CIVIL ENGINEERING STUDIES

STRUCTURAL RESEARCH SERIES NO. 508

Illinois Cooperative Highway and Transportation Series No. 204

UILU-ENG-83-2012

ISSN: 0069-4274

MATHEMATICAL MODELING OF LOADING HISTORIES FOR STEEL BEAM OR GIRDER HIGHWAY BRIDGES

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A Report of the Investigation on
Steel Bridge Design Criteria to Help
Minimize the Probability of Fracture
Project IHR-304
Illinois Cooperative Highway and
Transportation Research Program

A COOPERATIVE INVESTIGATION
Conducted by
THE STRUCTURAL RESEARCH LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
ENGINEERING EXPERIMENT STATION
UNIVERSITY OF ILLINOIS
AT URBANA-CHAMPAIGN

in cooperation with the
STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION
and the
U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

UNIVERSITY OF ILLINOIS
AT URBANA-CHAMPAIGN
URBANA, ILLINOIS
MARCH 1983

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No. FHWA/IL/UI-204	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Mathematical Modeling of Loading Histories for Steel Beams or Girder Highway Bridges		5. Report Date March 1983
		6. Performing Organization Code
7. Author(s) Douglas Miller and W. H. Munse		8. Performing Organization Report No. UILU-ENG-83-2012
9. Performing Organization Name and Address Department of Civil Engineering, Engineering Experiment Station, University of Illinois at Urbana-Champaign		10. Work Unit No.
		11. Contract or Grant No. IHR-304
12. Sponsoring Agency Name and Address Illinois Department of Transportation, 126 E. Ash Street, Springfield, Illinois 62706		13. Type of Report and Period Covered Interim Report
		14. Sponsoring Agency Code

15. Supplementary Notes

Study was conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration.

16. Abstract

In this study the nature of actual random traffic stress histories in steel beam or girder highway bridges has been investigated. Quantitative field test results have been used to develop mathematical models that can be used to represent the stress histories for such bridges.

The beta distribution function has been found to be an effective mathematical model for the actual stress-range histograms and has been used to establish random stress factors and can be used in fatigue design to account for the random nature of loadings in highway bridges.

17. Key Words Loading histories, fatigue, mathematical models, highway bridges, fatigue design, distribution functions	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	54	

Form DOT F 1700.7 (8-69)

Acknowledgements

This study was conducted as a part of the Illinois Cooperative Highway and Transportation Research Program, Project IHR-304, "Steel Bridge Design Criteria to Help Minimize the Probability of Fracture," by the Department of Civil Engineering, Engineering Experiment Station, University of Illinois at Urbana-Champaign, in cooperation with the Illinois Department of Transportation, Federal Highway Administration.

The research reported herein was conducted by Douglas Miller, Research Assistant in Civil Engineering, working under the direction of W. H. Munse, Professor of Civil Engineering at the University of Illinois. Appreciation is extended to the technicians, draftsmen, and secretaries who assisted in the preparation of this report.

The authors also wish to acknowledge the assistance of the IHR-304 Project Advisory Committee in the development and evaluation of the programs and the resulting report, and of D. R. Schwartz and R. Taylor of the Bureau of Materials and Physical Research of the Illinois Department of Transportation. This committee consisted of R. M. Kiel, J. C. Stuemke and J. J. Wavering of the Illinois Department of Transportation, of R. A. Scarr and F. Grabski of the Federal Highway Administration, and Professors F. V. Lawrence and J. E. Stallmeyer of the University of Illinois.

Finally, it should be noted that the opinions, findings and conclusions expressed herein are those of the authors and not necessarily those of the Illinois Division of Highways, the Federal Highway Administration, or the University of Illinois. This report does not constitute a standard, specification, or regulation.

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1.0 INTRODUCTION

1.1 Description of Problem

1.1.1 Highway Bridge Loadings

Highway bridges by their very nature of supporting moving traffic loads are subject to cycles of stress in large numbers and fatigue becomes a concern. Both intuitively and on the basis of many in-field bridge tests where strain measurements under traffic loading have been recorded, it is clear that the stress cycles are of variable amplitude. Indeed, highway loadings vary greatly from highly overloaded trucks (either illegally or by special permit) to relatively light automobiles whose effect upon fatigue life may be small or essentially nonexistent.

It is unfortunate, in view of the random nature of highway loads, that most laboratory studies of the fatigue resistance of structural details have focused on constant amplitude tests. Fatigue behavior under variable amplitude stress cycles has only recently received increased attention.

While the field is not yet thoroughly understood, the Miner (18) linear damage hypothesis has been generally accepted as a reasonable working rule for random loadings such as occur in highway bridge fatigue. This allows known random loading distributions to be dealt with using fatigue information from constant amplitude tests.

We are also left with another problem in bridge fatigue analysis. While it is clear that highway bridge loadings may be variable or random, it is not clear what the proportions of the loading distribution or the resulting stress-frequency distribution are to which the bridge members are subjected. To employ the Miner approach, or any other approach, the relative frequencies of the various magnitudes of stress cycles need to be known or approximated as realistically as possible. That is to say, the bridge members' predicted stress histories are needed. Also, the overall volume of traffic that determines the total number of cycles in the bridge's expected life needs to be known to complete the loading description. Thus, any study of the fatigue behavior of highway bridges requires investigation into the nature of the variable traffic stress histories to which bridges are subjected.

It is known, even before such a study is started, that there will be some degree of variety in bridge stress histories. For this reason, a better understanding of bridge loadings should make possible a refinement in the AASHTO Bridge Specification (2) fatigue provisions which base allowable fatigue stress on number of cycles expected and the detail geometry with only limited provisions for the influence of the variations in loading a bridge receives (2).

1.1.2 Stress Histories

A number of studies have been made in recent years in which bridges have been instrumented with strain gages to provide measurements of live load stresses. Figure 1 is an example of a strain gage output for a single truck passage from one of these studies (12). The stress range in the figure is the total range from lowest trough to highest peak for one truck passage. This "stress range" definition of a stress event is the most commonly used in the literature. During the life of a bridge, the truck passages can be expected to produce many stress events which can be plotted as a stress-frequency histogram, an example of which is found in Fig. 2. Each bridge member has its own histogram, and even for different parts of the same bridge the histrograms may differ both in overall magnitude and in shape. However, it is the shape of the histogram that is of primary importance. In designing a member the overall stress magnitude can be changed by the choice of member size, but the proportions of the stress history generally will not be changed. What needs to be done, therefore, is to establish the histogram shapes that are realistic for the stress histories of highway bridges.

1.2 Object and Scope of Investigation

In this study, the nature of actual random traffic stress histories in steel highway beam or girder bridges has been investigated. Quantitative field test results have been used in developing mathematical models that can be used to represent the stress histories for such highway bridges. A qualitative understanding of some of the factors affecting these stress histories has been sought also.

The study has not dealt with the subject of the fatigue resistance of the bridges or details since a great deal has already been written about that aspect (1,9,13,14,15). The intent has been to model the loading function aspects of the fatigue problem in such a way that the function can readily be used in available fatigue reliability theory. In particular, the general theory presented in references 1, 9 and 19 has been utilized.

The study is intended to cover welded steel highway beam or girder bridges with a definite slant toward bridges of the short to medium span, slab-stringer type, found typically on interstate highways. Most of the bridges tested have been of this type and are the bridges that are of most concern with respect to fatigue.

2.0 LOADING HISTORY TESTS

The experimental data that was used in this study comes from loading history investigations on a number of bridges in Illinois, Ohio, Maryland and Connecticut, and one bridge in Brazil. Numerous stress histograms taken from these bridges have been examined in the study. The spans range (with one exception) from 34 to 129 feet, some being simple and some continuous. The bridge locations include urban, suburban, and rural areas. Table 1 briefly describes the test bridges considered.

2.1 Review of Data

A summary of the histogram data is found in Table 2 with the mean μ, standard deviation σ and maximum value of stress range S_o being used to characterize the histogram. The normalized mean, μ', is

$$\mu' = \frac{\mu}{S_o}$$

and the coefficient of variation, C.O.V., is given also.

$$C.O.V. = \frac{\sigma}{\mu}$$

These parameters are especially useful since they characterize the shape of a histogram. The normalized mean, μ', shows where the bulk of the stress events occurred, and the C.O.V. indicates the degree to which these stress magnitudes are spread. It was observed that μ' ranged between 0.13 and 0.51, while the C.O.V. ranged from 0.24 to 0.99. Very few C.O.V.'s were found to be greater than 0.60.

3.0 MATHEMATICAL MODELING OF LOADING HISTORIES

3.1 General

Information about the nature of the distribution of stress cycle magnitudes is available from the many stress-range histograms generated in the field tests mentioned above. Modeling this information methematically, for the purposes of this study, consisted of finding a function whose shape can act essentially as a mathematical replacement for the shapes of the observed stress range histograms. The ordinate of the function then represents the probability density for the occurrence of a particular stress range.

The general mathematical function or functions chosen should represent the cyclic loading conditions in bridges and be suitable to represent the various shapes of histograms that have been recorded or can be expected. The function can then be used to provide simple expressions for various fatigue design parameters. Using shapes derived from one or more basic functions and distinguished from one another by shape parameters makes it possible to produce functions applicable to a variety of situations and conditions.

3.2 Models and Critical Features

In choosing the basic functions it is necessary to develop criteria by which to judge a possible choice. These criteria have been developed directly from a consideration of the role that the model has in the fatigue analysis and design and from a consideration of interfacing this loading information with procedures for fatigue reliability analysis.

First of all, it is known that the proportions of the stress history are important. Therefore, when plotted, the proportions of the mathematical function and the histogram modeled should be very similar. This can be judged visually or by statistical measures of fit.

In the context of the reliability theory of reference 1, the main role that the stress function plays is to provide a base on which a realistic determination of an equivalent constant amplitude stress range S_c can be made that would produce the same fatigue damage as the distribution of expected stress amplitudes. At a given life, S_c should accurately reproduce the damage expected from histogram by the mathematical model. To establish S_c, the Miner hypothesis is used. (18)

$$\sum \text{Damage} = \sum \left(\frac{n_s}{C/S^m} \right) = \frac{\sum (n_s \cdot S^m)}{C} = \frac{n_{total} \cdot S_c^m}{C}$$

$$S_c^m = \frac{\sum (n_s \cdot S^m)}{n_{total}} = E(S^m) \quad \text{or} \quad [E(S^m)]^{1/m} = S_c$$

where S = a stress range magnitude

n_s = the number of cycles at stress range S

n_{total} = the total number of stress cycles

m = exponent of S-N relationship ($\frac{1}{k}$, where k is the negative slope of the straight-line S-N curve)

C = intercept of the S-N diagram at $S=1$

Therefore, accurate reproduction of S_c in the mathematical model requires that $E(S^m)$ for the mathematical model match $E(S^m)$ for the measured histogram. This of course depends on m, the slope exponent of the S-N relationship, which can vary from 3 to about 12, depending on the geometry of a given detail.

