FINAL REPORT

A LOADING HISTORY STUDY OF TWO HIGHWAY BRIDGES IN VIRGINIA

by

Wallace T. McKeel, Jr.
Highway Research Engineer

Charles E. Maddox, Jr.
Graduate Engineer Assistant

Henry L. Kinnier
Consultant
Virginia Highway Research Council

and

Charles F. Galambos
Structural Research Engineer
Federal Highway Administration

Virginia Highway Research Council
(A Cooperative Organization Sponsored Jointly by the Virginia
Department of Highways and the University of Virginia)

In Cooperation with the U. S. Department of Transportation
Federal Highway Administration

Charlottesville, Virginia

June 1971
VHRC 70-R48

SUMMARY

An evaluation of the stress ranges in two typical highway bridge spans under service loadings was made in a cooperative study by the Virginia Highway Research Council and the Federal Highway Administration. The strains at selected points on the superstructure elements of the spans were recorded continuously for periods of four and five days under normal traffic conditions by means of an automatic computer controlled data acquisition system, and converted to stress on the basis of assumed moduli of elasticity. The weights, axle spacings, and lateral positions by lane of trucks crossing the instrumented spans during the test periods were also recorded, to the degree possible.

The study proved the feasibility of utilizing the data acquisition system, which was developed for the FHWA, to obtain an indication of the service life to be expected of the test structures under today's truck traffic. The magnitudes of the stress ranges measured in the two simply supported test spans, a 76 foot steel beam composite span and a 60 foot prestressed concrete beam span, were considered acceptable, and it was concluded that both structures could safely accommodate an increase in traffic volume under current load limitations. The stress ranges recorded in the prestressed concrete beam span were low, indicating that fatigue considerations may not be critical in the case of such relatively massive spans of moderate length. However, since fatigue life is a function of the number of loadings, which will increase, as well as the magnitude of the stress ranges, the experimental results were not interpreted as justifying an increase in allowable weight limits, particularly in the case of the steel beam span.

A theoretical correlation between the recorded truck characteristics and the measured bridge response is also presented. While refinement of the analytical methodology is necessary to improve the correlation, the analysis, which is based on accepted theory, did serve to verify the magnitude of the experimentally obtained stress ranges.

FINAL REPORT

A LOADING HISTORY STUDY OF TWO HIGHWAY BRIDGES IN VIRGINIA

by

Wallace T. McKeel, Jr.
Charles E. Maddox, Jr.
Henry L. Kinnier
and
Charles F. Galambos

INTRODUCTION

The life expectancy of highway bridges is of increasing interest to bridge engineers throughout the United States. Bridges are being subjected to loading by an ever increasing number of heavy trucks whose allowable axle and gross weights are periodically being increased. A significant increase in the frequency and magnitude of the loadings to which a bridge is subjected could cause fatigue problems that shorten the service life of medium and short span structures.

Many field tests of bridges, employing loading by a variety of test vehicles, have been conducted over the past several years, but the need to evaluate the stresses produced by normal truck traffic remained, largely because of difficulties involved in obtaining the data with existing strain measuring equipment. A contract for an instrumentation system which provides a practical means of assessing the structural behavior of bridges under service loadings was awarded by the Structures and Applied Mechanics Division, Office of Research and Development of the Federal Highway Administration (FHWA) to the Scientific Data Systems Corporation of Santa Monica, California, in 1965.[1] After delivery of the system in 1966, a nationwide program of cooperative studies guided by committees of the American Society of Civil Engineers and the Highway Research Board was inaugurated by the FHWA. Such a study, reported herein, was begun in Virginia in July 1968.

PURPOSE AND SCOPE

The primary purpose of the subject study was to determine experimentally the stresses produced by service loadings at selected points on the superstructures of typical highway bridges in Virginia. Two structures, a steel beam composite span bridge on Interstate Route 95 and a prestressed concrete beam span bridge on Interstate Route 81, were selected for testing. One span on each bridge was instrumented, and data were collected during periods of roughly four to five consecutive days at each structure. No attempt was made to determine the effect of forces due to temperature differentials or to evaluate the effect of seasonal variations in traffic volume. A theoretical study was conducted to verify the magnitude of the strains measured during the experimental phase of the study.

EXPERIMENTAL PROCEDURE

The two structures included in the study, a steel beam composite span bridge carrying the northbound lane of Route 95 over Quantico Creek and Route 629 near Dumfries, Virginia, and a prestressed concrete beam bridge carrying the northbound lane of Route 81 over Cedar Creek near Middletown, were chosen because of their proximity to permanent weighing stations which were in operation 24 hours per day. In general, the experimental procedure included monitoring strains at selected points on the superstructures and noting the type and lateral position by lane of all trucks crossing the bridge. Separate records of axle weights and spacings were kept at the nearby weighing stations for subsequent correlation with the vehicle data taken at the bridge. The trucks were classified by the axle types shown in Figure 1 in conjunction with a body type description such as van, flatbed, tank, or car carrier and an identification of the operating company. Data collection was continuous over a period of 105 hours at the Route 95 site and 84 hours at the Route 81 site. Data collection instrumentation and procedures are described in more detail in the following sections.

Strain Monitoring Instrumentation

The automated, computer controlled data acquisition system developed by Scientific Data Systems for the FHWA has been described in detail in other publications, and only a summary of its functions will be given here.(1) Essentially, the equipment, which is housed in a test trailer, takes the output from a maximum of ten resistance type strain gages in the form of analog voltages, digitizes the voltages, and stores and tabulates strain ranges for printing out at specified intervals. The equipment produces no visual record of a strain trace, nor does it relate individual strain ranges to specific trucks.

The system, which operated continuously during testing, was programmed to record strains for periods of one hour and type out the results. Four minutes of each hour were consumed in printing the stored data, and since strains were not monitored, vehicles crossing the structure during this period were stricken from the truck records. The output, Figure 2, consisted of an array showing the number of occurrences at each of nine strain ranges — listed in the left-hand column — for each of the ten gages, represented by channels 1-10 in the top row. Thus, in the printout at 1401 hours, shown in Figure 2, the circled figure indicates that five strain ranges between predetermined levels 5 and 6 were recorded during the preceding hour for gage number 2. No higher strain ranges were recorded, as indicated by the zeroes above the circled number.

The event recorded as a strain range during the Virginia tests is shown in Figure 3. The minimum test level shown in Figure 3 is a preselected level below which no strain ranges were recorded in order to eliminate the effect of automobiles crossing the span. A strain range is measured from peak to valley; the computer seeks a peak strain when the signal exceeds the minimum test level and it seeks a valley when the signal drops to or below the zero level. An event is counted each time the signal passes the minimum test level and returns to zero. It is, therefore, possible, and not uncommon, for a single vehicle to produce more than one recorded strain range; the ensuing events are referred to as secondary strain ranges.

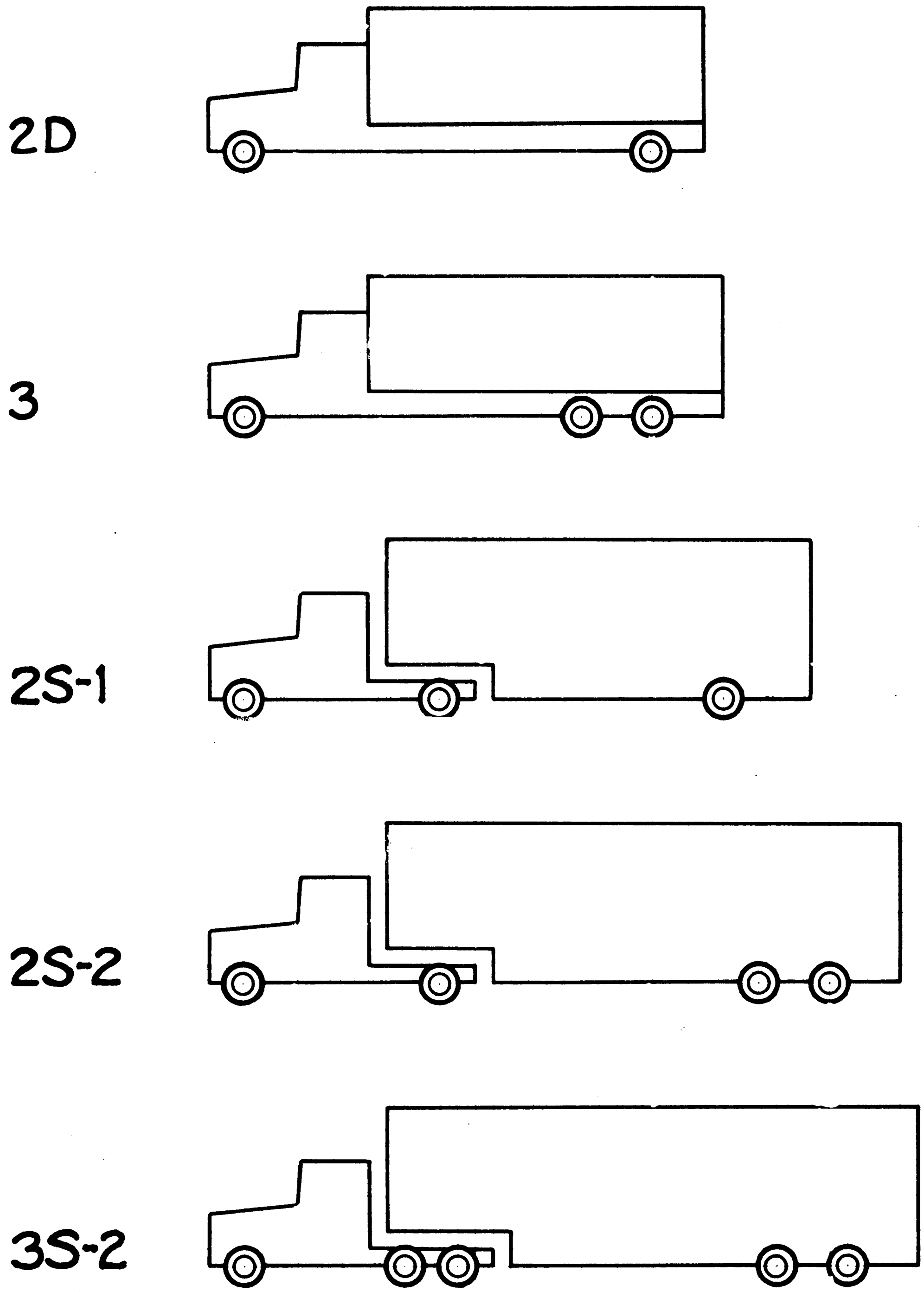


Figure 1. Axle type designations.

2292

TIME: 1401

STRAIN RANGE	CHANNEL 1	2	3	4	5	6	7	8	9	10
	0	0	0	0	0	0	0	0	0	0
1										
	0	0	0	0	0	0	0	0	0	0
2										
	0	0	0	0	0	0	0	0	0	0
3										
	0	0	0	1	0	0	0	0	0	0
4										
	0	0	1	0	0	0	0	0	1	0
5										
	0	5	2	2	0	2	3	0	5	1
6										
	6	13	9	3	0	4	12	0	9	2
7										
	27	10	25	18	10	17	21	1	14	14
8										
	27	15	20	45	16	38	18	13	5	12
9										
	0	0	0	0	0	0	0	0	0	0

Figure 2. Sample of hourly data printout.

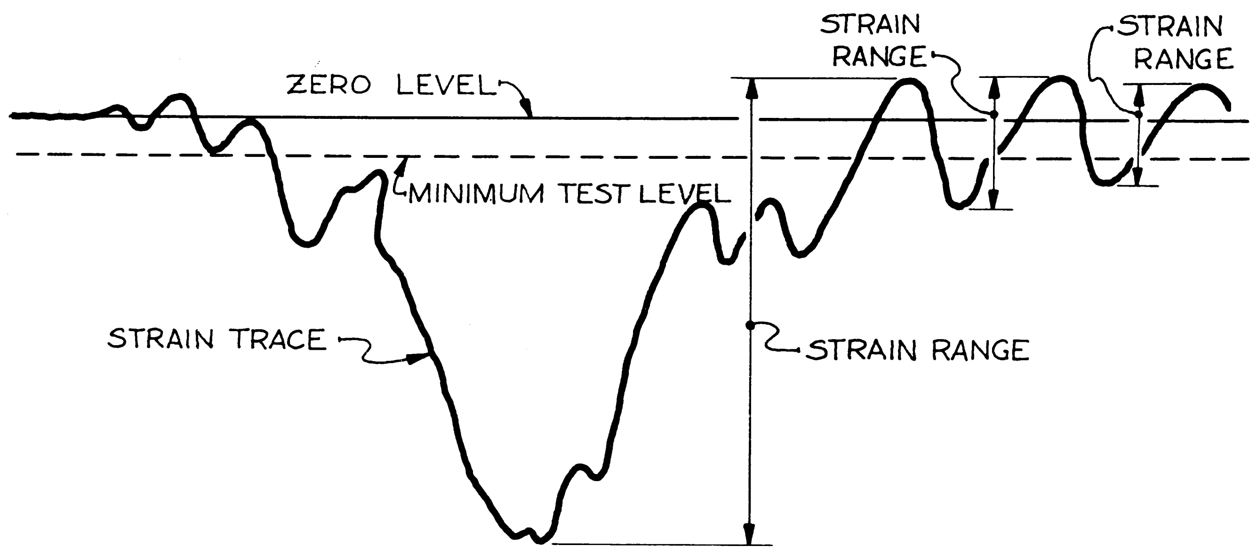


Figure 3. Strain ranges as defined by FHWA equipment in the Virginia tests.

