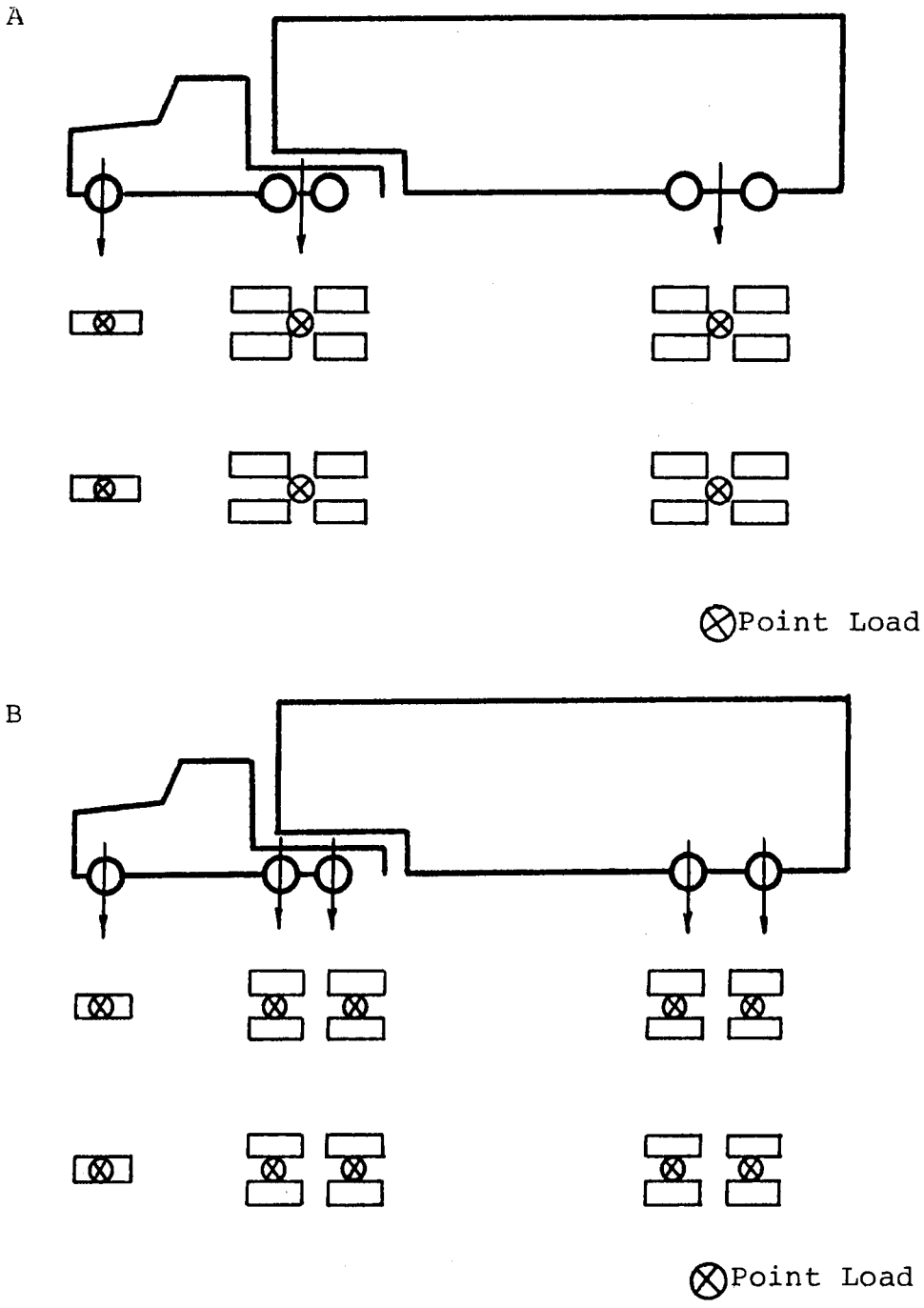


Figure 9. Control Truck Axle Loading Variation
(A) Original Loading; (B) Revised Loading.

into the previously defined bridge model. These refinements would hopefully bring the theoretical stresses down to a more acceptable comparison. The results of the refinements introduced were compared to the average actual stresses and the theoretical stresses according to lane position in Tables X, XI and XII.

In conclusion, the newly developed theoretical analysis provided a good means by which actual stresses could be calculated for a truck in the outside lanes. The middle lane comparison was not as accurate as that of the outside lanes, but was still acceptable. The reason for this was that an average value for the distance from the neutral axis to the bottom of the girders was used for the total cross section in calculating the theoretical stresses. The general consensus of the research personnel and the author was that the actual value for this distance varies in such a manner as to give higher stresses on the outside lanes and lower stresses in the middle lane as compared to using the theoretical value. A procedure was developed that could be used to calculate the sum of the girder stresses on the cross section of the tested bridge without the use of a computer analysis.

TABLE X

LEFT LANE STRESS COMPARISONS (KSI)

Analysis	I	I	I	I	I	I	I	Σ
Measured	1.261	2.207	2.001	0.989	0.346	0.176	0.192	7.173
Original Loading	1.529	2.750	2.511	1.120	0.343	-0.008	-0.111	8.134
Revised Loading	1.587	2.586	2.407	1.170	0.360	-0.008	-0.116	7.977
Revised Loading with Handrail	1.377	2.483	2.406	1.182	0.370	0.003	-0.097	7.724

TABLE XI

RIGHT LANE STRESS COMPARISONS (KSI)

Analysis	I	I	I	I	I	I	I	Σ
Measured	0.166	0.159	0.338	0.843	1.551	2.335	1.458	6.849
Original Loading	-0.101	-0.034	0.244	0.910	2.247	2.847	1.906	8.019
Revised Loading	-0.106	-0.035	0.255	0.958	2.178	2.681	1.936	7.866
Revised Loading with Handrail	-0.088	-0.023	0.266	0.968	2.168	2.555	1.682	7.528

TABLE XII

MIDDLE LANE STRESS COMPARISONS (KSI)

Analysis	I	I	I	I	I	I	I	Σ
Measured	0.292	0.690	1.537	2.482	1.350	0.677	0.234	7.262
Original Loading	0.001	0.622	1.890	2.998	2.196	0.771	0.064	8.541
Revised Loading	0.001	0.659	1.881	2.822	2.146	0.817	0.067	8.392
Revised Loading with Handrail	0.010	0.655	1.903	2.860	2.171	0.809	0.068	8.475

CHAPTER VI

SIMPLIFIED ANALYSIS

1. Initial Assumptions

The initial assumptions made to establish the analytical method developed were that (1) the bridge has known dimensions and has been designed under present design criteria methods, and (2) the bridge was adequately designed for strength. With these assumptions, the basic idea behind the simplified analysis developed was to estimate the life span of the bridge using fatigue criteria. From the results of the above comparisons, the designer would establish the controlling criteria and recognize the true life span. This avenue of analysis was taken because all stress history research programs throughout the nation had established the fact that a large percentage of existing highway bridges had been more than adequately designed for fatigue due to present day truck traffic. Hence, to simply continue designing highway bridges under established design procedures and check the fatigue limits for adequacy was considered the best method.

2. Estimating Truck Traffic

The first step in determining the fatigue analysis was to estimate the number of cycles of loading that will occur

in each lane. To fulfill this first step, a general method was developed to estimate the number of times the major types of trucks crossed a bridge. The designer must first have the percentage of the ADT which constitutes truck traffic for the bridge under consideration. If the bridge had already been built and in use, the designer need only call the State Highway Department of Transportation, and they would most probably have this information. If they did not, or if the bridge was unconstructed, there are developed methods for estimating future traffic volumes. This study will not discuss traffic volume estimating methods, but simply inform the designer that the percentage of the ADT depicting truck traffic can be determined.

Once the average dialy truck traffic (ADTT) is established, it becomes necessary to develop a method for describing a breakdown of the truck types in percent of the ADTT for various locations of the bridge. From the study previously mentioned in Chapter IV, Table V was developed from the superposition of data collected in several states. With the use of this table, along with the ADTT, it becomes possible to describe the total truck traffic volume according to truck type.

In order to estimate the number of cycles of loadings that occurs in each lane, it is necessary to know the approximate percentages of the total volume of truck traffic traveling in each lane. Since fatigue is usually not the major contributing factor regarding the life of highway

bridges, assume that all the truck traffic will travel in one lane. If this ultraconservative approach exceeds the failure limit due to fatigue, approximate percentages of the truck traffic in each lane could be assumed depending on the number of lanes. For example, for a four lane bridge assume 60 percent of the truck traffic will travel in the far right lane, 25 percent in the next lane, 10 percent in the next lane, and 5 percent in the extreme left lane. For a three lane bridge, assume 70 percent of the trucks will travel in the right lane, 20 percent in the middle lane, and 10 percent in the left lane. For a two lane bridge, assume that 80 percent of the trucks will travel in the right lane, and 20 percent in the left lane.

If the bridge has an exit or entrance as one of its lanes, the approximation of the truck traffic in each lane must be determined accordingly. Referring to Table I on page 22, the right lane on the westbound bridge is an exit lane, so it is apparent that more trucks will travel in the other two lanes. Similarly, the right lane on the eastbound bridge is an entrance lane, thus more traffic occurs in the other two lanes. The largest percentage of trucks in any one lane shown in Table I is 60 percent in the middle lane of the eastbound bridge due to type 3 trucks. The middle lane on the eastbound bridge is the far right lane for flowing traffic; hence, assuming a large percentage (70 percent or more)

of trucks in one lane will be a conservative approach for estimating the number of cycles of loadings that occurs in each lane.

3. Formulating the Simplified Stress Equation

Detailed Procedure

The overall analysis developed within this study resulted in a single equation describing the stress induced on the total cross section of a bridge. The equation was formed by following statical procedures and incorporating linearizations of the behavior patterns of stresses induced by truck variables. The linerizations which were used and shown to be true in an earlier study[1] were that stresses induced on a bridge varied linearly with (1) the percent weight distribution of the middle axle using the middle and rear axle weights only, and (2) the gross vehicle weight, and varied inversely with axle spacing.

The procedure began by using a 3S2 truck type configuration. Three variables of the 3S2 truck were fixed at specific values with standard deviations as shown:

(1) Front axle weight = 8.75 kips $\pm$ 1.0 kip

(2) Front axle spacing = 12.75 feet $\pm$ 1.0 foot

(3) Gross vehicle weight = 50.0 kips

The values of the above variables were justified by field data and stress relationships. A total of 1744 3S2 truck samples were taken at the state weigh station approximately

ten miles from the bridge. The front axle weight was based on 1382 axle weight samples. The front axle spacing was determined from 315 spacing samples, and a close comparison with the results shown in Table IV on page 26. In brief, the stress included in a bridge cross section was negligible due to small variations of the front axle weight as shown in Figure 10. The field data proved the front axle weight and spacing of a 3S2 truck varied very little. The gross vehicle weight was fixed at 50 kips, but in actuality could be chosen to be any value from 10 kips to 120 kips.

The simplified loading configuration of a 3S2 truck (concentrated loads placed at the centerline of the wheels as in diagram A of Figure 8 on page 39) was placed on the bridge so as to produce the maximum moment at the center line of the span. Four separate calculations of the maximum moment were done by altering the rear spacing and percent weight distribution of the middle axle as shown in Figure 11. Recall that the percent weight distribution to the middle and rear axles was the weight remaining of the gross vehicle weight without the front axle weight (i.e., 50-8.75 = 41.25 kips).

The next step was to calculate an average section modulus of the bridge. This was accomplished by calculating the total moment of inertia of the entire cross section. The calculated moment of inertia was divided by the distance from the neutral axis to the bottom of the girders of the entire bridge cross section to find the average section modulus.

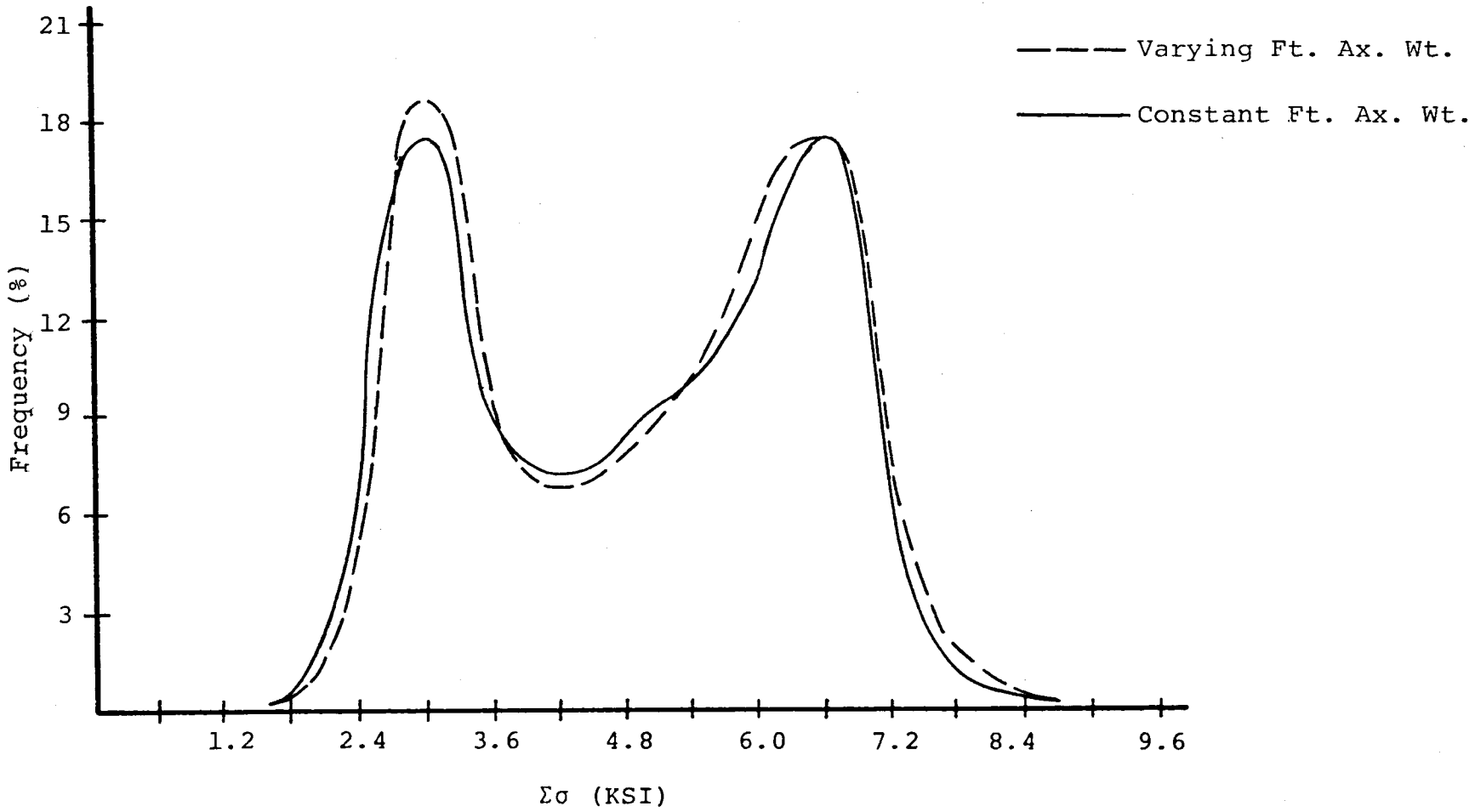


Figure 10. Comparison of Varying Front Axle Weight and Constant Front Axle Weight Summation of Stresses - 3S2 Trucks.

