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STRESS HISTORY STUDIES AND THE FATIGUE LIFE EXPECTANCY OF HIGHWAY BRIDGES

By
W. H. WALKER

Issued as the Final Report on
Life Expectancy of Highway Bridges — Stress
History Studies
Project IHR-301
Illinois Cooperative Highway Research Program

Conducted by
THE 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
APRIL 1980

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16. Abstract

Increased volumes of truck traffic, pressure for increased axle and gross weight limits, and the use of new materials and structural details make the study of live-load stresses induced in bridges subjected to heavy truck traffic of increasing importance. The present study is directed to the collection, analysis, and interpretation of data on stresses at critical locations in bridges. It is focused on the development of a probabilistic technique for forecasting the stress-range environment, and ultimately the mean fatigue life of the bridge.

The study makes use of a comprehensive, computer based, data acquisition, analysis and interpretation system.

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The contents of this report reflect the view of the author who is responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Illinois Department of Transportation or the Federal Highway Administration. This report does not constitute a standard specification or regulation.

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I. INTRODUCTION

1.1 Stress Histories and Fatigue Life Expectancy

Research on live load stresses induced in highway bridges by heavy truck traffic, often termed the stress history problem, has been motivated by concern for the effect of increases in truck traffic volume and truck gross loads. Forecasts by government agencies and trade organizations concerned with the trucking industry confirm both of these growth trends to the end of the century, despite real and potential shortages of energy. The need to increase productivity and energy efficiency in the trucking industry will be reflected in the future demand for and trend toward higher gross weight limits, increased vehicle size, and a reduction in the number of trucks being operated on the highway with partial payloads. Statistically, the mean gross vehicle weight will increase with time and the distribution of gross weights will be skewed more to the high side.

For medium and short span bridges the trends just noted could be mitigated, in part, by the increased use of longer wheel base trucks -- including double or triple unit vehicles. In the congested urban areas, a counter-trend might be more widespread, that is, the use of light, fuel efficient, single-unit vehicles for delivery or local freight distribution service.

The adverse trends in live loading of bridges, the increases in frequency and severity of truck loadings, are of concern in relation to the fatigue life expectancy of existing structures and the fatigue design for future structures. Reliable fatigue life predictions could be of great value to the planner and should be made a significant component of the rating of existing older bridges for load and traffic capacity so that the determination of priorities for the replacement of such structures can be made on a rational basis.

The term stress history has been used to denote the occurrence of significant or fatigue critical live load stress events and usually implies (1) a specific definition of a critical stress event, for example maximum stress range, and, (2) a statistical description of these events and perhaps a crude correlation with a certain population of loading events. A parallel term is load history which denotes the statistical description of the live load events -- vehicles and their crossings -- to which the bridge or structural element is subjected, and which are significant for fatigue. The relationship between loads and stresses important for fatigue behavior at critical locations in the bridge is complex and not efficiently handled by exact direct methods of structural analysis or computer simulation. This is due in part to the dynamic, three-dimensional analytical model and the numerous problem parameters needed to describe the bridge - vehicle system, and because of the uncertainties in the definition of these parameters. Hence the direct study of bridge stresses, rather than applied loads, will be emphasized. The task of forecasting stress histories bassed on knowledge of axle loads, truck gross weights, dimensions and traffic conditions remains important.

Thus the study of stress histories in highway bridges has been undertaken in Illinois (1,2,3,4) and elsewhere in order to define the live load stress environment in highway bridges for the purpose of assessing the susceptibility of these structures to fatigue damage, that is, to predict their expected life under the action of repeated random loads.

The present study is not directed to the modeling of material fatigue behavior and the usual assumptions taken from the literature, primarily the Miner hypothesis, will be used for the fatigue analysis presented herein. A reliability based fatigue analysis will be used in keeping with the inherent uncertainties in fatigue behavior and the statistical descriptions required for the live load stress environment.

There are two concerns in interpretation of the stress history environment which require particular consideration: First, to evaluate the stress-time variation measured at a critical location in the bridge, under the action of an individual moving vehicle, one must decide which features of the stress-time variation are significant from the point of view of fatigue. Such a decision involves both the identification of critical stress ranges or events and a corresponding scheme for counting their occurrence. Second, the basic stress-time event or block of events for which the counting scheme has been devised must be related to the traffic stream characteristics. Both the present study and work elsewhere has emphasized sequential single vehicle crossings of relatively simple bridges loaded with traffic mainly in a single lane. At the other extreme, for example in the measurements taken on the Dan Ryan Expressway, the loading pattern is complex and a given stress-time event may correspond to the action of one or several vehicles with a longitudinal and transverse placement which is not readily measured or precisely predicted.

It should be noted that while emphasis is placed on the stress environment, information on deflections is available. Deflections may be considered from the point of view of user reaction (pedestrian or vehicle occupant) and expressed either as a deflection amplitude relative to the span of the bridge, a velocity or an acceleration. Although an analysis of the problem of user reaction to vibration is not within the scope of the study, the bridge deflections are statistically highly correlated with stress ranges.

Embedded within the data for stress histories is a dynamic component to be associated with each vehicle crossing event, that is, a dynamic increment in stress to be associated with a live-load impact factor as used in design. It is difficult to separate the impact factor out of the data presented because no direct means exist to measure the corresponding exact equivalent static response produced by the traffic stream. While an approximate calculation of the static response can be made, the uncertainties in the selection of the loads and the analysis would represent a substantial variability, perhaps equal to the apparent increment due to dynamic response.

1.2 Fatigue Damage Model

Much of the discussion and interpretation in this report will be based on a formulation drawn from a linear fatigue damage model, the Miner hypothesis, drawing upon laboratory fatigue data for constant amplitude tests. Although described previously (2, 5, 6) the derivation of the expression for mean fatigue life will be summarized in the following.

Fatigue data for constant amplitude tests is conventionally represented by the S-N diagram plotted on a logarithmic scale, where S is the constant amplitude stress and N is the corresponding life. It should be emphasized that fatigue life under constant amplitude loadings is also a random variable. The relationship between stress range and mean fatigue life may be written as:

$$\bar{n} = c/S^m \tag{1.1}$$

The fatigue life, $\bar{n}$, determined by this equation is the mean value representing the test results for the stress level, S. There is indeed scatter about the mean S-N regression line. In Eq. 1.1 the parameters c and m depend upon the steel type, connection type or weld detail, stress raisers, etc. The parameter S denotes the live-load stress range.

Under the application of mixed random cycles of stress application with stress ranges of s_1, s_2, s_3, ... s_k, which are applied for n_1, n_2, n_3, ... n_k cycles of application, the Miner's linear damage law is stated as follows:

$$\text{Damage} = D = n_1/n(s_1) + n_2/n(s_2) + \ldots$$
$$n_k/n(s_k) \tag{1.2}$$

where $n(s_k)$ denotes the life determined from Eq. 1.1 based on the stress level s_k applied in a constant amplitude fatigue test. Failure is assumed to occur when the damage level D = 1. Equation 1.2 is clearly a function of random variables, and we can write an expression to describe the mean or expected value of the damage level, D, denoted as E(D):

$$E(D) = E\left(\sum_{i=1}^{k} \frac{n_i}{n(s_i)}\right) = \sum_{i=1}^{k} \frac{\bar{n}_i}{\bar{n}(s_i)} \tag{1.3}$$

Usually the stress ranges will be described by probability density function $f_S(s)$. Given an appropriate density function, Eq. 1.3 may be evaluated in a continuous form:

$$E(D) = \int_0^\infty \frac{n\ f_S(s)ds}{\bar{n}(s)} \tag{1.4}$$

Introducing the failure criterion that the expected value of damage must be unity, and noting that $\bar{n}(s) = c/s^m$, Eq. 1.4 may be written:

$$n = \frac{1}{\frac{\int_0^\infty f_S(s)ds}{\bar{n}(s)}} = \frac{c}{\int_0^\infty s^m f_S(s)ds} \tag{1.5}$$

But, in Eq. 1.5 the quantity

$$\int_0^\infty s^m f_S(s)ds$$

is the mean value of the quantity S^m, and is denoted $E(S^m)$. Finally, the expression for the mean life for random amplitude stress ranges reduces to:

$$\bar{n} = c/E(S^m) \tag{1.6}$$

The above formulation for mean fatigue life was initially presented and explored in detail by Ang and Munse (6). Laboratory fatigue tests have confirmed the dependence of constant amplitude fatigue tests on stress range as the primary variable. Thus for random stress (load) applications, fatigue life depends upon:

1. The material and structural parameters, c and m, which are deduced from laboratory tests, and
2. the quantity $E(S^m)$ which describes the applied random stress applications.

The determination of the quantity $E(S^m)$ is the main focus of the interpretive phase of the present research.

From the above it is seen that the use of a linear fatigue damage law yields a useful and significant statistical parameter, $E(S^m)$. Other fatigue damage laws will yield different formulations and will require for application at least one of two additional steps: 1) the computation of accumulated damage on a numerical basis, either directly from the histograms of stress range or on the basis of a representative stress-block; or 2) a closed form derivation of a mean life expectancy based on a density function model for the stress environment (for example, the beta-density function).

As an example of the second step noted above, Ang (5) has shown it is convenient to use the beta density model with a linear fatigue damage law, but where a lower endurance limit is required. However, it should be noted that more complex damage criteria or damage laws consider both the amplitude and the sequence or order of occurrence of stress cycles. Analysis of damage under these circumstances requires a scheme for describing a representative ordering of high and low stress cycles or conversely the effect of ordering must be demonstrated to be not significant in the fatigue damage model. This matter has been handled in laboratory tests using representative stress blocks with a prescribed ordering of cycles. Work by Socie (8,9) and others and has shown the significance of the rainfall counting scheme for stress cycles which can take into account the damage accumulated in hysteretic stress loops.

1.3 Scope of Field Program and Data Reduction

The initial and an important phase of the IHR-301 study was the development of the field test capability for the Illinois Department of Transportation. In the previous study, IHR-85, the field measurements were undertaken as a cooperative effort between the University of Illinois, Department of Civil Engineering, the State of Illinois, and the Federal Highway Administration, with FHWA providing part of the data collection equipment. When the present study IHR-301 was initiated, it was clear that the data collection capability of the FHWA should be shifted to the State. Thus the early work on the project and several of the early bridge field studies were directed toward development of a field measurement capability. This aspect of the research has been described in the Phase 1 interim report (1) and will not be reviewed here.

Also, Phase 1 included the revision of the automated, computer-based, data reduction programs. The same computer programs have been used, with modifications, over a ten year period from the start of IHR-85 (1967) through the completion of the data reduction for IHR-301. It should be noted that during this period there has been an extremely rapid advance in computer technology and undoubtedly as this project drew to a close the entire data reduction system was in need of extensive revision, or replacement, to reflect present hardware capabilities. These programs are not considered transferable to another computer facility and are working tools for the present investigation. Specific details of the data reduction scheme have been presented in three reports: two on IHR-85, (3,4) and in the interim report (2) of the present study.

The criteria for selecting test bridges were described in the interim report (2) and are substantially the same as those used in the previous study, IHR-85 (3,4). Sketches and descriptive information on the test bridges and data on instrumentation locations was presented in the interim report and will not be repeated herein in detail. Only a summary tabulation is repeated in Table 1.1.

In all instances the effort to gather truck data was undertaken making use of state truck weighing stations which are maintained for law enforcement and research by the State of Illinois, Some descriptive information on the weighing stations is presented in Refs. 2, 3, and 4. In general, the weighing devices are electronic scales which are substantially automated; these are calibrated and sealed at frequent intervals since they are used for law enforcement. The measurements of the vehicle wheel-base were made manually with the exception that photogrammetric measurements were attempted, but with limited success. Also, photographic studies of the vehicle traffic making use of the stop-motion camera were used and are presented in Ref. 2. These photographic studies were useful in identifying multiple vehicle events and the relative proportion of trucks and passenger cars in the traffic crossing the bridge. It should be noted that the reduction of photographic records was tedious and served to shift the manpower requirements from the field to the laboratory. The photographic records are subject to changing shadow and exposure problems during the course of the day's measurements.

1.4 Objectives of Report

This report, as the final report on Project IHR-301, is intended to provide an overview of the project research, interpretation of the results, and certain recommendations on the use of stress history information for fatigue damage analysis and fatigue design. Specific objectives may be listed as follows:

1. To present data not included in the interim project reports (1,2).
2. To provide an interpretation of the stress history data including the fitting of useful probability density functions.
3. To outline the fatigue analysis problem in order to indicate suitable approaches to the representation of the stress history data.
4. To show the development of an analysis technique which is independent of a probability density function model.
5. To make suggestions concerning the implementation of the research in fatigue design practice.
6. To provide suggestions for future research.

No attempt will be made to repeat the descriptive information, summaries and tabulations of data presented in the interim report, but selected statistical data for significant characteristics such as mean, variance, and random stress analysis factors, etc. will be presented for all data developed in the study.

TABLE 1.1 SUMMARY OF STRUCTURAL DATA FOR BRIDGES TESTED

Bridge Designation	Bridge Type	Girders	Deck	Other Design Information	Test Duration	Load Events Recorded	Dynamic Properties: Freq. Hz, Meas.	Dynamic Properties: Freq. Hz, Computed: C	Dynamic Properties: Freq. Hz, Computed: NC.	Dynamic Properties: Damping % Critical
Spring Creek F.A. Rt. 154 Sect. 14-B Bureau County Sta. 400+20	3 span- Continuous 57'-3" 66'-11" 57'-3"	6 - W36x150 @ 6'-7"	7" thick R.C. 35'-8" out-to-out 9"-2'-10" curbs	0° Skew Non-composite design-1955 HS20-44 loading Rocker supports	11/12/71 to 11/18/71					
Sangamon River F.A.I. Rt. 55 Sect. 84-2B-F sta. 462+25 Sangamon County	4 span- Continuous 71'-9" 91'-10" 91'-10" 71'-9"	6 - W36x182 @ 6'-0" Cover PL. over interior pier #1 11'X11/16"x18'-6" bottom	7" thick R.C. 35'-8" out-to-out 9"x2'-10" curbs	0° Skew Non-composite design-1958 HS20-S16-44 + Alt.	6/02/72 to 6/09/72	1625 truck traffic crossings				
EJ & E (I,II) F.A.I. Rt. 57 Sect. 0303- 1002 VB Cook County Sta. 111+77.30	3 span- Continuous 60'-10" 73'-4" 60'-10"	6 - W36x170 @ 7'-2-1/2" Cover PL. over interior piers 10-1/2"x7/16"x16'-5" Top and bottom	7-1/2" thick R.C. 42'-0" out-to-out 9'x1'-9" curbs	11° Skew Composite design- 1966 HS20-44 loading Rocker supports	8/18/72 to 8/25/72 11/25/72 to 11/30/72	3680 truck traffic crossings 1687 truck traffic crossings		5.2	2.9	
Camp Creek (I) F.A.I. Rt. 20 Sect. 26-30-2(2) Fayette County Sta. 610+43.10	1 simple span in 14 spans 33'-10"	5 - W30x116 @ 7'-6"	7" thick R.C. 34'-0" out-to-out 9"x3'-0" curbs	0° Skew Non-composite design HS20-44 loading	10/23/73 to 10/26/73	1600 truck traffic crossings	11.0	14.4	9.2	
Poplar Street Approach F.A.I. Rt. 70 Sect. 82-3HVB East St. Louis St. Clair County Sta. 48+48	3 span- Continuous in a multi- span bridge 92'-0" 117'-0' 92'-0"	Girders - 72" deep welded Floor beams - W36x170 Stringers - 3W18x50 @ 8'-0"	7" thick R.C. 36'-0" out-to-out 9"x3'-0" curbs	Curved Non-composite design-1963 HS20-44 loading	3/25/74 to 3/27/74					
Shaffer Creek F.A.I. Rt. 74 Sect. 81-3B-1 Sta. 594+68.00	2 span- Continuous 43'-0" 43'-0"	9 - W24x100 5 @ 5'-6" and 4 @ 5'-4" Longitudinal separation (joint) between groups of 4 & 5 beams	7" thick R.C. 43'-8" out-to-out 9"x2'-10" north curb, 9"x1'-10" south curb	0° Skew Non-composite design-1958 HS20-44 loading and modified	6/03/74 to 6/07/74	1100 truck traffic crossings	7.8	8.2	5.2	1.6

TABLE 1.1 SUMMARY OF STRUCTURAL DATA FOR BRIDGES TESTED (CONTINUED)

Bridge Designation	Bridge Type	Girders	Deck	Other Design Information	Test Duration	Load Events Recorded	Dynamic Properties: Freq. Hz Meas.	Computed: C	Computed: NC.	Damping % Critical
Span 31, 18th Street Bridge Dan Ryan Expwy. F.A.I. Rt. 94 Sect. 5-2525.3 -AA & AHF Cook County	1 simple span 92'-6"	6 PL. Girders 60"x3/8" web Flanges 12x1" top @ center 16x1-1/2" bot. Flanges 12x3/4" top @ ends 14x3/4" bottom Girder spacing 3 @ 8'-4" 2 variable: 2 @ 7'-3-1/2 @ s. end; 4'-3", 4'-6" @ n. end	7" thick R.C. Deck tapers 42'-5-1/2" to 35'-5-1/2"	Spiral tangent Composite design 1961, H20-S16-44 mod. Intermediate cross frames 3 - 4"x3-1/2"x5/10" members; 1 @ bottom flange, 2 @ x-bracing	5/15/75 to 5/16/75 Continuous recording	1185				
Gallatin County Saline River S.B.I. Rt. 140 Sect. 113-B Gallatin County Sta. 343+48	1 simple truss 200'-0" 10 panels 1 of 6 simple appr. spans	10 @ 10I24.5 stringers inflow 24I95 int. floorbeams 2 facia @ 24I 94 4 int. @ 24I85	6" thick R.C. 24'-4" out-to-out R.C. parapet and rail	0° Skew Non-composite design-1929 deck system and approach spans	7/24/75	160 crossings for stress range studies				
Green River F.A. Rt. 141 Sect. 114-8-WPH Lee County Sta. 340+65	3 span-Continuous 43'-0" 77'-0"	5 @ W30x108 11-1/2"x5/8" Cover PL. 17'-6" long over intermediate piers	7" thick R.C. 32'-6" out-to-out 12"x9" curb	46° 25' Skew Non-composite design-1936 Widened, new curbs & handrails, 1968	7/19/76 to 7/27/76					
Camp Creek (II) F.A.I. Rt. 70 Sect. 26-3B-2(2) Fayette County Sta. 610+43.10	1 simple span unit 33'-10" 3 span-Continuous 39'-10" 50'-0" 39'-10"	2 Facia @ W30x99 5 Int. @ W30x116 2 Facia @ W30x99 5 Int. @ W30x108 Spacing for both units: 1 @ 2'-9" 4 @ 7'-6" 1 @ 6'-9"	7" thick R.C. 42'-0" out-to-out 21"x27" curbs	0° Skew Non-composite design-1946 (Rt. 40) H20-S16-44 and Alt.	9/20/76 to 9/28/76					

** Load event count to be verified

2. PRESENTATION OF DATA

2.1 General

The objective of this Chapter is to present certain additional data which was not described in the interim project report (2) and to characterize in brief the complete body of information available for analysis.

Data on gross vehicle weight, axle load, and axle spacing was collected on heavy truck traffic at or near the bridge test sites. The truck data does not coincide on an event-by-event basis with the recordings made at the bridge test sites. A vehicle identification scheme was not available to provide exact correlation between vehicle information and bridge crossing events. A full correlation was obtained for one bridge in the previous study, IHR-85 (3,4), and has been useful in the interpretation of the present data.

Vehicle data was taken during approximately the same time period as the bridge test recordings. In one instance, the second test at the EJE Bridge, the truck weighing station serving the test site was closed when the bridge recordings were made, but additional data on vehicles was taken several months later.

The test bridges, selected according to criteria described previously (2), were instrumented to determine strains at midspan cross sections and other locations such as cover plate cutoff points, e.g. locations significant for fatigue analysis.

The basic permanent project record of bridge strain (or deflection) information for each truck crossing event is an analog magnetic tape record of the strain-time histories which in the course of data processing are searched for maximum and minimum strain ranges or partial ranges, as appropriate. It is strain or stress range quantities which are tabulated herein to describe the bridge response. However the strain-time histories are retained as a part of the project record. But, it should be emphasized that the manipulation of the complete strain-time histories is a tedious task. The present summaries will emphasize information on stress range (strain range) and no attempt will be made to provide an extensive summary of plotted stress-time histories at various locations on the bridge. This latter aspect of bridge data interpretation has been dealt with theoretically in several previous reports on the research underlying the present study (3, 4). While little attention is given to the bridge stress-time history herein, its characteristics are well understood and are useful in interpreting the histograms and other data presented.

2.2 Truck Data

The data presented in Tables 2.1 through 2.3 represents measurements on over 10,000 vehicles, 7,600 of which were collected under the present program. Overall, about 80 percent of these vehicles are of the 3S-2 type, the 5-axle semi-trailer, tractor combination. This data represents, for the most part, rural interstate truck traffic in the State of Illinois in the period 1968 through 1976.

A basic question is, has the mean gross vehicle weight increased during the time period studied? To answer the question, the mean gross vehicle weight values in Table 2.1 were paired to obtain those sets of data for which significant shifts in the mean gross weight occurred. Three such combinations have been studied and the results are summarized in Table 2.4. The results are for the EJ&E Bridge, Shaffer Creek Bridge, and

TABLE 2.1 GROSS VEHICLE WEIGHT STATISTICS

Bridge	Mean GVW**	Std. Dev.	c.o.v.	Count
	(a) For All Trucks			
E.J. & E. (11/72)	42.05	18.47	0.439	270
E.J. & E. (3/73)	41.17	18.66	0.442	1,977
Camp Creek I (10/73)	48.49	18.39	0.379	1,003
Shaffer Creek (6/74)	37.80	17.56	0.456	1,422
E. St. Louis (10/74)	49.09	17.90	0.365	515
Green River (7/76)	50.66	19.10	0.377	272
Camp Creek II (9/21/76)	51.63	17.83	0.345	1,053
Camp Creek II (9/23/76)	50.92	17.63	0.346	1,076
*Shaffer Creek (1968)	40.50	20.60	0.510	249
*Shaffer Creek (1969)	34.40	20.40	0.590	862
*C.B. & Q. (1969)	46.00	18.70	0.410	1,482
	(b) For Type 3S-2 Trucks			
E.J. & E. (11/72)	45.27	18.75	0.414	204
E.J. & E. (3/73)	47.07	18.12	0.385	1,481
Camp Creek I (10/73)	52.91	16.84	0.318	772
Shaffer Creek (6/74)	49.03	19.04	0.388	1,164
E. St. Louis (10/74)	50.45	17.58	0.348	409
Green River (7/76)	55.04	15.07	0.274	231
Camp Creek II (9/21/76)	51.53	18.05	0.350	954
Camp Creek II (9/23/76)	51.28	17.62	0.343	971
*Shaffer Creek (1968)	-	-	-	-
*Shaffer Creek (1969)	44.60	18.30	0.410	512
*C.B. & Q. (1969)	51.20	17.10	0.330	1,027

* See Reference 3.

** GVW in kips (1 kip = 4.448 kN)

TABLE 2.2 AXLE LOAD STATISTICS FOR 3S-2 TRUCKS

Bridge	Axle Load Designation A	B	C	D	E
E.J. & E. (11/72)	8.8[a] (14)[b]	10.4 (42)	9.8 (45)	8.9 (55)	8.9 (55)
E.J. & E. (3/73)	8.7 (13)	10.6 (40)	10.0 (42)	9.0 (53)	9.0 (53)
Camp Creek I (10/73)	9.2 (11)	11.7 (38)	11.1 (37)	10.8 (42)	10.8 (42)
Shaffer Creek (6/74)	9.0 (15)	10.8 (39)	10.4 (42)	9.5 (53)	9.5 (53)
E. St. Louis (10/74)	8.8 (16)	11.1 (35)	10.7 (37)	10.1 (46)	10.1 (46)
Green River (7/76)	9.0 (11)	11.7 (30)	11.6 (30)	11.4 (37)	11.4 (37)
Camp Creek II (9/21/76)	9.5 (20)	11.3 (37)	10.9 (40)	10.0 (47)	10.0 (47)
Camp Creek II (9/23/76)	9.5 (16)	11.1 (37)	10.7 (39)	10.0 (46)	10.0 (46)
Shaffer Creek (1969)[c]	8.4 (15)	10.0 (41)	9.4 (45)	8.5 (56)	8.5 (56)
C.B. & Q. (1969)[c]	8.6 (13)	11.4 (35)	10.9 (37)	10.2 (44)	10.2 (44)

[a]Mean axle load in kips (1 kip = 4.448 kN)

[b]Coefficient of variation in percent

[c]Reference 3

TABLE 2.3 AXLE SPACING STATISTICS FOR 3S-2 TRUCKS

Bridge	Axle Spacings			
	Tractor Steering A - B	Tractor Drive Tandem B - C	Trailer C - D	Trailer Tandem D - E
E.J. & E. (11/72)	11.1[a] (13)[b]	4.3 (17)	25.5 (13)	4.3 (35)
E.J. & E. (3/73)	11.3 (13)	4.3 (12)	25.8 (12)	4.6 (28)
Camp Creek I (10/73)	11.1 (12)	4.3 (6)	26.8 (10)	4.4 (18)
Shaffer Creek (6/74)	- (-)	- (-)	- (-)	- (-)
E. St. Louis (10/74)	11.0 (13)	4.3 (3)	25.8 (17)	4.3 (15)
Green River(7/76)	10.6 (11)	4.1 (30)	24.8 (13)	4.0 (8)
Camp Creek II (9/21/76)	11.2 (14)	4.7 (-)	26.8 (14)	4.7 (57)
Camp Creek II (9/23/76)	11.4 (13)	4.2 (17)	27.1 (11)	4.2 (25)
Shaffer Creek (1969)[c]	10.3 (11)	3.7 (5)	24.6 (13)	3.7 (3)
C.B. & Q. (1969)[c]	10.5 (13)	3.7 (8)	25.7 (9)	3.5 (25)

[a]Mean axle spacing in feet (1 ft = 0.305m)

[b]Coefficient of variation in percent

[c]See Reference 3

Camp Creek Bridge and consider consistent sets of data for both the aggregated averages for all trucks measured and for only 3S-2 trucks. As might be expected for the short time span of the measurements at the EJ&E Bridge, there are no statistically significant differences in the mean gross weight. For Shaffer Creek over the approximate five year span between measurement periods there are statistically significant shifts in the gross vehicle weight: 2 kips in the case of all trucks and 3 kips in the case of the 3S-2 vehicles. These differences are statistically significant at the 95% confidence level based on a standard t-statistic hypothesis test of the differences between means. The data for the Camp Creek Bridge over a two year span shows a 2 kip increase in mean gross weight in the data for all trucks measured but no corresponding shift of the mean for the population consisting of only 3S-2 trucks. Indeed in the case of the 3S-2 vehicles there is a slight decrease in distribution mean although this decrease is not statistically significant at the 95% confidence level.

Where the change in mean gross weight is largest, for the Shaffer Creek Bridge, there was a significant change in traffic conditions at the site. In 1969 the bridge served only local traffic in the Quad Cities area since the interstate by-pass using I-280 had not been completed. In 1974 the bridge over the Mississippi River on this route was completed and the interstate segment of which Shaffer Creek Bridge is a part, became a south by-pass to the Quad Cities for traffic on I-80.

All data on gross vehicle weight was taken at state truck weighing stations which are used primarily for law enforcement. The presence of the stations and the schedule of their operation influences the flow of overloads. Thus, realistically, the characteristic bimodal shape of this

TABLE 2.4 STATISTICAL TESTS OF DIFFERENCES IN MEAN GVW

Bridge Pairing	Mean GVW (kips)	Std. Dev. (kips)	Count N	Test Results t-Statistic	D.O.F.[b]	Δ[a] (kips)
(a) All Trucks						
E.J.E. (11/72) E.J.E. (3/73)	42.05 41.17	18.47 18.66	270 1977	0.738	348	0
Shaffer Creek (7/69) Shaffer Creek (6/74)	34.40 37.80	20.40 17.56	862 1422	4.069	1611	2
Camp Creek (6/74) Camp Creek (9/21/76)	48.49 51.63	18.39 17.83	1003 1053	-3.923	2041	2
(b) 3S-2 Trucks						
E.J.E. (11/72) E.J.E. (3/73)	45.27 47.07	18.75 18.21	204 1481	-1.285	259	0
Shaffer Creek (7/69) Shaffer Creek (6/74)	44.60 49.03	18.30 19.04	512 1164	4.513	1013	3
Camp Creek (6/74) Camp Creek (9/21/76)	52.91 51.53	16.84 18.05	772 954	1.644	1690	0

[a]Δ = difference in mean values supported by data at 95% confidence level. (t-test)

[b]D.O.F. = Degrees of freedom in t-test

histogram for GVW should be considered to be incomplete in the GVW range above 73,280 lbs, that is, above the legal limit. Undoubtedly a tail to the distribution extends beyond the cut off in the data shown; but, it is very difficult to develop data on high GVW levels. Arrest records for truck weight violations show that GVW levels can be much higher than 73,280 lbs for 5-axle truck-trailer combinations. The degree to which such vehicles operate in the truck traffic stream is not known.

Histograms for two sets of data, Green River Bridge and Camp Creek Bridge - 1976, describing gross weight, axle load and axle spacing which were not included in the interim report are presented in Appendix A. The Green River histograms are shown in Figs. A1-A11. Testing at Camp Creek was conducted in two phases, on September 21 and 23, 1976, respectively; truck data for the two dates have been kept separate and two sets of histograms are presented in Figs. A12-A33. The general features of the histograms are familiar from previous data; as before, for gross vehicle weight and for corresponding heavily loaded axles which correlate well with the gross weight, the characteristic bimodal histogram shape is seen.

Finally, from Table 2.3 it may be seen that the vehicle dimensions for the 3S-2 trucks are consistent and suggest a model five axle vehicle with a tractor wheelbase of 11 ft, tandem axle spacings of 4 ft for both tractor and trailer, and a trailer wheel base of about 26 ft.

2.3 Bridge Data

A major portion of the bridge data collected in the field program has been presented in the project interim report (2). In this section data for three field tests will be presented: Camp Creek with two phases, Green River and the Dan Ryan 18th Street bridge.

The presentation of data for the Camp Creek and Green River bridges follows the format set in previous presentations in the interim report. Significant stress events for the latter test can be associated with individual vehicle crossing events; the traffic pattern on the bridge is simple. However, Green River is one of the bridges in the study where two-way traffic was present. For the bridge on the Dan Ryan Expressway the traffic pattern is more complex and at peak travel times the structure is continuously loaded by trucks in multiple lanes. Such a loading pattern, when viewed in terms of the shape of the resulting-time history, produces stress events of much longer duration than are characteristic of other test results. Indeed, these long events correspond to an unknown number of multiple vehicles acting simultaneously. The data record at the Dan Ryan site was made continuously over a 24 hr period with interruptions only for changing magnetic tapes, calibration and adjustments.

Camp Creek and Green River Bridges

The data collected on these bridges represent a total of 29 strain gage locations on three separate structures for a total of three test periods, and are summarized in Appendix B. The data is presented in the form of tabulated data for histograms, including mean strains and coefficients of variation, and plotted histograms. These plots and tabulations are in the same format which was used and discussed in detail in the interim report. All data is aggregated and not separated by day or hour of test; the levels of mean strain and maximum stress observed follow the patterns reported previously. However, the coefficient of variation for the data at Green River Bridge is somewhat higher, on the order of 0.6 to 0.8, than has been the common for data taken at interstate highway locations. The

Green River Bridge site is on a two-lane, state route and carries two-way traffic of relatively low volume serving local industry. Also, the Green River is significantly skewed (46^o); the combination of two-way traffic pattern and skewed structure may account for the greater variability in the stress range data. Further comments on the statistical descriptions of all data for both sites will be made in Chapter 3.

The Camp Creek data is presented in two groupings, designated Phase 1 and Phase 2. In Phase 1 the measurements were taken primarily on one single-span structure of the multi-span Camp Creek Bridge complex, and one data channel was added from a three-span unit in the complex. The structure differs from that tested in 1973, denoted Camp Creek (I), in that it represents a modification following a widening project in which an additional longitudinal beam was added together with a new curb and wider deck slab. The second phase of testing at the site concentrated on the above noted three-span structure, with one gage from the single-span structure, previously tested, recorded simultaneously. This selection of strain gages provided an opportunity to test the consistency of the data for the two phases for bottom flange strains on beam 4; there were no statistically significant differences between the mean levels seen, although the number of events recorded was small, 86 and 43, respectively in Phases 1 and 2. Differences in mean levels were evaluated on the basis of the standard t-statistic test for the comparison of means.

Dan Ryan Expressway - Eighteenth Street Bridge, Span 31

The Eighteenth Street Bridge on the Dan Ryan Expressway was the subject of a continuous twenty-two hour recording session. This is a difficult field test location, the structure being part of the elevated roadway immediately before the 18th Street on-ramp of the northbound segment of the Dan Ryan Expressway. The instrumentation trailer was placed in a parking lot under the elevated structure; the deck surface was not accessible to the field party and no convenient vantage point was available to observe traffic during the course of the recordings. The test site was costly and hazardous to occupy and it was not feasible to attempt to repeat the studies or to occupy the site for more than 24 hours with the resources allocated.

The recording system and software to be used were designed for data acquisition where bridge stress and truck crossing events are readily distinguished; the recording equipment can be started using vehicle detector, a calibration step inserted before each event, the data be recorded, and the system stopped automatically. However, when recording in a continuous mode, it was necessary to manually switch in calibration signals and be concerned with setting zero signal levels, a particularly difficult problem with continuous traffic.

The recording session began at approximately 5 p.m. on May 20, 1975, and continued until 7:00 p.m. the next day. The information gathered is contained on fourteen 3500 ft analog magnetic tape reels, i.e., seven from each of two recorders. The total recording time is approximately 22 hours on tape, plus the time required to change the tape reels and make adjustments. Calibration steps were inserted in the record every thirty to sixty minutes. The calibration events, plus some electrical noise, and zero shifts required particular attention in editing the tape record before data reduction.

In view of the volume of data taken and the difficulties associated with editing the data to make it suitable for automatic processing, several alternate reduction procedures were tried. It was hoped that the procedure using an automatic repetitive application of the event-by-event data reduction system use might prove adequate; however, because of the great length of the record and the need to edit out unwanted events this proved prohibitively costly, if the standard sampling rates and techniques were used. Hence, strip chart plots were made of all data and these charts were scanned to select suitable intervals for detailed analysis.

The Dan Ryan structure consists of 60 in. deep plate girders on a 90 ft span and the live load stress levels were low; few of the strain measurements were significant for fatigue analysis. Two locations, the bottom flange strain on beam five, one of the heavily loaded beams, and the horizontal strains at a welded vertical stiffener cutoff point on the same beam, were chosen for editing and sampling. The editing consisted of selecting half-hour segments at intervals of three to four hours for a total of eight segments in the twenty-two hour record. These eight segments were then sampled for strain range events.

While selecting suitable time segments for study, three qualitative characteristics of the bridge response to traffic became apparent. First, when traffic volume was light and the vehicle speeds approached normal maximum levels, the records show events on the strain trace which, when examined on an expanded time scale, appear identical to the expected dynamic influence line for typical heavy highway vehicles. Second, when traffic becomes heavier, and speeds are reduced, there are large strain events but of greater duration. These correspond either to slow moving heavy single vehicles or to multiple vehicles in two lanes closely staggered such that one long vehicle-crossing event is developed. Third, the bridge is on the northbound lane of the Dan Ryan carrying traffic into Chicago, and traffic serving the city has some expected characteristics. In the early morning hours from 3:00 to 6:00 a.m., there are a number of large strain events which suggest loaded heavy trucks entering the city for the business day. These events continue through the 8:00 to 9:00 a.m. period, although masked somewhat by heavy traffic conditions at rush hour. In contrast, at the 5 o'clock rush hour there are fewer large amplitude events although there is considerable activity on the bridge due to local truck and automobile traffic.

A general picture of the data editing can be obtained from Table 2.5, wherein the time periods for sampling of bottom flange strains on beam 5 are listed. Also given are the mean strain range of the histogram, corresponding standard deviation and coefficient of variation, the maximum strain event and the count of strain range events in each time period. The mean strain levels are between 39 and 53 microstrain, that is, mean stresses between 8 and 10 MPa. The variability in the data does not exceed that which would be expected from studies at rural interstate locations, i.e., coefficients of variation between 0.4 and 0.6. The highest strain level encountered is about 120 microstrain, or a stress of 24 MPa. A total of 1,185 strain events were recorded in the eight sample periods. A histogram for the composite of the eight samples is shown in Fig. 2.1; the corresponding data is given in Table 2.6. For the composite of all events, the mean strain is 46.1, standard deviation 22.6 and coefficient of variation of 0.49. The shape of the composite histogram does not differ from what might be

TABLE 2.5 DAN RYAN BRIDGE; SUMMARY OF STATISTICS BY HALF-HOUR

Date	Time	Mean Strain	Std. Dev.	C.O.V.	Max Strain	Count
5/20/75	8:00 - 8:30 p.m.	47.678	21.517	0.413	94	90
5/20 - 5/21/75	11:43 p.m. - 12:13 a.m.	40.000	25.086	0.627	91	139
5/21/75	3:16 - 3:46 a.m.	48.670	20.339	0.418	100	112
"	5:40 - 6:10 a.m.	52.486	23.143	0.441	118	177
"	9:01 - 9:36	48.564	23.513	0.484	107	234
"	11:31 a.m.- 12:01 p.m.	46.370	22.197	0.479	119	192
"	2:28 - 3:00 p.m.	39.101	20.268	0.518	85	149
"	5:14 - 5:44 p.m.	42.239	20.365	0.482	106	92
					TOTAL	1185

TABLE 2.6 HISTOGRAM DATA FOR STRESS RANGE
DAN RYAN BRIDGE--18TH STREET

STRAIN GAGE= 125 NOTE= 1 TOTAL EVENTS= 1185

MEAN STRAIN	STD.DEV.	MEAN STRESS	STD.DEV.
46.052	22.594	9.2104	4.5188

COEFFICIENT OF VARIATION= 0.491

NO COUNTS AFTER 12 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	182	0.154
30	6	176	0.149
40	8	207	0.175
50	10	144	0.122
60	12	144	0.122
70	14	108	0.091
80	16	130	0.11
90	18	67	0.057
100	20	19	0.016
110	22	6	0.005
120	24	2	0.002
------	------	------	------

expected from previous studies on less heavily traveled interstate locations. There is little suggestion of a bimodal shape for the histogram; that is, it would be unrealistic to assume for this histogram a shape identical to that for the gross vehicle weight histogram typical for heavy truck traffic as measured at weight stations (although no direct information is available on gross weight histograms for the Dan Ryan location).

In addition to the composite histogram, eight individual histograms for bottom flange strain in beam 5 for each of the sampling periods (per Table 2.5) are presented in Appendix C. These histograms have an appearance similar to that for the composite plot, particularly for those periods when the heavier loads predominate, 3-6 a.m. and 8-9 a.m. An exception is seen for the midnight hour where there is a high percentage, 30%, of very small strain events in the histogram. This is due in part to the fact that during this quiet period a particularly clear record was obtained on which small and moderate amplitude events were clearly defined, easily detected and sampled. Perhaps with a quieter recording at other hours additional small events might have been detected. In any case, these small events are not significant for fatigue and it can be argued that these should be edited from the record.

Data was sampled for longitudinal strain at a welded stiffener cutoff point located at midspan on beam 5. As would be expected from structural theory, the results for this location are similar in shape of the time-histories to those recorded for the bottom flange of beam 5. Typical histogram data and a plotted histogram are given in Table 2.7 and Fig. 2.2, respectively. Since stress measurements cannot be made for every potential

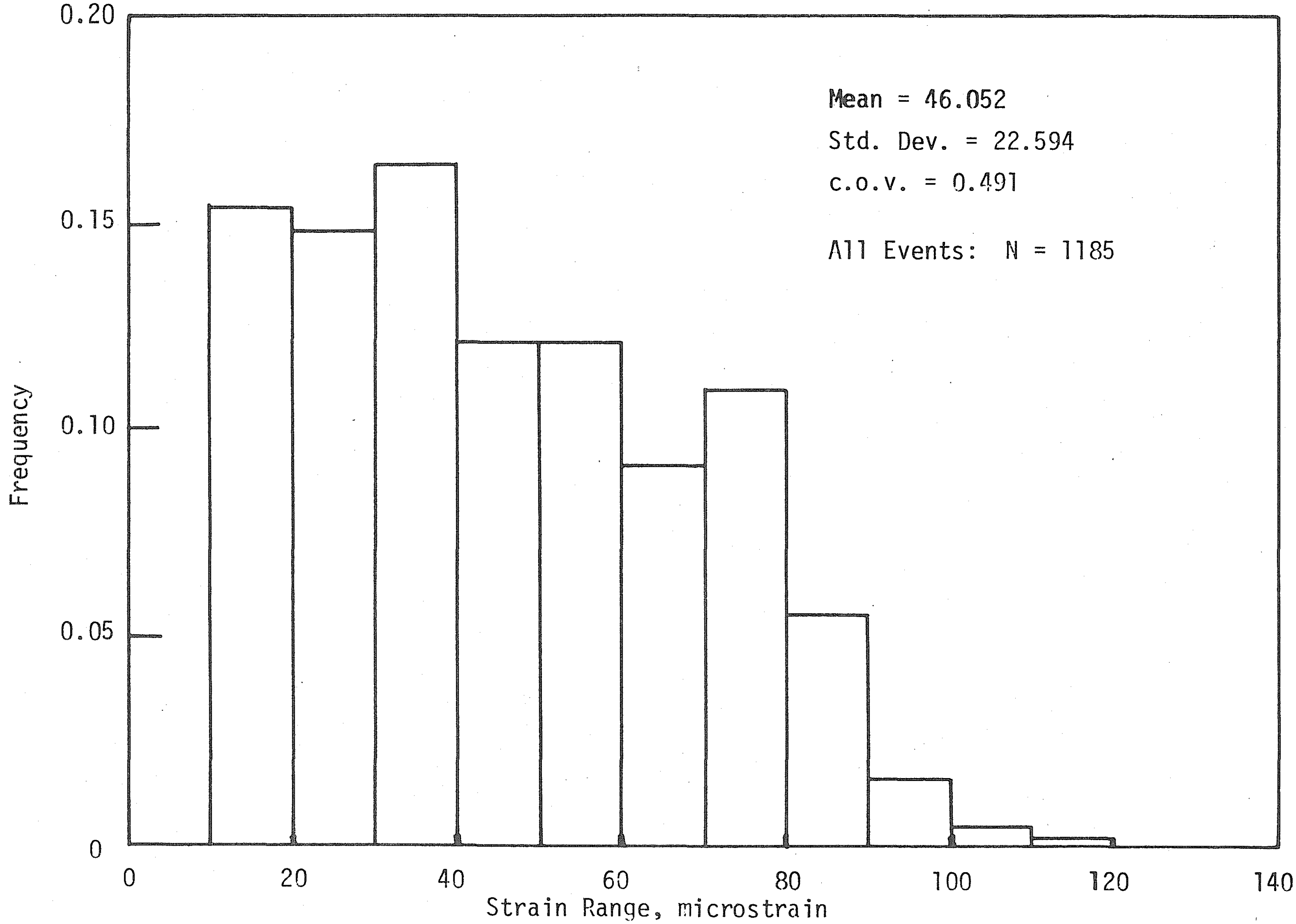

Fig. 2.1 Histogram for Strain Range -- 18th Street Bridge, Dan Ryan Expressway Bottom Flange Strain, Beam 5

TABLE 2.7 HISTOGRAM DATA FOR STRESS RANGE
DAN RYAN BRIDGE--18TH STREET

STRAIN GAGE= 135 NOTE= 6 TOTAL EVENTS= 216

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
45.097	21.418	9.0194	4.2836

COEFFICIENT OF VARIATION= 0.475

NO COUNTS AFTER 11 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	1	0.005
20	4	37	0.171
30	6	30	0.139
40	8	23	0.106
50	10	41	0.19
60	12	24	0.111
70	14	27	0.125
80	16	22	0.102
90	18	9	0.042
100	20	1	0.005
110	22	1	0.005
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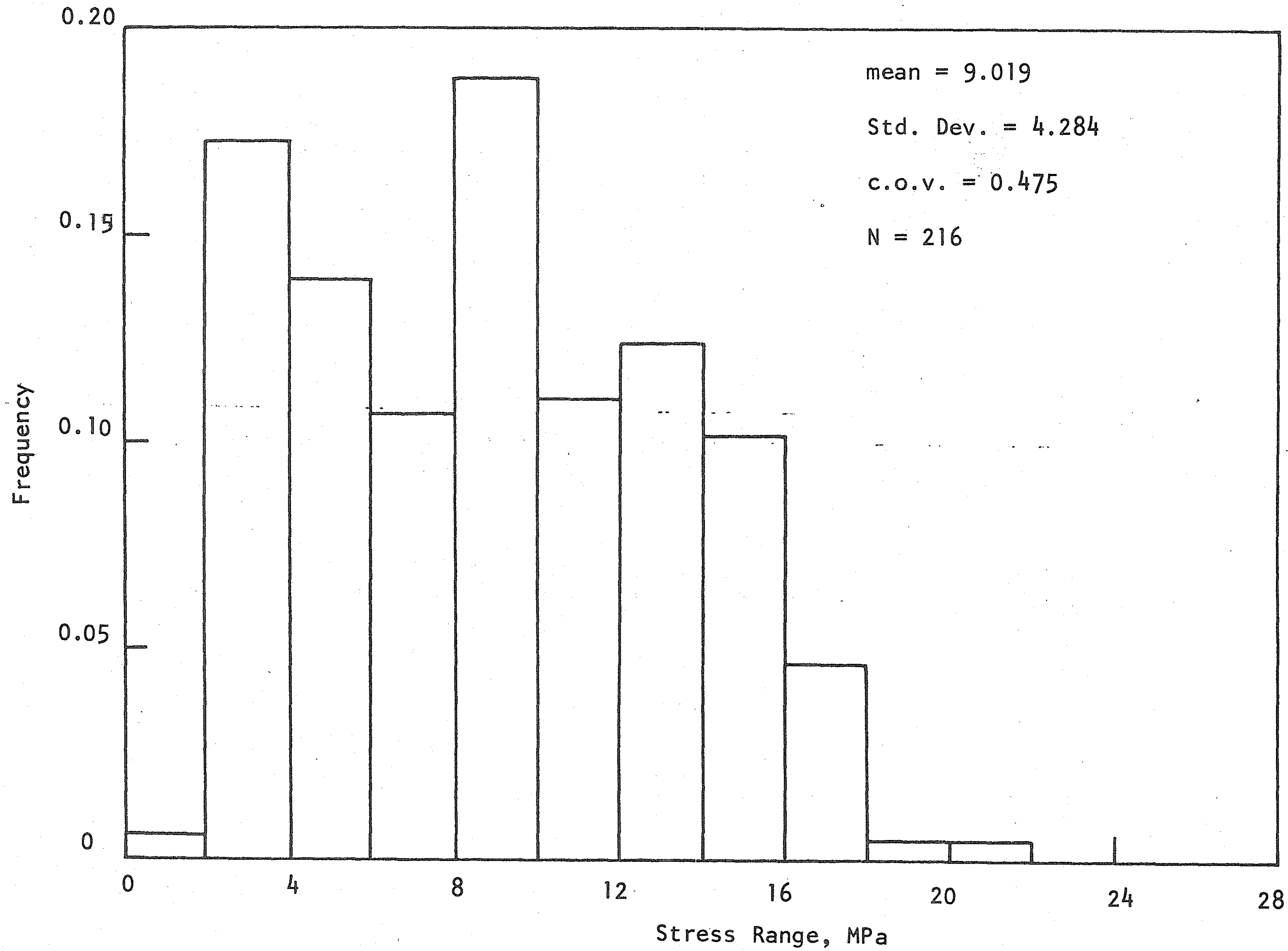

Fig. 2.2 Histogram for Stress Range--18 th Street Bridge, Dan Ryan Expressway--9:01 to 9:36 am Bottom Termination of Vertical Stiffener--Beam 5

fatigue sensitive location, it is useful to explore the statistical correlation between a typical midspan measurement location and a possible fatigue critical location. For the stiffener cutoff point and the bottom flange strain, the records were matched in time, and event-by-event the peak amplitudes were compared in a linear regression analysis. This sampling was made for two time periods of two minutes each, one in the early morning hours at approximately 6:00 a.m. and the other at 9:00 a.m. The results of these studies shows that one correlation coefficient ranges between 0.90 and 0.94, as would be expected since these are structurally related. The slope of the linear regression lines range between 0.84 and 0.87 for stiffener strain vs bottom flange strain. This slope is approximately correct based on a linear distribution of strains on the beam cross section.

$E(S^m)$, critical for fatigue life estimates, was investigated and will be discussed in detail in Chapters 4 and 5.

In Table 2.5 differences in mean stress level are seen for the histograms for the eight sampling periods. These differences should be interpreted with caution in view of the variance of the data. A t-statistic test for the differences in the paired mean stresses for successive periods of sampling shows that the largest difference in mean stress which can be supported at the 95 percent confidence level is 3 microstrain; the largest difference supported between any pair of sampling periods is 9 microstrain, i.e. between the largest and the smallest observed mean values in the table.

Finally, from this study of the Dan Ryan strain information it is seen that a major revision in data handling is needed to cope with continuous data recordings of this size. A revision could use one of two approaches: (1) to digitize and perform counting and strain event analysis on line in the field, or (2) to perform the analysis on edited analog magnetic tape recordings. The recording technique for the second approach should be modified for the use of an additional recording channel to signal the processor of unwanted events, such as calibration steps and equipment adjustments and permit the automated data processing to edit out these events.

3. DENSITY FUNCTION MODELS FOR STRESS-HISTORY DATA

3.1 General

The data on stress range may be analyzed explicitly to determine the critical statistical parameter, $E(S^m)$. However, the needs of the analyst are perhaps better served by fitting one, or more, useful probability density functions to model the stress range data. Simple unimodal density functions will be investigated. The virtues of the beta density function have been described in Ref. 2. Other functions, the truncated Rayleigh and lognormal, will be discussed because of their use in the literature on fatigue analysis (Rayleigh) or as one limiting shape for a distribution to describe the stress range data (lognormal).

The basis for deciding the suitability or "goodness-of-fit" of the distribution should be the adequacy of the density function in predicting the quantity $E(S^m)$, rather than a formal statistical test. The fatigue parameter, m, will be retained as a variable in the study; m-values from 3 to 5 will be studied herein. Laboratory research on the fatigue behavior of various steels and selected structural details shows that this range of m values encompasses, approximately, most fatigue behavior of interest. Intuitively, the larger the m-value, the more sensitive the modeling process will be. A small mismatch in modeling higher stress range levels will be exaggerated for the higher exponent when tested in terms of the predicted $E(S^m)$.

It has been shown in previous work (3,4) that there is a strong correlation between gross vehicle weight and the stress range despite all other parameters which enter into the bridge-vehicle behavior problem. Also, the distribution for gross vehicle weight tends to be strongly bimodal in probability distribution. These two facts taken together suggest that a bimodal distribution is needed to describe adequately the stress range data. Alternatively, one can argue that the stress range can be described by the superposition of two density functions to be directly associated with the two peaks in the gross vehicle weight data, e.g. for loaded and empty trucks. This bimodal approach would yield a second generation theory shich involves a fitting process which could be undertaken only for a relatively larger body of stress range data to better represent the extreme value or high stress portion of the stress range distribution. To support such a theory, a study of gross vehicle weights in the high, overload, range should also be undertaken to insure an adequate modeling of extreme values in the distribution.

3.2 Data Sample and Basic Statistics

The evaluation of density function models for the data collected will be made using a selection of forty two channels of data taken from tests on five bridges. These bridges are, respectively, reading from top to bottom of the order of presentation to be used in tabulations: the EJE(I) Bridge--gages 221-225, 533,534,433, and 434; EJE(II) Bridge -- gages 221-225, 533, 534, 433, and 434; Camp Creek (I) Bridge -- gages 113 and 114 and 121-125; Gallatin County Bridge -- gages 222, 224-226, 221, 121-123, 113 and 221; and, Shaffer Creek Bridge -- gages 121-125, 114 and 115. This body of data represents single-span and two-span and three-span continuous bridges and the deck stringer system of a truss bridge.

The primary statistical characteristics of the stress range data are shown in Table 3.1, including the gage designation, mean (MPa), coefficient of variation (c.o.v.), skew, kurtosis, ratio of maximum to mean stress level, and coefficient of variation based on analysis of the grouped data. The mean and coefficient of variation were calculated from the original stress range data points before grouping for histogram construction. The values of skew and kurtosis were calculated on the basis of the grouped data, that is, on the weighted histogram information. The maximum stress is simply the maximum range value observed for the channel. The column indicating the coefficient of variation based on a grouped data calculation is shown for comparison with the corresponding values taken from the ungrouped data. This comparison will provide a basis for judging the adequacy of the grouped vs. ungrouped statistical calculations.

In Table 3.1 the mean values of stress range have a wide range -- from as low as 4.6 MPa to as high as 47.6 MPa. Coefficients of variation range from 0.2 to over 0.6, with one high value of 0.75. Both positive and negative skews are represented, but it should be noted that the negatively skewed distributions are limited to the data for the Gallatin County bridge. At the Gallatin County test site only loaded trucks were included in the sample--heavy ore trucks operating in conjunction with a mine operation in the vicinity. Furthermore, the bridge was on a two-lane road and the instrumented beams were under the lanes used by the loaded trucks rather than the returning empty vehicles.

No conclusion can be drawn regarding the values of kurtosis other than that they vary widely. It will be seen that there is considerable difficulty in finding probability density functions to match the four statistical moments presented in Table 3.1. To give an indication of the variability of the coefficients of variation for the data, these have been summarized

TABLE 3.1 SUMMARY OF STRESS RANGE HISTOGRAM STATISTICS FOR 42 DATA SETS

GAGE	MEAN (MPa)	C.O.V.	SKEW	KURTOSIS	MAX/MN	C.O.V.-GRP
221	7.4774	0.399	0.827	4.37	4.28	0.4002
222	11.2556	0.4525	0.828	2.79	2.84	0.4551
223	14.3458	0.464	0.759	2.611	2.65	0.4478
224	11.793	0.4502	0.884	3.041	2.88	0.4523
225	7.4314	0.4515	1.643	8.515	4.31	0.4677
533	12.1286	0.4557	0.611	2.719	2.8	0.4599
534	8.5166	0.4244	0.616	3.699	3.76	0.4167
433	11.601	0.4405	0.626	2.363	2.76	0.4318
434	8.2352	0.441	0.588	2.023	2.67	0.4095
221	8.2296	0.4387	3.573	32.087	5.35	0.4332
222	11.0688	0.4156	1.788	10.773	3.98	0.413
223	12.8474	0.4118	1.277	5.501	3.27	0.411
224	10.9414	0.4105	1.902	11.694	4.02	0.4121
225	7.792	0.4698	3.646	29.985	5.39	0.4742
533	11.1634	0.4087	1.297	7.56	3.76	0.4063
534	8.2276	0.4284	3.215	27.597	4.86	0.4308
433	10.6754	0.4147	2.232	15.191	4.31	0.4157
434	8.036	0.4412	3.718	30.808	5.23	0.4436
113	4.2296	0.5337	0.986	5.03	4.26	0.5221
114	6.7008	0.5743	1.868	7.947	4.18	0.5782
121	4.6072	0.7727	3.575	22.725	7.81	0.7599
122	7.9232	0.8009	2.752	15.31	7.57	0.7598
123	19.2588	0.6014	2.365	9.991	4.15	0.6143
124	22.779	0.5775	1.57	5.82	3.42	0.5521
125	13.3624	0.5584	1.696	6.516	3.89	0.5385
222	28.1954	0.2804	-1.527	3.853	1.42	0.2604
224	32.5126	0.2787	-1.283	3.734	1.41	0.2799
225	38.935	0.2585	-1.107	3.593	1.44	0.2568
226	32.0398	0.2385	-0.843	3.9	1.5	0.2385
211	46.65	0.3073	-0.888	3.137	1.63	0.308
121	36.1548	0.3519	-1.09	3.244	1.55	0.3528
122	34.9376	0.3723	-1.001	3.021	1.6	0.3715
123	38.1808	0.3359	-1.026	3.112	1.52	0.3368
113	27.1212	0.3493	-1.028	3.059	1.55	0.3518
221	47.5672	0.3151	-0.818	3.01	1.64	0.3156
121	4.9994	0.3748	1.394	6.235	2.8	0.3951
122	9.0418	0.5798	0.427	1.674	3.32	0.4295
123	15.6612	0.4396	1.374	7.218	3.83	0.4611
124	20.3042	0.4739	1.065	4.883	3.15	0.4636
125	21.0102	0.4184	0.043	2.651	2.86	0.4191
114	16.6656	0.6048	2.053	15.77	6.6	0.6028
115	10.775	0.7507	1.332	5.764	5.57	0.6954

in the form of a histogram in Fig. 3.1. The mode of this distribution falls between 0.40 and 0.45. The distribution is nearly symmetrical with a range from 0.2 to 0.8. The significance of this wide range in coefficients of variation will be noted subsequently in the discussion of the choice of specific probability density function.

The ability of various density functions to match skew and kurtosis was discussed in detail in the interim report (3).

Corresponding data for the Camp Creek (II) Bridge, both phases, and the Green River Bridge are presented in Table 3.2, using the same format as Table 3.1. Because of the smaller number of truck crossing events represented, this data was not merged into Table 3.1. These results are seen to be much the same as discussed above, except for more instances of near zero or slightly negative skew.

