UTILIZATION OF STRESS HISTORY DATA IN BRIDGE DESIGN

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A method for checking the adequacy of steel stringer highway bridges for fatigue is presented. Truck types and weights are utilized with stress analyses to predict the fatigue life of bridges. The distributions of truck weights, axle weights and truck types were based on recent field measurements. A simplified method of establishing stress ranges due to typical trucks is summarized and an example is presented. However, any method may be used to obtain the stress ranges and the procedure outlined in the paper may be used. The method may be used in design or in checking existing bridges.

1. Background

The fatigue behavior of highway bridges has been the subject of numerous research projects in the past ten years (1,2,3,4,5,6,7,8,9). This increase in interest was caused in part by the failure of the Point Pleasant bridge in which forty-six persons were killed. As a result much has been learned about material behavior and traffic characteristics, but this knowledge requires time to be disseminated and to be converted into an easily utilized format. This paper presents a method of utilization of data generally known to the bridge designer and which enables him to be as sophisticated as desired in his analysis. It is presumed that the bridge design is dictated by factors other than fatigue and that the proposed method is used as a checking procedure to establish the expected life of a bridge.

The proposed method was developed as a part of a stress history research project conducted by the Department of Civil Engineering of The University of Tennessee and sponsored by the Bureau of Planning and Programming of the Tennessee Department of Transportation and the Federal Highway Administration. The contents of this paper reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of Tennessee or the Federal Highway Administration.

2. Fatigue Design

The four distinct phases of the design procedure are summarized below and described in detail in the remainder of the paper.

Step 1. Determine the traffic characteristics for the proposed bridge such as average daily traffic, type of truck mix, etc.

Step 2. Compute the range of stresses due to the loads of step 1. The analysis may be as simple or complex as desired. A simple method is described herein.

Step 3. A factor of safety must be established, probably in two parts. One part should account for the uncertainty in weight and volume increases during the life of the structure. The second part should reflect the uncertainties in the analysis method chosen or the variations in material properties.

Step 4. The expected life of the bridge is computed and compared to the design life. If the computed life is not sufficient then alterations in the design are in order.

3. Detailed Procedure

3.1 Truck Traffic Characteristics. The designer must establish the number of each type of truck using the bridge. This is accomplished by obtaining the average daily traffic (ADT) plus the percentage of trucks present or by obtaining the average daily truck traffic (ADTT). After the total number of trucks per day is known, the different types present and their percentage may be determined. Table I contains the usual nomenclature used to identify truck types. Reference 7 contains a table which presents the various truck type percentages based on whether the bridge is located in a metropolitan, urban or rural setting. Surveys of present traffic in the vicinity of the bridge and projections of future traffic may suffice to furnish the number of each type of truck to be expected. Overloads may be accounted for if the number is known or can be approximated.

3.2 Calculations of Stress Range. A simplified method is proposed herein to compute the stress ranges caused by the various types of truck traffic. The method was developed and compared to a STRUDL finite element analysis. Reference 10 describes the method in detail and presents an example for its use. The weight and axle spacing data shown in Table I are very similar to the average values from four other states and presented in Reference 7. Two stress ranges are determined for each truck type based on a graph constructed as described below. A 3S2 truck with:

(a) front axle weight = 8.75 kips (38.92 KN)
(b) front axle spacing = 12.75 feet (3.89 m)
(c) gross vehicle weight - 50.0 kips (222.4 KN)

is placed on the bridge to produce maximum moment at the bridge centerline for four cases. The four loading variations are shown in Figure 1. The bridge may be idealized as a wide beam in order to obtain a summation value for plotting. The four stress sums are plotted as shown in Figure 2. The slopes of the two lines (B and C) and the vertical axis intercept (A) must be determined. Also the rate of change in the slopes of change in the slopes (D) is computed by dividing the difference in the two slopes by 38.0. These values are used in the following equation.

$$\Sigma\sigma = \{A - [\, B - D\,(X - 31.0)\,]\; Y\,\}\,\frac{Z}{50.0} \qquad (1)$$

where:

X = percent of gross vehicle weight (GVW) on middle group for tractor-trailer trucks or the percent of GVW to rear axle group for single unit trucks,
Y = rear axle spacing in feet,
Z = gross vehicle weight,
$\Sigma\sigma$ = summation of stress ranges on all girders.