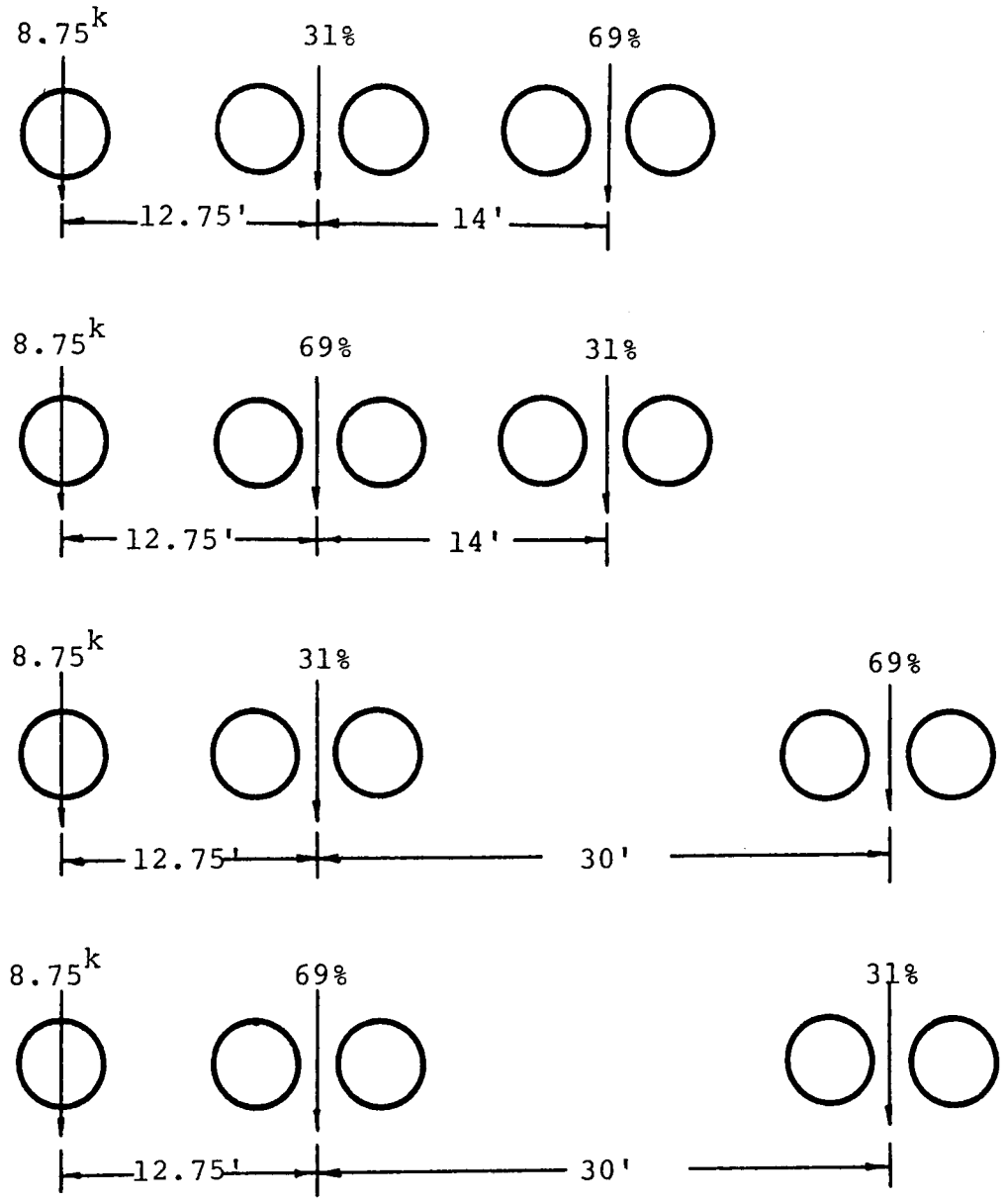


Figure 11. 3S2 Truck Loading Variation.

From the moments and the section modulus previously calculated, the sum of the girder stresses due to the loading was determined by dividing the moments by the section modulus and multiplying by the number of girders. Henceforth, wherever the word "stress" appears, the meaning refers to "the sum of the girder stresses" unless otherwise defined. A graph was plotted using the four stresses calculated versus the rear axle spacing as shown in Figure 12. Points 1 and 3 in Figure 12 represent the stresses induced on the bridge due to 31 percent of the middle-to-rear axle weight being distributed to the middle axle with the rear axle spacings at 14 feet and 30 feet respectively. Points 2 and 4 represent the stresses caused by 69 percent of the middle-to-rear axle weight being distributed to the middle axle with the rear axle spacing at 14 feet and 30 feet respectively.

The graph plotted in Figure 12 was used to determine the point of intersection of the two lines (point A), and the slopes of each line (indicated as B and C). From these values, three constants of the final equation was determined. The first value was simply the number calculated at point A (as a check on the calculations thus far, point A should be the same value whether determined by using the top or bottom line). The second number was the absolute value of the slope of the bottom line (indicated as B on the graph). The third and last constant was the change of rate of the slopes from the bottom line to the top line. This constant was calculated as follows:

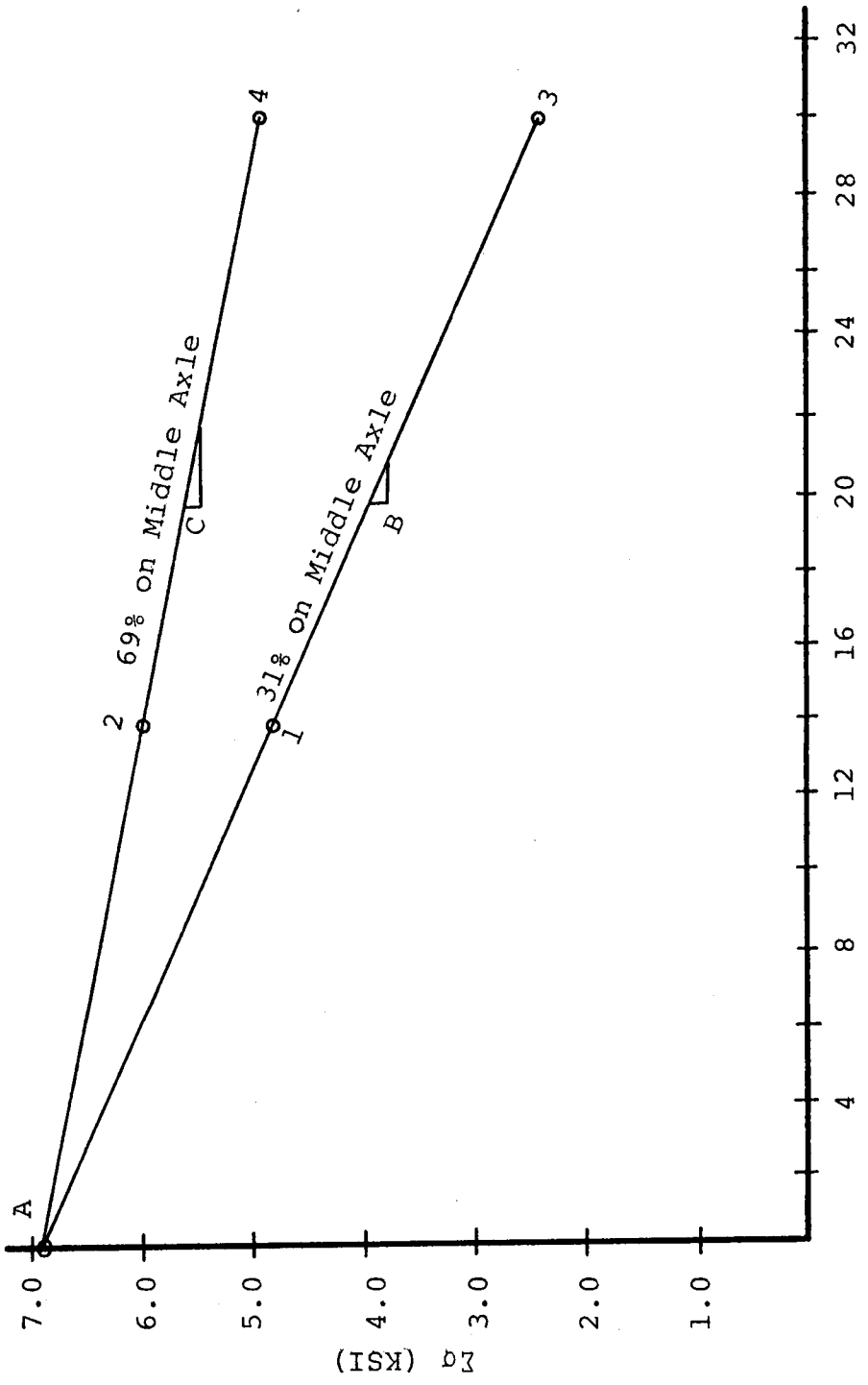


Figure 12. Summation of Stresses Versus Rear Axle Spacing

$$D = \left| \frac{B-C}{38.0} \right|$$

where D = change in rate of slopes;

B = slope of the bottom line; and

C = slope of the top line.

With the three constants previously determined, the simplified equation for the stress induced across the transverse section of the bridge under evaluation was generated as follows:

$$\Sigma\sigma = \left[A - \left(\left[B - (D)\ (X - 31.0) \right] Y \right) \right] \frac{Z}{50}$$

where A, B, and D are the constants previously defined;

X = percent distribution to middle axle of middle-to-rear axle distribution percentages;

Y = rear axle spacing (feet);

Z = gross vehicle weight of truck under inspection (kips); and

$\Sigma\sigma$ = stress induced on entire bridge cross section (ksi).

The simplified equation made it possible to find the stress caused by any truck, with variables X, Y, and Z as defined above known, on most highway bridge configurations. The maximum and minimum stress ranges induced by various truck types could be determined by using Table II on page 24, and placing the variables into the above equation. For 2D and 3 type trucks let X equal the percent distribution to front

axle, Y equal the spacing and Z remain the gross weight. The stress calculated represents the stress range because zero was used as the base.

At this point, the possibility of determining the number of cycles each truck type recorded, and the stress range induced by each truck was available. If only one truck type was predominant, the above procedure could be further simplified by using Table II and calculating the stress range directly by placing the maximum and minimum loading conditions onto the bridge.

Simplified Procedure for Simple Span Bridges

Simple span bridges are statically determinate, thus simplifying the method for determining where to place the truck loadings so as to induce maximum moments. The same placement could also be used on short span continuous bridges with little error. In an effort to make calculations easier, a chart was formed tabulating the maximum and minimum moments induced by each truck type using Table II to determine the maximum and minimum loading conditions.

The procedure began with calculating the loading cycles of each truck type in each lane. This step was accomplished by following the same procedure previously described. Naturally the following step was to determine maximum and minimum moments induced on the simple span bridge under consideration caused by each truck type. Shown in Table XIII are the maximum and minimum moments tabulated according to truck type

TABLE XIII

MAXIMUM AND MINIMUM MOMENTS FOR SIMPLE SPANS

FOR EACH TRUCK TYPE (K-FT)

Truck Type \ Span (ft)	40 Max/Min	60 Max/Min	80 Max/Min	100 Max/Min	120 Max/Min	140 Max/Min
3S2	378.9 / 147.6	623.9 / 246.1	959.4 / 382.3	1249.9 / 547.8	1630.4 / 713.3	1974.9 / 878.8
2D	152.9 / 64.1	253.4 / 106.1	355.2 / 148.3	457.3 / 190.7	559.5 / 233.1	661.9 / 275.5
2S2	245.3 / 109.7	422.1 / 185.0	651.1 / 301.0	880.1 / 424.0	1109.1 / 545.0	1338.1 / 670.0
2S1	208.6 / 107.0	354.7 / 179.7	536.7 / 280.2	718.7 / 395.2	900.7 / 510.2	1082.7 / 625.2
3	282.0 / 106.3	461.8 / 190.2	642.4 / 275.3	823.4 / 361.0	1004.6 / 447.0	1185.8 / 533.1

and span length. Table XIII was further simplified by making a plot for each truck type of moment versus span length, thus enabling the designer to avoid interpolation. By reading values from the graphs of Firgures 13 through 17, the designer could quickly find the maximum and minimum moments induced by each truck type.

Next, the average section modulus of the entire bridge cross section had to be determined. The procedure defined earlier was used once again as the form for calculations. With the maximum and minimum moments and average section modulus known, the calculation of maximum and minimum stress range for each truck type was possible.

The simplified procedure developed within this study enables the designer to determine the stress range on an entire bridge cross section for general I-beam stringer deck bridge configurations. In order to determine the adequacy of the bridge due to fatigue, the stress range must be distributed to find the highest individual girder stress range known as the peak stress range. The number of cycles of loadings can be conservatively approximated for each lane due to each truck type. With the peak stress range and the number of loadings known, the bridge can be checked for fatigue.