3.3 Beta Distribution

For reasons which have been noted previously, and based on studies reported in the interim report, major emphasis will be placed on the use of the beta distribution for describing the stress range data. The beta probability density function is taken in the form

$$f(s) = \frac{\Gamma(Q + R)}{\Gamma(Q) \quad (R)} (x-a)^{Q-1} \; (b-x)^{R-1}$$

where $\Gamma(--)$ denotes the gamma function given by

$$\Gamma(z) = \int_0^\infty t^{z-1} \, e^{-t} \, dt$$

The gamma function is tabulated in standard mathematical handbooks, but is readily evaluated using a simple recurrence formula and a series approximation for $\Gamma(z + 1)$; the numerical evaluation is readily made with

TABLE 3.2 SUMMARY OF STRESS RANGE HISTOGRAM STATISTICS FOR CAMP CREEK(II) AND GREEN RIVER BRIDGES

GAGE	MEAN (MPa)	C.O.V.	SKEW	KURTOSIS	MAX/MN	C.O.V.-GRP
121	3.7196	0.5046	2.824	16.51	4.3	0.5024
122	4.2138	0.5232	2.493	11.193	3.8	0.5321
123	8.4376	0.6691	2.581	11.692	4.5	0.6548
124	18.9176	0.4459	0.217	2.884	2.43	0.4499
125	19.9638	0.4384	-0.05	1.933	1.9	0.4413
126	15.2548	0.4596	0.214	2.157	2.1	0.4624
127	6.4358	0.3031	0.23	2.899	1.86	0.326
114	6.0744	0.4173	1.01	4.499	2.63	0.4223
115	9.9388	0.5385	0.385	1.816	2.21	0.5359
224	17.0372	0.3839	-0.031	2.777	2	0.3877
121	3.4418	0.4085	1.833	7.379	2.91	0.4041
122	4.4558	0.4549	1.174	4.279	2.69	0.4541
123	10.0512	0.4301	1.546	6.926	2.79	0.4384
124	17.4046	0.3184	-0.032	2.048	1.72	0.3194
125	15.5534	0.3616	-0.117	1.92	1.8	0.3553
126	14.0372	0.3856	-0.178	1.627	1.57	0.3743
127	7.3096	0.3213	0.413	4.56	2.19	0.3442
114	6.4976	0.3409	0.055	2.168	1.85	0.3418
115	10.3442	0.3598	-0.18	1.751	1.74	0.356
224	17.862	0.3555	-0.212	1.788	1.68	0.3625
121	14.3984	0.7875	1.133	3.652	3.47	0.7896
122	18.5016	0.7111	0.896	3.448	3.13	0.7129
123	15.1924	0.6095	0.412	2.388	2.76	0.6131
124	21.336	0.6742	0.782	2.701	2.91	0.6755
125	19.5496	0.8141	1.264	3.807	3.68	0.8186
126	19.0456	0.7856	1.161	3.746	3.47	0.7837
127	8.3046	0.6715	0.795	3.202	2.89	0.6832
114	21.4756	0.6556	0.775	2.692	2.89	0.6561
115	15.9612	0.7353	1.11	3.321	3.26	0.7383

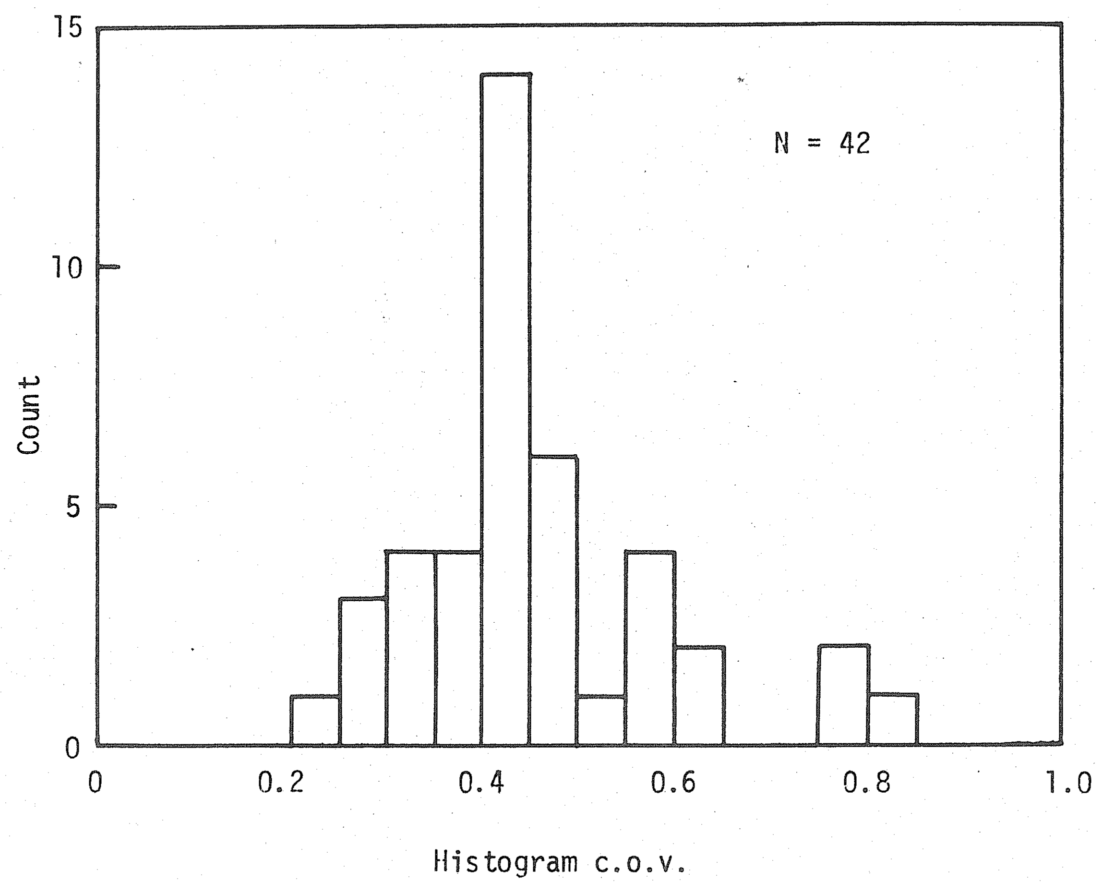

Fig. 3.1 Distribution of Histogram Coefficient of Variation
42 Data Sets

a programmable pocket calculator. Thus, the beta function poses no difficulties for use in fatigue analysis. The quantities a and b are the lower and upper limits of the distribution; the lower limit, a, is taken as zero herein.

The Q and R distribution parameters obtained in fitting the beta distribution to the 42 histograms represented by the information in Table 3.1 are summarized in Table 3.3, with the same order of presentation as in Table 3.1. The gage locations and designations and bridges are identical. Table 3.3 represents a wide variation in shape of the beta density functions; several sample plots are shown in Fig. 3.2.

The criteria for matching the beta distributions to the histograms was to fit identically the distribution mean stress and coefficient of variation, and to equate b to the maximum observed stress (strain) range. It may be readily shown (2) that these matching criteria lead to a simple formulation for the distribution parameters Q and R which describe the beta function. In addition, in Table 3.3 values of skew and kurtosis calculated from the beta function as fitted are summarized. These also are explicit functions of Q, R, and coefficient of variation. Little emphasis should be placed on the skew and kurtosis since, although the matches of mean and variance are exact, the values of skew and kurtosis based on the beta distribution are a poor match for the corresponding values for histograms; this may be seen by comparing the appropriate columns in Tables 3.1 and 3.3. Indeed it is seen that although the beta distribution is characterized by four independent parameters, minimum value, maximum value, Q and R, it does not have sufficient freedom to match all of the first four statistical moments. However, it should be noted with emphasis that there is no difficulty

TABLE 3.3 SUMMARY TABULATION OF BETA DISTRIBUTION PARAMETERS
42 DATA SETS AS IN TABLE 3.1

MEAN	MAX	C.O.V.	SKEW	KURTOSIS	Q	R	GAGE
7.48	32	0.399	0.529	3.136	4.58	15.021	221
11.26	32	0.452	0.373	2.644	2.814	5.187	222
14.35	38	0.464	0.323	2.519	2.514	4.145	223
11.79	34	0.45	0.381	2.667	2.876	5.415	224
7.43	32	0.452	0.593	3.17	3.534	11.683	225
12.13	34	0.456	0.364	2.619	2.741	4.943	533
8.52	32	0.424	0.508	3.018	3.807	10.498	534
11.6	32	0.441	0.342	2.617	2.922	5.138	433
8.24	22	0.441	0.317	2.571	2.843	4.753	434
8.23	44	0.439	0.647	3.358	4.037	17.547	221
11.07	44	0.416	0.522	3.075	4.082	12.143	222
12.85	42	0.412	0.429	2.868	3.788	8.595	223
10.94	44	0.411	0.52	3.085	4.21	12.719	224
7.79	42	0.47	0.691	3.409	3.505	15.389	225
11.16	42	0.409	0.492	3.019	4.129	11.406	533
8.23	40	0.428	0.606	3.267	4.122	15.918	534
10.68	46	0.415	0.55	3.15	4.234	14.011	433
8.04	42	0.441	0.644	3.343	3.962	16.746	434
4.23	18	0.534	0.68	3.195	2.451	7.979	113
6.7	28	0.574	0.713	3.182	2.067	6.569	1[illegible]
4.61	36	0.773	1.212	4.593	1.333	9.08	121
7.92	60	0.801	1.237	4.619	1.221	8.027	122
19.26	80	0.601	0.737	3.179	1.859	5.863	123
22.78	78	0.577	0.596	2.829	1.831	4.438	124
13.36	52	0.558	0.659	3.062	2.126	6.148	125
28.2	40	0.28	-0.655	2.738	3.05	1.277	222
32.51	46	0.279	-0.662	2.751	3.069	1.273	224
38.94	56	0.259	-0.575	2.737	3.864	1.694	225
32.04	48	0.239	-0.431	2.695	5.177	2.579	226
46.65	76	0.307	-0.315	2.439	3.476	2.187	211
36.15	56	0.352	-0.472	2.349	2.216	1.216	121
34.94	56	0.372	-0.399	2.256	2.09	1.26	122
38.18	58	0.336	-0.511	2.424	2.37	1.23	123
27.12	42	0.349	-0.47	2.357	2.258	1.239	113
47.57	78	0.315	-0.307	2.414	3.321	2.125	221
5	14	0.375	0.309	2.729	4.218	7.595	121
9.04	30	0.58	0.576	2.767	1.777	4.118	122
15.66	60	0.44	0.532	3.039	3.563	10.089	123
20.3	64	0.474	0.46	2.771	2.719	5.851	124
21.01	60	0.418	0.353	2.696	3.362	6.238	125
16.67	110	0.605	0.933	3.883	2.168	12.141	114
10.78	60	0.751	1.044	3.879	1.276	5.83	115

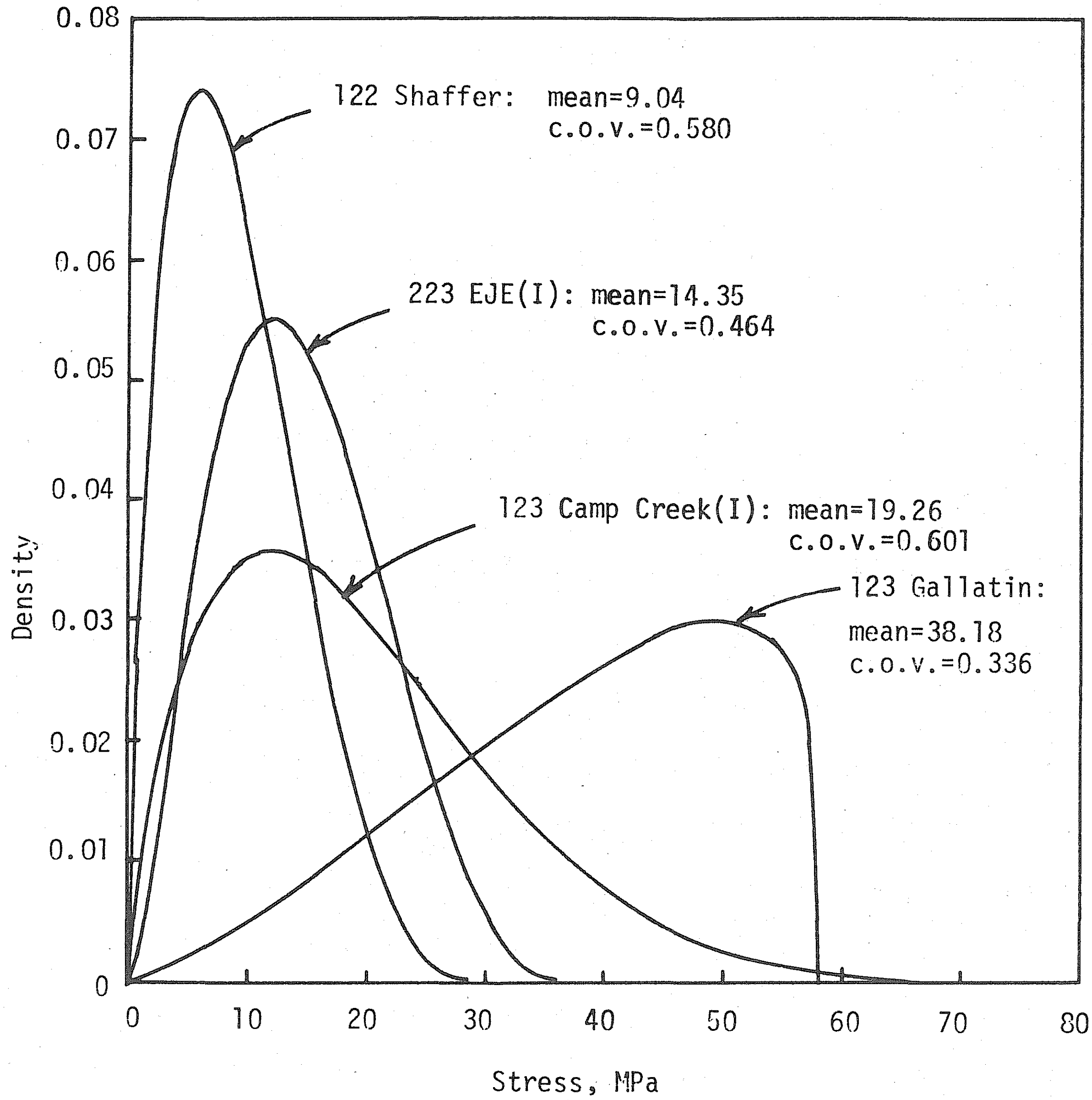

Fig. 3.2 Comparison of Selected Beta Density Functions Fitted to Various Histograms

whatsoever in matching mean, maximum and coefficient of variation for this data. The skew and kurtosis and are not useful measures of engineering significance. It was seen also that statistical tests of goodness-of-fit of a distribution, i.e. Chi-square and Kolmogorov-Smirnov, were not helpful.

3.4 Prediction of $E(S^m)$ using the Beta Density Function

The significant property of the stress-range data is the quantity $E(S^m)$; it should be the primary object of any test of the adequacy of a probability density function model. Herein the beta distribution will be tested by taking the ratio of the predicted value of $E(S^m)$ to the value calculated from the histogram data, using the grouped data:

$$E(S^m) = \sum_{j=1}^{N} S_j^m \frac{n_j}{N}$$

where S_j is the average stress range in the jth histogram stress interval, n_j is the count in the j-th interval and N is the total data count for the histogram.

For all histograms presented for comparison herein, the beta distribution is fitted by matching the mean stress level, variance, and the apparent maximum stress level, as discussed previously. To test the beta-model, again the 42 sets of data presented in Table 3.1 are used.

The value of $E(S^m)$ for the beta-function is calculated from the expression(5):

$$E(S^m) = s_o^m \frac{\Gamma(m+Q)\ \Gamma(Q+R)}{\Gamma(Q)\ \Gamma(m+Q+R)}$$

where Q, R, and s_o are the parameters of the beta function (s_o is the maximum stress). The calculation proceeds using a series approximation and recurrence formula for the gamma function as needed in the above expression. The Q, R, and s_o (max) values are given in Table 3.3.

The calculated values of $E(S^m)$ based on the histogram data, and corresponding values obtained using the beta function are shown in Table 3.4. Also included in this tabulation are the mean stress, c.o.v. and the ratio of the beta model prediction to the histogram calculation of $E(S^m)$. The tabulated results are presented for the fatigue parameter m = 3, only. However, the comparison was investigated for a range of m-values from 2.75 to 5. The errors in the use of the beta function for calculating $E(S^m)$ are seen to increase with increasing m-value. The mean error ratios, the standard deviation of the error ratios, and the c.o.v. of the error ratios are tabulated as function of the m-value below:

m	Mean Error Ratio	Std. Dev. of Error Ratio	c.o.v. of Error Ratio
2.75	0.9952	0.0711	0.071
3	0.9794	0.0926	0.095
3.5	0.9372	0.1473	0.157
4	0.8859	0.2101	0.237
4.5	0.8324	0.2718	0.327
5	0.7820	0.3266	0.418

From the above tabulation, the beta model prediction of $E(S^m)$ is seen to fall, on the average, below the value calculated from the histogram data and the variance associated with the average error ratio has a coefficient of variation ranging from 0.10 to over 0.40. The larger variance corresponds to m = 5. If one restricts the range of the fatigue parameter

TABLE 3.4 COMPARISON OF E(S**m) VALUES CALCULATED WITH BETA MODEL WITH HISTOGRAM DATA; m = 3

E(S**M:H)	E(S**M:BTMOD)	MEAN (MPa)	C.O.V.	BTMOD/H
640	631	7.48	0.399	0.985
2421	2351	11.26	0.452	0.97
4684	4954	14.35	0.464	1.057
2779	2694	11.79	0.45	0.969
741	683	7.43	0.452	0.921
3019	2957	12.13	0.456	0.979
931	975	8.52	0.424	1.047
2454	2515	11.6	0.441	1.025
858	899	8.24	0.441	1.048
1035	909	8.23	0.439	0.878
2217	2109	11.07	0.416	0.951
3376	3262	12.85	0.412	0.966
2149	2019	10.94	0.411	0.939
970	820	7.79	0.47	0.844
2196	2135	11.16	0.409	0.972
1007	890	8.23	0.428	0.883
2040	1891	10.68	0.415	0.927
991	850	8.04	0.441	0.858
144	148	4.23	0.534	1.028
707	639	6.7	0.574	0.904
427	327	4.61	0.773	0.767
2046	1770	7.92	0.801	0.865
18904	16037	19.26	0.601	0.848
25063	25002	22.78	0.577	0.997
5100	4891	13.36	0.558	0.959
22266	27376	28.2	0.28	1.229
41600	41881	32.51	0.279	1.006
69528	70270	38.94	0.259	1.01
38296	38311	32.04	0.239	1
127922	129351	46.65	0.307	1.011
62580	63846	36.15	0.352	1.02
57914	59498	34.94	0.372	1.027
72277	73420	38.18	0.336	1.015
26352	26850	27.12	0.349	1.018
137273	138644	47.57	0.315	1.01
183	179	5	0.375	0.979
1209	1567	9.04	0.58	1.296
6573	6241	15.66	0.44	0.949
14498	14420	20.3	0.474	0.994
14157	14385	21.01	0.418	1.016
11772	10663	16.67	0.605	0.905
3686	3918	10.78	0.751	1.063

m to those values near 3 then a coefficient of variation of about 10% is a reasonable choice to describe the uncertainty in the prediction using the model. Of course, there is also an average bias to the low side, i.e., an error ratio which is less than 1, to be associated with this model.

3.5 Truncated Rayleigh Distribution

A truncated Rayleigh density function was used by Schilling, et. al., (11) to represent stress range data in highway bridges and to define a random loading sequence for laboratory tests. It consists of the Rayleigh function, which has an infinite upper limit, truncated so that the upper limit stress is three times the mode of the distribution; the lower limit stress is taken as zero. For a non-zero lower limit stress the entire distribution can be shifted without a change in function shape. The truncated Rayleigh distribution is shown in Fig. 3.3. Also shown in the figure is a beta density function with matching values of mean, variance, lower and upper limits. The stress levels are expressed in arbitrary units. The significant parameters of the two distributions are as follows:

Parameter	Truncated Rayleigh	Beta
Mean Stress	1.230	1.230
Mode Stress	1.000	1.026
Maximum Stress	3.000	3.000
Minimum Stress	0.0	0.0
c.o.v.	0.505	0.505
Skew Coeff.	0.431	0.270
RMS Stress	1.378	1.378

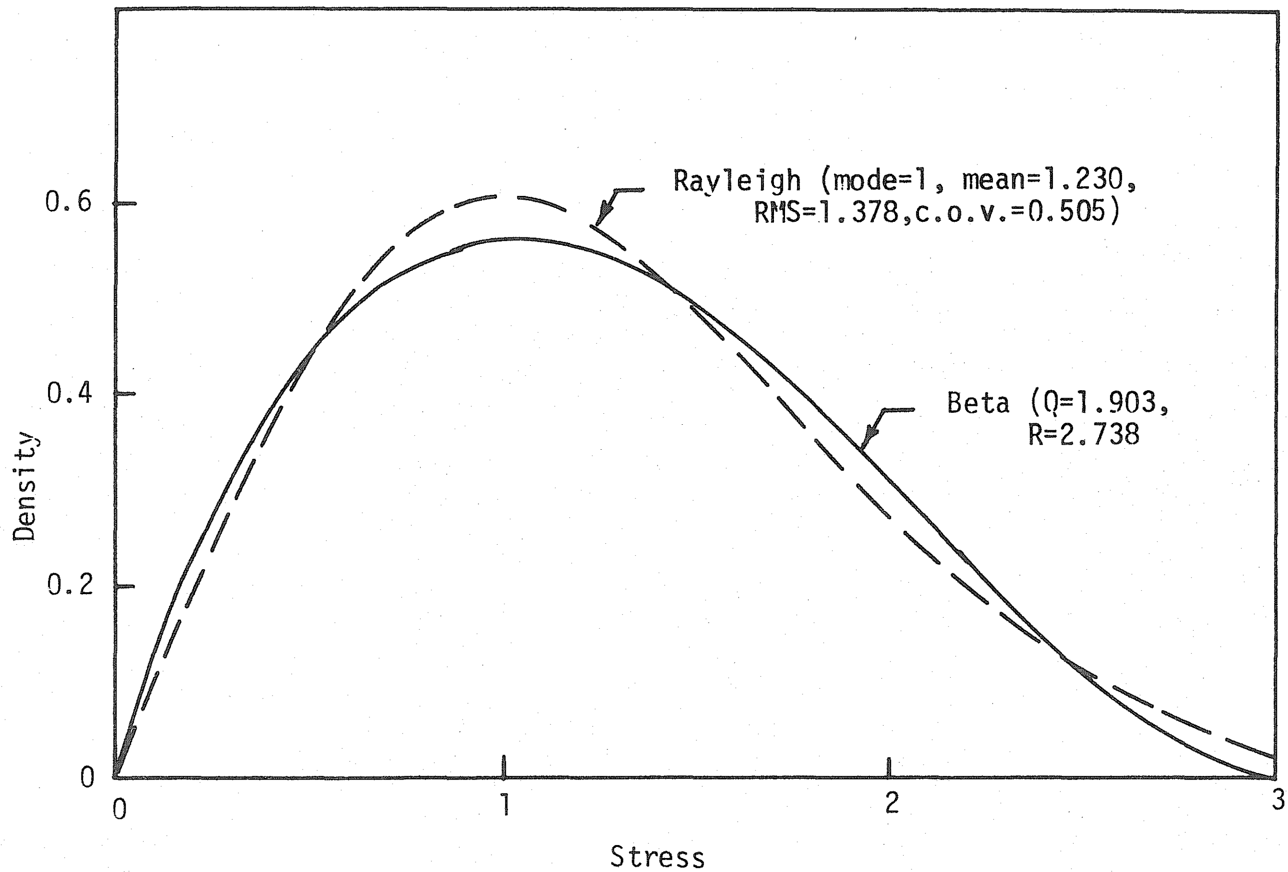

Fig. 3.3 Comparison of Truncated Raleigh and Beta Density Functions

Both distributions have potential for being useful for predicting $E(S^m)$ provided that the mean and variance of the data to be modeled are matched. However, the Rayleigh distribution has fixed values of coefficient of variation and skewness and will be less flexible in application to the analysis of histogram data.

The effect of a failure to match variance (c.o.v.) may be illustrated by comparing the predicted values of $E(S^m)$ for the Rayleigh and beta functions. For the comparison, the mean, maximum and minimum stress values are matched and, by changing beta parameters Q and R, values of coefficients of variation of 0.3, 0.4, 0.6, and 0.505 are taken for the beta distribution. The latter value, 0.505, represents the exact match of variance between the beta and truncated Rayleigh distributions. Computed data for $E(S^m)$ and a random stress analysis factor will be used to compare the two density functions.

These comparisons are presented in Table 3.5 in which results for values for m = 3, 4 and 5 are given; shown are computed values of $E(S^m)$ expressed in terms of the appropriate power of arbitrary stress units, e. g., the maximum stress is 3 units, means is 1.230 units, etc. Also tabulated are values of RSAF, random stress analysis factor, which will be defined and discussed in Chapter 4.

Consider first the comparisons of $E(S^m)$ for c.o.v. = 0.505, the most favorable condition for matching the distributions:

m-Value	Ratio: [Rayleigh $E(S^m)$]/[Beta $E(S^m)$]
3	1.04
4	1.04
5	1.08

Table 3.5 - Comparison of Predicted Values of $E(S^m)$ for Various Values of C.O.V.

min. = 0 max. = 3 mean = 1.230

m	C.O.V.	Q R	Beta RSAF	Rayleigh RSAF	Beta $E(S^m)$	Rayleigh $E(S^m)$
3	0.3	6.146 8.844	1.199		2.374	
	0.4	3.328 4.716	1.225		2.778	
	0.505	1.903 2.738	1.237	1.220	3.350	3.494
	0.6	1.229 1.768	1.241		3.987	
4	0.3	Same	1.159		3.619	
	0.4	↓	1.166		4.760	
	0.505	↓	1.162	1.151	6.443	6.695
	0.6	↓	1.155		8.442	
5	0.3	Same	1.125		5.796	
	0.4	↓	1.118		8.663	
	0.505	↓	1.105	1.088	13.223	14.227
	0.6	↓	1.093		18.934	

From the above it is seen that there is no more than an 8 percent difference in the prediction of $E(S^m)$ and thus of mean fatigue life, even at the more extreme value of m = 5. It should be noted that many fatigue designs may be adequately analyzed, i.e., conservatively represented, for fatigue behavior with m values near 3.

Consider next similar comparisons for the other values of c.o.v. for the Beta functions:

m-Value	[Rayleigh $E(S^m)$] / [Beta $E(S^m)$]		
	C.O.V. = 0.3	C.O.V. = 0.4	C.O.V. = 0.6
3	1.47	1.26	0.88
4	1.85	1.41	0.79
5	2.45	1.64	0.75

From the above, major differences are seen to occur in the prediction of $E(S^m)$ when the match of variance is not close; the differences increase with increasing m-value. However, it should be noted that the usefulness of the Rayleigh distribution can be extended by an appropriate shift in the minimum, mean and maximum stress values to compensate. Since such a shift will retain the shape of the function, the ratio of shift, C, can be estimated and for the larger m-values will be relatively small:

$$C = \left(\frac{Max^{Shifted}}{Max,\ m=3}\right) = \left(\frac{1}{Error\ Ratio}\right)^{\frac{1}{m}}$$

Thus for C.O.V. = 0.3 and m = 3; $C = \left(\frac{1}{1.47}\right)^{1/3} = 0.88$, and for C.O.V. = 0.3 and m = 5; $C = \left(\frac{1}{2.45}\right)^{1/5} = 0.84$. Thus, a modest shift of the maximum value on the order of 12-16% will bring the Rayleigh distribution into agreement with the beta prediction, and based on tests of the beta function in representing data herein, it should be possible to use the Rayleigh distribution. However, no routine basis for making this shift for the Rayleigh using statistics of mean and variance is readily available.

Truncated Rayleigh Functions and Histogram Data

In a manner parallel to the use of the beta distribution in Section 3.4, one may explore the use of the truncated Rayleigh distribution to predict $E(S^m)$ for the data of Table 3.1. This will be presented for only m = 3. Since the Rayleigh distribution has a constant coefficient of variation and a fixed ratio of the maximum stress to the mean or mode stress, the match to the experimental histograms will be made on the basis of mean stress of the histogram. The resulting predictions for $E(S^m)$ based on this matching are given in Table 3.6. In Table 3.6 results for $E(S^m)$ are shown for both the truncated Rayleigh and the corresponding results for the beta distribution, as well as the corresponding calculated value taken from the histogram data. In the last column of the table, the symbol RAY/H denotes the error ratio and is taken with respect to the histogram data calculation of $E(S^m)$. The mean error ratio is 1.0895 with a standard deviation of 0.289. The corresponding coefficient of variation is 0.265.

Thus it is seen that the uncertainty in the use of the Rayleigh model in terms of coefficient of variation for error ratio is substantially larger than that for the beta distribution model. As noted previously, the truncated Rayleigh could yield improved results by slight shifts in the function relative to the mean and maximum values. It is more convenient to use a density function with sufficient degrees of freedom to permit match of both mean and variance.

TABLE 3.6 COMPARISON OF E(S**m) VALUES PREDICTED USING TRUNCATED RAYLEIGH WITH HISTOGRAM DATA; m = 3

E(S**M:H)	E(S**M:BTMOD)	E(S**M)-RAY	MEAN(MPa)	RAY/H
640	631	751	7.4774	1.173
2421	2351	2564	11.2556	1.059
4684	4954	5309	14.3458	1.133
2779	2694	2949	11.793	1.061
741	683	738	7.4314	0.994
3019	2957	3208	12.1286	1.062
931	975	1110	8.5166	1.193
2454	2515	2807	11.601	1.143
858	899	1004	8.2352	1.17
1035	909	1002	8.2296	0.968
2217	2109	2438	11.0688	1.099
3376	3262	3813	12.8474	1.129
2149	2019	2355	10.9414	1.095
970	820	850	7.792	0.876
2196	2135	2501	11.1634	1.139
1007	890	1001	8.2276	0.993
2040	1891	2187	10.6754	1.072
991	850	933	8.036	0.941
144	148	136	4.2296	0.944
707	639	541	6.7008	0.765
427	327	175	4.6072	0.411
2046	1770	894	7.9232	0.437
18904	16037	12845	19.2588	0.679
25063	25002	21254	22.779	0.848
5100	4891	4290	13.3624	0.841
22266	27376	40307	28.1954	1.81
41600	41881	61802	32.5126	1.485
69528	70270	106137	38.935	1.526
38296	38311	59145	32.0398	1.544
127922	129351	182559	46.65	1.427
62580	63846	84986	36.1548	1.358
57914	59498	76683	34.9376	1.324
72277	73420	100088	38.1808	1.384
26352	26850	35873	27.1212	1.361
137273	138644	193540	47.5672	1.409
183	179	224	4.9994	1.225
1209	1567	1329	9.0418	1.099
6573	6241	6907	15.6612	1.05
14498	14420	15052	20.3042	1.038
14157	14385	16677	21.0102	1.178
11772	10663	8323	16.6656	0.707
3686	3918	2249	10.775	0.61

4. RANDOM STRESS ANALYSIS FACTOR

4.1 General

In Chapter 3, the use of a density function model for stress range for the calculation of $E(S^m)$, the beta distribution being preferred, has been demonstrated. The values of $E(S^m)$ so calculated are, on the average, below the values calculated directly from the histogram for measured stress ranges, although by a small amount. Plots of various density functions (beta, Raleigh, lognormal, etc.) compared with measured histograms (2) show considerable agreement, but significant differences between the model and measured data. In some instances the predicted value of $E(S^m)$ based on a fitted density function is adequate because even though the density function underestimates some important higher stress ranges, it may compensate with an over estimate of lesser stress range values. In short, the correct result for $E(S^m)$ is sometimes obtained for the wrong reasons.

Furthermore, it is seen that even a versatile, four parameter density function model such as the beta function lacks significant range in the first four statistical moments to match simultaneously the mean, variance, skew, and kurtosis. A bimodal density function, e.g. constructed from two or more beta functions, or others, may well be needed for a more precise match of the histogram data.

Thus, a scheme for calculating $E(S^m)$ which is independent of a probability density function model would be attractive. Any proposed scheme must fit the field data adequately, and be demonstrated to be substantially independent of both histogram shape (or probability density function). Such a scheme will be presented in this chapter.

4.2 Random Stress Analysis Factor

The objective of this section is to present the experimental evidence for an alternative representation of the quantity $E(S^m)$. Depending on the m-value and the general stress levels, the numerical value of $E(S^m)$ can vary widely, such that the variation offers little aid to the intuitive judgment of the analyst. A method for normalization of $E(S^m)$ is very useful.

In the work by Ang and Munse (6) on a design formulation for reliability approach to fatigue, $E(S^m)$ is equated with the m-th power of a reduced critical design stress. This critical stress is one which is related to the constant-cycle fatigue behavior of material and is reduced by a factor which represents the influence of random stress range effects, in contrast to constant stress range fatigue behavior. The formulation results in the following:

$$\left(\frac{S_c}{\xi_c}\right)^m = E(S^m)$$

In the above, S_c is the critical stress level, ξ_c is the reduction factor corresponding to this critical stress and as before, m is the slope of the conventional S-N fatigue diagram. The numerical value of ξ_c thus depends on the value of S_c chosen to represent the stress data, that is:

$$c = \frac{S_c}{[E(S^m)]^{\frac{1}{m}}} \qquad (4.1)$$

The first step in seeking an alternate representation of $E(S^m)$ was to study the variation in ξ_c, redefined for an analysis role rather than as a design parameter, for various definitions of the characteristic stress, S_c.

A perhaps obvious choice for S_c is simply the maximum stress observed in the data, or perhaps the mean stress. Other possibilities were drawn observing the data; it might be reasonable to select an upper limit stress at either a specified percentile exceedance level, at a prescribed number of standard deviations from the mean, or at a prescribed ratio of critical stress to the mean stress.

In the present study the following S_c definitions were used:

a) S_c = mean stress

b) S_c = mean stress plus one standard deviation

c) S_c = mean stress plus two standard deviations

d) S_c = maximum stress observed in histogram data.

Using the S_c values defined above, ξ_c values to be interpreted now as random stress analysis factors (RSAF) were calculated for a set of 42 histograms, representing the various gage locations and selected bridges described in Chapter 3. This aggregation of data represents a variety of histogram shapes, mean stress levels, coefficients of variation and maximum values of stress range. If such a diverse set of histogram data can yield a mathematically well behaved and useful random analysis factor, then extrapolation to other bridge types and traffic situations can be made.

The calculated random analysis factors, RSAF values, were studied statistically to define the average RSAF and the corresponding variance. A linear regression analysis was made to show the variation of RSAF with the mean stress of the distribution and also to show the change of the mean RSAF levels as a function of the fatigue parameter, m.

The calculation of the RSAF parameter by Eq. 4.1 was carried out using values of $E(S^m)$ calculated on the basis of the m-th moment of the grouped histogram data; this procedure was adopted to avoid repeated and expensive manipulation of the original stress range data set from which the histograms were drawn, which would be necessary for each new value of m. As noted in Chapter 3, the errors introduced by considering the grouped data rather than point estimates are not significant.

A set of calculated RSAF values for the set of 42 gages, per Table 3.1, is presented in Tables 4.1 through 4.3. These tables represent about 8,000 stress events and are presented for m-values of 3, 4, and 5.

Considering in more detail the data for m = 3 presented in Table 4.1a, and treating the aggregated results without regard to the possible dependence on the mean stress level or coefficient of variation, there are significant trends in the data which are emphasized in summary form in Table 4.4. In Table 4.4 the various definitions of S_c, denoted by a, b, c, and d, represent respectively the four definitions of S_c indicated previously. The table includes information on the minimum, average, and maximum RSAF, and the coefficient of variation for RSAF. In addition, the last column in the table shows the range of the data for maximum to minimum expressed in terms of the mean level of RSAF. In reviewing the results, it should be remembered that the definition of S_c denoted c), the mean plus two standard deviations, may yield a S_c which is larger than the observed maximum stress; for that reason emphasis is placed on definition b), that is, S_c = mean stress plus one standard deviation. These results are confirmed for the data from Camp Creek (II) and Green River Bridges shown in Table 4.1b.

To varying extents the RSAF data presented in Tables 4.1 through 4.3 by inspection appears to be dependent upon the mean stress level of the

TABLE 4.1a SUMMARY OF RANDOM STRESS ANALYSIS FACTORS --Various Definitions of S_c (Characteristic Stress), m = 3

MEAN STRESS MPA	RSAF(MN)	RSAF(1SD)	RSAF(2SD)	RSAF(MAX)
7.48	0.867	1.213	1.559	3.711
11.26	0.838	1.217	1.597	2.383
14.35	0.857	1.255	1.653	2.271
11.79	0.839	1.216	1.594	2.418
7.43	0.821	1.192	1.562	3.535
12.13	0.839	1.222	1.604	2.352
8.52	0.872	1.242	1.613	3.277
11.6	0.86	1.239	1.618	2.372
8.24	0.867	1.249	1.631	2.315
8.23	0.813	1.17	1.527	4.349
11.07	0.849	1.202	1.554	3.374
12.85	0.856	1.209	1.562	2.8
10.94	0.848	1.196	1.544	3.409
7.79	0.787	1.157	1.526	4.241
11.16	0.859	1.21	1.561	3.231
8.23	0.821	1.172	1.524	3.99
10.68	0.842	1.191	1.54	3.626
8.04	0.806	1.162	1.517	4.212
4.23	0.807	1.238	1.668	3.434
6.7	0.752	1.184	1.616	3.143
4.61	0.612	1.085	1.557	4.78
7.92	0.624	1.124	1.624	4.726
19.26	0.723	1.158	1.592	3.003
22.78	0.778	1.228	1.677	2.665
13.36	0.776	1.21	1.643	3.021
28.2	1.002	1.283	1.564	1.422
32.51	0.938	1.2	1.461	1.328
38.94	0.947	1.192	1.436	1.362
32.04	0.951	1.177	1.404	1.424
46.65	0.926	1.21	1.495	1.508
36.15	0.911	1.231	1.552	1.411
34.94	0.903	1.239	1.575	1.447
38.18	0.917	1.224	1.532	1.392
27.12	0.911	1.23	1.548	1.411
47.57	0.922	1.213	1.503	1.512
5	0.88	1.21	1.54	2.464
9.04	0.849	1.341	1.833	2.816
15.66	0.836	1.204	1.571	3.203
20.3	0.833	1.227	1.622	2.625
21.01	0.868	1.232	1.595	2.48
16.67	0.733	1.176	1.619	4.836
10.78	0.698	1.221	1.745	3.884

TABLE 4.1b SUMMARY OF RANDOM STRESS ANALYSIS FACTORS FOR CAMP CREEK AND GREEN RIVER BRIDGES -- m = 3

MEAN STRESS MPA	RSAF(MN)	RSAF(1SD)	RSAF(2SD)	RSAF(MAX)
3.72	0.779	1.172	1.565	3.351
4.21	0.774	1.178	1.583	2.937
8.44	0.694	1.158	1.623	3.125
18.92	0.856	1.238	1.619	2.081
19.96	0.859	1.236	1.613	1.635
15.25	0.849	1.239	1.629	1.78
6.44	0.919	1.197	1.476	1.713
6.07	0.856	1.213	1.57	2.255
9.94	0.809	1.245	1.681	1.791
17.04	0.89	1.232	1.573	1.776
3.44	0.835	1.176	1.518	2.427
4.46	0.85	1.237	1.624	2.29
10.05	0.837	1.197	1.557	2.332
17.4	0.924	1.218	1.513	1.593
15.55	0.901	1.227	1.554	1.623
14.04	0.892	1.236	1.581	1.399
7.31	0.907	1.199	1.49	1.986
6.5	0.913	1.224	1.535	1.686
10.34	0.903	1.227	1.552	1.571
17.86	0.899	1.219	1.538	1.51
14.4	0.665	1.189	1.712	2.309
18.5	0.706	1.207	1.709	2.212
15.19	0.769	1.238	1.707	2.126
21.34	0.728	1.218	1.709	2.114
19.55	0.648	1.175	1.702	2.385
19.05	0.666	1.189	1.712	2.307
8.3	0.728	1.217	1.706	2.104
21.48	0.737	1.22	1.703	2.128
15.96	0.689	1.195	1.702	2.244

TABLE 4.2 SUMMARY OF RANDOM STRESS ANALYSIS FACTORS --Various Definitions of S_c (Characteristic Stress), m = 4

MEAN STRESS MPA	RSAF(MN)	RSAF(1SD)	RSAF(2SD)	RSAF(MAX)
7.48	0.814	1.138	1.463	3.482
11.26	0.783	1.137	1.491	2.225
14.35	0.794	1.162	1.531	2.103
11.79	0.782	1.134	1.486	2.254
7.43	0.744	1.079	1.415	3.202
12.13	0.786	1.144	1.503	2.204
8.52	0.814	1.16	1.505	3.059
11.6	0.806	1.161	1.516	2.223
8.24	0.815	1.175	1.535	2.178
8.23	0.686	0.987	1.288	3.668
11.07	0.77	1.089	1.409	3.059
12.85	0.793	1.12	1.447	2.594
10.94	0.768	1.083	1.398	3.088
7.79	0.658	0.968	1.277	3.549
11.16	0.791	1.115	1.438	2.978
8.23	0.705	1.007	1.309	3.426
10.68	0.752	1.064	1.376	3.242
8.04	0.682	0.983	1.284	3.564
4.23	0.728	1.116	1.505	3.098
6.7	0.66	1.039	1.418	2.757
4.61	0.478	0.847	1.217	3.736
7.92	0.497	0.895	1.293	3.764
19.26	0.623	0.998	1.373	2.589
22.78	0.684	1.079	1.474	2.342
13.36	0.686	1.069	1.452	2.669
28.2	0.985	1.261	1.537	1.397
32.51	0.922	1.179	1.435	1.304
38.94	0.93	1.171	1.412	1.338
32.04	0.934	1.157	1.379	1.399
46.65	0.904	1.182	1.46	1.473
36.15	0.889	1.201	1.514	1.376
34.94	0.879	1.206	1.534	1.409
38.18	0.895	1.195	1.496	1.359
27.12	0.889	1.199	1.509	1.376
47.57	0.899	1.183	1.466	1.475
5	0.816	1.122	1.428	2.285
9.04	0.785	1.241	1.696	2.606
15.66	0.763	1.098	1.433	2.922
20.3	0.763	1.124	1.486	2.404
21.01	0.826	1.172	1.518	2.36
16.67	0.612	0.983	1.353	4.043
10.78	0.597	1.044	1.492	3.322

TABLE 4.3 SUMMARY OF RANDOM STRESS ANALYSIS FACTORS --Various Definitions of S_c (Characteristic Stress), m = 5

MEAN STRESS MPA	RSAF(MN)	RSAF(1SD)	RSAF(2SD)	RSAF(MAX)
7.48	0.763	1.068	1.372	3.266
11.26	0.74	1.074	1.409	2.103
14.35	0.745	1.091	1.437	1.975
11.79	0.737	1.069	1.401	2.125
7.43	0.673	0.977	1.281	2.899
12.13	0.745	1.085	1.425	2.089
8.52	0.764	1.088	1.412	2.87
11.6	0.764	1.1	1.437	2.107
8.24	0.774	1.116	1.457	2.069
8.23	0.571	0.822	1.072	3.054
11.07	0.69	0.977	1.264	2.744
12.85	0.736	1.04	1.343	2.408
10.94	0.688	0.971	1.253	2.768
7.79	0.549	0.807	1.065	2.959
11.16	0.725	1.021	1.317	2.727
8.23	0.596	0.851	1.107	2.898
10.68	0.663	0.938	1.212	2.856
8.04	0.572	0.824	1.077	2.99
4.23	0.662	1.015	1.368	2.816
6.7	0.589	0.927	1.266	2.462
4.61	0.392	0.696	0.999	3.066
7.92	0.412	0.742	1.072	3.12
19.26	0.551	0.883	1.214	2.289
22.78	0.615	0.97	1.325	2.106
13.36	0.617	0.961	1.306	2.4
28.2	0.971	1.244	1.516	1.378
32.51	0.909	1.162	1.416	1.286
38.94	0.917	1.155	1.392	1.32
32.04	0.92	1.139	1.358	1.378
46.65	0.887	1.16	1.432	1.445
36.15	0.872	1.179	1.486	1.351
34.94	0.861	1.182	1.503	1.381
38.18	0.878	1.173	1.468	1.334
27.12	0.872	1.176	1.48	1.35
47.57	0.882	1.159	1.437	1.446
5	0.761	1.046	1.331	2.131
9.04	0.729	1.152	1.575	2.419
15.66	0.699	1.006	1.313	2.678
20.3	0.703	1.036	1.37	2.217
21.01	0.792	1.124	1.455	2.262
16.67	0.508	0.815	1.122	3.352
10.78	0.522	0.913	1.305	2.905

TABLE 4.4 STATISTICS FOR MEAN RANDOM STRESS ANALYSIS FACTORS

M-value	Definition[1] of S_c	RSAF min.	RSAF Avg.	RSAF Max.	c.o.v.	Range[2]
3	a	0.612	0.839	1.002	0.096	0.46
	b	1.085	1.209	1.341	0.034	0.
	c	1.404	1.578	1.833	0.048	0.27
	d	1.328	2.837	4.836	0.370	1.24
4	a	0.478	0.771	0.934	0.147	0.59
	b	0.847	1.106	1.261	0.082	0.37
	c	1.217	1.442	1.696	0.063	0.33
	d	1.304	2.545	4.043	0.321	1.08
5	a	0.412	0.715	0.971	0.195	0.78
	b	0.696	1.022	1.182	0.132	0.48
	c	0.999	1.330	1.575	0.107	0.43
	d	1.286	2.305	3.352	0.277	0.90

Notes: (1) a) S_c = mean stress

b) S_c = mean + 1 x Std. dev.

c) S_c = mean + 2 x std. dev.

d) S_c = max. stress

(2) Range for RSAF = (max-min)/mean

histograms. The following linear regression relationship may be formulated:

$$\text{RSAF} = \alpha + \beta \times (\text{mean stress})$$

The coefficients α and β were evaluated using standard regression techniques on the data in Table 4.1a, representing the 42 histograms and various definitions of S_c. One may also determine the conditional variance for the regression line, denoted as Var(RSAF|mean stress) = Var. The variance, Var, is assumed to be constant over the range of the regression. The quantity $\sqrt{Var}$ has the meaning of a standard deviation which is constant for all values of RSAF given by the regression equation, i.e. independent of mean stress. Considering the three definitions of S_c and m = 3, the following results are obtained:

S_c Definition	α	β	$\sqrt{Var}$
mean + (std. dev.)	1.1980	0.0006042	0.041
mean + 2(std. dev.)	1.6256	-0.002798	0.069
Maximum	3.9858	-0.06687	0.69

The first definition of S_c in the above yields an RSAF nearly independent of mean stress (small β) with a small variance. The RSAF for S_c = maximum is the least desirable with a much larger β value and a variance which is ten fold greater than for the other definitions.

In Fig. 4.3 the results tabulated above are presented graphically as the mean regression lines with scatter bands of $\pm \sqrt{Var}$. Seen in these plots the definition of S_c at the mean stress plus one standard deviation has closely banded RSAF values which are well behaved over the full range of mean stress levels. It may be shown that more scatter, i.e. larger

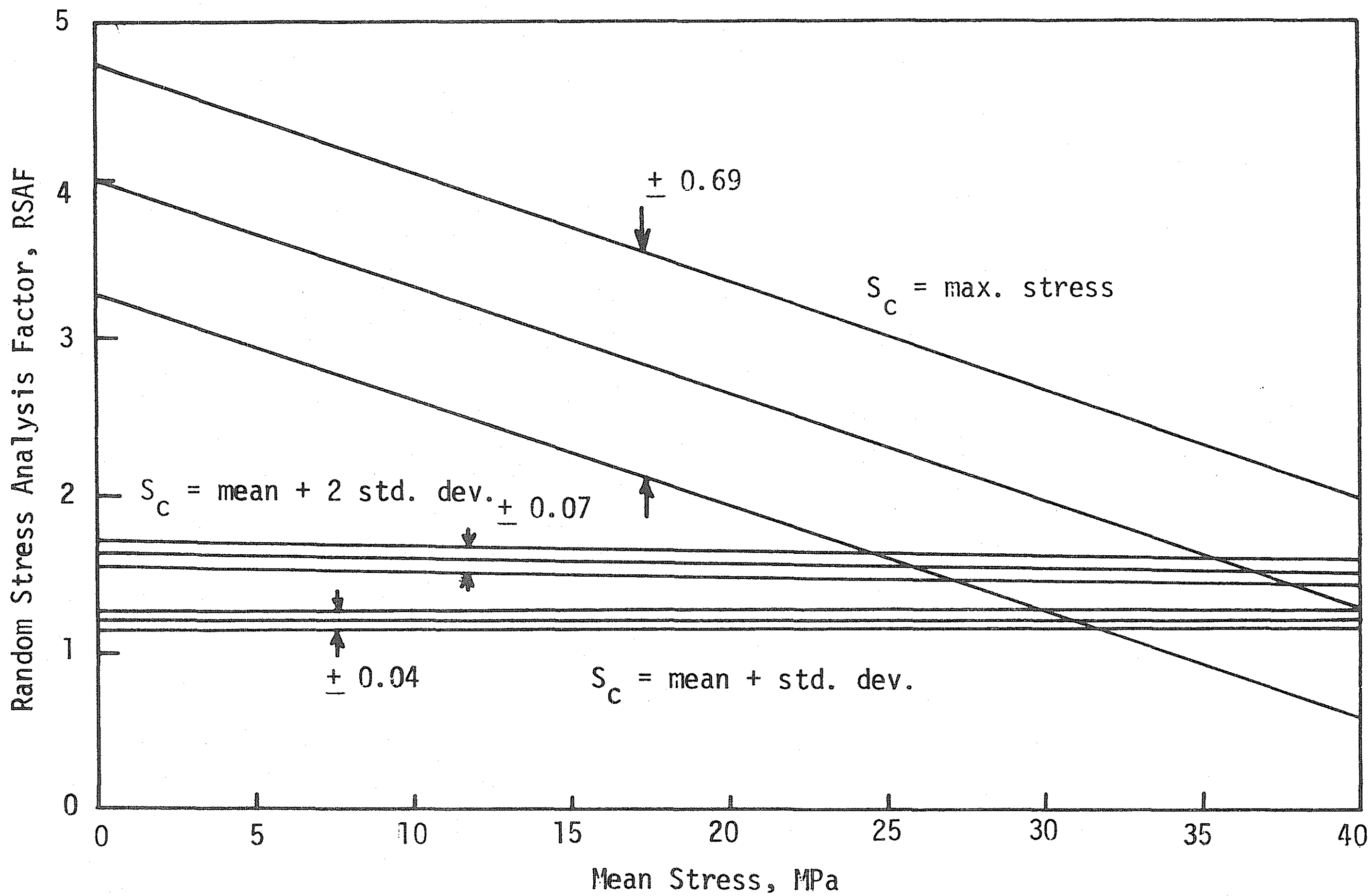

Fig. 4.3 Regression Studies of Random Stress Analysis Factor 42 Data Sets, m = 3

conditional variance, will be present for values of m = 4 or 5. Given that m = 3 represents a large number of practical fatigue problems, the RSAF based on mean plus one standard deviation seems to be a reasonable basis for the RSAF studies in the following sections.

The conclusions drawn above from histogram data have excluded the results of the measurements on the Dan Ryan Bridge. It is to be noted that the intuitive statement concerning its similarity to other locations made in the summary of the data in Chapter 2 is borne out in the detailed statistical properties and in the definition of RSAF, which for bottom flange stress, beam 5, are as follows:

Quantity	Value
Mean, MPa	9.21
c.o.v.	0.491
Skew	0.322
Kurtosis	2.446
Q,R	2.176, 3.495
RSAF (m = 3)	1.240
Count	1185

4.3 Effect of Distribution Shape and m-Value

From the results presented in Tables 4.1 to 4.3 it is seen that RSAF is a function of the fatigue parameter, m. This may be derived explicitly for a variety of simple probability density functions. For example, for uniform, right-triangular and symmetrical-triangular density functions the resultant expressions are quite simple and are summarized for S_c = (mean + std. dev.) in Table 4.5. In a similar fashion the RSAF can be derived for the lognormal distribution (which, in contrast to the above, has no upper stress bound) and for the truncated Rayleigh distribution, discussed in Section 3.6, for which the ratio of maximum stress to modal stress is three.

These various distributions are studied to illustrate the behavior of the random stress analysis factor for a wide range of probability distribution shapes and are not proposed as specific stress history models. The resulting RSAF values are shown in Table 4.6 for a range of m-values from 2 to 5, and the data is plotted in Fig. 4.4. There is a smooth variation of RSAF with m-value which, although not linear, approaches a linear variation over a limited range of m, say, between 3 and 4, which would cover a wide variety of fatigue conditions. For m = 3 the range in RSAF is quite limited, that is, between approximately 1.2 and 1.25; this is indeed a desirable result and would confirm that the random stress analysis factor is a well behaved parameter for analysis and design purposes.

The effect of m-value may also be determined from the field data used in the previous section by taking intermediate values of m and studying the average values of RSAF for the set of 42 histograms as a function of m-value. This was done and there is a smooth variation of the average RSAF with m. A linear regression fitting indicates that a good m-value relationship is as follows:

$$\text{RSAF} = 1.491 - 0.0950\ m$$

Again it should be remembered that the RSAF equation presented above is for S_c = mean + (std. dev.). This empirical relationship for RSAF was

TABLE 4.5 RSAF EXPRESSIONS FOR SIMPLE DENSITY FUNCTIONS--
S_c = mean + 1 x Std. Dev.

FUNCTION	RSAF
UNIFORM	$\dfrac{\frac{1}{2}(1 + 1/\sqrt{3})}{\left(\frac{1}{m+1}\right)^{\frac{1}{m}}}$
RIGHT TRIANGLE	$\dfrac{\frac{1}{3}(1 + 1/\sqrt{2})}{\left(\frac{2}{m+1} - \frac{2}{m+2}\right)^{\frac{1}{m}}}$
SYMMETRICAL TRIANGLE	$\dfrac{\frac{1}{2}(1 + 1/\sqrt{6})}{\left[\frac{1}{2^{m-1}}\left(\frac{1}{m+2} - \frac{1}{m+1}\right) + 4\left(\frac{1}{m+1} - \frac{1}{m+2}\right)\right]^{\frac{1}{m}}}$
LOGNORMAL	$\dfrac{1 + \sigma/\mu}{\left\{1 + \ln\left(1+\frac{\sigma^2}{\mu^2}\right)\right\}^{\frac{m-1}{2}}}$

TABLE 4.6 EFFECT OF FATIGUE PARAMETER m ON RANDOM STRESS ANALYSIS FACTOR -- Simple Density Function Models

	RSAF Values for S_c = mean + std. dev.					
m/Case	A	B	C	D	E	F
2	1.366	1.394	1.304	1.356	1.247	1.344
2.5	1.302	1.299	1.264	1.290	1.222	1.284
3	1.252	1.226	1.230	1.226	1.197	1.233
3.5	1.212	1.168	1.201	1.166	1.172	1.189
4	1.179	1.120	1.175	1.109	1.148	1.151
4.5	1.152	1.080	1.151	1.054	1.125	1.118
5	1.128	1.046	1.130	1.003	1.102	1.089

Density Functions:

Case A = rectangle

B = right triangle

C = symmetrical triangle

D = lognormal, c.o.v. = 0.5

E = lognormal, c.o.v. = 0.3

F = truncated Rayleigh

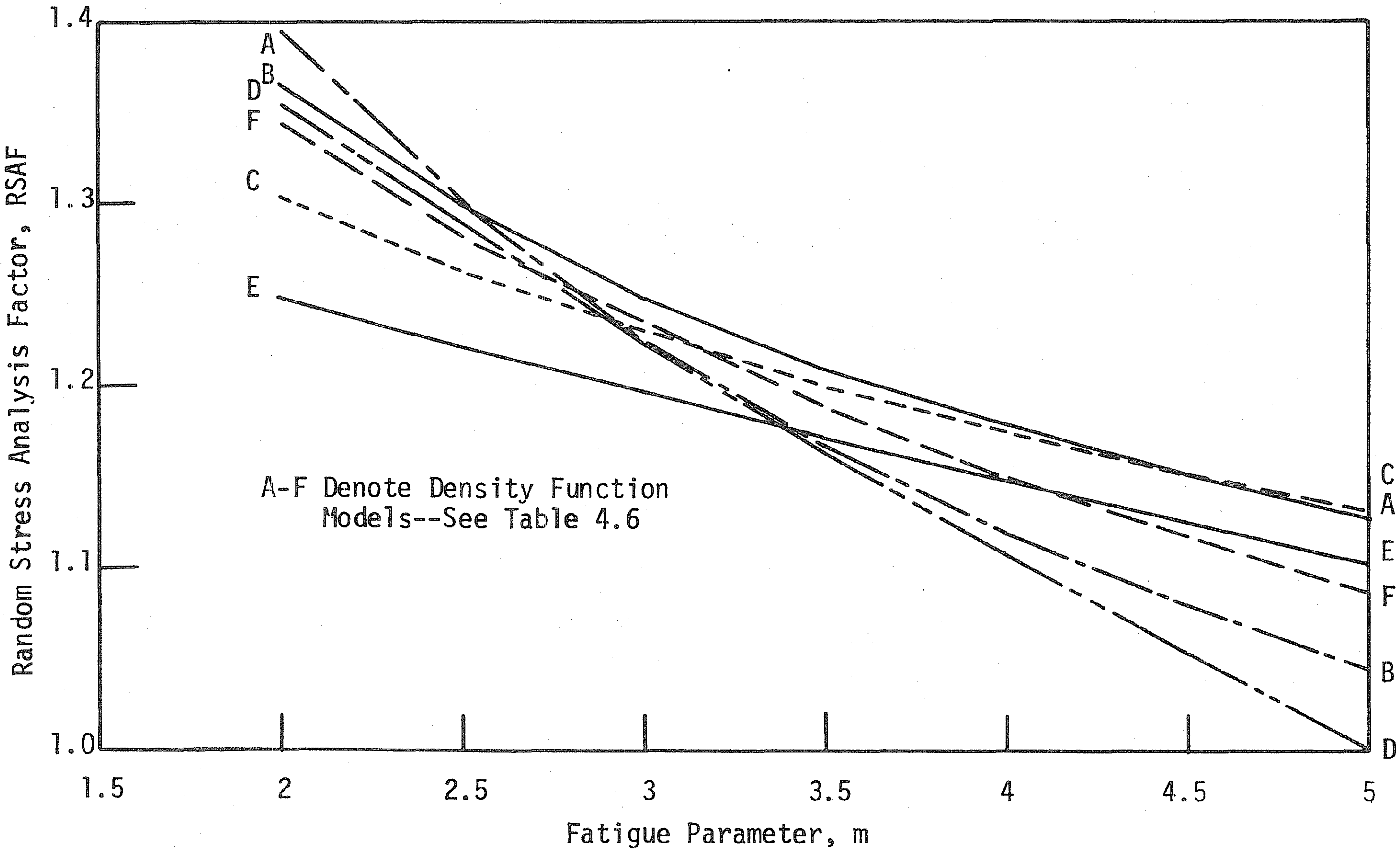

Fig. 4.4 Effect of Fatigue Parameter m on Random Stress Analysis Factor for Simple Density Function Models

applied to the calculation of $E(S^m)$ for the 42 data sets; it is seen to work well but with increasing errors for larger m-values as shown in the results tabulated below:

m	Mean Error Ratio	Std. Dev. of Error Ratio	C.O.V. of Error Ratio
2.75	1.017	0.080	.079
3	1.010	0.104	.103
3.5	1.005	0.186	.185
4	1.024	0.300	.293
4.5	1.082	0.448	.414
5	1.204	0.653	.542

4.4 Effect of Coefficient of Variation and Maximum Stress

It was seen in Section 3.4 that the beta distribution correlates well with histogram data although it underestimates somewhat the value of $E(S^m)$. The correlation between the beta model and the measured results is close enough to use the beta distribution to study the effects of parameters such as maximum stress and the coefficient of variation on RSAF. Again, the RSAF is defined for S_c = mean + std. dev. The results of a set of such studies for m = 3 is shown in Fig. 4.5. Included on this figure are results for the truncated Rayleigh model and a lognormal distribution. In Fig. 4.5, RSAF is plotted as a function of coefficient of variation for various values of maximum stress, expressed as the ratio of the maximum to the mean stress, and denoted b. It should be noted that b = 2 corresponds to a symmetrical distribution, that is, skew = 0. A value of $b < 2$ corresponds to a negative skew and a value of $b > 2$ to positive skew. The solid triangle represents the truncated Rayleigh distribution and falls at a level corresponding to b = 3. The beta distribution can be fitted closely to the shape of the Rayleigh and hence the RSAF results should be in close correspondence for c.o.v. = 0.505.

Figure 4.5 shows the relative insensitivity of RSAF to the coefficient of variation over a range covering most of the field data presented in the study. An exception to this statement is the result which has negative skew values which would be represented approximately by the curve denoted b = 1.5; this would represent the Gallatin County Bridge data. The other important conclusion to be drawn from this figure is the relative insensitivity of RSAF to the maximum stress limit (b value). If the maximum apparent stress had been used to define S_c (instead of mean + std. dev.), then the RSAF would have behaved quite erratically depending on one choice or perception of the maximum stress.