An impact factor to account for the dynamic nature of the loading could be introduced into the equation above if desired by the designer. A Monte Carlo simulation was used to obtain a typical truck type and weight distribution. Stresses computed from this procedure are plotted in Figure 3 as the simplified method and compare very well with the results of a STRUDL finite element analysis also shown on the figure. There is general agreement with the values obtained by field measurements reported in Reference 9.

3.3 Factor of Safety. The determination of the factor of safety is beyond the scope of this paper; however, a brief discussion of the subject is in order. It was suggested earlier that the factor of safety should be separated into two parts. The first part, used to account for growth in traffic volume and in truck weights, is very difficult to estimate. There has been a significant increase in the number of 3S2 trucks on the highways in the past several years. The gasoline shortage in 1974 led to a substantial decrease in traffic volume and, therefore, the trend to higher volumes of traffic each year has altered. The long term effects are even harder to define. Recently, increases in maximum truck weights have been sought both nationally and in Tennessee. It seems prudent to include a factor in the design of the bridge that reflects these possible increases. Reference 8 includes a factor of 1.5 to account for future truck weight and volume growth.

The second part of the factor of safety is for the uncertainty in material properties and methods of analysis. For steel structures designed on the basis of elastic behavior this factor is taken to be 1.8 (AASHTO). For fatigue this factor should be altered depending on the confidence limit of the allowable stress values for a given welded or bolted detail. If mean values are used in establishing the allowable stress range, then the safety factor should be higher than when the mean is reduced by two standard deviations as suggested in Reference 3.

3.4 Determination of Fatigue Life. There are a number of methods for determining the fatigue life of steel structures. Most design specifications utilize an allowable stress range based on laboratory fatigue test data. This method works very well if the design stress range occurs with great frequency relative to lower stress ranges. However, this is not the case for highway bridges where the loading varies from automobiles to trucks weighing in excess of 100,000 lbs. (444.8 KN). Recently a research project (NCHRP 12-12) was conducted to study the effects of variable cycle fatigue loadings on welded steel bridge members. The results of this research indicated that the vast amount of laboratory fatigue test data conducted at constant stress range magnitude could be used in estimating the life of structures loaded by variable stress ranges (11).

There are at least three methods for estimating fatigue life of a structure. Two of these methods, Miner's theory and root-mean-square method (RMS) are widely known, whereas the third method, root-mean-cube method (RMC), was recently proposed by Yamada and Albrecht in Reference 12. Miner's theory assumes that a fraction of the total fatigue life is expended with each loading cycle. If the stress ranges can be grouped together in a reasonable number of intervals, then the fractional life expended at each stress level may be computed by using the number of cycles to failure obtained from a S-N curve and the actual number of stress range cycles experienced. The usual procedure is to compute the sum of these fractional parts for a year. The reciprocal of this fraction is then the number of years to failure provided that the loading history for future years is the same as that assumed in the computation. The RMS and RMC methods allow the designer to compute one equivalent stress range that has the same effect as the variable stress ranges. The total number of variable cycles is used with the equivalent stress range to determine the amount of damage experienced based on the S-N curve of the particular steel and structural detail.

4. Example

The preceding steps may be clarified by the use of an example problem. Therefore, the fatigue life of a bridge used in the tests reported in Reference 9 will be presented.

The truck traffic at the bridge site is approximately 11% of the ADT based on data obtained during continuous sampling periods and the estimated 1974 ADT of 40,000 vehicles for the bridge.

The percentages of the various truck types are given below.