The actual and theoretical data were used to determine a method for calculating the peak stress range occurring on the seven girder bridge tested within this study. For each case, the stress on the entire bridge cross section was

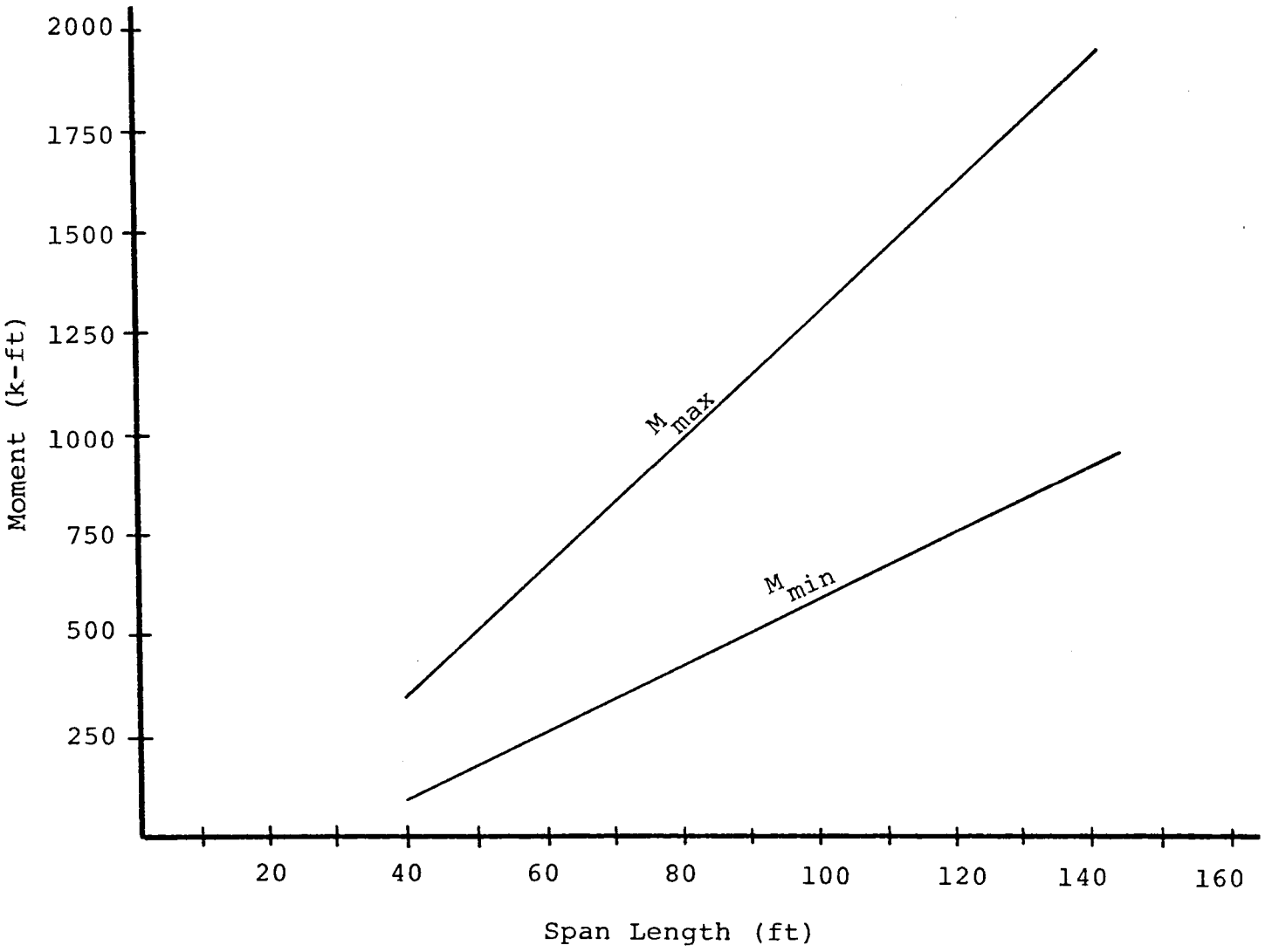


Figure 13. Moment Versus Span Length - 3S2 Trucks.

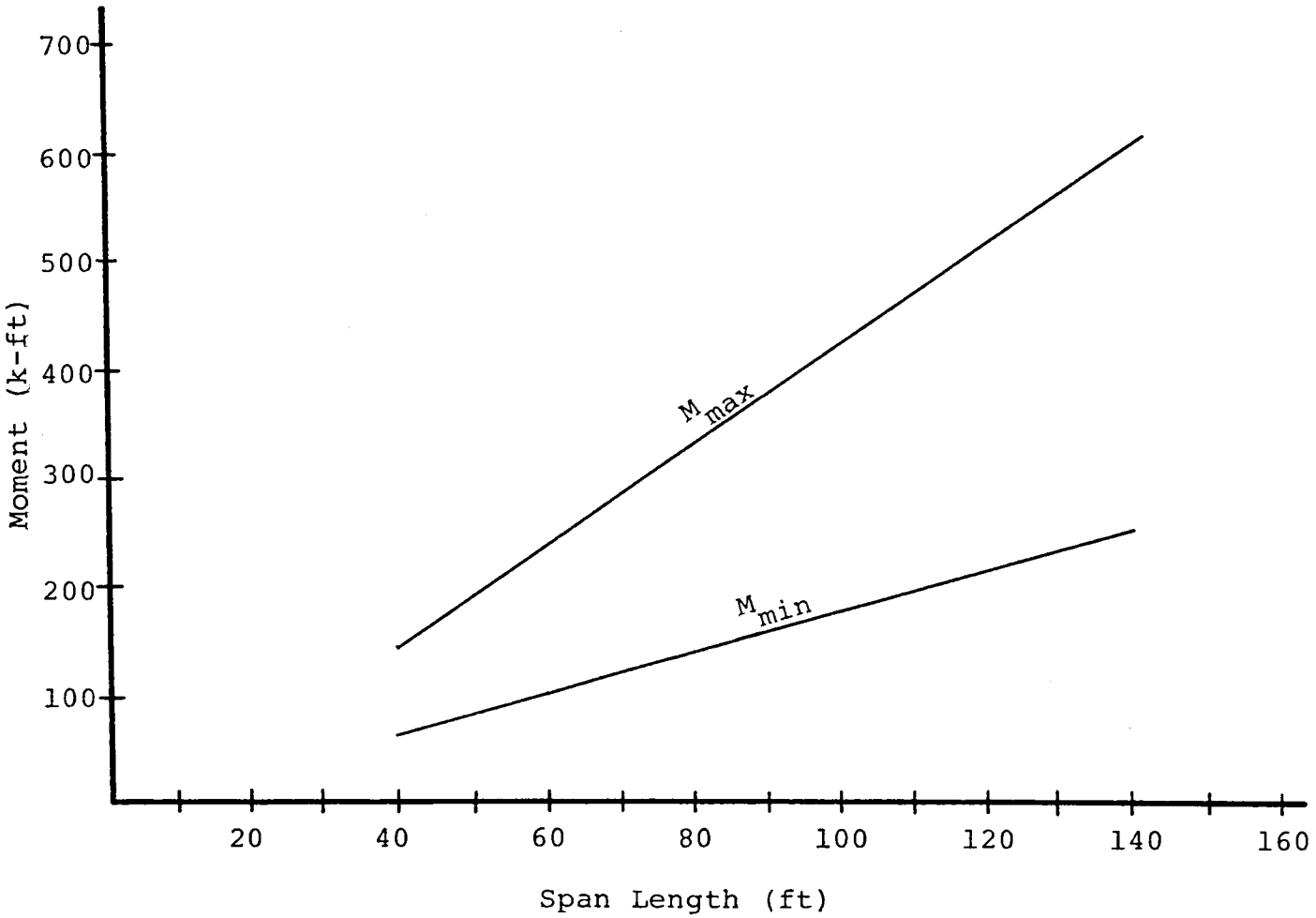


Figure 14. Moment Versus Span Length - 2D Trucks.

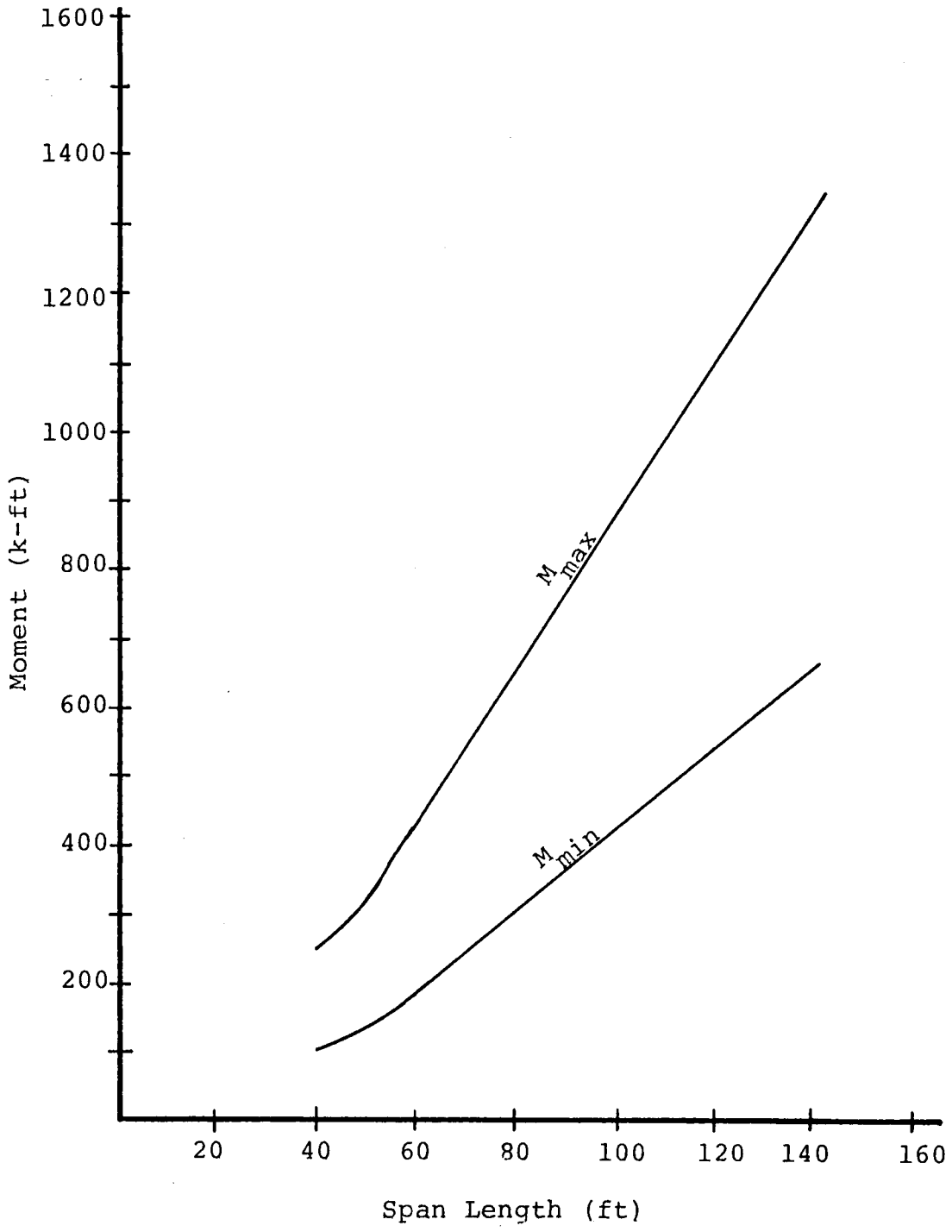


Figure 15. Moment Versus Span Length - 2S2 Trucks.

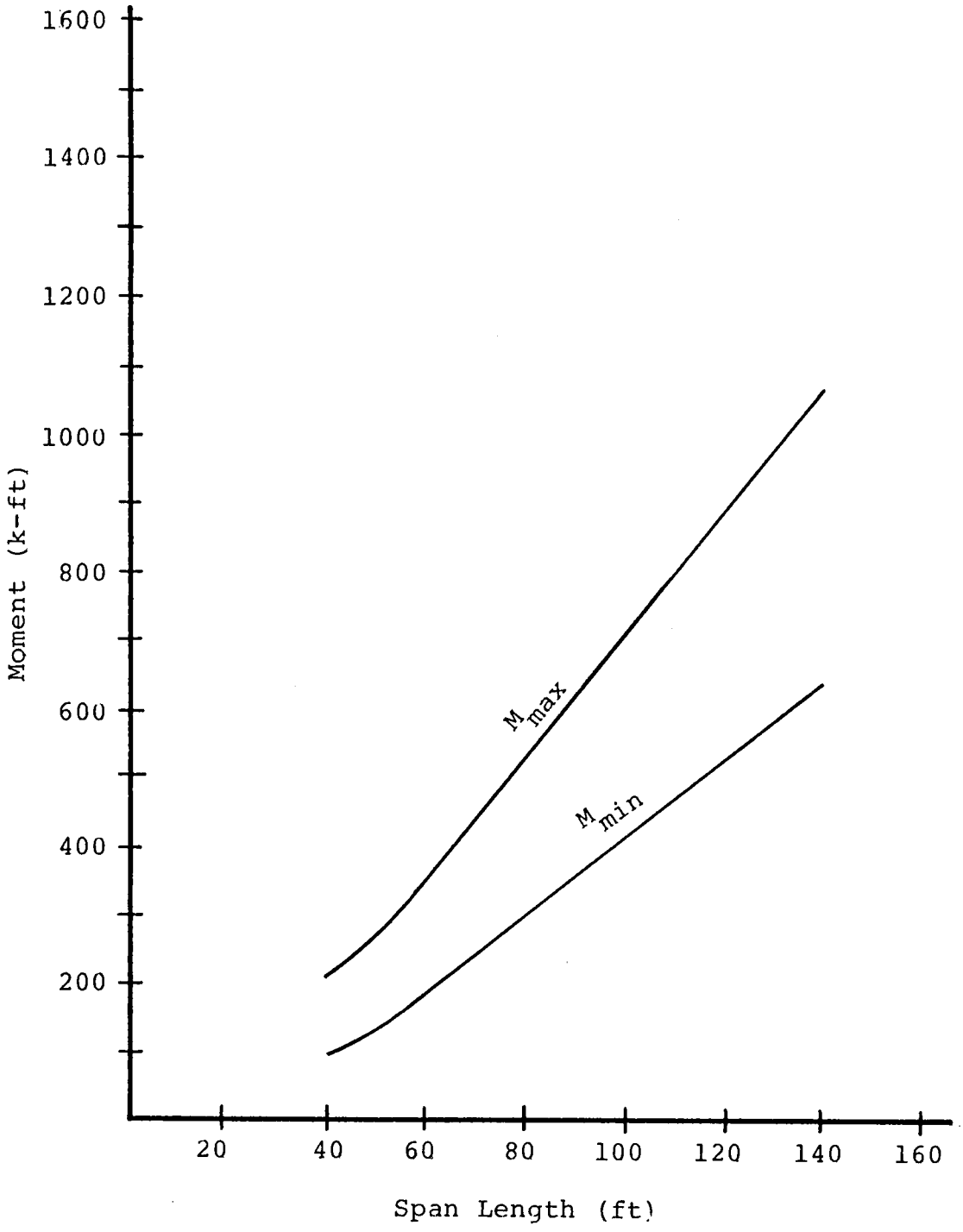


Figure 16. Moment Versus Span Length - 2S1 Trucks.