Finally, it is observed that the RSAF for the lognormal model does not drop off as rapidly with increasing coefficient of variation as does the beta model with large b values. This result is perhaps contrary to the anticipated since the lognormal model has what would correspond to a b-value of infinity. This would suggest a reduced usefulness of the lognormal model in that it would underestimate the value of $E(S^m)$ for large coefficients of variation.

Summarizing the significance of these studies of RSAF using simple probability density functions, clearly the parameter is well behaved and not excessively sensitive to the major parameters used in fitting it to the experimental data. Hence the empirical results for a design value of RSAF namely, 1.419 - 0.0955 m, may prove to be reliable for use in predicting values of $E(S^m)$ for use with the class of bridges and traffic conditions considered in the present study. Also, this study serves to suggest the

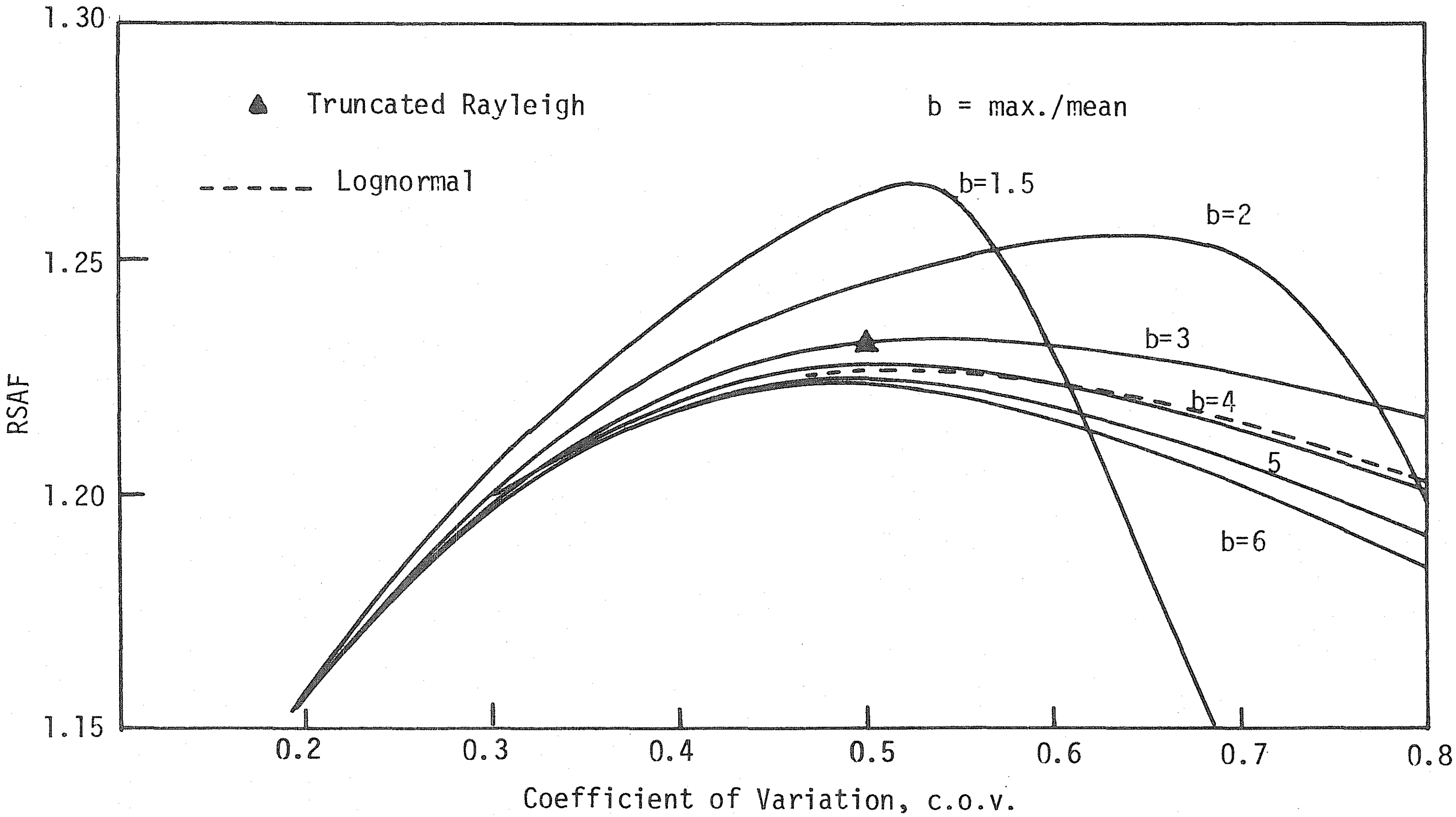

Fig. 4.5 Effect of Coefficient of Variation and Maximum Stress on RSAF

limitations of simple probability density functions in that one cannot fully take advantage of the apparent number of degrees of freedom, e.g. changing the upper limit stress. If the mean and variance of the data is modeled, even large changes in the upper limit stress for the distribution will not produce a significant shift in RSAF or the resulting value of $E(S^m)$.

5. APPLICATIONS IN ANALYSIS AND DESIGN FOR FATIGUE LIFE EXPECTANCY

5.1 General

The designer must establish allowable design live load stress limits to assure that fatigue failure will not occur within a specified number of load cycles. For example, the current AASHTO specification sets live load stress limits by class of structure, i.e. whether it is redundant or nonredundant in load path, for four classes of traffic volume: 100,000, 500,000, 2,000,000, or over 2,000,000 cycles of load application. Given these design goals, the results presented herein have direct application and can help provide an understanding of some of the problems associated with formulating a rational fatigue design procedure.

In the design process there are a number of additional questions suggested by the present study. The data on stress histograms shows a degree of uncertainty or variability, leading to uncertainty to be associated with the prediction of $E(S^m)$, the major load or stress parameter needed to predict mean fatigue life. There is also an uncertainty in the analysis of fatigue behavior even with constant amplitude load applications. Fatigue life is a random variable under controlled laboratory conditions. In this chapter techniques for analysis of fatigue reliability under a given design life will be applied, including estimates of the variability of constant amplitude fatigue information. This analysis will be applied to the 42 histograms sets presented in Chapter 3, recognizing that the stress levels obtained in the field measurements are well below those allowed under the AASHTO specifications. However, for the Gallatin County Bridge, if category E details had been present, there would be some difficulties with fatigue life expectancy. It is important to note, however, that such details were not present where stress measurements were made.

The analysis of fatigue life expectancy can be approached from the point of view of the researcher-analyst making use of available data to predict fatigue life, or of the designer who seeks a rational determination of fatigue stress limits with an acceptable reliability against a specified fatigue life. Guidance is needed on several matters:

1. Selection of the design fatigue life, that is, the number of cycles of a given or projected traffic situation which the structure should be designed to resist.

2. A corresponding permissible design live load stress level which should be based on the overall induced live load stress effect, expressed as $E(S^m)$, and which is associated with the design traffic volume.

3. An estimate of the reliability of the design or alternatively the reliability of the estimate of the fatigue life expectancy of an existing structure.

The points of view of the researcher-analyst and the design engineer merge when they are faced with making a quantitative estimate of the remaining life of an existing structure which is to be scheduled for replacment under perhaps a financial constraint which may result in uncertain delays. One of the benefits of being able to make a reliable estimate of fatigue life is to be found in the rational planning of bridge replacements.

The purpose of this chapter is to discuss topics related to the use of field data and the RSAF concept for analysis and design for fatigue life expectancy. First, to illustrate the variability in the reliability levels to be associated with the design process, a fatigue reliability analysis will be applied to the stress histogram data, using the formulation by Ang and Munse. The random stress analysis factor developed in Chapter 4 shows promise of being useful in a simplified design formulation and will be explored, including two other suggestions for fatigue design which have appeared in the literature.

Finally, assigning a value to a design fatigue life in terms of cycles has two important implications. One must know the relationships between design load cycles and a vehicle crossing events. Then given the design life, one must be able to predict statistics of the stress range data to be associated with a fatigue critical location under study. More specifically, work is needed on the task of calculating mean stress range and predicting variance for the stress range. However, the designer usually seeks to specify a maximum permissable live load stress instead of a mean stress range and variance. Thus, further guidance is needed on the relationship between maximum stress and mean stress and associated variance. Given the degrees of freedom in the problem of describing stress range histograms, there is interchangeability in approaches to specifying design stress variables.

5.2 Fatigue Reliability Analysis

The development of techniques for analysis of fatigue behavior is not within the scope of the present study and the analysis and interpretation presented in this section draws upon the damage rule presented in Section 1.2. The work by Ang (5) and Ang and Munse (6) has led to a convenient formulation for estimates of the reliability of design for fatigue with a specified life. This formulation for fatigue reliability is discussed and applied in the following.

The material properties and the influence of structural details are represented by m and c parameters of the S-N diagram for fatigue under constant amplitude tests. Since the present studies involve structures which do not appear to be fatigue critical, values of fatigue parameters to be studied will be varied over a realistic range to represent usual steels and structural details but will not be selected in specific combinations to represent a fatigue-critical situation.

The formulation for mean fatigue life, $\bar{n}$, under the action of variable cycles of loading was developed in Section 1.2, Eq. 1.6: $n = \frac{c}{E(S^m)}$.

The quantity $E(S^m)$ has been discussed in previous sections as a statistical property of the stress range data. It has been computed using the stress range histograms directly or using a density function model of the stress range data. It is subject to uncertainties as a function of random variables.

It is possible to determine a corresponding reliability function for fatigue life, that is, to express the probability that there will be no failure at a specific design life due to fatigue as a function of the life selected. Ang (5) has reviewed this area of application of reliability theory and suggests certain simplifications based on the Weibull probability density function for fatigue life under constant amplitude cyclic loading. Resulting reliability function is:

$$L(n) = \exp\left[-\frac{n}{\bar{n}}\,\Gamma(1+\Omega^{1.08})\right]\Omega^{-1.08} \quad (5.1)$$

n represents the design fatigue life, $\bar{n}$ the mean life from Eq. (1.6) and $\Omega = \Omega_N$ is the coefficient of variation associated with the determination of mean fatigue life. The coefficient of variation, Ω_N, thus represents the uncertainty associated with all aspects of the determination of fatigue life including inaccuracy of estimating mean stress range, the uncertainty in using the Miner's damage rule, and the uncertainty associated with estimating the parameters m and c in the S-N fatigue diagram. The uncertainty analysis principles are well formulated (5) but not within this presentation; representative values of Ω_N are seen to be between 0.6 and 0.9. This range will be used to illustrate the effect of uncertainty, and should be interpreted as including the uncertainty associated with $\bar{n}$, c, m, etc.

The behavior of the reliability function for the 42 histogram sets considered previously has been analyzed for three design lifes: 100,000, 500,000, and 2,000,000 cycles of loading. The expression given in Eq. (5.1) has been evaluated for the 42 cases for m = 3, c = 10^{10} and Ω_N = 0.7. The results are shown in Tables 5.1 through 5.3. Also summarized in the tabulation are the gage identification, mean, coefficient of variation, maximum stress, m-value, RSAF, c value, Ω_N, and in the last column, the fatigue reliability represented as [1-L(n)]. Again, the fatigue reliability, L(n), is the probability, for the combination of parameters considered, that the estimate of mean fatigue life, $\bar{n}$, will exceed the required design life, n; conversely, [1-L(n)] is a probability of failure. Scanning the three tables, it is seen that for most combinations of parameters the fatigue reliability is very high, as would be expected, because the mean stress levels are low. For locations having the higher mean stresses, mainly for the Gallatin River Bridge, there is some reduction in the fatigue reliability, L(n), but rarely below 0.98.

To focus more specifically on those locations having higher mean stress levels the results for six locations presented in Table 5.4 were selected. The format of the presentation of the statistical quantities and [1-L(n)] is the same as before. In order to indicate where probabilities of failure, [1-L(n)], may become unacceptable, the results in Table 5.4 have been extended to include m = 3, 4 and 5 with Ω_N values of 0.6, 0.7, 0.8 and 0.9. Thus for mean stress of 46.7 MPa the reliability is unsatisfactory for a fatigue design life at 2,000,000 cycles for all values of Ω_N, 0.6 to 0.9. Again, these results are presented only on a comparative basis to show the effect of the uncertainty parameter Ω_N, and should not be taken as substantive evidence of an inadequate design at the Gallatin County location. However, this result does suggest some difficulty if the heavy vehicles encountered in the measurement program were to be present in traffic volumes on the order of 2,000,000 vehicles. Also, higher m-values, 5 or larger, correspond to a cover plate cut-off detail which was not present at the location considered in Table 5.4. Otherwise, in scanning these tabulations for m-values on the order of 3 and Ω_N between 0.6 and 0.9, the general probability of failure against a 2,000,000 cycle fatigue life is on the order of 10^{-3} to 10^{-5}, an entirely acceptable level. Since the higher stress values shown in Table 5.4 are for a rural location it would be more realistic to think in terms of the 500,000 cycle design fatigue life for which the largest probability of failure is on the order of 10^{-2} even for the severe condition at the

TABLE 5.1 FATIGUE RELIABILITY STUDY--DESIGN LIFE(N)=100000
(MEAN and MAX are for Stress Range in MPa)

GAGE	MEAN	C.O.V.	MAX	M	RSF(S)	LOG-C	OMEGA	1 - L(n)
221	7.5	0.4	32	3	1.219	10	0.7	0
222	11.3	0.45	32	3	1.229	10	0.7	1.0E-6
223	14.3	0.46	38	3	1.232	10	0.7	2.0E-6
224	11.8	0.45	34	3	1.229	10	0.7	1.0E-6
225	7.4	0.45	32	3	1.224	10	0.7	0
533	12.1	0.46	34	3	1.23	10	0.7	1.0E-6
534	8.5	0.42	32	3	1.223	10	0.7	0
433	11.6	0.44	32	3	1.229	10	0.7	1.0E-6
434	8.2	0.44	22	3	1.229	10	0.7	0
221	8.2	0.44	44	3	1.222	10	0.7	0
222	11.1	0.42	44	3	1.222	10	0.7	1.0E-6
223	12.8	0.41	42	3	1.223	10	0.7	1.0E-6
224	10.9	0.41	44	3	1.221	10	0.7	1.0E-6
225	7.8	0.47	42	3	1.223	10	0.7	0
533	11.2	0.41	42	3	1.221	10	0.7	1.0E-6
534	8.2	0.43	40	3	1.222	10	0.7	0
433	10.7	0.41	46	3	1.221	10	0.7	1.0E-6
434	8	0.44	42	3	1.222	10	0.7	0
113	4.2	0.53	18	3	1.226	10	0.7	0
114	6.7	0.57	28	3	1.225	10	0.7	0
121	4.6	0.77	36	3	1.185	10	0.7	0
122	7.9	0.8	60	3	1.179	10	0.7	1.0E-6
123	19.3	0.6	80	3	1.223	10	0.7	1.4E-5
124	22.8	0.58	78	3	1.229	10	0.7	2.6E-5
125	13.4	0.56	52	3	1.227	10	0.7	2.0E-6
222	28.2	0.28	40	3	1.198	10	0.7	3.0E-5
224	32.5	0.28	46	3	1.197	10	0.7	5.6E-5
225	38.9	0.26	56	3	1.187	10	0.7	1.2E-4
226	32	0.24	48	3	1.177	10	0.7	4.9E-5
211	46.7	0.31	76	3	1.206	10	0.7	2.93E-4
121	36.2	0.35	56	3	1.223	10	0.7	1.04E-4
122	34.9	0.37	56	3	1.228	10	0.7	9.4E-5
123	38.2	0.34	58	3	1.218	10	0.7	1.28E-4
113	27.1	0.35	42	3	1.222	10	0.7	2.9E-5
221	47.6	0.32	78	3	1.209	10	0.7	3.25E-4
121	5	0.37	14	3	1.218	10	0.7	0
122	9	0.58	30	3	1.23	10	0.7	0
123	15.7	0.44	60	3	1.225	10	0.7	3.0E-6
124	20.3	0.47	64	3	1.23	10	0.7	1.2E-5
125	21	0.42	60	3	1.225	10	0.7	1.2E-5
114	16.7	0.6	110	3	1.215	10	0.7	7.0E-6
115	10.8	0.75	60	3	1.197	10	0.7	2.0E-6

TABLE 5.2 FATIGUE RELIABILITY STUDY--DESIGN LIFE(n)=500000
(MEAN and MAX are for Stress Range in MPa)

GAGE	MEAN	C.O.V.	MAX	M	RSF(S)	LOG-C	OMEGA	1 - L(n)
221	7.5	0.4	32	3	1.219	10	0.7	1.0E-6
222	11.3	0.45	32	3	1.229	10	0.7	9.0E-6
223	14.3	0.46	38	3	1.232	10	0.7	2.6E-5
224	11.8	0.45	34	3	1.229	10	0.7	1.1E-5
225	7.4	0.45	32	3	1.224	10	0.7	1.0E-6
533	12.1	0.46	34	3	1.23	10	0.7	1.2E-5
534	8.5	0.42	32	3	1.223	10	0.7	2.0E-6
433	11.6	0.44	32	3	1.229	10	0.7	1.0E-5
434	8.2	0.44	22	3	1.229	10	0.7	2.0E-6
221	8.2	0.44	44	3	1.222	10	0.7	2.0E-6
222	11.1	0.42	44	3	1.222	10	0.7	7.0E-6
223	12.8	0.41	42	3	1.223	10	0.7	1.4E-5
224	10.9	0.41	44	3	1.221	10	0.7	7.0E-6
225	7.8	0.47	42	3	1.223	10	0.7	2.0E-6
533	11.2	0.41	42	3	1.221	10	0.7	7.0E-6
534	8.2	0.43	40	3	1.222	10	0.7	2.0E-6
433	10.7	0.41	46	3	1.221	10	0.7	6.0E-6
434	8	0.44	42	3	1.222	10	0.7	2.0E-6
113	4.2	0.53	18	3	1.226	10	0.7	0
114	6.7	0.57	28	3	1.225	10	0.7	1.0E-6
121	4.6	0.77	36	3	1.185	10	0.7	0
122	7.9	0.8	60	3	1.179	10	0.7	6.0E-6
123	19.3	0.6	80	3	1.223	10	0.7	1.45E-4
124	22.8	0.58	78	3	1.229	10	0.7	2.79E-4
125	13.4	0.56	52	3	1.227	10	0.7	2.5E-5
222	28.2	0.28	40	3	1.198	10	0.7	3.19E-4
224	32.5	0.28	46	3	1.197	10	0.7	5.95E-4
225	38.9	0.26	56	3	1.187	10	0.7	0.001273
226	32	0.24	48	3	1.177	10	0.7	5.22E-4
211	46.7	0.31	76	3	1.206	10	0.7	0.00312
121	36.2	0.35	56	3	1.223	10	0.7	0.001106
122	34.9	0.37	56	3	1.228	10	0.7	9.97E-4
123	38.2	0.34	58	3	1.218	10	0.7	0.001358
113	27.1	0.35	42	3	1.222	10	0.7	3.1E-4
221	47.6	0.32	78	3	1.209	10	0.7	0.003454
121	5	0.37	14	3	1.218	10	0.7	0
122	9	0.58	30	3	1.23	10	0.7	5.0E-6
123	15.7	0.44	60	3	1.225	10	0.7	3.6E-5
124	20.3	0.47	64	3	1.23	10	0.7	1.24E-4
125	21	0.42	60	3	1.225	10	0.7	1.24E-4
114	16.7	0.6	110	3	1.215	10	0.7	8.0E-5
115	10.8	0.75	60	3	1.197	10	0.7	1.8E-5

TABLE 5.3 FATIGUE RELIABILITY STUDY--DESIGN LIFE(N)= 2000000
(MEAN and MAX are for Stress Range in MPa)

GAGE	MEAN	C.O.V.	MAX	M	RSF(S)	LOG-C	OMEGA	1 - L(N)
221	7.5	0.4	32	3	1.219	10	0.7	1.0E-5
222	11.3	0.45	32	3	1.229	10	0.7	6.6E-5
223	14.3	0.46	38	3	1.232	10	0.7	1.98E-4
224	11.8	0.45	34	3	1.229	10	0.7	8.1E-5
225	7.4	0.45	32	3	1.224	10	0.7	1.1E-5
533	12.1	0.46	34	3	1.23	10	0.7	9.3E-5
534	8.5	0.42	32	3	1.223	10	0.7	1.8E-5
433	11.6	0.44	32	3	1.229	10	0.7	7.3E-5
434	8.2	0.44	22	3	1.229	10	0.7	1.6E-5
221	8.2	0.44	44	3	1.222	10	0.7	1.6E-5
222	11.1	0.42	44	3	1.222	10	0.7	5.7E-5
223	12.8	0.41	42	3	1.223	10	0.7	1.07E-4
224	10.9	0.41	44	3	1.221	10	0.7	5.3E-5
225	7.8	0.47	42	3	1.223	10	0.7	1.4E-5
533	11.2	0.41	42	3	1.221	10	0.7	5.8E-5
534	8.2	0.43	40	3	1.222	10	0.7	1.6E-5
433	10.7	0.41	46	3	1.221	10	0.7	4.8E-5
434	8	0.44	42	3	1.222	10	0.7	1.5E-5
113	4.2	0.53	18	3	1.226	10	0.7	1.0E-6
114	6.7	0.57	28	3	1.225	10	0.7	1.0E-5
121	4.6	0.77	36	3	1.185	10	0.7	4.0E-6
122	7.9	0.8	60	3	1.179	10	0.7	4.4E-5
123	19.3	0.6	80	3	1.223	10	0.7	0.001114
124	22.8	0.58	78	3	1.229	10	0.7	0.002138
125	13.4	0.56	52	3	1.227	10	0.7	1.95E-4
222	28.2	0.28	40	3	1.198	10	0.7	0.002443
224	32.5	0.28	46	3	1.197	10	0.7	0.004559
225	38.9	0.26	56	3	1.187	10	0.7	0.00973
226	32	0.24	48	3	1.177	10	0.7	0.004001
211	46.7	0.31	76	3	1.206	10	0.7	0.023691
121	36.2	0.35	56	3	1.223	10	0.7	0.008457
122	34.9	0.37	56	3	1.228	10	0.7	0.007627
123	38.2	0.34	58	3	1.218	10	0.7	0.010375
113	27.1	0.35	42	3	1.222	10	0.7	0.002375
221	47.6	0.32	78	3	1.209	10	0.7	0.026201
121	5	0.37	14	3	1.218	10	0.7	2.0E-6
122	9	0.58	30	3	1.23	10	0.7	3.7E-5
123	15.7	0.44	60	3	1.225	10	0.7	2.78E-4
124	20.3	0.47	64	3	1.23	10	0.7	9.54E-4
125	21	0.42	60	3	1.225	10	0.7	9.49E-4
114	16.7	0.6	110	3	1.215	10	0.7	6.12E-4
115	10.8	0.75	60	3	1.197	10	0.7	1.4E-4

Gallatin County location where the histogram is skewed to the heavy side. Again, measurements for this location are biased by the fact that the recordings were made only for the heaviest vehicles, and on beams under the lane carrying trucks on the loaded portion of their travel cycle.

The results presented in Tables 5.1 to 5.4 used a beta function model for calculating $E(S^m)$ and $\bar{n}$ in the evaluation of Eq. (5.1). This was done for economy in calculation for a wide range of m and Ω_N values. However, virtually identical values of [1-L(n)] are obtained when the histogram calculation for $E(S^m)$ is used; this may be verified by comparing the results presented in Table 5.5 with Table 5.3. Both consider a design fatigue life of 2,000,000 cycles.

RSAF in the Reliability Formulation -- The reliability formulation of Eq. (5.1) may be recast in terms of the RSAF formulation for $E(S^m)$ and consequently for mean life, $\bar{n}$. The result will be an estimate of fatigue reliability which is expressed directly in terms of mean stress. For example:

$$\bar{n} = c/E(S^m) \text{ where } E(S^m) = [\text{mean}(1+\text{c.o.v.})/\text{RSAF}]^m \qquad (5.2)$$

To test this approximation take RSAF = 1.2 and m = 3. Thus,

$$n/\bar{n} = (n/c)E(S^m) = (n/c)[\text{mean}(1+\text{c.o.v.})/1.2]^3 \qquad (5.3)$$

To further simplify Eq. (5.3), consider a central value of c.o.v. which will represent the data collected, say, c.o.v. = 0.45:

$$n/\bar{n} = (n/c)(1.45/1.2)^3(\text{mean})^3 = 1.7643(n/c)(\text{mean})^3$$

With the above, the reliability expression, Eq. (5.1), becomes:

$$L(n) = \exp\{-(n/c).7643(\text{mean})^3\ \Gamma(1+\Omega^{1.08})\Omega^{1.08}\} \qquad (5.4)$$

TABLE 5.4 FATIGUE RELIABILITY STUDY--DESIGN LIFE(N)=2000000
TABULATION OF FAILURE PROBABILITIES
(MEAN and MAX are for Stress Range in MPa)

GAGE	MEAN	C.O.V.	MAX	M	RSF(S)	LOG-C	OMEGA	1 - L(n)
533	12.1	0.46	34	3	1.23	10	0.6	1.7E-5
533	12.1	0.46	34	3	1.23	10	0.7	9.3E-5
533	12.1	0.46	34	3	1.23	10	0.8	3.33E-4
533	12.1	0.46	34	3	1.23	10	0.9	0.0009
533	12.1	0.46	34	4	1.158	10	0.6	9.7E-5
533	12.1	0.46	34	4	1.158	10	0.7	4.09E-4
533	12.1	0.46	34	4	1.158	10	0.8	0.001202
533	12.1	0.46	34	4	1.158	10	0.9	0.002783
533	12.1	0.46	34	5	1.101	10	0.6	6.3E-4
533	12.1	0.46	34	5	1.101	10	0.7	0.001995
533	12.1	0.46	34	5	1.101	10	0.8	0.004737
533	12.1	0.46	34	5	1.101	10	0.9	0.009295
223	12.8	0.41	42	3	1.223	10	0.6	2.0E-5
223	12.8	0.41	42	3	1.223	10	0.7	1.07E-4
223	12.8	0.41	42	3	1.223	10	0.8	3.78E-4
223	12.8	0.41	42	3	1.223	10	0.9	0.001005
223	12.8	0.41	42	4	1.157	10	0.6	1.18E-4
223	12.8	0.41	42	4	1.157	10	0.7	4.82E-4
223	12.8	0.41	42	4	1.157	10	0.8	0.001387
223	12.8	0.41	42	4	1.157	10	0.9	0.003157
223	12.8	0.41	42	5	1.102	10	0.6	7.9E-4
223	12.8	0.41	42	5	1.102	10	0.7	0.002415
223	12.8	0.41	42	5	1.102	10	0.8	0.005589
223	12.8	0.41	42	5	1.102	10	0.9	0.010749
124	22.8	0.58	78	3	1.229	10	0.6	6.84E-4
124	22.8	0.58	78	3	1.229	10	0.7	0.002138
124	22.8	0.58	78	3	1.229	10	0.8	0.005031
124	22.8	0.58	78	3	1.229	10	0.9	0.0098
124	22.8	0.58	78	4	1.132	10	0.6	0.015668
124	22.8	0.58	78	4	1.132	10	0.7	0.030066
124	22.8	0.58	78	4	1.132	10	0.8	0.049087
124	22.8	0.58	78	4	1.132	10	0.9	0.071952
124	22.8	0.58	78	5	1.059	10	0.6	0.344852
124	22.8	0.58	78	5	1.059	10	0.7	0.389675
124	22.8	0.58	78	5	1.059	10	0.8	0.428902
124	22.8	0.58	78	5	1.059	10	0.9	0.463789

TABLE 5.4 FATIGUE RELIABILITY STUDY--DESIGN LIFE(N)=2000000
TABULATION OF FAILURE PROBABILITIES
(Continued)

GAGE	MEAN	C.O.V.	MAX	M	RSF(S)	LOG-C	OMEGA	1 - L(n)
211	46.7	0.31	76	3	1.206	10	0.6	0.011802
211	46.7	0.31	76	3	1.206	10	0.7	0.023691
211	46.7	0.31	76	3	1.206	10	0.8	0.040012
211	46.7	0.31	76	3	1.206	10	0.9	0.060224
211	46.7	0.31	76	4	1.171	10	0.6	0.387322
211	46.7	0.31	76	4	1.171	10	0.7	0.428365
211	46.7	0.31	76	4	1.171	10	0.8	0.464188
211	46.7	0.31	76	4	1.171	10	0.9	0.49606
211	46.7	0.31	76	5	1.143	10	0.6	1
211	46.7	0.31	76	5	1.143	10	0.7	0.999999
211	46.7	0.31	76	5	1.143	10	0.8	0.999953
211	46.7	0.31	76	5	1.143	10	0.9	0.999613
123	38.2	0.34	58	3	1.218	10	0.6	0.004431
123	38.2	0.34	58	3	1.218	10	0.7	0.010375
123	38.2	0.34	58	3	1.218	10	0.8	0.019667
123	38.2	0.34	58	3	1.218	10	0.9	0.032394
123	38.2	0.34	58	4	1.182	10	0.6	0.124344
123	38.2	0.34	58	4	1.182	10	0.7	0.169041
123	38.2	0.34	58	4	1.182	10	0.8	0.213108
123	38.2	0.34	58	4	1.182	10	0.9	0.255536
123	38.2	0.34	58	5	1.154	10	0.6	0.984701
123	38.2	0.34	58	5	1.154	10	0.7	0.967761
123	38.2	0.34	58	5	1.154	10	0.8	0.950365
123	38.2	0.34	58	5	1.154	10	0.9	0.935029
125	21	0.42	60	3	1.225	10	0.6	2.62E-4
125	21	0.42	60	3	1.225	10	0.7	9.49E-4
125	21	0.42	60	3	1.225	10	0.8	0.002493
125	21	0.42	60	3	1.225	10	0.9	0.005287
125	21	0.42	60	4	1.16	10	0.6	0.003623
125	21	0.42	60	4	1.16	10	0.7	0.008753
125	21	0.42	60	4	1.16	10	0.8	0.016987
125	21	0.42	60	4	1.16	10	0.9	0.028496
125	21	0.42	60	5	1.107	10	0.6	0.055152
125	21	0.42	60	5	1.107	10	0.7	0.086194
125	21	0.42	60	5	1.107	10	0.8	0.12059
125	21	0.42	60	5	1.107	10	0.9	0.156717

TABLE 5.5 STUDY OF FATIGUE RELIABILITY
ESTIMATED LIFE CALCULATED FROM HISTOGRAM DATA

SUMMARY TABULATION FOR 42 SETS
M= 3 C= 10000000000

OMEGA=0.7

MEAN	E(S***M)	RSAF(S)	1 - L(n)
7.477	2.132	1.219	9.6E-6
11.256	7.935	1.229	6.63E-5
14.346	16.721	1.232	1.982E-4
11.793	9.094	1.229	8.1E-5
7.431	2.308	1.224	1.08E-5
12.129	9.98	1.23	9.28E-5
8.517	3.293	1.223	1.82E-5
11.601	8.491	1.229	7.32E-5
8.235	3.036	1.229	1.61E-5
8.23	3.07	1.222	1.64E-5
11.069	7.12	1.222	5.65E-5
12.847	11.011	1.223	1.073E-4
10.941	6.815	1.221	5.3E-5
7.792	2.768	1.223	1.41E-5
11.163	7.206	1.221	5.75E-5
8.228	3.004	1.222	1.59E-5
10.675	6.385	1.221	4.82E-5
8.036	2.871	1.222	1.49E-5
4.23	0.5	1.226	1.1E-6
6.701	2.158	1.225	9.8E-6
4.607	1.106	1.185	3.7E-6
7.923	5.976	1.179	4.37E-5
19.259	54.126	1.223	0.0011139
22.779	84.383	1.229	0.0021384
13.362	16.509	1.227	1.945E-4
28.195	92.396	1.198	0.0024431
32.513	141.35	1.197	0.0045593
38.935	237.163	1.187	0.0097304
32.04	129.302	1.177	0.0040008
46.65	436.562	1.206	0.0236911
36.155	215.482	1.223	0.0084569
34.938	200.809	1.228	0.0076273
38.181	247.796	1.218	0.010375
27.121	90.621	1.222	0.0023745
47.567	467.924	1.209	0.0262009
4.999	0.606	1.218	1.5E-6
9.042	5.291	1.23	3.65E-5
15.661	21.065	1.225	2.784E-4
20.304	48.669	1.23	9.529E-4
21.01	48.55	1.225	9.495E-4
16.666	35.99	1.215	6.116E-4
10.775	13.225	1.197	1.404E-4

It is noted that the uncertainty measure, $\Omega_N = \Omega$, includes all sources of uncertainty, both in the fatigue behavior of the material and in the loading and method of analysis. $\Gamma(-)$ represents the gamma function.

Considering a set of fatigue parameters in which $c = 10^{10}$, $n =$ 2,000,000, and $\Omega_N = 0.7$ yields:

$$L(n) = \exp \{-[(n/c)1.7643(\text{mean})^3\ 0.90505]^{1.46922}\} \tag{5.5}$$

where $n/c = (2\text{x}10^6)/10^{10} = 2\text{x}10^{-4}$ and thus

$$L(n) = \exp -[\frac{3.19356}{10^4}\ (\text{mean})^3]^{1.46922} \tag{5.6}$$

Equation (5.6) is plotted in Fig. 5.1 and compared with selected data from the 42 data sets. The comparison is quite good and the RSAF approach for estimating the fatigue life appears valid over the entire range of reliability values. Similar results for other design fatigue lives, n, ($m = 3$, $\Omega_N = 0.7$) can be obtained using Eq. (5.5). In view of the RSAF behavior for larger m-values, greater differences could be expected for $m = 4$ or 5.

5.3 RSAF in Design Formulations

The RSAF concept frees the user from the task of selecting a probability density function to model the design live-load stress environment. Several other approaches have been suggested in the literature and will be investigated in this section in the light of the findings on the RSAF concept.

One approach to the analysis of fatigue for design is to eliminate the statistical parameters for stress or load and define classifications to categorize the expected load levels. For example, rather than specifying mean, variance and a probability density function for stress, a maximum stress level and a designated class of loading condition is specified. The class of loading condition is in effect a designated histogram shape or density function. The categories to be considered may be derived, for example, by setting beta distribution parameters to the yield representative shapes with either a positive skew, negative skew, or a symmetrical or zero skew shape. These three classes of distribution shape are interpreted as meaning that for a constant maximum stress range, the populations of stress events are such that they are dominated either by low level events, high level events or a mix equally divided between the two. Such an approach has been used by Munse (7). An important feature of this approach is that the maximum stress range is specified rather than deduced from the statistical properties of the stress range environment.

However, from the point of view of structural analysis, from field data, the more readily predictable statistics of stress range are mean stress level and variance. The variance can be studied by using a first order error analysis; problems in such an analysis will be noted in Section 5.5. The RSAF parameter can represent a wide range of possible stress range probability density functions, and for design a single RSAF can be specified for a given m-value. For example, taking the density functions used by Munse (7), the corresponding RSAF values are well behaved, even for the wide range of skew coefficients represented by the three suggested density functions. These are presented in Table 5.6 for m values of 3, 4, and 5, and two values of S_c. A value of RSAF = 1.2 could be used to represent these classes for design with acceptable errors in the estimates of reliability of the design (e.g. Fig. 5.1).

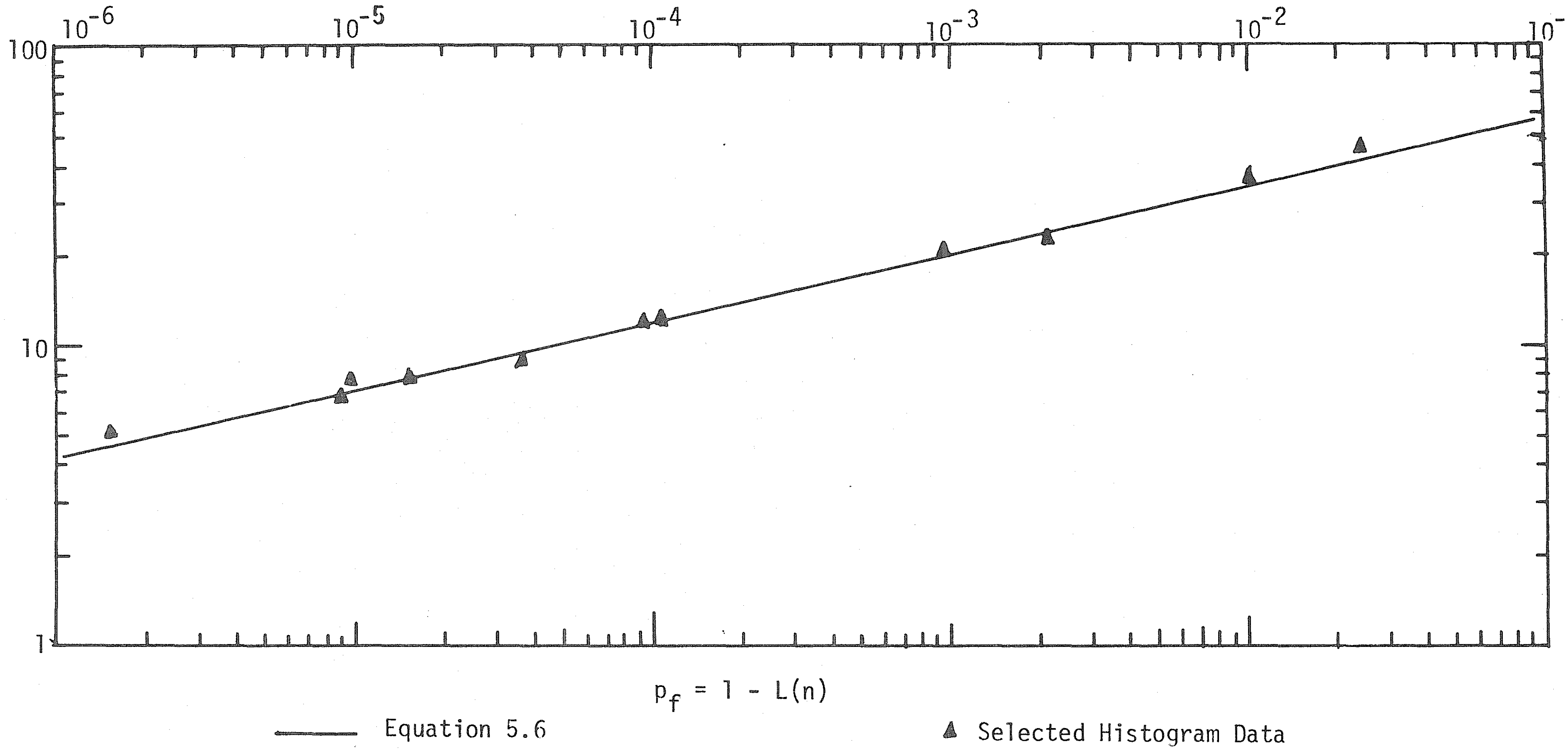

Fig. 5.1 Study of Fatigue Design Reliability--Design Life=2000000

Using the RSAF concept the problems of specifying the density function or shape are avoided, but a selection of mean and variance for the stress environment must be made. Mean and variance of live load stresses can be calculated directly by structural analysis. Of course parameters such as bridge type, span length, number of traffic lanes loaded, geometry, etc., as well as the specific fatigue critical location on the structural member or the particular structural detail must be incorporated into the analysis. It should be recalled that there is a high degree of correlation between stress range and the gross vehicle weight (4). This correlation has been clearly shown when bridge response data are matched exactly with corresponding vehicle data. Multiple, side-by-side crossing events may modify the relationship.

Another simplification of the fatigue damage calculation for design is given by Schilling and Klippstein (12) wherein the fatigue life, denoted N, is expressed as

$$N = \frac{A}{(F_{sr})^3}$$

where F_{sr} denotes a design stress range and A the intercept (denoted herein as c) on the S-N fatigue diagram. Schilling and Klippstein have standardized on B=m=3 to describe the slope of the S-N diagram for all categories of weld details. They define the design stress range, F_{sr}, as that stress range produced by a fatigue design vehicle of a gross weight, W_F.

If one interprets the life N as equivalent to the mean fatigue life $\bar{n}$, developed herein, an interpretation not made explicitly, then one can interpret this information using the RSAF concept:

TABLE 5.6 RSAF VALUES FOR BETA DISTRIBUTION STRESS RANGE MODELS -- Ref. (7)

Beta Distribution Properties						RSAF Values (Two S_c definitions)					
						m=3		m=4		m=5	
Model	Q,R	Max (MPa)	Mean (MPa)	c.o.v.	skew	max	$\mu^* + \sigma$	max	$\mu + \sigma$	max	$\mu + \sigma$
I	2,5	100 $B/\mu=3.5$	28.57	0.559	0.60	2.759	1.229	2.546	1.134	2.384	1.062
II	5,5	100 $B/\mu=2$	50	0.302	0	1.846	1.201	1.788	1.163	1.739	1.131
III	5,2	100 $B/\mu=1.4$	71.43	0.224	-0.60	1.339	1.170	1.316	1.150	1.297	1.133

* μ = mean stress range
σ = standard deviation

$$\frac{A}{(F_{sr})^3} = \frac{c}{(\frac{\overline{F} + \sigma_F}{RSAF})^3} = \frac{c}{(\overline{F})^3(\frac{1+c.o.v.}{RSAF})^3} \quad (5.7)$$

In establishing the above relationship the present notation (c) for the S-N diagram has been substituted. In Eq. (5.7) σ_F denotes the standard deviation of the stress histogram and $\overline{F}$ denotes the mean stress range corresponding to the histogram. This equation yields a simple relationship between the design stress range and mean stress range

$$F_{sr} = \overline{F} \times (\frac{1+c.o.v.}{RSAF})$$

While m=3 is considered, the above is independent of m, except for its influence on RSAF. Note that the relationship between design stress range and the mean stress depends upon the shape of the histogram only as reflected in the RSAF and the variance (c.o.v.); it is related to the shape of the histogram for applied loads only if there is a one-to-one relationship between histograms for load and fatigue critical stress.

However, using the relationship above, it is interesting to determine the relationship between mean stress and the design stress range for the truncated Rayleigh distribution, which Schilling and Klippstein have found useful in their fatigue studies. A single value of coefficient of variation is defined for the Rayleigh distribution, 0.505, which, with a value m = B = 3, corresponds to an RSAF = 1.233. Thus,

$$F_{sr} = F \times \frac{1+0.505}{1.233} = 1.22\ \overline{F}$$

Finally, with respect to the derivation of the design fatigue load, which is defined as producing F_{sr}, it is noted that Schilling and

Klippstein's computations for W_r are equivalent to treating the histogram for gross vehicle weight as a stress histogram and calculating the expected value of (gross weight)^3 in a way which is analagous to that for $E(S^m)$. Specifically where α_i denotes the percentage of counts in the i-th gross weight histogram interval, W_i:

$$W_F = (\Sigma\ \alpha_i\ W_i^3)^{1/3}$$

or

$$(W_F)^3 = E(W^3)$$

It is also noted by the writers that W_F may be taken as 50 kips if histogram data is lacking. Based on the present study this is a value close to the mean GVW for typical interstate truck data.

Effectively, the above corresponds to using the present analysis with a random stress analysis factor of unity and a defining stress (S_c) equal to the mean of the distribution. For the 42 histograms studied herein, the RSAF value for a definition based on the mean ranges from 0.6 to 1.0 with a coefficient of variation on the order of 10% for the RSAF data. Hence, the fatigue life calculations based on $(F_{sr})^3$ can be expected to be in error by a factor ranging from $(0.6)^3$ to $(1.0)^3$.

Another limitation is seen in relying upon the load histogram to define the histogram for the stress range for a fatigue critical location in a bridge structure; this correspondence is not demonstrated in the data herein. The use of an RSAF value set at a conservative and representative level could compensate for this since it is shown herein that RSAF is not sensitive to histogram shape, particularly when it is defined using a stress level of the mean plus one standard deviation. The RSAF can be adjusted readily if m-values other than 3 are used.

The above concerns are essentially refinements in view of the degree of statistical correlation which exists between stress range and gross vehicle weight.

5.4 Stress Range Definition and Counting Methods

For the data which has been presented it has been assumed that the maximum stress range is the significant quantity to represent each vehicle crossing event. This interpretation is common, but it is meaningful to count only those stress events which have potential for producing fatigue damage. Low amplitude stress cycles are to be included or excluded as need be in the fatigue damage model. Large amplitude cycles of stress applications, under some damage criteria, are significant not only in amplitude but in the order of occurrence.

Also, the count of stress events must be related to the total traffic count and the count of various truck classifications. For simple span bridges loaded mainly with a single lane of heavy vehicles, a pattern of significant stress events per vehicle crossing may be developed. However, crossings by mutliple vehicles with close longitudinal spacing in the traffic and occupation of adjacent traffic lanes greatly increases the difficulty of determining the relationship between the traffic history and the stress cycles seen in measurements on the bridge. Within the scope of the present study insufficient resources were available to measure traffic conditions in a complex loading situation such as on the Dan Ryan structure.

Counting Methods -- The histogram data tabulated here and in the interim report is for stress range, that is, the stress excursion from the maximum positive to the maximum negative value during a truck crossing.

Numerous alternative methods for counting significant stress events are available in the literature, some of which are more suitable for large strain events, that is, events which produce significant non-linear strain accumulation. The measured strain events associated with the highway bridge response are small; however, these events represent nominal values measured at locations which are not necessarily associated with a fatigue critical location. For the prediction of crack initiation and propagation, the effect of geometry, residual stresses and notches must be taken into account and will lead to the conclusion that the stress or strain cycles at the critical fatigue location may be much larger.

The subject of cycle counting is critical to the prediction of the behavior of both prototype structures and laboratory specimens under complex random load cycles. Several excellent reviews are available on the subject, for example works by Dowling and Socie (8, 9, 10). They include a list of alternative counting methods:

1. Peak counting
2. Count of mean crossing events
3. Level crossing events
4. Fatigue-meter schemes
5. Range counting
6. Range-mean counting
7. Range-pair counting
8. Rainfall counting method

Of the various method cited in the above list, the last two have been shown to be particularly useful in the prediction of crack propagation and mean life of laboratory specimens.

Rainfall Counting -- Perhaps the most useful scheme for counting significant stress cycles is the so-called rainfall method. Its description and interpretation requires more extensive discussion than is appropriate here, and is well documented in the literature (9, 10).

To provide an indication of the effect of that counting scheme on specific blocks of data the following example is presented. Data from Day 1 taken at the Camp Creek Bridge (I) has been analyzed for 50 typical events. These events, corresponding to individual truck crossings, were characterized as events which would begin at the zero or dead load stress level, rise to positive maximum, drop to zero or a negative or minimum value in the free vibration era and then return to zero. The edited sequence of data consisting of 50 such crossing events occurring in the random order recorded in the field is analyzed by the rainfall counting method. These data had been analyzed previously using stress range counting scheme. The histograms resulting from these two counts are shown in Fig. 5.2. Tabulated data for the rainfall count scheme are shown in Table 5.6 and include a Miner hypothesis damage calculation for the number of blocks of stress cycles needed for failure.

The stress range and rainfall count have comparable statistics as shown below:

Quantity	Rainfall	Range
mean strain	73.57	73.90
c.o.v.	0.423	0.458
skew	1.151	1.296
kurtosis	4.260	4.55
Q	2.90	2.40
R	4.195	3.438
max/mean	2.45	2.44

These results confirm that the two counting methods, for the data considered, yield histograms which are statistically very similar.

The essence of the rainfall counting method is that it assesses the occurrence of strain cycles counting them only if they form closed loops or hysteretic cycles. For data such as that taken on the Dan Ryan Expressway, where a relatively long-span, simple bridge is subjected to vehicles producing events which look very much like static influence lines, the rainfall counting scheme is essentially the same as the stress range counting scheme. For every instance of the development of a single strain peak, following that peak there is a return to the zero level or thus a single hysteretic loop. The usefulness of the rainfall counting scheme for such data would be in accepting or rejecting small oscillations in the record, usually occurring at the frequency of the fundamental of the bridge. On the other hand, when the time histories of bridge response are more complex and reflect large components of dynamic increment proportional to vehicle motions, then the rainfall count will produce different results from the maximum range counting scheme. The time histories for shorter bridges with larger vehicle oscillations often contain several intermediate cycles of variation which may be as significant as the primary crawl or influence line load cycle. The rainfall count does not require the identification of the start or end of a vehicle crossing event in order to register an event count. It can accept minor shifts in the zero level due to drift in the instrumentation, but has the disadvantage that if applied without editing of the digitized field record, it requires searching every data point taken and will process both electrical noise and other spikes in the records. Hence, data editing should be used even though one need not seek to correlate the edited record with vehicle crossing information.

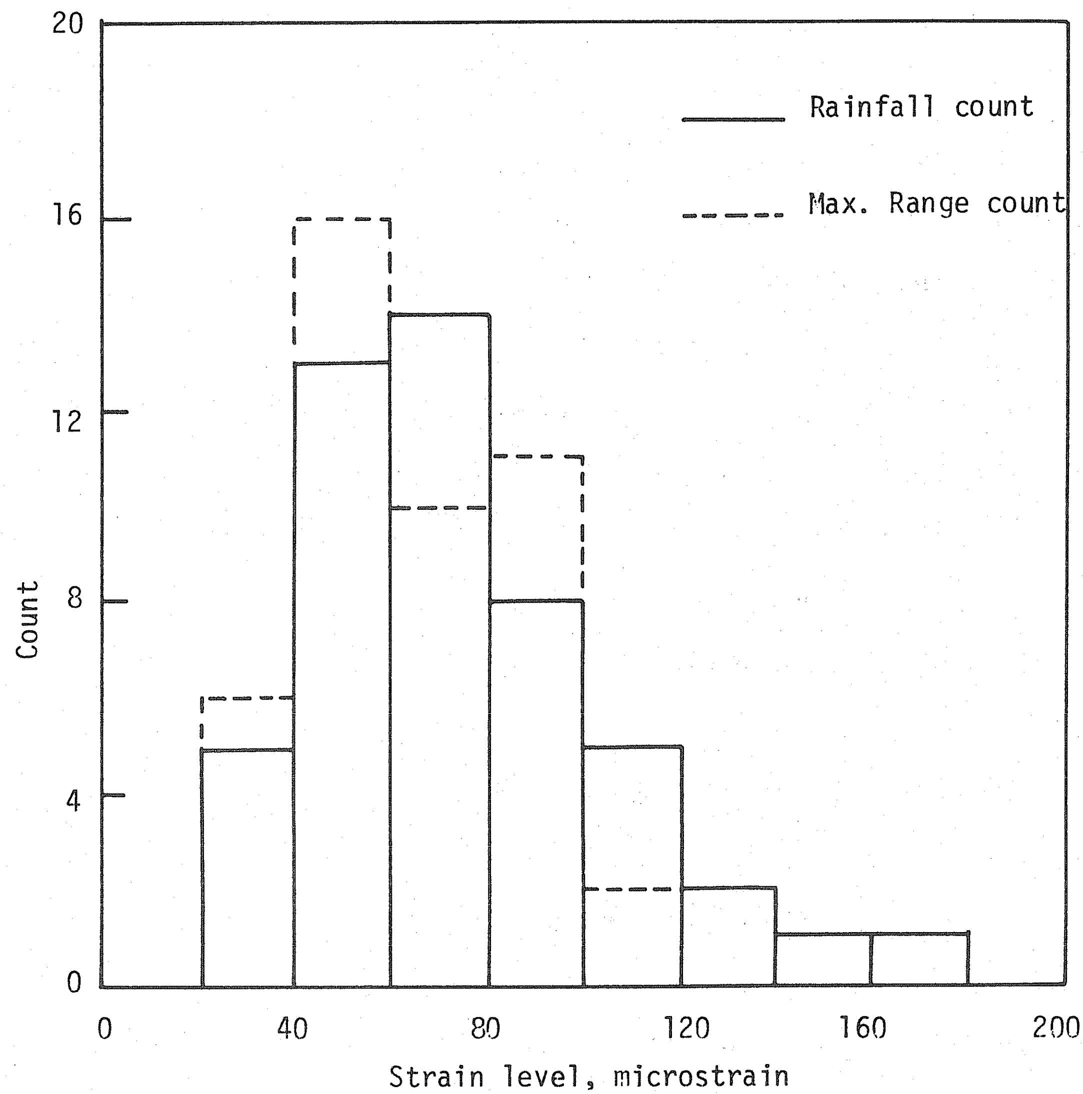

Fig. 5.2 Comparison of Rainfall and Maximum Stress Range Counts Camp Creek Bridge (I)--50 events

TABLE 5.7 SAMPLE RAINFALL COUNT AND DAMAGE CALCULATION
CAMP CREEK (II) BRIDGE

FATIGUE LIFE= C/S**M
M=3, LOG-10 OF C = 10

NUMBER OF DATA POINTS= 100

BLOCKS TO FAILURE= 6001321

DATA FOR HISTOGRAM PLOT

STRAIN	COUNT	FREQUENCY
10	0	0
20	0	0
30	0	0
40	5	.10204081633
50	7	.14285714286
60	6	.12244897959
70	11	.22448979592
80	3	.06122448980
90	4	.08163265306
100	4	.08163265306
110	4	.08163265306
120	1	.02040816327
130	1	.02040816327
140	1	.02040816327
150	0	0
160	1	.02040816327
170	0	0
180	1	.02040816327
190	0	0

TOTAL COUNT	49	

5.5 On Predicting Stress Range Statistics

If the RSAF technique proposed in Chapter 4 is used to calculate the expected life of a bridge subjected to traffic loadings, there is no need to make an estimate of histogram shape or to choose a probability density function. However, the mean stress range and the variance of the stress range data must be known as well as the fatigue parameters $\underline{m}$ and $\underline{c}$ based on an analysis of fatigue critical details in the structure. In the design situation the mean and variance of the stress environment must be predicted.

It is possible to use a first order linearization of an explicit formulation for stress range to predict its coefficient of variation. The mean stress range can be predicted directly based on mean applied live loads and structural theory.

For example, if stress range is written in the form (3)

$$S = K_1 \; GVW \; \Phi_{xy}(1 + I) \qquad (5.7)$$

where S is the stress range, K_1 a static scale factor, Φ_{xy} an influence surface, GVW is gross vehicle weight, and (1 + I) a dynamic impact effect. Using Eq. (5.7) the mean stress range, $\overline{S}$, becomes

$$\overline{S} = K_1 \; \overline{GVW} \; \Phi_{xy}(\max) \; (1 + \overline{I})$$

where $\overline{I}$ and $\overline{GVW}$ are mean values of the respective parameters and Φ_{xy} is taken at its maximum value consistent with the lane loaded and location on the bridge for which the analysis is being made.

The variance of the stress range by linear first order error theory is

$$Var(S) = \sum_i \{(\frac{g}{X_i})^2 \Big|_{X_i = \overline{X}_i} Var(X_i)\} \qquad (5.8)$$

where Var( ) denotes variance or standard deviation squared, and g denotes the function for stress range given in Eq. (5.7). The partial derivatives in Eq. 5.8 are evaluated for the mean values of the parameters ($\overline{GVW}$, $\overline{I}$, etc.). This formulation, after simplification for the Φ_{xy} factor, yields:

$$\left(\frac{\sigma_S}{\overline{S}}\right)^2 = \delta_S^2 = \delta_{GVW}^2 + \delta_{K_1}^2 + \delta_I^2 \qquad (5.9)$$

where δ_S, δ_{GVW}, δ_{K_1} and δ_I are coefficients of variation for stress range, GVW, analysis factor and impact, respectively. In Eq. (5.9) all parameters of the formulation are taken to be statistically independent. There are obvious difficulties with this assumption to be implied from the fact that the c.o.v. for the GVW can equal or exceed that of the stress range data:

Test	GVW c.o.v.	Stress Range c.o.v.
EJE(I)	0.44	0.46
EJE(II)	0.44	0.41
Camp Creek (I)	0.38	0.58

The prediction of the mean stress level requires defining a mean vehicle weight and wheel base, and then a static analysis for maximum moment and distribution to the fatigue critical location. This calculation is subject to undertainties in material properties, composite action, etc. and was explored on Project IHR-85 (4).

As can be seen from the formulation above for the coefficient of variation, a simple first order model of the response assuming the parameters of the problem to be statistically independent may be faulty. If we turn to the expression for the variance based on a first order approximation but including covariance:

$$\mathrm{Var}(S) = \sum_{i=1}^{n} c_i^2 \, V(X_i) + \sum_i \sum_j c_i c_j \, \mathrm{Cov}(X_i, x_j)$$

where c_i and c_j are derivatives of the stress range function, Eq. (5.7), with respect to X_i and X_j evaluated at their respective mean values, then it is seen that this covariance term can lead to a smaller c.o.v. in stress range. This is true provided that pairs of variables exist which have a negative correlation and for which c_i can be paired with a negative c_j. However, a detailed analysis of the interaction of problem parameters and their statistical correlations is needed.

6. SUMMARY AND CONCLUSIONS

The objectives, scope, physical arrangements, test procedures, data reduction, presentation of results and interpretation have been described for Project IHR-301, Investigation of the Life Expectancy of Highway Bridges - Stress History Studies. The report emphasizes the interpretation of the data for live load stress range and the prediction of mean fatigue life. A method of analysis which does not require the definition of a probability density function model is presented.

The project used computer-based data acquisition, analysis, and interpretation programs which were developed, in part, on project IHR-85, Dynamic Stresses in Highway Bridges.

An important first result of the research project was the development of a field test capability for the State of Illinois which became the responsibility of the bridge research group of the Illinois Department of Transportation based in Ottawa, Illinois, under the direction of F. K. Jacobson. With project funding, this research group set specifications for and acquired a field instrumentation van with equipment for 28 channels of high speed analog tape recording using two tape drives, signal conditioning, portable power supply and support apparatus. In the first phase of the project the recording equipment, van and field methods for data acquisition were tested and refined to the point where the field program, aside from initial selection of test structures, was the full responsibility of the IDOT bridge research group. This field test capability remains available for field research not only on highway bridges, but on other static and dynamic test situations, for example, on pilings or pavement.

This final report together with the interim reports on the project have presented the results of the field program; these represent measurements made on seventeen bridges and considered the effects of over 10,000 heavy vehicle crossings recorded during the various field test periods. A block of 42 histograms representing tests on five bridges and selected gage locations were taken to form a data base for detailed statistical analysis and interpretation. In general, the measured live-load stresses are moderate or low with mean values rarely exceeding 14 to 30 MPa (2,000 to 4,500 psi). The largest stresses recorded were at a rural location on a two-lane highway on a road carrying heavy ore trucks; instrumentation was such that only those lanes carrying heavy traffic were recorded. In this latter case, mean stresses on the order of 30 to 50 MPa were encountered. The histograms comprising the major summary of field test results have been presented both in an interim report (2) and herein. The maximum stress range was used throughout in the presentation of data although the question of alternative stress counting methods was investigated and is discussed herein. For low levels of stress with large number of applications it is seen that the maximum range concept is essentially equivalent to the more rational rainfall counting method.

In the literature on fatigue reliability analysis, and in investigations conducted on IHR-85 and IHR-301, the beta density function model was found to be particularly useful in representing the live-load stress range data. In addition, the use of the truncated Rayleigh model was investigated. The beta function is seen to model adequately the histogram data for the calculation of $E(S^m)$, the expected value of the stress range raised to the m-value, significant for predicting fatigue life expectancy.