Type	Percentage
2D	32.4
3	7.8
2S1	2.2
2S2	10.2
3S2	45.6
Other	1.8

Based on the percentages above and 11% of 40,000 vehicles, the following number of trucks can be expected each day.

2D - 1426
3 - 343
2S1 - 97
2S2 - 449
3S2 - 2006
others - 79 (neglected)

Next the stress ranges caused by these trucks will be estimated. This step may be as simple or as precise as the designer wishes. A simple method that agrees with measured values for steel girder bridges with composite concrete decks was developed and is summarized herein.

Reference 13 gives details about the derivation of an equation for the summation of the girder stresses for the example bridge. For illustration a 3S2 truck is chosen with a front axle weight of 8.75 kips (38.9 KN), a front axle spacing of 12.75 feet (3.89 m), and a gross vehicle weight of 50.0 kips (222.4 KN). The weights on the tractor drive axles and on the trailer axles are distributed in a 31% to 69% ratio for two rear axle spacings of 14 feet (4.3 m) and 30 feet (9.1 m) as shown in Figure 1. These four loading arrangements were used to compute the maximum moment in the bridge and, therefore, the resulting average stress across the cross-section. The average stress was then multiplied by the number of girders present to obtain the summation of girder stresses. It was determined that GVW correlates very well with the sum of the girder stresses (13). These four stress values were plotted on the graph shown in Figure 2. The slopes of the two lines were determined and used to obtain the equation shown below.

$$\Sigma\sigma = \{8.333 - [0.14575 - (0.00138322)(X - 31)]\, Y\} = \frac{Z}{50} \quad (2)$$

where X, Y, and Z were defined in section 3.2.

Distributions of gross vehicle weights indicate that the majority of trucks are either heavily loaded or essentially empty with smaller percentage with weights between these two extremes (9). Therefore, one half of the trucks of a given type were considered to have a weight equal to the mean value less one standard deviation and the other half were considered to have a weight equal to the mean value plus one standard deviation as indicated in Table I. The weights of the various types of trucks are summarized in Table II. These weights were then used with the tabulated distribution percentages and axle spacings shown in Table I to obtain maximum and minimum loading configurations given in Table II as columns A and B, respectively. The sum of the girder stresses was then divided by the number of girders and multiplied by a lateral load distribution factor. The lateral load distribution factor may be obtained theoretically or the AASHTO factor may be used. In this example, there were seven girders and the lateral load distribution factor of 2.80 was obtained experimentally (9). These stress ranges were then used to estimate the fatigue life of the bridge using Miner's theory, RMS and RMC methods. The stresses shown in Table II were for the center of the span, but the critical locations for fatigue on this bridge were at the end of the cover plates. Experimental results (9) indicate that the stress range at the end of a cover plate was from 0.6 to 1.11 times that at the center of the span for this bridge. The computed stress ranges at the cover plate ends and the number of cycles to failure from S-N curves in Reference 11 are shown in Table III. This information and the number of trucks causing these stress ranges are sufficient for the calculation of the expected life in fatigue. If the present traffic volume and weight limits are assumed to continue for the lift of the bridge, then fatigue cracks at the cover plate end-welds would be expected to occur after 107, 113, or 82 years when predicted using Miner's theory, the RMS method or the RMC method, respectively. It is also assumed that all of these trucks are using the same lane which is very conservative.

Sample calculations illustrating each of the three methods are shown below.

(A) Miner's theory: damage in one year

$$365 \left\{ \frac{713}{7.33 \times 10^8} + \frac{713}{1.33 \times 10^{10}} + \frac{171.5}{8.39 \times 10^7} + \frac{171.5}{1.11 \times 10^9} + \frac{48.5}{7.32 \times 10^9} + \frac{48.5}{1.48 \times 10^9} + \frac{224.5}{1.39 \times 10^8} + \frac{224.5}{1.12 \times 10^9} + \frac{1003}{3.94 \times 10^7} + \frac{1003}{5.49 \times 10^8} \right\} = 0.00929$$