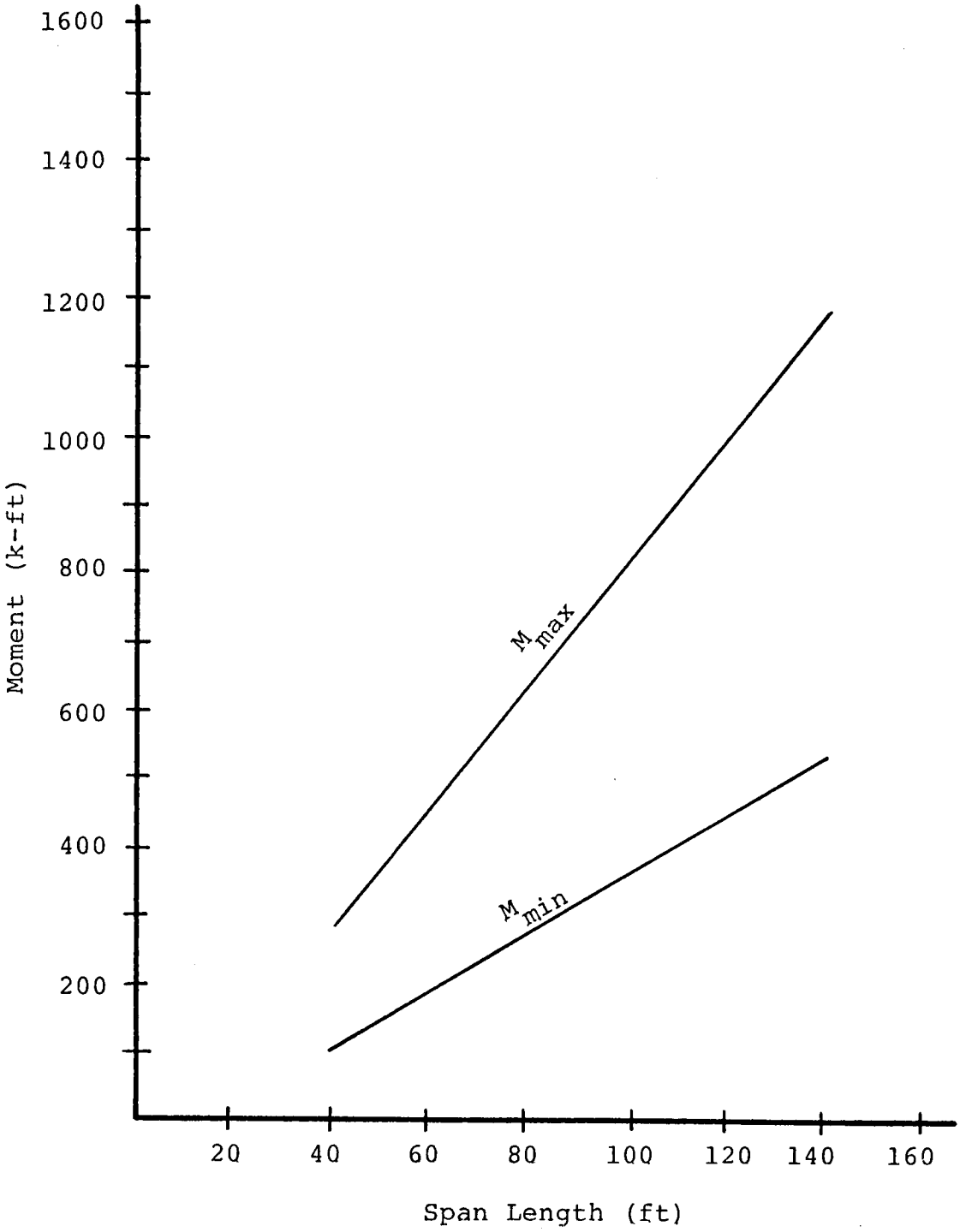


Figure 17. Moment Versus Span Length - 3 Type Trucks.

divided by the number of girders to obtain the average stress range per girder. The peak stress range was divided by the average stress range per girder to obtain a distribution factor. In the case of the theoretical data, the same truck was placed at approximately two foot intervals transversely across the bridge. The distribution factor was determined for each loading. The results of these calculations are shown in Table XIV. The distribution factor using the actual data was determined by randomly choosing a truck for any test recorded in the field. The results of the actual distribution factors is shown in Table XV.

The conclusion of the theoretical data was to use a distribution factor of 2.50 to have a 95 percent confidence level of obtaining the peak stress range or higher. The actual data indicated that a distribution factor of 2.80 must be used in order to have a 95 percent confidence level of obtaining the peak stress range or higher. Since the actual data better represents a random sample and a closer approximation to the stresses induced on the bridge, the conclusion was to use the distribution factor of 2.80.

Emphasis is directed to the fact that the distribution factor determined relates only to the seven girder bridge tested in this study. To better understand the relationship between the individual girder stress and the total cross sectional stress, a study correlating peak stress range distribution factors for bridges having four to eight girders

TABLE XIV

PEAK STRESS FACTORS USING THEORETICAL DATA

Trial	Distance from Left Curb to Center of Truck (ft)	Peak Stress Range (ksi)	Distribution Factor
1	3.8750	2.591	2.44
2*	5.8125	2.483	2.25
3	7.7500	2.700	2.37
4	9.6875	2.830	2.42
5*	11.6250	2.777	2.34
6	13.5625	2.679	2.23
7	15.5000	2.824	2.34
8	17.4375	2.861	2.36
9*	19.3750	2.748	2.28
10	21.3125	2.745	2.30
11	23.2500	2.836	2.41
12*	25.1875	2.744	2.39
13	27.1250	2.529	2.26
14	29.0625	2.555	2.38
15*	31.000	2.598	2.52
		Average =	2.35
		Standard Deviation =	0.08

* Wheel loads directly over girder

TABLE XV

PEAK STRESS FACTORS USING MEASURED DATA

Trial	Truck Type and Lane	Peak Stress Range (ksi)	Distribution Factor
1	352-R	2.161	2.24
2	352-R	1.610	2.19
3	352-R	2.028	2.58
4	352-R	1.834	2.58
5	352-R	2.538	2.69
6	352-R	2.365	2.64
7	352-R	2.161	2.82
8	352-R	1.712	2.69
9	352-R	1.987	2.38
10	352-R	2.691	2.51
11	352-L	1.668	2.14
12	352-L	2.178	2.47
13	352-L	1.784	2.39
14	352-L	1.950	2.35
15	352-L	2.073	2.31
16	352-L	2.131	2.60
17	352-L	2.131	2.30
18	352-L	1.807	2.05
19	352-L	1.761	2.10
20	352-L	2.247	2.33
21	352-M	1.982	2.52
22	352-M	1.501	2.33
23	352-M	2.181	2.45
24	352-M	1.970	2.32
25	352-M	2.252	2.54
		Average =	2.42
		Standard Deviation =	0.21

should be conducted. This is a matter subject to future research in order to better utilize the simplified approach developed.

The final step was to determine whether or not the calculated stress ranges determined from the simplified procedure will cause fatigue failure. If the stress range versus the number of loadings (S-N) diagram is available, the fatigue limit can be approximated.

Two methods available for estimating the fatigue limit are (1) Miner's Law of Cumulative Damage and (2) Root Mean Square. A general summary discussing the simple concepts involved within the two methods mentioned above is given in the Final Report on Stress History of Highway Bridges by D. W. Goodpasture.[2] The emphasis presented herein is on the application of the two procedures with regard to the information obtained by the simplified analysis.

Calculate the stress range due to the maximum and minimum loadings for each truck type, as depicted by Table II, from the simplified stress equation to find the maximum and minimum peak stress ranges (S_{max} and S_{min} respectively). The maximum and minimum loadings represent the average load for each truck type plus and minus one standard deviation respectively. Divide each stress range by the number of girders (seven) and multiply by the peak stress range distribution factor (2.80). To apply Miner's method, divide the yearly total number of cycles of loadings of a particular truck type by

two. At the number of cycles obtained, plot S_{max} and S_{min} on the S-N diagram. Find the number of cycles corresponding to failure at the stress levels of S_{max} and S_{min}. Sum the ratios of half the actual number of cycles to the fatigue limit cycle for each peak stress range and each truck type. The above procedure can be written into the following equation:

$$\Sigma \frac{N'_n}{N_n^{max}} + \frac{N'_n}{N_n^{min}} = \frac{N'_{3S2}}{N_{3S2}^{max}} + \frac{N'_{3S2}}{N_{3S2}^{min}} \ldots + \frac{N'_n}{N_n^{max}} + \frac{N'_n}{N_n^{min}}$$

where N'_n = the number of repetitions for truck type n calculated by the simplified procedure divided by two;

N_n^{max} = the number of stress cycles to failure when the stress level is S_{max} for truck type n; and

N_n^{min} = the number of stress cycles to failure when the stress level is S_{min} for truck type n.

The graphical interpretation of Miner's method is shown in Figure 18. Fatigue design criteria is adequate given that the summation of the ratios presented above is less than 1.0. In contrast, fatigue will cause failure if the summation is greater than 1.0.

Miner's method disregards important variables such as loading sequence, notch effect, and mean stress.[3] The ability to define the loading cycles due to a truck being fully loaded or unloaded is a matter of engineering judgment and subject to

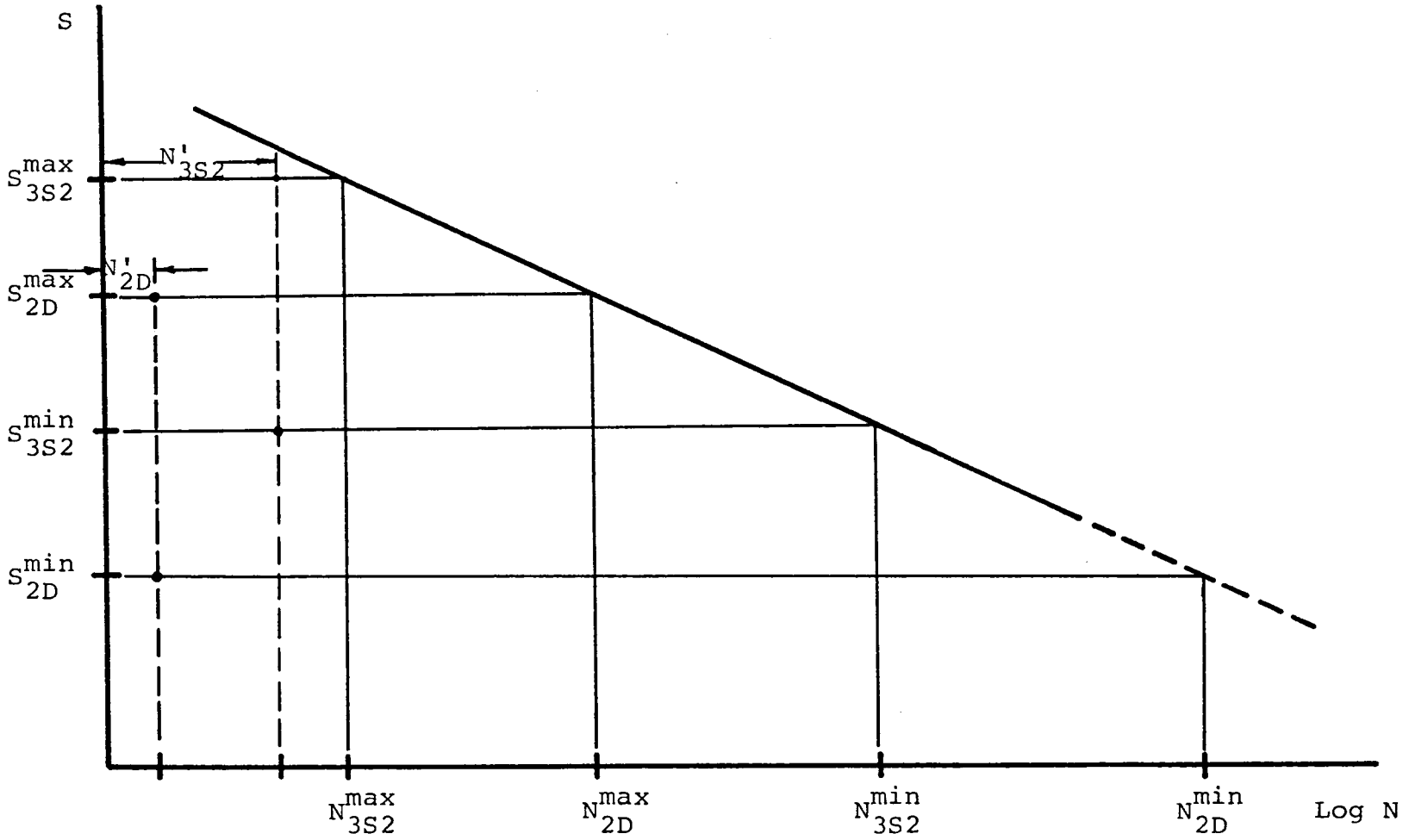


Figure 18. Illustration of Miner's Cumulative Damage Theory.

inaccuracy. The total loading cycle was divided in half in the procedure described above giving the same importance to loaded trucks as well as unloaded trucks. The factor of one half was based on gross weight distributions collected within this study which indicated a bi-modal distribution that reflects the fact that the majority of trucks were either completely loaded or empty. The distribution of loading cycles concerning loaded to unloaded trucks is a matter subject to further research. The life span in years is determined by taking the reciprocal of the summation of the above equation.