While much emphasis is given to the use of $m = 3$, this modeling was adequate for values of m ranging from 3 to 5. It should be noted that as the fatigue parameter m increases, because of its amplification of the effect of higher stress ranges, and thus need for proper fit in this range, the use of the beta model or the Rayleigh model becomes less satisfactory for the higher m-values. The adequacy of a probability density function model was evaluated in terms of its ability to predict the parameter $E(S^m)$ and not by standard "goodness-of-fit" tests. The beta model was found satisfactory to fit the mean, standard deviation, and maximum stress range of the histogram data, but did not have sufficient freedom to fully match skewness and kurtosis. It is noted that the Rayleigh distribution, having a fixed coefficient of variation, could match satisfactorily only the mean and maximum stress.

Drawing upon the fact that there exist scaled or normalized parameters to represent $E(S^m)$ that are useful analysis tools, and to extend work by Ang and Munse where a random stress factor was introduced, the concept of the random stress analysis factor (RSAF) was developed. RSAF represents a coefficient which when used to modify the mean stress range plus the standard deviation yields an effective stress level which when raised to the mth power equals the quantity $E(S^m)$. The RSAF was found to be largely independent of mean stress level for all bridge types and measurement locations among the 42 histogram sets studied. RSAF has low variance as a random variable. Further it was seen that the RSAF could be related by linear regression to the m value over a range of 3 to 5. In brief, the RSAF concept permits the mean fatigue life to be expressed in terms of the mean histogram stress, coefficient of variation and the RSAF, which can be assigned on the basis of fatigue parameter, m, and engineering judgment reflecting the bridge type and other circumstances. This was demonstrated for a range of distribution shapes, including selected simple density function models of triangular and rectangular shape.

Using the RSAF concept to describe stress range histogram characteristics and predict the mean fatigue life (using the Miners hypothesis), the concept was further tested by study of the fatigue reliability of selected gage locations for the three design lives specified in the AASHTO: 100,000, 500,000, and 2,000,000 cycles. The computed reliability levels were seen to be realistic in that with the low mean stress levels no difficulty was predicted in the adequacy of the fatigue design of these bridges; the reliability levels were on the order of 99.9 percent or greater, that is, with probabilities of failure, against specified design fatigue lives, of 10^{-3} to 10^{-6}. Thus the RSAF concept for estimating $E(S^m)$ may be introduced into both the expression for mean fatigue life and the reliability formulation and will be a useful parameter to represent the meaured histogram data.

For use in design and analysis the following method is suggested for estimating mean fatigue life:

1. Using the RSAF concept, choose a value of RSAF = 1.2 and modify according to a linear rule to take into account m-values other than m = 3. The results presented in Section 4.3 can be used, or an alternative relationship can be established based on additional field data.

2. Establish the design mean stress range level and coefficient of variation. The mean stress range can be calculated on the basis of a mean

gross vehicle weight adequate for the location; based on current experience, a mean GVW of 50 kips is not unreasonable. The coefficient of variation can be calculated directly using field data if available, or the coefficient of variation of the GVW histograms can be used as an approximation, although there are difficulties with this assumption. A representative value of c.o.v. = 0.45 could be used: this is suggested on the basis of a central value representing the data presented herein.

3. Calculate $E(S^m) = \left[\frac{\text{mean } (1+\text{c.o.v.})}{\text{RSAF}} \right]^m$ and the corresponding mean fatigue life $\bar{n} = c/E(S^m)$.

4. Finally, compare the predicted mean fatigue life with the specified design fatigue life using a fatigue reliability analysis or an appropriate "factor of safety".

The field measurement program included study of an urban location with very high volume traffic, the 18th Street Bridge on the Dan Ryan Expressway; the results suggest several problems which need further research. Locations such as the Dan Ryan Expressway require continuous 24 hour recording and cannot be analyzed with a method which is based upon the identification of individual truck crossing events. A rainfall counting scheme incorporated into the field data collection would be particularly useful in this situation. There are micro-processor based field data acquisition systems (8) for doing rainfall counts on single data channels which can be placed in the field and do not require an instrumentation van. This type of device should be acquired and tested in future studies. The rainfall counting method can be implemented with the present field equipment, but will require changes in recording technique to aid the editing of long duration records.

There remains an ongoing need to obtain additional information on live load stress histograms in bridges under a wider variety of traffic conditions. The data presently are not complete enough to establish with statistical validity the dependence of stress range on daily or seasonal traffic conditions, or shifts in stress levels to be associated with systematic changes in vehicle speed patterns or gross weight trends or legislation. Of course, a high degree of correlation between gross truck weight mean stress range has been established. A limited number of field test sites, with high quality truck data, traffic surveillance and bridge stress measurements would be most fruitful.

Finally, the RSAF concept is seen to be useful in linking field measurements to the design process. The technique has been used as a design concept when defined in terms of maximum design stress levels (6) or defined in terms of the mean stress level (12). The RSAF approach is more stable if used in a formulation involving both mean stress and variance. Ways in which this can be introduced into fatigue design specifications should be investigated. Alternatively, the beta density function model is seen to be adequate for representing the stress history data. Its applicability to design formulations (7) is supported by the field measurements.

The use of the beta function to model the bimodal nature of both the gross weight histogram and the resulting stress range data should be explored. It would be appropriate to investigate the applicability of extreme value statistics to predicting the very highest stress levels to be forecasted for extreme (perhpas illegal or permit type) highway bridge vehicle loadings.

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APPENDIX A

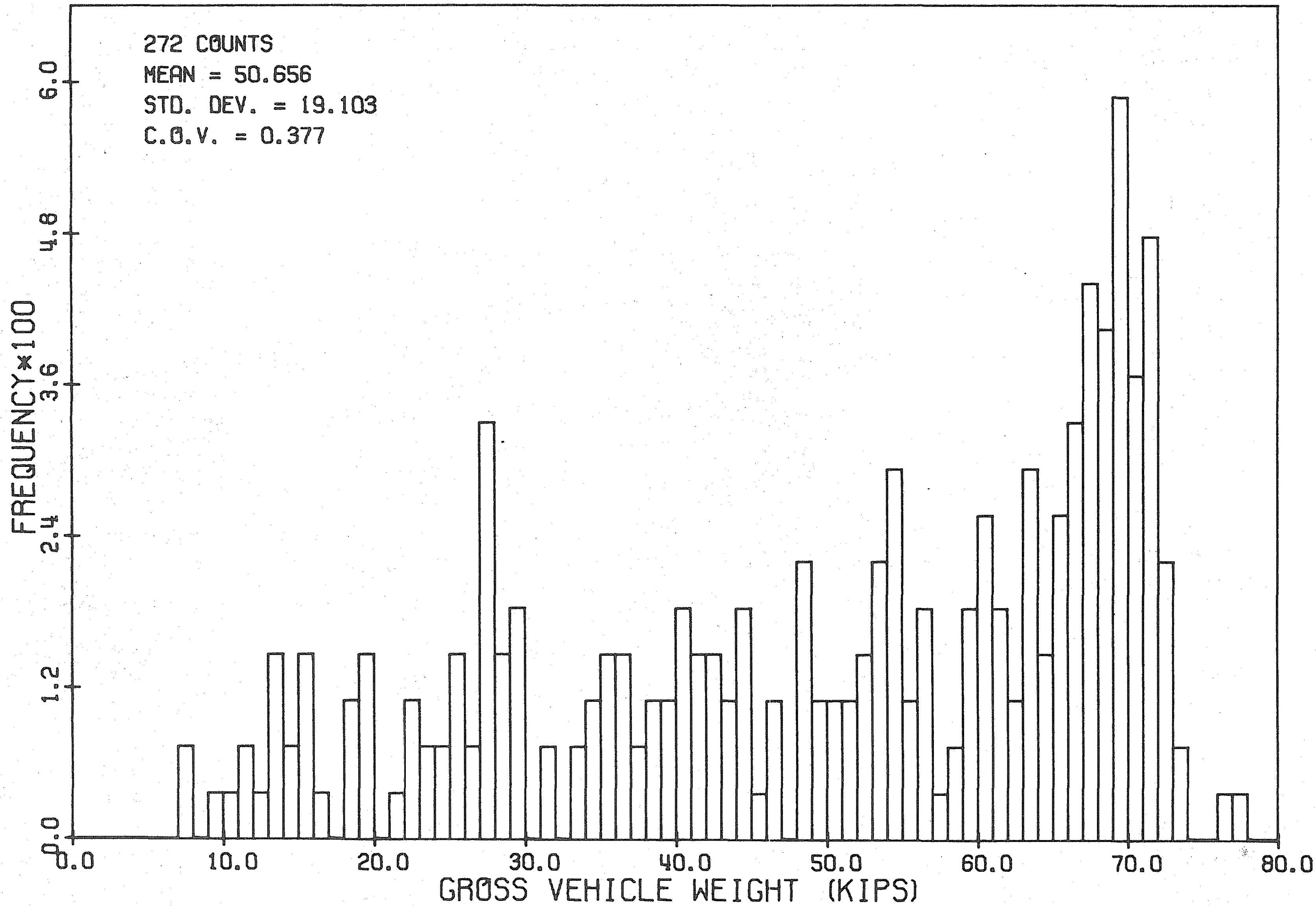

Fig. A.1 Histogram for Gross Vehicle Weight--Green River Bridge--All Trucks, 7/19/76

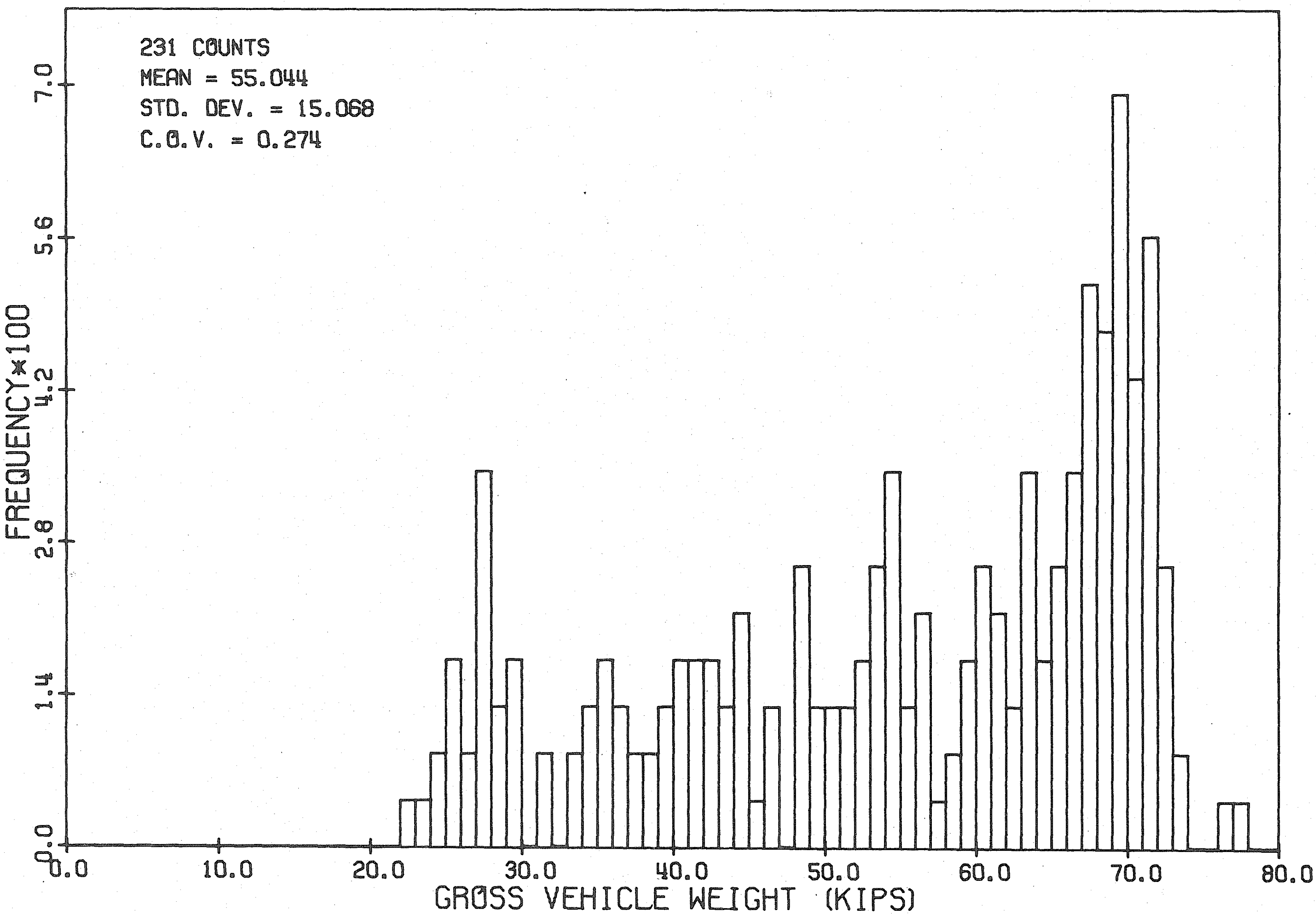

Fig. A.2 Histogram for Gross Vehicle Weight--Green River Bridge, 3S-2 Trucks, 7/19/76

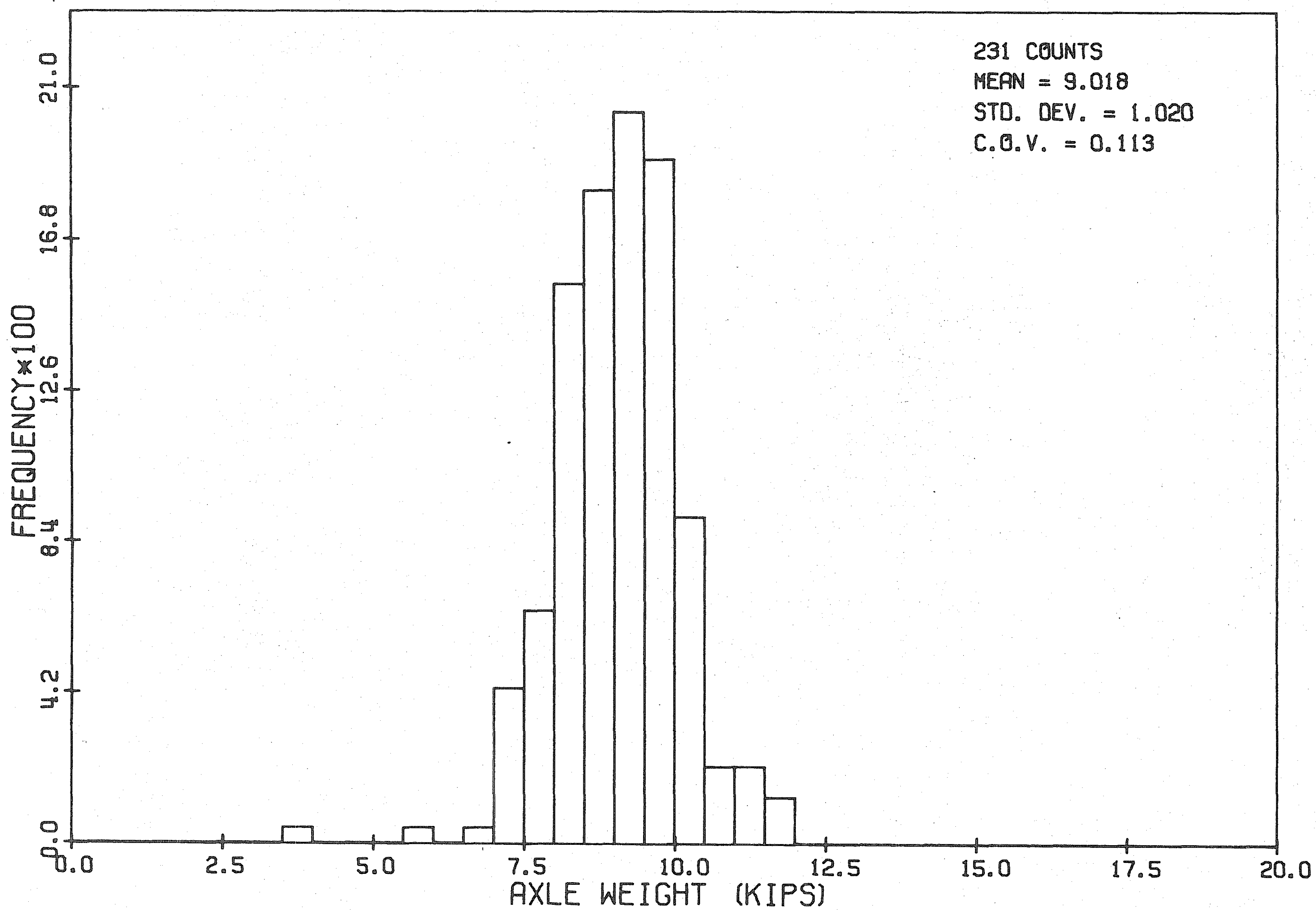

Fig. A.3 Histogram for Axle Weight A, 3S-2 Trucks, Green River Bridge, 7/19/76

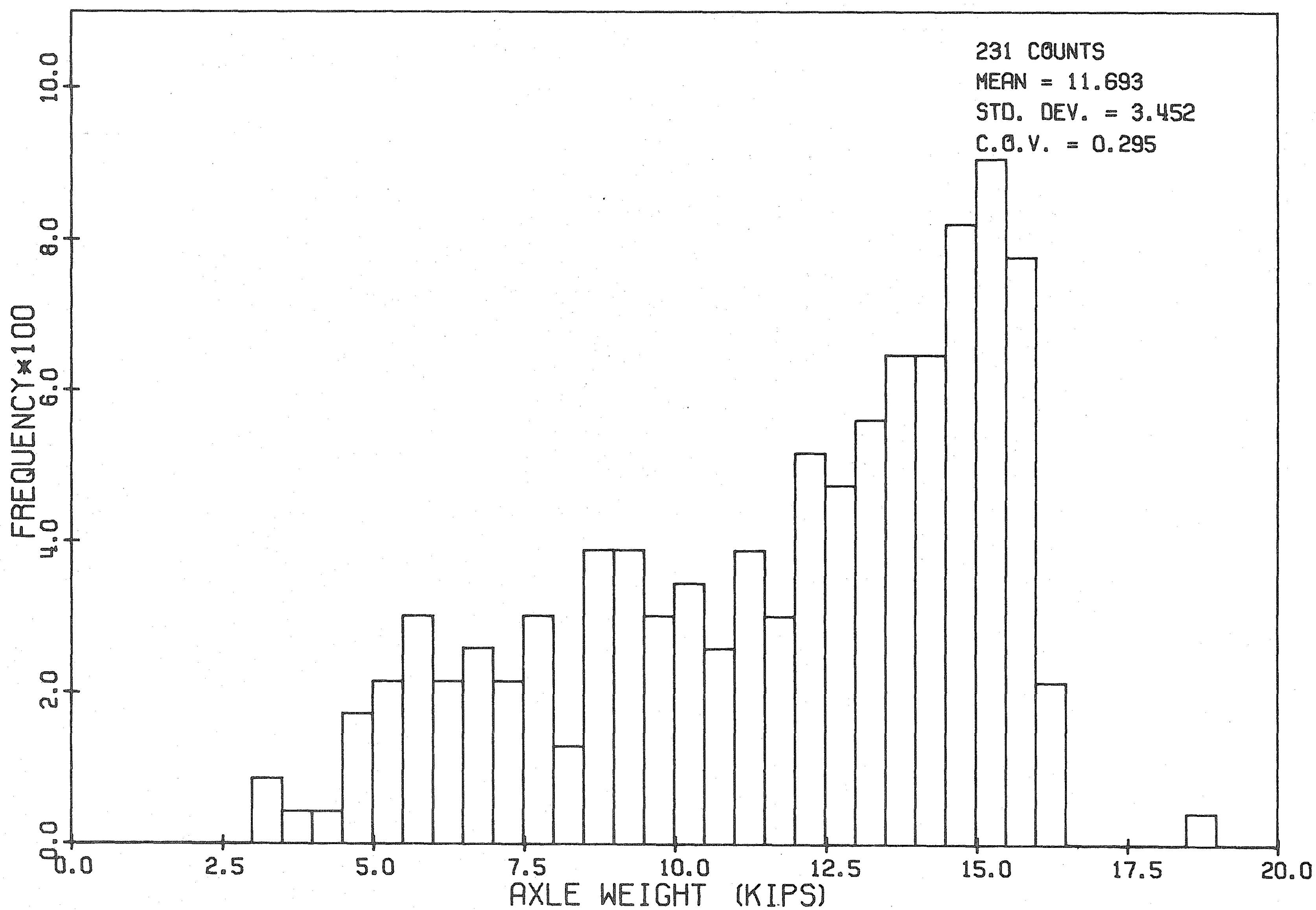

Fig. A.4 Histogram for Axle Weight B, 3S-2 Trucks, Green River Bridge, 7/19/76

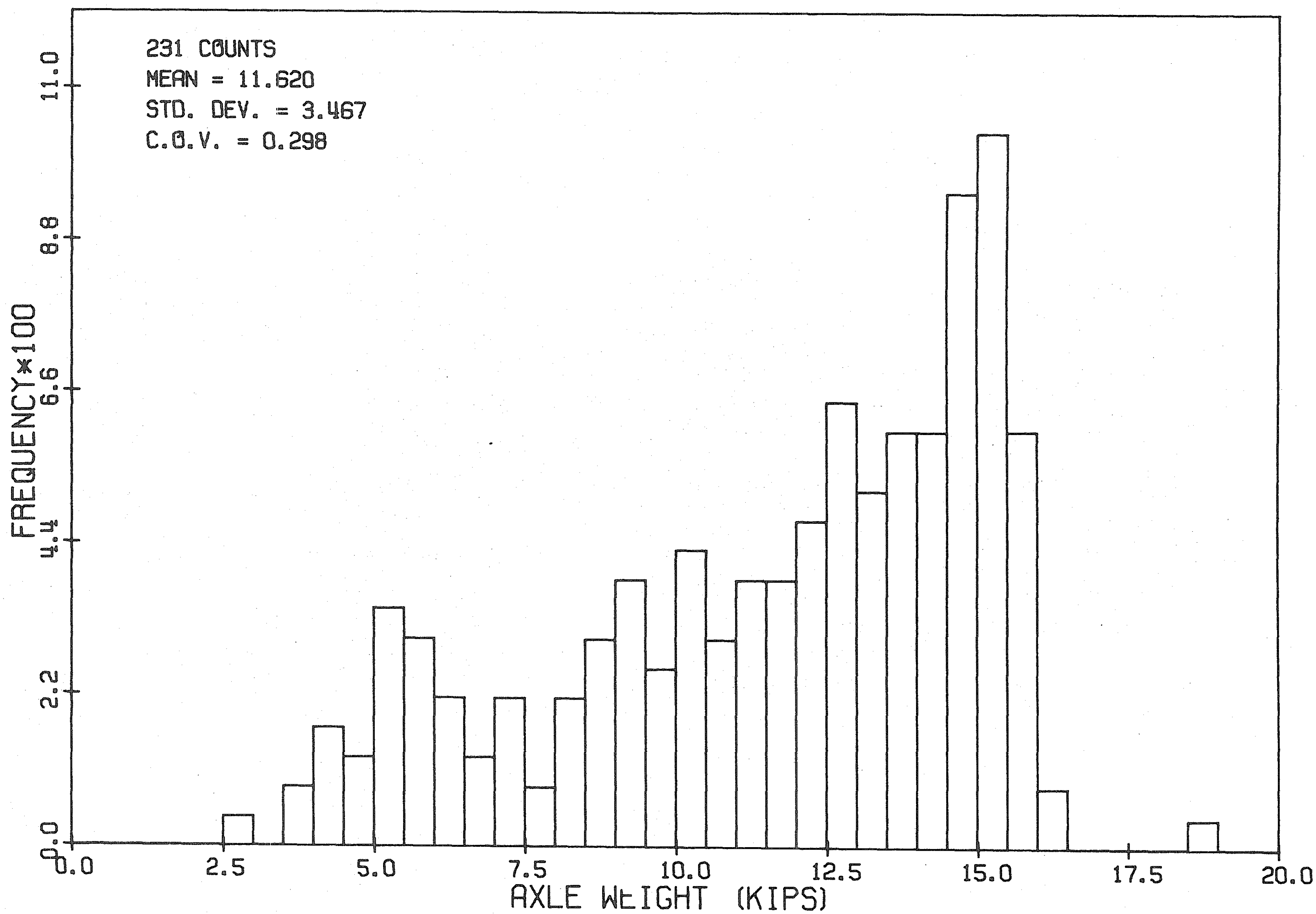

Fig. A.5 Histogram for Axle Weight C, 3S-2 Trucks, Green River Bridge, 7/19/76

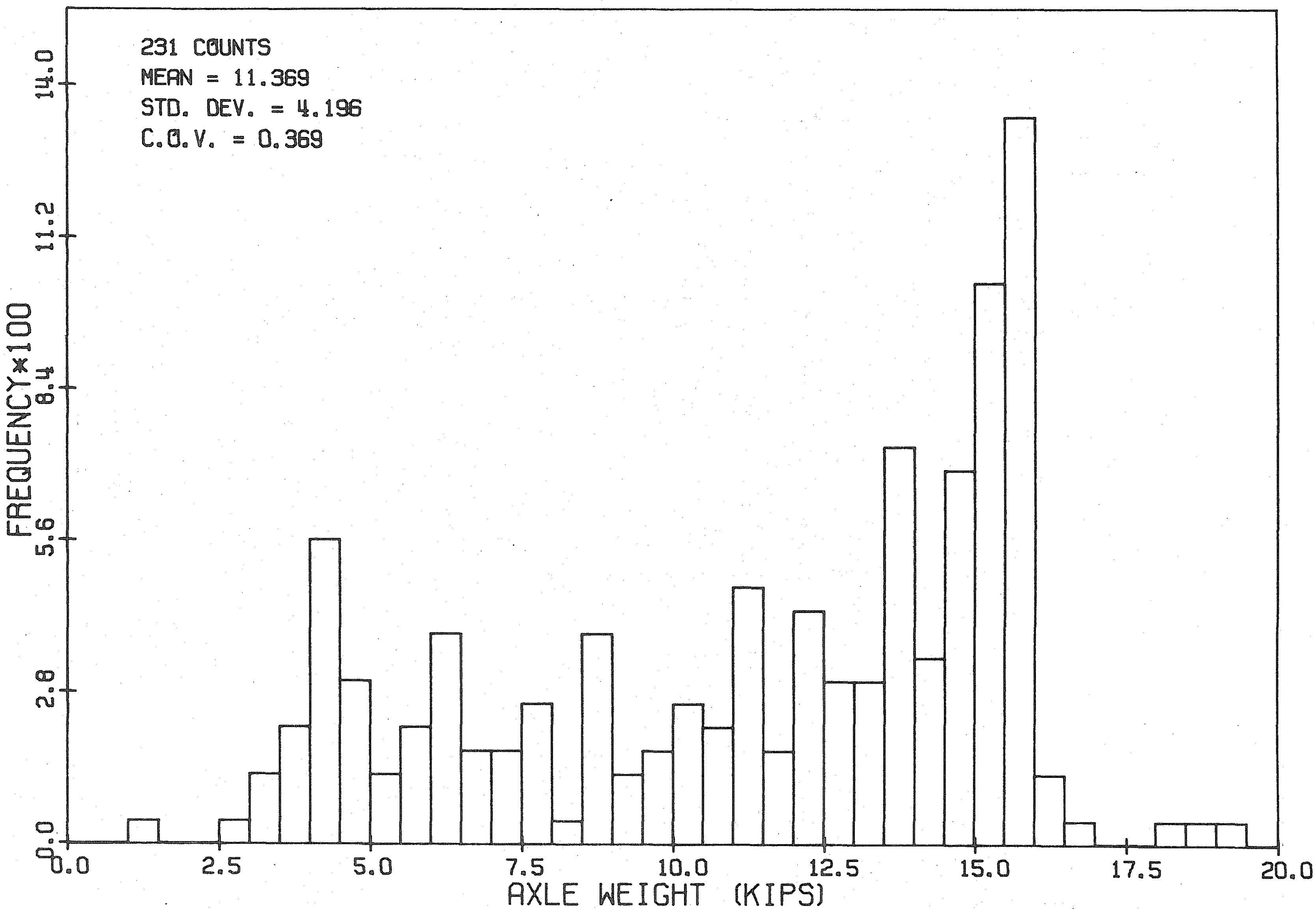

Fig. A.6 Histogram for Axle Weight D, 3S-2 Trucks, Green River Bridge, 7/19/76

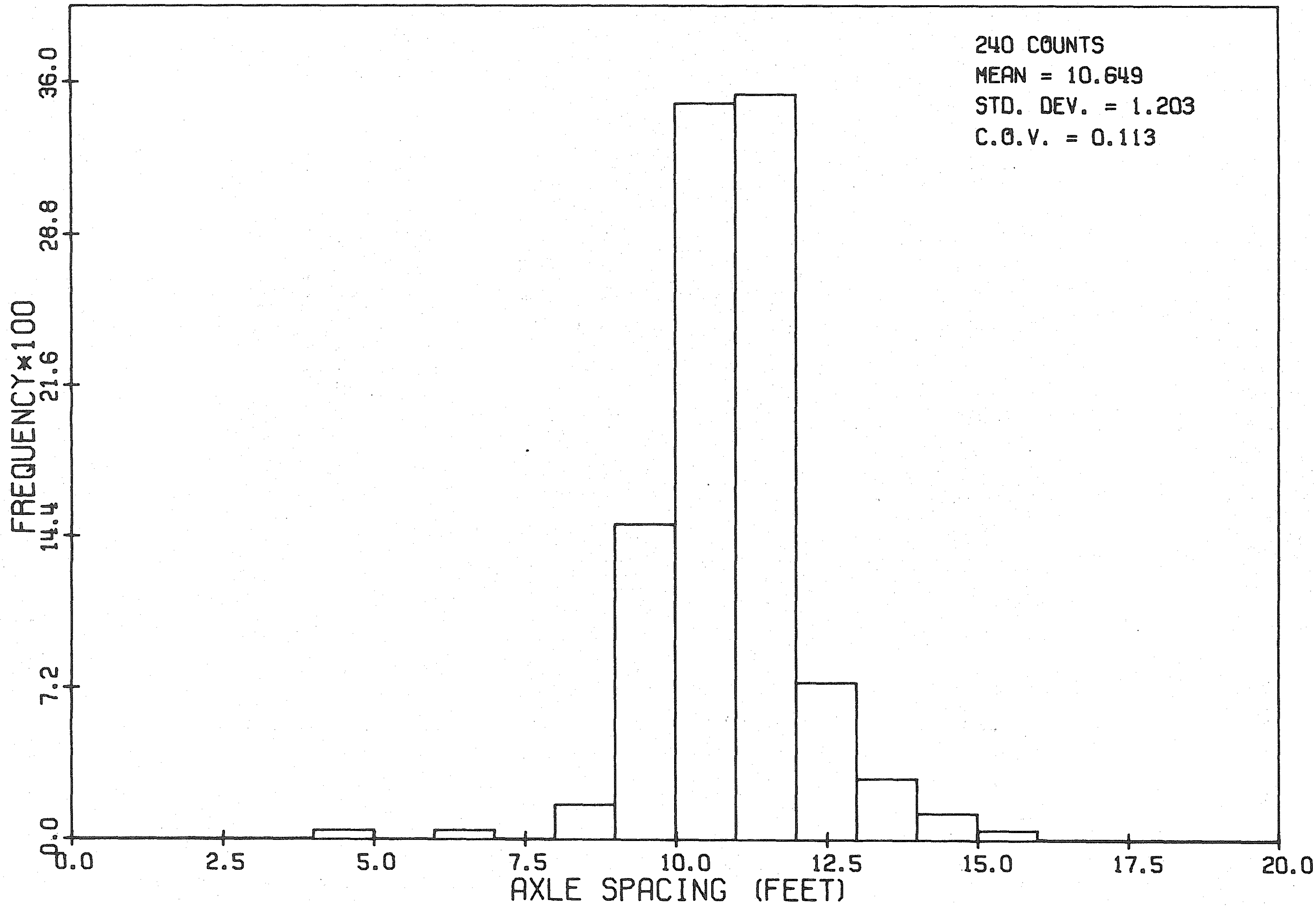

Fig. A.7 Histogram for Axle Spacing A-B, 3S-2 Trucks, Green River Bridge, 7/19/76

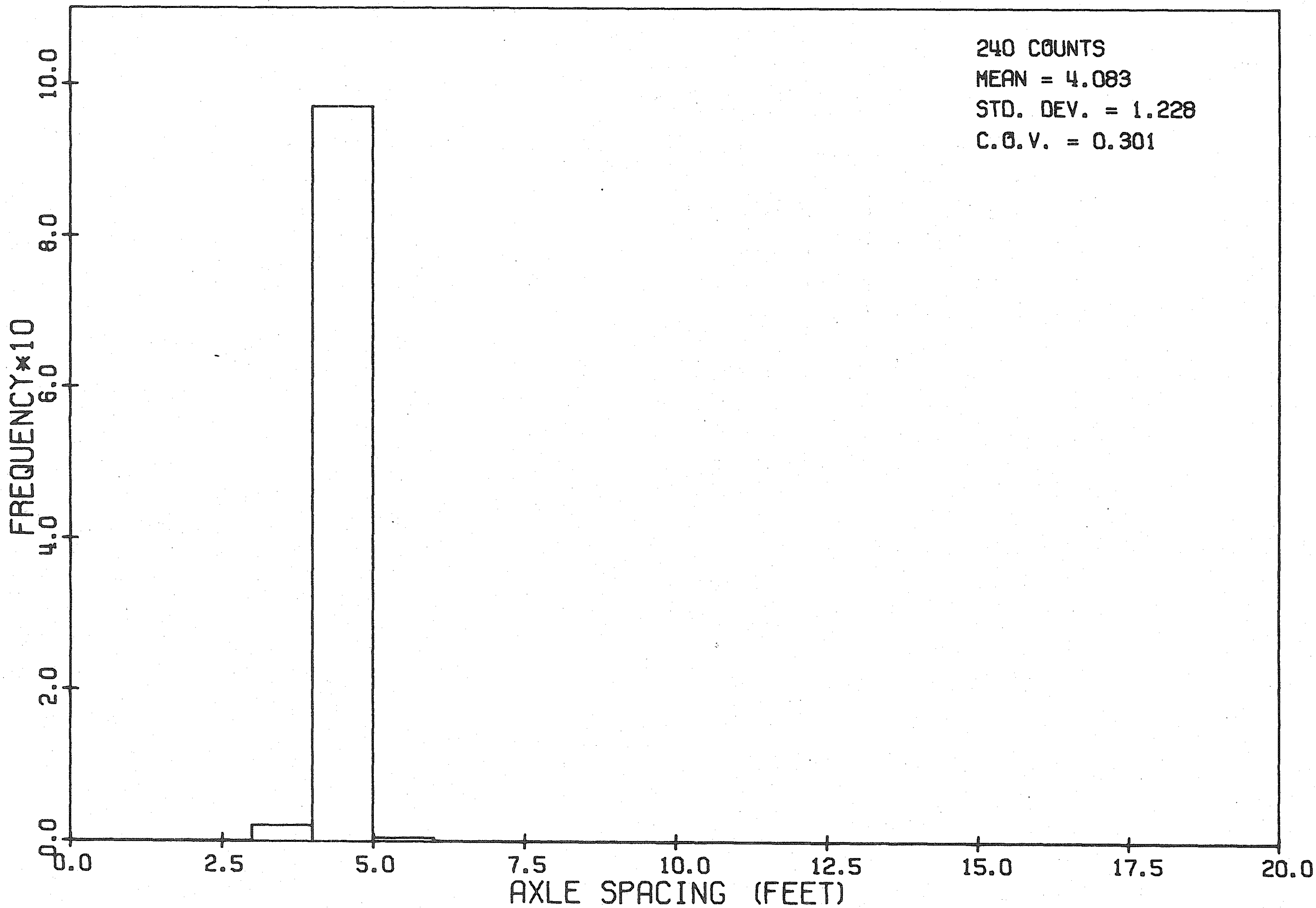

Fig. A.8 Histogram for Axle Spacing B-C, 3S-2 Trucks, Green River Bridge, 7/19/76

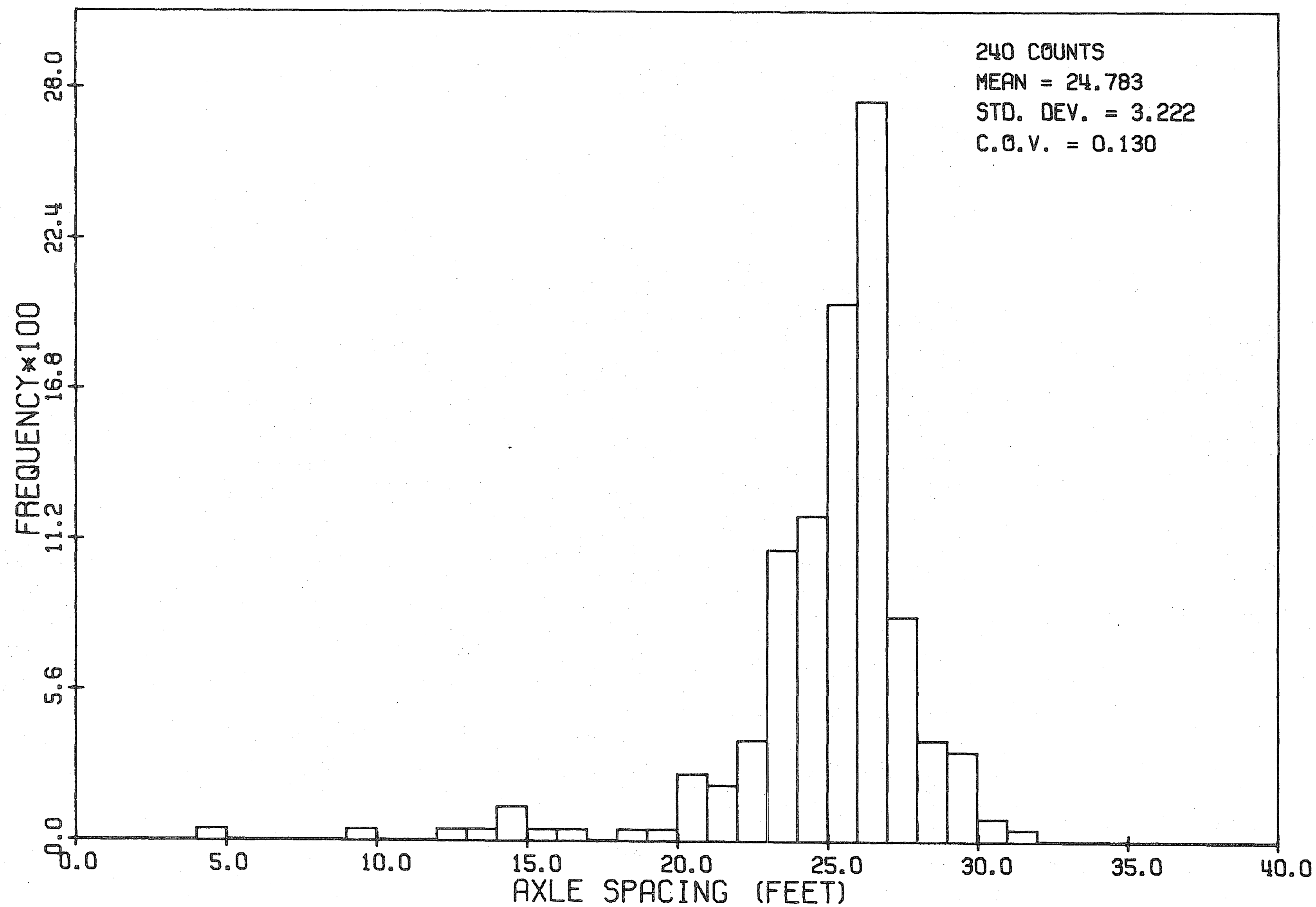

Fig. A.9 Histogram for Axle Spacing C-D, 3S-2 Trucks, Green River Bridge, 7/19/76

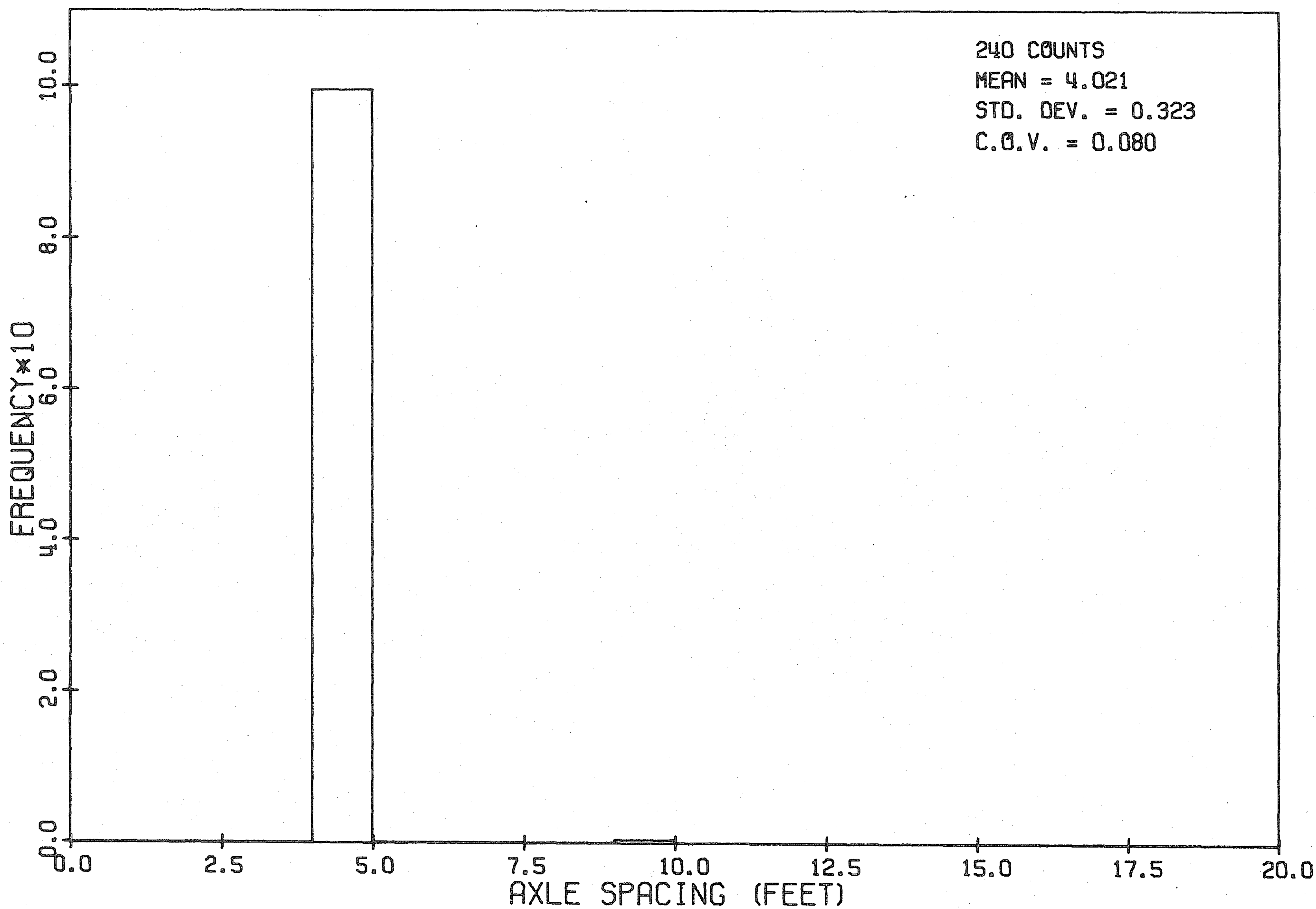

Fig. A.10 Histogram for Axle Spacing D-E, 3S-2 Trucks, Green River Bridge, 7/19/76

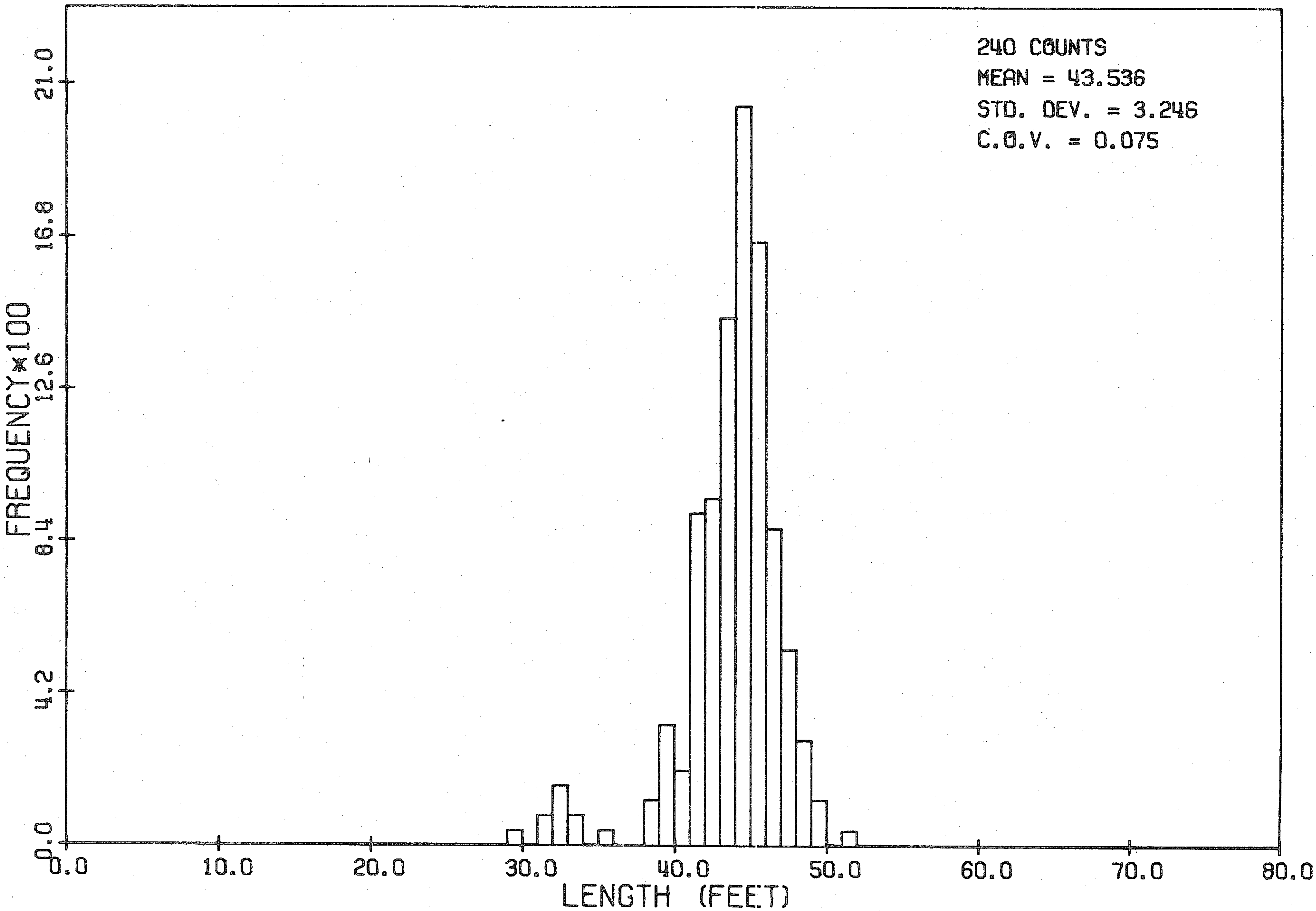

Fig. A.11 Histogram for Total Length, 3S-2 Trucks, Green River Bridge, 7/19/76

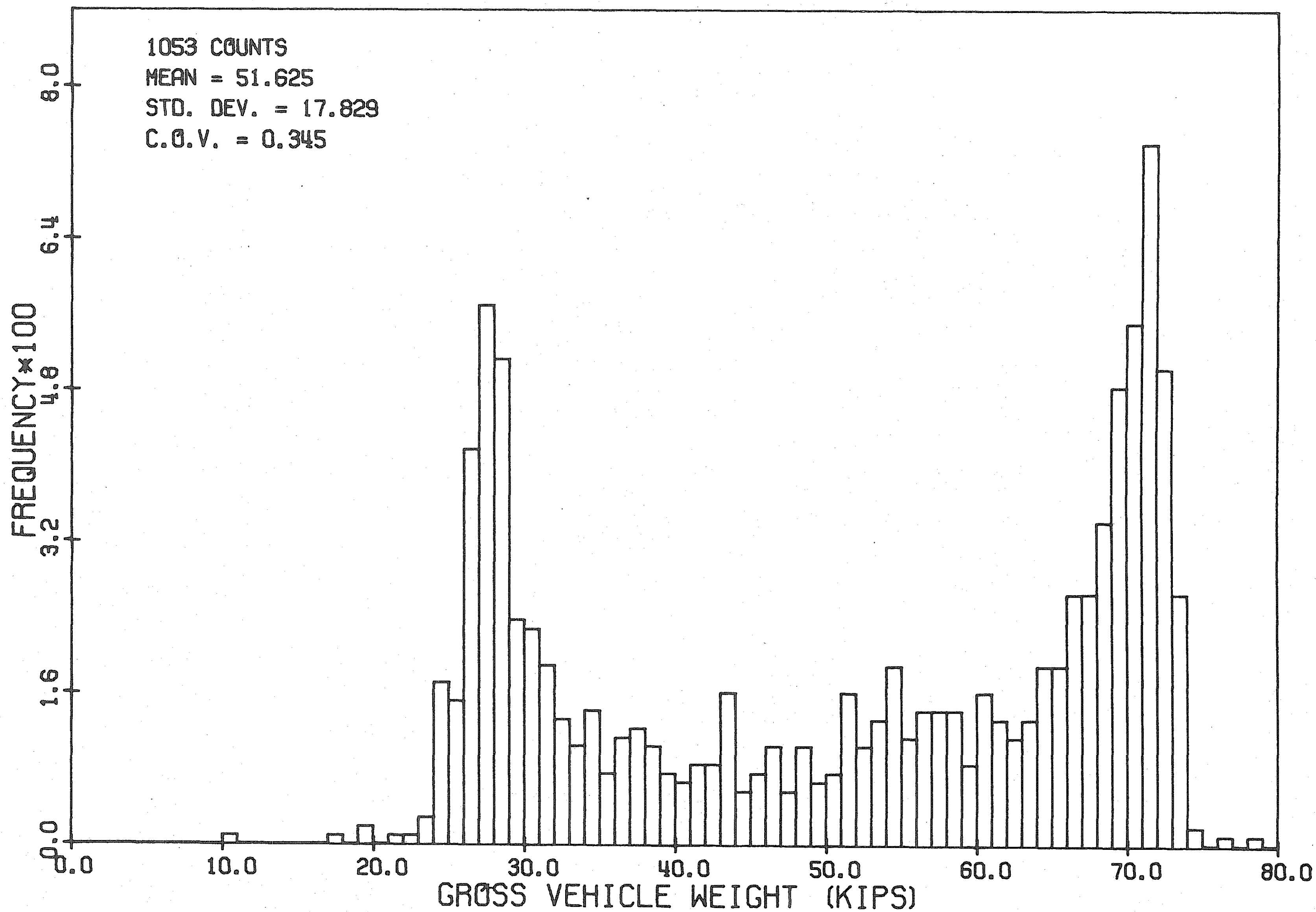

Fig. A.12 Histogram for Gross Vehicle Weight--Camp Creek, Phase I, All Trucks, 9/21/76

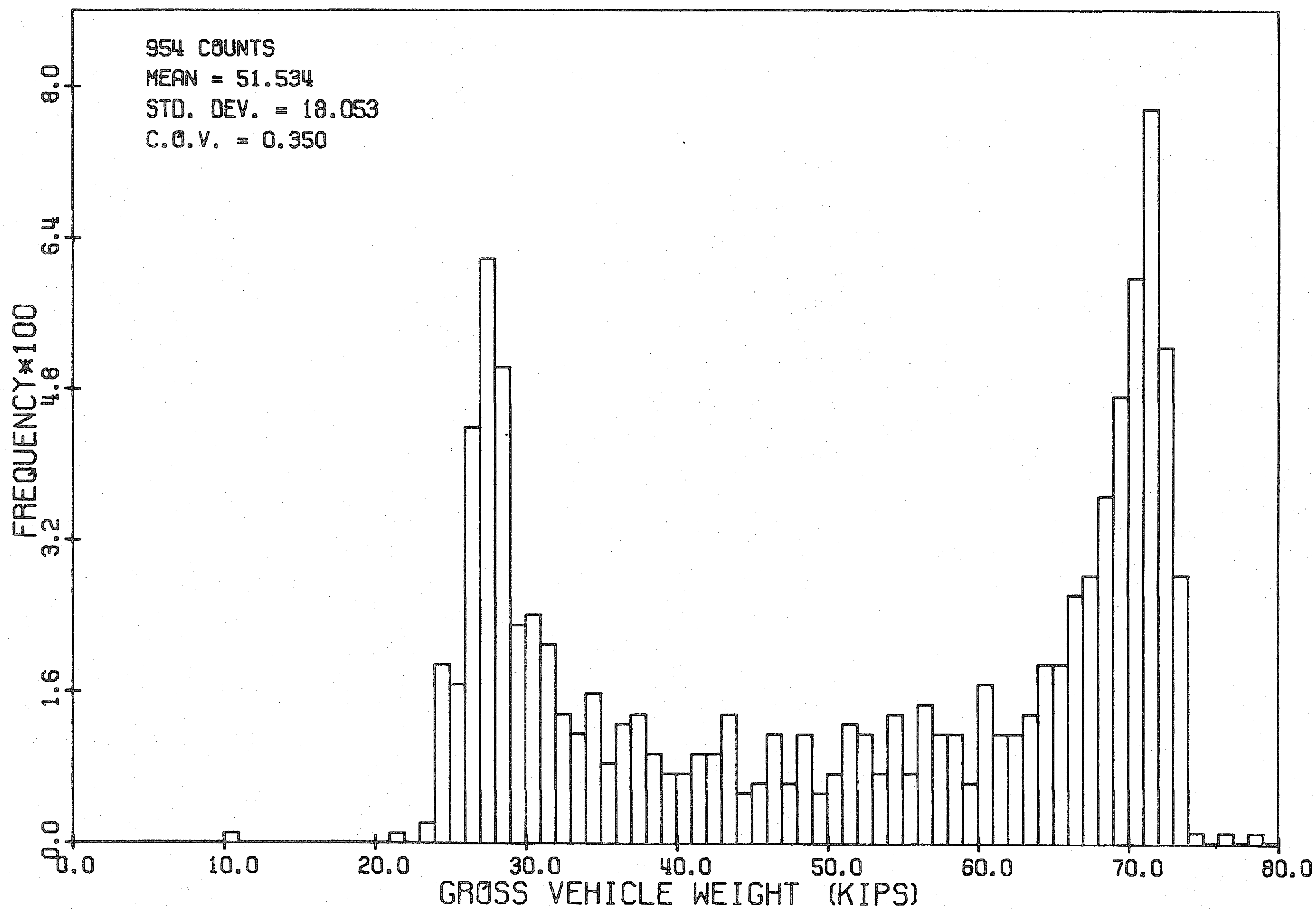

Fig. A.13 Histogram for Gross Vehicle Weight--Camp Creek, Phase I, 3S-2 Trucks, 9/21/76