Another feature that the model must provide for, to be applicable for practical design, is an upper limiting value of S which can be taken to be an "allowable fatigue stress range." This enables a design procedure to be developed in which checking for fatigue simply involves comparing a design live load stress range to this maximum value. Given the existence of S_o, a random stress factor (1,9,19), ξ can be defined

$$S_o = \xi \cdot S_c \quad \text{or} \quad \xi = \frac{S_o}{S_c}$$

so that, given a magnitude of allowable constant amplitude stress, the maximum allowable stress range, S_o, may be found using ξ.

The above criteria provide a means to judge how well a given function models a specific histogram. The criteria can be applied to a number of differently shaped histograms and mathematical models in order to choose the best model or models for all types of histograms. Additionally, it must be remembered that any mathematical model used can represent a number of shape functions so that in practice the objective will be to describe a stress history for which the function was not specifically fitted.

3.3 Modeling Functions

Probability distribution functions that can be used to model random traffic stresses include the Rayleigh distribution, the beta distribution, the log-normal distribution, the Weibull distribution, the exponential distribution, a shifted exponential distribution, and a variation on the Beta distribution that has been called the double beta distribution. Some of the basic properties of these distributions are given in Table 2 and the general shapes are shown in Fig. 3. Expressions for $E(S^m)$ and ξ for the various distributions are given in Table 3 (19).

The beta distribution is a versatile function, with finite lower and upper bounds, that may be skewed in one direction or the other, depending on the relative values of the shape parameters q and r (see Fig. 3(a)). This distribution has been used in fatigue analyses by Ang and Munse (1) to model the random loadings in highway bridges subjected to heavy truck loadings. Howevever, the beta distribution, with its upper bound limitation, does not appear to be as compatible with the loading history data at the higher loads as one would like. Consequently, a variety of other distributions have been considered.

The lognormal distribution is a lower bounded non-negative probability distribution with a "tail" that trails off to the right (Fig. 3(b)). The two-parameter Weibull distribution, like the lognormal, is a lower bounded non-negative distribution with a tail to the right (Fig. 3(c)). However, the Weibull distribution can take on many different shapes depending on the shape parameter, k. The exponential and Rayleigh distributions (Figs. 3(d) and 3(e)) are special cases of the Weibull distribution, where k=1 and k=2, respectively.

The shifted exponential distribution is simply an exponential distribution which starts at a non-zero value (Fig. 3(f)). This distribution can be considered as a special case of the exponential distribution. However, the introduction of the non-zero lower bound implies that the lowest possible stress variation is a non-zero value which is not physically true and tends to complicate the analysis.

A double beta distribution is nothing more than two ordinary beta distributions with ordinates added. The double beta has been considered due to a concern that in cases where histograms had two distinct humps, ordinary modeling distributions underestimated the frequency of stresses corresponding to the second hump. Fitting a double beta distribution involves splitting a histogram into a left group and a right group and fitting beta distributions to each, using in both cases the maximum value of the histogram. The ordinates of the two distributions are then weighted (by factors equal to the respective fractions of total occurrences for that side) and added.

3.4 Choice of Function

Initial consideration of the type of function best suited to model stress histories was made by fitting each of several functions to a given histogram. For the Rayleigh distribution the mean was matched, and for the two beta distributions and the log-normal distribution both means and standard deviations were matched. For the beta distributions the maximum value of stress was also matched. This was done for a number of histograms with fairly consistent results.

The fittings to one particular histogram is shown in Fig. 4. By visual inspection of the four fits, not much more can be said other than that all four functions can be made to fit the histrogram in a rather general way, but none will fit it exactly.

Plotting the deviations of $E(S^m)$ of the mathematical models, from $E(S^m)$ of the histogram is probably more informative. This plot is shown in Fig. 5.

The beta and the double beta distributions model $E(S^m)$ within 10 percent when m is less than 6 to 7, but on the unconservative side. The log-normal models $E(S^m)$ well, but only when m is less than about 3. The Rayleigh distribution models $E(S^m)$ much too high, mostly due to its having a coefficient of variation* fixed at 0.52 while the histogram's C.O.V. is only 0.38.

These results and similar results for other histograms that have been fitted indicate the following observations.

First, the double beta function, for reasonably shaped histograms does not provide enough additional accuracy to justify its considerable additional complexity. The log-normal does not model $E(S^m)$ as well as the beta distribution and has the disadvantage of requiring truncation so it was not considered further. The beta distribution's only problem is that while it models $E(S^m)$ fairly accurately, its error tends to be on the unconservative side, particularly at the higher stress levels. The Rayleigh distribution models $E(S^m)$ much too high, but this is a conservative modeling, predicting fatigue damage greater than it should. The Rayleigh's conservativeness combined with its simplicity may explain its widespread use. Also, it can be argued that when one is in the position of the designer, not knowing the histogram shape in advance, then the extent or degree to which a distribution fits a known histogram may not be too important. However, assuming that one has some knowledge of the general shape of stress distribution expected in a given situation, then the additional versatility of the beta distribution is of considerable value.

The question of the degree of improvement in modeling that can be realistically expected in practice by using a more versatile function like the beta distribution as compared to the simpler Rayleigh distribution was explored by

* The coefficient of variation, C.O.V., is equal to the ratio of the standard deviation to the mean of the histogram.

testing the modeling performance of these two functions as they relate to two other bridge loading histories.

First, span #10 of the Yellow Mill Pond bridge on Interstate 95 in Connencticut (3) was modeled for stresses off the cover plate in beam 4 and then the E, J and E bridge on Interstate 80 in Illinois (17) was modeled for midspan stresses in beam 4. Taking the position of a designer having some idea of what distribution to expect but not possessing a measured histogram, we selected a normalized mean stress $\mu' = 0.30$, a coefficient of variation C.O.V. = 0.35 for the Yellow Mill Pond bridge, and $\mu' = 0.37$ and C.O.V. = 0.45 for the E, J and E bridge as parameters for the mathematical models. In reality the parameters of the actual histograms differ somewhat with $\mu' = 0.29$, C.O.V. = 0.32 for the Yellow Mill Pond bridge, and $\mu' = 0.34$, C.O.V. = 0.45 for E, J and E Bridge. The μ' and C.O.V. have been chosen on the conservative side.

The histograms and modeling functions are shown in Fig. 6 and an evaluation of the modeling in Fig. 7. The results in Fig. 7 are shown as deviations of the random stress factor ξ instead of $E(S^m)$. Zero deviation of ξ is equivalent to zero deviation of $E(S^m)$, but while $E(S^m)$ tends to vary widely and is very difficult to match closely for large m, ξ has been used to illustrate the quality of modeling over the range of m.

The results illustrate that the beta distribution does model the distribution of stresses somewhat more closely than the Rayleigh distribution does. Notice that the choice of modeling curves has indeed resulted in conservative modeling for both the beta distribution and the Rayleigh distribution. The conservative result was of course produced intentionally by choosing a model more severe than the actual histogram. Making conservative assumptions for distribution function shape is what seems to be necessary in general to get safe modeling. The degree of conservativeness appropriate will depend upon the accuracy with which the loading history shape can be predicted.

The results of this last test indicate that, of the distributions studied, the beta distribution is probably the best model of the loading histories for use in design. It can be used conservatively, but is not as over-conservative as the Rayleigh distribution. Additional advantage in use of the beta disbtribution is the increasing importance of its versatility as more information is assembled and allows improved ability to predict the distribution function shape. Also, the fact that the beta disbtribution possess an upper limit and a greater range in shapes allows a cleaner development of its mathematic properties for deisgn than does a truncated curve such as the Rayleigh disbtribution.

Another distribution function that should be given further consideration in future studies is the Weibull distribution. This distribution function has been found to provide a very effective mathematical model for ship design (19).

3.5 Proposed Model - Mean Stress and C.O.V.

Assuming that the beta distribution is chosen to model stress histories, both a non-dimensional mean stress and a C.O.V. can be specified for each of a set of distributions. Nine beta distributions have been chosen collectively to model the stress histories examined in this study. The nine shapes are shown in Figs. 8 and 9, and random stress factors for these distributions and for a range of values of m are given in Table 4.

These shapes were arrived at by first grouping the histogram data of Table 5 into groups according to their value of μ' and then by their value of C.O.V. In order to prevent unconservative modeling, the upper value of μ' was chosen to model the histograms of each group. Thus $\mu' = 0.3$ was used for histograms with $\mu' < 0.3$, $\mu' = 0.37$ for those with $0.30 \leq \mu' \leq 0.37$, $\mu' = 0.45$ for those with $0.37 \leq \mu' \leq 0.45$, and $\mu' = .60$ for those with $\mu' > 0.45$. Next, the range of values of C.O.V. in each group was noted and model C.O.V.'s in each group were chosen to represent the group. The histogram data as used to choose the beta distribution shapes is shown in Table 6.

The beta distribution curves were thus chosen to reflect the recorded stress histories in the study. Should improved knowledge indicate that different combinations of μ' and C.O.V. would better model the loading on bridges, revised beta distributions could be chosen using the best available values of μ' and C.O.V. and the following formulas. The beta distribution shape factors r and q are given by,

$$r = \frac{(1 - \mu')^2}{\mu'(C.O.V.)^2} - (1 - \mu')$$

$$q = \left(\frac{\mu'}{1 - \mu'}\right) r$$

The random stress factors are then determined from,

$$\xi = \left[\frac{\Gamma(q)\ \Gamma(m + q)}{\Gamma(q)\ \Gamma(m + q + r)}\right]^{\frac{1}{m}}$$

A summary of random stress factors for various values of μ', C.O.V., and m are given in Table 7. Interpolations in this table could also be used.

In order to establish an appropriate value for the random stress factor for a bridge member, it is necessary to establish the following:

(a) The stresses to which a member can be expected to be subjected during its lifetime.

(b) The variability (C.O.V.) that can be expected in this loading ((a) and (b) establish the necessary histogram).

(c) The number of cycles of truck loading expected during the lifetime of the bridge member.

(d) The fatigue resistance of the member in question. (The slope of the S-N curve, $\frac{1}{m}$, and the equation for the S-N curve.) See reference (19) for example.

With the above information it is easily possible to evaluate the fatigue adequacy of any bridge member, to establish allowable fatigue design stresses for bridge members, or to rate the expected life of bridge members, or remaining life of bridge members that have been in service.

A simple example can be used to show the application of the general procedure presented. First, assume the design provides the following:

Bridge span = 115 ft.