The second procedure used in estimating the fatigue life of highway bridges deals with obtaining the root mean square (RMS) of the various peak stress ranges induced by random truck samplings. The RMS is defined as:

$$RMS = \frac{\Sigma S_i^2}{n}$$

where S_i is the variable peak stress range at the point in question and n is the total number of observations. Once this stress range is determined, the estimate of cycles of loads needed to produce failure can be calculated by an empirical formula, or by direct use of the S-N curve. The calculated loading cycles to cause failure is compared to the calculated loading cycles determined by the simplified analysis to establish if failure due to fatigue will occur.

First a bi-modal distribution of gross weights is assumed for each truck type (which is a good assumption based on

weight histograms collected throughout stress history studies). The root mean square simplifies to the following:

$$S_{rms} = \sqrt{\frac{\Sigma \left(\frac{S^2_{max_n} + S^2_{min_n}}{2} \right) \chi_n}{\Sigma \chi_n}}$$

where S_{max_n} = the maximum peak stress range calculated for truck type n;

S_{min_n} = the minimum peak stress range calculated for truck type n;

χ_n = number of cycles of loadings for truck type n; and

n = type of truck (i.e., 3S2, 2D, 2S2, 2S1, etc.).

Refer to the S-N diagram, as depicted by Figure 18, and find the number of cycles that will cause failure at the stress range of S_{rms}. The Root Mean Square procedure enables the designer to check the actual number of cycles of loadings to the calculated number of cycles of loading to cause failure.

In conclusion, the main objective in this study has been completed with the development of the simplified procedure described. An example problem illustrating the calculations necessary to develop the stress equation is shown in Appendix A. The procedure used in Appendix A for determining the fatigue life is not the present procedure used by designers. The designer uses the AASHTO Specifications as described in the conclusion of the appendix.

A conservative method was also presented within this study to approximately describe the truck traffic characteristics. With the simplified procedure developed, the next step was to check the validity of the analysis by the use of comparisons to actual field data and computer results.

CHAPTER VII

COMPARISON OF RESULTS

1. Westbound Comparisons

Since the simplified analysis was developed without emphasis on any particular bridge, the information collected on the westbound bridge of field tests earlier discussed were used for comparisons. The control test truck used during the field tests was used for emphasis towards individual comparisons. The test truck was not used on the eastbound bridge, hence, the eastbound bridge was compared only to computer analysis results.

The control test vehicle was loaded with the maximum weight allowed by state and federal laws (although in actuality it was slightly overweight). This provided the possibility of using both simplified analyses developed to compare with results of actual field analysis and computer analysis. The simplified analyses were (1) the detailed procedure involving the development of the summation of the stress equation, then placing the control truck variables into the equation to determine the stress induced on the bridge cross section, and (2) the simple span procedure for simple span bridges which involves finding the

moment induced on the centerline of the span by a specific truck type (see Figure 13 page 58 and Figure 17 page 62), then dividing by the section modulus of the entire bridge cross section.

For the purpose of comparison, the stress representing the measured analysis results was derived by using the average stress induced from all three lanes. Likewise, the value used from the computer analysis results was the average of the stresses caused by loading the control truck simulation in every lane. These values were compared to the results obtained by using the detailed and simple span procedures developed. The results of these comparisons are shown in Table XVI.

The next comparison was of maximum and minimum stress ranges. Since the computer analysis was shown to be an adequate method for calculating stresses on the bridge, the two simplified procedures were compared to the computer analysis results. Similar equations were developed by the same procedure from the computer analysis. Maximum and minimum truck configurations for a 3S2 truck type were placed in the equations to determine the stress ranges. The results of these comparisons are shown in Table XVI.

The last method of comparison was that of stress versus frequency graphs developed by each analysis. The measured stresses were placed in a histogram, thus, providing one of the graphs used in the comparisons. In order to develop the histograms for the computer and simplified analyses, a

TABLE XVI

COMPARISON OF MEASURED, THEORETICAL

AND SIMPLIFIED RESULTS

Bridge	Analysis	Control Truck Sum of Stresses (ksi)	Maximum Sum of Stress Ranges (ksi)	Minimum Sum of Stress Ranges (ksi)
	Measured	7.10		
	Theoretical	7.91	7.401	2.586
Westbound	Detailed Process	7.719	7.244	2.697
	Simple Span Process	7.536	7.536	2.898
	Theoretical		7.00	2.36
Eastbound	Detailed Process		7.21	2.73
	Simple Span Process		7.11	2.74

process called the Monte Carlo Simulation[1] was used. In brief, this process simulates random truck loadings on the bridge and calculates the stress induced by each truck forming a histogram of stress versus frequency. Figure 19 shows the results of these comparisons. Emphasis is given to the fact that the truck characteristics used in the Monte Carlo Simulation were developed from data collected at the weigh station previously mentioned in Chapter III.

2. Eastbound Comparisons

The main comparison for the eastbound bridge was of the computer and simplified analyses. Two methods for comparisons were used. They were (1) maximum and minimum stress ranges and (2) stress versus frequency graphs generated by the Monte Carlo Simulation.

The first comparison was derived by calculating the stress equations from the computer and simplified analyses. Each equation was used to calculate maximum and minimum stress ranges induced by 3S2 truck loadings. The results of these comparisons are shown in Table XVI.

The final comparison was that of the stress versus frequency graphs produced by placing the calculated stress equations in the Monte Carlo Simulation of the computer and simplified analyses. Actual field measurements were too small in number to justify comparing the measured stress histogram of the eastbound bridge. The results of the above

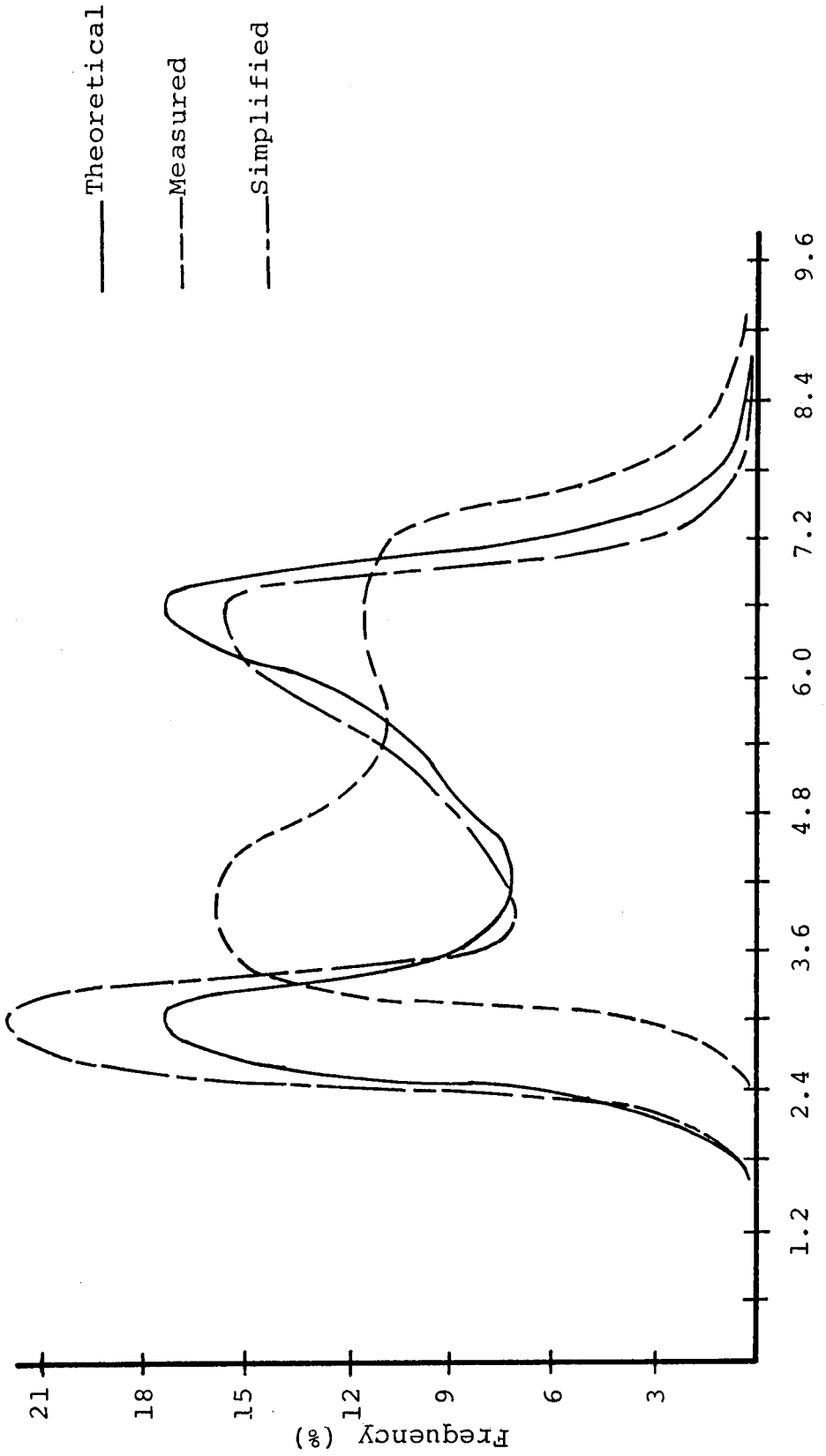


Figure 19. Comparison of Measured, Theoretical and Simplified Summation of Stresses - 3S2 Trucks.

comparison is shown in Figure 20. Conclusions and discussions of these comparisons presented will be shown in the following chapters.

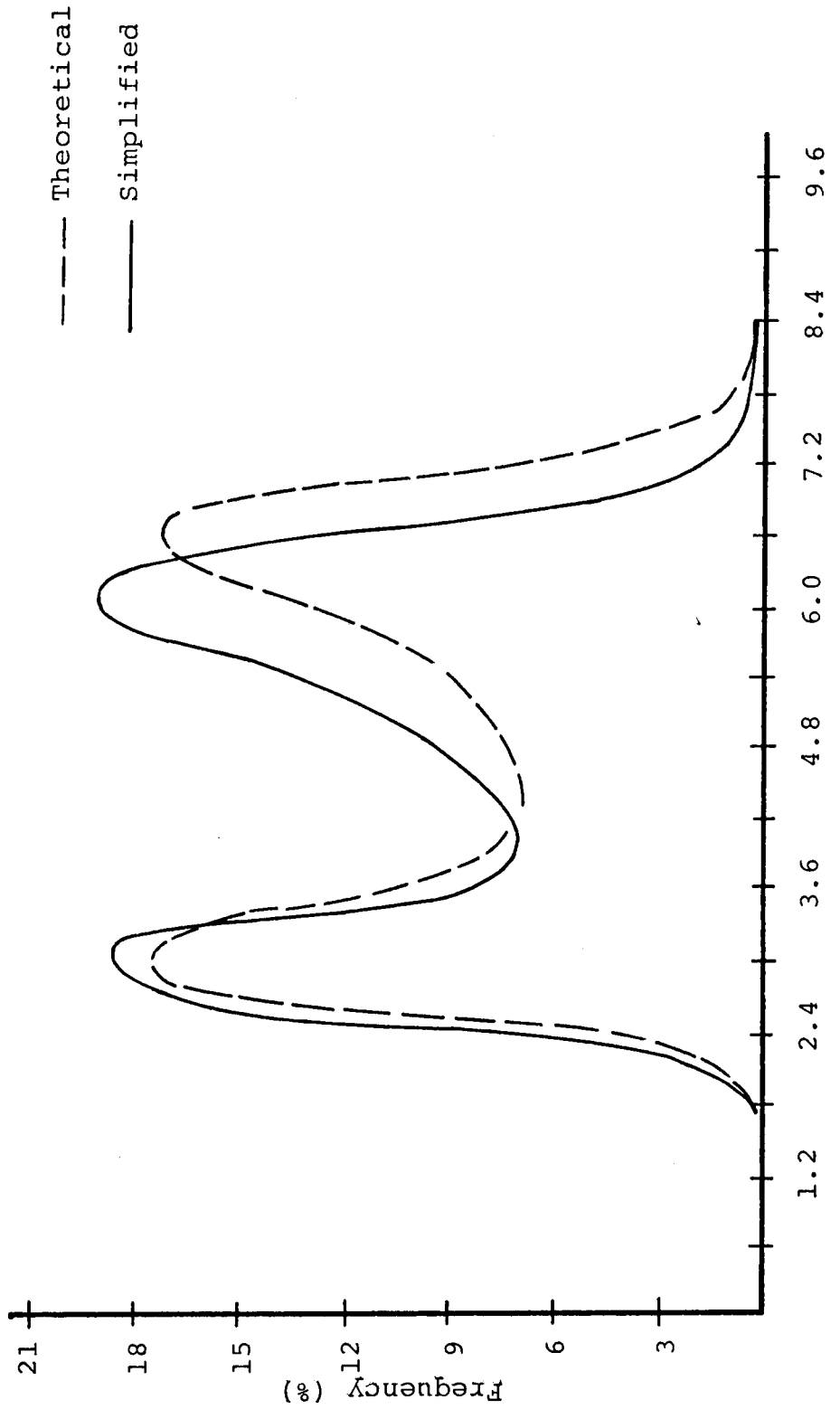


Figure 20. Comparison of Theoretical and Simplified Summation of Stresses - 3S2 Trucks.

CHAPTER VIII

DISCUSSION OF RESULTS

1. General

The overall results of the comparisons presented in the previous chapter were considered to be a good validation for the simplified analysis procedure developed. Close relationships were evident in the individual truck loadings as well as the stress histograms generated by the Monte Carlo Simulation. Both bridges showed comparisons that were clearly evident of the close compatibility between the three methods of stress determinations (i.e., actual field measurements, computer analysis, and the simplified analysis).

The two main sources of error were (1) the comparison of the theoretical stresses to the actual stresses for the control load, and (2) the phase shift of the actual stress histogram compared to the Monte Carlo Simulations of the computer and simplified analyses. Otherwise, all the results were within an acceptable percentage of error.

The reason behind the error (11 percent off the actual stress) of the computer analysis with the control load was due to the inclusion of the center lane stress calculation into the overall average. Earlier it was shown that the actual stiffness of the bridge was best represented by the stiffness

near the outside lanes. If the center lane stress was excluded, the comparison would be more acceptable (7 percent off the actual stress).