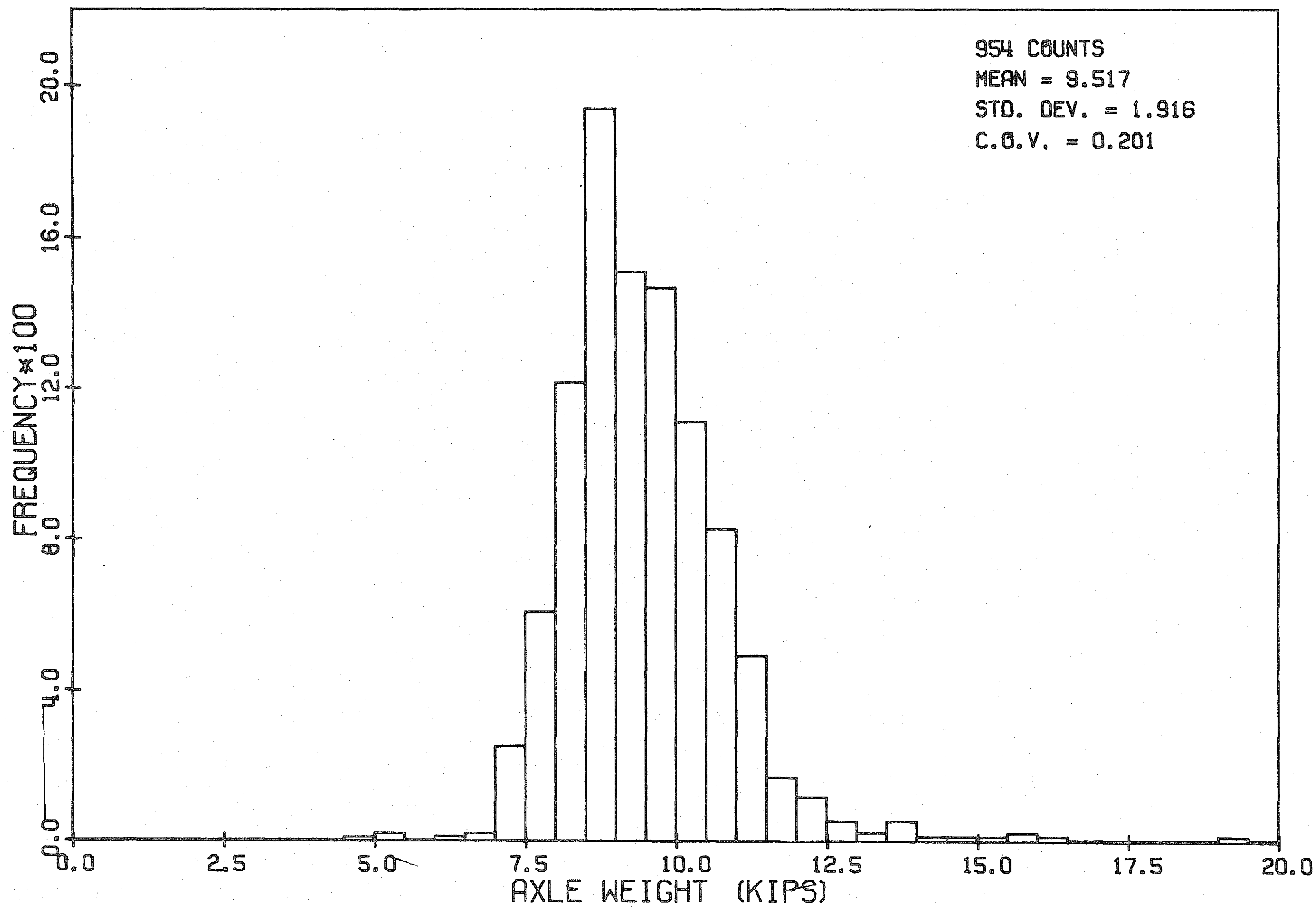

Fig. A.14 Histogram for Axle Weight A, 3S-2 Trucks, Camp Creek Phase I, 9/21/76

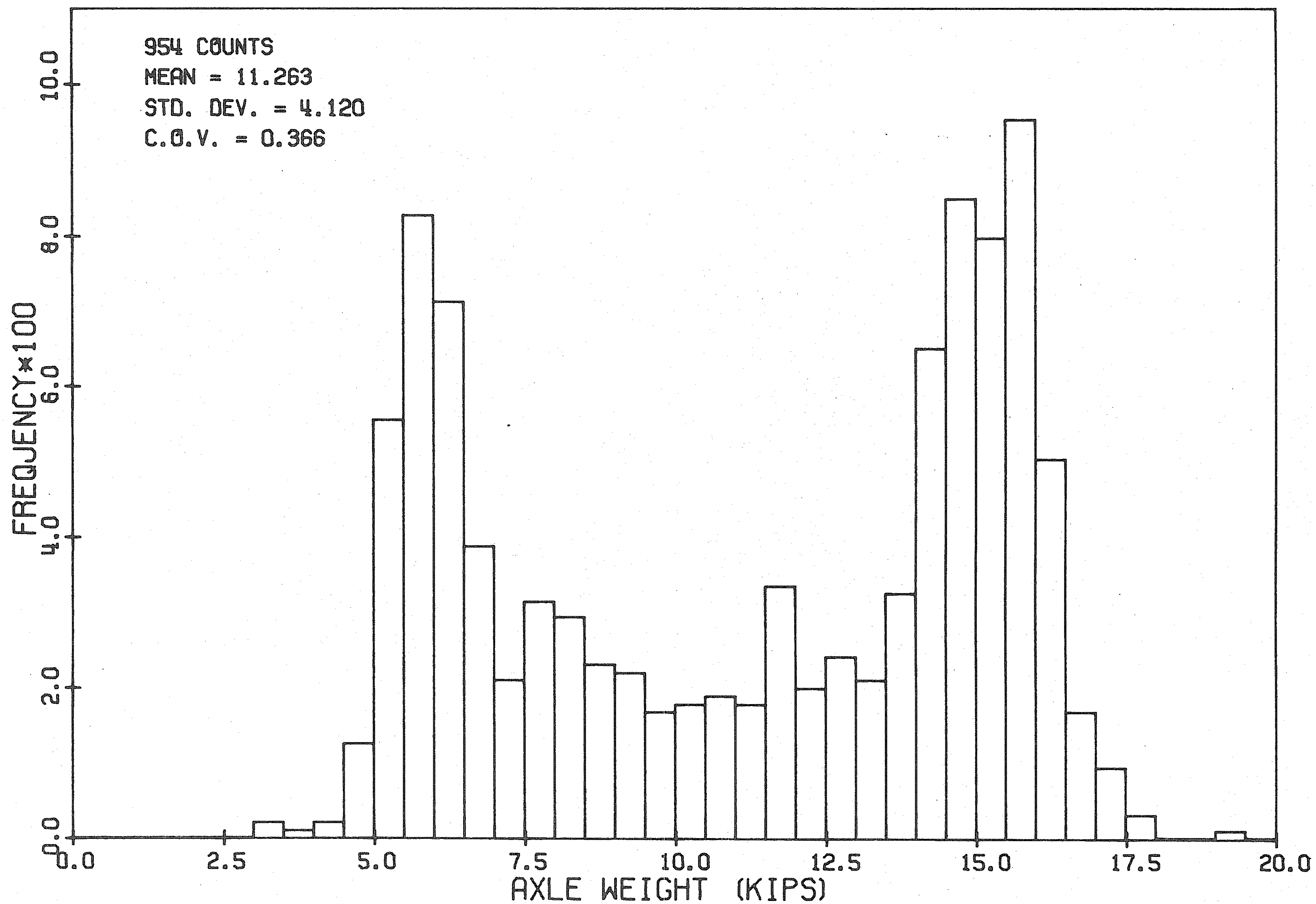

Fig. A.15 Histogram for Axle Weight B, 3S-2 Trucks, Camp Creek Phase I, 9/21/76

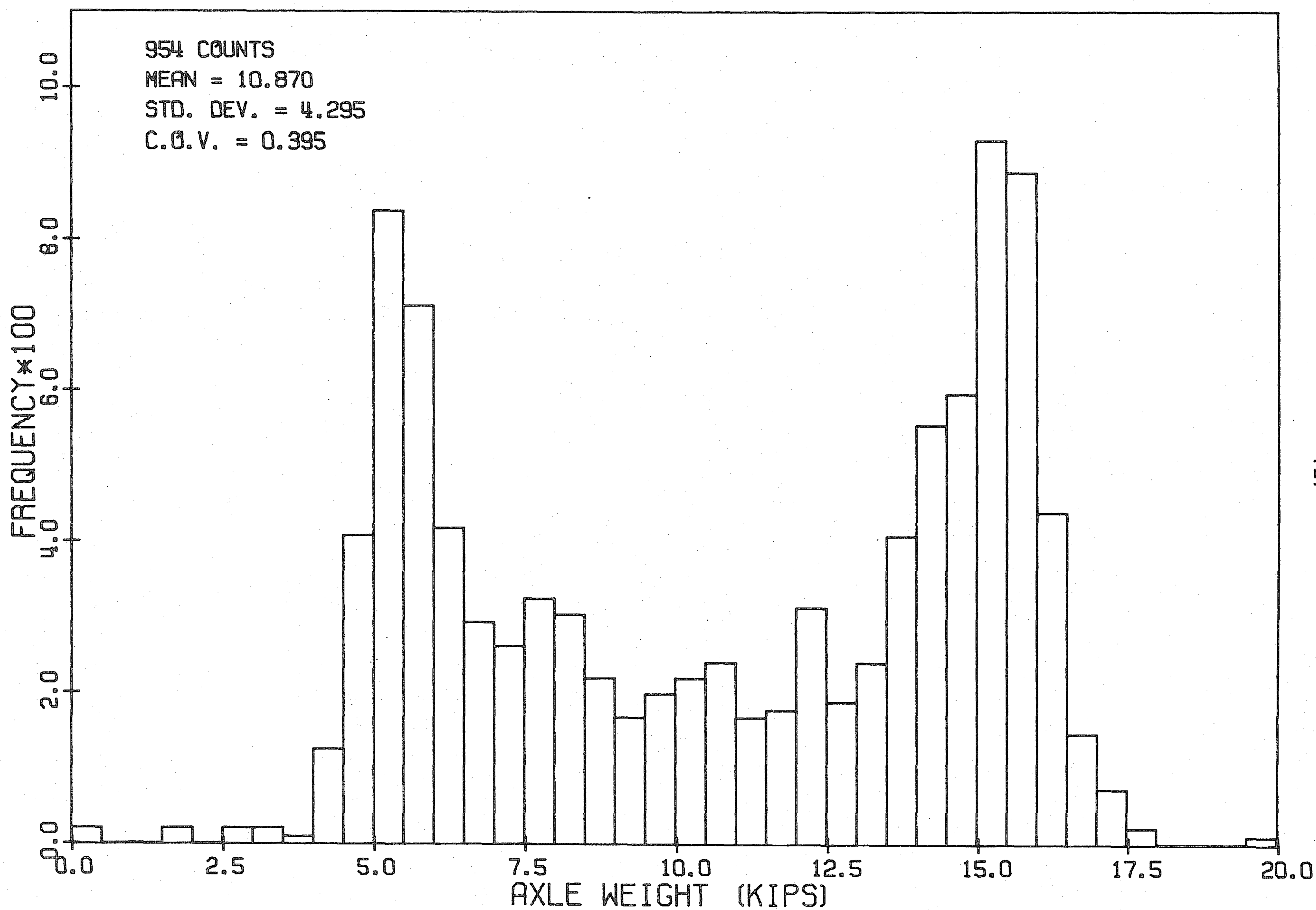

Fig. A.16 Histogram for Axle Weight C, 3S-2 Trucks, Camp Creek Phase I, 9/21/76

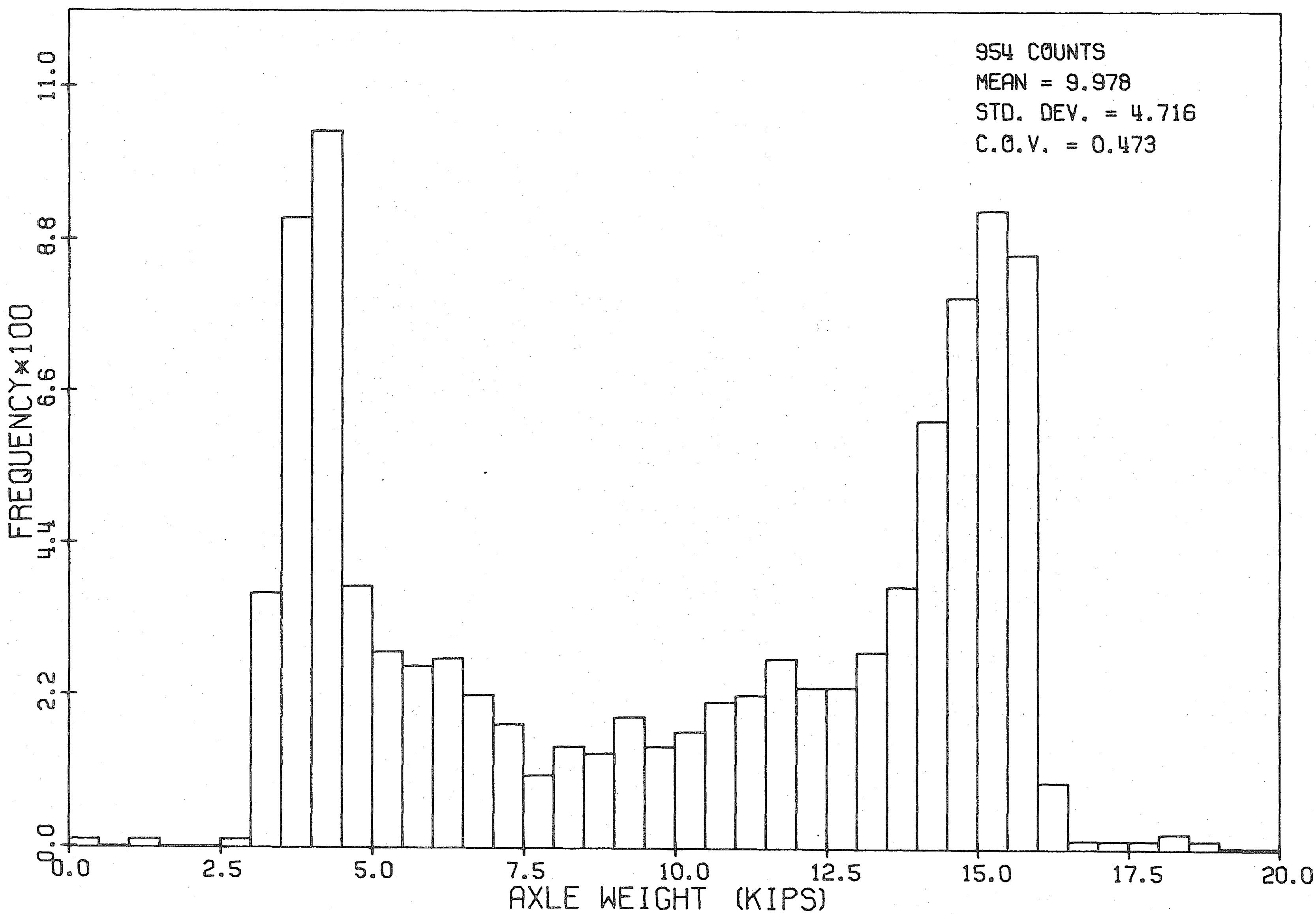

Fig. A.17 Histogram for Axle Weight D, 3S-2 Trucks, Camp Creek Phase I, 9/21/76

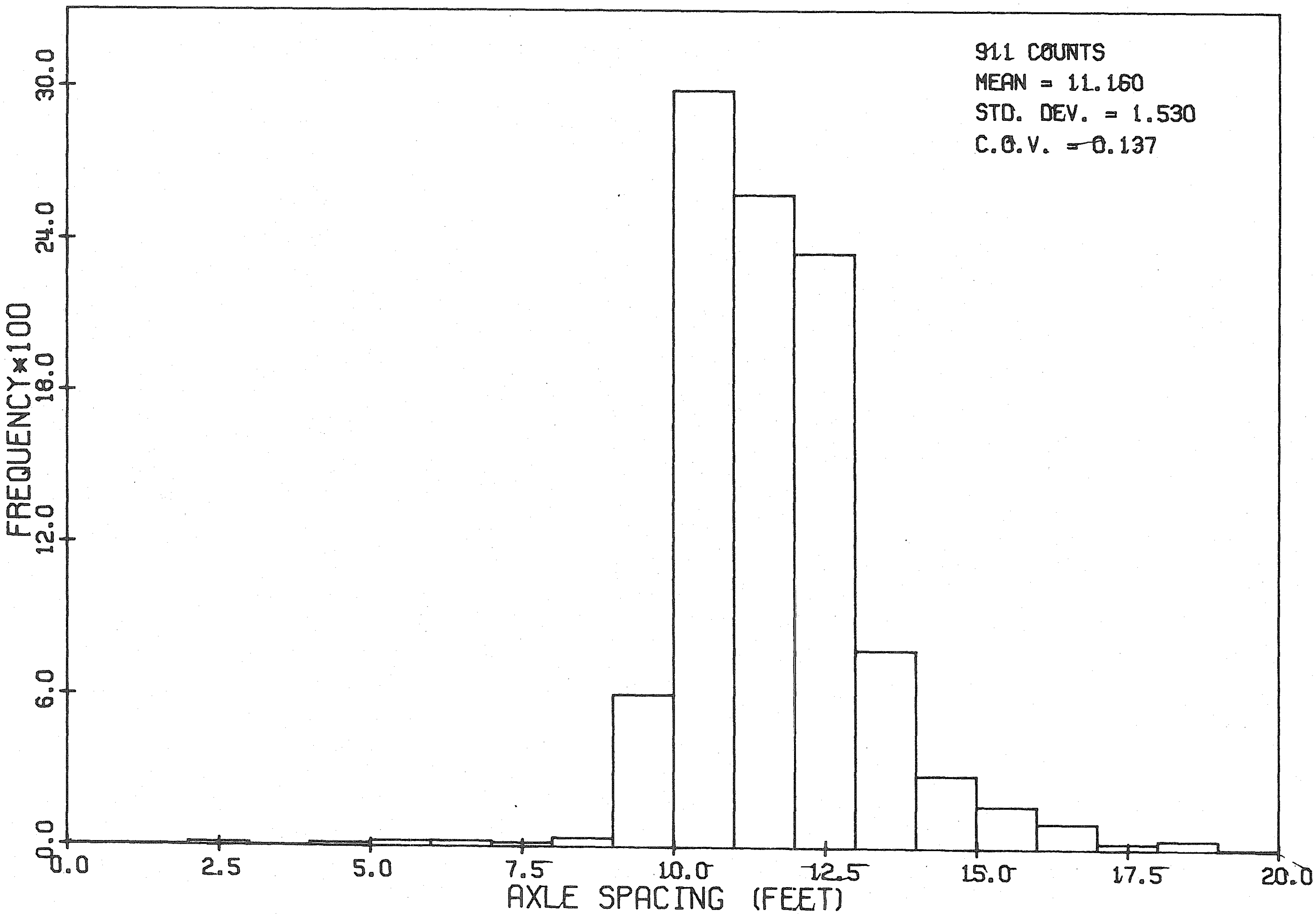

Fig. A.18 Histogram for Axle Spacing A-B, 3S-2 Trucks, Camp Creek Phase 1, 9/21/76

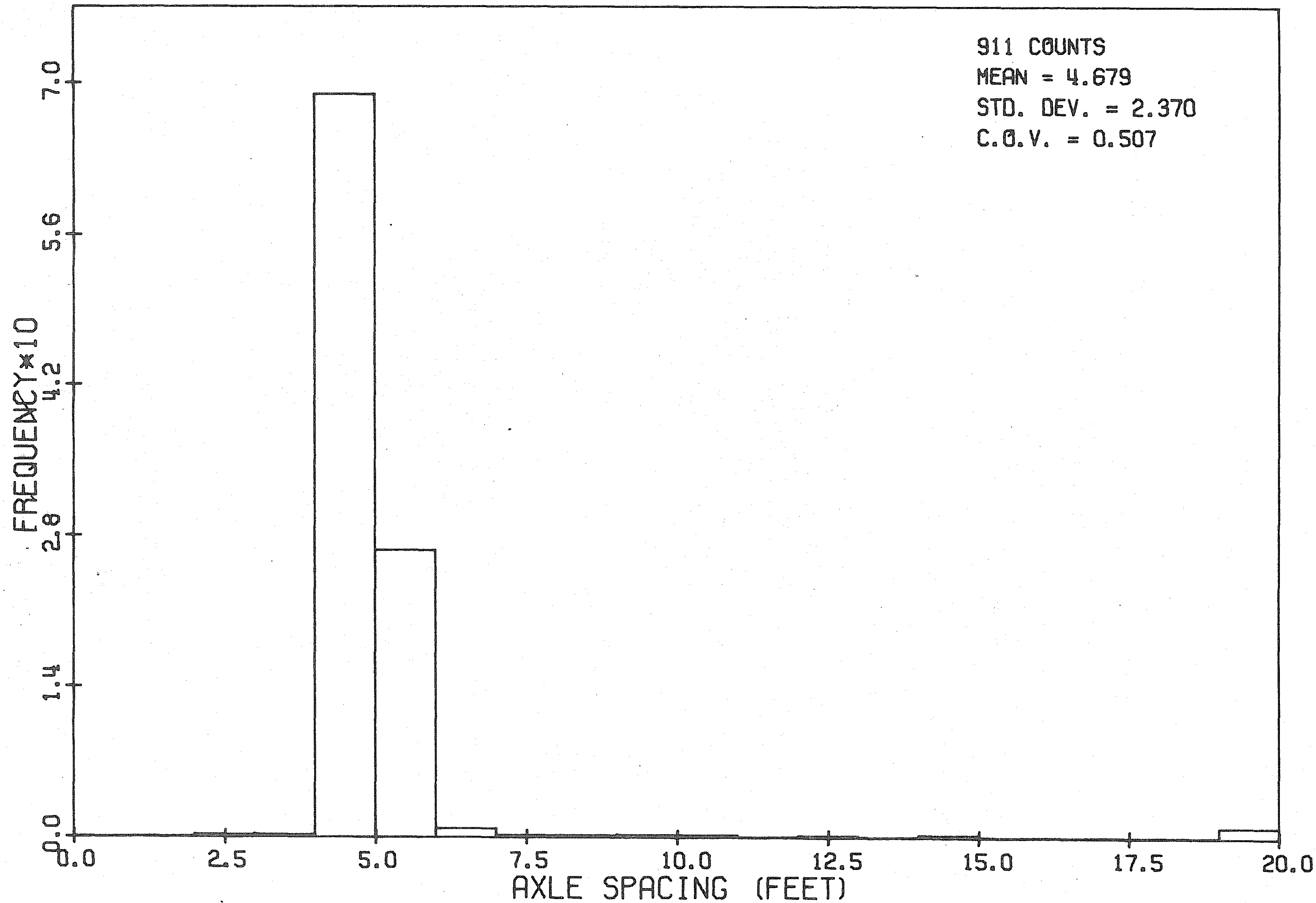

Fig. A.19 Histogram for Axle Spacing B-C, 3S-2 Trucks, Camp Creek Phase I, 9/21/76

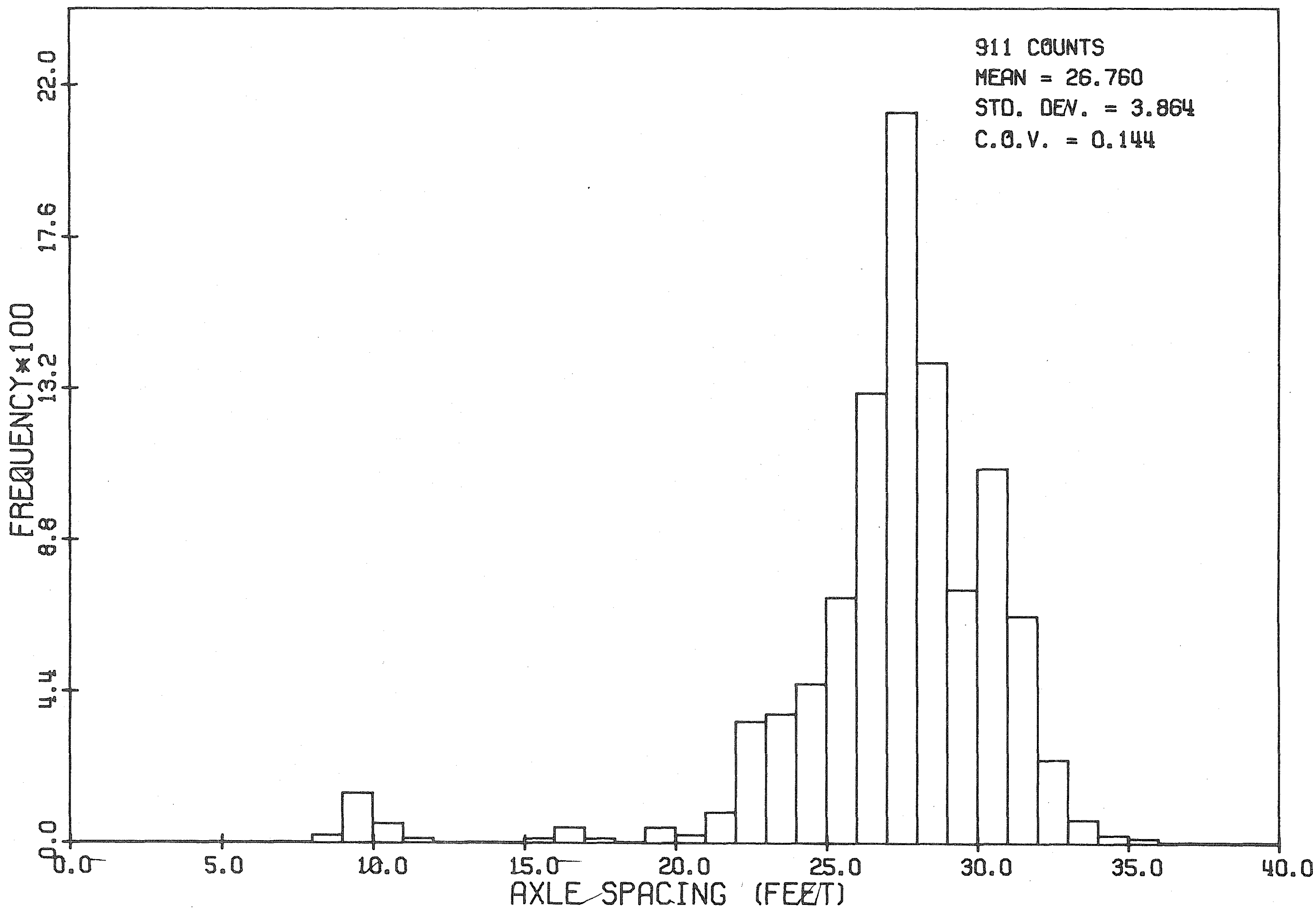

Fig. A.20 Histogram for Axle Spacing C-D, 3S-2 Trucks, Camp Creek Phase I, 9/21/76

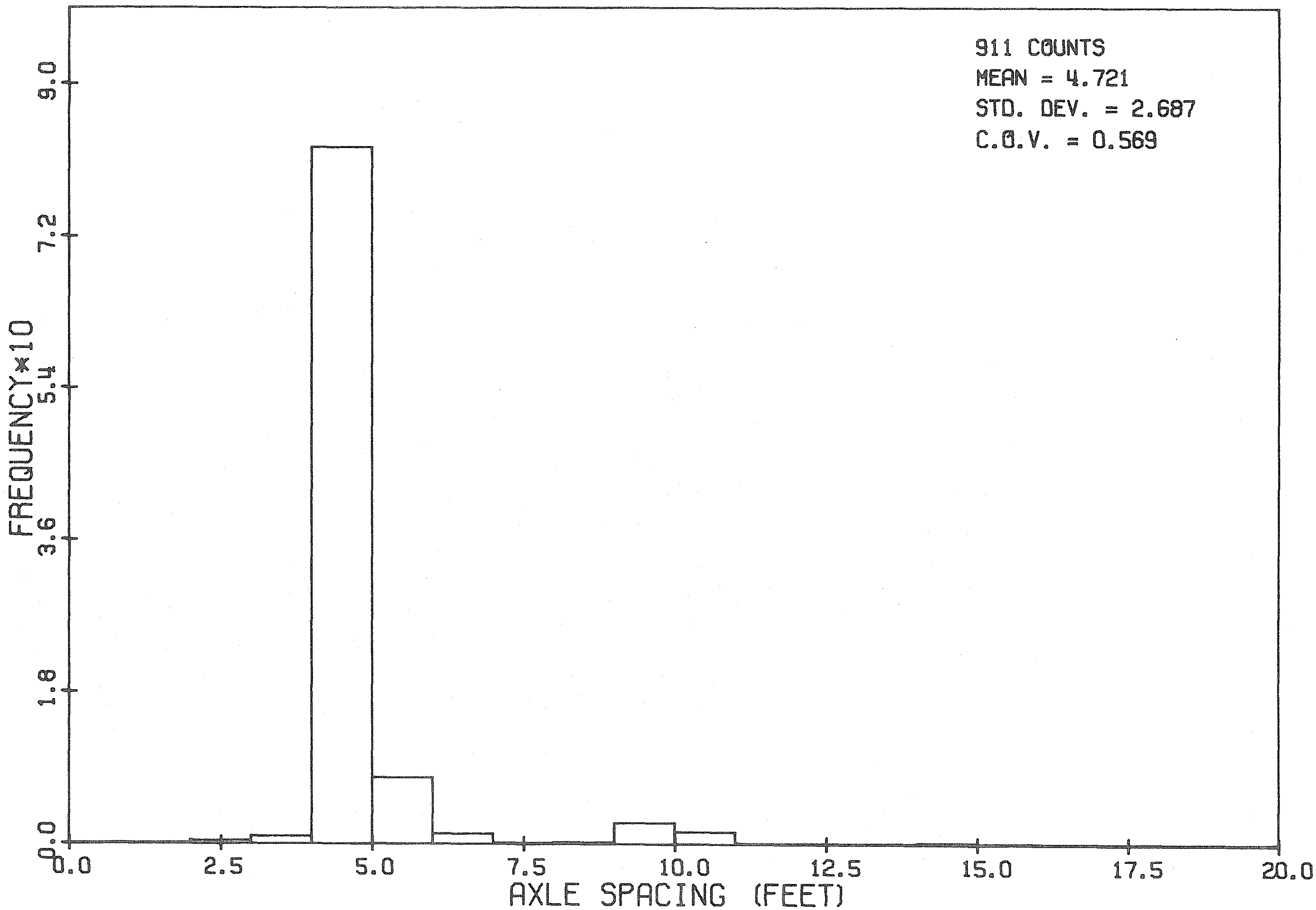

Fig. A.21 Histogram for Axle Spacing D-E, 3S-2 Trucks, Camp Creek Phase I, 9/21/76

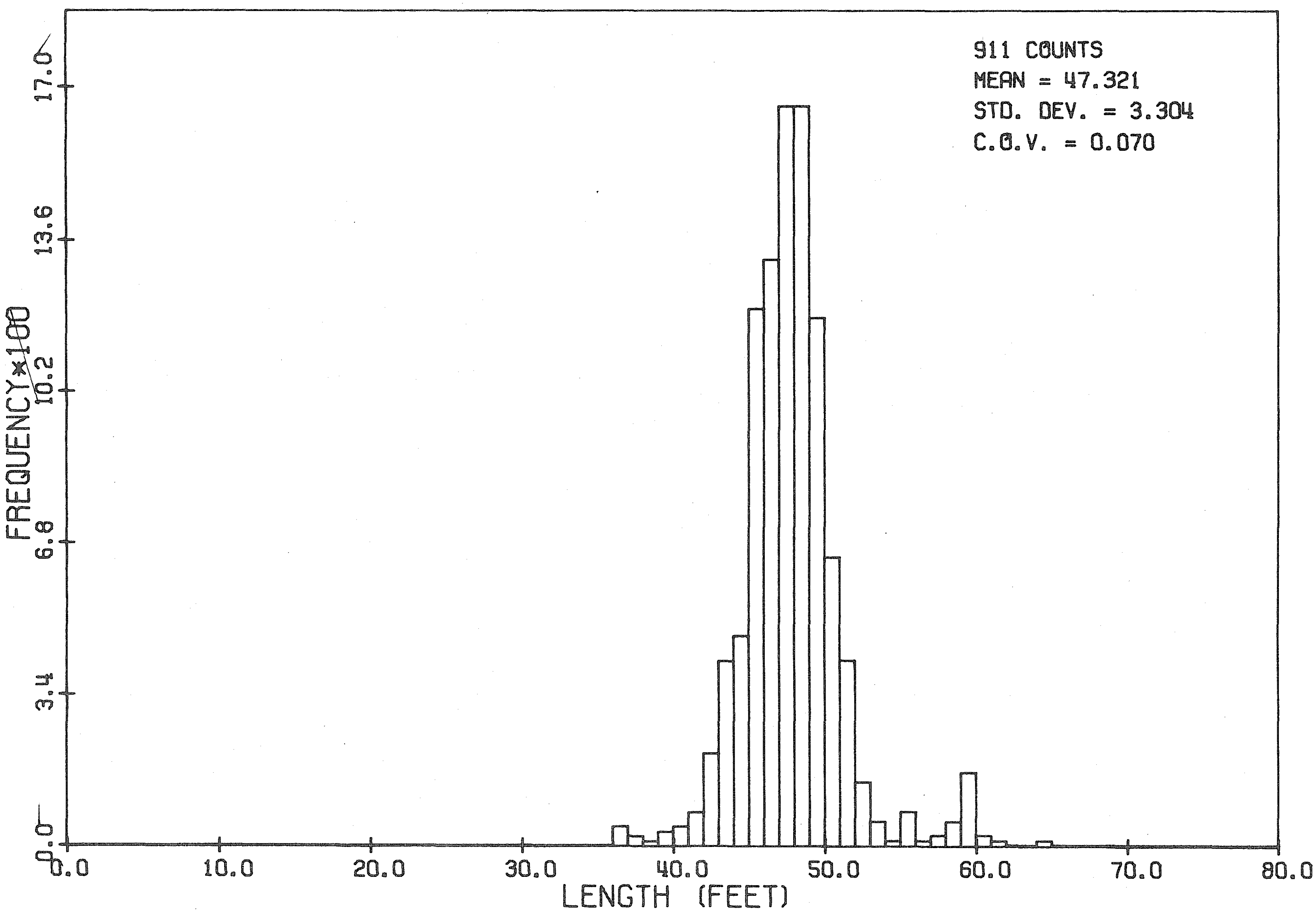

Fig. A.22 Histogram for Total Length, 3S-2 Trucks, Camp Creek Phase I, 9/21/76

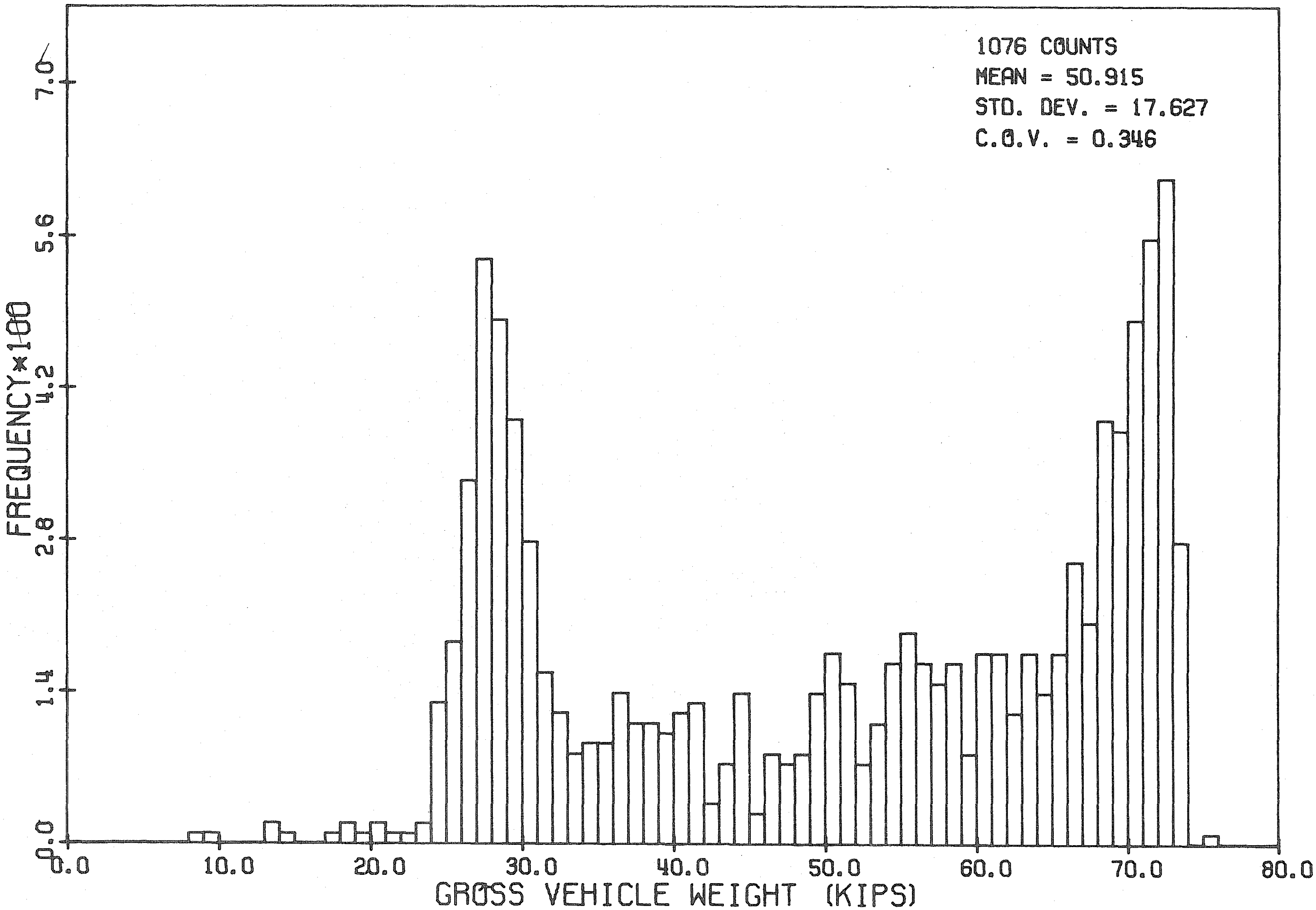

Fig. A.23 Histogram for Gross Vehicle Weight--Camp Creek Phase II, All Trucks, 9/23/76

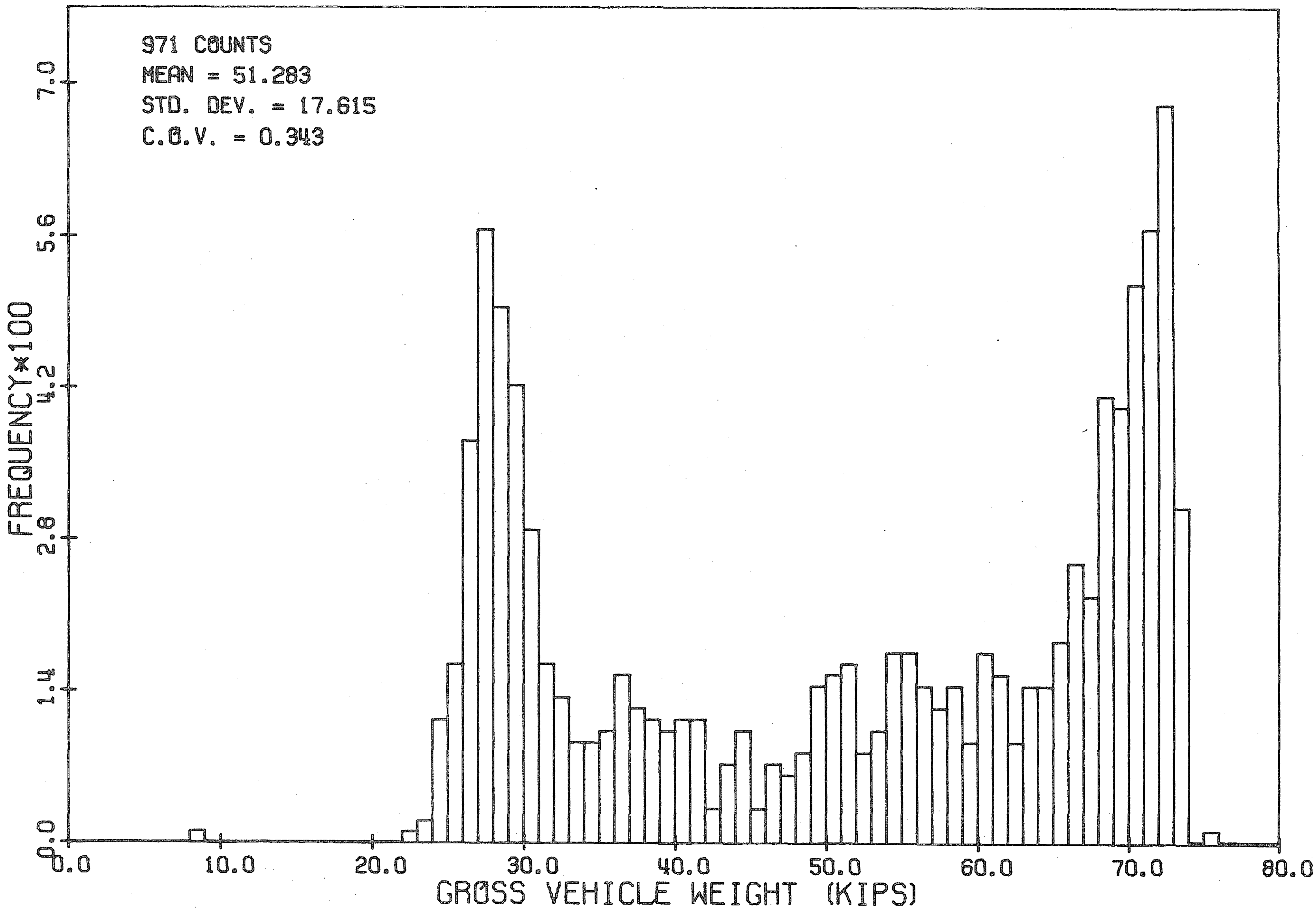

Fig. A.24 Histogram for Gross Vehicle Weight--Camp Creek Phase II, 3S-2 Trucks, 9/23/76

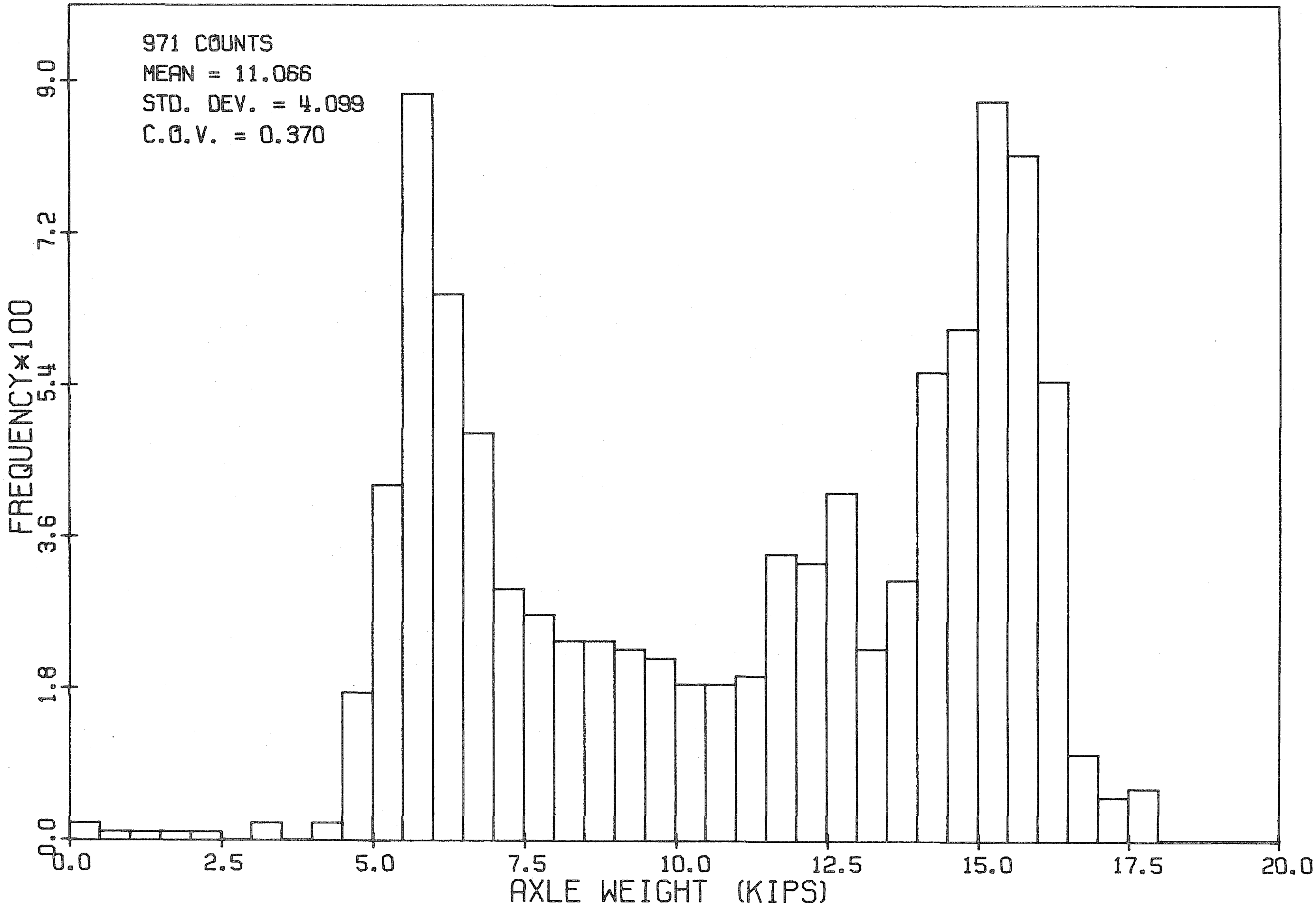

Fig. A.26 Histogram for Axle Weight B, 3S-2 Trucks, Camp Creek Phase II, 9/23/76

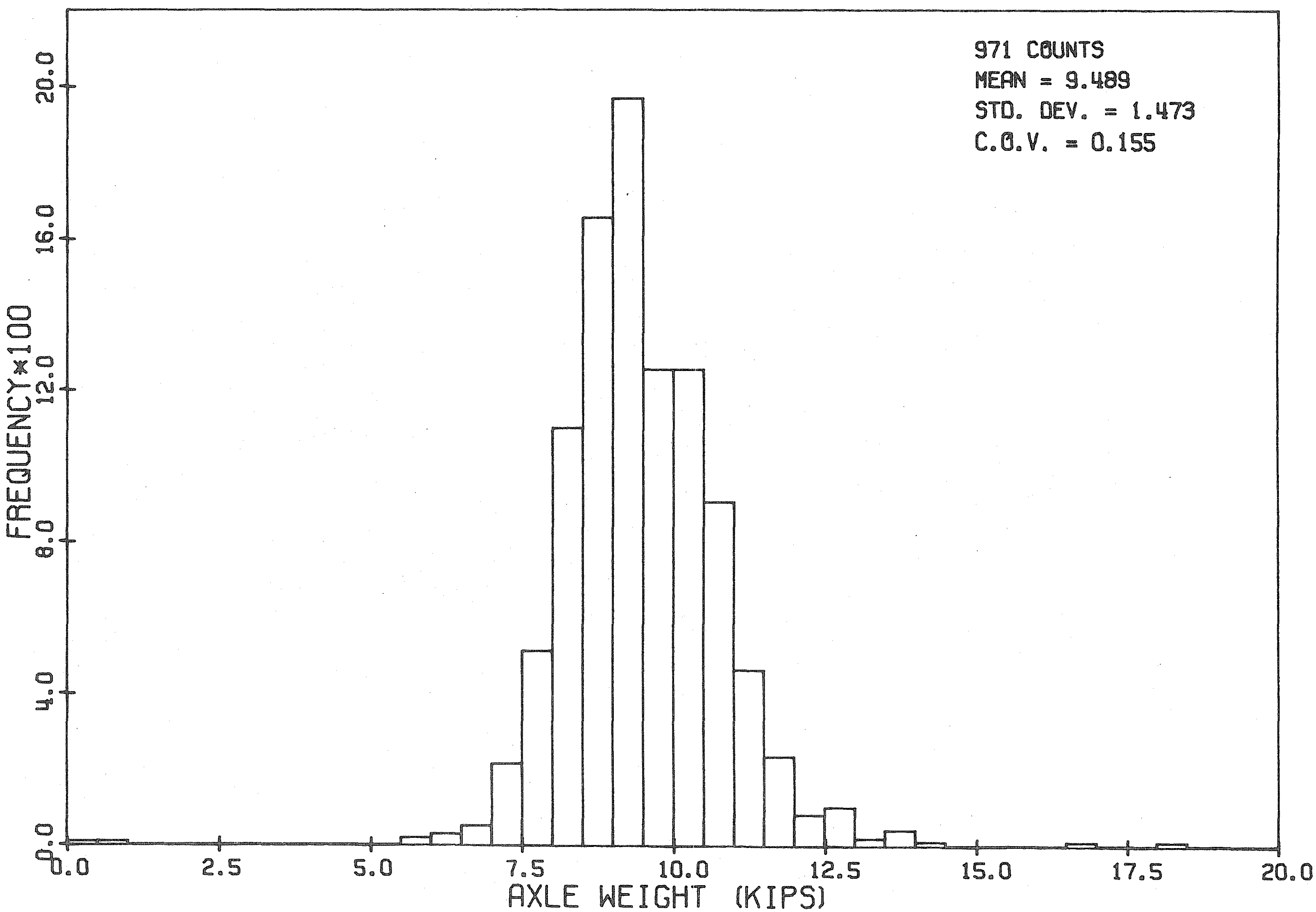

Fig. A.25 Histogram for Axle Weight A, 3S-2 Trucks, Camp Creek Phase II, 9/23/76

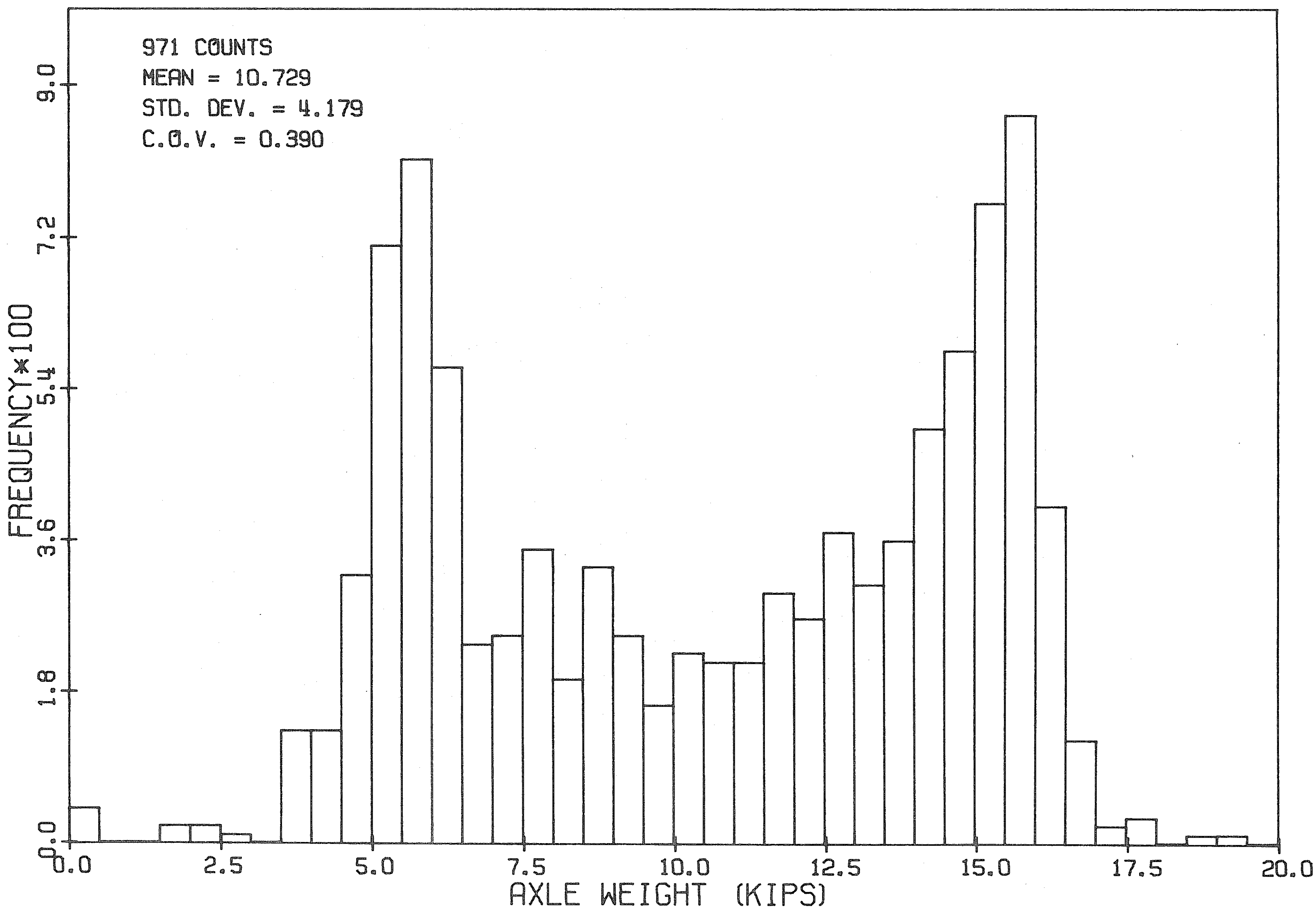

Fig. A.27 Histogram for Axle Weight C, 3S-2 Trucks, Camp Creek Phase II, 9/23/76

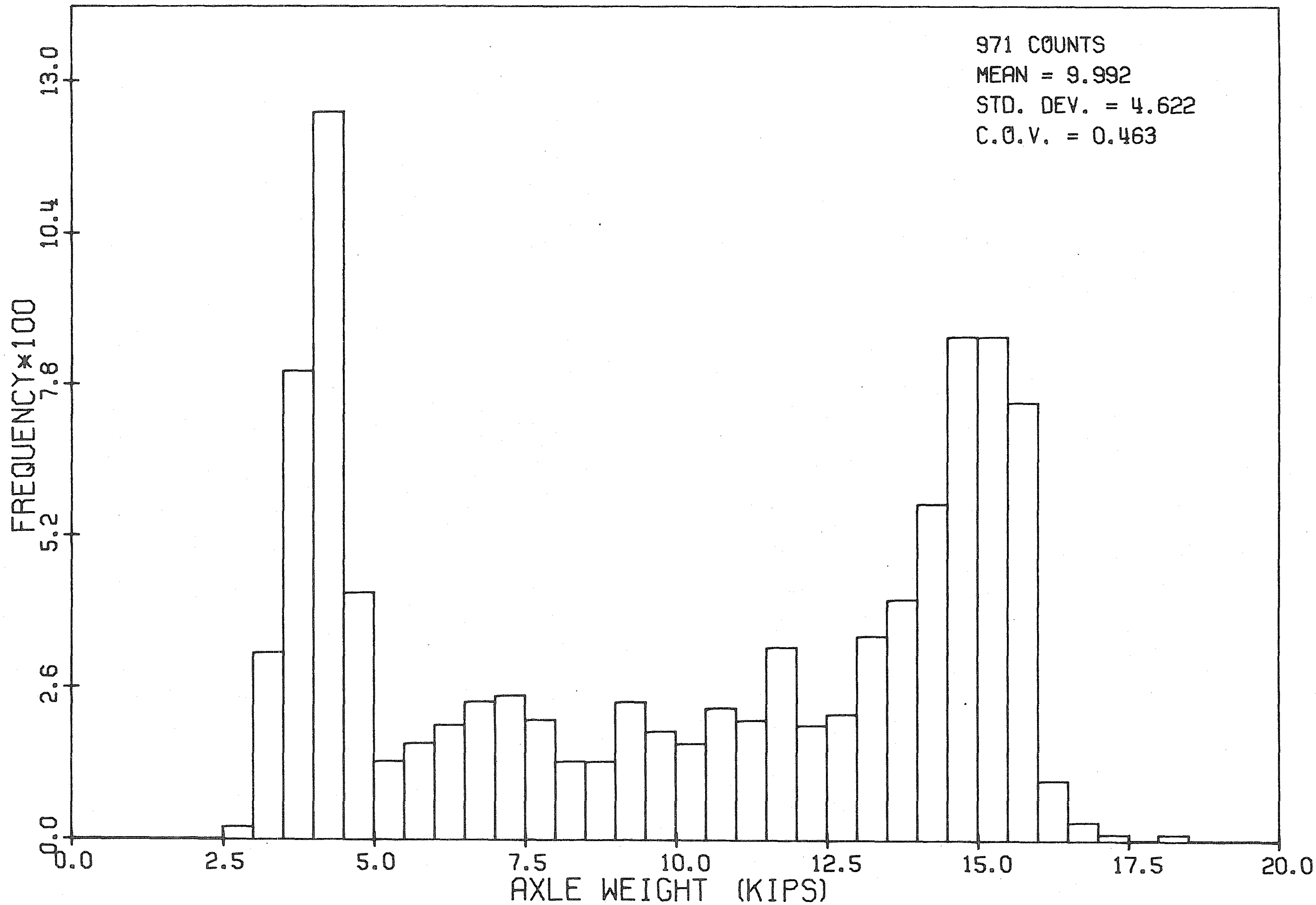

Fig. A.28, Histogram for Axle Weight D, 3S-2 Trucks, Camp Creek Phase II, 9/23/76

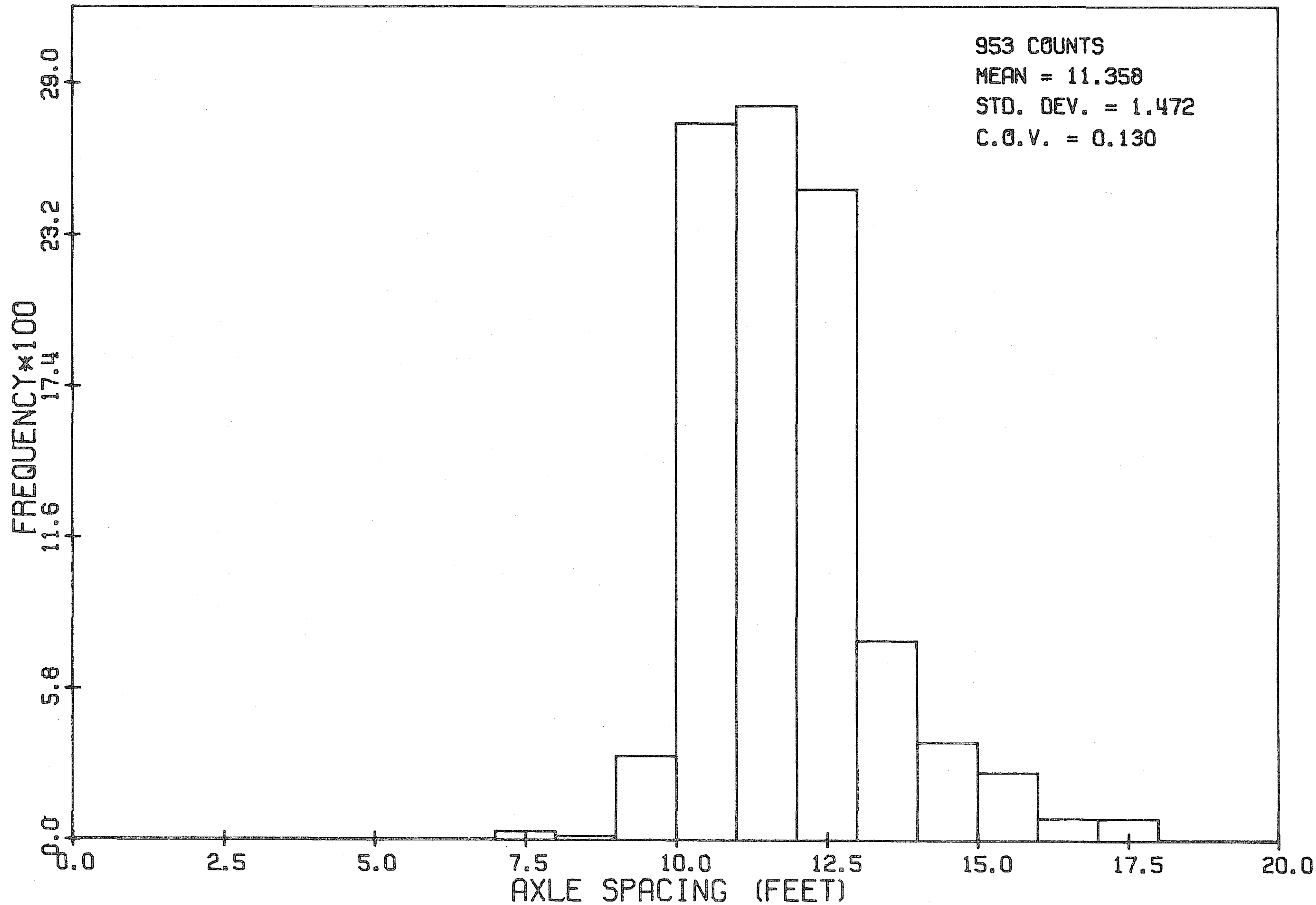

Fig. A.29 Histogram for Axle Spacing A-B, 3S-2 Trucks, Camp Creek Phase II, 9/23/76

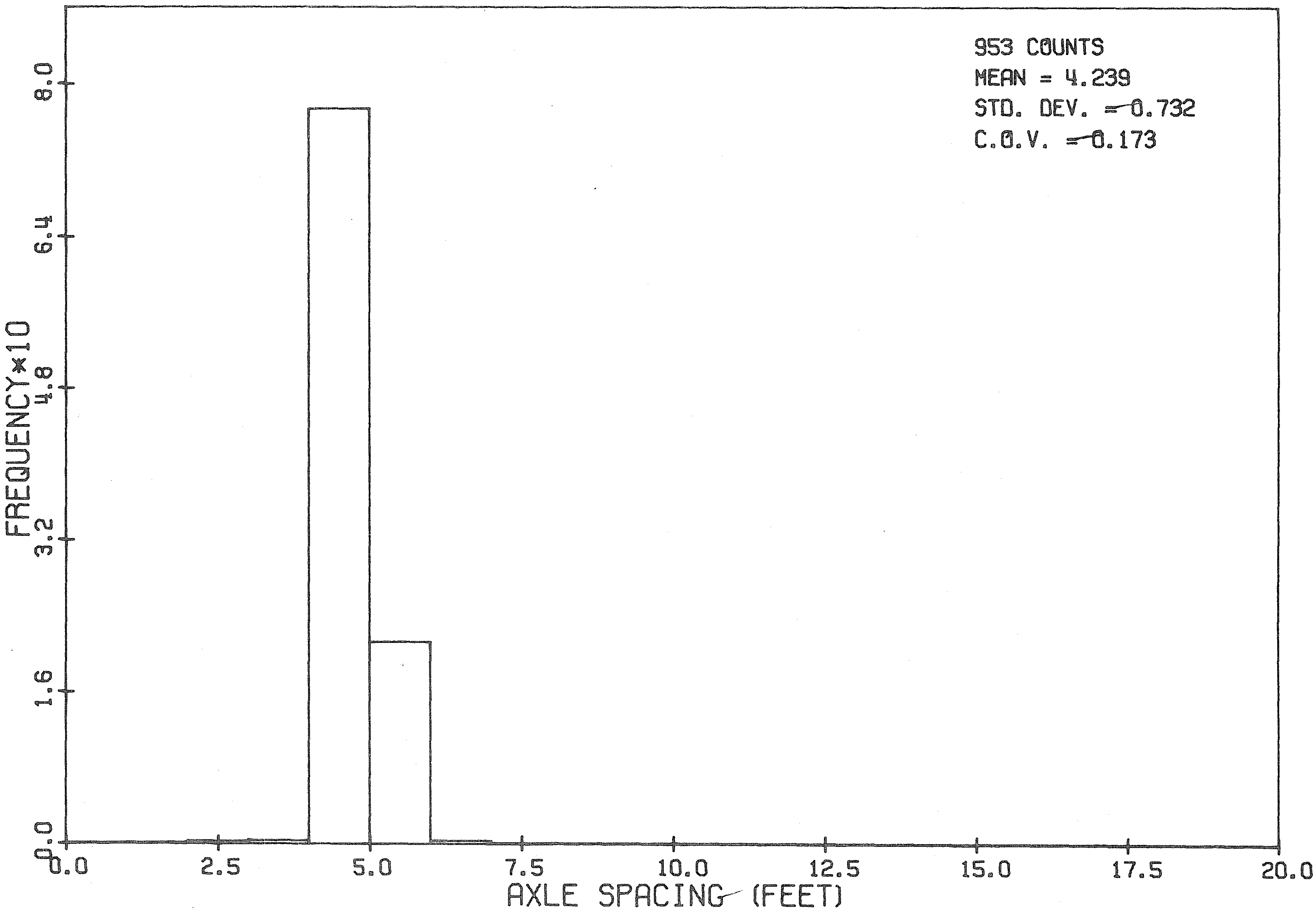

Fig. A.30 Histogram for Axle Spacing B-C, 3S-2 Trucks, Camp Creek Phase II, 9/23/76