Predicted loading conditions

$N = 10^7$ cycles in 50 yrs. (548 ADTT)

S_o = 12.0 ksi., maximum stress range in random loading history; at point in question

μ' = 0.6, mean stress-range ratio in loading history (a high density of heavily loaded trucks - on the high side of values in study).

C.O.V. = 0.4, variability expected in loading. (mean of values in Fig. 10)

m = 4, slope of S-N curve for detail in question.

ξ = 1.4, the random stress factor (from Table 7).

From the above, the following can be determined.

$S_c = \frac{S_o}{\xi}$ = 8.57 ksi., the equivalent constant stress range for the stress history with S_o = 12.0 ksi.

Then, if the allowable fatigue design stress (the mean fatigue resistance for the detail divided by a reliability factor (19) or factor of safety) is greater than 8.57 ksi., the detail is adequate; if it is lower than 8.57 ksi., the detail is not adequate and the design should be changed. Thus, the random loading, a <u>very important</u> fatigue factor, can be taken into account simply and directly in the design.

3.6 Factors Influencing Stress Histories

In considering the principal factors that influence stress histories, only beam and girder bridges of the type studied in this investigation are considered. For longer span major bridges, such as long span box girder bridges and truss bridges, more study is necessary to define the necessary values of μ' and C.O.V.

In general, for bridges of the type included in this study, the non-dimensional mean stress, μ', will be largely a function of span length, and of the type and distribution of truck loadings to which the bridge will be subjected. For short spans that are subjected to a high density of heavily loaded trucks, the value of μ' would be high. For short spans subjected to infrequent and only lightly loaded trucks, the value of μ' would be relatively low. For the longer span bridges that are subjected to a high density of heavily loaded trucks, the value of μ' would be relatively high. For longer spans that are subjected to infrequent and only lightly loaded trucks, the value of μ' would be low. Thus, the normalized mean stress can be expected to vary considerably depending upon the particular member in question, the type of bridge, and the type of traffic.

Also of importance is the C.O.V. Again, an examination of the data for the bridges studied (simple and continuous spans ranging from 34 ft. to 113 ft.), indicates that a considerable variation in the C.O.V. can exist for any given span length (see Fig. 10). However, it is evident that (a) the C.O.V. tends to decrease with an increase in span length and (b) tends to be lower for a continuous span than for a simple span. Both of these observations are logical and should be expected.

For the range in values of μ', C.O.V. and m shown in Table 4 for the bridges studied, the values of random stress factor are seen to vary from 2.99 to 1.27. Thus, the random stress factor indicates that, because the loadings are not all at the maximum stress level, the fatigue damage at a given number of cycles of loading is not as great as the damage that would be caused if all stress cycles were at the maximum. The random stress factor (1) then provides a means whereby this effect can be taken into account in design.

The factors determining the values of μ' and C.O.V. break clearly into two groups: the first group pertains to the traffic on the bridge and the second group to the structural configuration of the bridge and the members which distribute the loads to various portions of the structure.

The influence of the type of traffic is clearly dominated by the frequency distribution of gross vehicle weights of trucks crossing the bridge (16). Loadometer studies have been made in a number of states to gather information locally on this question. Other important factors are the relative frequency of the various axle configurations, the frequency of multiple truck loadings, the possibility of more than one cycle of stress from one truck, and dynamic effects which in turn depend on truck speed and road roughness. In addition to these factors, the difference in frequency of lane use by trucks causes differences between the loading histories of the beams of a multi-beam bridge (3).

Among the structural influences on the loading history, the following are important: (a) the type of member, (b) the span length, (c) the lateral distribution system, (d) the continuity in spans, and (e) the skewness of span. The type of member being considered is important with respect to whether the member is a main spanning member or a secondary member. This is due to the fact that a main member is generally affected by the overall load on the bridge while members such as floor beams are generally affected mainly by local wheel loads. The span length is important in the sense that very short spans are affected most by axle loads, medium spans by individual vehicles and the primary members of very long spans get relatively few cycles of load since they are affected by long steady trains of load which may cycle only a few times a day (13). In beam-slab bridges the degree of lateral sharing of the load between beams is important in determining the degree to which histograms for the different beams differ. This degree of lateral distribution is determined by the beam spacing, the size and spacing of lateral framing and the stiffness of the slab. Other important structural factors include whether or not a bridge is continuous over its supports, whether or not its deck slab has been designed to act compositely, and the degree to which the bridge is skewed.

3.7 Number of Lifetime Cycles

One of the factors noted above that is necessary in order to complete a description of the fatigue loading environment of a bridge (item c) is the number of cycles the bridge will be subjected to in its life. Although this information is as important as the proportions of the loading distribution, it is not the main emphasis of this investigation. However, in the interest of bringing a sense of scale to the problem, Table 8 has been prepared. For many bridge members, one cycle per truck would apply. However, for very short spans or for floor beams, every truck axle may produce a cycle; then, 2 or 3 cycles per truck would be more appropriate.

4.0 Summary and Conclusions

The nature of cyclic stress conditions in a number of actual highway beam and girder bridges has been investigated. The beta distribution function has been found to be an effective mathematical model for actual stress range histograms, and a set of nine beta distributions has been determined for use in fatigue design of such bridges. Which of these distributions should be used in any particular situation depends on traffic characteristics and the structural configuration of the bridge. Additional investigations need to be conducted to better define the individual distribution functions for bridges with the characteristics that produce them. Then, it will be possible to readily evaluate the fatigue adequacy of bridge members subjected to random loadings, to establish allowable fatigue design stresses, or to rate the life expectancy of bridges.

5. Nomenclature

μ' = The normalized mean value of stress range.

μ = The mean value of stress range.

S_0 = The maximum value of stress range.

σ = The standard deviation of stress range.

C.O.V. = The coefficient of variation for the stress range.

n_s = The number of cycles at stress range S.

m = The slope exponent of the S-N relationship ($=\frac{1}{K}$, where K is the negative slope of the straignt-line S-N curve).

C = Intercept of the S-N diagram at S=1.

S = The stress range magnitude.

n_{total} = The total number of stress cycles.

S_c = Equivalent constant amplitude stress range.

$E(S^m)$ = The m^{th} moment of S or expected value of S^m.

ξ	=	Random stress factor.
K	=	Negative slope of a constant-life straight line S-N curve, or 1/m.
$F_s(s)$	=	Probability density function of S.
$s_{10^{-8}}$	=	The value of S at which the probability of exceedance is 10^{-8}.
Γ	=	The gamma function.
q	=	Beta distribution shape parameter(See Table 2).
r	=	Beta distribution shape parameter (See Table 2).
λ	=	Lognormal distribution parameter (See Table 2).
ζ	=	Lognormal distribution parameter (See Table 2).
k	=	The Weibull scale or shape parameter (See Table 2).
w	=	The characteristic life, or Weibull distribution parameter (See Table 2).
λ_e	=	The mean value of experimental and shifted experimental distributions (See Table 2).
S_{RMS}	=	Root-mean-square value of S (See Table 2).
a	=	The lower limit value of shifted exponential distribution (See Table 2).
s	=	Stress parameter (tension, compression, bending, etc.).
L	=	$a/S_{10^{-8}}$. Shifted exponential distribution parameters (See Table 3).
ε_s	=	Coefficient of variation of S.

6.0 References

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Table 1. Brief Description of Test Bridges

Bridge Name (Ref.)	Type Structure	Span Lengths	Setting	Highway Type
Shaffer Creek (10)	Steel Stringer	43'-43'	Suburban, Illinois	Interstate
Camp Creek (17)	Steel Stringer	34'	Rural, Illinois	Interstate
E, J & E (17)	Steel Stringer	61'-73'-61'	Suburban, Illinois	Interstate
C, B & Q (10)	Steel Stringer	50'-76'-50'	Rural, Illinois	Interstate
Cuy 271-721 (8)	Steel Stringer	41'-66'-41'	Urban, Ohio	Interstate
Cuy 71-384 (8)	Steel Stringer	37'-46'-37'	Suburban, Ohio	Interstate
Cuy 71-100 (8)	Steel Stringer	44'-63'-44'	Suburban, Ohio	Interstate
Atb 90-145D (8)	Steel Stringer	49'-81'-49'	Rural, Ohio	Interstate
Por 805-130 (8)	Steel Stringer	48'-60'-48'	Suburban, Ohio	Interstate
Cuy 90-2181 (8)	Steel Stringer	75'	Urban, Ohio	Interstate
Sum 21-778 (8)	Steel Stringer	36'-45'-36'	Suburban, Ohio	Divided Highway
Por 14-1131 (8)	Steel Stringer	48'-60'-60'-48'	Suburban, Ohio	2 lane
Yellow Mill Pond Westbound (3)	Steel Stringer	113'	Urban, Connecticut	Interstate
Yellow Mill Pond Eastbound (3)	Steel Stringer	113'	Urban, Connecticut	Interstate
U.S. 301 at Western Branch (11)	Steel Stringer	42'	Suburban, Maryland	Divided Highway
U.S. 301 - Marlboro (12)	Steel Stringer	42'-52'-42'	Suburban, Maryland	Divided Highway
Md 301 at Md 5 (6)	Steel Stringer	76'	Suburban, Maryland	Divided Highway
U.S. 1 at I495 (6)	Steel Stringer	38'	Suburban, Maryland	Capitol Beltway
I-83-N (6)	Steel Stringer	47'	Rural, Maryland	Interstate
I-83-S (6)	Steel Stringer	47'	Rural, Maryland	Interstate
Rio-Niteroi(4):	Box girders with an Orthotropic Steel Deck	200m-300m-200m	Major crossing in Brazil	

Table 2. Properties of Probability Distributions (R^2).