The actual stress histogram was formed due to the stresses caused by the truck traffic crossing the bridge site, whereas the stress histograms developed by the Monte Carlo Simulation were formed under the basis of the truck traffic at the state weigh station. The truck traffic characterisitcs were statistically inconsistent for the samples taken at the weigh station as compared to the samples taken at the bridge. Hence, the Monte Carlo Simulation stress histogram did not adequately compare with the actual stress histogram.

The overall results presented a strong case for the acceptance of the method generated for determining stresses induced by present day truck traffic. In any case, the procedures developed within this study, and the results of various comparisons were considered to be the direction in which future research should be oriented.

CHAPTER IX

CONCLUSIONS

1. Summary

Two bridges carrying eastbound and westbound traffic in Knoxville, Tennessee were instrumented with strain gages. A computerized data collection and reduction process was utilized in conjunction with field personnel recording truck traffic characteristics on the bridges and a nearby state weigh station. Included in the field testing was the use of a control loaded 3S2 truck. From the data collected in the field, stress and weight histograms were developed.

A theoretical analysis using finite element methods was refined and developed in order to simulate actual stresses induced by truck traffic. Once the theoretical analysis was comparable to the actual field analysis, a simplified method was developed for calculating stresses.

The simplified method developed resulted in a single equation for determining the stress induced on the entire cross section of most I-beam stringer deck bridge configurations. The actual, computer, and simplified analyses using the control truck vehicle were compared, along with stress histrograms developed by Monte Carlo Simulations of the computer and simplified analyses. The results of the comparisons confirmed the validity of the simplified analysis method developed.

2. Applications of Findings

The simplified analysis developed may be applied to general I-beam stringer deck bridge configurations for determining the stress induced on the bridge due to truck traffic. The simplified procedure can be used on steel girder or concrete bridges since it is only dependent upon the designer being able to calculate member properties of the bridge. Emphasis is given to the fact that this procedure is only a basis for which further research may be directed to establish a more generalized method of fatigue analysis.

3. Future Research Suggestions

Future research should be directed towards three main avenues. These avenues which should be investigated in depth are (1) finding the appropriate factors of safety to be applied to the simplified analysis results in order to account for such items as future truck traffic, probable material defects, and errors in the analysis method, (2) determining a method for accurately distributing the stress to individual girders, and (3) determining a more generalized procedure for estimating the fatigue life of highway bridges.

The first suggestion of future research could be generated by collecting even more data of truck traffic characteristics (e.g., gross weights, truck types, axle spacings, etc.), and by using an even more detailed testing

procedure for bridge materials specifically subjected to fatigue. Factors of safety could then be generated based on firm results of the generated data.

The second process of distributing the stress to each girder would enable the designer to check each main load carrying member of the bridge for fatigue. This, in essence, has been the overall objective of stress history research. There has been research directed towards the goal suggested here which reflects the necessity of further investigation.

The final suggestion is merely to use the simplified procedure developed within this study on bridges already tested. From the comparisons of the simplified analysis results to the actual field measurements of as many tested bridges as deemed necessary, the validity of the procedure generated herein would become clearer. The results of the comparisons would also bring forth possibilities for altering the simplified analysis in order to make it more concise.

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APPENDIX

APPENDIX

EXAMPLE PROBLEM USING

SIMPLIFIED APPROACH

GIVEN: Bridge with following information known:

a) Simple span of 62 feet;

b) Seven girder, three lane, I-beam stringer deck;

c) Section modulus of entire bridge cross section = $S = 7250 \text{ in.}^3$;

d) Located in rural area;

e) ADT = 45,000 of which 15 percent are trucks; and

f) Peak stress range distribution factor = 2.80.

REQUIRED: Using Simplified approach, along with tables and figures given in this study, determine the life span of the bridge due to fatigue.

1) Use a 3S2 truck configuration with three fixed variables shown below:

   a) Front axle weight = 8.75 kips

   b) Front axle spacing = 12.75 feet

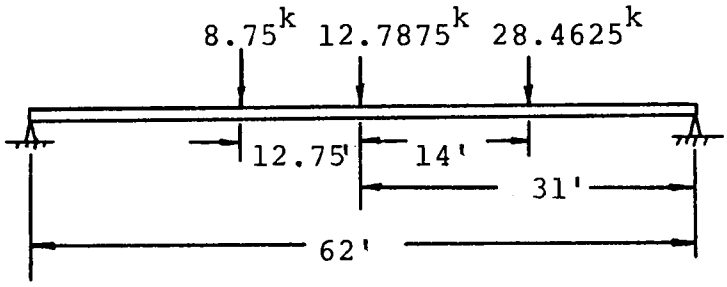
   c) Gross vehicle weight = 50.0 kips

2) Place 3S2 truck configuration on span to produce maximum moment at the span centerline in the following four different loadings: (As shown in Figure 10 on page 50).

   Note: 0.31 (50.0 - 8.75) = 12.7875

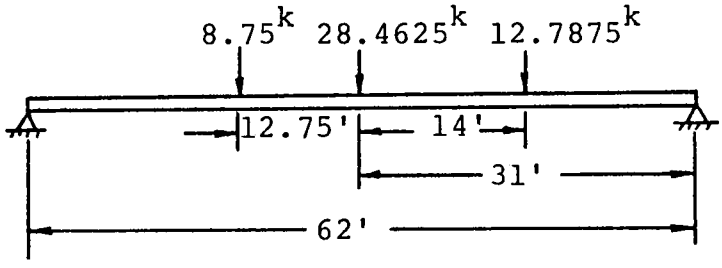
   0.69 (50.0 - 8.75) = 28.4625

a)



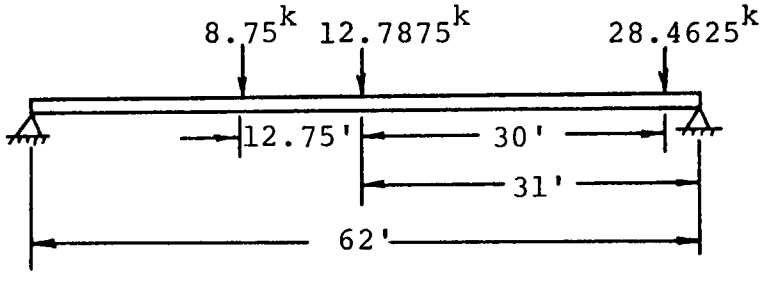
$M_{\ell} = 6517$ k-in.

b)



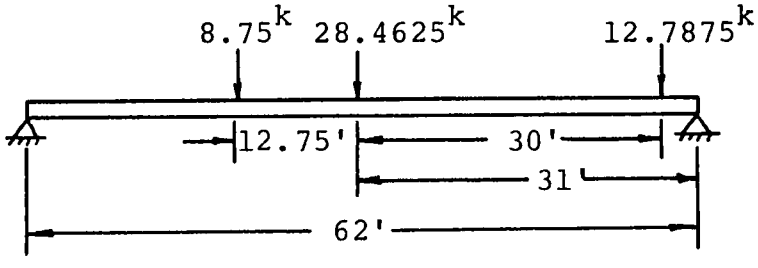
$M_{\ell} = 7279$ k-in.

c)



$M_{\ell} = 4101$ k-in.

d)



$M_{\ell} = 5735$ k-in.

3) Determine the stress due to each loading used in steps 2 (a) through 2 (d) with S = 7250 in.^{3}.

a) $\sigma_a = \frac{M_{\text{¢}_a}}{S} = \frac{6157}{7250} = 0.8989$ ksi

b) $\sigma_b = \frac{M_{\text{¢}_b}}{S} = \frac{7279}{7250} = 1.004$ ksi

c) $\sigma_c = \frac{M_{\text{¢}_c}}{S} = \frac{4101}{7250} = 0.5657$ ksi

d) $\sigma_d = \frac{M_{\text{¢}_d}}{S} = \frac{5735}{7250} = 0.7910$ ksi

4) Multiply the stresses determined in step 3 by the number of girders (7) to find the total stress on the entire bridge cross section.

a) $\Sigma\sigma_a = (0.8989)(7.0) = 6.292$ ksi

b) $\Sigma\sigma_b = (1.004)(7.0) = 7.028$ ksi

c) $\Sigma\sigma_c = (0.5657)(7.0) = 3.960$ ksi

d) $\Sigma\sigma_d = (0.7910)(7.0) = 5.537$ ksi

5) Plot the graph of $\Sigma\sigma$ vs Rear axle spacing (shown in Figure A-1) as follows:

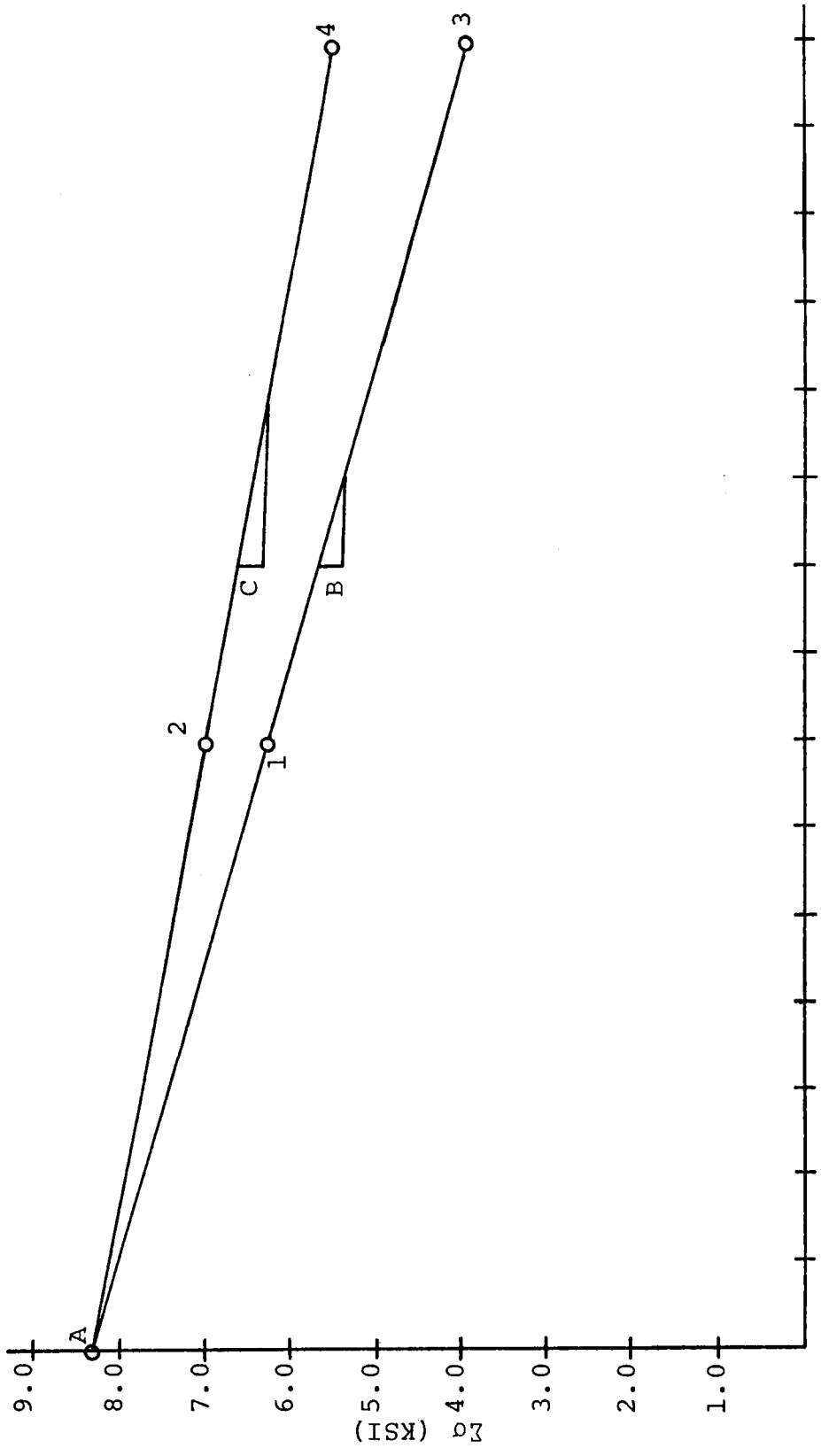


Figure A-1. Summation of Stresses Versus Rear Axle Spacing for the Example Problem.

6) Calculate the slope of each line and the point of intersection (indicated as points B, C, and A respectively).

a) Slope of bottom line = B = $\frac{\sigma c - \sigma a}{30 - 14}$

$$B = \frac{3.960 - 6.292}{16} = \underline{-0.14575}$$

b) Slope of top line = C = $\frac{\sigma d - \sigma b}{30 - 14}$

$$C = \frac{5.537 - 7.028}{16} = \underline{-0.0931875}$$

c) Point of intersection = A

1) Using bottom line:

$Y = -0.14575\ x + A$

at $Y = 6.292$, $x = 14.0$

therefore:

$6.292 = -0.14575(140.0) + A$

$\underline{A = 8.333}$

2) Check A by using top line:

$Y = -0.0931875\ x + A$

at $Y = 7.028$, $x = 14.0$

therefore:

$7.028 = -0.0931875(14.0) + A$

$\underline{A = 8.333}$ ok

7) Calculate the change in rate of slopes of the two lines on the graph of step 5 and formulate $\Sigma\sigma$ equation as follows:

a) Change in rate of slopes = $D = \left|\frac{B - C}{38.0}\right|$

$$D = \left|\frac{(-0.14575) - (-0.0931875)}{38.0}\right|$$

$$D = \left|\frac{-0.0525625}{38.0}\right| = 0.00138322$$

b) Formulation of $\Sigma\sigma$ equation:

$$\Sigma\sigma = \left[A - \left(\left[B - (D)(X-31.0)\right] Y\right)\right] \frac{Z}{50.0}$$

Substituting values of A, B, and D gives the following:

(Note: Use the absolute value of B in the equation).

$$\Sigma\sigma = \left[8.333 - \left(\left[0.14575 - (.00138322)(X-31.0)\right] Y\right)\right] \frac{Z}{50.0}$$

where X = percent distribution to middle axle of middle-to-rear axle distribution percentages;

Y = rear axle spacing (feet);

Z = gross vehicle weight of truck under inspection (kips); and

$\Sigma\sigma$ = stress induced on entire bridge cross section (ksi).