Truck Data Acquisition

As mentioned previously, an observer at the bridge attempted to catalog each truck as it crossed the instrumented span, identifying the operating company and noting the axle and body type of the vehicle and its lane of travel. Records of axle weights and spacings along with similar identifying characteristics were compiled at the weighing station for correlation with the site data.

The number of trucks crossing the bridge is compared with the number for which weighing station data were completed in Figures 4 and 5. As shown in these figures, a greater number of vehicles were recorded at the bridge than at the weighing station. There are several causes for the discrepancy, which was not serious. There was an exit between the bridge and the weighing station in each case, which allowed local vehicles and some overloaded vehicles to leave the highway. Some vehicles were eliminated because of insufficient identification, and, finally, data of dubious accuracy occurred during periods of heavy traffic or inclement weather. Overall, however, weighing station records were completed on slightly more than 85% of the trucks.

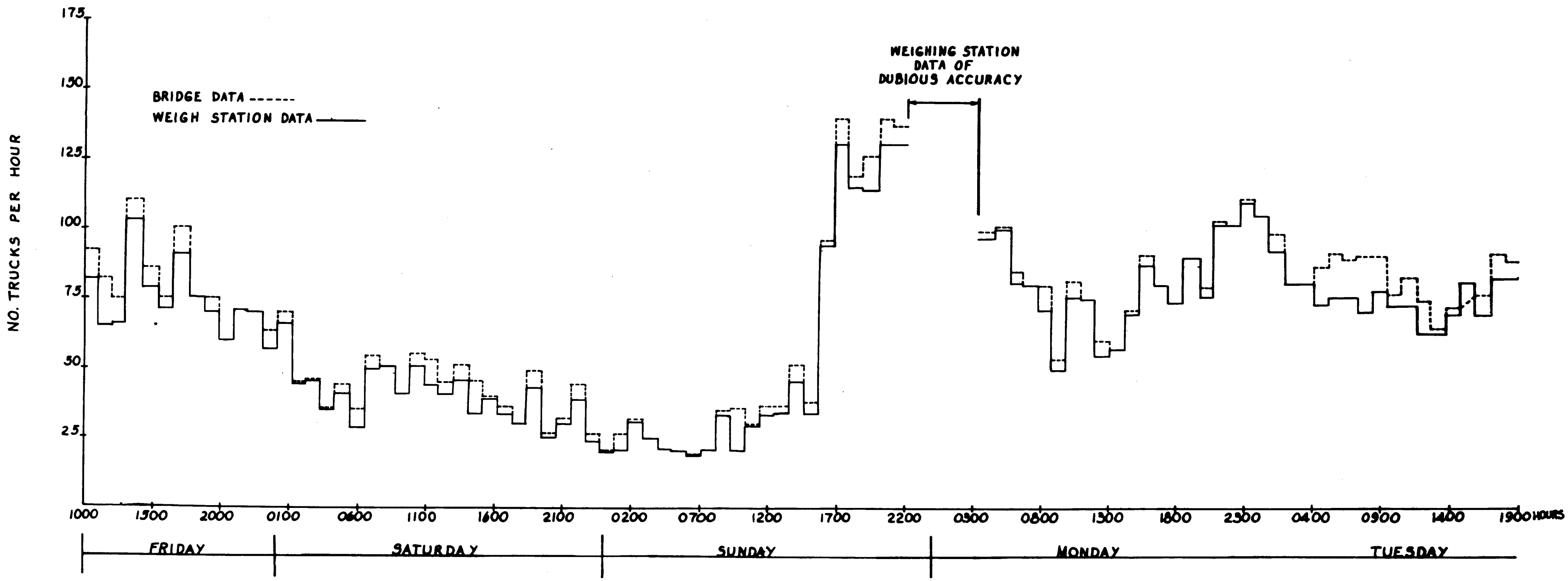


Figure 4. Number of trucks per hour recorded at the bridge and weigh station, Rte. 95 tests.

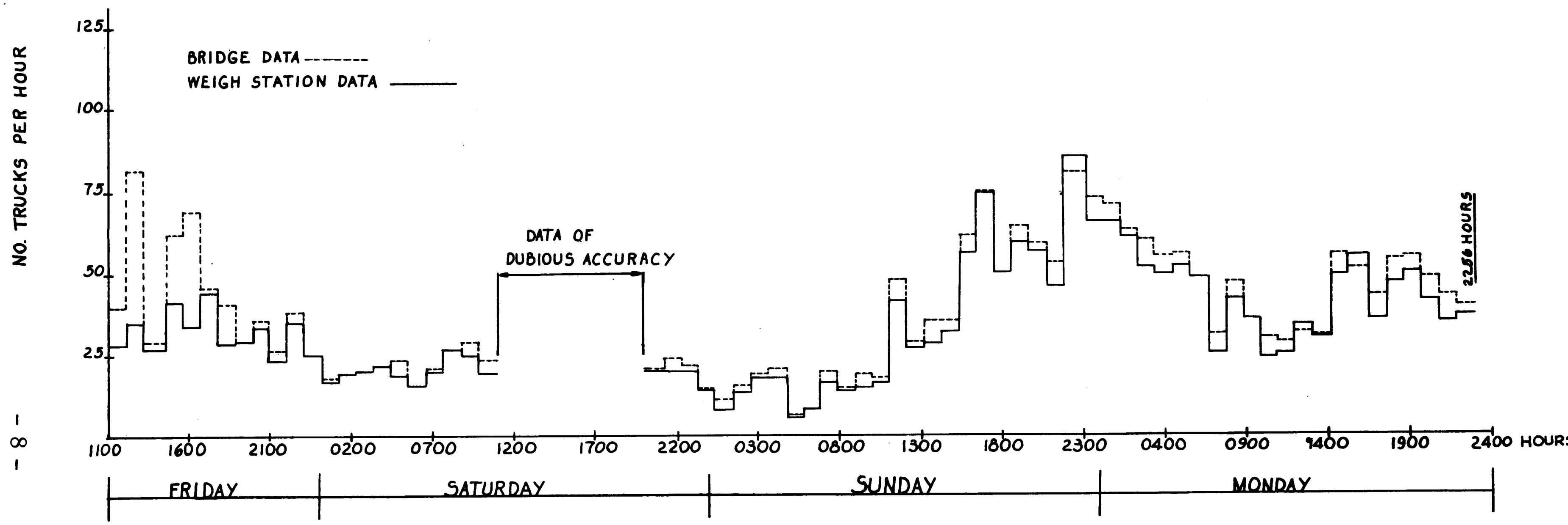


Figure 5. Number of trucks per hour recorded at the bridge and weigh station, Rte. 81 tests.

General Description

The bridge carrying the northbound lane of Route 95 over Quantico Creek and Route 629, located in a generally urban environment 30 miles south of Washington, contains a series of three simply supported steel beam composite spans 69 feet in length and one span 76 feet in length, which was chosen for testing. The bridge is located at the low point of a 1,000 foot vertical curve, and it lies on a slight horizontal curve. The spans are skewed at $3^{\circ} 46'$.

Average daily traffic volumes at the site during the years 1968-1970, based on figures published by the Virginia Department of Highways, are shown in Table I which also presents the percentage of trucks and buses.[3] The buses, which were not required to stop at the weighing station, constitute only 1% of the total average daily traffic.

TABLE I

AVERAGE DAILY TRAFFIC VOLUMES

Rte. 95 (NBL) from Rte. 619 W. of Triangle to Rte. 234 N. of Dumfries (From reference 3)

		1968*		1969		1970	
		No.	%	No.	%	No.	%
	A.D.T. — all vehicles	15,060	100	16,160	100	18,185	100
Truck Type	2D (4-6 tires)	620	4.1	1,000	6.2	1,120	6.2
Truck Type	3 (6-10 tires)	30	0.2	35	0.2	65	0.3
Truck Type	2S-1, 2S-2, and 3S-2 (combined)	2,000	13.3	2,100	13.0	2,150	11.8
	Total Trucks	2,650	17.6	3,135	19.4	3,335	18.3
	Buses	160	1.1	175	1.1	150	0.8

*Year of Test

2298

Details of Test

Test Span

The instrumented span, detailed in Figure 6, measures 74'6" center to center of bearings. The supporting elements are six 36" wide flange beams with partial length cover plates over the central portion of the span. Welded stud shear connectors ensure composite action between the beams and the 8" concrete deck. For the purposes of this study the beams were numbered 1 through 6 from the east side of the structure. The 42'-0" clear roadway is divided into three traffic lanes, numbered from the east side of the bridge such that lane 1 is the right-hand lane.

Gage Locations

The locations of the ten strain gages on the instrumented span are shown in Figure 7. Gages 1-5, which were positioned as shown in Figure 7, will be referred to as midspan gages. Gage number 9 was placed on a transverse reinforcing bar in the lower level of the deck steel, 49" from the west side of beam 2 and 14" south of the center of the diaphragm, near the point of maximum positive moment in the slab. Gage number 10 was placed on the bottom flange of the diaphragm, midway between beams 2 and 3. All gages were type SR-4 wire gages.

Results

Traffic Data

The test ran continuously at the Route 95 site from 10:00 a.m. on Friday, September 6, 1968 to 11:29 p.m. on Tuesday, September 10, a period of 105 hours. Approximately 6,906 trucks crossed the bridge during 98 sampling periods, and complete records were obtained at the weighing station for 5,916 of these vehicles, approximately 85% of those reported at the structure.

Histograms showing the percentage of trucks in each of eight weight ranges are presented in Figure 8 for each of the three traffic lanes and for the bridge as a whole. As indicated by Figure 8, more than three-fourths of the trucks crossed the structure in the right-hand traffic lane (lane 1), while less than 1% used the left-hand passing lane. The majority of the trucks are more or less evenly distributed by weight between 20 and 70 kips, probably because of the large number of local delivery vehicles in the urban setting.

It is obvious that some overloaded vehicles may have left the highway after crossing the bridge in order to avoid the weighing station, but it is not believed that there were many severely overloaded vehicles. The weight limitations in Virginia are well enforced through both permanent weighing stations and mobile units, and the percentage of overweight vehicles in the vehicles weighed by all units has been one-half of one percent or less since 1956.[4]