Distribution	Probability Density Function	Characteristic Parameters	Mean Stress, μ_S	Standard Deviation, σ_S
Beta	$f_S^B(s) = \frac{\Gamma(q+r)}{\Gamma(q)\Gamma(r)} \frac{S^{q-1}(S_o-S)^{r-1}}{S_o^{q+r-1}}$ $0 \le S \le S_o$	q, r, S_o	$\frac{qS_o}{q+r}$	$\frac{S_o}{(q+r)} \sqrt{\frac{qr}{q+r+1}}$
Lognormal	$f_S^L(s) = \frac{1}{\sqrt{2\pi}\,\zeta s} \exp\left[-\frac{1}{2}\left(\frac{\ln S-\lambda}{\zeta}\right)^2\right]$ $S \ge 0$	λ, ζ	$\exp(\lambda+\frac{1}{2}\zeta^2)$	$\sqrt{e^{\zeta^2}-1}\ \exp(\lambda+\frac{1}{2}\zeta^2)$
Weibull	$f_S^W(s) = \frac{k}{w}\left(\frac{s}{w}\right)^{k-1} \exp\left[-\left(\frac{s}{w}\right)^k\right]$ $S \ge 0$	k, w	$w\Gamma(1+\frac{1}{k})$	$w\left[\Gamma(1+\frac{2}{k}) - \Gamma^2(1+\frac{1}{k})\right]^{\frac{1}{2}}$
Exponential	$f_S^E(s) = \frac{1}{\lambda_e} \exp\left[-\left(\frac{s}{\lambda}\right)\right]$ $S \ge 0$	λ_e	λ_e	λ_e
Rayleigh	$f_S^R(s) = \frac{2}{S_{RMS}}\left(\frac{s}{S_{RMS}}\right) \exp\left[-\left(\frac{s}{S_{RMS}}\right)^2\right]$ $S \ge 0$	S_{RMS}	$\frac{\sqrt{\pi}}{2} S_{RMS}$	$\sqrt{1-\frac{\pi}{4}}\ S_{RMS}$
Shifted Exponential	$f_S^{SE}(s) = \frac{1}{\lambda_e} \exp\left[-\left(\frac{s-a}{\lambda}\right)\right]$ $S \ge a$	λ_e, a	$\lambda_e + a$	λ_e

Table 3. Expressions for $E(S^m)$ and ξ for Probability Distributions in Terms of S_o or $S_{10^{-8}}$ Stress-Range.

Distribution	$E(S^m)$ (Characteristic Parameters)	$E(S^m)$ (In Terms of Stress Range)	Maximum Stress-Range*	Random Stress Factor, ξ
Beta	$S_o^m\left[\frac{\Gamma(m+q)\Gamma(q+r)}{\Gamma(q)\Gamma(m+q+r)}\right]$	$S_o^m\left[\frac{\Gamma(m+q)\Gamma(q+r)}{\Gamma(q)\Gamma(m+q+r)}\right]$	S_o	$\left[\frac{\Gamma(q)\Gamma(m+q+r)}{\Gamma(m+q)\Gamma(q+r)}\right]^{1/m}$
Lognormal	$\mu_s^m[1+\delta_s^2]^{\frac{1}{2}m(m-1)}$ where $\delta_s = \frac{\sigma_s}{\mu_s}$	$S_{10^{-8}}^m(1+\delta_s^2)^{m^2/2}\exp[-5.6m\sqrt{\ln(1+\delta_s^2)}]$	$S_{10^{-8}}$	$(1+\delta_s^2)^{-m/2}[\exp(5.60\sqrt{\ln(1+\delta_s^2)})]$
Weibull	$w^m\Gamma(1+\frac{m}{k})$	$S_{10^{-8}}^m(18.42)^{-m/k}\Gamma(1+\frac{m}{k})$	$S_{10^{-8}}$	$(18.42)^{1/k}[\Gamma(1+\frac{m}{k})]^{-1/m}$
Exponential	$\lambda_e^m\Gamma(1+m)$	$S_{10^{-8}}^m(18.42)^{-m}\Gamma(1+m)$	$S_{10^{-8}}$	$18.42[\Gamma(1+m)]^{-1/m}$
Rayleigh	$S_{RMS}^m\Gamma(1+\frac{m}{2})$	$S_{10^{-8}}^m(18.42)^{-m/2}\Gamma(1+\frac{m}{2})$	$S_{10^{-8}}$	$\sqrt{18.42}[\Gamma(1+\frac{m}{2})]^{-1/m}$
Shifted Exponential	$\sum_{n=0}^{m}\frac{m!}{(m-n)!}\lambda_e^n a^{m-n}$	$S_{10^{-8}}^m[\sum_{n=0}^{m}\frac{m!}{(m-n)!}(18.42)^{-n}(1-\alpha)^n\alpha^{m-n}]$ where $\alpha = a/S_{10^{-8}}$	$S_{10^{-8}}$	$[\sum_{n=0}^{m}\frac{m!}{(m-n)!}(18.42)^{-n}(1-\alpha)^n\alpha^{m-n}]^{-1/m}$

*The Beta distribution is a "limited" distribution with S_o being the maximum stress range of the distribution for any specified life. For all other distributions shown, $S_{10^{-8}}$ is the maximum stress range that is expected only once in 10^8 cycles of loading. For any other life N in which the maximum stress range is expected only once, the values for Random Stress Factor must be multiplied by $\frac{(\ln N)^m}{(18.42)^m}$.

Table 4. Values of the Random Stress Factor for Nine Beta Distributions Representing Loading Histories Studied

	m			
	3	5	7	9
μ' = 0.30, C.O.V. = 0.70	2.39	2.02	1.82	1.70
μ' = 0.30, C.O.V. = 0.50	2.73	2.40	2.19	2.04
μ' = 0.30, C.O.V. = 0.35	2.99	2.75	2.57	2.42
μ' = 0.37, C.O.V. = 0.60	2.09	1.82	1.67	1.57
μ' = 0.37, C.O.V. = 0.45	2.29	2.06	1.91	1.79
μ' = 0.37, C.O.V. = 0.35	2.43	2.24	2.10	2.00
μ' = 0.45, C.O.V. = 0.55	1.78	1.59	1.49	1.41
μ' = 0.45, C.O.V. = 0.40	1.95	1.78	1.68	1.60
μ' = 0.60, C.O.V. = 0.40	1.47	1.37	1.31	1.27
Constant Cycle	1.00	1.00	1.00	1.00

Table 5. Summary of Histogram Data Representing Loading Histories Studied

Bridge and Gage	HISTOGRAM DATA, ksi μ	HISTOGRAM DATA, ksi σ	HISTOGRAM DATA, ksi $S_{R_{max}}$	NORMALIZED DATA μ'	NORMALIZED DATA C.O.V.	FITTED PARAMETERS r	FITTED PARAMETERS q
<u>Shaffer Creek</u> <u>1968</u> (10)							
Beam 2, Sec. A	1.331	.650	6.24	.21	.52	11.59	3.08
Beam 3, Sec. A	2.300	1.247	6.24	.37	.54	3.05	1.79
Beam 4, Sec. A	3.207	1.641	7.11	.45	.51	2.03	1.66
Beam 5, Sec. A	3.740	1.93	8.80	.42	.51	2.38	1.75
<u>1969</u>							
Beam 3, Sec. A	2.137	.951	6.67	.32	.45	6.46	3.04
Beam 4, Sec. A	2.87	1.308	7.98	.36	.46	4.74	2.66
Beam 5, Sec. A	3.410	1.482	8.12	.43	.43	3.674	2.660
<u>Camp Creek</u> (17)							
Gage 124	3.304	1.908	11.31	.29	.58	4.46	1.82
Gage 123	2.793	1.680	11.60	.24	.60	5.93	1.87
Gage 122	1.149	.920	8.85	.13	.80	8.23	1.23
<u>E, J & E (I)</u> (17)							
Gage 221	1.089	.433	4.79	.23	.40	15.34	4.58
Gage 222	1.632	.738	4.79	.34	.45	5.67	2.92
Gage 223	2.081	.965	5.51	.38	.46	4.16	2.55
Gage 224	1.711	.770	5.08	.34	.45	5.67	2.92
Gage 225	1.078	.486	4.79	.23	.45	11.96	3.57
Gage 533	1.759	.802	5.08	.35	.46	5.05	2.72
<u>E, J & E (II)</u> (17)							
Gage 221	1.194	.524	6.53	.18	.44	18.48	4.06
Gage 222	1.605	.667	6.53	.25	.42	12.01	4.00
Gage 223	1.863	.767	6.24	.30	.41	9.02	3.86

Table 5. (Continued)

	HISTOGRAM DATA			NORMALIZED DATA		FITTED PARAMETERS	
Bridge and Gage	μ	σ	$S_{R_{max}}$	μ'	C.O.V.	r	q
C, B & Q (10)							
Bottom flg, beam 2, Sec. A	1.740	.603	4.8	.36	.35	8.69	4.94
Bottom flg, beam 2, Sec. B	2.216	.806	5.4	.41	.36	5.81	4.04
Bottom flg, beam 3, Sec. C	1.398	.513	3.2	.44	.37	4.82	3.74
Top flg, beam 2, Sec. A	.658	.249	1.3	.51	.38	2.86	2.93
Bottom flg, beam 3, Sec. B	1.879	.699	5.08	.37	.37	7.21	4.23
Bottom flg, beam 4, Sec. B	.740	.386	4.79	.15	.52	16.96	2.99
Bottom flg, beam 1, Sec. A	1.494	.539	4.93	.30	.36	11.90	5.10
Bottom flg, beam 3, Sec. A	1.407	.479	4.64	.30	.34	13.43	5.76
Cuy 271-721 (8)							
Gage 8	1.118	.439	5.0	.22	.39	16.68	4.81
Cuy 71-384 (8)							
Gage 1	1.321	.669	5.0	.26	.51	7.25	2.60
Gage 4	1.699	1.065	6.0	.28	.63	3.90	1.54
Cuy 71-100 (8)							
Gage 1	1.607	.913	5.0	.32	.57	3.76	1.78
Gage 4	1.481	.826	4.5	.33	.56	3.72	1.83
Gage 2	1.346	.702	4.5	.30	.52	5.34	2.28
Gage 8 (Diaphram)	1.420	.761	5.0	.28	.54	5.57	2.21
Atb 90-145D (8)							
Gage 4	1.376	.603	3.5	.39	.44	4.27	2.76

Table 5. (Continued)

	HISTOGRAM DATA			NORMALIZED DATA		FITTED PARAMETERS	
Bridge and Gage	μ	σ	$S_{R_{max}}$	μ'	C.O.V.	r	q
Por 805-130 (8)							
Gage 1	1.313	.599	4.5	.29	.46	7.55	3.11
Gage 4	1.270	.576	4.0	.32	.45	6.46	3.00
Gage 8 (diaphram)	1.156	.431	3.5	.33	.37	9.09	4.48
Cuy 90-2181 (8)							
diaphram	1.734	1.081	7.0	.25	.62	5.13	1.69
Sum 21-778 (8)							
Gage 4	1.480	.788	4.0	.37	.53	3.15	1.85
Por 14-1131 (8)	1.499	.891	6.0	.25	.59	5.63	1.88
Yellow Mill Pond Bridge (3)							
Eastbound midspan, Beam 2	1.397	.548	4.0	.35	.39	7.24	3.88
Beam 3	1.480	.610	4.0	.37	.41	5.69	3.34
Beam 4	1.422	.535	4.0	.36	.38	7.62	4.20
Beam 5	1.447	.570	4.3	.34	.39	7.76	3.94
Beam 6	1.449	.549	4.3	.34	.38	8.41	4.27
Eastbound off cover plate Beam 2	.988	.249	2.2	.38	.25	10.09	8.22
Beam 3	1.237	.449	3.4	.36	.36	7.79	4.46
Beam 4	1.058	.277	2.6	.41	.26	11.97	8.21
Beam 5	1.073	.322	2.6	.41	.30	8.67	6.09
Beam 6	.993	.255	3.1	.32	.26	21.20	9.99