and the expected life is the reciprocal of the damage

$$\frac{1}{0.00929} = 107 \text{ years}$$

(B) Root-mean-square (RMS)

$$\left\{ \frac{[713 (0.979)^2 + 713 (0.358)^2 + 171.5 (2.08)^2 + 171.5 (0.847)^2 + 48.5 (0.98)^2 + 48.5 (0.768)^2 + 224.5 (1.745)^2 + 224.5 (0.845)^2 + 1003 (2.705)^2 + 1003 (1.083)^2]}{4321} \right\}^{\frac{1}{2}}$$

$$= 1.601 \text{ ksi}$$

$$\text{Log } N = 8.839 - 2.877 \text{ Log } (1.601)$$

$$N = 178{,}255{,}294 \text{ cycles to failure}$$

$$\text{life} = \frac{N}{4321 \times 365} = 113 \text{ years}$$

(C) Root-mean-cube (RMC)

$$\left\{ \frac{[713\ (0.979)^3 + 713\ (0.358)^3 + 171.5\ (2.08)^3 + 171.5\ (0.847)^3 + 48.5\ (0.98)^3 + 48.5\ (0.768)^3 + 224.5\ (1.745)^3 + 224.5\ (0.845)^3 + 1003\ (2.705)^3 + 1003\ (1.083)^3]}{4321} \right\}^{1/3}$$

$$= 1.792 \text{ ksi}$$

$$\text{Log } N = 8.839 - 2.877 \text{ Log } (1.792)$$

$$N = 128{,}840{,}409 \text{ cycles to failure}$$

$$\text{life} = \frac{N}{4321 \times 365} = 82 \text{ years}$$

It might be more realistic to expect that the traffic volume may increase and the number of trucks using the bridge would also increase. If a 4% growth rate is assumed, the the expected life decreases to approximately 40 years. Increases in the allowable loads on trucks would also decrease this life. An increase of 10% of present truck weights would result in an expected life of approximately 60 years even if there were no increase in truck traffic volume.

5. Conclusions

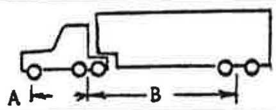
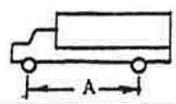
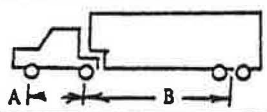
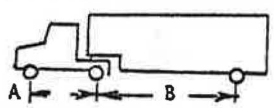
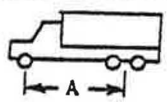
This paper described a method of fatigue design that incorporates significant data obtained from field measurements of truck weights and girder stresses in highway bridges. The method allows the designer with the flexibility of obtaining girder stresses with as much accuracy as deemed necessary.

References

1. Cudney, G. R., "Stress Histories of Highway Bridges", Journal of the Structural Division, ASCE, Vol. 94, No. ST12, December 1968.
2. Fisher, J. W., K. H. Frank, M. A. Hirt, and B. M. McNamee, "Effect of Weldments on the Fatigue Strength of Steel Beams", NCHRP Report 102, Highway Research Board, 1970.
3. Goodpasture, D. W., "Final Report on Stress History of Highway Bridges", University of Tennessee, December 31, 1972.
4. Walker, W. H. and J. A. Ruhl, "Investigation of Dynamic Stresses in Highway Bridges - An Interim Report", Civil Engineering Studies Structural Research Series No. 404, University of Illinois at Urbana - Champaign, October 1973.
5. Bowers, D. G., "Loading History, Span No. 10, Yellow Mill Pond Bridge I-95, Bridgeport, Connecticut", Department of Transportation, State of Connecticut, Research project HRP 175-332, May 1972.
6. Fisher, J. W., P. A. Albrecht, B. T. Yen, D. J. Klingerman, and B. M. McNamee, "Fatigue Strength of Steel Beams with Welded Stiffness and Attachments, NCHRP Report 147, Transportation Research Board, 1974.
7. Heins, C. P. Jr., and C. F. Galambos, "Bridge Fatigue due to Daily Traffic", Transportation Research Board Record No. 507, 1974.
8. Moses, Fred and Robert Garson, "New Procedure for Fatigue Desing of Highway Bridge Girders", Transportation Research Board Record No. 507, 1974.
9. Goodpasture, D. W., "Final Report on Stress History of Highway Bridges - II", University of Tennessee, June 30, 1976.
10. Bolt, Charles R., "A Simplified Approach for Determining Stresses on Highway Bridges", Master's thesis, The University of Tennessee, Knoxville, March 1976.
11. "Fatigue of Welded Steel Bridge Members Under Variable - Amplitude Loadings", Research Results Digest 60, National Cooperative Highway Research Program, April 1974.
12. Yamada, K. and P. Albrecht, "Fatigue Design of Welded Bridge Details for Service Stresses", A paper presented at the 54th Annual Transportation Board Meeting, January 1975.
13. Patrick, Perry C., Jr., "Comparison of Measured and Theoretical Stresses in a Simple Span Bridge", Master's thesis, The University of Tennessee, Knoxville, June 1975.