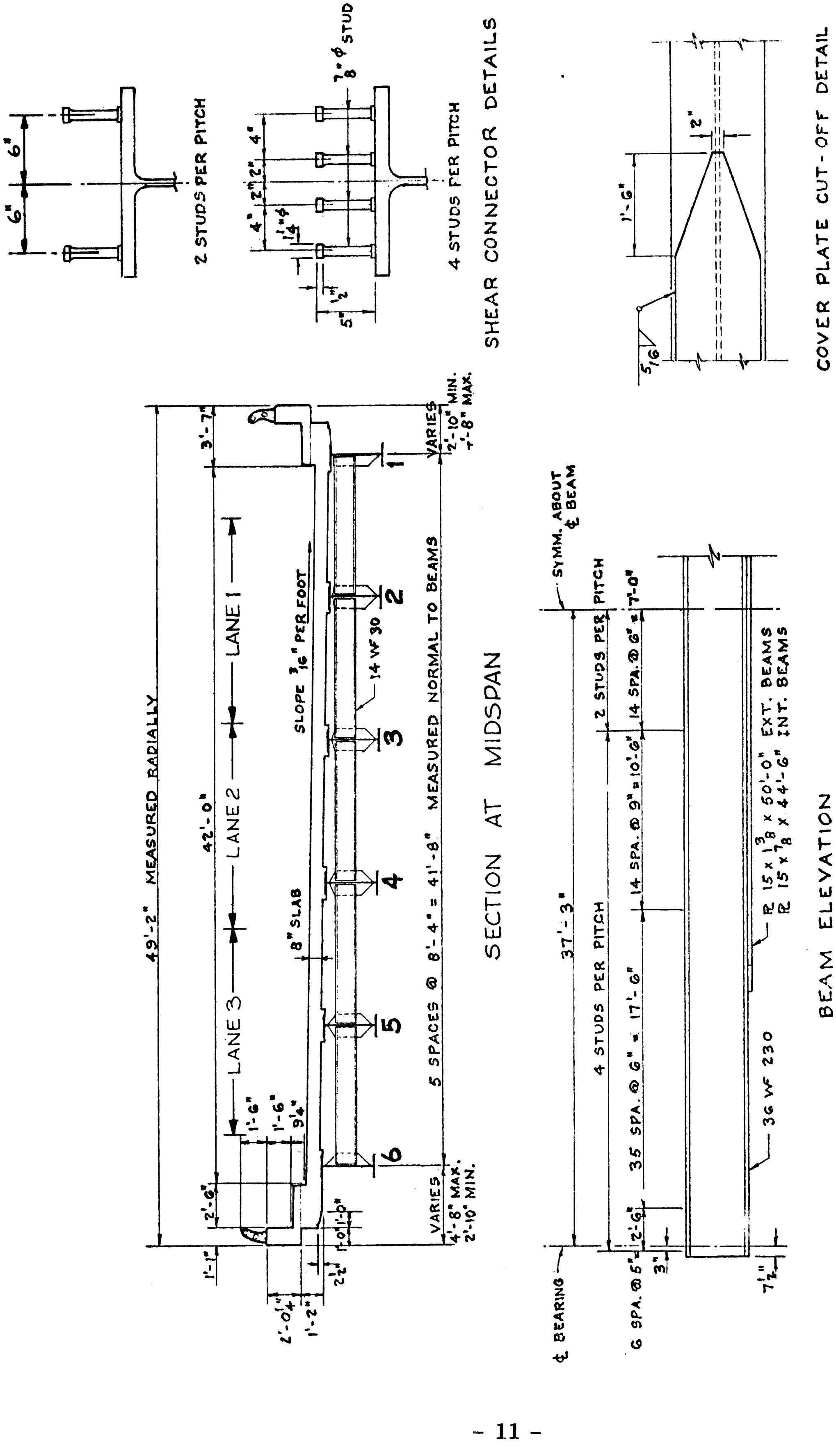
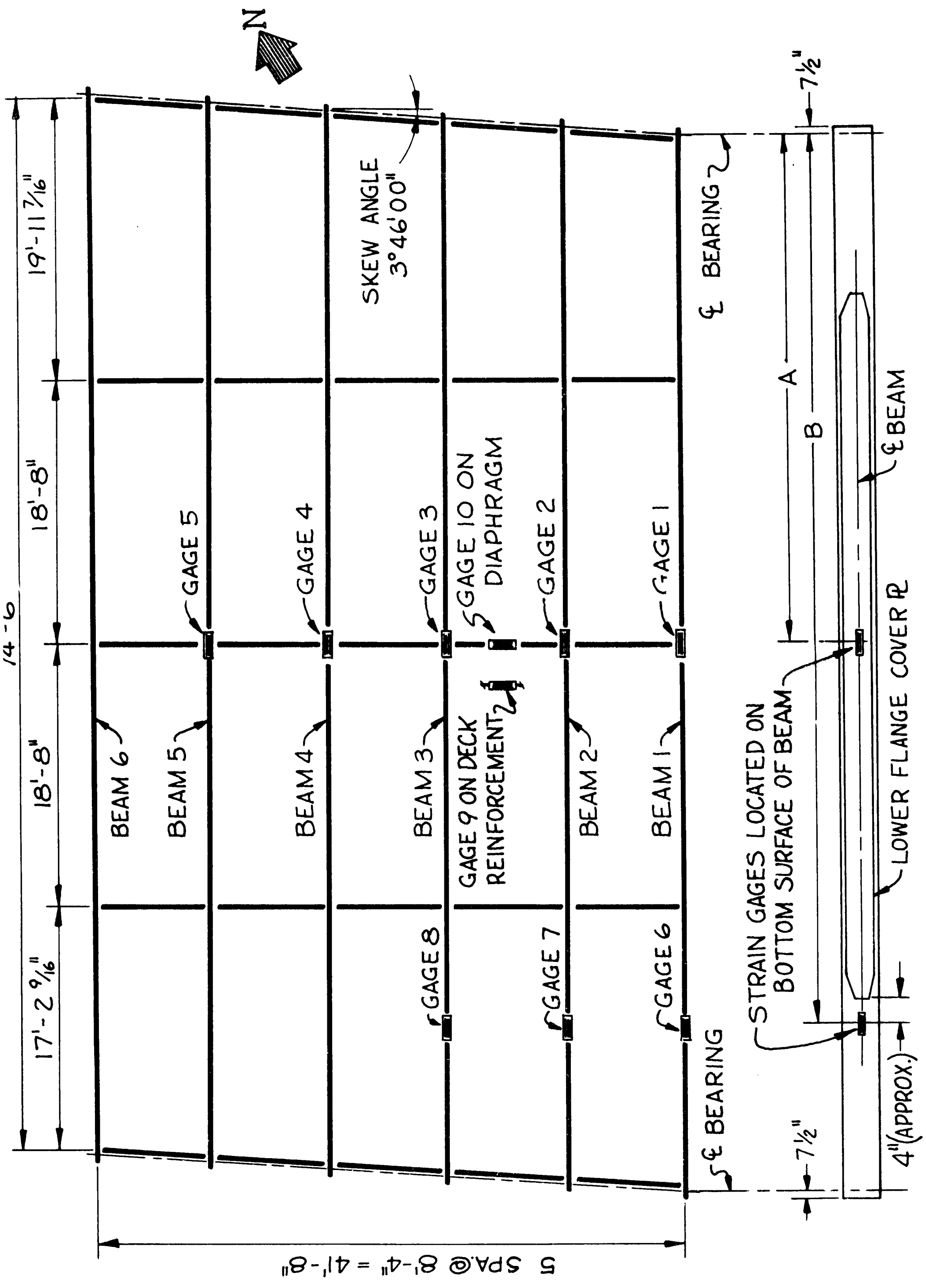


Figure 6. Details of the instrumented span on the Route 95 bridge. (From Reference 1, Figure 6.)



DIMENSION	BEAM 1	BEAM 2	BEAM 3	BEAM 4	BEAM 5	BEAM 6
A	35.9'	36.5'	37.0'	37.6'	38.1'	NOT GAGED
B	62.6'	59.8'	59.9'	—	—	

Figure 7. Gage locations, Route 95 bridge.

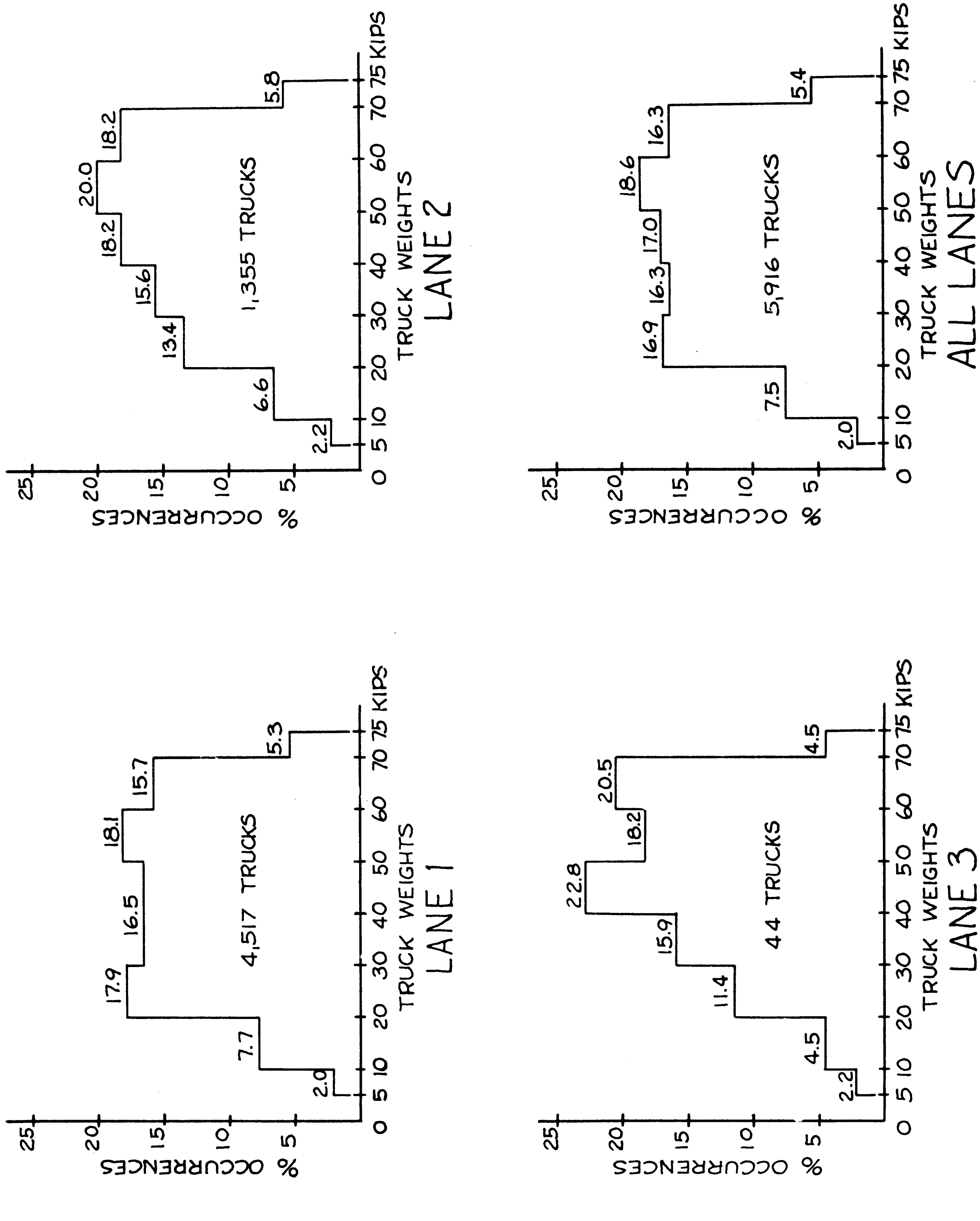


Figure 8. Truck weight histograms based on weighing station data, Route 95 bridge.

The Virginia Code limits the single axle weight of any vehicle or combination to no more than 18,000 pounds and the tandem axle weight to no more than 32,000 pounds, with no one axle of the unit exceeding 18,000 pounds. The allowable gross weight is limited by the distance between the extremes of all axles under the vehicle or combination, and the maximum gross weight is 70,000 pounds for a vehicle having a length of 42 feet between the extremes of its axles.[5] Conventionally, though not by law, a 5% tolerance is allowed. The weighing station data obtained during the subject study reflect the requirements of the code; no vehicles larger than 75,000 pounds in gross weight were recorded.

Stress Data

The primary purpose of this experimental study was to determine the stress ranges produced by service loadings at selected points on the instrumented span. The strain ranges given in the printouts, Figure 2, from the data acquisition system were combined and tabulated for each of the ten gages shown in Figure 7 and converted to stress assuming 30×10^6 psi as the modulus of elasticity of steel. The combined data are shown in Table II, and histograms showing the percent occurrence of events in each strain range for each gage except number 6 follow in Figures 9-17. The data for gage number 6, which behaved erratically throughout the tests, were discarded. The minimum test level was set at a strain of 15 micro-inches per inch (450 psi stress) for gages 1-7 and 40 micro-inches per inch (1,200 psi) for gages 8-10.

The strain ranges recorded in the beams were fairly low, but there is no reason to doubt their accuracy. The data from the various gages are consistent, and the measured strains are similar to those obtained in an earlier pilot study by the FHWA.[1]

The highest stress ranges recorded at a midspan gage were in beam 4, represented by gage number 4, which experienced two occurrences of ranges between 3,150 and 3,600 psi. A single occurrence of a range between 3,600 and 4,200 psi was also recorded at gage number 8, which was located four inches beyond the end of the lower flange cover plate on beam 3. The greatest number of stress ranges between 1,350 and 2,700 psi occurred in beams 2 and 3, while beam 1 experienced the greatest number of low stress ranges.

Beams 2 and 3 had the greatest number of relatively high stress ranges because, as indicated by the typical section in Figure 6, they are the supporting elements beneath the right-hand lane, in which most of the trucks crossed the structure. As shown in Figure 6, beam 1 is under the curb at the edge of the roadway. Similarly beams 3 and 4 are the supporting elements for the center lane, which carries fewer trucks, and beams 5 and 6 are primarily loaded by trucks in the seldom used left-hand lane. Although beams 4 and 5 experienced more stress ranges above 2,700 psi, the much greater number of loadings between 1,350 and 2,700 psi in beams 2 and 3 would have more influence on the service life of the structure.

TABLE II

TOTAL NUMBER OF OCCURRENCES AT EACH STRAIN LEVEL, ROUTE 95 BRIDGE
(Note: Position of Gages is shown in Figure 7.)