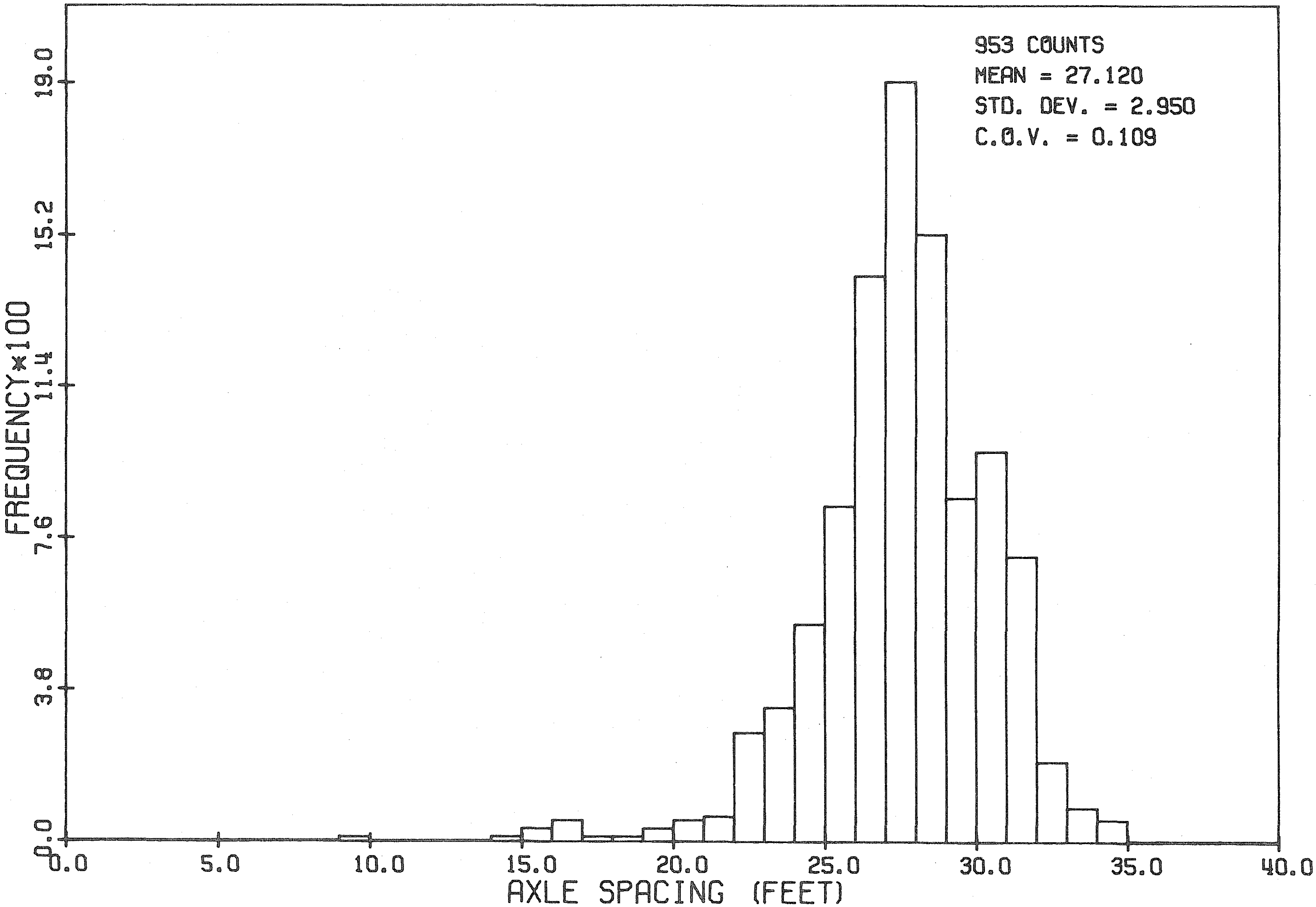

Fig. A.31 Histogram for Axle Spacing C-D, 3S-2 Trucks, Camp Creek Phase II, 9/23/76

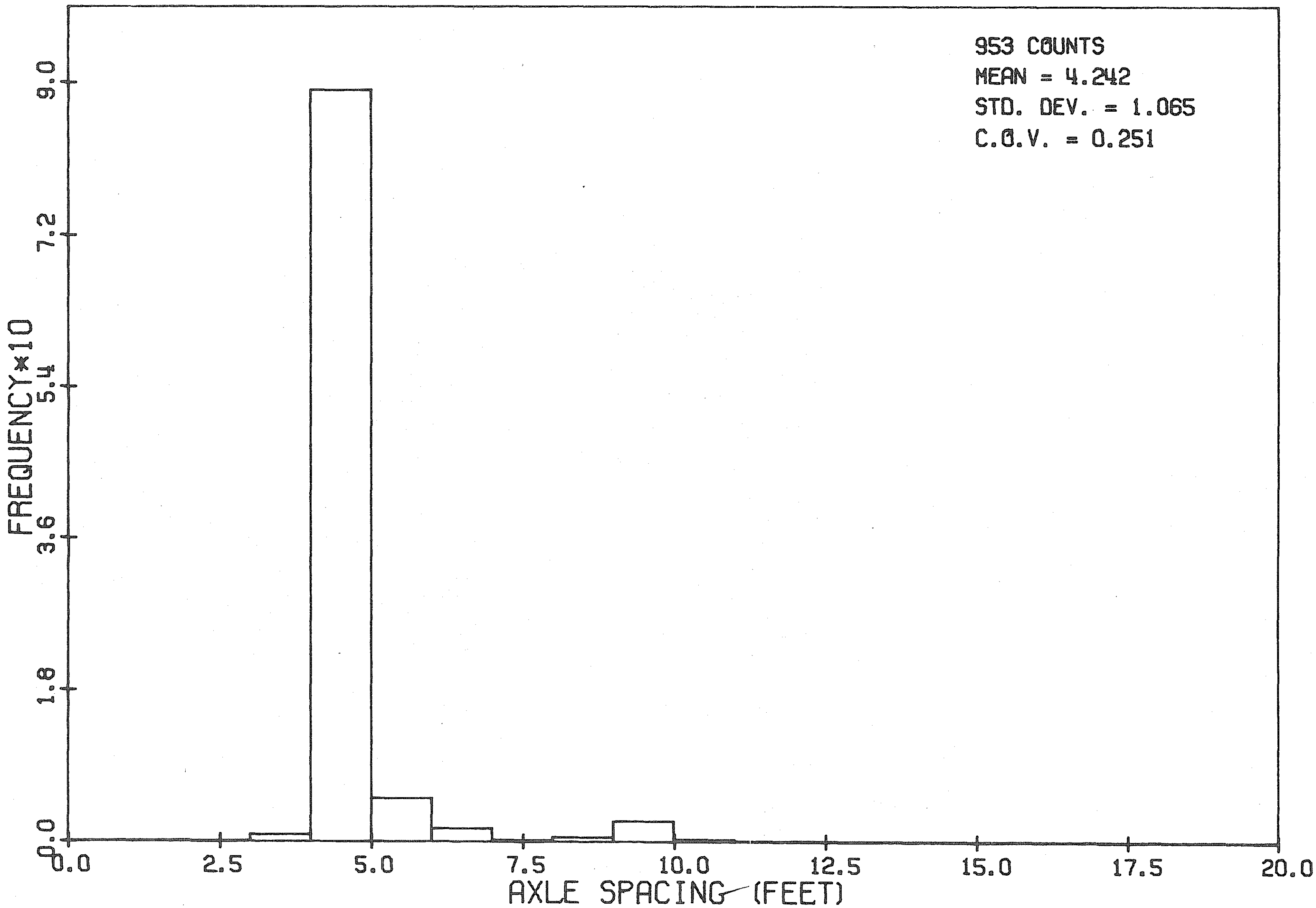

Fig. A.32 Histogram for Axle Spacing D-E, 3S-2 Trucks, Camp Creek Phase II, 9/23/76

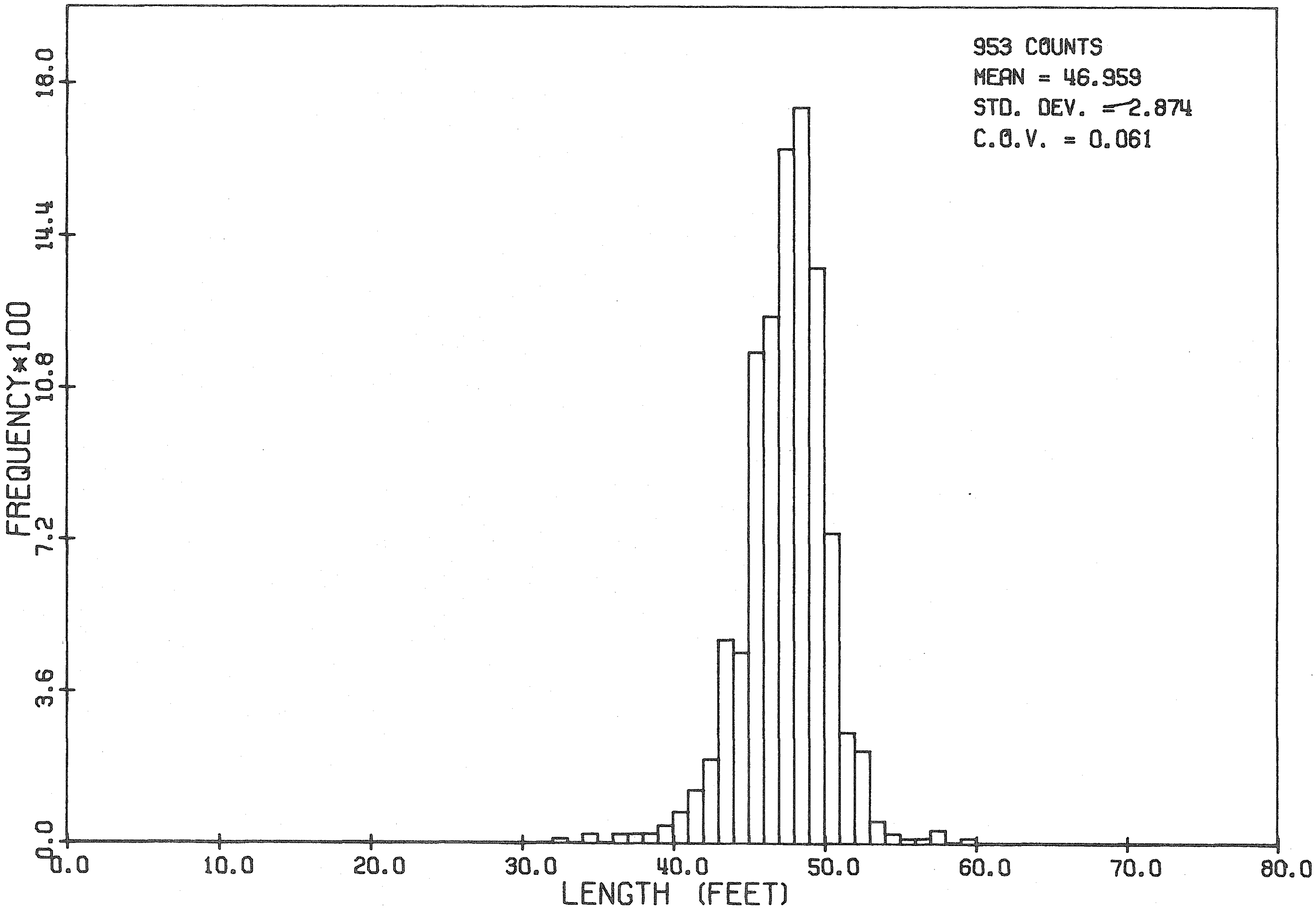

Fig. A.33 Histogram for Total Length, 3S-2 Trucks, Camp Creek Phase II, 9/23/76

APPENDIX B

TABLE B.1 DATA FOR STRESS RANGE HISTOGRAMS
GREEN RIVER BRIDGE

STRAIN GAGE= 121 DAY= 2 TOTAL EVENTS= 129

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
71.992	56.696	14.3984	11.3392

COEFFICIENT OF VARIATION= 0.788

NO COUNTS AFTER 25 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	3	0.023
20	4	19	0.147
30	6	19	0.147
40	8	6	0.047
50	10	12	0.093
60	12	10	0.078
70	14	8	0.062
80	16	9	0.07
90	18	6	0.047
100	20	2	0.016
110	22	8	0.062
120	24	6	0.047
130	26	2	0.016
140	28	1	0.008
150	30	3	0.023
160	32	3	0.023
170	34	1	0.008
180	36	1	0.008
190	38	3	0.023
200	40	1	0.008
210	42	1	0.008
220	44	2	0.016
230	46	1	0.008
240	48	1	0.008
250	50	1	0.008
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TABLE B.2 DATA FOR STRESS RANGE HISTOGRAMS
GREEN RIVER BRIDGE

STRAIN GAGE= 122 DAY= 2 TOTAL EVENTS= 130

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
92.508	65.781	18.5016	13.1562

COEFFICIENT OF VARIATION= 0.711

NO COUNTS AFTER 29 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	2	0.015
20	4	14	0.108
30	6	14	0.108
40	8	7	0.054
50	10	8	0.062
60	12	4	0.031
70	14	2	0.015
80	16	10	0.077
90	18	4	0.031
100	20	11	0.085
110	22	5	0.038
120	24	9	0.069
130	26	14	0.108
140	28	6	0.046
150	30	1	0.008
160	32	0	0
170	34	2	0.015
180	36	2	0.015
190	38	4	0.031
200	40	1	0.008
210	42	0	0
220	44	2	0.015
230	46	1	0.008
240	48	2	0.015
250	50	0	0
260	52	1	0.008
270	54	1	0.008
280	56	2	0.015
290	58	1	0.008
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TABLE B.3 DATA FOR STRESS RANGE HISTOGRAMS
GREEN RIVER BRIDGE

STRAIN GAGE= 123 DAY= 2 TOTAL EVENTS= 131

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
75.962	46.297	15.1924	9.2594

COEFFICIENT OF VARIATION= 0.609

NO COUNTS AFTER 21 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	4	0.031
20	4	14	0.107
30	6	9	0.069
40	8	11	0.084
50	10	3	0.023
60	12	14	0.107
70	14	13	0.099
80	16	8	0.061
90	18	11	0.084
100	20	4	0.031
110	22	5	0.038
120	24	7	0.053
130	26	9	0.069
140	28	6	0.046
150	30	4	0.031
160	32	3	0.023
170	34	2	0.015
180	36	3	0.023
190	38	0	0
200	40	0	0
210	42	1	0.008
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TABLE B.4 DATA FOR STRESS RANGE HISTOGRAMS
GREEN RIVER BRIDGE

STRAIN GAGE= 124 DAY= 2 TOTAL EVENTS= 128

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
106.68	71.923	21.336	14.3846

COEFFICIENT OF VARIATION= 0.674

NO COUNTS AFTER 31 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	7	0.055
30	6	12	0.094
40	8	5	0.039
50	10	4	0.031
60	12	12	0.094
70	14	10	0.078
80	16	11	0.086
90	18	5	0.039
100	20	12	0.094
110	22	2	0.016
120	24	5	0.039
130	26	1	0.008
140	28	4	0.031
150	30	3	0.023
160	32	3	0.023
170	34	6	0.047
180	36	3	0.023
190	38	3	0.023
200	40	2	0.016
210	42	2	0.016
220	44	2	0.016
230	46	4	0.031
240	48	3	0.023
250	50	2	0.016
260	52	1	0.008
270	54	1	0.008
280	56	1	0.008
290	58	0	0
300	60	1	0.008
310	62	1	0.008
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TABLE B.5 DATA FOR STRESS RANGE HISTOGRAMS
GREEN RIVER BRIDGE

STRAIN GAGE= 125 DAY= 2 TOTAL EVENTS= 123

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
97.748	79.58	19.5496	15.916

COEFFICIENT OF VARIATION= 0.814

NO COUNTS AFTER 36 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	1	0.008
20	4	10	0.081
30	6	8	0.065
40	8	13	0.106
50	10	12	0.098
60	12	9	0.073
70	14	6	0.049
80	16	10	0.081
90	18	8	0.065
100	20	7	0.057
110	22	5	0.041
120	24	3	0.024
130	26	0	0
140	28	2	0.016
150	30	0	0
160	32	3	0.024
170	34	2	0.016
180	36	2	0.016
190	38	3	0.024
200	40	1	0.008
210	42	3	0.024
220	44	1	0.008
230	46	0	0
240	48	3	0.024
250	50	4	0.033
260	52	1	0.008
270	54	0	0
280	56	1	0.008
290	58	1	0.008
300	60	1	0.008
310	62	1	0.008
320	64	0	0
330	66	0	0
340	68	1	0.008
350	70	0	0
360	72	1	0.008
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TABLE B.6 DATA FOR STRESS RANGE HISTOGRAMS
GREEN RIVER BRIDGE

STRAIN GAGE= 126 DAY= 2 TOTAL EVENTS= 127

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
95.228	74.811	19.0456	14.9622

COEFFICIENT OF VARIATION= 0.786

NO COUNTS AFTER 33 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	1	0.008
20	4	11	0.087
30	6	14	0.11
40	8	10	0.079
50	10	10	0.079
60	12	4	0.031
70	14	10	0.079
80	16	8	0.063
90	18	5	0.039
100	20	11	0.087
110	22	2	0.016
120	24	6	0.047
130	26	3	0.024
140	28	2	0.016
150	30	5	0.039
160	32	2	0.016
170	34	4	0.031
180	36	3	0.024
190	38	1	0.008
200	40	1	0.008
210	42	2	0.016
220	44	0	0
230	46	3	0.024
240	48	1	0.008
250	50	0	0
260	52	2	0.016
270	54	0	0
280	56	0	0
290	58	3	0.024
300	60	0	0
310	62	2	0.016
320	64	0	0
330	66	1	0.008
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TABLE B.7 DATA FOR STRESS RANGE HISTOGRAMS
GREEN RIVER BRIDGE

STRAIN GAGE= 127 DAY= 2 TOTAL EVENTS= 130

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
41.523	27.881	8.3046	5.5762

COEFFICIENT OF VARIATION= 0.671

NO COUNTS AFTER 12 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	15	0.115
20	4	24	0.185
30	6	15	0.115
40	8	14	0.108
50	10	20	0.154
60	12	17	0.131
70	14	8	0.062
80	16	1	0.008
90	18	4	0.031
100	20	5	0.038
110	22	4	0.031
120	24	3	0.023
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TABLE B.8 DATA FOR STRESS RANGE HISTOGRAMS
GREEN RIVER BRIDGE

STRAIN GAGE= 114 DAY= 2 TOTAL EVENTS= 127

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
107.378	70.396	21.4756	14.0792

COEFFICIENT OF VARIATION= 0.656

NO COUNTS AFTER 31 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	8	0.063
30	6	8	0.063
40	8	6	0.047
50	10	3	0.024
60	12	9	0.071
70	14	13	0.102
80	16	10	0.079
90	18	11	0.087
100	20	8	0.063
110	22	3	0.024
120	24	4	0.031
130	26	7	0.055
140	28	1	0.008
150	30	2	0.016
160	32	4	0.031
170	34	5	0.039
180	36	1	0.008
190	38	3	0.024
200	40	1	0.008
210	42	4	0.031
220	44	4	0.031
230	46	1	0.008
240	48	4	0.031
250	50	3	0.024
260	52	1	0.008
270	54	1	0.008
280	56	0	0
290	58	1	0.008
300	60	0	0
310	62	1	0.008
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TABLE B.9 DATA FOR STRESS RANGE HISTOGRAMS
GREEN RIVER BRIDGE

STRAIN GAGE= 115 DAY= 2 TOTAL EVENTS= 129

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
79.806	58.683	15.9612	11.7366

COEFFICIENT OF VARIATION= 0.735

NO COUNTS AFTER 26 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	11	0.085
30	6	10	0.078
40	8	21	0.163
50	10	12	0.093
60	12	11	0.085
70	14	9	0.07
80	16	11	0.085
90	18	7	0.054
100	20	2	0.016
110	22	2	0.016
120	24	2	0.016
130	26	5	0.039
140	28	5	0.039
150	30	2	0.016
160	32	2	0.016
170	34	1	0.008
180	36	1	0.008
190	38	5	0.039
200	40	4	0.031
210	42	2	0.016
220	44	0	0
230	46	2	0.016
240	48	0	0
250	50	1	0.008
260	52	1	0.008
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TABLE B.10 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 1

STRAIN GAGE= 121 DAY= 2 TOTAL EVENTS= 87

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
18.598	9.385	3.7196	1.877

COEFFICIENT OF VARIATION= 0.505

NO COUNTS AFTER 8 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	6	0.069
20	4	55	0.632
30	6	20	0.23
40	8	4	0.046
50	10	1	0.011
60	12	0	0
70	14	0	0
80	16	1	0.011
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TABLE B.11 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 1

STRAIN GAGE= 122 DAY= 2 TOTAL EVENTS= 87

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
21.069	11.023	4.2138	2.2046

COEFFICIENT OF VARIATION= 0.523

NO COUNTS AFTER 8 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	3	0.034
20	4	52	0.598
30	6	21	0.241
40	8	7	0.08
50	10	1	0.011
60	12	1	0.011
70	14	1	0.011
80	16	1	0.011
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TABLE B.12 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 1

STRAIN GAGE= 123 DAY= 2 TOTAL EVENTS= 85

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
42.188	28.23	8.4376	5.646

COEFFICIENT OF VARIATION= 0.669

NO COUNTS AFTER 19 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	1	0.012
20	4	6	0.071
30	6	25	0.294
40	8	23	0.271
50	10	13	0.153
60	12	5	0.059
70	14	6	0.071
80	16	1	0.012
90	18	1	0.012
100	20	0	0
110	22	1	0.012
120	24	0	0
130	26	0	0
140	28	1	0.012
150	30	1	0.012
160	32	0	0
170	34	0	0
180	36	0	0
190	38	1	0.012
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TABLE B.13 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 1

STRAIN GAGE= 124 DAY= 2 TOTAL EVENTS= 85

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
94.588	42.175	18.9176	8.435

COEFFICIENT OF VARIATION= 0.446

NO COUNTS AFTER 23 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	1	0.012
20	4	3	0.035
30	6	0	0
40	8	2	0.024
50	10	6	0.071
60	12	9	0.106
70	14	9	0.106
80	16	8	0.094
90	18	5	0.059
100	20	3	0.035
110	22	7	0.082
120	24	5	0.059
130	26	9	0.106
140	28	5	0.059
150	30	7	0.082
160	32	2	0.024
170	34	2	0.024
180	36	0	0
190	38	1	0.012
200	40	0	0
210	42	0	0
220	44	0	0
230	46	1	0.012
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TABLE B.14 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 1

STRAIN GAGE= 125 DAY= 2 TOTAL EVENTS= 83

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
99.819	43.76	19.9638	8.752

COEFFICIENT OF VARIATION= 0.438

NO COUNTS AFTER 19 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	1	0.012
20	4	2	0.024
30	6	1	0.012
40	8	0	0
50	10	7	0.084
60	12	6	0.072
70	14	11	0.133
80	16	7	0.084
90	18	5	0.06
100	20	3	0.036
110	22	3	0.036
120	24	2	0.024
130	26	7	0.084
140	28	6	0.072
150	30	11	0.133
160	32	6	0.072
170	34	2	0.024
180	36	2	0.024
190	38	1	0.012
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TABLE B.15 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 1

STRAIN GAGE= 126 DAY= 2 TOTAL EVENTS= 84

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
76.274	35.056	15.2548	7.0112

COEFFICIENT OF VARIATION= 0.46

NO COUNTS AFTER 16 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	4	0.048
30	6	2	0.024
40	8	7	0.083
50	10	12	0.143
60	12	10	0.119
70	14	5	0.06
80	16	6	0.071
90	18	10	0.119
100	20	6	0.071
110	22	4	0.048
120	24	6	0.071
130	26	6	0.071
140	28	3	0.036
150	30	2	0.024
160	32	1	0.012
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TABLE B.16 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 1

STRAIN GAGE= 127 DAY= 2 TOTAL EVENTS= 84

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
32.179	9.752	6.4358	1.9504

COEFFICIENT OF VARIATION= 0.303

NO COUNTS AFTER 6 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	9	0.107
30	6	34	0.405
40	8	19	0.226
50	10	19	0.226
60	12	3	0.036
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TABLE B.17 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 1

STRAIN GAGE= 114 DAY= 2 TOTAL EVENTS= 86

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
30.372	12.673	6.0744	2.5346

COEFFICIENT OF VARIATION= 0.417

NO COUNTS AFTER 8 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	1	0.012
20	4	16	0.186
30	6	31	0.36
40	8	24	0.279
50	10	7	0.081
60	12	4	0.047
70	14	2	0.023
80	16	1	0.012
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TABLE B.18 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 1

STRAIN GAGE= 115 DAY= 2 TOTAL EVENTS= 85

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
49.694	26.761	9.9388	5.3522

COEFFICIENT OF VARIATION= 0.539

NO COUNTS AFTER 11 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	8	0.094
30	6	23	0.271
40	8	12	0.141
50	10	4	0.047
60	12	6	0.071
70	14	9	0.106
80	16	8	0.094
90	18	9	0.106
100	20	3	0.035
110	22	3	0.035
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TABLE B.19 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 1

STRAIN GAGE= 224 DAY= 2 TOTAL EVENTS= 86

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
85.186	32.703	17.0372	6.5406

COEFFICIENT OF VARIATION= 0.384

NO COUNTS AFTER 17 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	1	0.012
20	4	2	0.023
30	6	1	0.012
40	8	1	0.012
50	10	6	0.07
60	12	14	0.163
70	14	8	0.093
80	16	4	0.047
90	18	9	0.105
100	20	14	0.163
110	22	4	0.047
120	24	11	0.128
130	26	5	0.058
140	28	2	0.023
150	30	1	0.012
160	32	2	0.023
170	34	1	0.012
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TABLE B.20 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 2

STRAIN GAGE= 121 DAY= 1 TOTAL EVENTS= 43

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
17.209	7.03	3.4418	1.406

COEFFICIENT OF VARIATION= 0.409

NO COUNTS AFTER 5 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	2	0.047
20	4	30	0.698
30	6	9	0.209
40	8	1	0.023
50	10	1	0.023
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TABLE B.21 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 2

STRAIN GAGE= 122 DAY= 1 TOTAL EVENTS= 43

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
22.279	10.135	4.4558	2.027

COEFFICIENT OF VARIATION= 0.455

NO COUNTS AFTER 6 -TH INTERVAL

STRAIN INTERVAL	STRESS (MPA) INTERVAL	COUNT	FREQUENCY
10	2	1	0.023
20	4	24	0.558
30	6	11	0.256
40	8	4	0.093
50	10	2	0.047
60	12	1	0.023
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TABLE B.22 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 2

STRAIN GAGE= 123 DAY= 1 TOTAL EVENTS= 43

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
50.256	21.614	10.0512	4.3228

COEFFICIENT OF VARIATION= 0.43

NO COUNTS AFTER 14 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	1	0.023
30	6	6	0.14
40	8	7	0.163
50	10	10	0.233
60	12	8	0.186
70	14	6	0.14
80	16	2	0.047
90	18	0	0
100	20	2	0.047
110	22	0	0
120	24	0	0
130	26	0	0
140	28	1	0.023
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TABLE B.23 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 2

STRAIN GAGE= 124 DAY= 1 TOTAL EVENTS= 43

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
87.023	27.712	17.4046	5.5424

COEFFICIENT OF VARIATION= 0.318

NO COUNTS AFTER 15 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	0	0
30	6	0	0
40	8	1	0.023
50	10	4	0.093
60	12	4	0.093
70	14	6	0.14
80	16	4	0.093
90	18	4	0.093
100	20	5	0.116
110	22	5	0.116
120	24	5	0.116
130	26	3	0.07
140	28	1	0.023
150	30	1	0.023
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TABLE B.24 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 2

STRAIN GAGE= 125 DAY= 1 TOTAL EVENTS= 43

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
77.767	28.124	15.5534	5.6248

COEFFICIENT OF VARIATION= 0.362

NO COUNTS AFTER 14 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	0	0
30	6	1	0.023
40	8	4	0.093
50	10	5	0.116
60	12	3	0.07
70	14	3	0.07
80	16	5	0.116
90	18	6	0.14
100	20	5	0.116
110	22	7	0.163
120	24	2	0.047
130	26	1	0.023
140	28	1	0.023
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TABLE B.25 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 2

STRAIN GAGE= 126 DAY= 1 TOTAL EVENTS= 43

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
70.186	27.062	14.0372	5.4124

COEFFICIENT OF VARIATION= 0.386

NO COUNTS AFTER 11 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	0	0
30	6	4	0.093
40	8	3	0.07
50	10	4	0.093
60	12	6	0.14
70	14	3	0.07
80	16	5	0.116
90	18	7	0.163
100	20	2	0.047
110	22	9	0.209
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TABLE B.26 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 2

STRAIN GAGE= 127 DAY= 1 TOTAL EVENTS= 42

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
36.548	11.742	7.3096	2.3484

COEFFICIENT OF VARIATION= 0.321

NO COUNTS AFTER 8 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	4	0.095
30	6	10	0.238
40	8	11	0.262
50	10	13	0.31
60	12	3	0.071
70	14	0	0
80	16	1	0.024
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TABLE B.27 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 2

STRAIN GAGE= 114 DAY= 1 TOTAL EVENTS= 41

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
32.488	11.074	6.4976	2.2148

COEFFICIENT OF VARIATION= 0.341

NO COUNTS AFTER 6 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	6	0.146
30	6	13	0.317
40	8	11	0.268
50	10	9	0.22
60	12	2	0.049
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TABLE B.28 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 2

STRAIN GAGE= 115 DAY= 1 TOTAL EVENTS= 43

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
51.721	18.609	10.3442	3.7218

COEFFICIENT OF VARIATION= 0.36

NO COUNTS AFTER 9 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	1	0.023
30	6	6	0.14
40	8	7	0.163
50	10	6	0.14
60	12	5	0.116
70	14	10	0.233
80	16	7	0.163
90	18	1	0.023
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TABLE B.29 DATA FOR STRESS RANGE HISTOGRAMS
CAMP CREEK BRIDGE(II)-Phase 2

STRAIN GAGE= 224 DAY= 1 TOTAL EVENTS= 42

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
89.31	31.754	17.862	6.3508

COEFFICIENT OF VARIATION= 0.356

NO COUNTS AFTER 15 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	0	0
30	6	1	0.024
40	8	0	0
50	10	5	0.119
60	12	6	0.143
70	14	5	0.119
80	16	1	0.024
90	18	0	0
100	20	3	0.071
110	22	6	0.143
120	24	7	0.167
130	26	5	0.119
140	28	2	0.048
150	30	1	0.024
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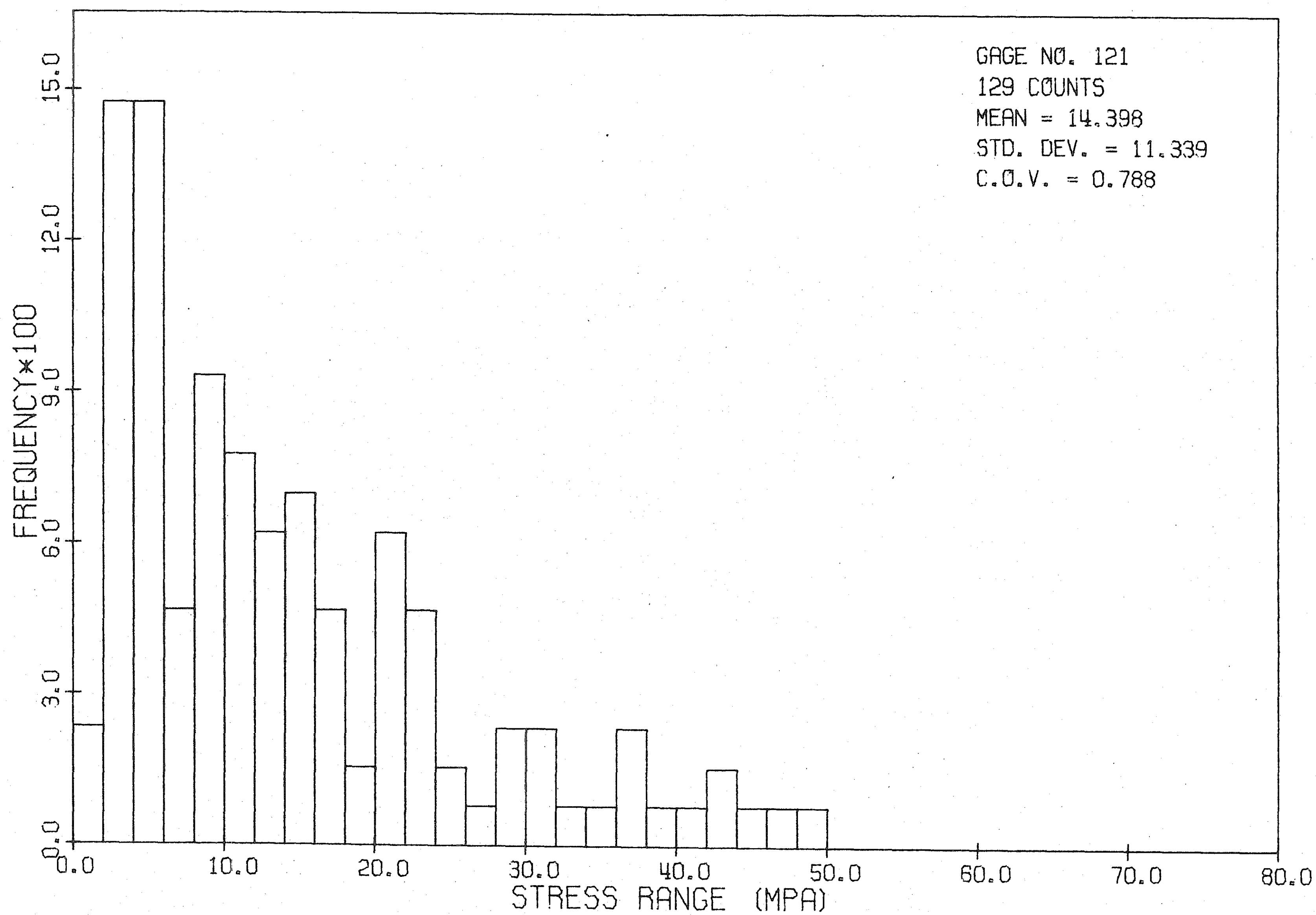