Table 5. (Continued)

Bridge and Gage	HISTOGRAM DATA			NORMALIZED DATA		FITTED PARAMETERS	
	μ	σ	$S_{R_{max}}$	μ'	C.O.V.	r	q
<u>Yellow Mill Pond Bridge</u> (3) (Con't)							
Westbound midspan, Beam 1	1.739	.865	7.5	.23	.50	9.52	2.87
Beam 2	1.561	.777	6.0	.26	.50	7.75	2.72
Beam 3	1.428	.625	4.2	.34	.44	6.02	3.10
Beam 4	1.306	.500	4.2	.31	.38	9.73	4.39
Westbound off cover plate Beam 2	1.239	.449	3.8	.33	.36	9.91	4.80
Beam 3	1.079	.316	3.8	.28	.29	20.31	8.05
Beam 4	1.120	.358	3.8	.29	.32	15.83	6.62
Beam 5	1.006	.243	2.7	.37	.24	17.49	10.39
Westbound off secondary cover plate Beam 1	1.428	.655	5.6	.26	.46	9.60	3.29
Beam 3	1.362	.509	3.3	.41	.37	5.40	3.79
diaphram	1.740	.835	6.0	.29	.48	6.84	2.79
<u>U.S. 301 at Western Branch</u> (11)							
off cover plate	1.344	.764	4.4	.31	.57	4.20	1.85
on cover plate	.925	.456	4.0	.23	.49	9.97	2.98
Center Line girders	1.641	.760	5.8	.28	.46	7.75	3.06
<u>U.S. 301 - Marlboro</u> (12)							
Section 1	1.897	.844	5.8	.32	.45	6.31	3.07
Section 2	1.587	.748	5.5	.29	.47	7.16	2.9
Section 3	1.124	.636	3.4	.33	.57	3.09	1.67

Table 5. (Continued)

Bridge and Gage	HISTOGRAM DATA, ksi μ	σ	$S_{R_{max}}$	NORMALIZED DATA μ'	C.O.V.	FITTED PARAMETERS r	q
Maryland 301 at Maryland 5 (6)							
off cover plate	1.024	.422	2.6	.39	.41	4.89	3.18
on cover plate	.686	.277	1.7	.40	.40	3.47	2.32
Center Line girders	.982	.381	2.6	.38	.39	6.20	3.76
U.S. 1 at I-495 (6)							
Center Line girders	1.361	.989	6.0	.23	.73	4.22	1.24
I-83-N Center Line Girders (6)	.947	.379	2.2	.43	.40	4.12	3.12
I-83-S (6)							
off cover plate	.789	.344	2.0	.39	.44	4.30	2.80
on cover plate	.345	.126	0.9	.37	.38	6.80	3.99
Rio-Niterio (4)							
transverse, stress in deck	.838	.829	4.79	.17	.99	3.30	0.68

Table 6. Histogram Data, Grouped by μ'

μ'	C.O.V.	μ'	C.O.V.	μ'	C.O.V.
.13	.80	.30	.36	.37	.41
.15	.52	.30	.41	.37	.53
.17	.99	.30	.52	.37	.54
.18	.44	.31	.38	.38	.25
.21	.52	.31	.57	.38	.39
.22	.39	.32	.26	.38	.46
.23	.40	.32	.45	.39	.41
.23	.45	.32	.45	.39	.44
.23	.49	.32	.45	.39	.44
.23	.50	.32	.57	.40	.40
.23	.73	.33	.36	.41	.26
.24	.60	.33	.37	.41	.30
.25	.42	.33	.56	.41	.36
.25	.59	.33	.57	.41	.37
.25	.62	.34	.38	.42	.51
.26	.46	.34	.39	.43	.40
.26	.50	.34	.44	.43	.43
.26	.51	.34	.45	.44	.37
.28	.29	.34	.45	.45	.51
.28	.46	.35	.39	.51	.38
.28	.54	.35	.46		
.28	.63	.36	.35		
.29	.32	.36	.36		
.29	.46	.36	.38		
.29	.47	.36	.46		
.29	.48	.37	.24		
.29	.58	.37	.37		
.30	.34	.37	.38		

Table 7. Values of Random Stress Factors for Various Values of μ', C.O.V. and m.

μ'	C.O.V.	m					
		2	3	4	5	7	9
0.20	0.30	4.79	4.60	4.44	4.29	4.03	3.81
	0.50	4.47	4.08	3.78	3.54	3.17	2.91
	0.60	4.29	3.81	3.47	3.21	2.83	2.58
	0.80	3.90	3.31	2.93	2.67	2.32	2.11
0.30	0.30	3.19	3.07	2.96	2.87	2.71	2.58
	0.50	2.98	2.73	2.55	2.40	2.19	2.04
	0.60	2.86	2.56	2.36	2.20	1.99	1.85
	0.80	2.61	2.24	2.03	1.88	1.69	1.57
0.40	0.30	2.39	2.30	2.23	2.16	2.05	1.96
	0.50	2.24	2.06	1.93	1.84	1.70	1.61
	0.60	2.14	1.94	1.80	1.70	1.57	1.48
	0.80	1.95	1.71	1.58	1.49	1.38	1.31
0.50	0.30	1.92	1.85	1.79	1.74	1.66	1.60
	0.50	1.79	1.66	1.57	1.51	1.42	1.36
	0.60	1.72	1.57	1.47	1.36	1.33	1.27
	0.80	1.56	1.40	1.31	1.26	1.20	1.16
0.60	0.30	1.60	1.54	1.46	1.43	1.38	1.35
	0.50	1.49	1.40	1.33	1.29	1.23	1.20
	0.60	1.43	1.32	1.26	1.22	1.17	1.14
	0.80	1.31	1.20	1.15	1.12	1.08	1.06

Table 8. Total Number of Lifetime Cycles (in millions)

		Cycles in Millions							
Average Daily Truck Traffic	Life in Years	20		30		40		50	
	Cycles/Truck	1	3	1	3	1	3	1	3
50		.37	1.1	.54	1.6	.73	2.2	.91	2.7
250		1.8	5.5	2.7	8.2	3.7	11.0	4.6	14.0
1000		7.3	22.0	11.0	33.0	15.0	44.0	18.0	55.0
2000		15.0	44.0	22.0	66.0	29.0	88.0	37.0	110.0
5000		37	130	73	200	91	260	110	330
10000		73	220	110	330	150	440	180	550
20000		150	440	220	660	290	880	370	1100

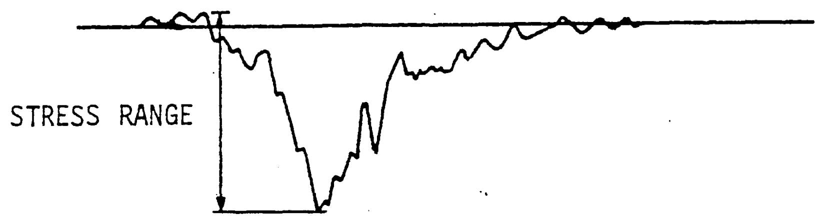


Figure 1. Typical Strain Gauge Output
From One Truck Passage (12)