TABLE I

CHARACTERISTICS OF EACH TRUCK TYPE

Truck Type	Configuration	Axles Spacing and Standard Deviation (feet)	Gross Weight and Standard Deviation (Kips)	Present Weight to Middle Axle and Standard Deviaiton
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 ± 2.5	35.2 ± 10.6 Fix front axle weight = 8.0	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

1 kip = 4.448 KN   1 ft = 0 .3048 meter

TABLE II

TRUCK LOADINGS AND RESULTING STRESS RANGES

Truck Type	Z Weight (kips)		Y Rear Axle Spacing (ft.)		X Weight Distribution (percent)		Girder Stress (ksi)	
	A	B	A	B	A	B	A	B
2D	20.4	8.6	12.75	17.25	43.5	30.5	1.093	0.400
3	36.3	17.3	14.85	19.65	44.0	32.0	1.869	0.761
2S1	36.4	23.0	26.0	33.0	60.1	45.1	1.628	0.768
2S2	45.8	24.6	26.0	31.0	55.0	42.2	1.981	0.845
3S2	67.0	33.2	27.45	34.05	59.2	46.0	2.896	1.083

1 kip = 4.448KN

1 ft = 0.3048M

A = maximum loading configuration

B = minimum loading configuration

TABLE III

FATIGUE CHARACTERISTICS FOR EXAMPLE BRIDGE

Truck Type	Stress at Coverplate end (ksi) A	Stress at Coverplate end (ksi) B	Number of Cycles to Failure Log N = 8.839 - 2.877 Log Sr. A	Number of Cycles to Failure Log N = 8.839 - 2.877 Log Sr. B
2D	0.979	0.358	7.33×10^{8}	1.33×10^{10}
3	2.080	0.847	8.39×10^{7}	1.11×10^{9}
2S1	0.980	0.768	7.32×10^{8}	1.48×10^{9}
2S2	1.745	0.845	1.39×10^{8}	1.12×10^{9}
3S2	2.705	1.083	3.94×10^{7}	5.49×10^{8}