8) Calculate $\Sigma\sigma_{max}$ and $\Sigma\sigma_{min}$ for each truck type of Table II on page 24.

a) Truck type 3S2

1) Maximum loading:

X = 52.6 + 6.6 = 59.2 percent

Y = 30.75 - 3.30 = 27.45 feet

Z = 50.10 + 16.90 = 67.0 kips

2) Minimum loading:

X = 52.6 - 6.6 = 46.0 percent

Y = 30.75 + 3.3 = 34.05 feet

Z = 50.10 - 16.9 = 33.2 kips

Substituting into stress equation of step 7:

$$\Sigma\sigma = \left[8.333 - \left(\left[0.14575 - (.00138322)(59.2-31.0)\right] 27.45\right)\right]\frac{67.0}{50.0}$$

$$= \left[8.333 - \left(\left[0.14575 - 0.0390069\right] 27.45\right)\right]\frac{67.0}{50.0}$$

$$= \left[8.333 - 3.89376\right]\frac{67.0}{50.0}$$

$$\underline{\Sigma\sigma_{max} = 5.949 \text{ ksi}}$$

$$\Sigma\sigma = \left[8.333 - \left(\left[0.14575 - (.00138322)\ (46.0-31.0)\right] 34.05\right)\right]\frac{33.2}{50.0}$$

$$= \left[8.333 - \left(\left[0.14575 - 02074835\right] 34.05\right)\right]\frac{33.2}{50.0}$$

$$= \left[8.333 - 4.256306\right]\frac{33.2}{50.0}$$

$$\underline{\Sigma\sigma_{min} = 2.707 \text{ ksi}}$$

b) Truck type 2D

1) Maximum loading:

X = 37.0 + 6.5 = 43.5 percent

Y = 15.0 - 2.25 = 12.75 feet

Z = 14.5 + 5.9 = 20.4 kips

2) Minimum loading:

X = 37.0 - 6.5 = 30.5 percent

Y = 15.0 + 2.25 = 17.25 feet

Z = 14.5 - 5.9 = 8.6 kips

Substituting into stress equation of step 7:

$\Sigma\sigma_{max}$ = 5.825 ksi

$\Sigma\sigma_{min}$ = 0.999 ksi use 1.0 ksi

c) Truck type 2S2

1) Maximum loading:

X = 48.6 + 6.4 = 55.0 percent

Y = 28.5 - 2.5 = 26.0 feet

Z = 35.2 + 10.6 = 45.8 kips

2) Minimum loading:

X = 48.6 - 6.4 = 42.2 percent

Y = 28.5 + 2.5 = 31.0 feet

Z = 35.2 - 10.6 = 24.6 kips

Substituting into stress equation of step 7:

$\Sigma\sigma_{max}$ = 4.952 ksi

$\Sigma\sigma_{min}$ = 2.113 ksi

d) Truck type 2S1

1) Maximum loading:

X = 52.6 + 7.5 = 60.1 percent

Y = 29.5 - 3.5 = 26.0 feet

Z = 29.7 + 6.7 = 36.4 kips

2) Minimum loading:

X = 52.6 - 7.5 = 45.1 percent

Y = 29.5 + 3.5 = 33.0 feet

Z = 29.7 - 6.7 = 23.0 kips

Substituting into stress equation:

$\Sigma\sigma_{max}$ = 4.070 ksi

$\Sigma\sigma_{min}$ = 1.917 ksi

e) Truck type 3

1) Maximum loading:

X = 38.0 + 6.0 = 48.0 percent

Y = 17.25 - 2.4 = 14.85 feet

Z = 26.8 + 9.5 = 36.3 kips

2) Minimum loading:

X = 38.0 - 6.0 = 32.0 percent

Y = 17.25 + 2.4 = 19.65 feet

Z = 26.8 - 9.5 = 17.3 kips

Substituting into stress equation:

$\Sigma\sigma_{max}$ = 4.732 ksi

$\Sigma\sigma_{min}$ = 1.902 ksi

9) Calculate the average sum of the girder stress ranges and determine the maximum and minimum peak stress range for each truck type (S_{max} and S_{min} respectively) with a total of 7 girders and a peak stress range distribution factor of 2.80.

a) Truck type 3S2

$$S_{max} = \frac{\Sigma\sigma_{max}}{\text{Number of girders}} \times \text{Peak stress range distribution factor}$$

$$S_{max}^{3S2} = \frac{5.949}{7.0} \times 2.80 = 2.380 \text{ ksi}$$

$$S_{min}^{3S2} = \frac{2.707}{7.0} \times 2.80 = 1.083 \text{ ksi}$$

b) Truck type 2D

$$S_{max}^{2D} = \frac{5.825}{7.0} \times 2.80 = 2.33 \text{ ksi}$$

$$S_{min}^{2D} = \frac{1.0}{7.0} \times 2.80 = 0.400 \text{ ksi}$$

c) Truck type 2S2:

$$S_{max}^{2S2} = \frac{4.952}{7.0} \times 2.80 = 0.845 \text{ ksi}$$

$$S_{min}^{2S2} = \frac{2.113}{7.0} \times 2.80 = 0.845 \text{ ksi}$$

d) Truck type 2S1

$$S_{max}^{2S1} = \frac{4.07}{7.0} \times 2.80 = 1.628$$

$$S_{min}^{2S1} = \frac{1.917}{7.0} \times 2.80 = 0.767 \text{ ksi}$$

e) Truck type 3

$$S_{max}^{3} = \frac{4.732}{7.0} \times 2.80 = 1.893 \text{ ksi}$$

$$S_{min}^{3} = \frac{1.902}{7.0} \times 2.80 = 0.761 \text{ ksi}$$

10) Calculate the number of cycles per day of each truck type using Table V on page 28, 15 percent of the ADT being truck traffic, and a bridge located in a rural section.

ADTT = 0.15(ADT) = 0.15(45000) = 6750

a) Truck type 3S2

$$N_{3S2} = (0.41)(6750) = 2767.5$$

b) Truck type 2D

$$N_{2D} = (0.21)(6750) = 1417.5$$

c) Truck type 2S2

$$N_{2S2} = (025)(6750) = 1687.5$$

d) Truck type 2S1

$$N_{2S1} = (0.07)(6750) = 472.5$$

e) Truck type 3

$N_3 = (0.06)(6750) = 405$

11) Summary:

Truck type	S_{max} (ksi)	S_{min} (ksi)	Total Number of Cycles per Day
3S2	2.380	1.083	2767.5
2D	2.330	0.400	1417.5
2S2	1.981	0.845	1687.5
2S1	1.628	0.767	472.5
3	1.893	0.761	405

12) Check fatigue capacity of bridge for two cases as follows using the Root Mean Square and Miner's procedures:

a) Plain rolled beam at the centerline

b) The welded end of cover plated beams.

The following conservative assumptions will be used:

1) Let all the trucks travel in one lane;

2) Let half the trucks be loaded and half the trucks be unloaded.

Case a:

Shown in Figure A-2 is the S-N diagram plot for tests on rolled beams.[10] The equation describing the plot is:

$$\text{Log } N_f = 11.121 - 3.178 \text{ Log } S_r$$

where N_f = number of cycles to failure

S_r = stress range (ksi)

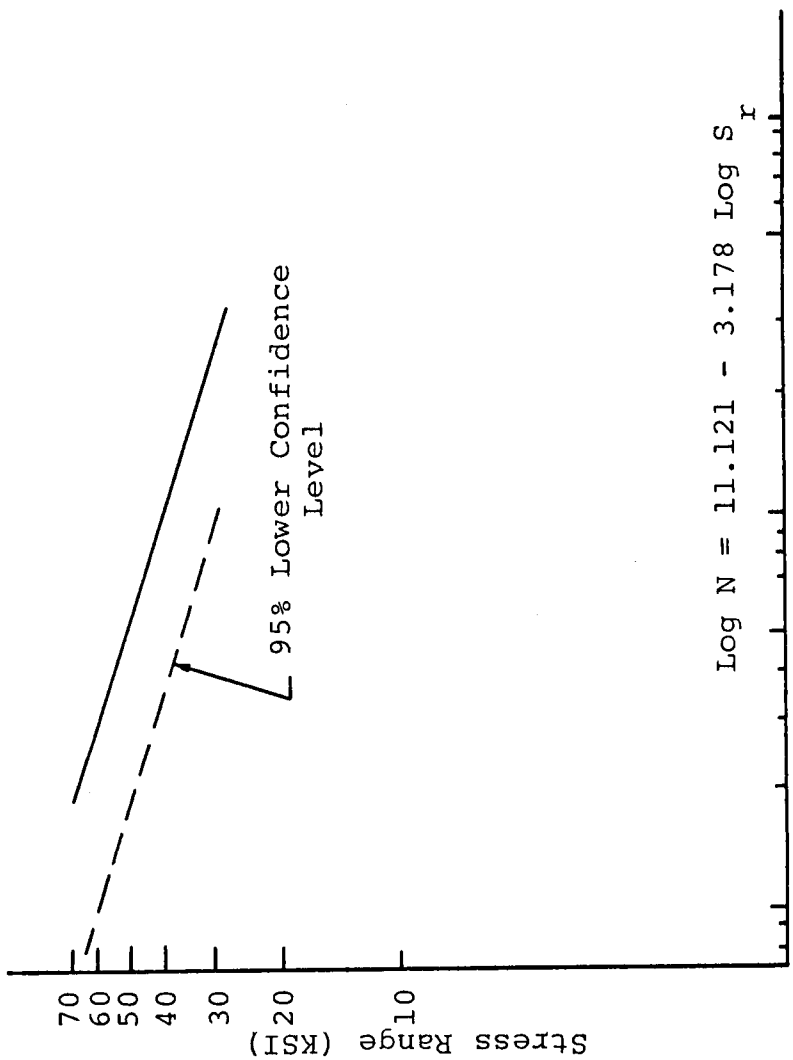


Figure A-2. Test Results on Rolled Beams.

Root Mean Square:

N_{act} = Actual Number of Cycles = 6750 cycles/day = 2,463,750 cycles/year

N_f = Number of cycles to failure

$$\text{Log } N_f = 11.121 - 3.178 \text{ Log } S_{rms}$$

where

$$S_{rms} = \sqrt{\frac{\Sigma\left(\frac{S^2_{max_n} + S^2_{min_n}}{2}\right)X_n}{\Sigma X_n}}$$

$$= \Bigg(\Bigg(\left[\frac{(2.38)^2 + (1.083)^2}{2}\right]2767.5 + \left[\frac{(2.33)^2 + 0.40)^2}{2}\right]1417.5$$

$$+ \left[\frac{(1.981)^2 + (0.845)^2}{2}\right]1687.5 + \left[\frac{(1.628)^2 + (0.767)^2}{2}\right]472.5$$

$$+ \left[\frac{(1.893)^2 + (0.761)^2}{2}\right]405\Bigg) \Big/ 6750\Bigg)^{1/2}$$

$$= ((9.4610986 \times 10^3 + 3.9611328 \times 10^3 + 3.9136381 \times 10^3 + 7.6513649 \times 10^2 + 8.4292042 \times 10^2) \Big/ 6750)^{1/2}$$

$$= \underline{1.6752633 \text{ ksi}}$$

$$\text{Log } N_f = 11.121 - 3.178 \text{ Log } (1.6752633)$$

$$= 11.121 - 3.178\ (0.22408307) = 10.408864$$

$$N_f = 2.564 \times 10^{10} \text{ cycles}$$

Life span:

$$\text{Life} = \frac{N_{failure}}{N_{actual}} = \frac{2.564 \times 10^{10} \text{ cycles}}{2.46375 \times 10^6 \text{ cycles/year}} = 1.04 \times 10^4 \text{ yrs}$$

$$\underline{\text{Life} = 10{,}400 \text{ years}}$$

Miner's Law:

Note: Since the S-N plot does not extend to the lower levels of stress ranges calculated, the N_n^{max} and N_n^{min} (number of cycles to failure at stress level of S_{max} and S_{min} for truck type n respectively) will be calculated from the equation of the plot shown Figure A-2.

1) Truck type 3S2:

$$\text{Log } N_{3S2}^{max} = 11.121 - 3.178 \text{ Log } (2.38)$$

$$N_{3S2}^{max} = 8.40 \times 10^{9}$$

$$\text{Log } N_{3S2}^{min} = 11.121 - 3.178 \text{ Log } (1.083)$$

$$N_{3S2}^{min} = 1.0255 \times 10^{11}$$

2) Truck type 2D:

$$\text{Log } N_{2D}^{max} = 11.121 - 3.178 \text{ Log } (2.33)$$

$$N_{2D}^{max} = 8.99 \times 10^{9}$$

$$\text{Log } N_{2D}^{min} = 11.121 - 3.178 \text{ Log } (0.400)$$

$$N_{2D}^{min} = 2.43 \times 10^{12}$$

3) Truck type 2S2:

$$\text{Log } N_{2S2}^{max} = 11.121 - 3.178 \text{ Log } (1.981)$$

$$N_{2S2}^{max} = 1.505 \times 10^{10}$$

$$\text{Log } N_{2S2}^{min} = 11.121 - 3.178 \text{ Log } (0.845)$$

$$N_{2S2}^{min} = 2.257 \times 10^{11}$$

4) Truck type 2S1:

$$\text{Log } N_{2S1}^{max} = 11.121 - 3.178 \text{ Log } (1.628)$$

$$N_{2S1}^{max} = 2.808 \times 10^{10}$$

$$\text{Log } N_{2S1}^{min} = 11.121 - 3.178 \text{ Log } (0.767)$$

$$N_{2S1}^{min} = 3.07 \times 10^{11}$$

5) Truck type 3:

$$\text{Log } N_{3}^{max} = 11.121 - 3.178 \text{ Log } (1.893)$$

$$N_{3}^{max} = 1.739 \times 10^{10}$$

$$\text{Log } N_{3}^{min} = 11.121 - 3.178 \text{ Log } (0.761)$$

$$N_{3}^{min} = 3.147 \times 10^{11}$$

$$\text{Life span} = \frac{1}{\Sigma\left(\frac{N_n'}{N_n^{max}} + \frac{N_n'}{N_n^{min}}\right)}$$

where N_n' = actual number of cycles for truck type n divided by 2.0;

and N_n^{max}, N_n^{min} as defined in note under Miner's Law

Summary for Miner's Law at center line span:

Truck type	N_n^{max}	N_n^{min}	N_n' (cycles/day)
3S2	8.4×10^{9}	1.0255×10^{11}	1383.75
2D	8.99×10^{9}	2.43×10^{12}	708.75
2S2	1.505×10^{10}	2.257×10^{11}	843.75
2S1	2.808×10^{10}	3.07×10^{11}	236.25
3	1.739×10^{10}	3.147×10^{11}	202.50

$$\Sigma \left(\frac{N_n'}{N_n^{max}} + \frac{N_n'}{N_n^{min}}\right) = \left(\frac{1383.75}{8.4 \times 10^9} + \frac{1383.75}{1.0225 \times 10^{11}}\right) + \left(\frac{708.75}{8.99 \times 10^9} + \frac{708.75}{2.43 \times 10^{12}}\right) + \left(\frac{843.75}{1.505 \times 10^{10}} + \frac{843.75}{2.257 \times 10^{11}}\right) + \left(\frac{236.25}{2.808 \times 10^{10}} + \frac{236.25}{3.07 \times 10^{11}}\right) + \left(\frac{202.5}{1.739 \times 10^{10}} + \frac{202.5}{3.147 \times 10^{11}}\right)$$

$$= 1.647 \times 10^{-7} + 1.349 \times 10^{-8} + 7.88 \times 10^{-8} + 2.92 \times 10^{-10} + 5.606 \times 10^{-8} + 3.74 \times 10^{-9} + 8.41 \times 10^{-9} + 7.7 \times 10^{-10} + 1.164 \times 10^{-8} + 6.43 \times 10^{-10} = 3.385 \times 10^{-7}$$

$$\text{Life span} = \frac{1 \text{ day}}{3.385 \times 10^{-7} \text{ cycles}} \times \frac{1 \text{ yr}}{365 \text{ days}} = 8.095 \times 10^3 \text{ years}$$

Life span = 8095 years

Case b: Find life span at the welded end of cover plated beam.