FIG. B.1. STRESS RANGE HISTØGRAM -- GREEN RIVER BRIDGE

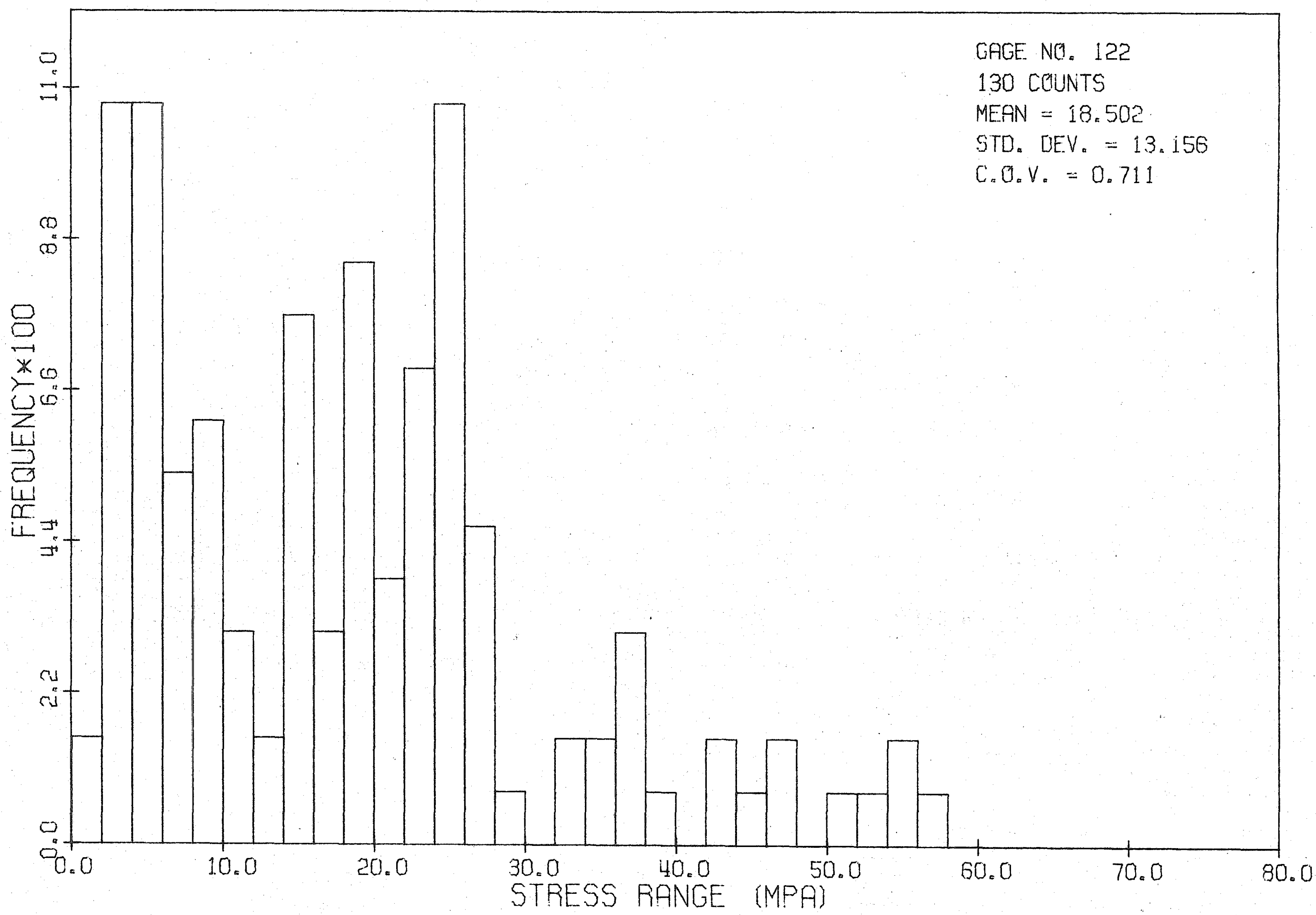

FIG. B.2 STRESS RANGE HISTOGRAM -- GREEN RIVER BRIDGE

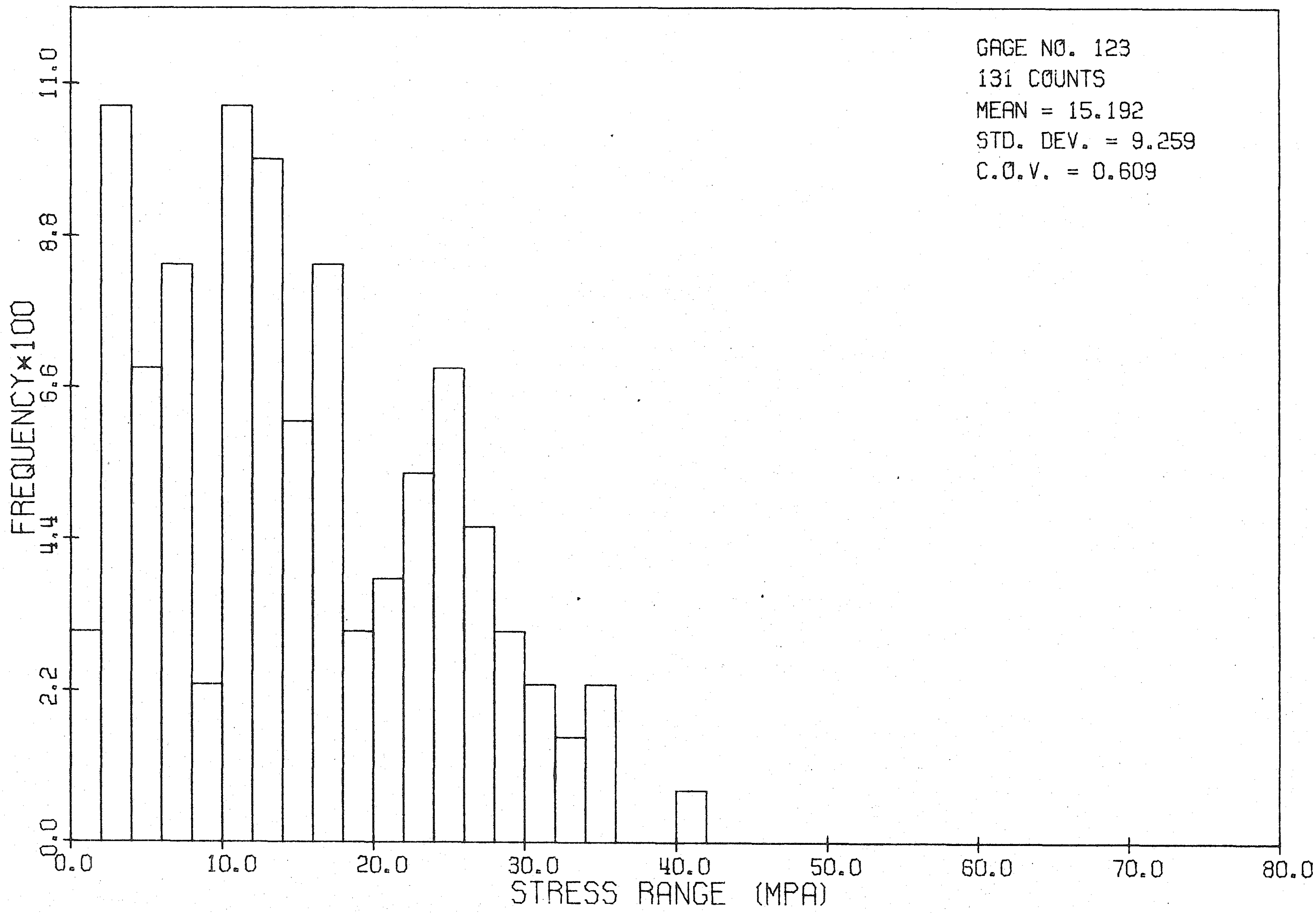

FIG. B.3. STRESS RANGE HISTOGRAM -- GREEN RIVER BRIDGE

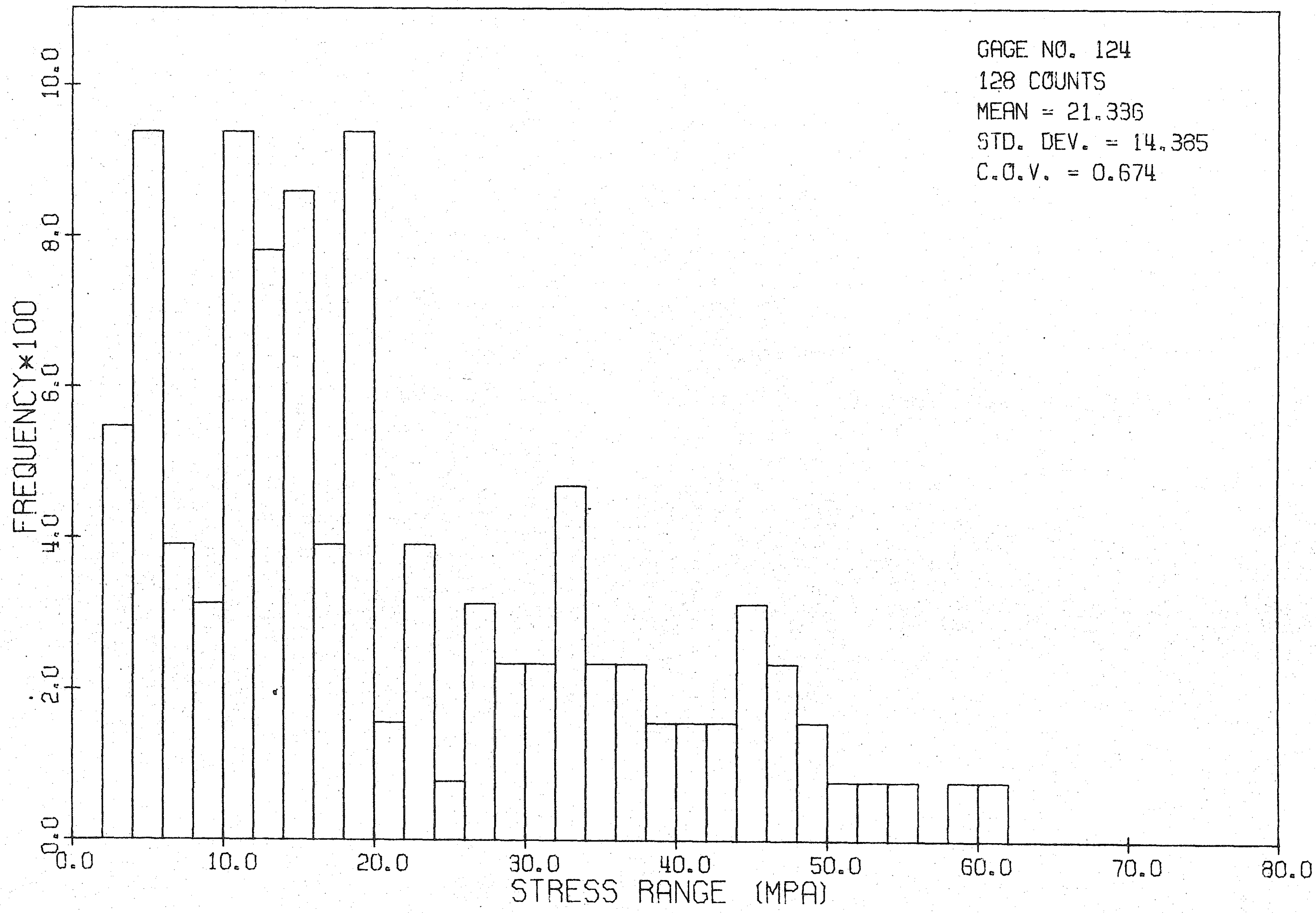

FIG. B.4. STRESS RANGE HISTOGRAM -- GREEN RIVER BRIDGE

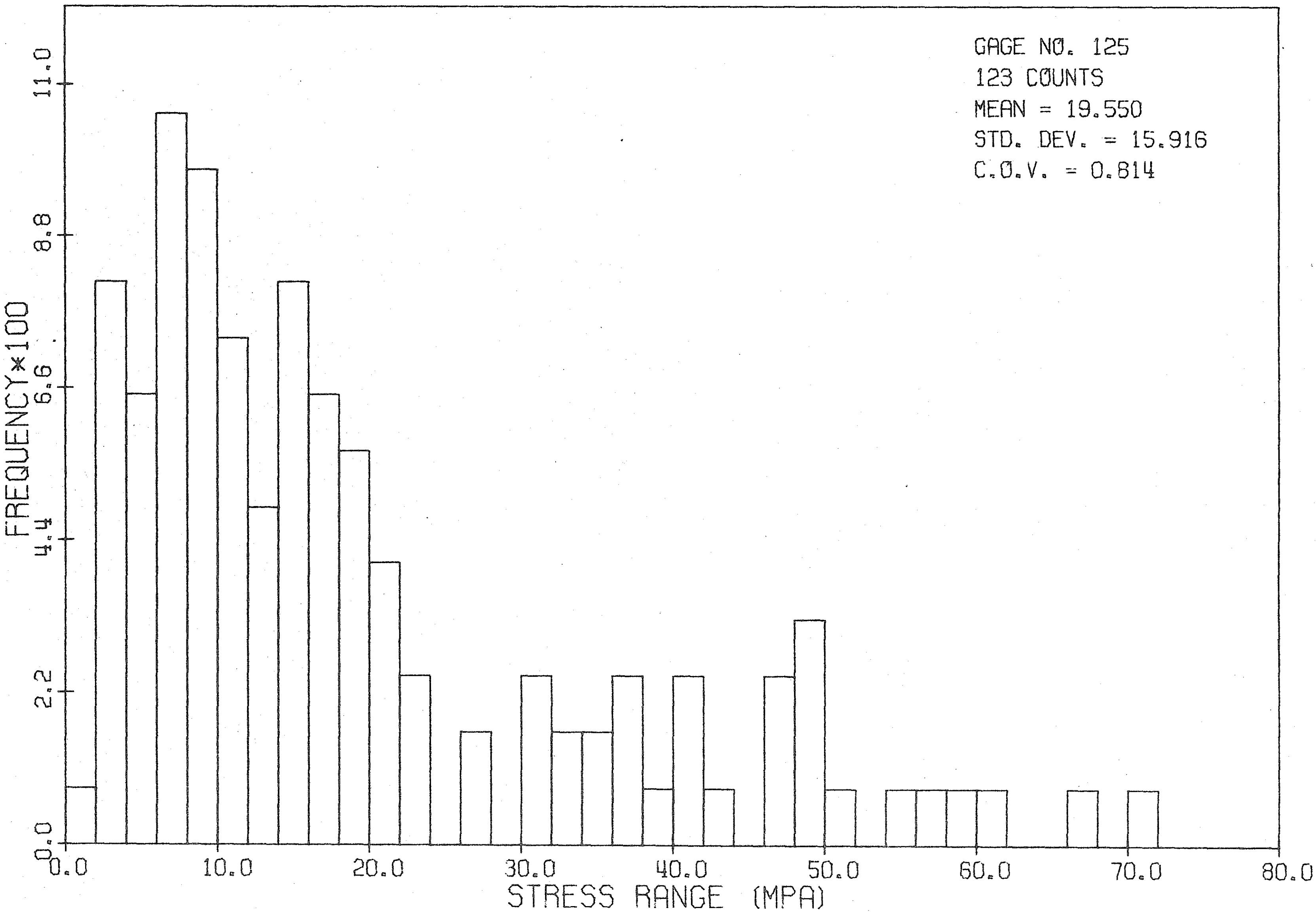

FIG.B.5. STRESS RANGE HISTØGRAM -- GREEN RIVER BRIDGE

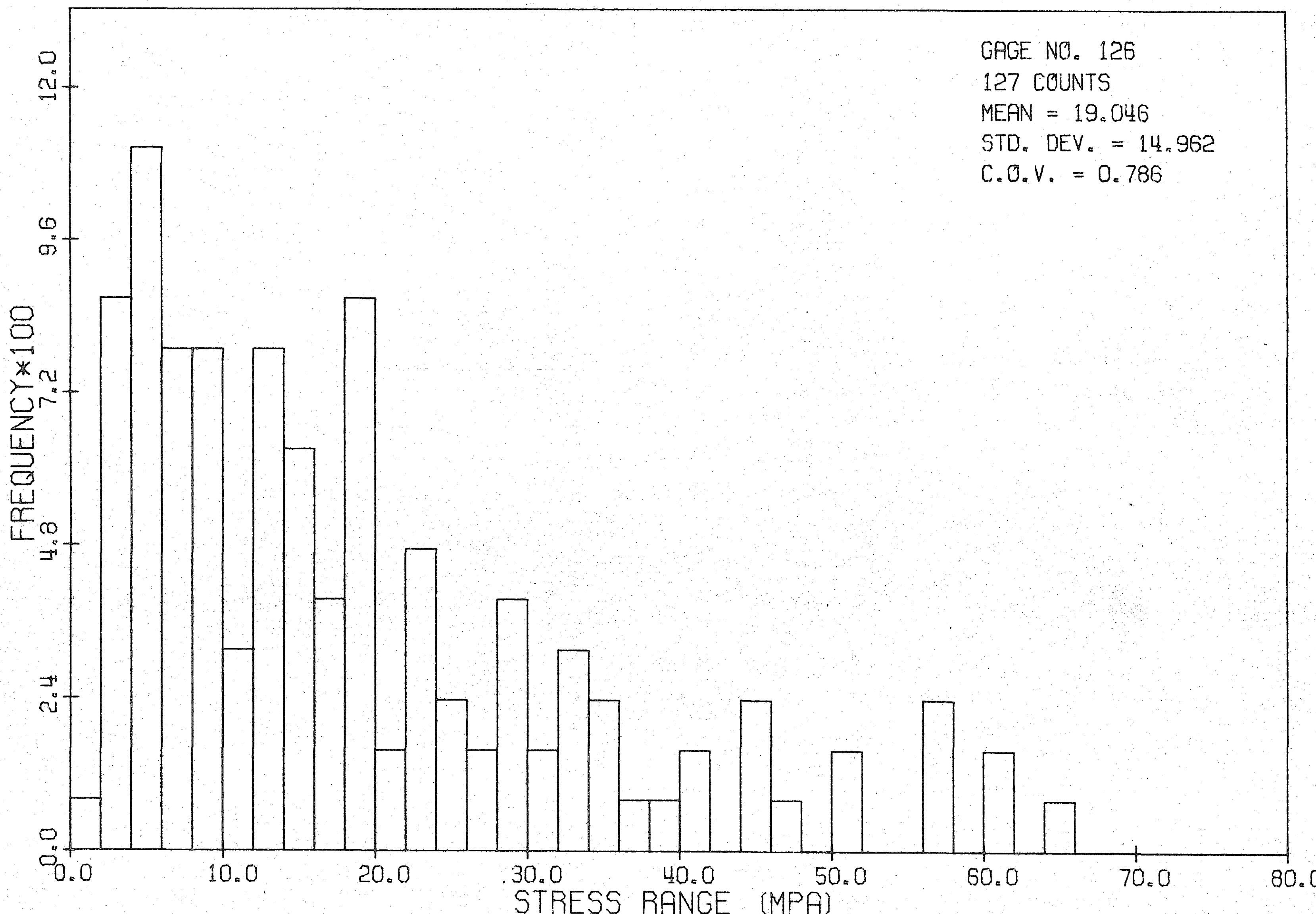

FIG. B.6. STRESS RANGE HISTOGRAM -- GREEN RIVER BRIDGE

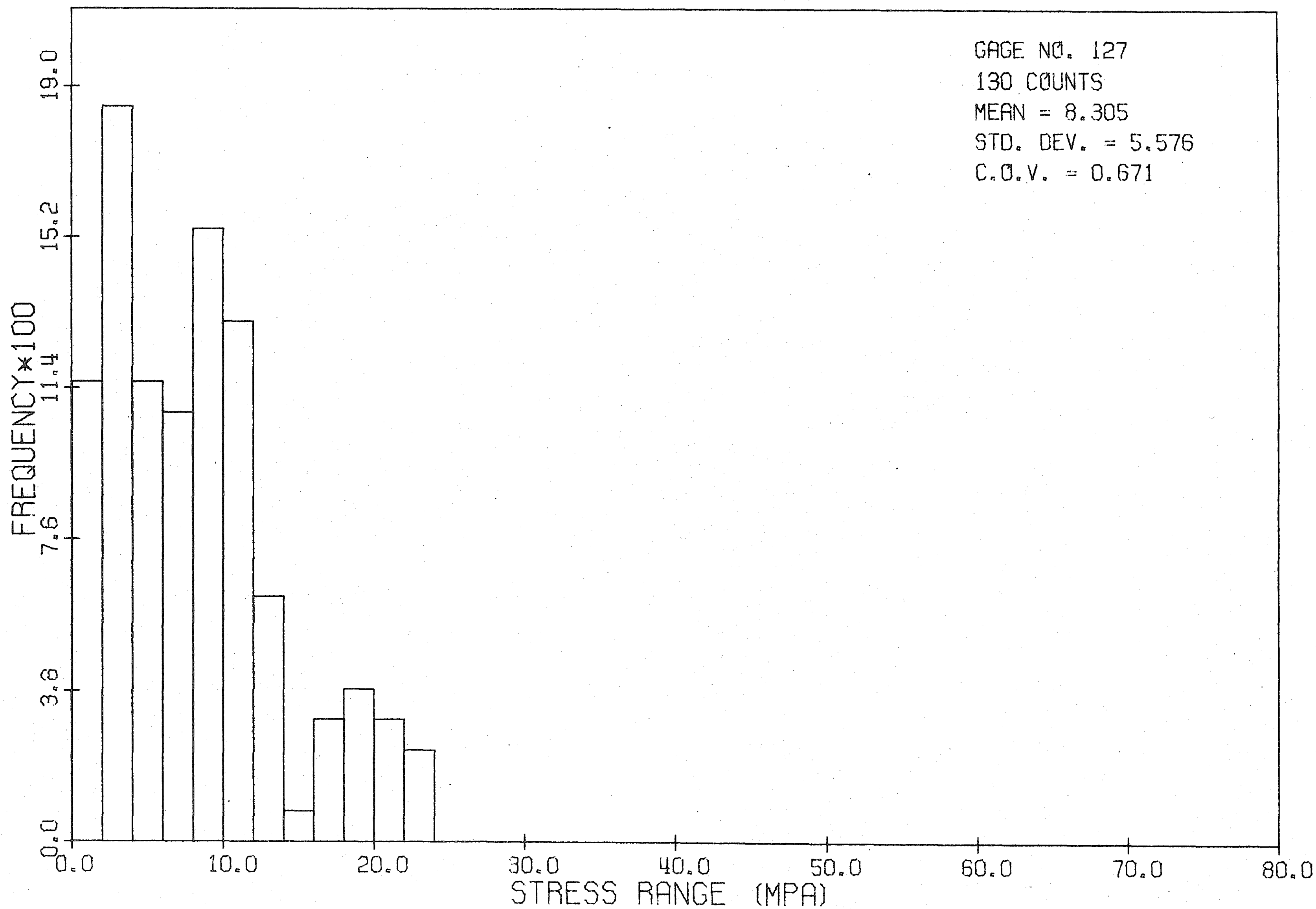

FIG. B.7. STRESS RANGE HISTOGRAM -- GREEN RIVER BRIDGE

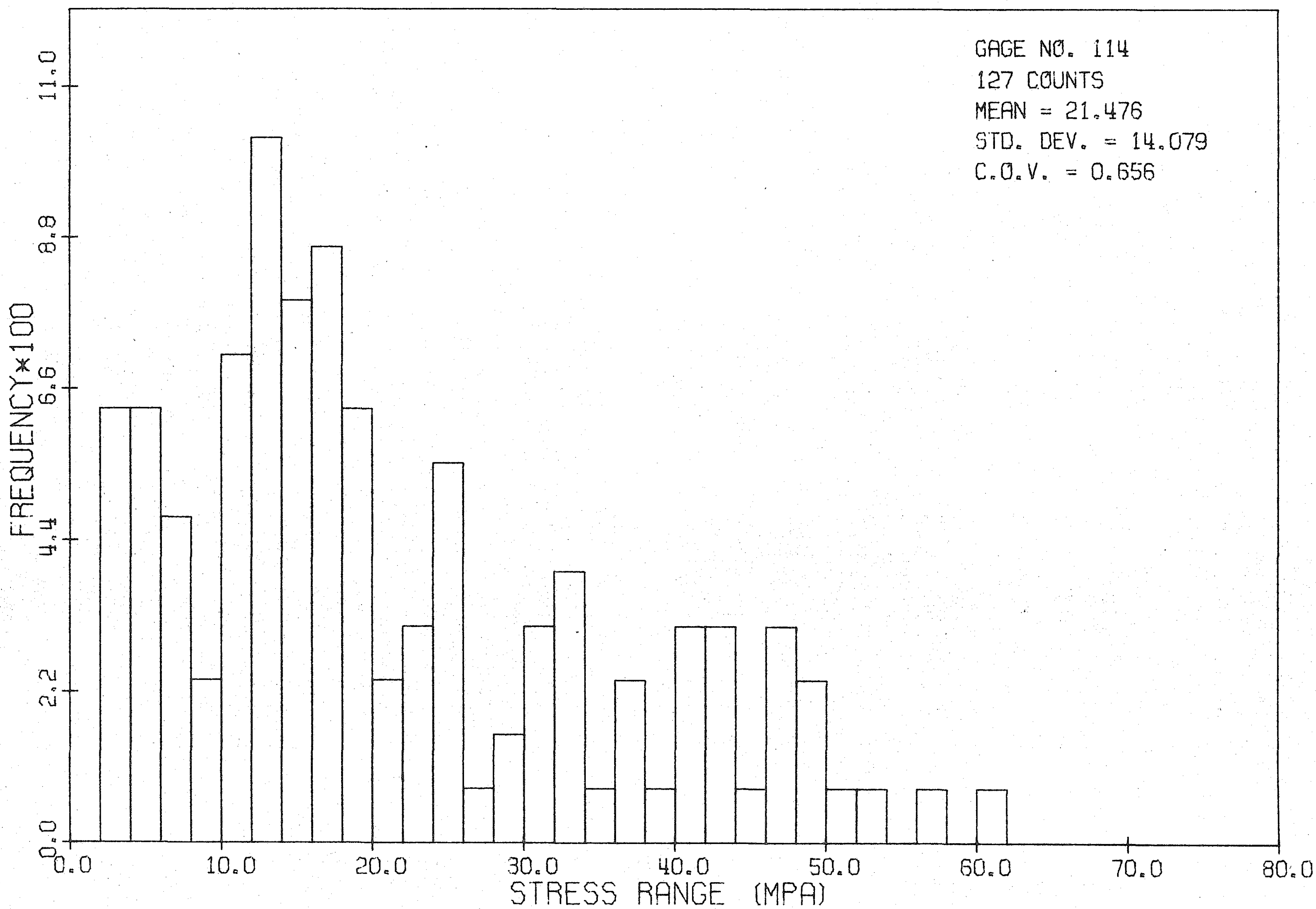

FIG. B.8. STRESS RANGE HISTOGRAM -- GREEN RIVER BRIDGE

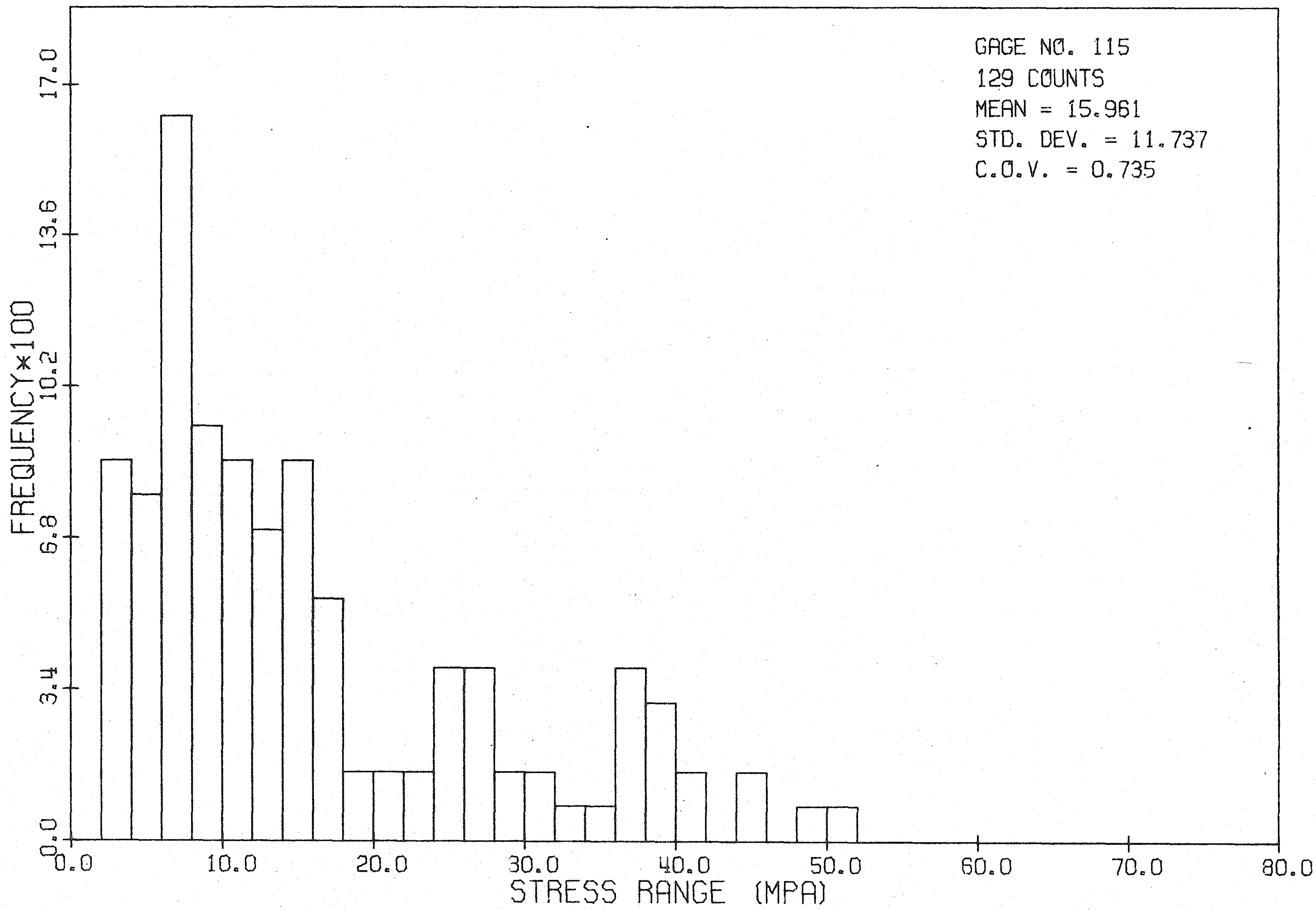

FIG. B.9 STRESS RANGE HISTOGRAM -- GREEN RIVER BRIDGE

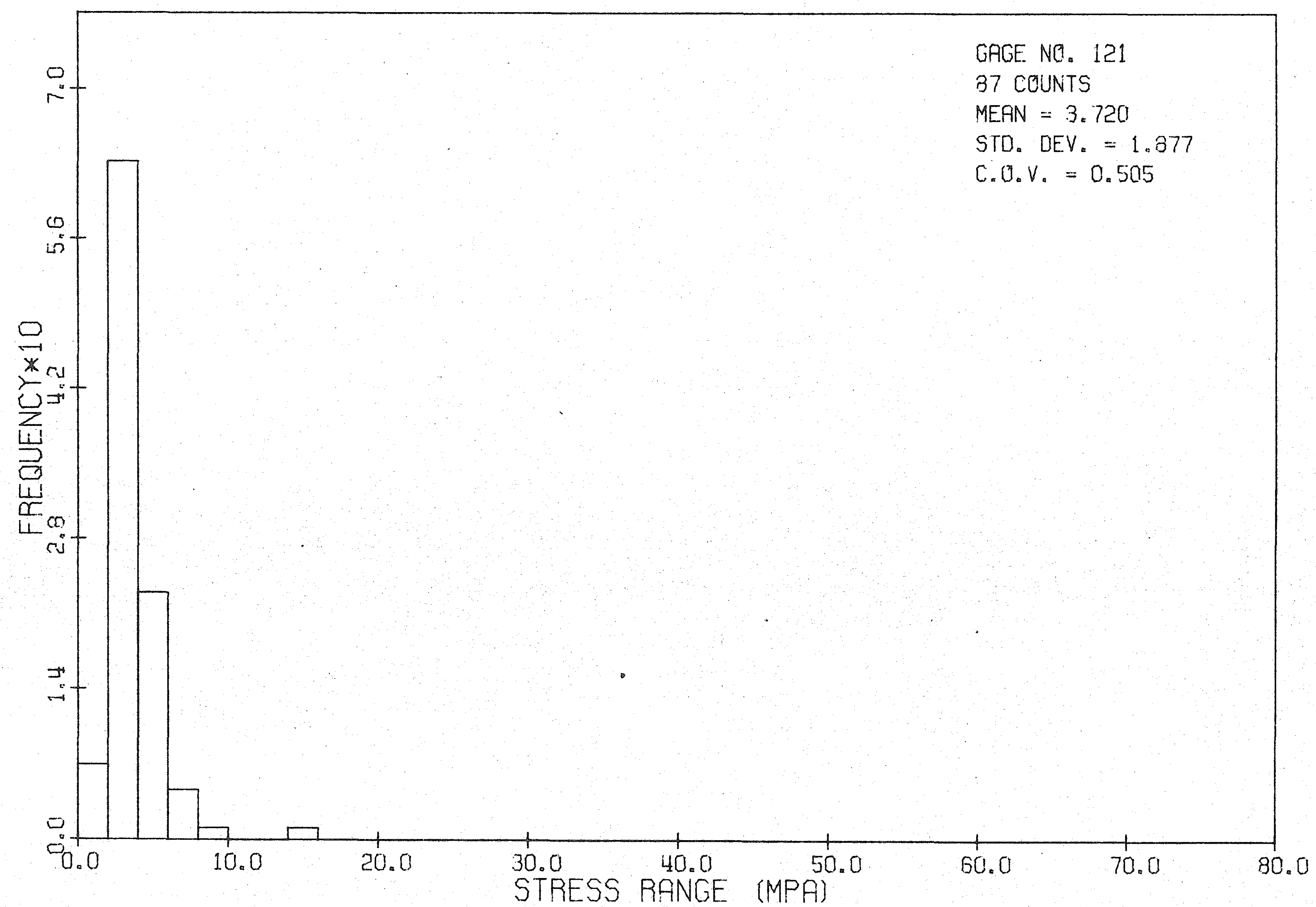

FIG. B.10. STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 1

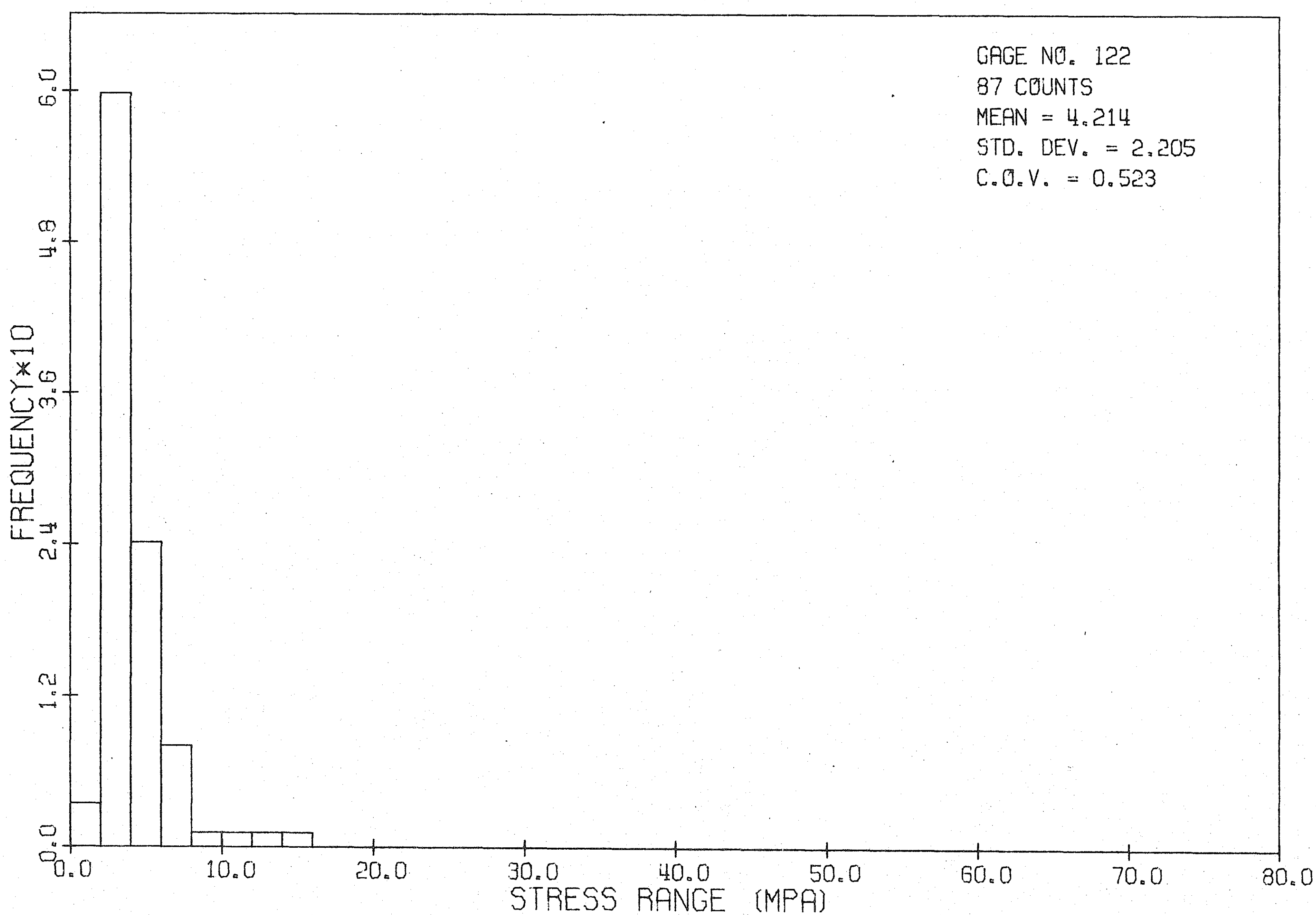

FIG. B.11 STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 1

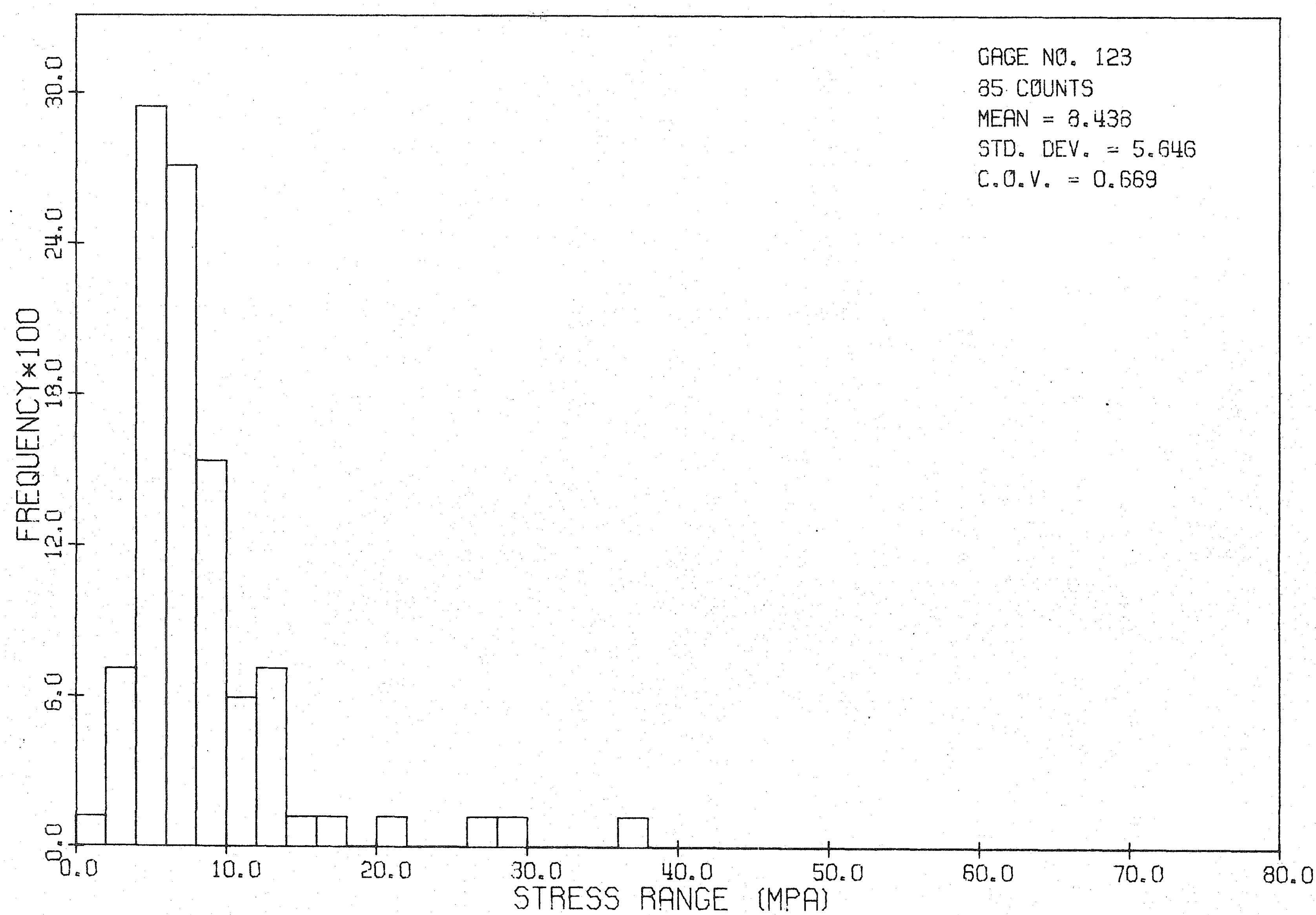

FIG. B.12 STRESS RANGE HISTØGRAM -- CAMP CREEK BRIDGE (II) -PHASE 1

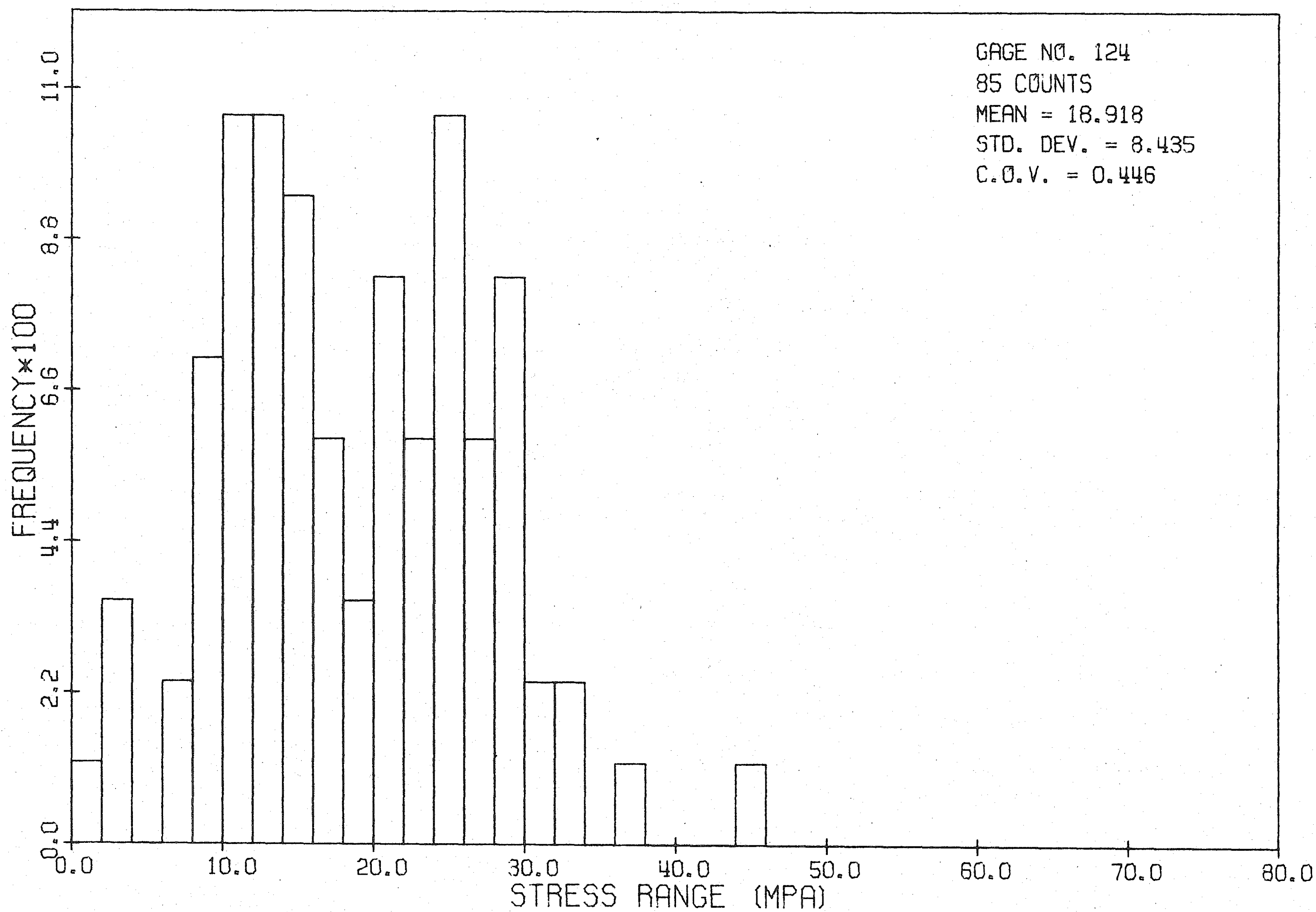

FIG. B.13 . STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 1

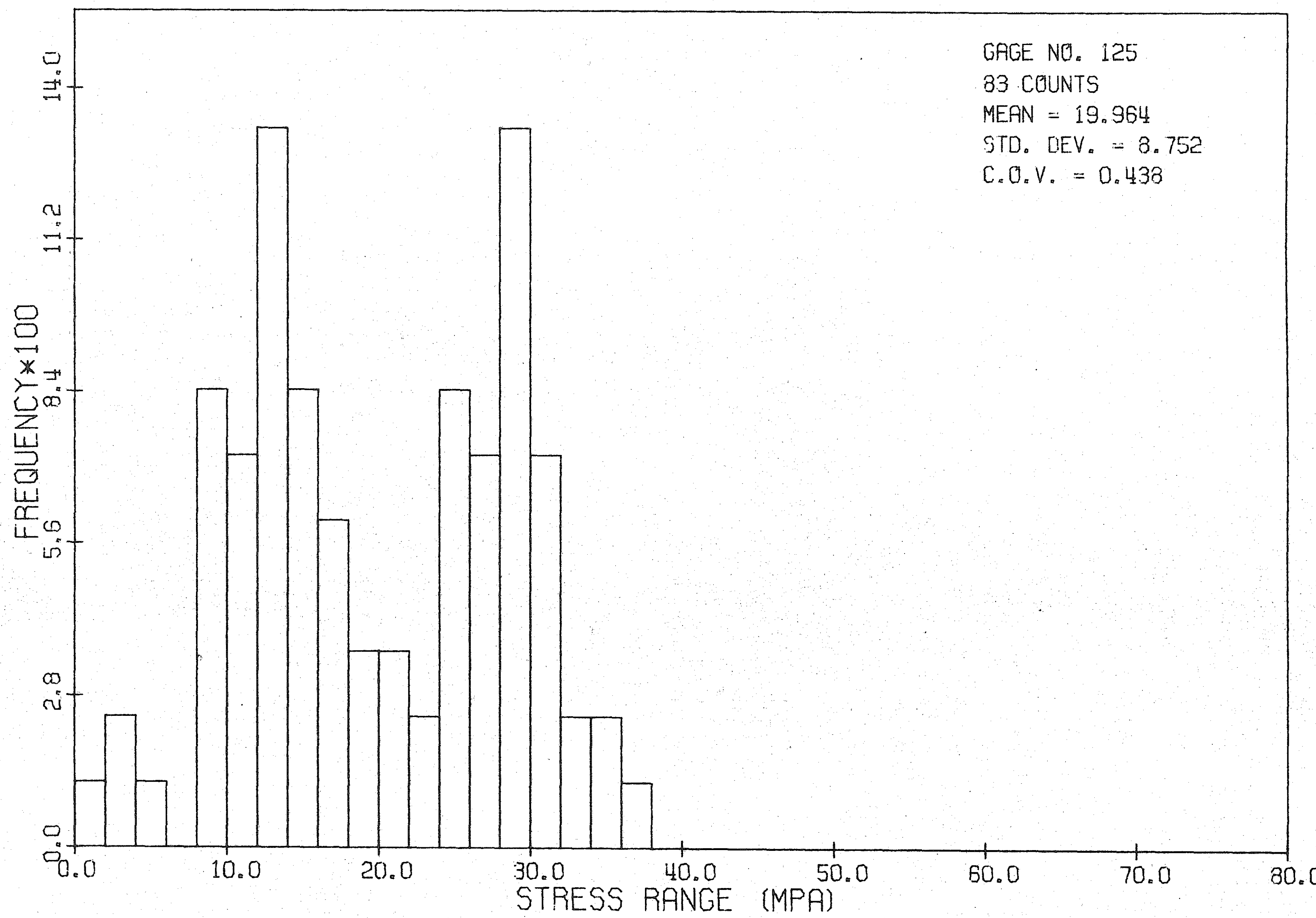

FIG. B.14 . STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 1

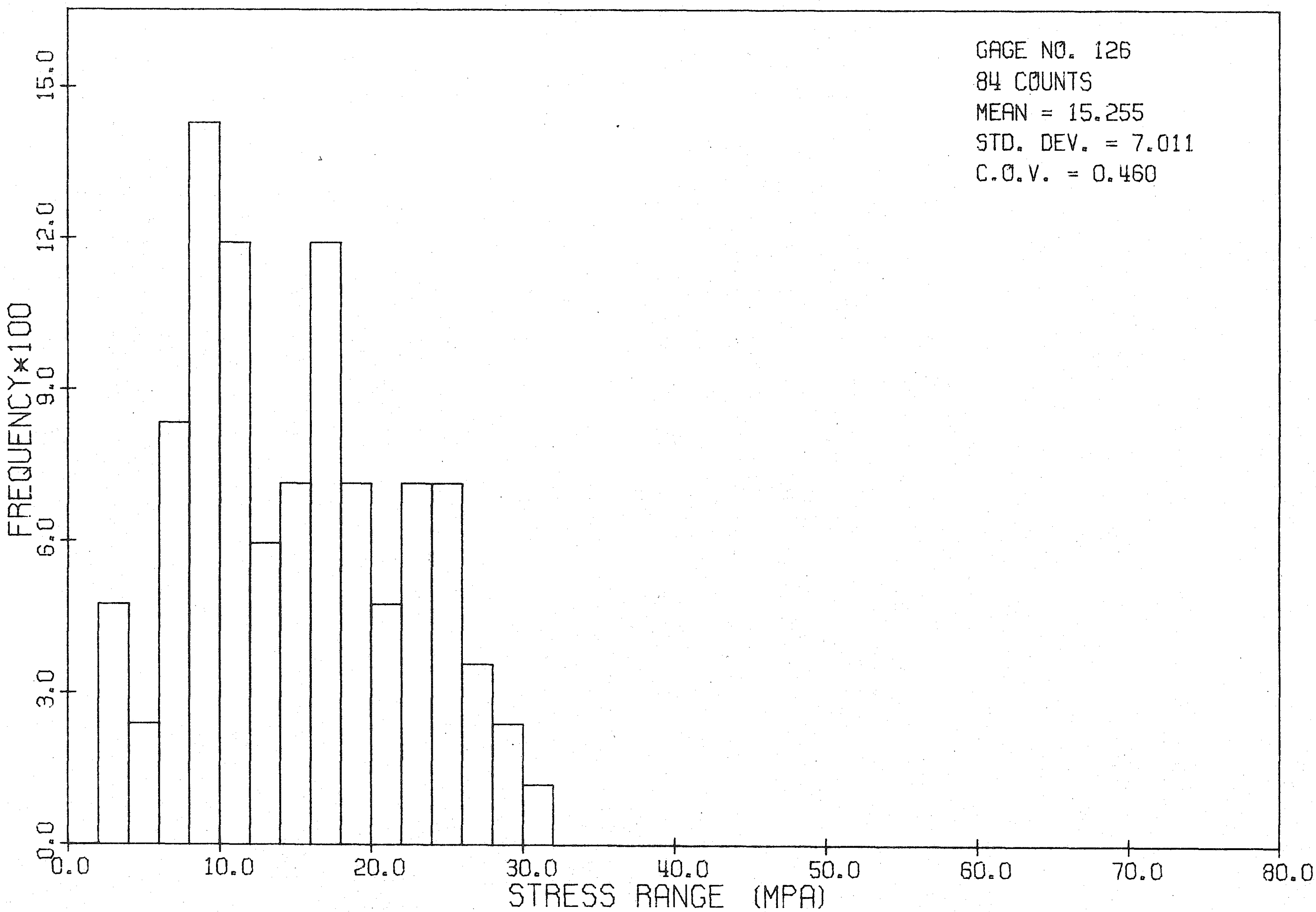

FIG. B.15 STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 1

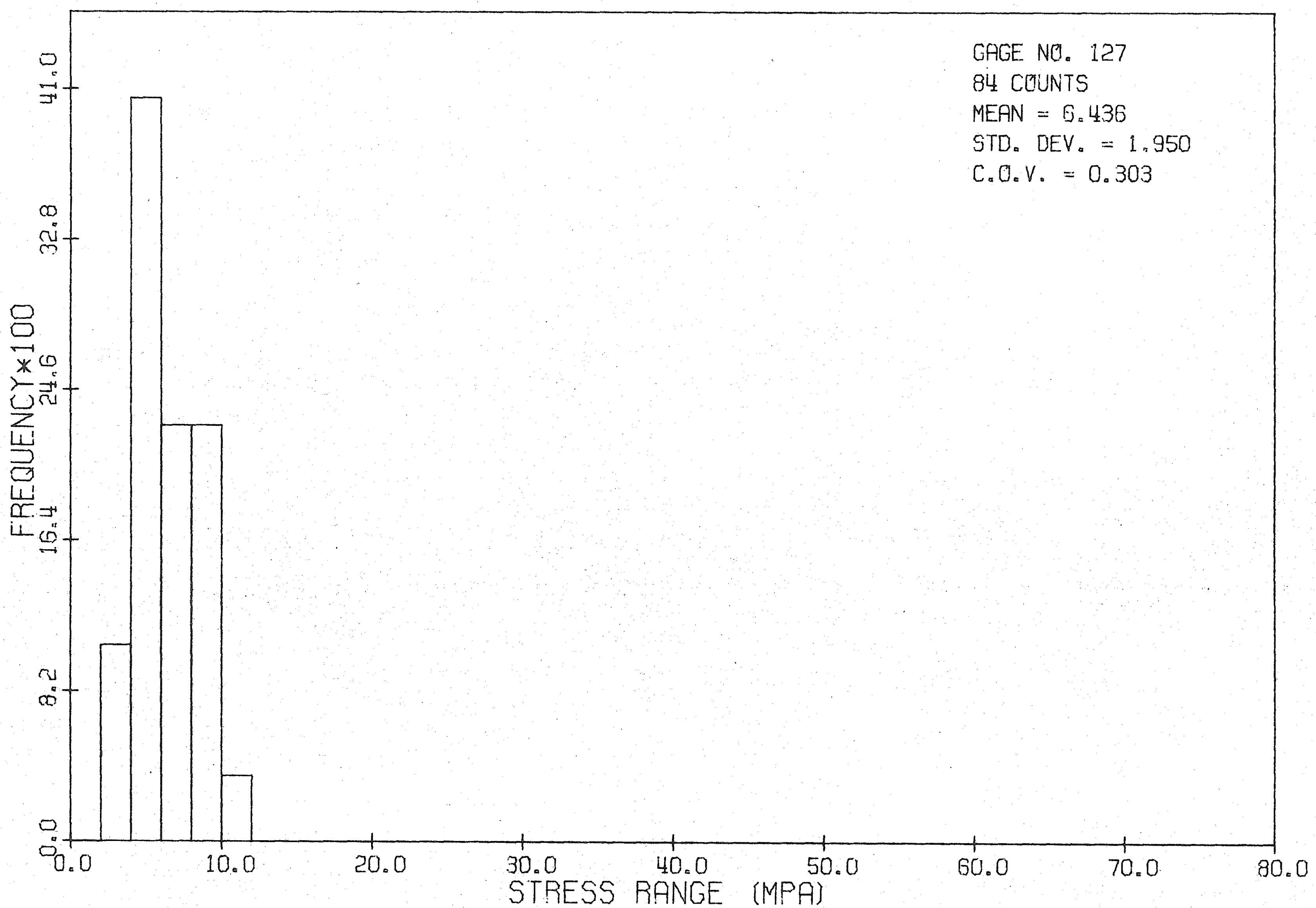

FIG. B.16 STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 1

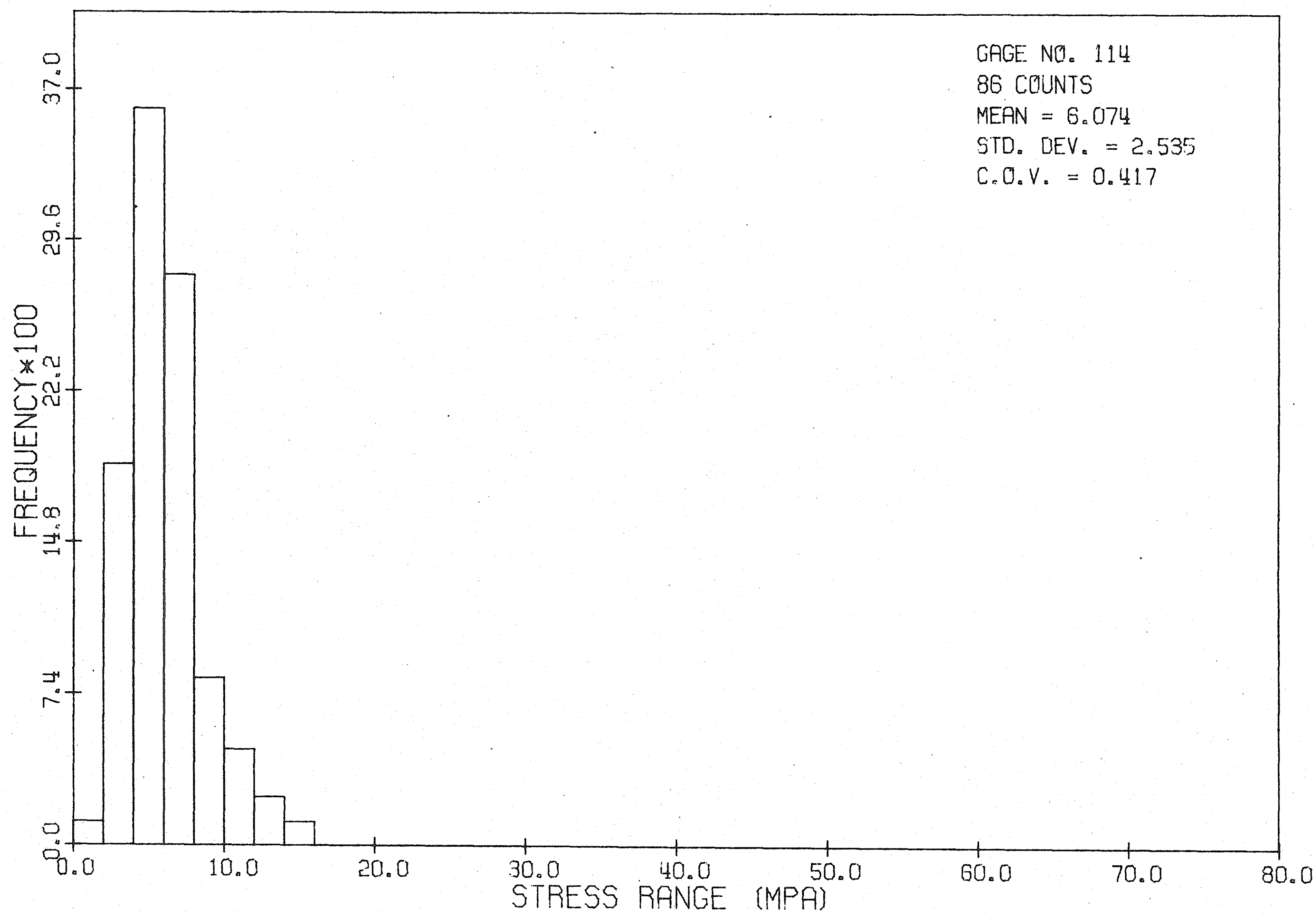

FIG. B.17 . STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 1

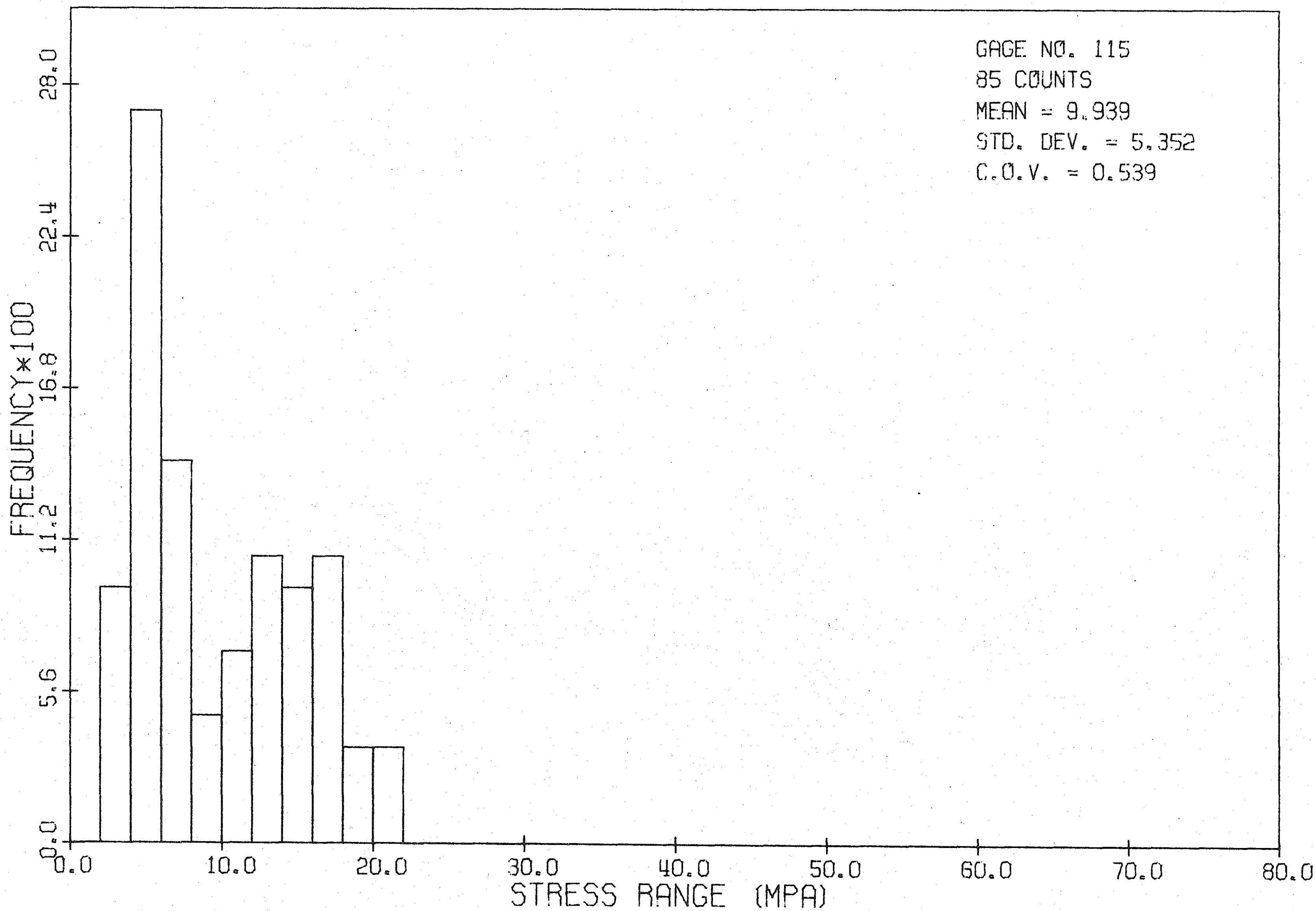

FIG. B.18 . STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 1

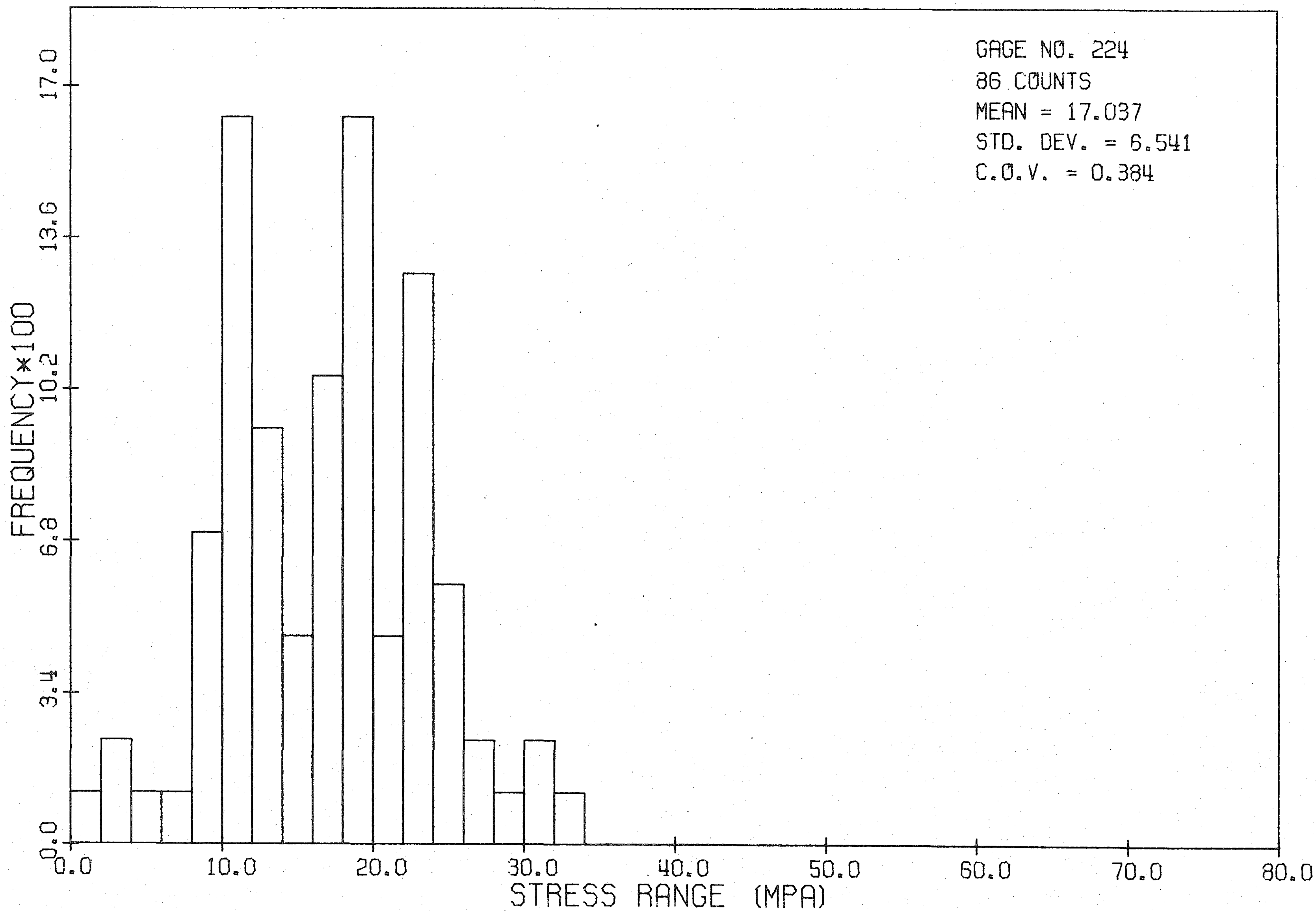

FIG. B.19 . STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 1

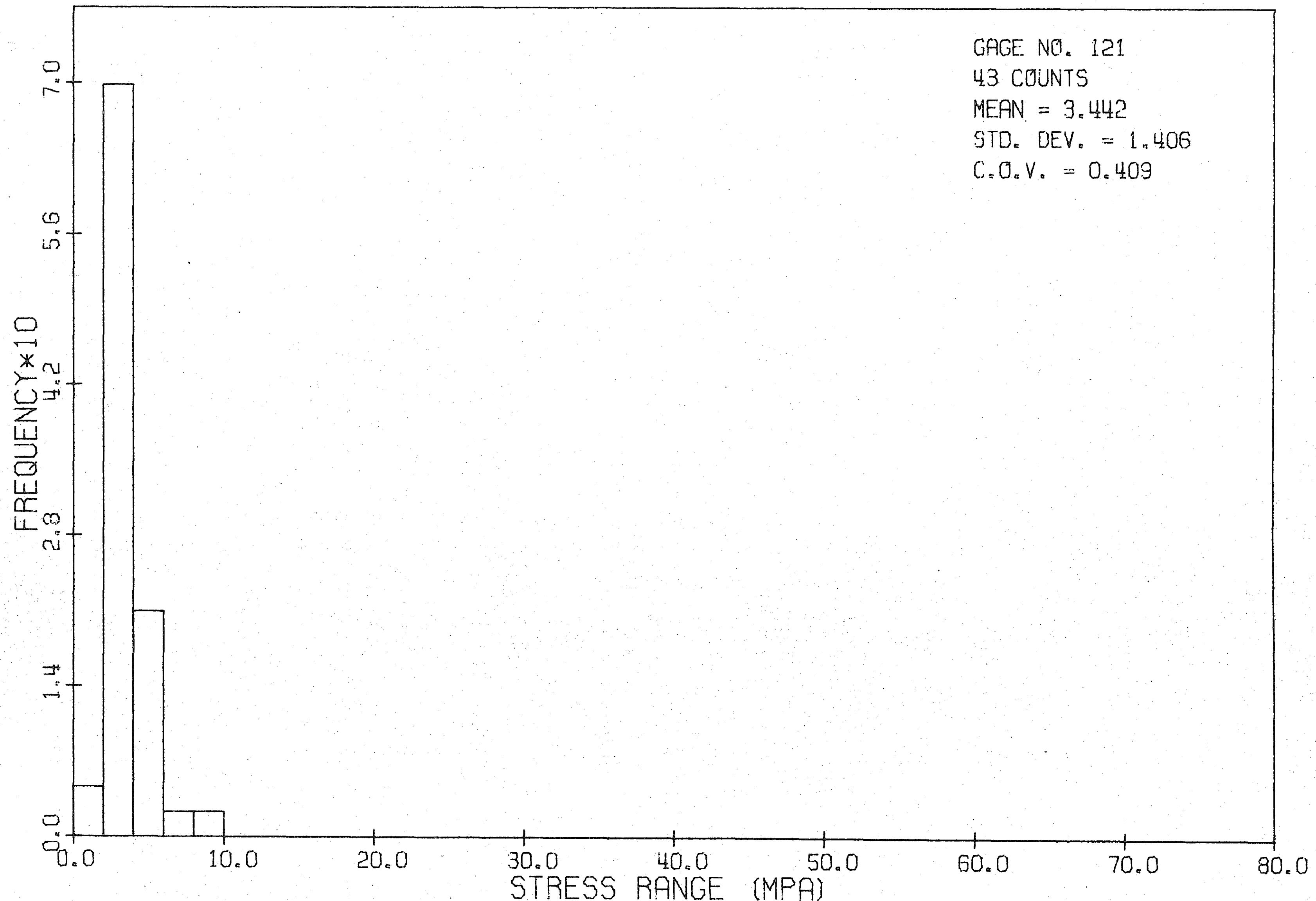

FIG. B.20 STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 2

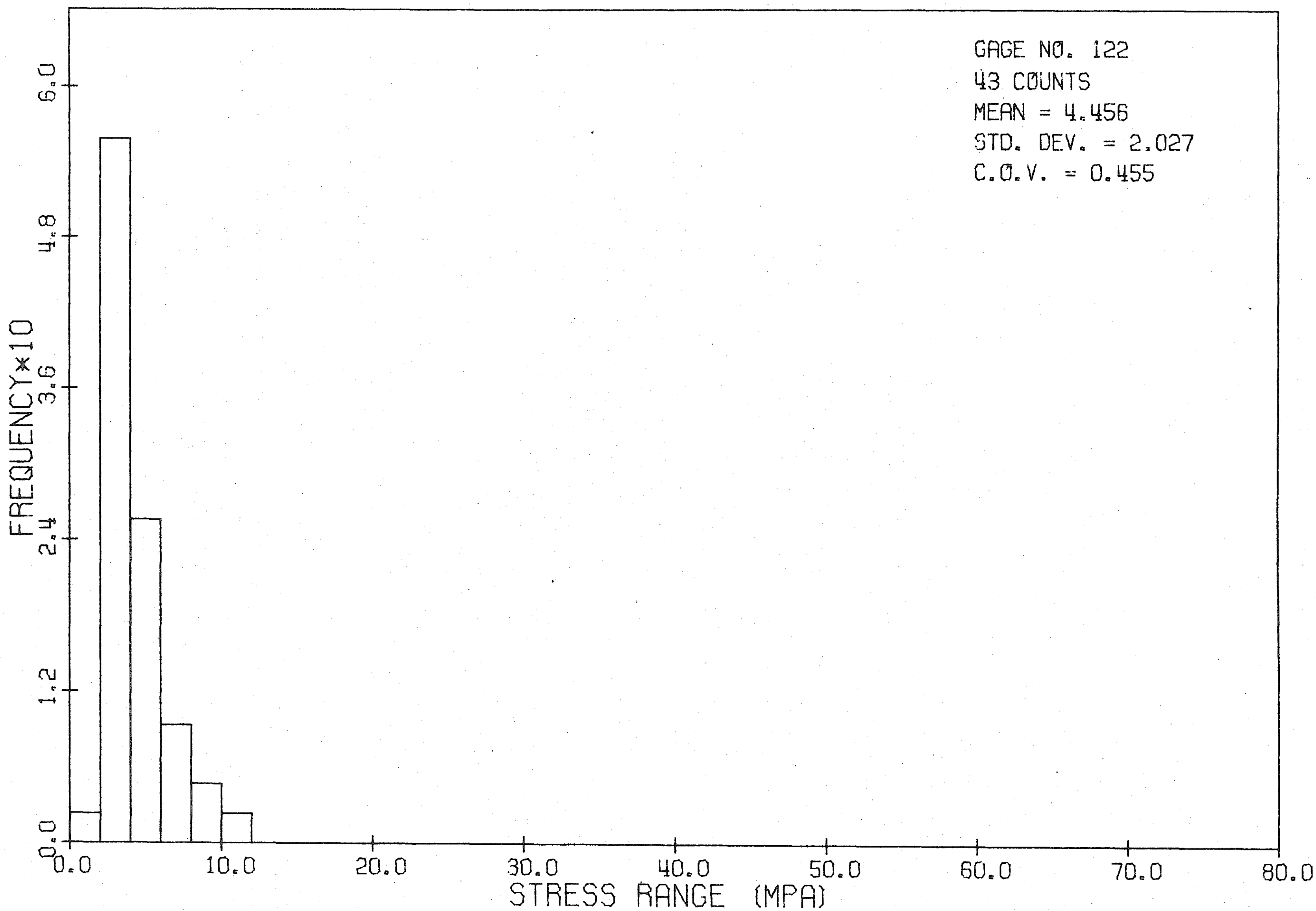

FIG. B.21 . STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 2

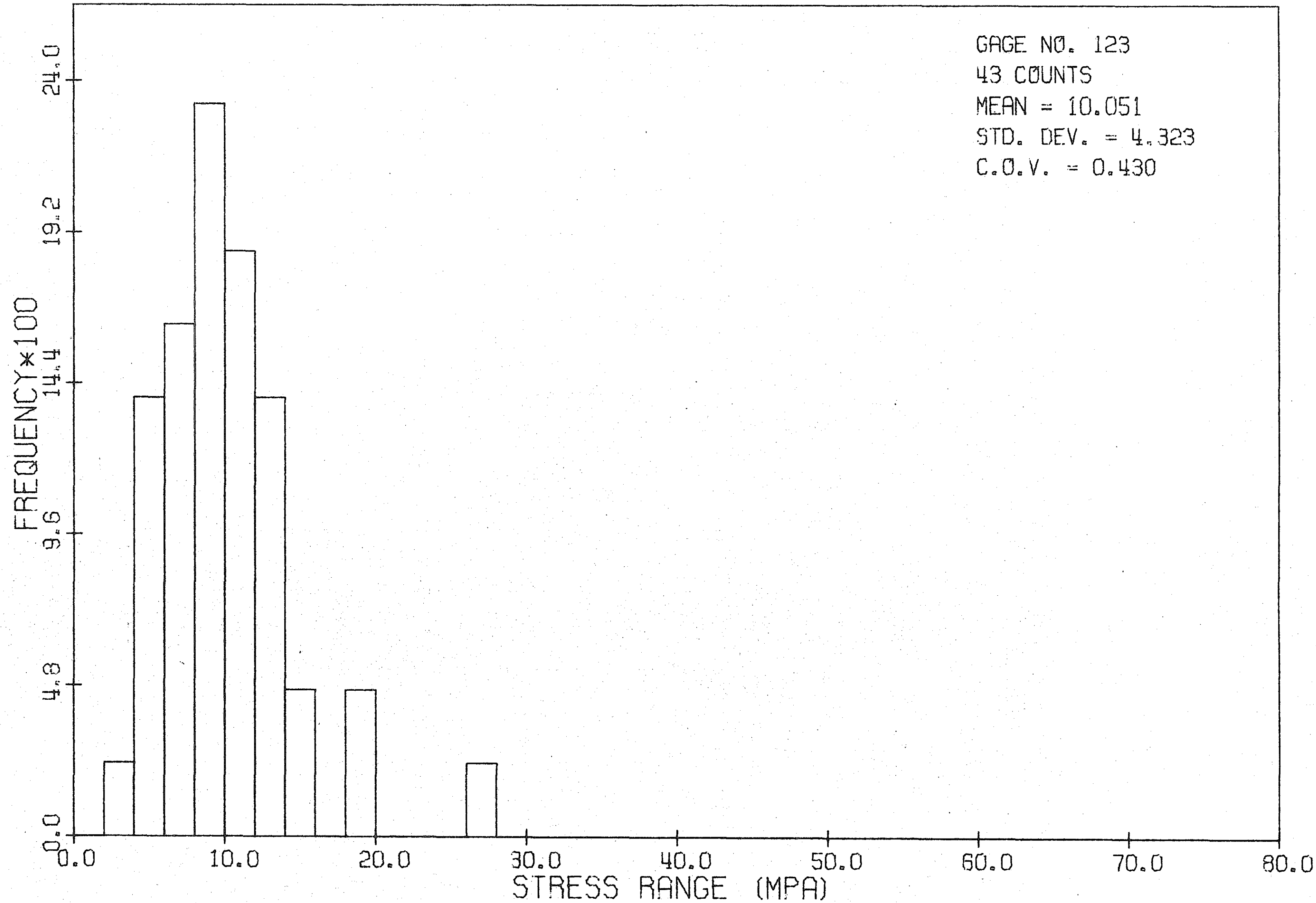

FIG. B.22 STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 2

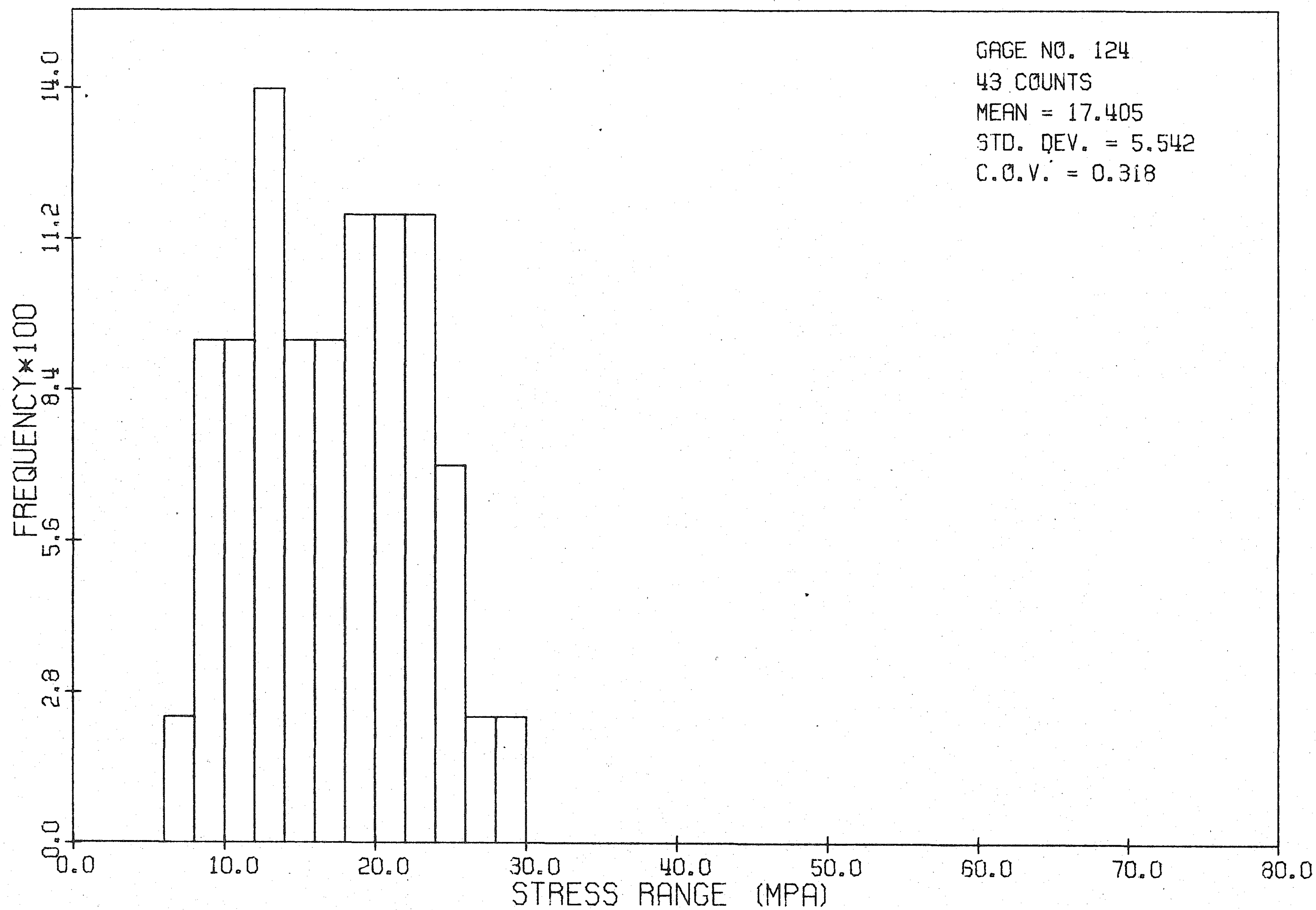

FIG. B.23 STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 2

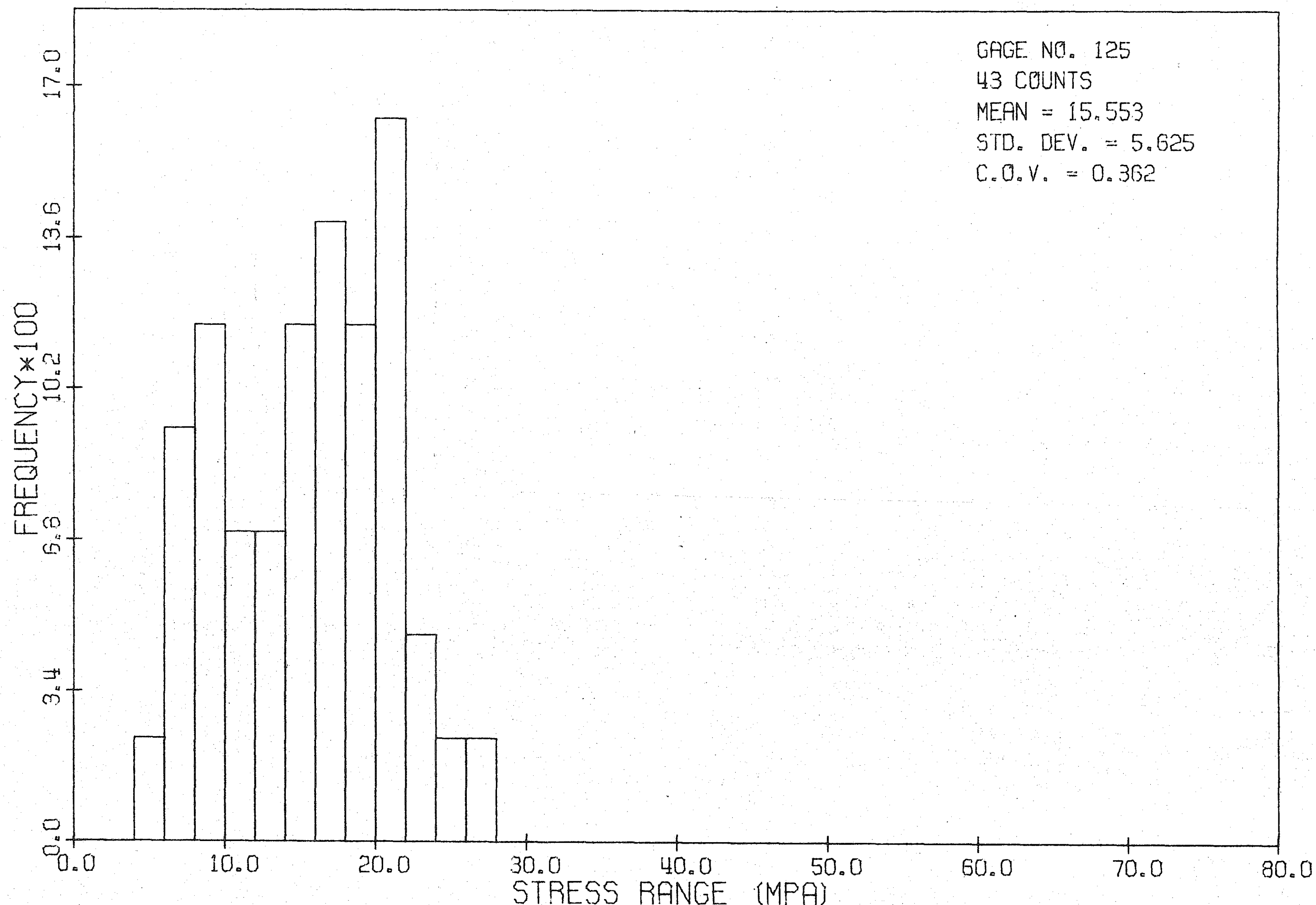

FIG. B.24 STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 2

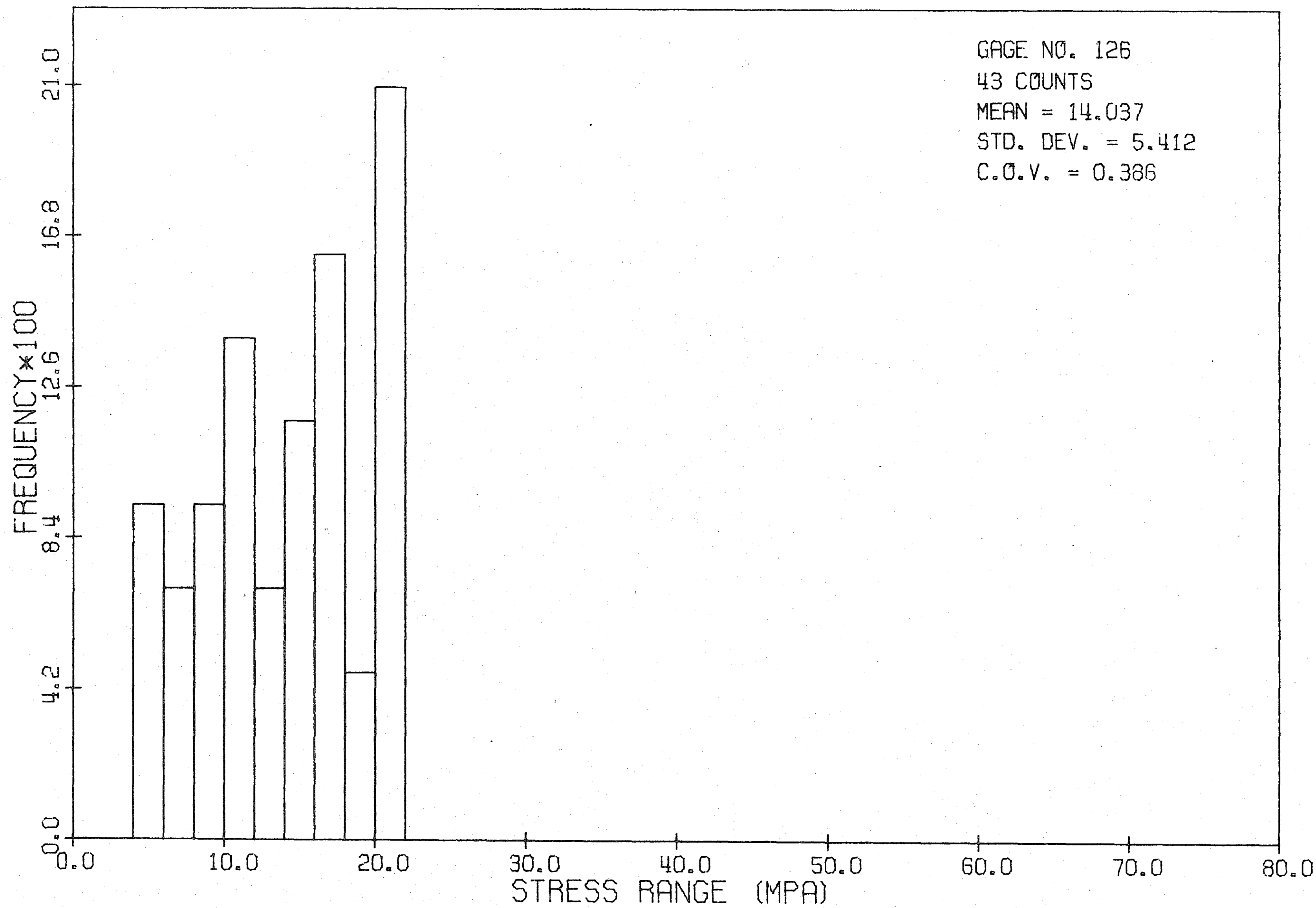

FIG. B.25 . STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 2

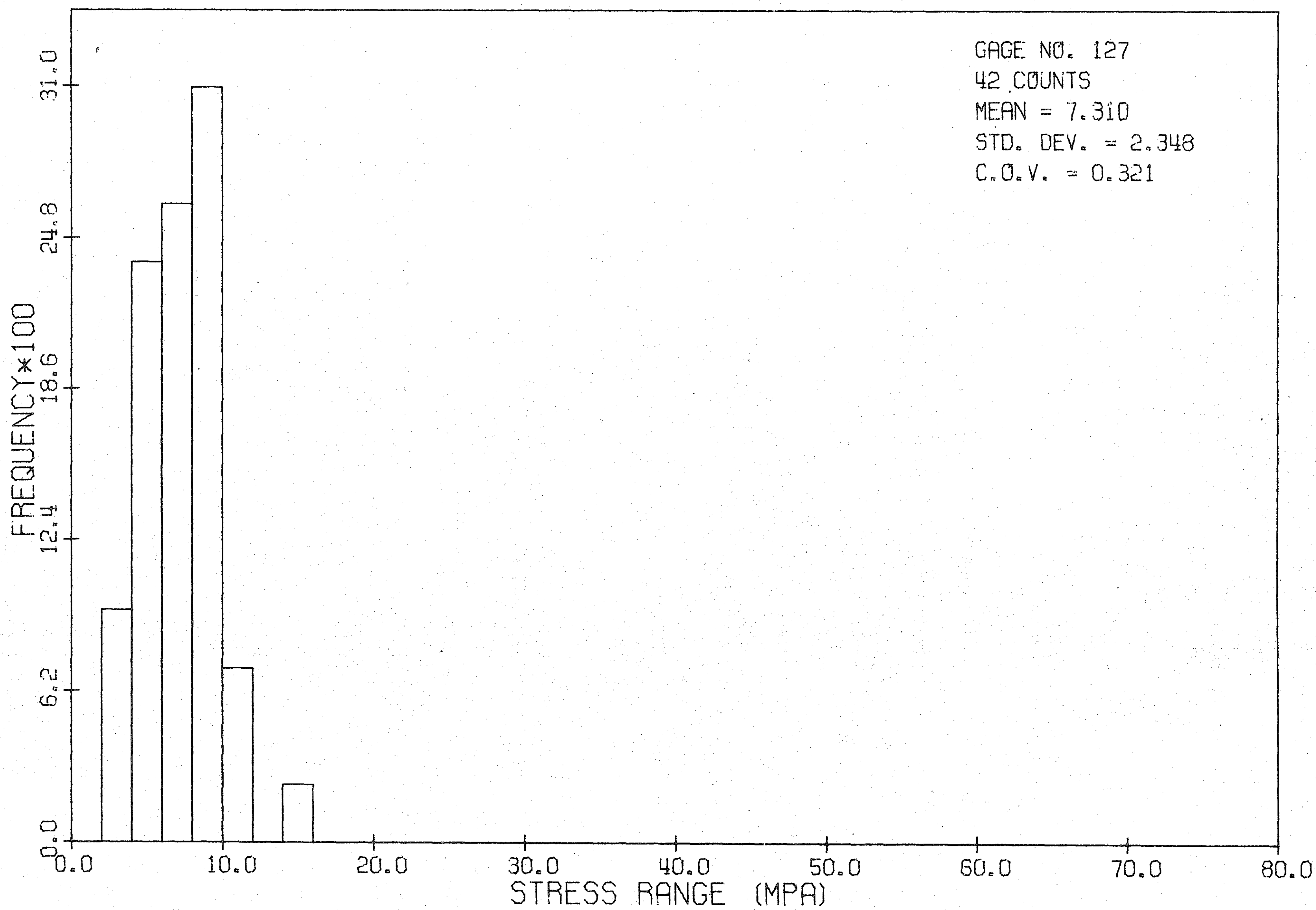

FIG. B.26 STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE(II)-PHASE 2

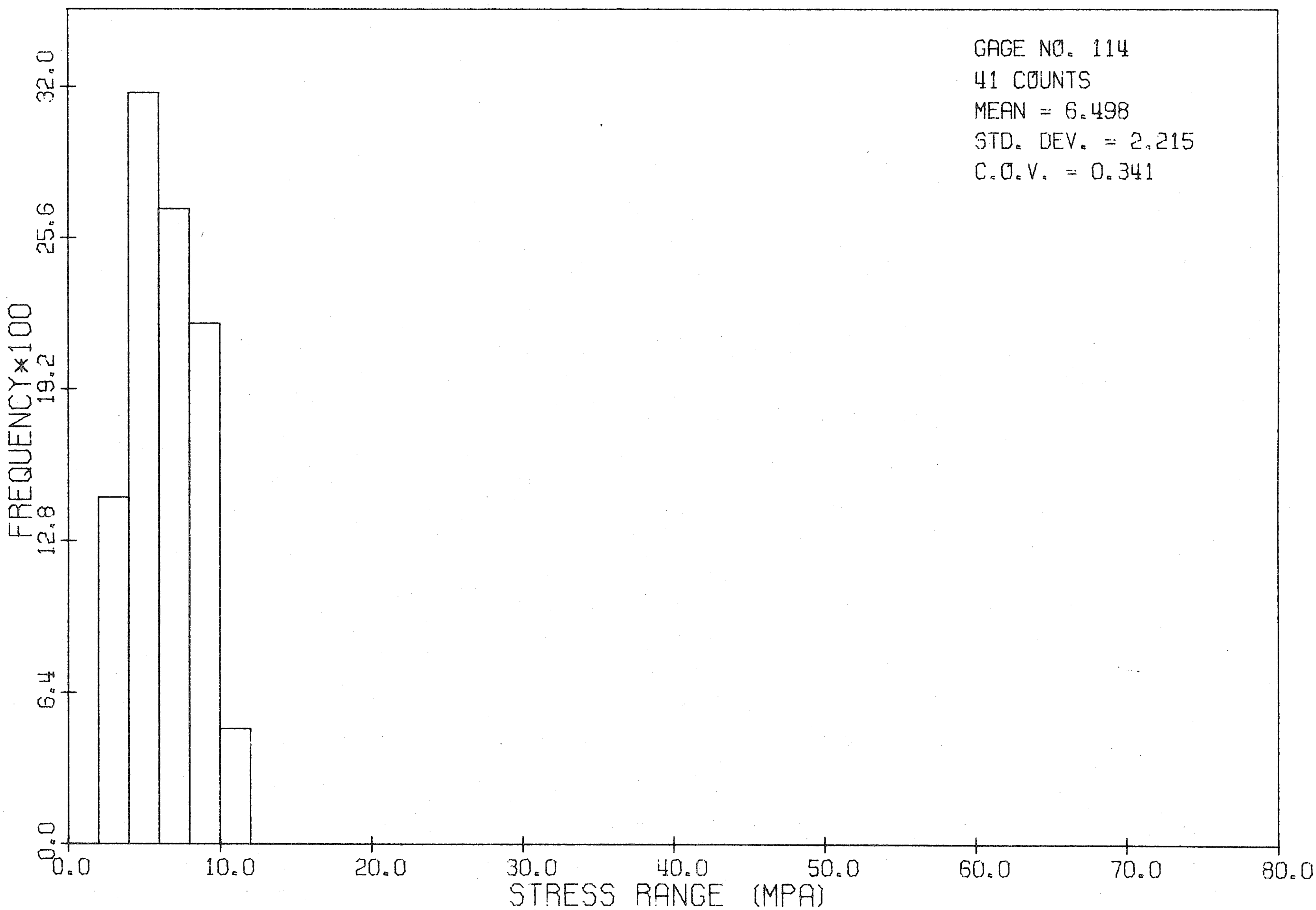

FIG. B.27 STRESS RANGE HISTØGRAM -- CAMP CREEK BRIDGE(II)-PHASE 2

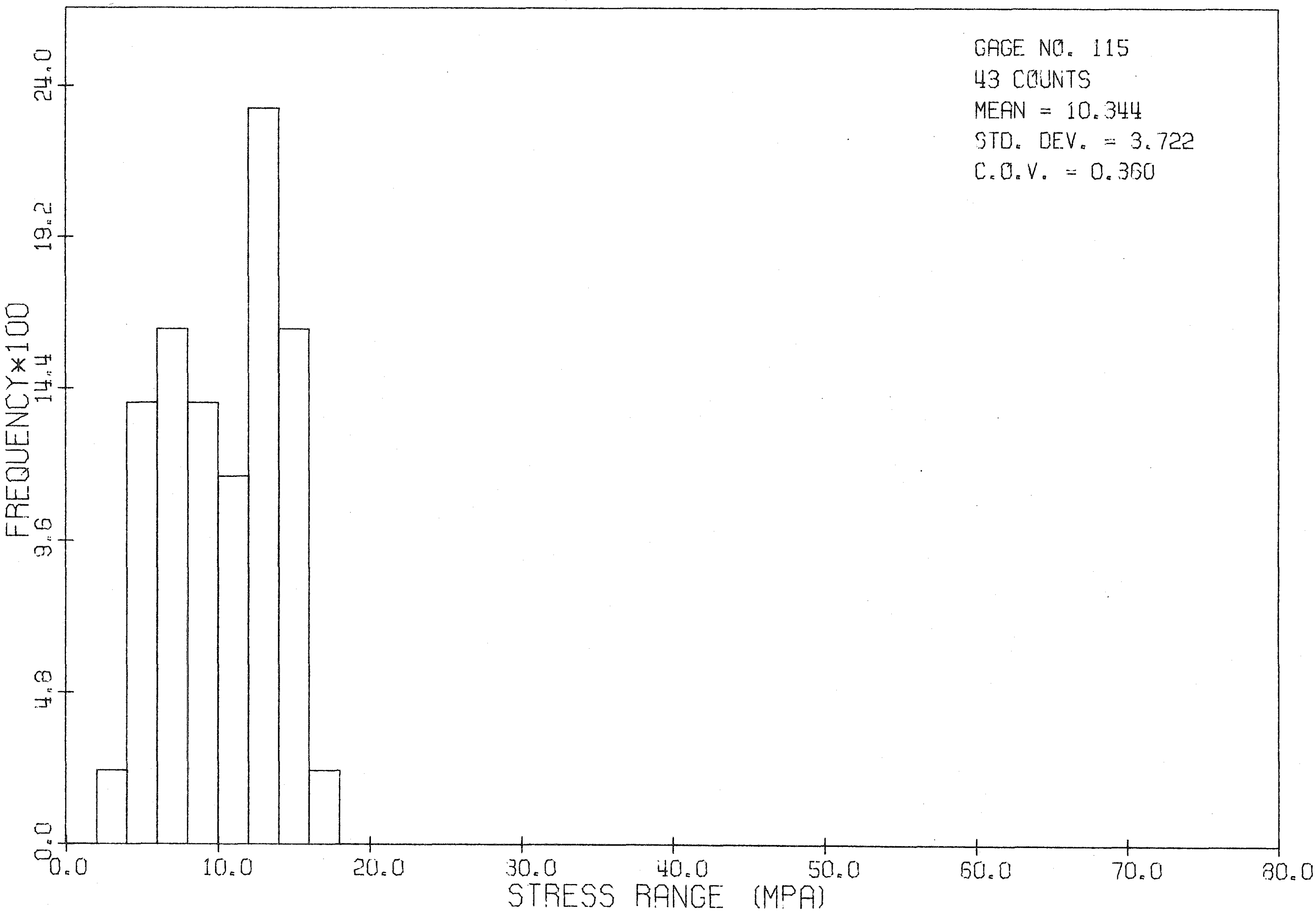

FIG. B.28 STRESS RANGE HISTØGRAM -- CAMP CREEK BRIDGE (II) -PHASE 2

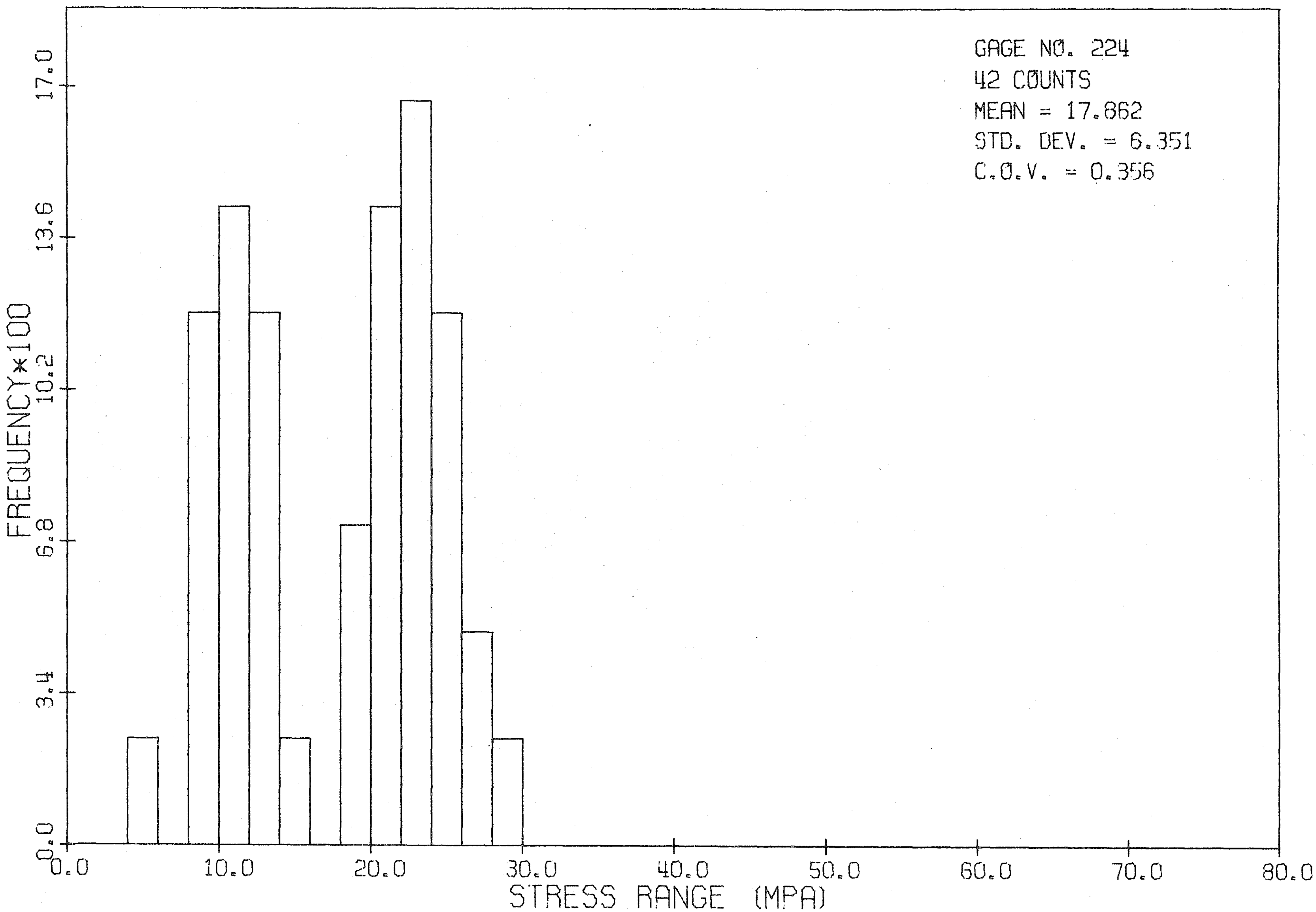

FIG.B.29 STRESS RANGE HISTOGRAM -- CAMP CREEK BRIDGE (II) -PHASE 2

APPENDIX C

TABLE C.1 HISTOGRAM DATA FOR STRESS RANGE
DAN RYAN BRIDGE--18TH STREET

STRAIN GAGE= 125 NOTE= 2 TOTAL EVENTS= 90

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
47.678	21.517	9.5356	4.3034

COEFFICIENT OF VARIATION= 0.451

NO COUNTS AFTER 10 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	13	0.144
30	6	7	0.078
40	8	20	0.222
50	10	10	0.111
60	12	12	0.133
70	14	6	0.067
80	16	18	0.2
90	18	2	0.022
100	20	2	0.022
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TABLE C.2 HISTOGRAM DATA FOR STRESS RANGE
DAN RYAN BRIDGE--18TH STREET

STRAIN GAGE= 125 NOTE= 3 TOTAL EVENTS= 139

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
40	25.086	8	5.0172

COEFFICIENT OF VARIATION= 0.627

NO COUNTS AFTER 10 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	42	0.302
30	6	20	0.144
40	8	21	0.151
50	10	11	0.079
60	12	9	0.065
70	14	11	0.079
80	16	16	0.115
90	18	7	0.05
100	20	2	0.014
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TABLE C.3 HISTOGRAM DATA FOR STRESS RANGE
DAN RYAN BRIDGE--18TH STREET

STRAIN GAGE= 125 NOTE= 4 TOTAL EVENTS= 112