Strain Level	Strain, μ, in./in.	Stress, psi	Gage Number 1	2	3	4	5	6	7	Strain μ, in./in.	Stress, psi	Gage Number 8	9	10
0								Data discarded due to erratic gage behavior.						
			0	0	0	0	0		0			0	4	0
1	135	4,050								200	6,000			
			0	0	0	0	0		0			0	3	0
2	120	3,600								180	5,400			
			0	0	0	2	0		0			0	13	1
3	105	3,150								160	4,800			
			0	2	7	10	8		2			0	61	2
4	90	2,700								140	4,200			
			1	81	82	39	15		42			1	316	10
5	75	2,250								120	3,600			
			58	809	548	227	31		642			7	718	90
6	60	1,800								100	3,000			
			905	1,985	2,054	714	106		1,993			55	1,169	803
7	45	1,350								80	2,400			
			4,646	3,669	3,914	3,167	1,073		3,874			703	1,422	1,186
8	30	900								60	1,800			
			4,481	2,935	2,680	4,539	2,738		3,602			2,050	1,133	2,026
9	15	450								40	1,200			
	TOTAL		10,084	9,481	9,285	8,698	3,971		10,155	TOTAL		2,816	4,839	4,118

2303

2304

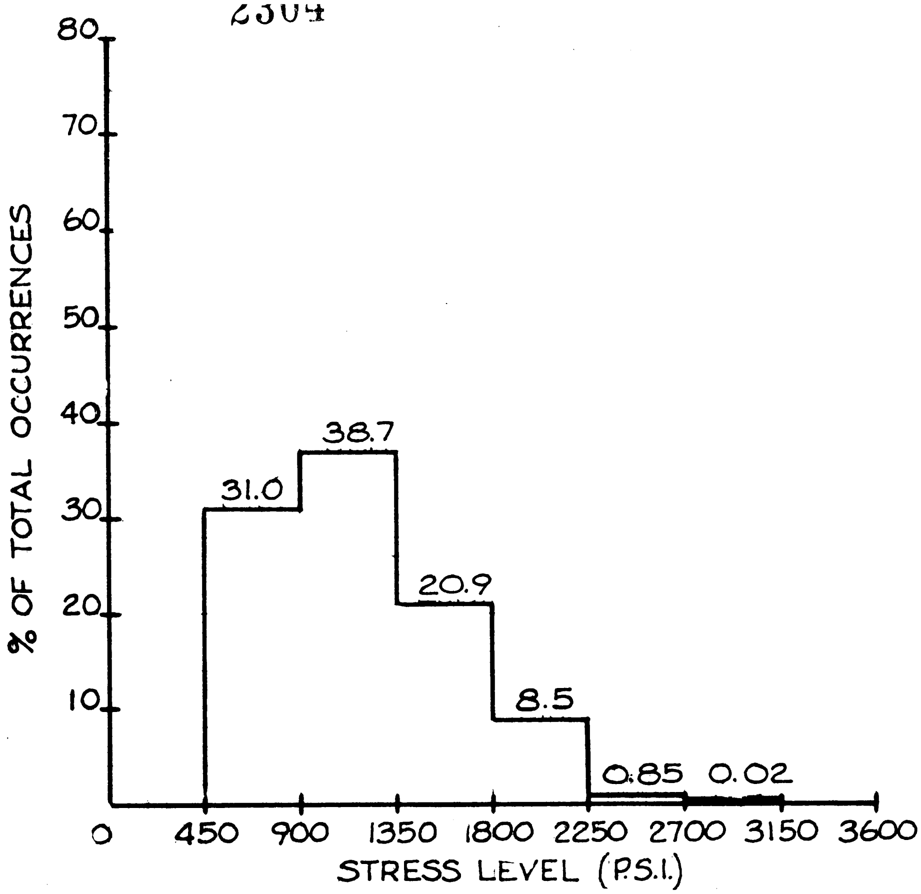


Figure 9. Stress histogram, gage #1, midspan beam 1.

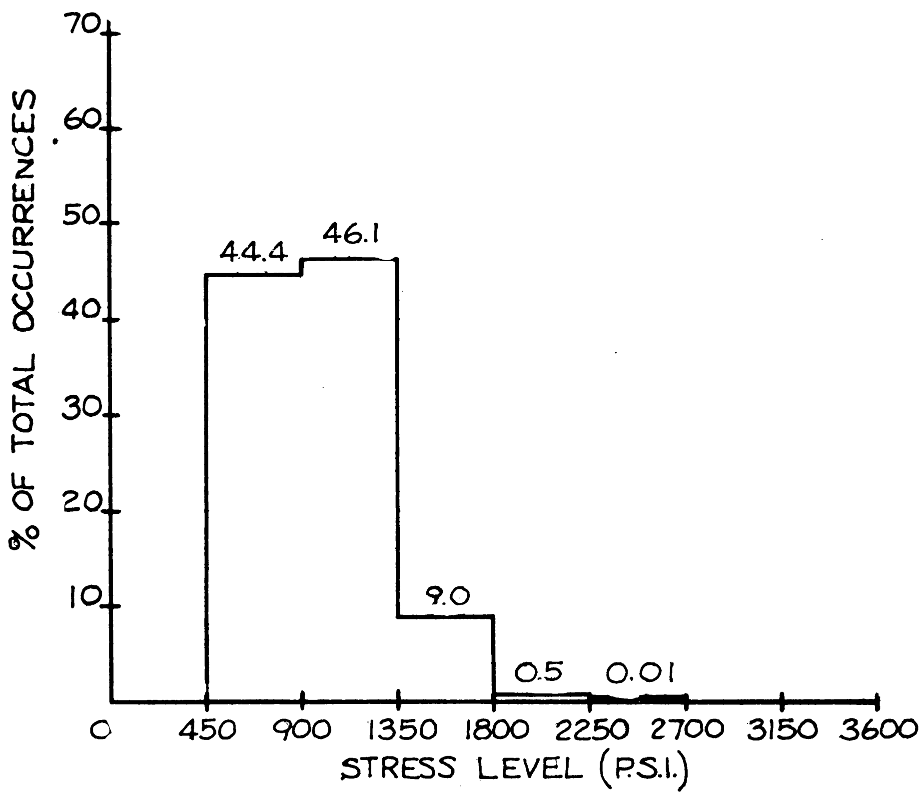


Figure 10. Stress histogram, gage #2, midspan beam 2.

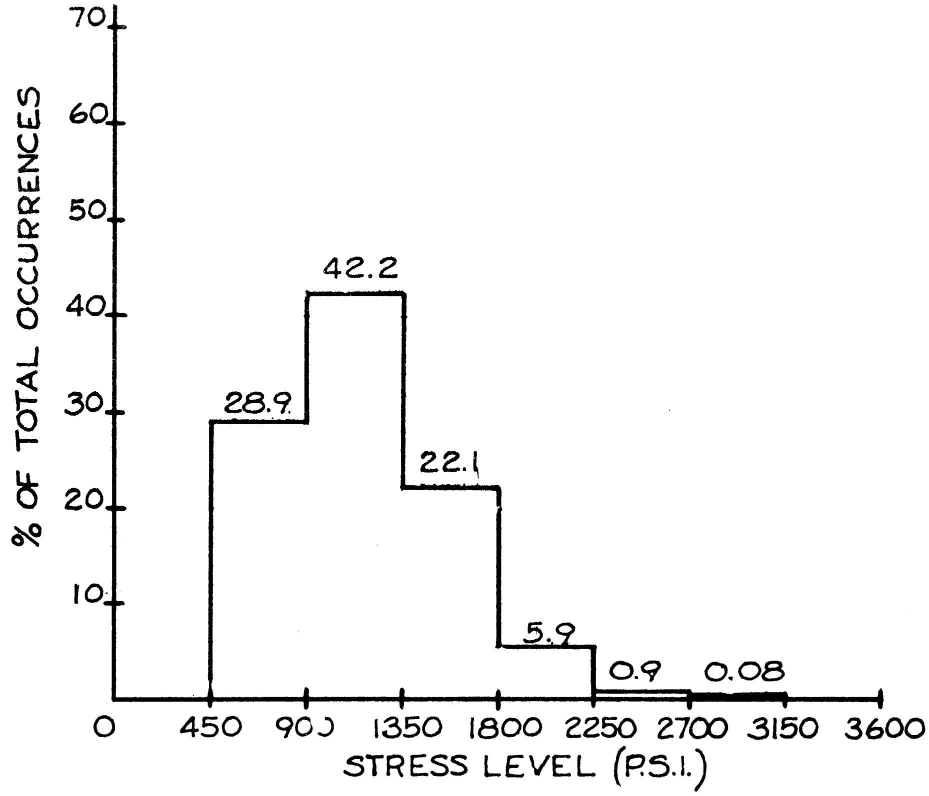


Figure 11. Stress histogram, gage #3, midspan beam 3.

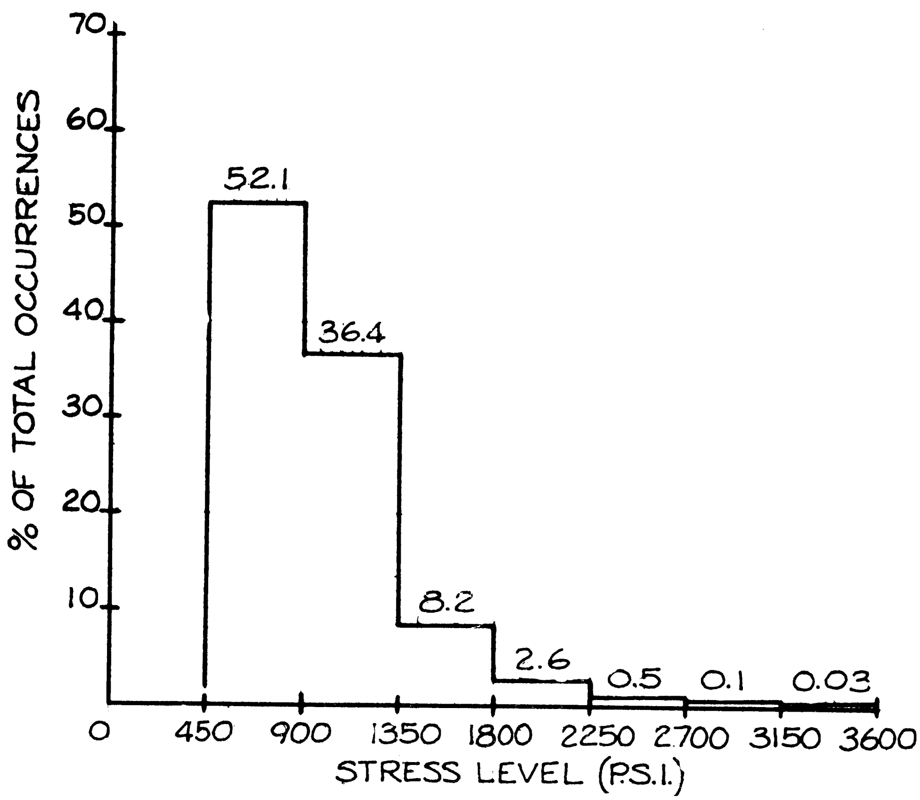


Figure 12. Stress histogram, gage #4, midspan beam 4.

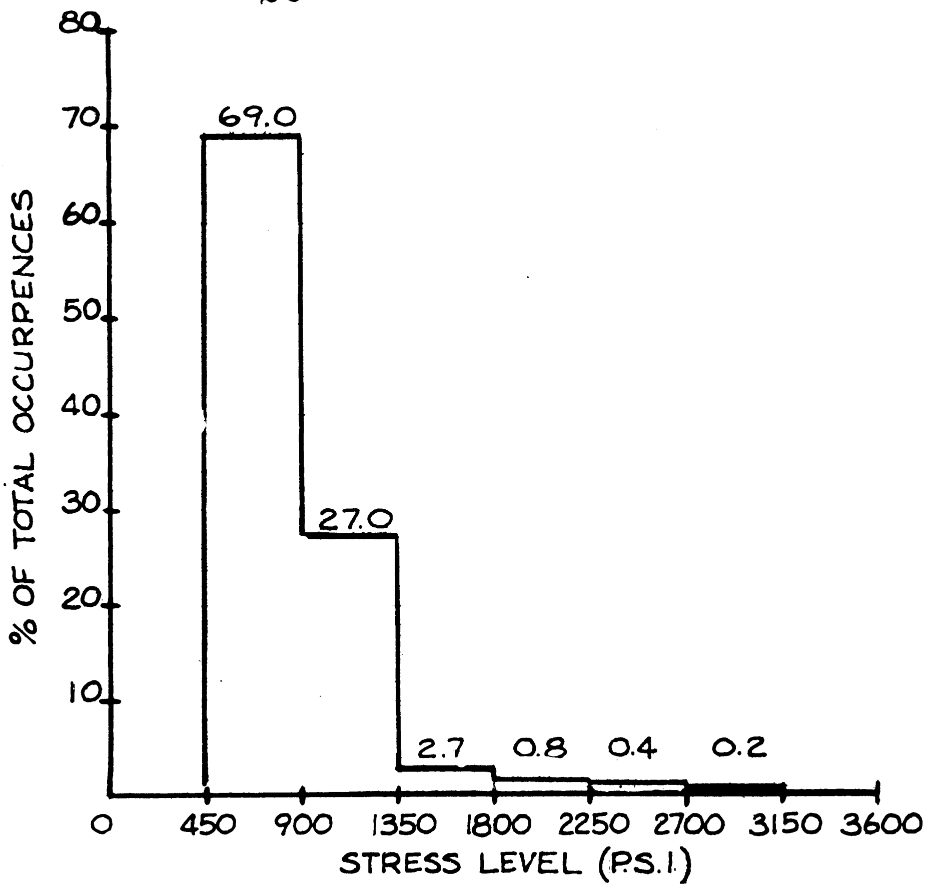


Figure 13. Stress histogram, gage #5, midspan beam 5.