Since the peak stress ranges previously calculated represent the peak stress at the centerline of the beams, it was necessary to find a factor to multiply the peak stress range at the centerline by to determine the nominal peak stress range at the coverplated ends. A carry over factor was determined by using the computer analysis of the westbound bridge developed within this study.

A loading (3S2 truck) was placed on the span centerline at approximately 2.0 feet intervals transversely across the bridge, and stresses were determined at

the centerline and the coverplate end. The carry over factor was determined by dividing the sum of the girder stresses at the centerline by the sum of the girder stresses at the coverplate end. Table A-1 shows the results of the calculations, and the conclusion was to use .49 as the carry over factor.

Summary of peak stress range at coverplate end:

Truck type	$S_{max_{\ell}}$ (ksi)	$S_{min_{\ell}}$ (ksi)	$S_{max_{f}}$ (ksi)	$S_{min_{f}}$ (ksi)	N (cycles/day)
3S2	2.38	1.083	1.19	0.531	2767.5
2D	2.33	0.400	1.17	0.196	1417.5
2S2	1.98	0.845	0.99	0.414	1687.5
2S1	1.63	0.767	0.82	0.376	472.5
3	1.89	0.761	0.95	0.373	405

Root Mean Square:

N_{act} = Actual Number of Cycles = 6750 cycles/day = 2,463,750 cycles/year

N_f = Number of cycles to failure

Show in Figure A-3 is the S-N diagram plot for tests on the welded end of cover-plated beams for A-36 steel (which is what the steel is in our example). The equation describing the plot is:

$$\text{Log } N_f = 8.975 - 2.877 \text{ Log } S_r$$

where N_f and S_r are the same variables described earlier.

$$\text{Log } N_f = 8.975 - 2.877 \text{ Log } S_{rms}$$

TABLE A-1

CARRY OVER FACTOR FOR

COVERPLATE

$\Sigma\sigma_{\ell}$ (ksi)	$\Sigma\sigma_{\ell}$ (ksi)	C.O. Factor
7.430	3.872	.521
7.724	3.883	.503
7.968	3.893	.486
8.173	3.902	.477
8.310	3.908	.470
8.420	3.913	.465
8.465	3.915	.462
8.479	3.916	.462
8.438	3.914	.464
8.350	3.910	.468
8.227	3.905	.475
8.037	3.896	.485
7.819	3.887	.497
7.528	3.876	.515
7.210	3.863	.536
8.039*	3.897*	.486*

*Average Value

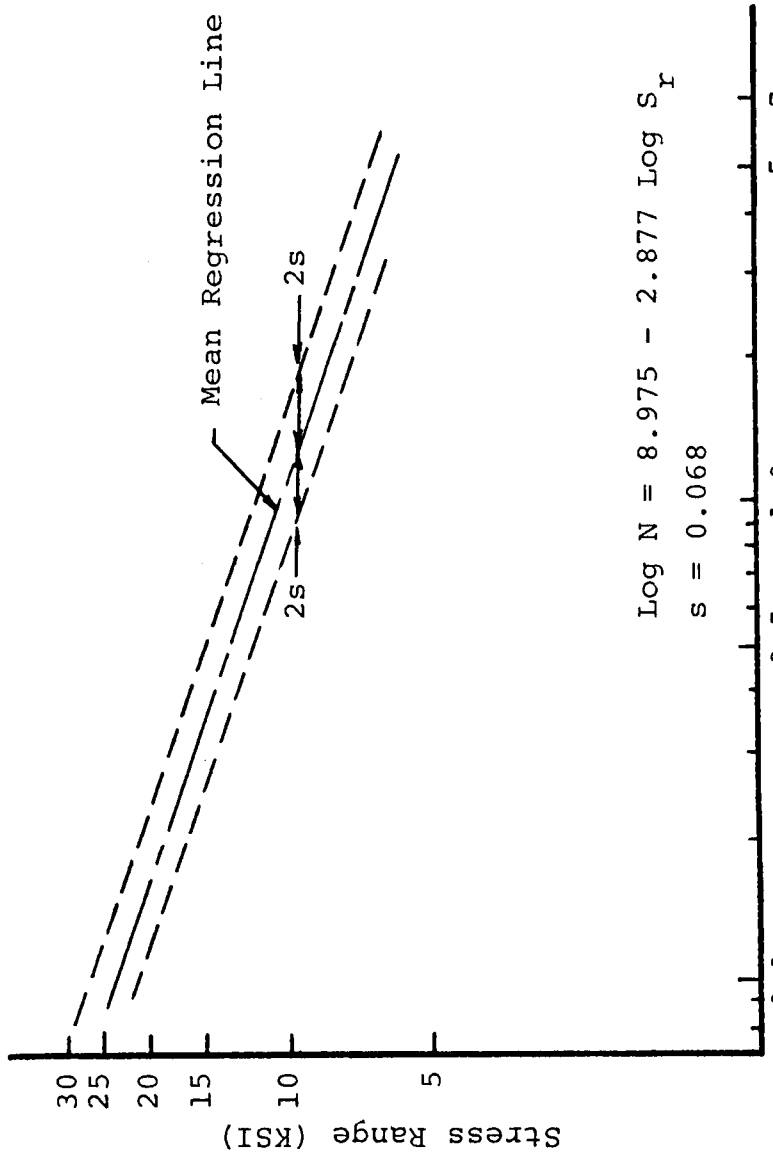


Figure A-3. Effect of Stress Range and Minimum Stress on the Cycle Life for the Welded End Cover - Plated Beams - A36 Steel.

where $S_{rms} = \sqrt{\frac{\Sigma\left(\frac{S^2_{max_n} + S^2_{min_n}}{2}\right) X_n}{\Sigma X_n}}$

$$= \Bigg(\Bigg(\left[\frac{(1.19)^2 + (0.531)^2}{2}\right] 2767.5 + \left[\frac{(1.17)^2 + (0.196)^2}{2}\right] 1417.5$$

$$+ \left[\frac{(0.99)^2 + (0.414)^2}{2}\right] 1687.5 + \left[\frac{(0.82)^2 + (0.376)^2}{2}\right] 472.5$$

$$+ \left[\frac{(0.95)^2 + (0.373)^2}{2}\right] 405\Bigg) \Big/ 6750\Bigg)^{1/2}$$

$$= ((2.3497 \times 10^3 + 9.974 \times 10^2 + 9.71575 \times 10^2 + 1.9225 \times 10^2$$

$$+ 2.1093 \times 10^2) / 6750)^{1/2}$$

$$\underline{S_{rms} = 0.83638 \text{ ksi}}$$

$$\text{Log } N_f = 8.975 - 2.877 \text{ Log } (0.83638)$$

$$N_f = 5.65 \times 10^8$$

$$\text{Life span} = \frac{N_f}{N_{act}} = \frac{5.65 \times 10^8 \text{ cycles}}{2.46375 \times 10^6 \text{ cycles/yr}} = 229 \text{ yrs}$$

Life span = 229 years

Miner's Law:

1) Truck type 3S2:

$$\text{Log } N^{max}_{3S2} = 8.975 - 2.877 \text{ Log } (1.19)$$

$$N^{max}_{3S2} = 5.723 \times 10^8$$

$$\text{Log } N^{min}_{3S2} = 8.975 - 2.877 \text{ Log } (0.531)$$

$$N^{min}_{3S2} = 5.833 \times 10^9$$

2) Truck type 2D:

$$\text{Log } N_{2D}^{max} = 8.975 - 2.877 \text{ Log } (1.17)$$

$$N_{2D}^{max} = 6.009 \times 10^{8}$$

$$\text{Log } N_{2D}^{min} = 8.975 - 2.877 \text{ Log } (0.196)$$

$$N_{2D}^{min} = 1.026 \times 10^{11}$$

3) Truck type 2S2:

$$\text{Log } N_{2S2}^{max} = 8.975 - 2.877 \text{ Log } (0.99)$$

$$N_{2S2}^{max} = 9.172 \times 10^{8}$$

$$\text{Log } N_{2S2}^{min} = 8.975 - 2.877 \text{ Log } (0.414)$$

$$N_{2S2}^{min} = 1.194 \times 10^{10}$$

4) Truck type 2S1:

$$\text{Log } N_{2S1}^{max} = 8.975 - 2.877 \text{ Log } (0.82)$$

$$N_{2S1}^{max} = 1.671 \times 10^{9}$$

$$\text{Log } N_{2S1}^{min} = 8.975 - 2.877 \text{ Log } (0.376)$$

$$N_{2S1}^{min} = 1.575 \times 10^{10}$$

5) Truck type 3:

$$\text{Log } N_{3}^{max} = 8.975 - 2.877 \text{ Log } (0.95)$$

$$N_{3}^{max} = 1.094 \times 10^{9}$$

$$\text{Log } N_{3}^{min} = 8.975 - 2.877 \text{ Log } (0.373)$$

$$N_{3}^{min} = 1.611 \times 10^{10}$$

$$\text{Life Span} = \frac{1}{\Sigma\left(\frac{N_n'}{N_n^{max}} + \frac{N_n'}{N_n^{min}}\right)}$$

where N_n' = actual number of cycles for truck type n divided by 2.0;

and N_n^{max}, N_n^{min} were defined earlier.

Summary for Miner's Law at coverplate end:

Truck type	N_n^{max}	N_n^{min}	N_n' (cycles/day)
3S2	5.723×10^8	5.833×10^9	1383.75
2D	6.009×10^8	1.026×10^{11}	708.75
2S2	9.172×10^8	1.194×10^{10}	843.75
2S1	1.671×10^9	1.575×10^{10}	236.25
3	1.094×10^9	1.611×10^{10}	202.50

$$\Sigma\left(\frac{N_n'}{N_n^{max}} + \frac{N_n'}{N_n^{min}}\right) = \left(\frac{1383.75}{5.723 \times 10^8} + \frac{1383.75}{5.833 \times 10^9}\right) + \left(\frac{708.75}{6.009 \times 10^8} + \frac{708.75}{1.026 \times 10^{11}}\right) + \left(\frac{843.75}{9.172 \times 10^8} + \frac{843.75}{1.194 \times 10^{10}}\right) + \left(\frac{236.25}{1.671 \times 10^9} + \frac{236.25}{1.575 \times 10^{10}}\right) + \left(\frac{202.5}{1.094 \times 10^9} + \frac{202.5}{1.611 \times 10^{10}}\right)$$

$$= 2.418 \times 10^{-6} + 2.372 \times 10^{-7} + 1.179 \times 10^{-6} + 6.905 \times 10^{-9} + 9.198 \times 10^{-7} + 7.067 \times 10^{-8} + 1.414 \times 10^{-7} + 1.50 \times 10^{-8} + 1.851 \times 10^{-7} + 1.257 \times 10^{-8} = 5.86 \times 10^{-6} \text{ cycles/day}$$

$$\text{Life span} = \frac{1 \text{ day}}{5.186 \times 10^{-6} \text{ cycles}} \times \frac{1 \text{ year}}{365 \text{ day}} = 528 \text{ years}$$

Life span = 528 years

Total Summary of Life Spans (years):

Method of Analysis	Centerline of Span	Welded Coverplate End
Root Mean Square	10,400	229
Miner's Law	8,095	528

In conclusion, it should be emphasized that the procedures shown within this appendix are not those which a designer would follow. The designer determines the number of cycles of maximum stress from Table 1.7.3(A) in the AASHTO specifications.[11] The allowable fatigue stress is determined from AASHTO specifications through equation A of section 1.7.3 or Figure 1.7.3A. The allowable fatigue stress is then incorporated into the design criteria.

VITA

Charles Raymond Bolt was born in Munich, Germany on August 25, 1952. He was declared a citizen of the United States at birth by virtue of his father being a member of the United States Air Force. The author began attending school in Sioux City, Iowa until his parents were transferred to Kansas City, Missouri. After attending school in Missouri for two years, the author moved with his parents overseas to the Far East island of Okinawa. He went to an American school in Okinawa for three years and returned to the United States with his parents. The remainder of the author's education was spent at schools in Waco, Texas, Tucson, Arizona, and Arlington, Virginia where he received his high school diploma in June 1970 at Wakefield High. The following August he entered Clemson University, Clemson, South Carolina. He received his Bachelor of Science degree in Civil Engineering on May 10, 1974 from Clemson University. He accepted a research assistantship at the Graduate School of The University of Tennessee where he began work towards his Master of Science Degree in September 1974. He received this degree in March 1976.