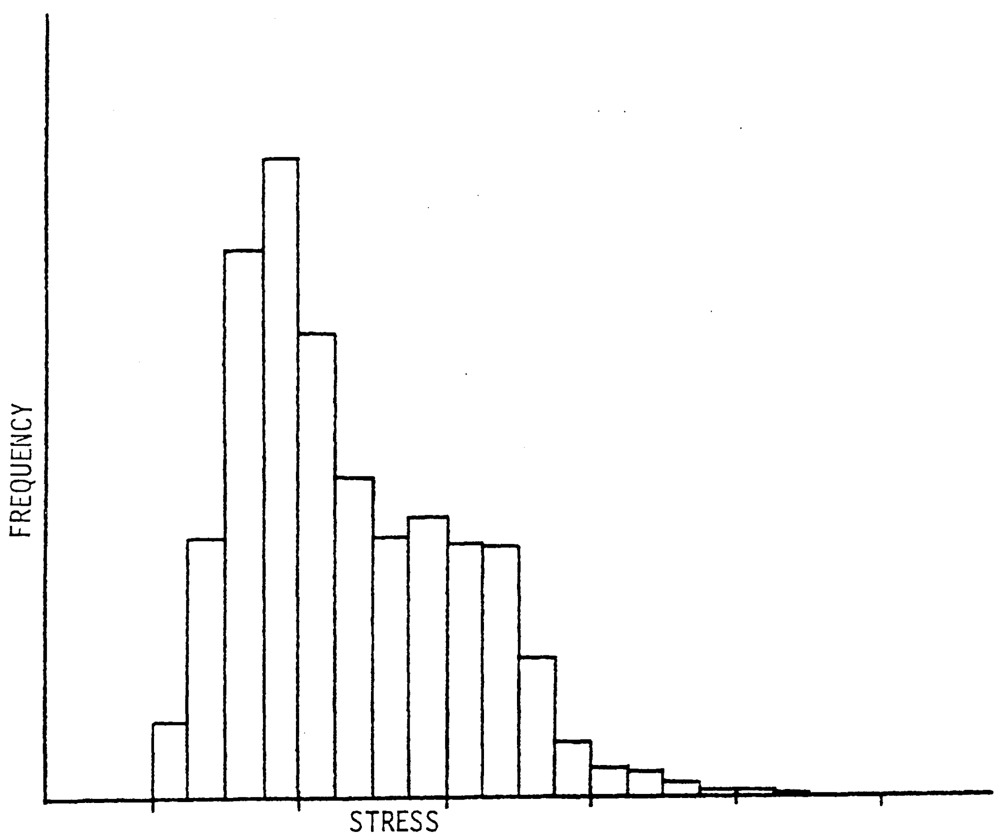


Figure 2. Typical Stress-Frequency Histogram

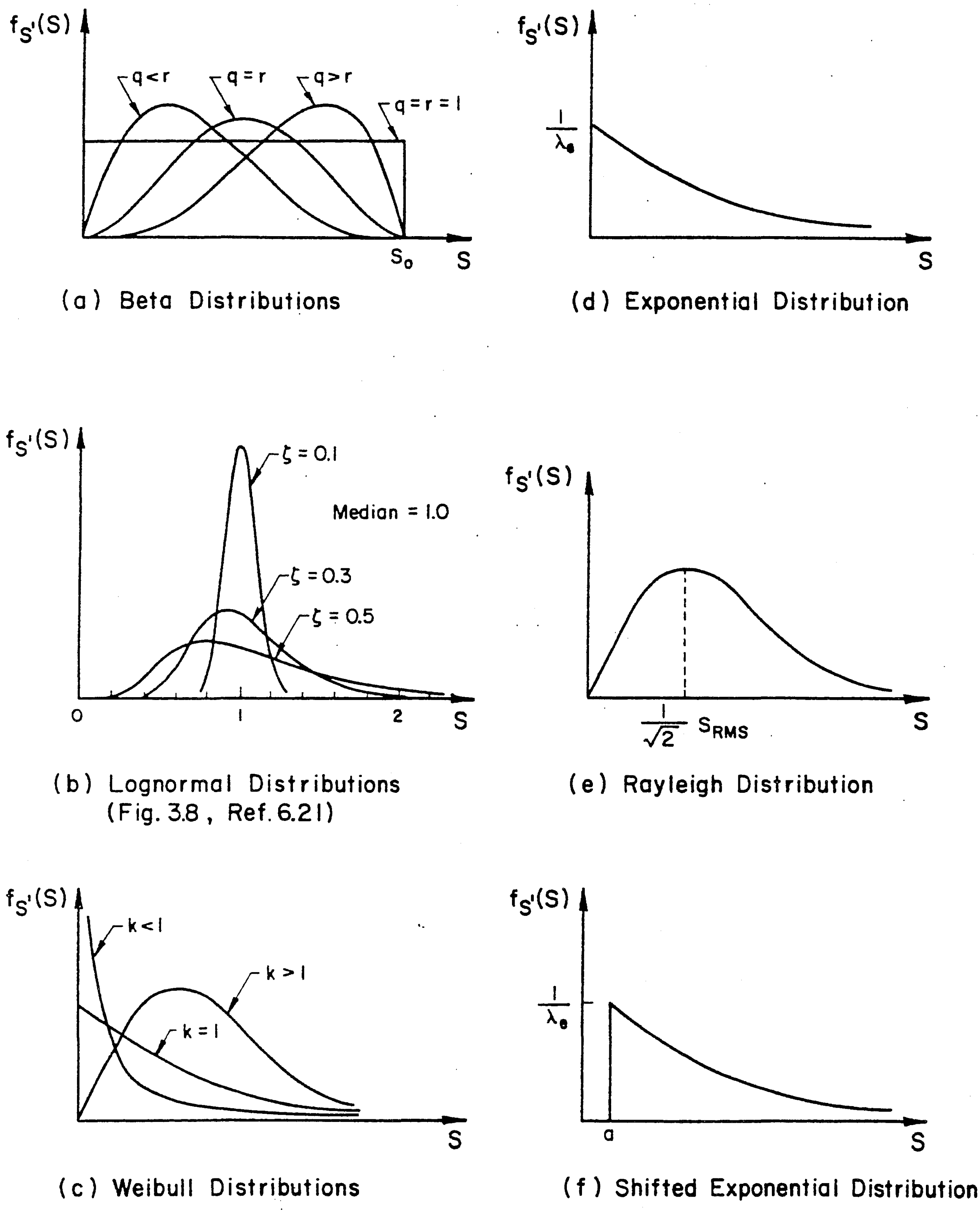


Figure 3. Shapes of Probability Density Functions (see Table 2 for Probability Density Functions). (2)

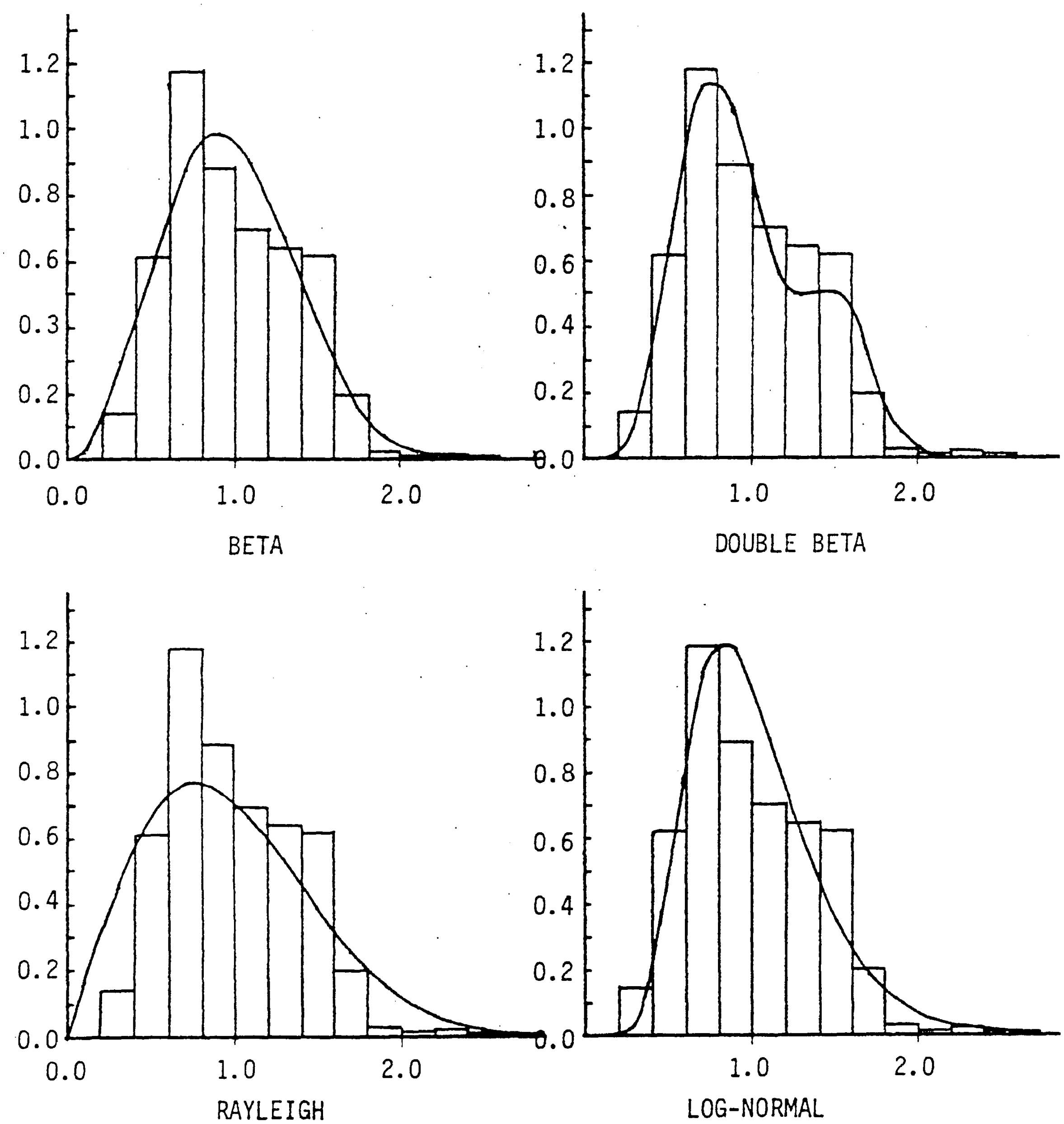


Figure 4. Various Mathematical Functions Fit to Histogram From MD 301 at MD 5, Center Line of Girders

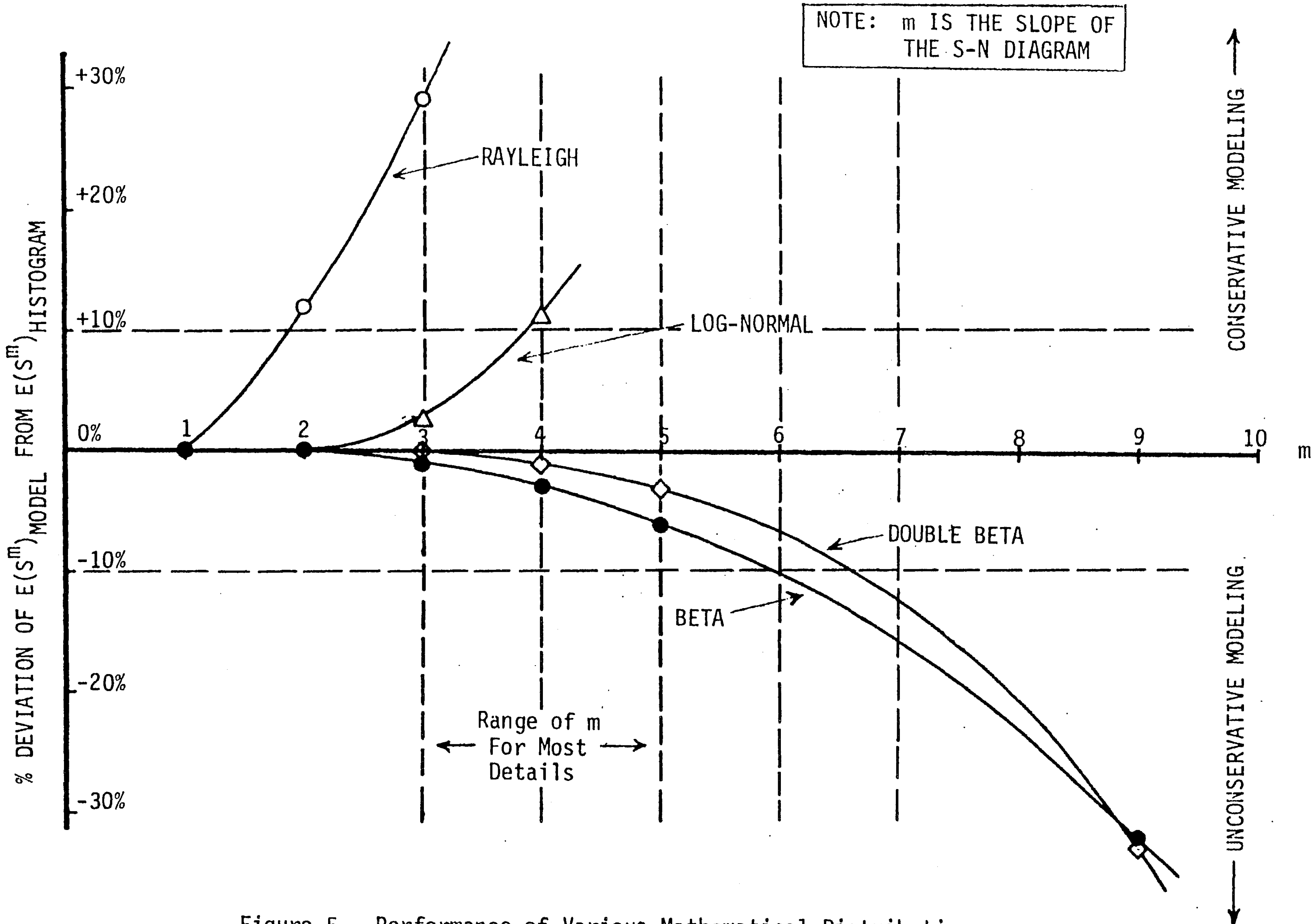
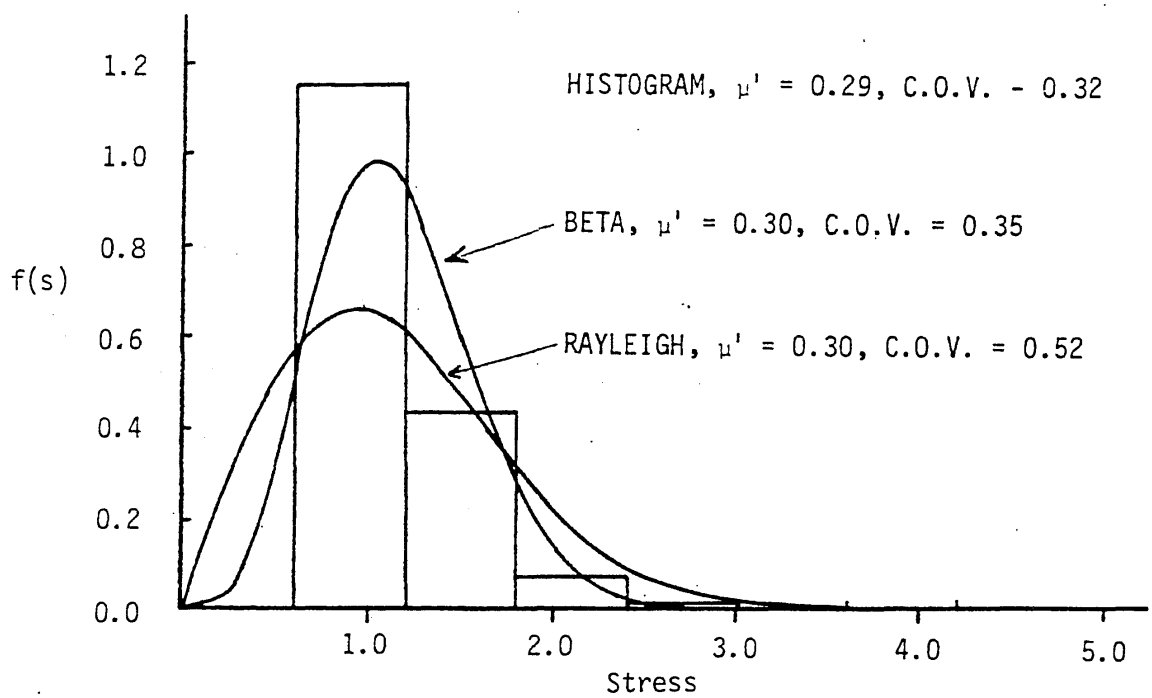