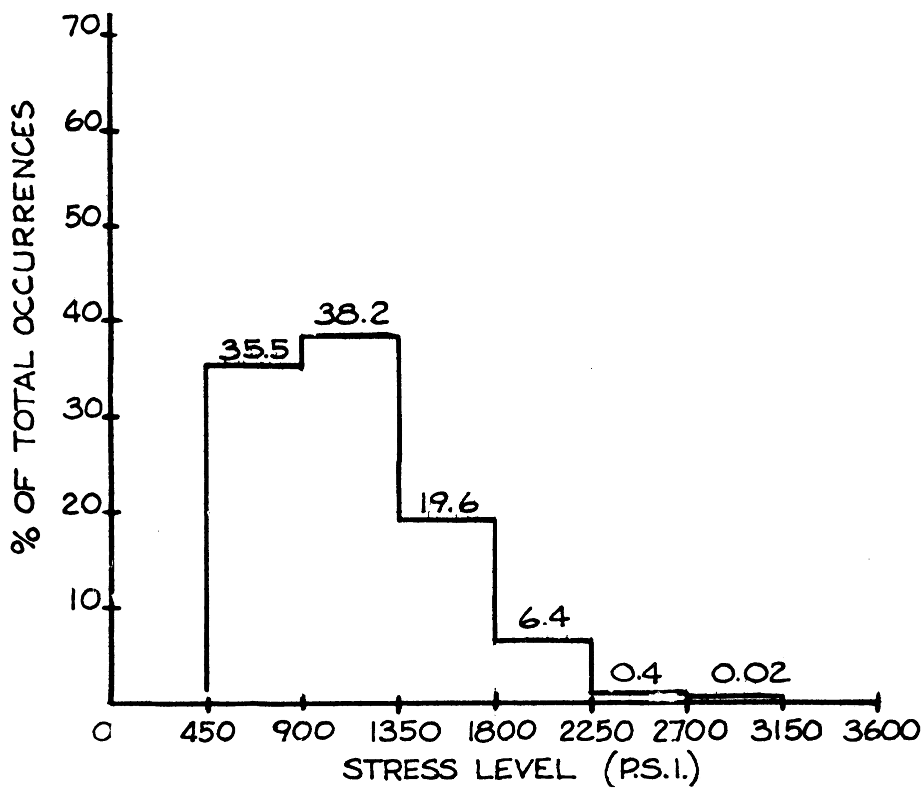


Figure 14. Stress histogram, gage #7, end of cover plate, beam 2.

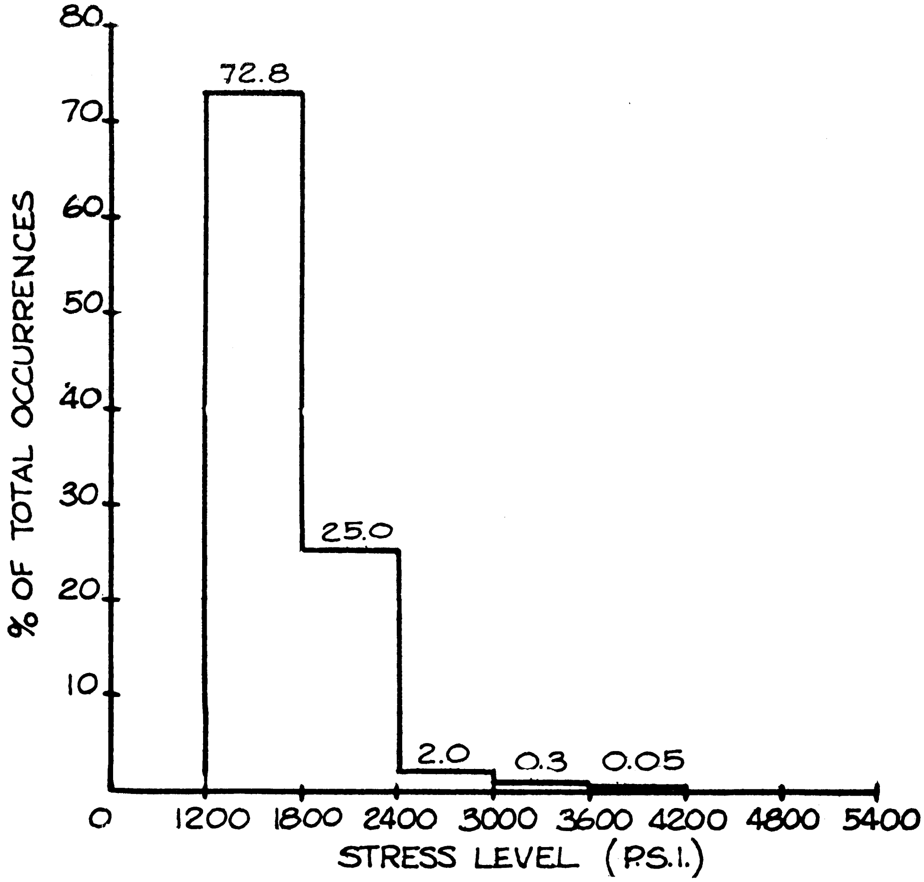


Figure 15. Stress histogram, gage #8, end of cover plate, beam 3.

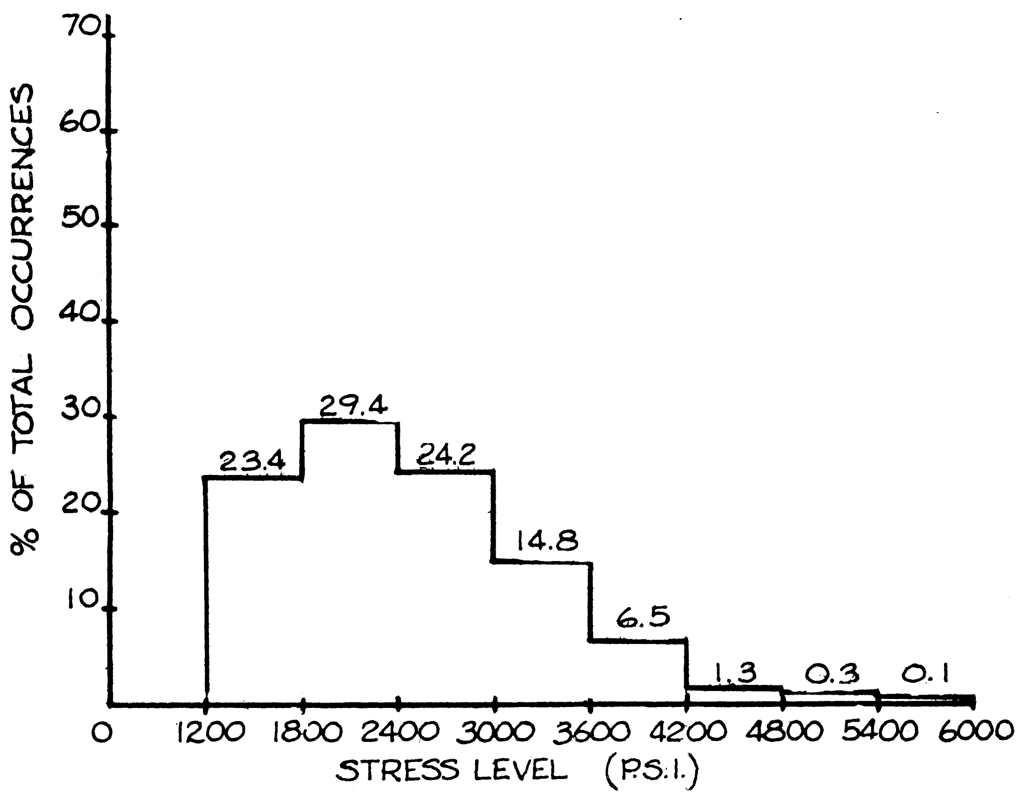


Figure 16. Stress histogram, gage #9, deck reinforcement.

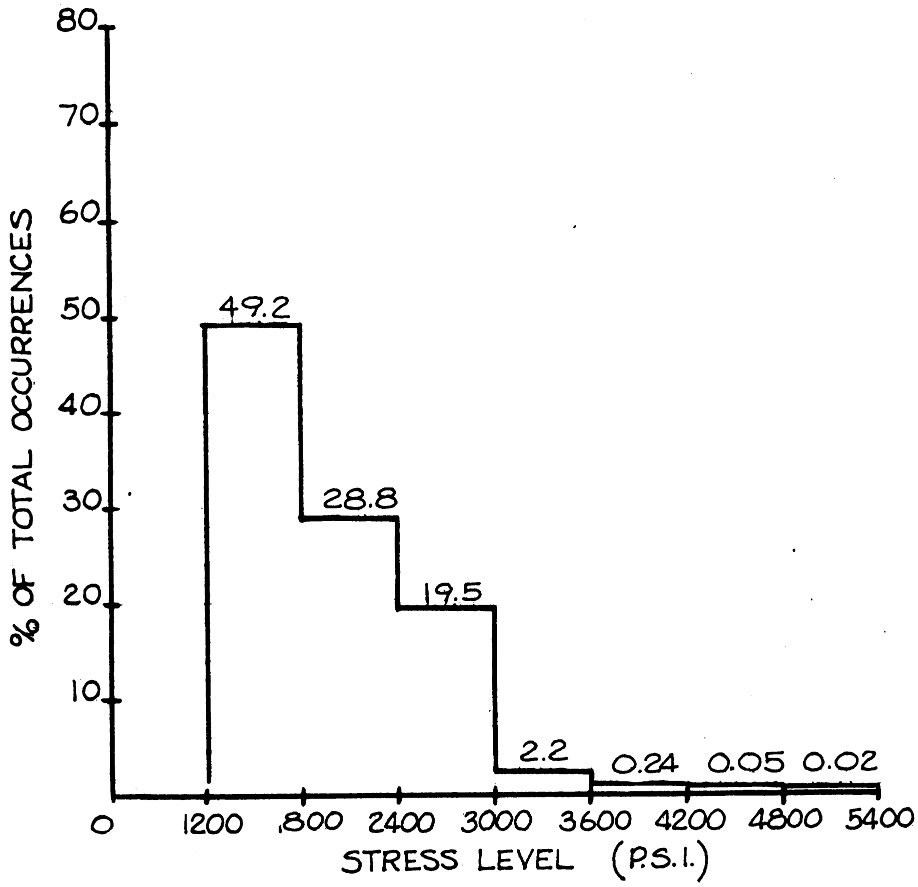


Figure 17. Stress histogram, gage #10, center of diaphragm.

Fortunately, the service life of the beams is not critical. The vast majority of the stress ranges to which the girders were subjected under loading by the approximately 6,906 trucks crossing the structure during the tests were below 2,250 psi. Very few stress ranges greater than approximately 3,000 psi were recorded, and it is believed that the girders can serve adequately under an increased volume of truck traffic under the current weight limitations.

It is interesting to note that several occurrences of stress ranges above 3,000 psi were recorded at gage number 10 on the diaphragm between beams 2 and 3. Relatively little is known about the loadings to which diaphragms are subjected, but it is apparent that the stress in these members is comparable to, and in some instances greater than, the stress in the girders. The highest stress ranges encountered in this study occurred in the transverse deck reinforcing bar, gage number 9. It is possible that fatigue considerations are important in such a case.

The data from the Route 95 tests indicate that secondary strain ranges of measurable magnitude were excited in the relatively flexible steel beam composite span. The total number of strain range occurrences, shown on the bottom line of Table II, indicate that the 6,909 trucks crossing the structure caused an average of approximately 1.3 strain events above 450 psi per vehicle in beams 2 and 3 and nearly 1.5 per vehicle in beam 1. The data acquisition system, as utilized in the Virginia tests, did not allow a determination of the magnitude of the secondary strain ranges, but additional research of this nature would be worthwhile. The effect of secondary strain ranges on service life must be considered if a theoretical approach, such as that described in the following section, is attempted.