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
48.67	20.339	9.734	4.0678

COEFFICIENT OF VARIATION= 0.418

NO COUNTS AFTER 10 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	9	0.08
30	6	16	0.143
40	8	15	0.134
50	10	19	0.17
60	12	18	0.161
70	14	14	0.125
80	16	15	0.134
90	18	4	0.036
100	20	2	0.018
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TABLE C.4 HISTOGRAM DATA FOR STRESS RANGE
DAN RYAN BRIDGE--18TH STREET

STRAIN GAGE= 125 NOTE= 5 TOTAL EVENTS= 177

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
52.486	23.143	10.4972	4.6286

COEFFICIENT OF VARIATION= 0.441

NO COUNTS AFTER 12 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	15	0.085
30	6	18	0.102
40	8	34	0.192
50	10	27	0.153
60	12	17	0.096
70	14	13	0.073
80	16	29	0.164
90	18	19	0.107
100	20	3	0.017
110	22	1	0.006
120	24	1	0.006
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TABLE C.5 HISTOGRAM DATA FOR STRESS RANGE
DAN RYAN BRIDGE--18TH STREET

STRAIN GAGE= 125 NOTE= 6 TOTAL EVENTS= 234

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
48.564	23.513	9.7128	4.7026

COEFFICIENT OF VARIATION= 0.484

NO COUNTS AFTER 11 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	25	0.107
30	6	42	0.179
40	8	37	0.158
50	10	24	0.103
60	12	36	0.154
70	14	23	0.098
80	16	20	0.085
90	18	14	0.06
100	20	9	0.038
110	22	4	0.017
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TABLE C.6 HISTOGRAM DATA FOR STRESS RANGE
DAN RYAN BRIDGE--18TH STREET

STRAIN GAGE= 125 NOTE= 7 TOTAL EVENTS= 192

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
46.367	22.197	9.2734	4.4394

COEFFICIENT OF VARIATION= 0.479

NO COUNTS AFTER 12 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	27	0.141
30	6	31	0.161
40	8	37	0.193
50	10	25	0.13
60	12	25	0.13
70	14	16	0.083
80	16	13	0.068
90	18	16	0.083
100	20	1	0.005
110	22	0	0
120	24	1	0.005
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TABLE C.7 HISTOGRAM DATA FOR STRESS RANGE
DAN RYAN BRIDGE--18TH STREET

STRAIN GAGE= 125 NOTE= 8 TOTAL EVENTS= 149

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
39.101	20.268	7.8202	4.0536

COEFFICIENT OF VARIATION= 0.518

NO COUNTS AFTER 9 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	31	0.208
30	6	33	0.221
40	8	24	0.161
50	10	14	0.094
60	12	15	0.101
70	14	18	0.121
80	16	12	0.081
90	18	2	0.013
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TABLE C.8 HISTOGRAM DATA FOR STRESS RANGE
DAN RYAN BRIDGE--18TH STREET

STRAIN GAGE= 125 NOTE= 9 TOTAL EVENTS= 92

MEAN STRAIN	STD.DEV.	MEAN STRESS (MPA)	STD.DEV. (MPA)
42.239	20.365	8.4478	4.073

COEFFICIENT OF VARIATION= 0.482

NO COUNTS AFTER 11 -TH INTERVAL

STRAIN INTERVAL	STRESS(MPA) INTERVAL	COUNT	FREQUENCY
10	2	0	0
20	4	20	0.217
30	6	9	0.098
40	8	19	0.207
50	10	14	0.152
60	12	12	0.13
70	14	7	0.076
80	16	7	0.076
90	18	3	0.033
100	20	0	0
110	22	1	0.011
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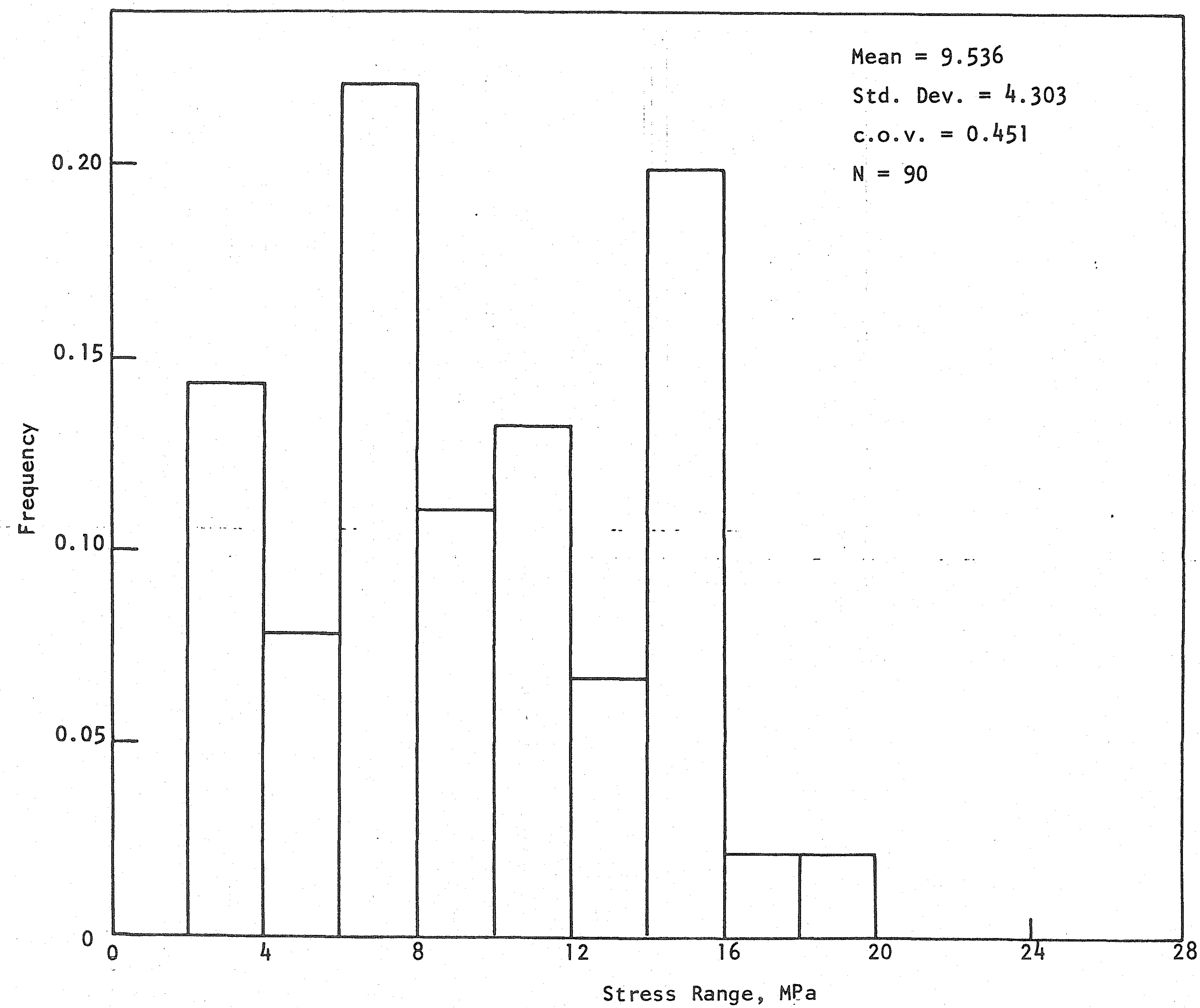

Fig. C.1 Histogram for Stress Range--18 th Street Bridge, Dan Ryan Expressway--8:00 to 8:30 pm
Bottom Flange, Beam 5

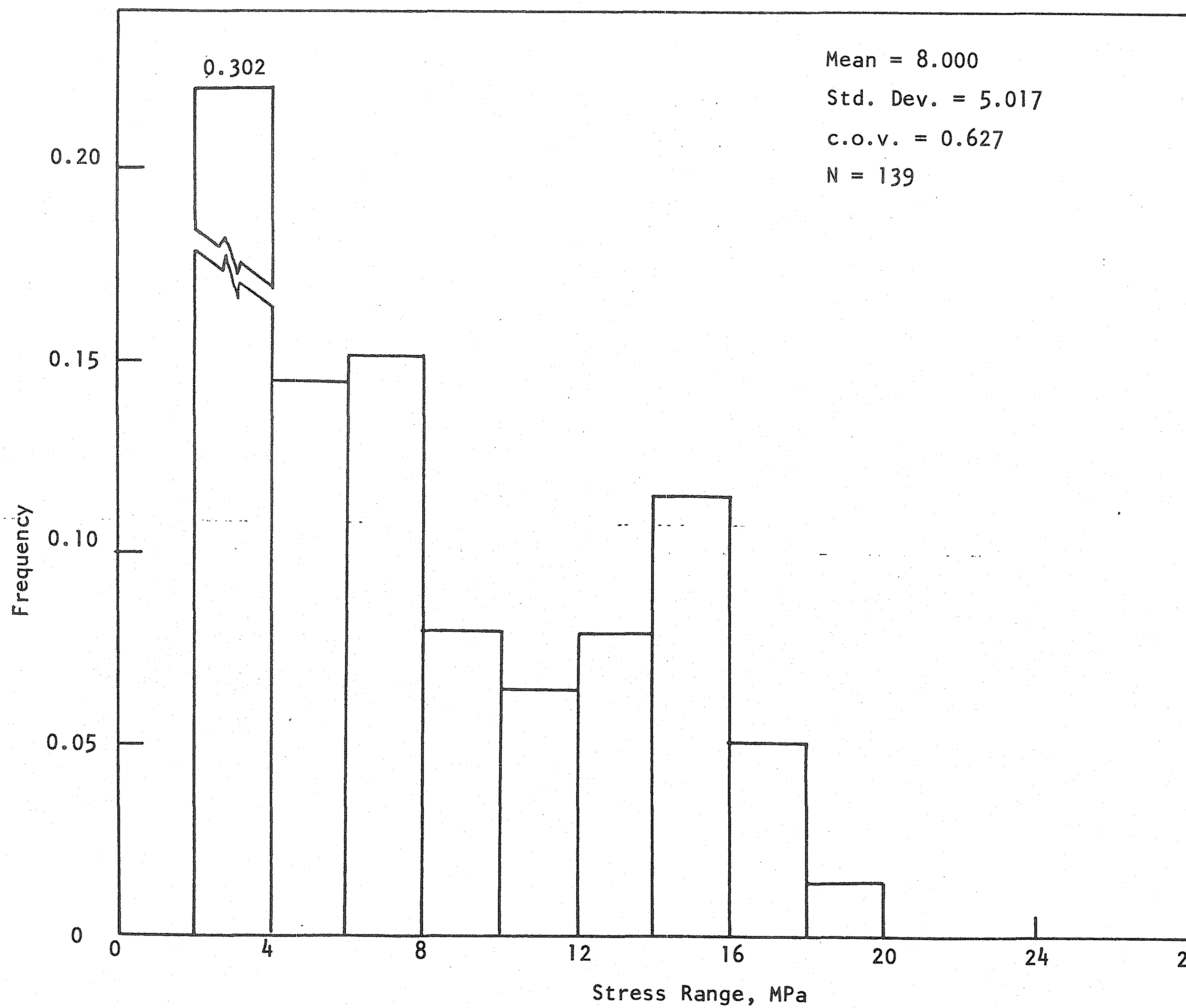

Fig. C.2 Histogram for Stress Range--18 th Street Bridge, Dan Ryan Expressway--11:43 pm to 12:13 am Bottom Flange, Beam 5

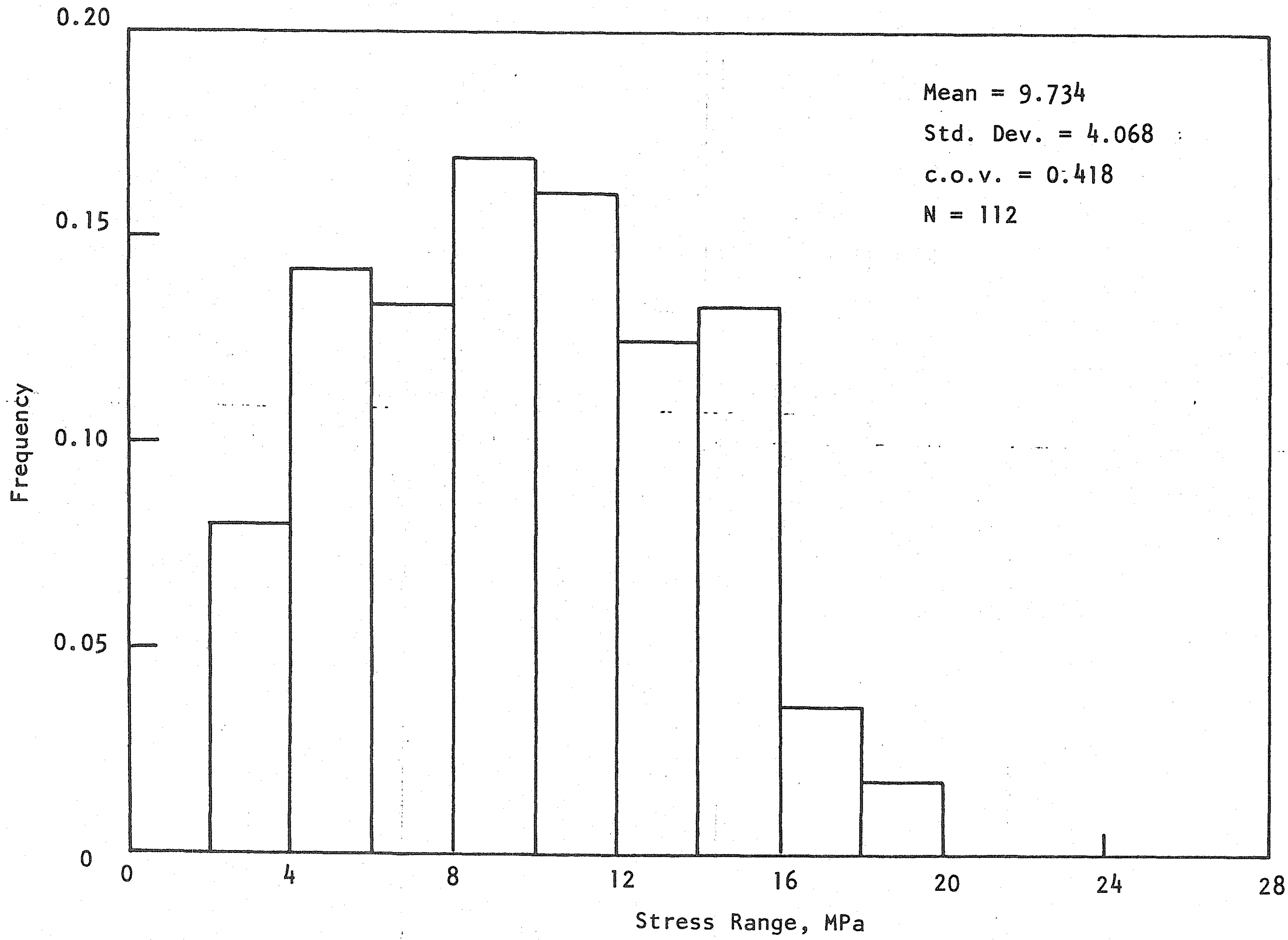

Fig. C.3 Histogram for Stress Range--18 th Street Bridge, Dan Ryan Expressway--3:16 to 3:46 am Bottom Flange, Beam 5

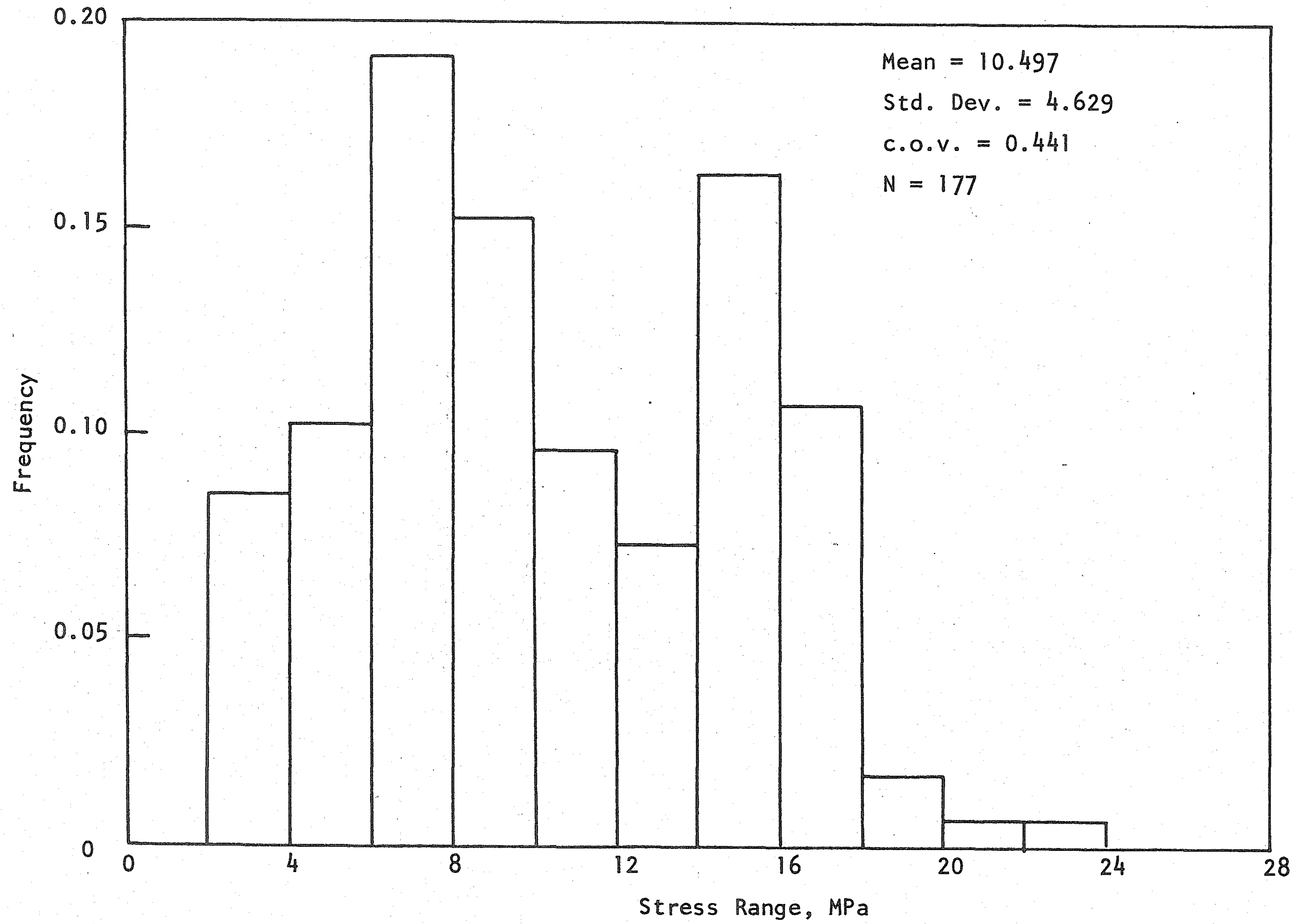

Fig. C.4 Histogram for Stress Range--18 th Street Bridge, Dan Ryan Expressway--5:40 to 6:10 am Bottom Flange, Beam 5

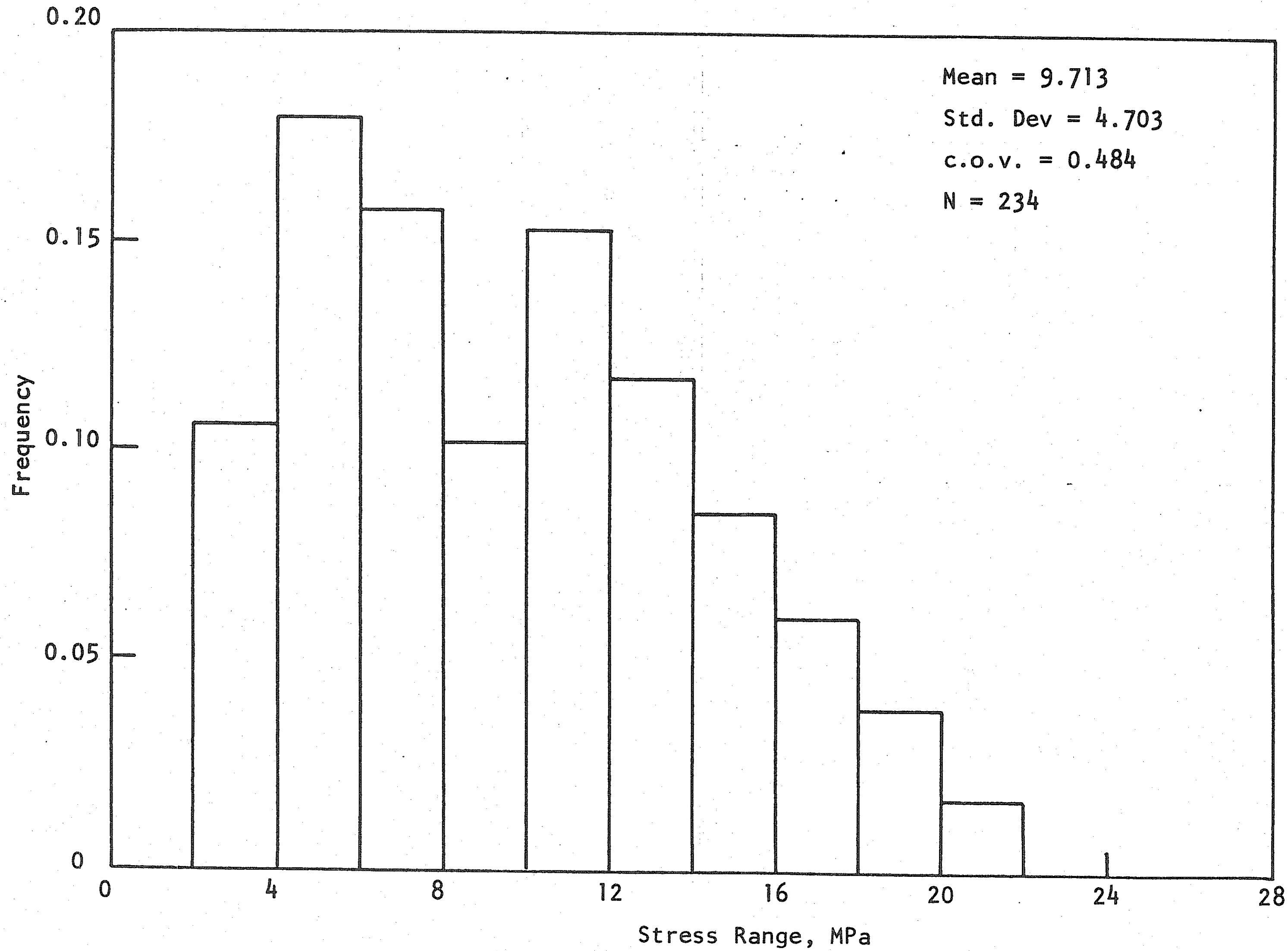

Fig. C.5 Histogram for Stress Range--18 th Street Bridge, Dan Ryan Expressway--9:01 to 9:36 am Bottom Flange, Beam 5

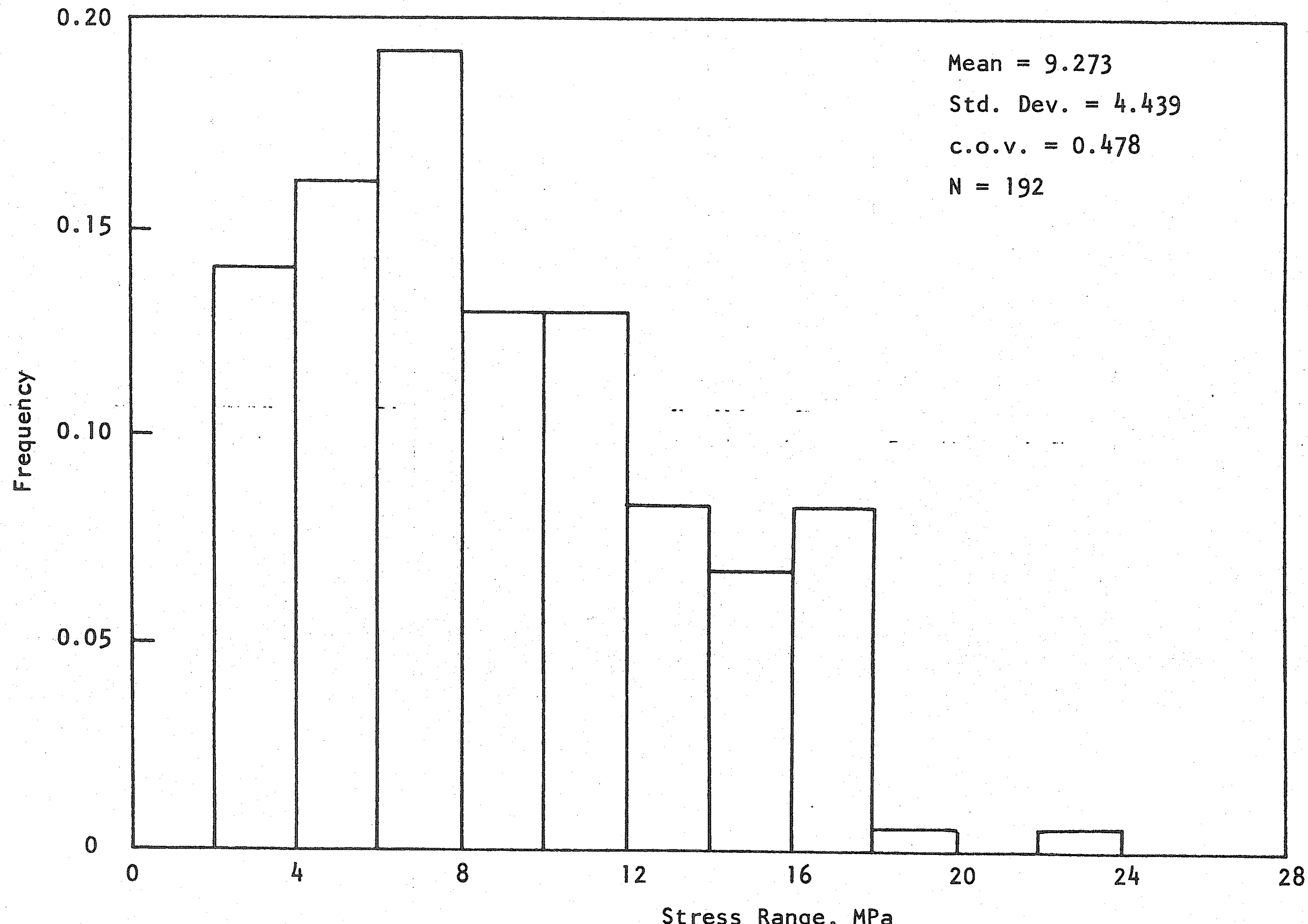

Fig. C.6 Histogram for Stress Range--18 th Street Bridge, Dan Ryan Expressway--11:31 am to 12:01 pm Bottom Flange, Beam 5

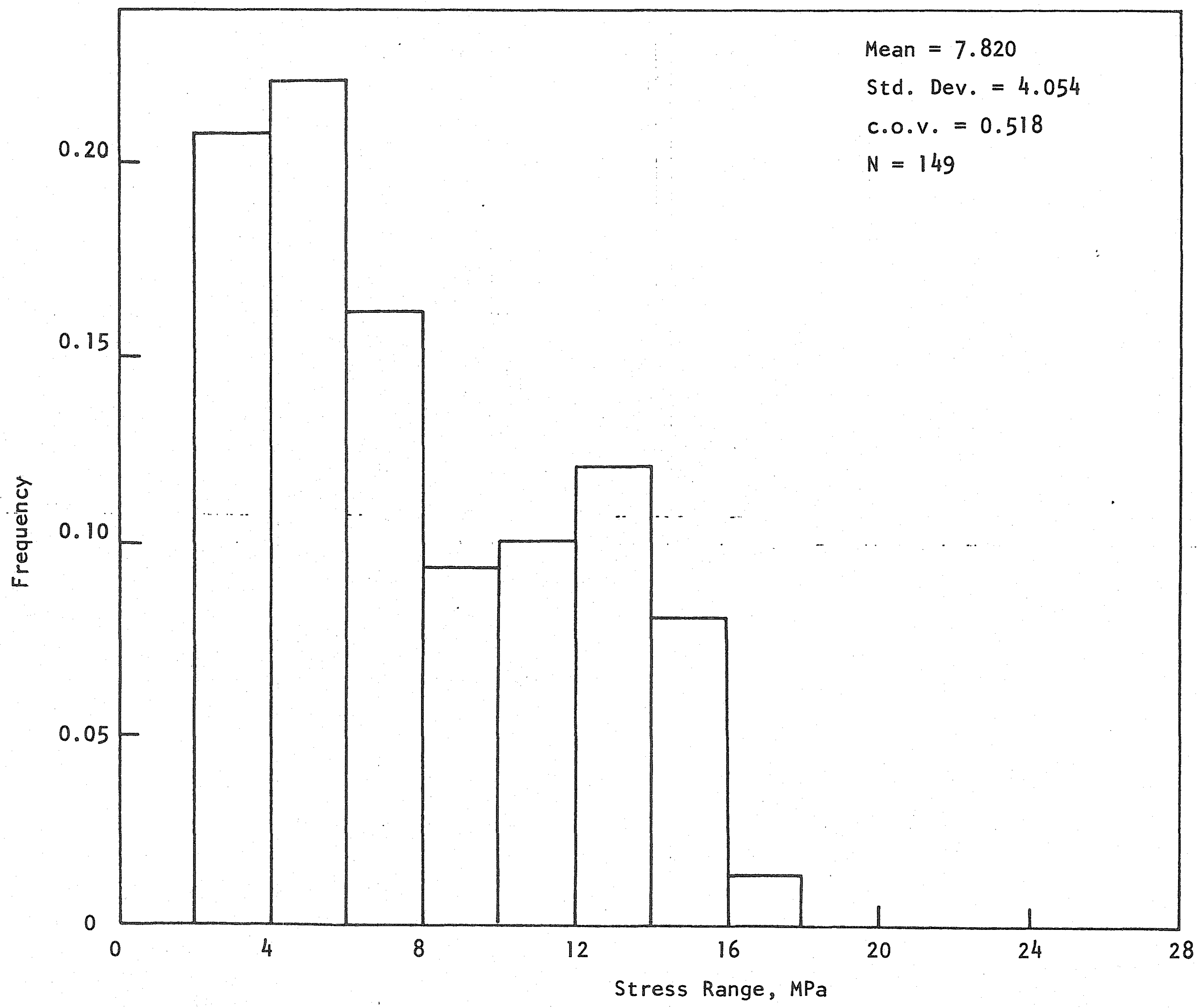

Fig. C.7 Histogram for Stress Range--18 th Street Bridge, Dan Ryan Expressway--2:28 to 3:00 pm
Bottom Flange, Beam 5

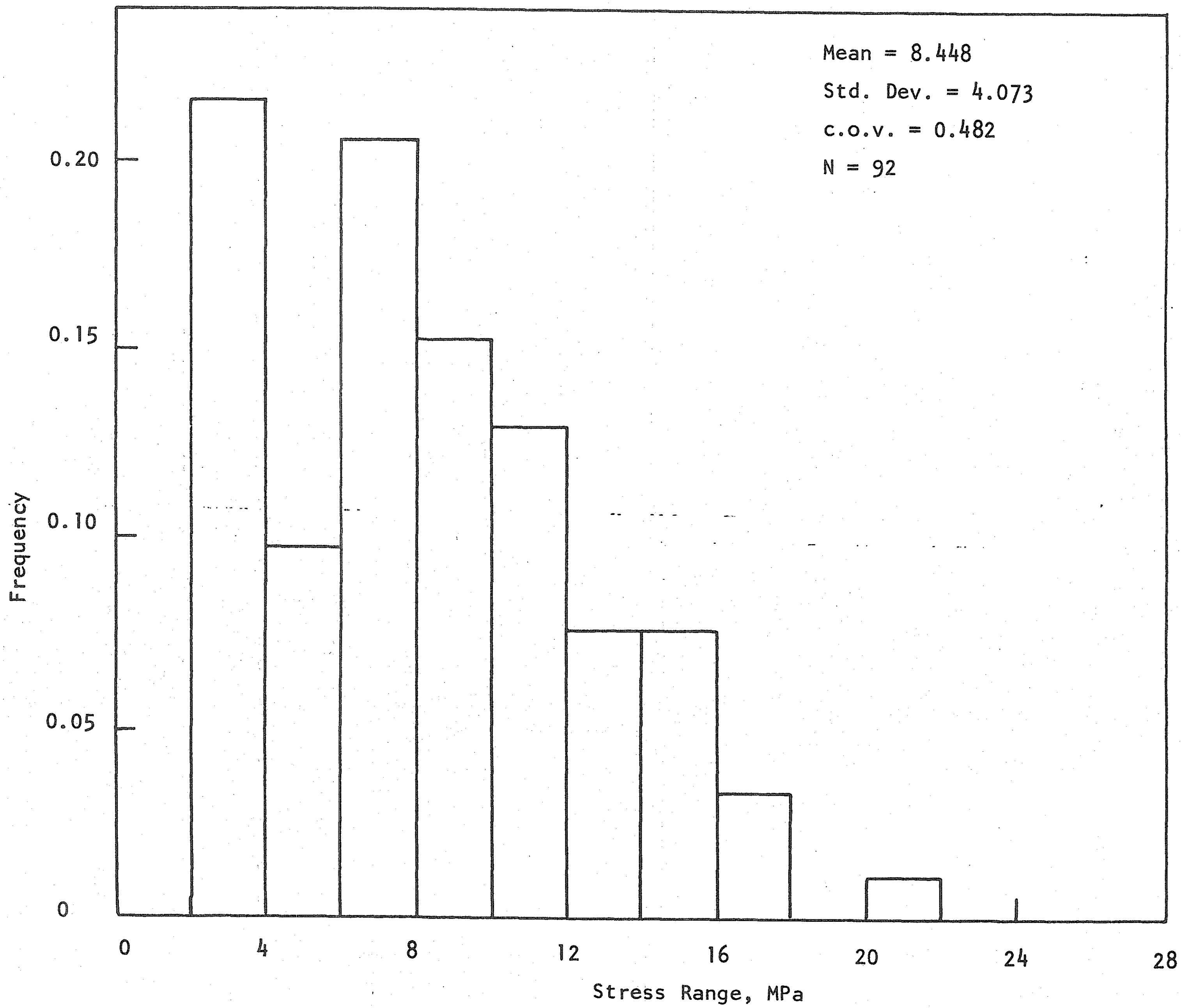

Fig. C.8 Histogram for Stress Range--18 th Street Bridge, Dan Ryan Expressway--5:14 to 5:44 pm Bottom Flange --Beam 5