Figure 5. Performance of Various Mathematical Distributions in Modeling Histogram from MD 301 at MD 5, Center Line of Girders



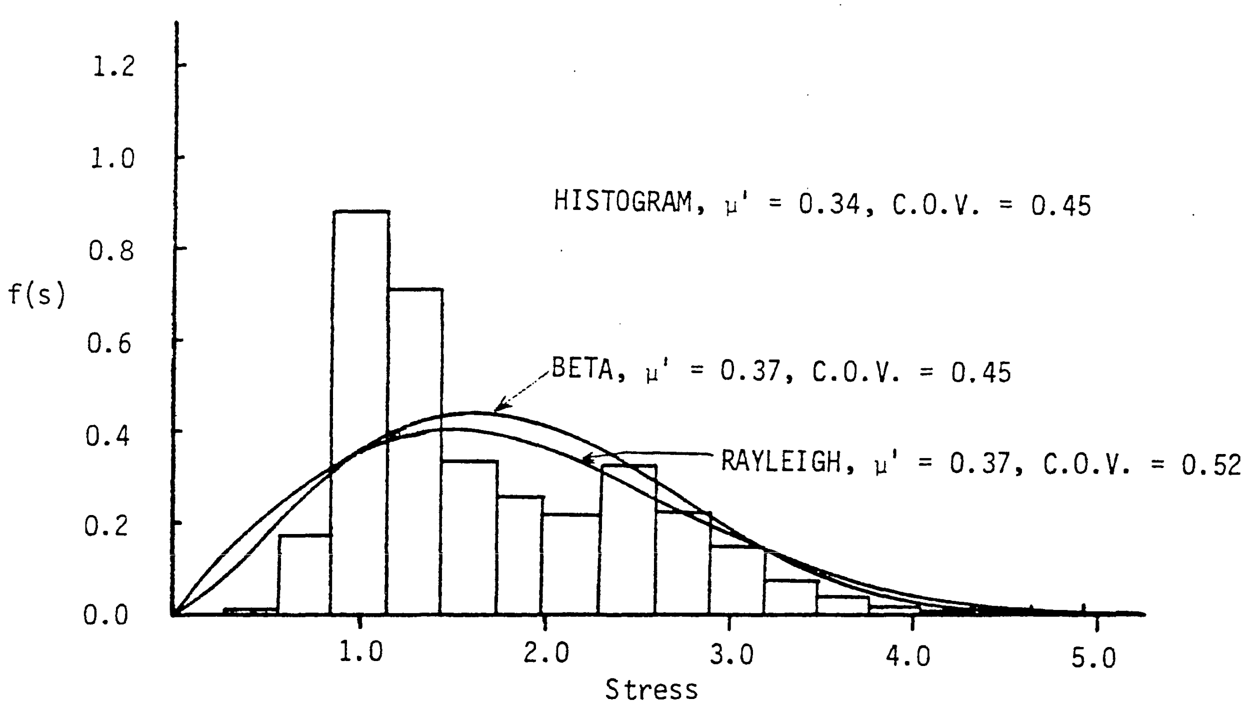


Figure 6. Histogram and Modeling Functions for Yellow Mill Pond Bridge (top) and for E, J and E Bridge (bottom)

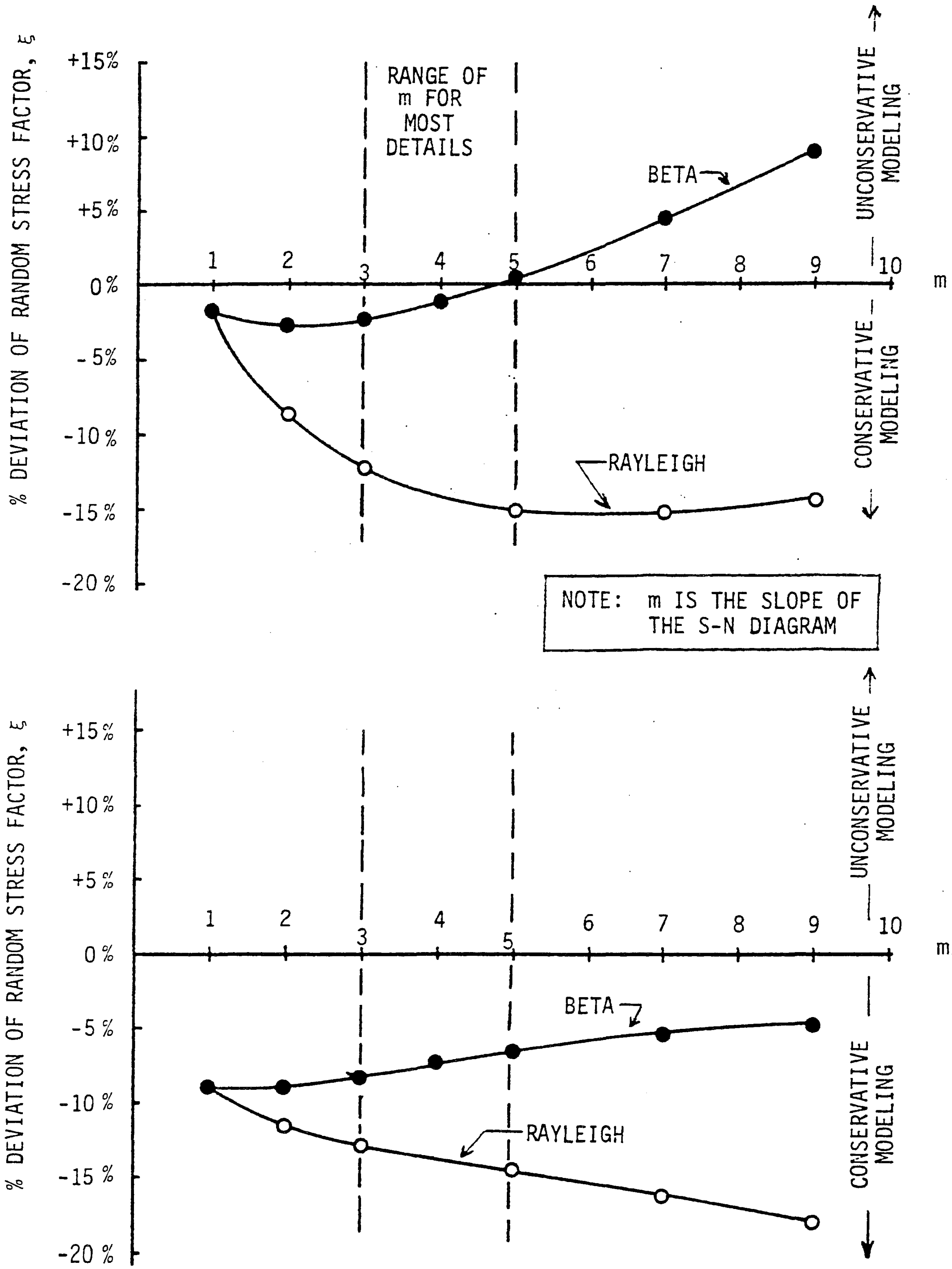


Figure 7. Performance of Beta and Rayleigh Distributions in Modeling Histograms from Yellow Mill Pond Bridge (top) and E, J and E Bridge (bottom)