Theoretical Correlation Study

The objective of this phase of the study was to provide a theoretical basis for predicting the stresses in a highway bridge under service loadings. This was desirable, first, to provide verification of the stresses based on the experimentally determined strain ranges and, second, to provide a method which would allow an engineer with a knowledge of the truck traffic characteristics on a given route to evaluate the loading history and life expectancy of a structure without the use of costly strain monitoring equipment. Such a procedure would be of significant value in the design and maintenance of highway bridges, and it is a major objective of the nationwide program of loading history studies.

Unfortunately, the development of a theoretical correlation is not without difficulties. The determination of a proper impact factor — which varies widely with the roughness of the approach surface, vehicle suspension characteristics, and the state of oscillation of the vehicle and the span — is difficult, as is an accurate assessment of the effect of the secondary stress ranges associated with dynamic loading. Field conditions also present practical limitations on the accuracy of the basic data, and a perfect correlation is not to be expected. The theoretical phase of this study is based on an accepted method of analysis, which is as simple as conditions permit. Further refinement is necessary, but the methodology, which will be described only briefly, is considered a sound approach by the authors.

In the analytical approach, beam theory was used to develop an algorithm for the magnitude of the maximum moment at midspan, which occurs in a simple span when the middle axle of a three axle truck is over the center of the span. Only one strain event per vehicle was assumed, and the effect of impact due to dynamic loading was not considered. The individual truck data obtained at the weighing station plus the lane in which the vehicle crossed the structure composed the loading input. The maximum moments caused by all trucks were distributed to the beams on the basis of factors computed for a representative vehicle, a three axle truck with axle spacings of 10 and 30 feet and weights of 10 kips on the front axle and 30 kips on the middle and rear axles, at the position of maximum moment in the right-hand and center lanes. The very few trucks in the left-hand lane were ignored.

Classical finite difference theory was used to determine the distribution factors. The deflection of the deck under any loading condition was expressed in terms of a symmetrical array of 42 points equally spaced at seven points along each of the six beams. The 42 linear, independent, simultaneous equations thus generated were solved by means of a standard library computer program.

The basis for the distribution factor analysis was an evaluation of the transverse and longitudinal stiffnesses, assuming no torsional restraint at the 42 points. This assumption of pinned intersection points may be inaccurate in the case of heavily skewed bridges, but it was considered acceptable in the case of the slightly skewed Route 95 structure. A typical transverse beam was taken as a rectangular section of concrete having a thickness equal to that of the deck and a width equal to the distance between the intersection points; the stiffness of the W 14 x 30 diaphragms was ignored. A longitudinal section had the properties of the composite section at the point under consideration. The beams were assumed to be hinged at the points of intersection to allow both rotational and translational independence except for continuity of vertical deflection. The conditions of compatability were identical to those of a stiffened plate.

The theoretical midspan stress levels based on that portion of the individual truck moments distributed to each of the beams are compared with the measured midspan stress levels for 10 selected hours in Figures 18-27, a series of histograms in which the number of occurrences at each stress level is plotted for the five instrumented beams. The theoretical results verify the accuracy of the experimental data, and, in view of the limitations cited previously, the agreement between theory and experiment is considered reasonable.

The comparison is made in a different form in Figures 28-37, in which the total number of occurrences of midspan stress ranges above the minimum test level of 450 psi is plotted for each of the six beams. It can be seen that the theoretical analysis consistently predicts fewer stress occurrences above 450 psi than were measured in the field, which indicates that refinements which would account for impact and secondary stress ranges due to dynamic loading are desirable. The often pronounced difference in the case of beam 1 is probably due to the assumption of too large a moment of inertia for the exterior member. The shapes of the curves are generally similar, however, and it is believed that the basic methodology — the use of finite difference theory to develop distribution factors — is a sound approach to attaining a correlation.

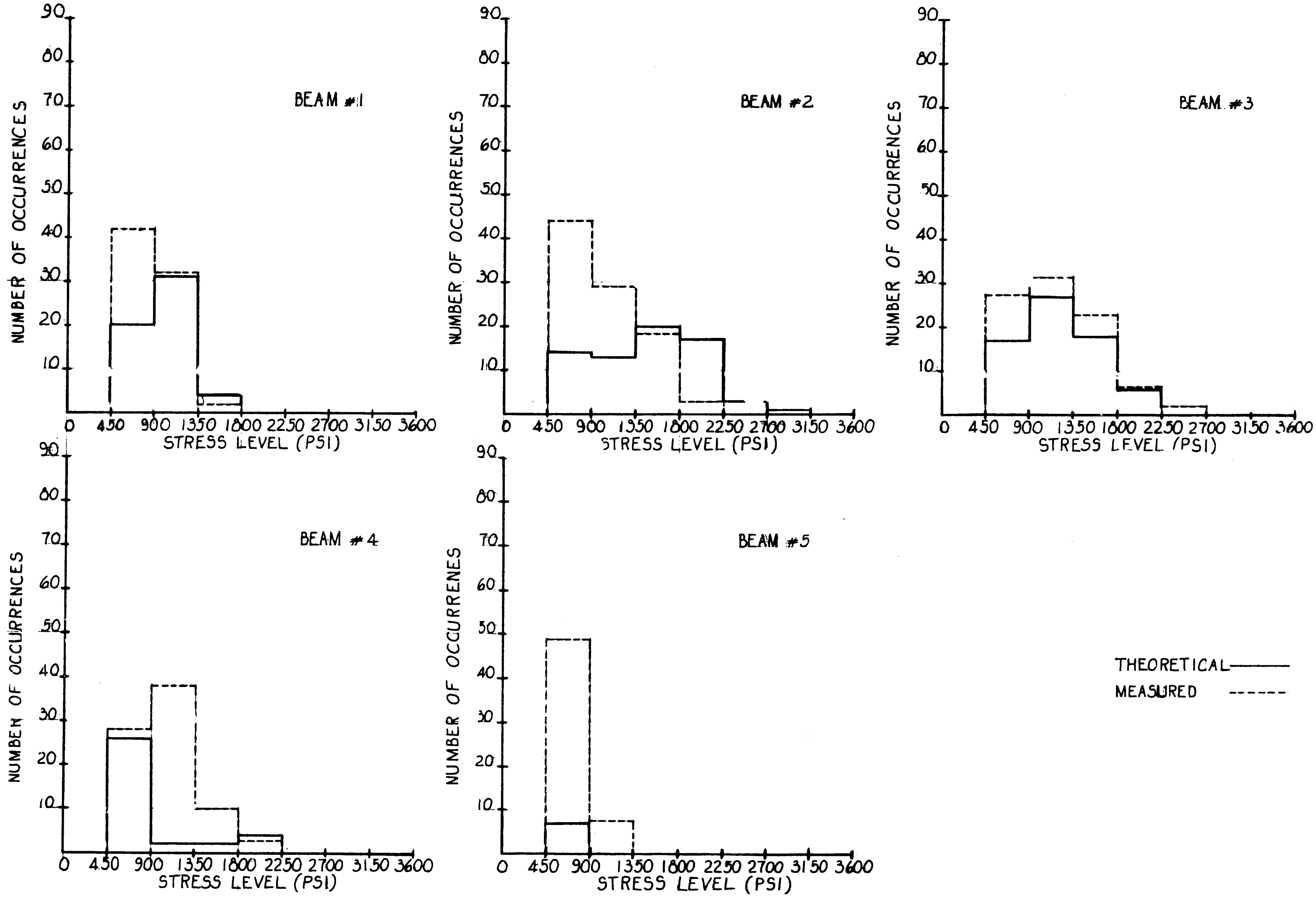


Figure 18. **Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 2157 hours 9/6/68.**

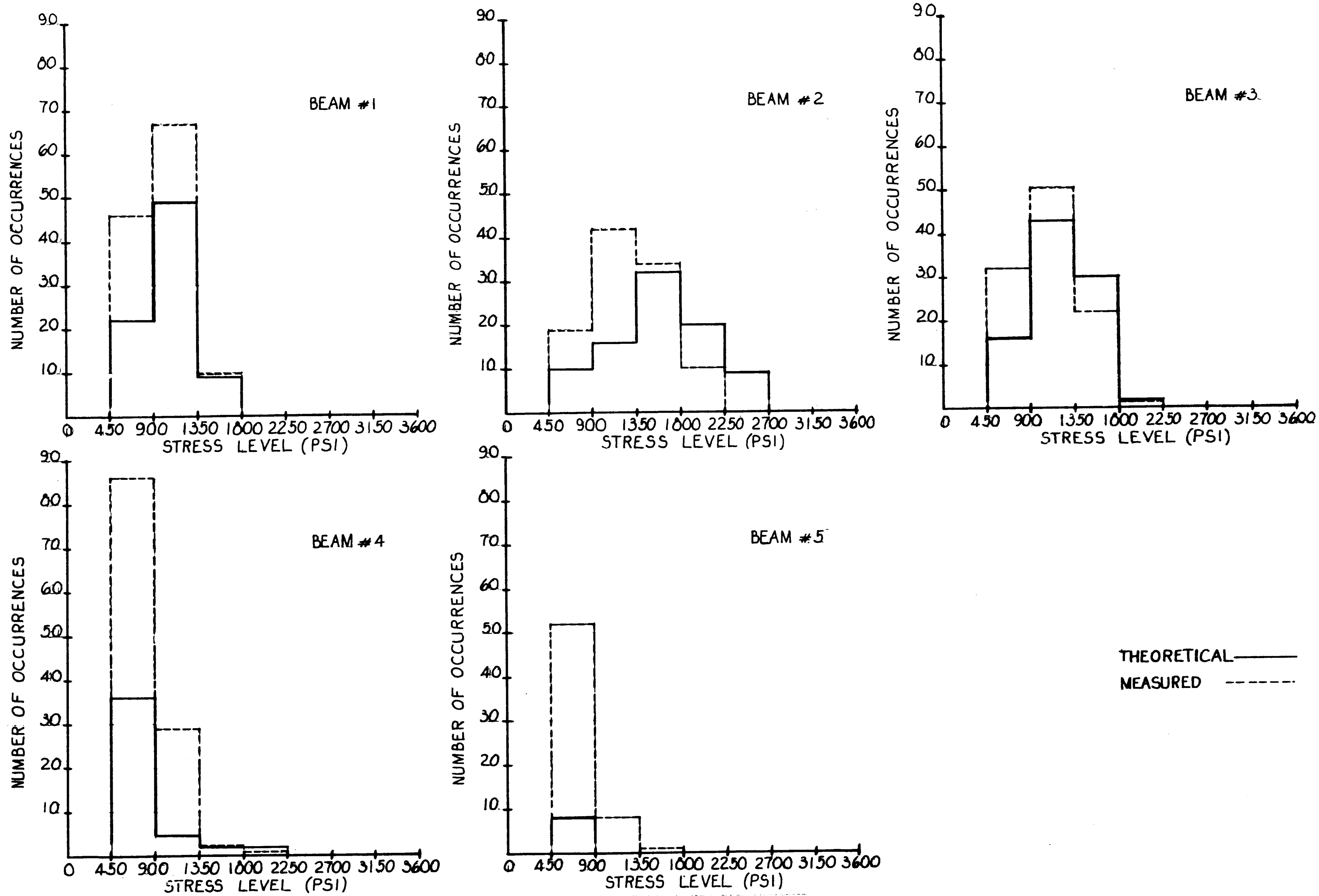


Figure 19. Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 1645 hours 9/8/68.