1 ksi = 6900 $KN/_{M}2$

A = maximum loading configuration

B = minimum loading configuration

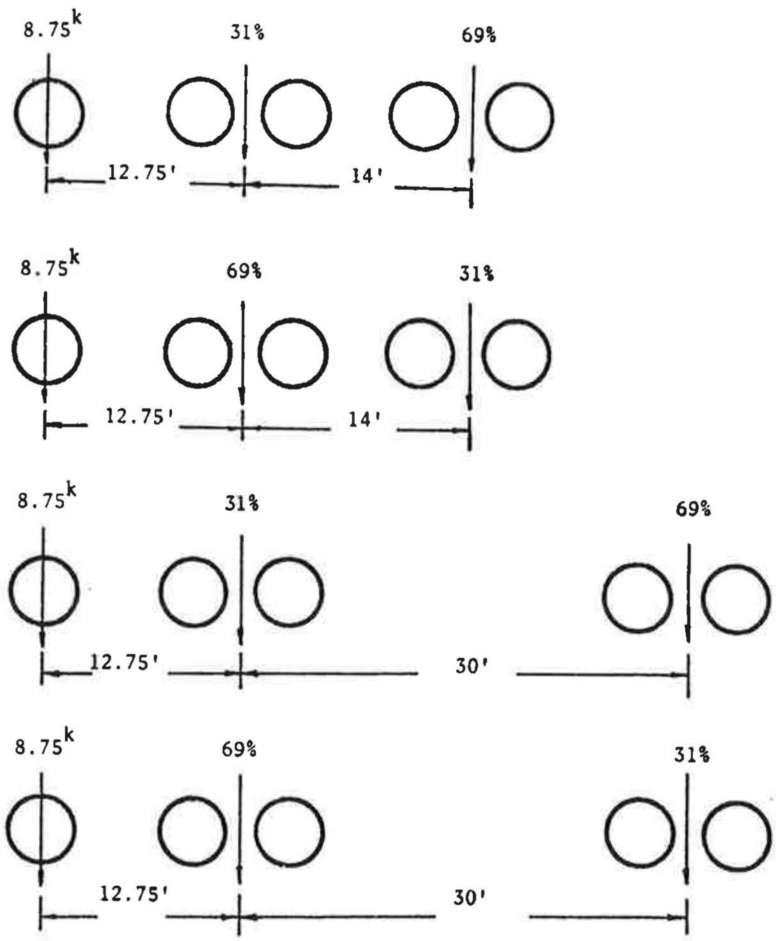


Figure 1 3S2 Truck Loading Variation

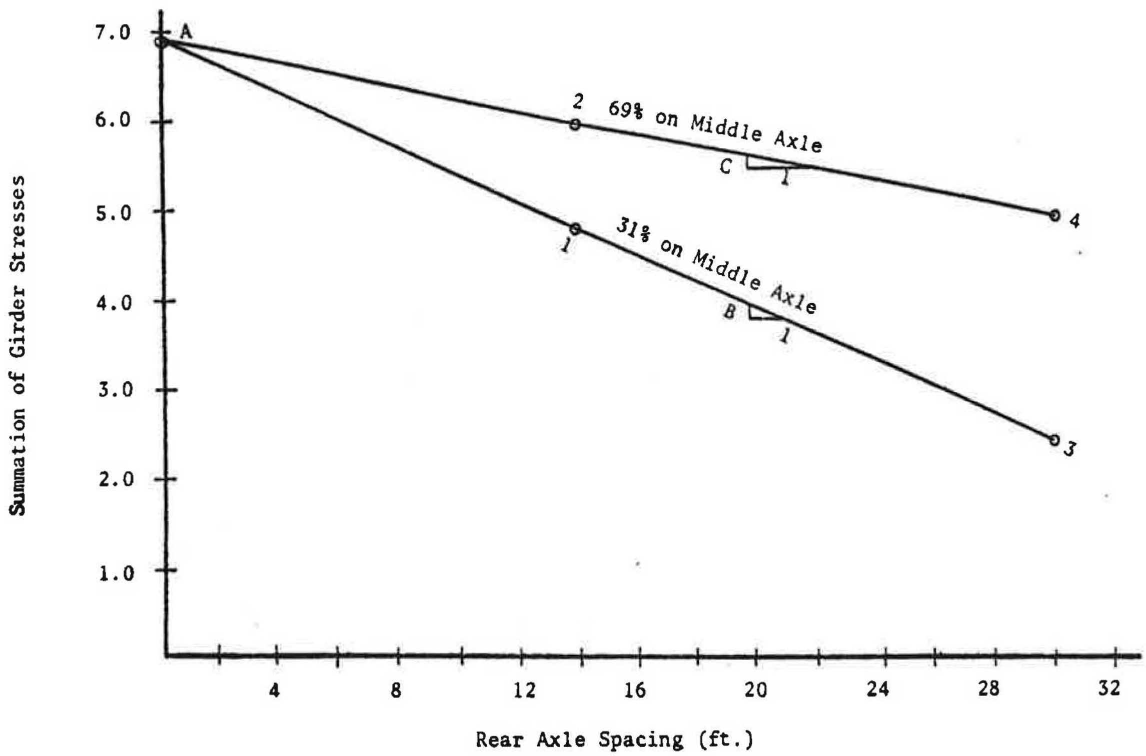