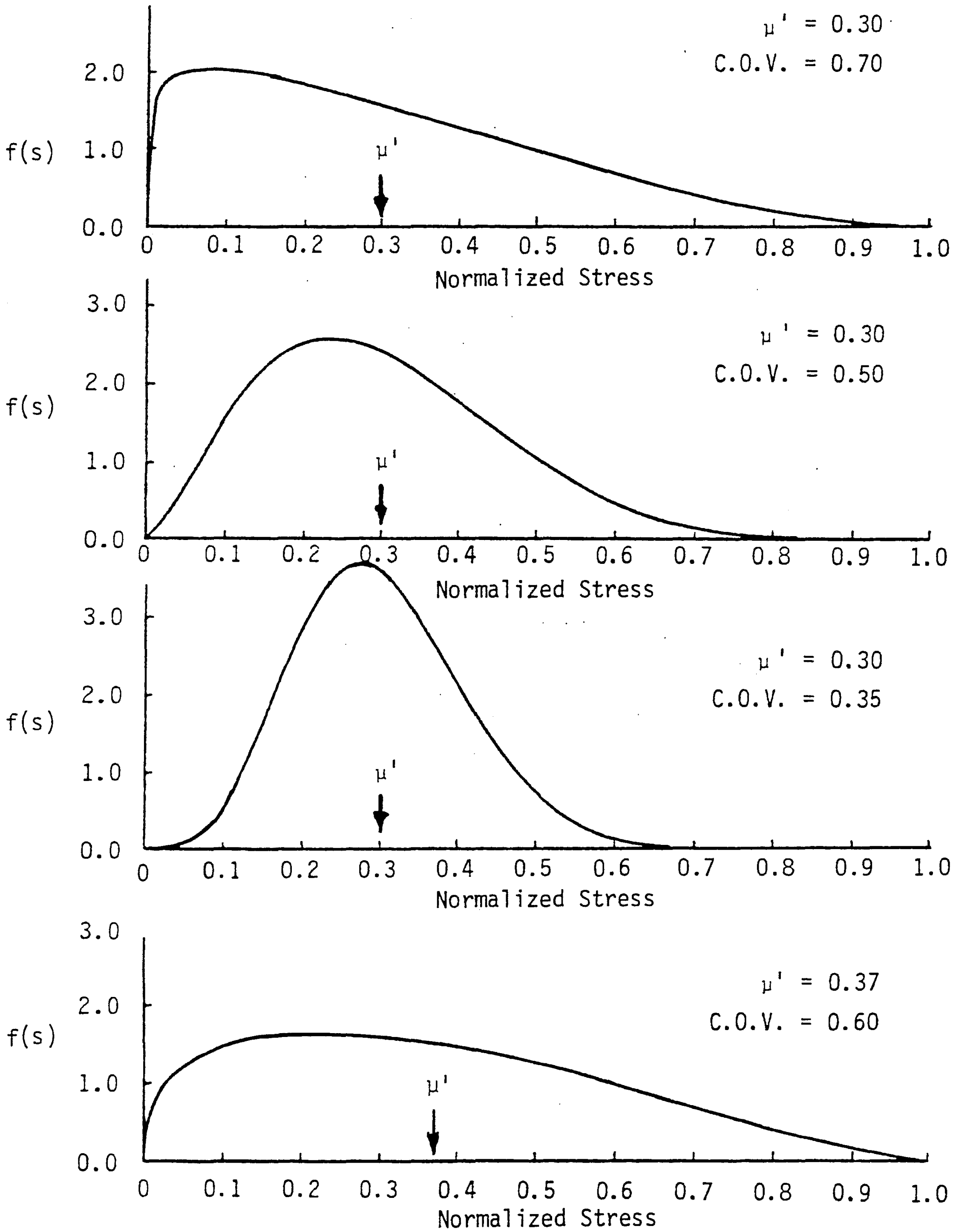


Figure 8. Normalized Beta Distributions

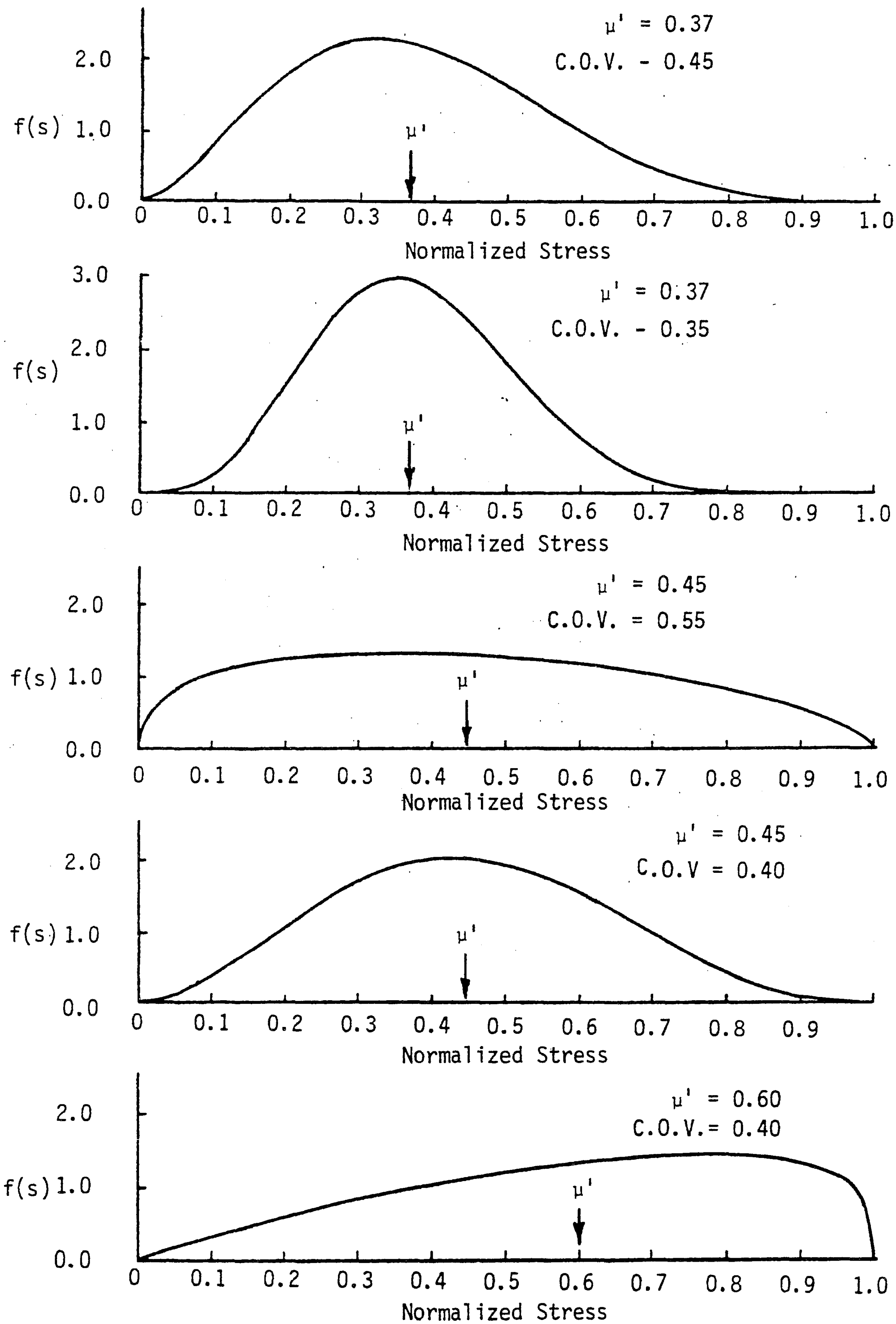


Figure 9. Normalized Beta Distributions

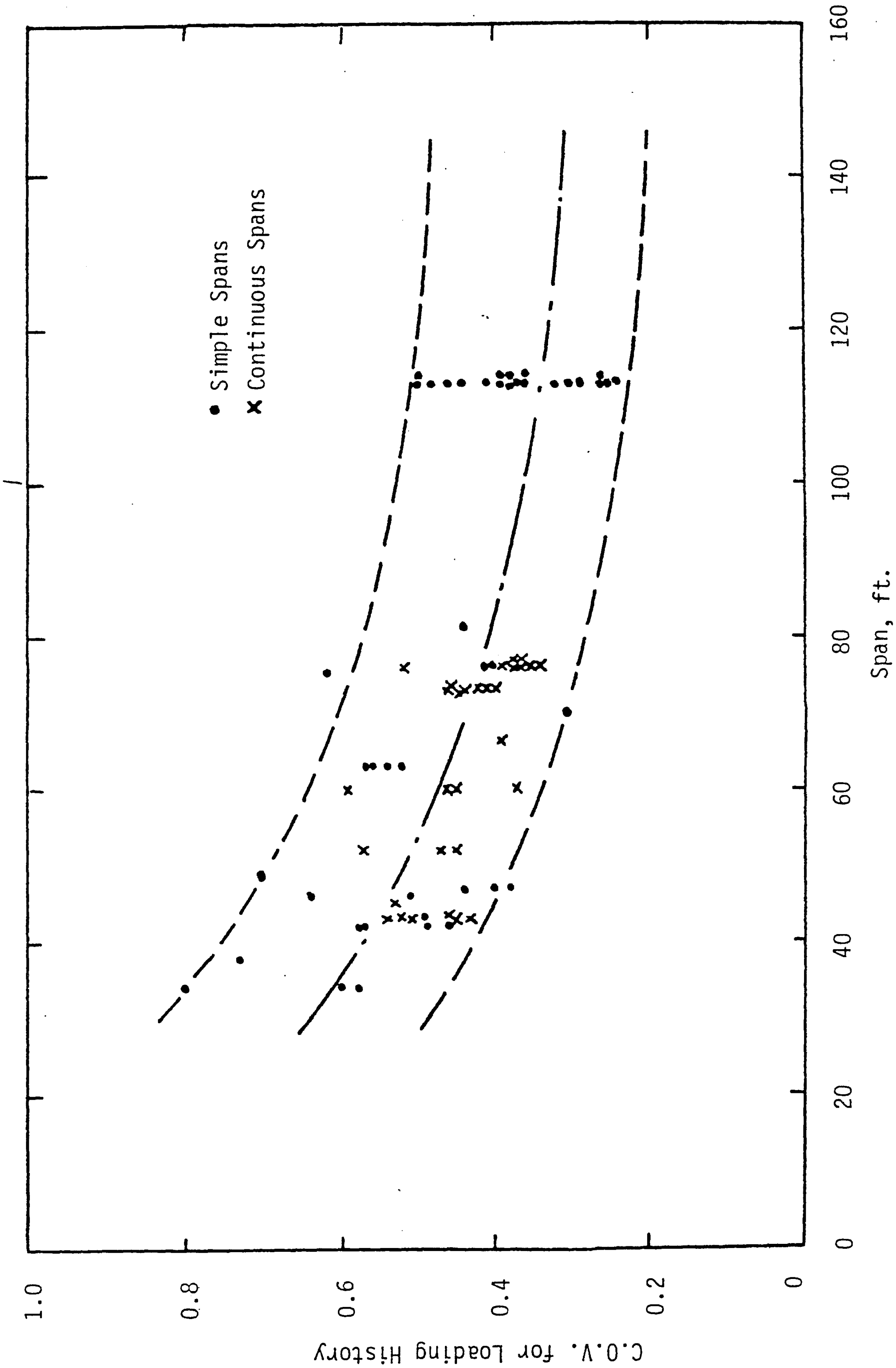


Figure 10. Values of C.O.V. for Various Bridges Studied