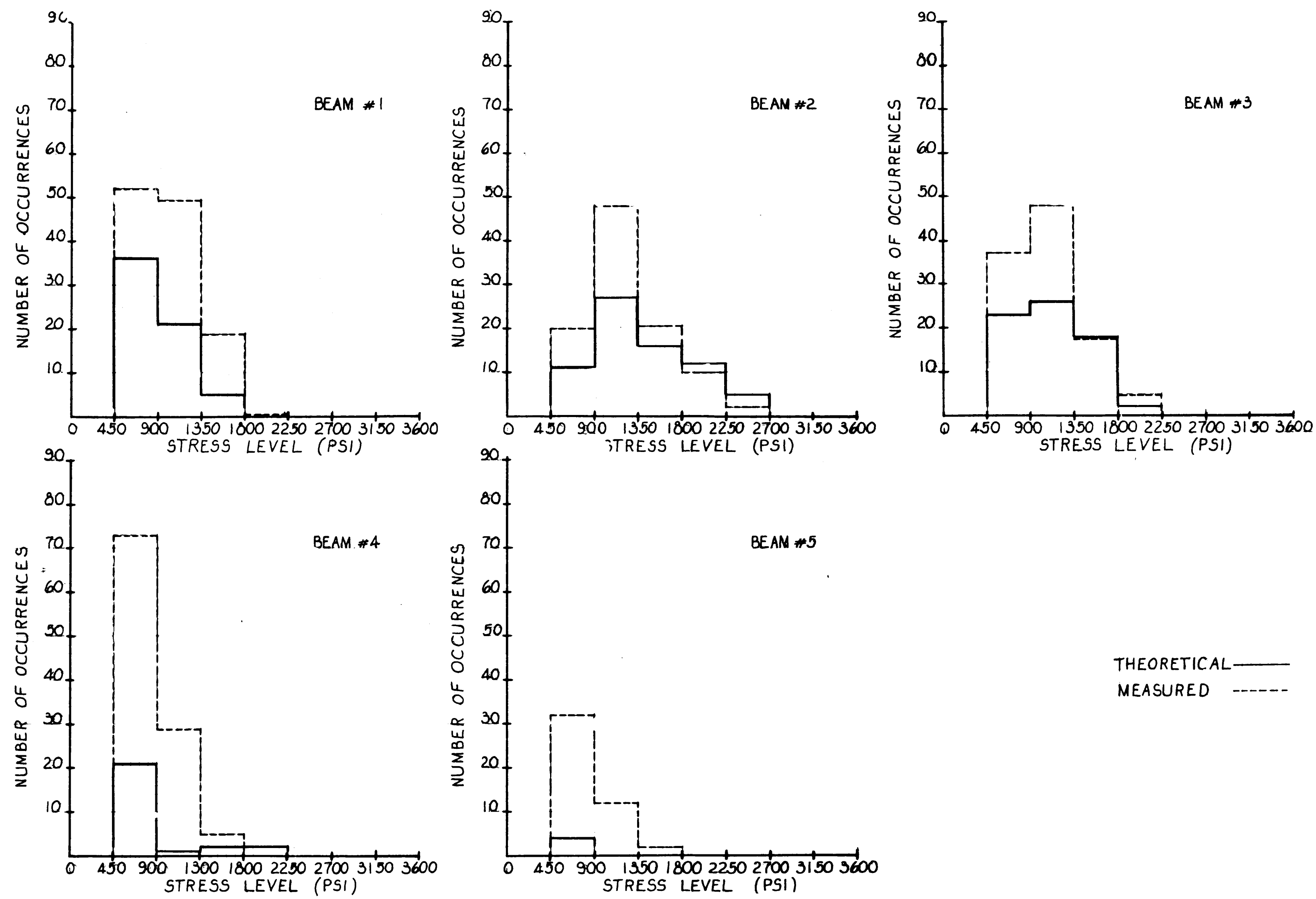


Figure 20. Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 0854 hours 9/9/68.

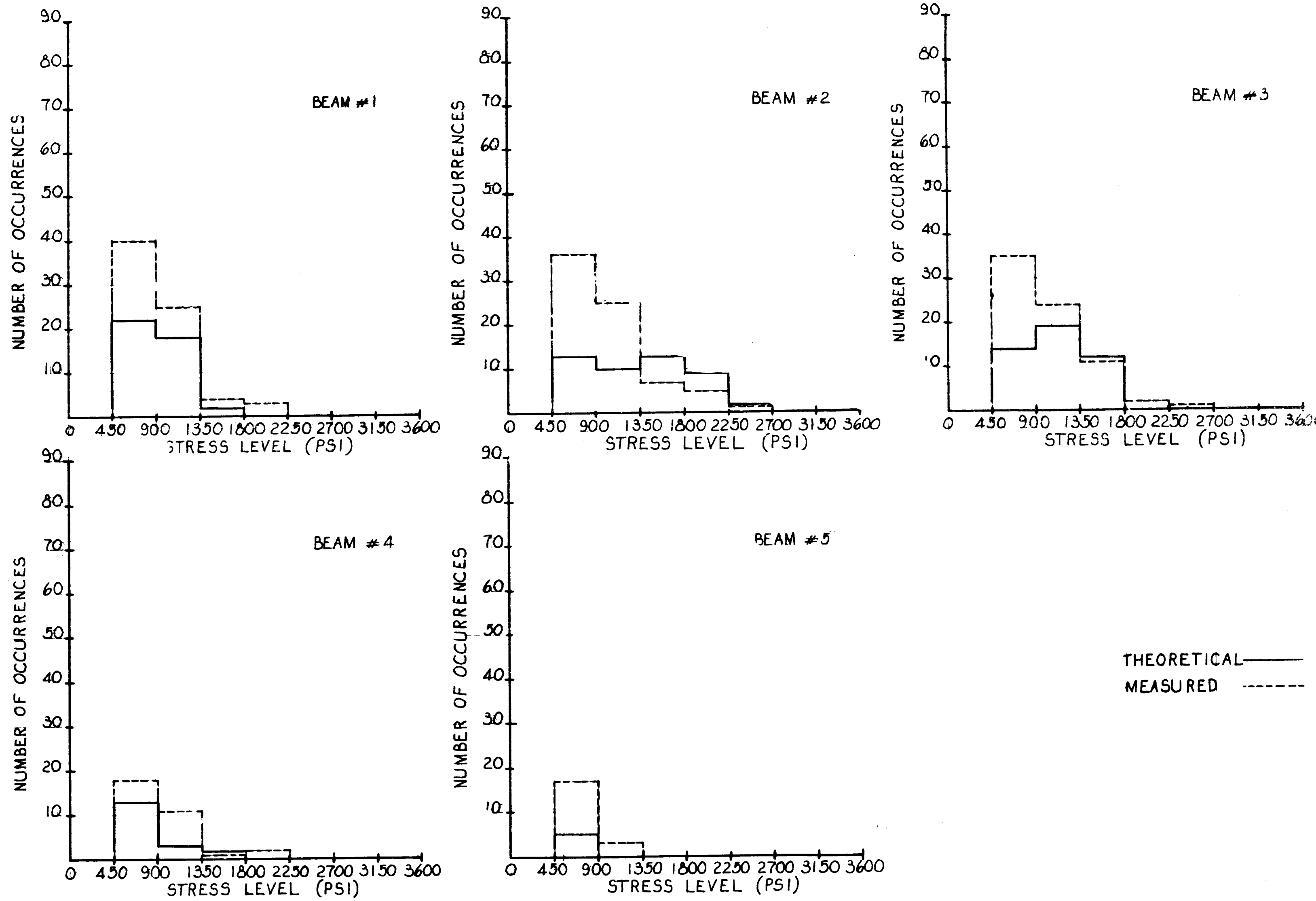


Figure 21. Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 0958 hours 9/9/68.

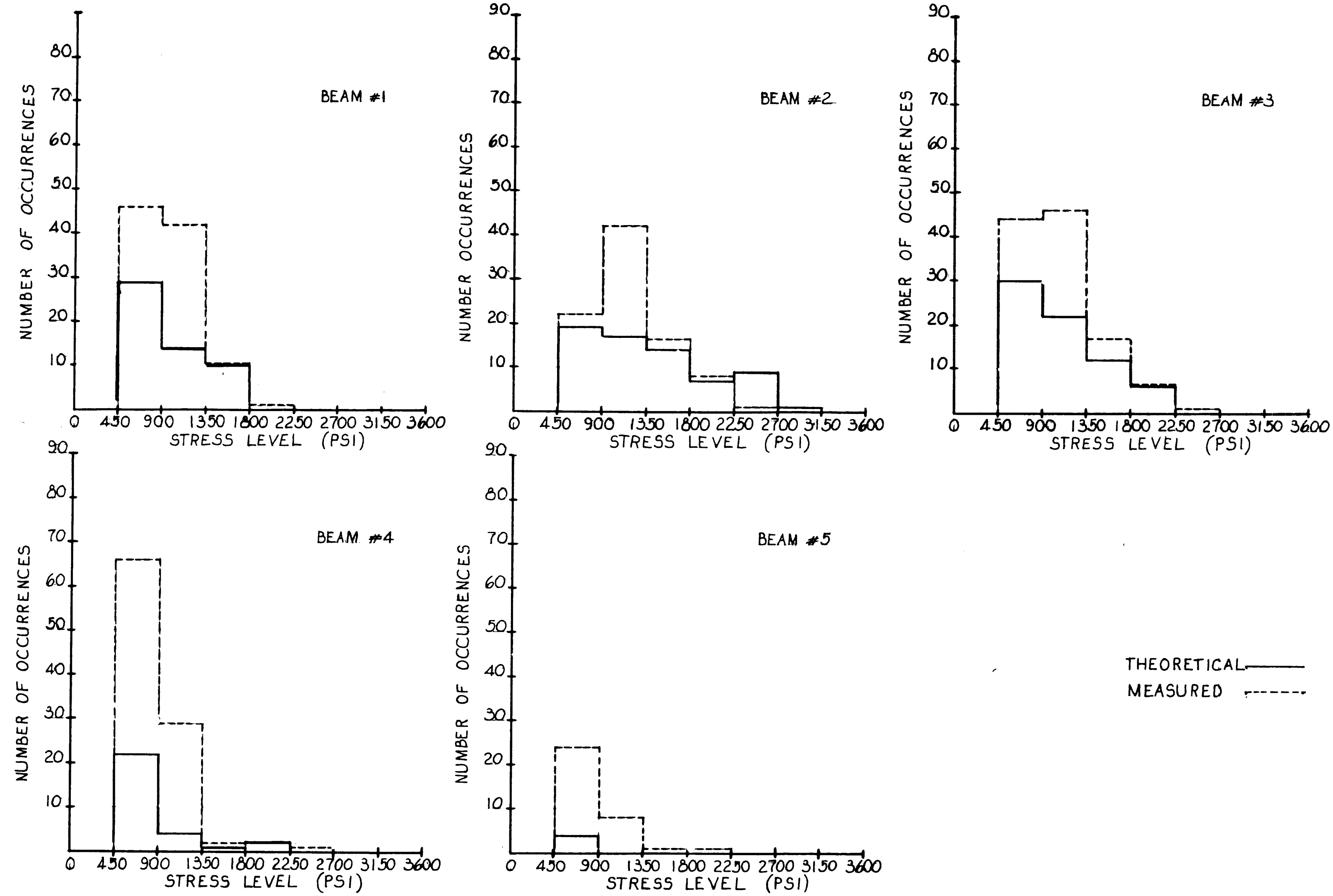


Figure 22. Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 1102 hours 9/9/68.

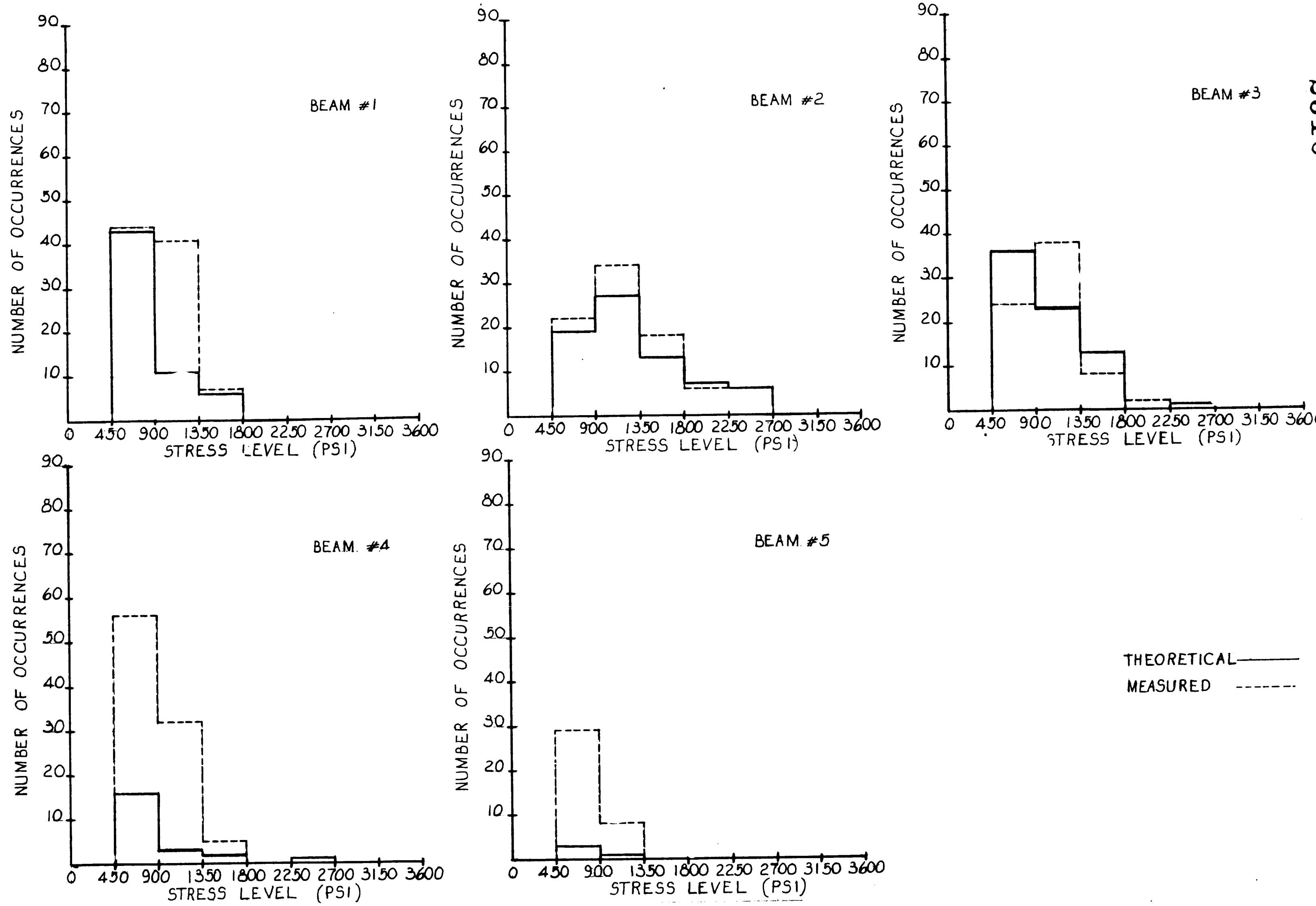


Figure 23. Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 1207 hours 9/9/68.