Figure 2 Summation of Stresses Versus Rear Axle Spacing

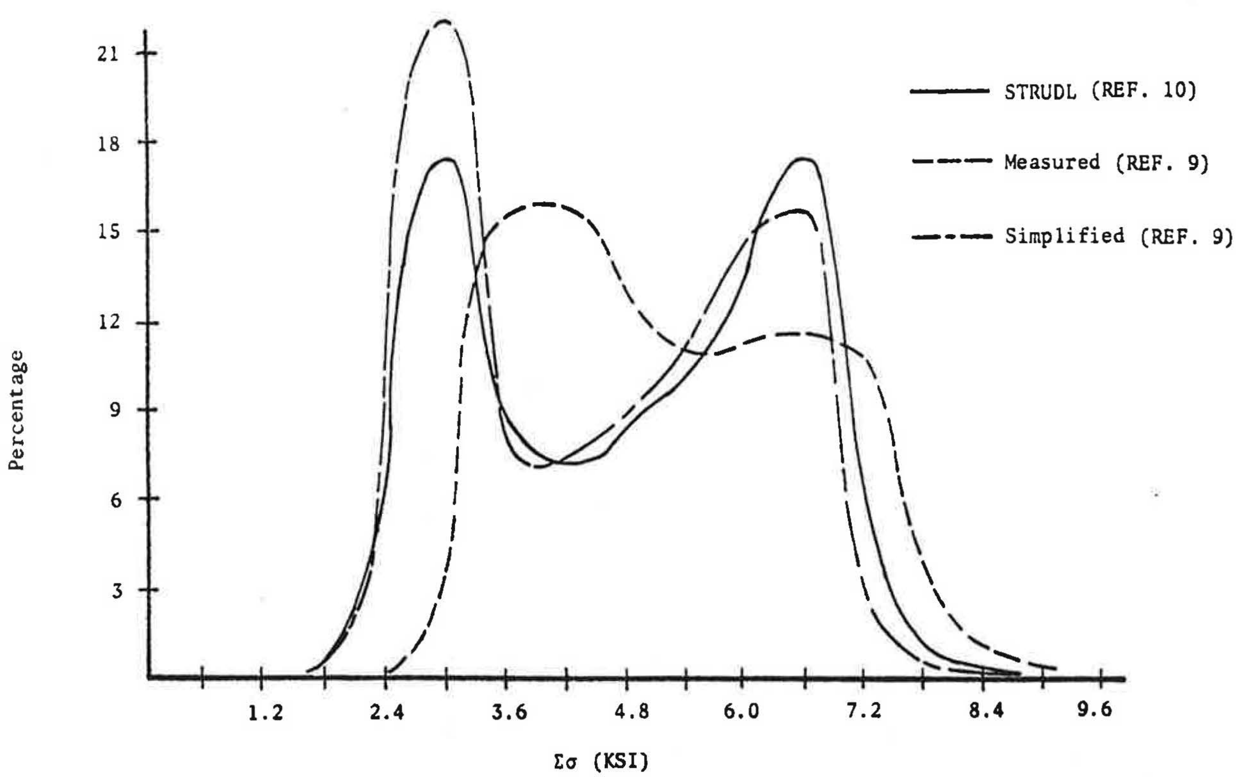


Figure 3 Comparison of Measured, Theoretical and Simplified Summation of Stresses - 3S2 Trucks.