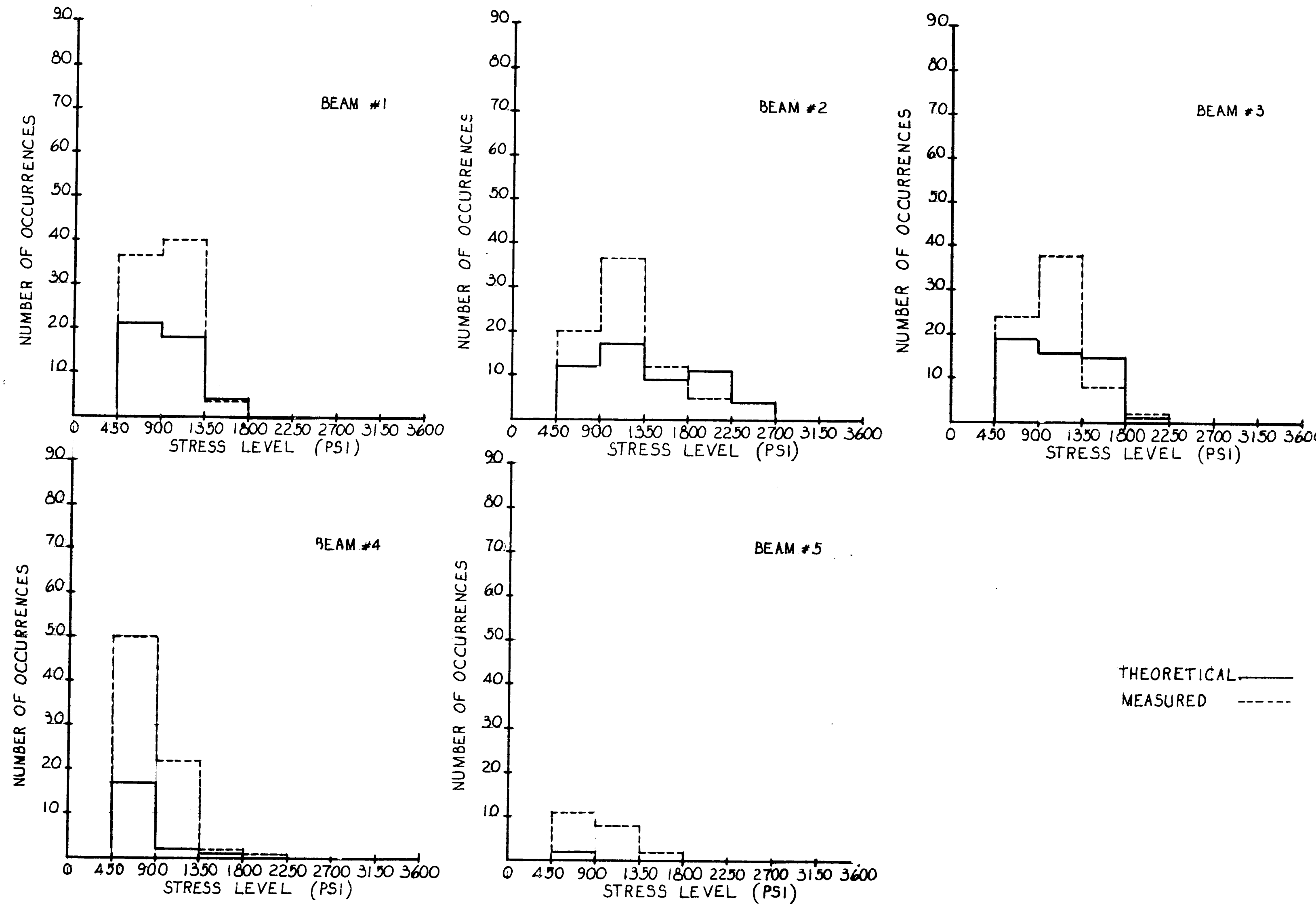


Figure 24. Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 1312 hours 9/9/68.

2317

2318

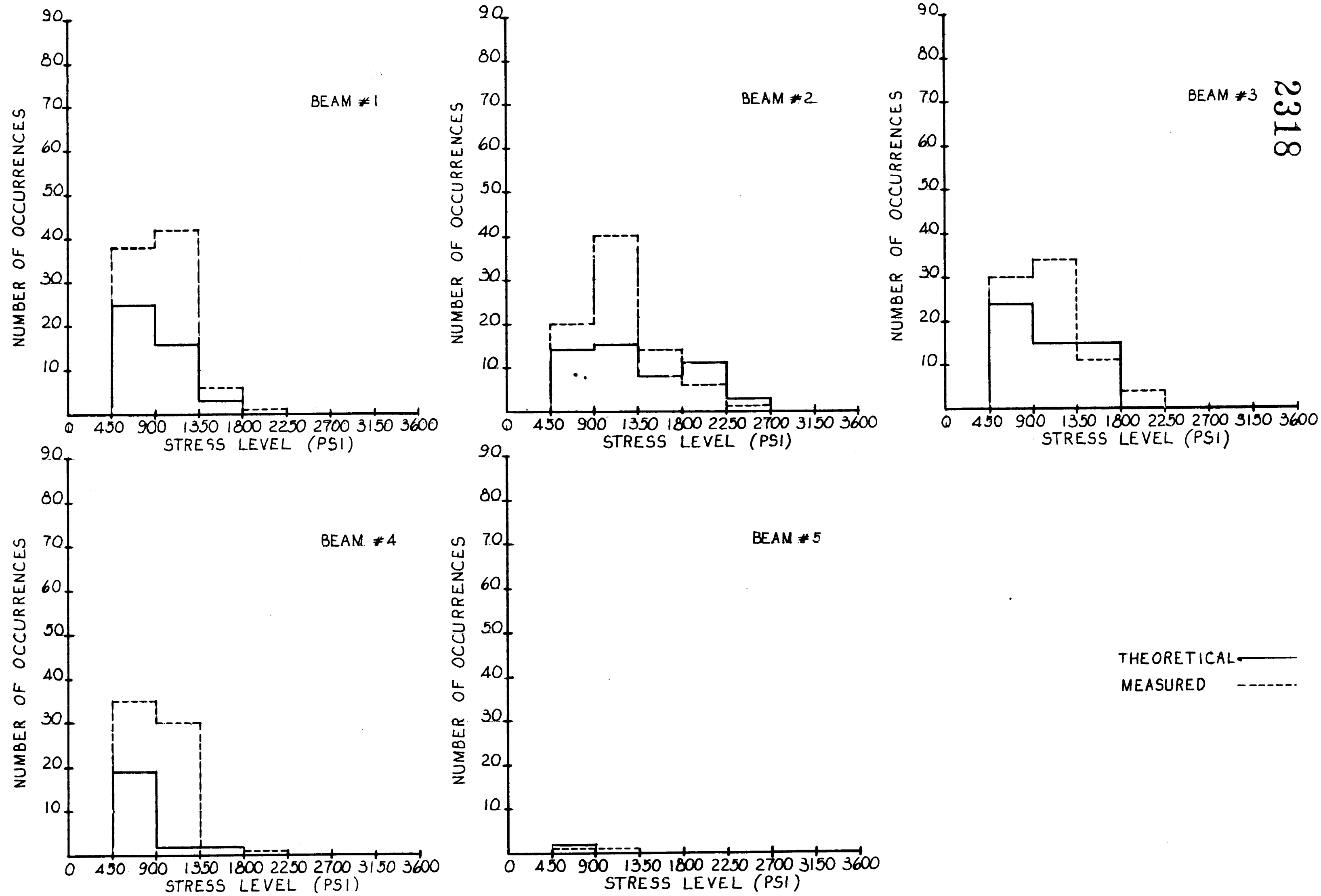


Figure 25. Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 1416 hours 9/9/68.

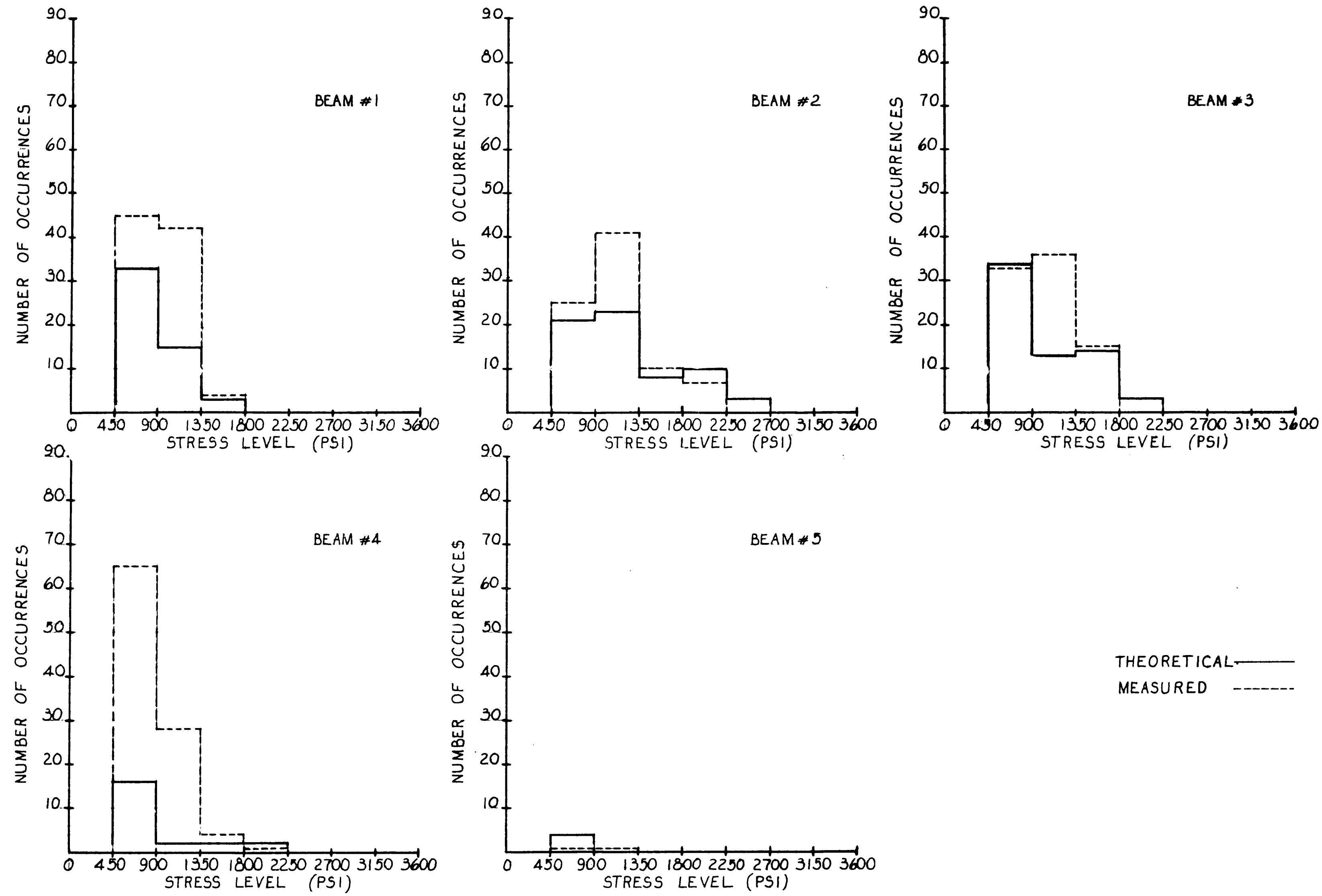


Figure 26. Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 1520 hours 9/9/68.

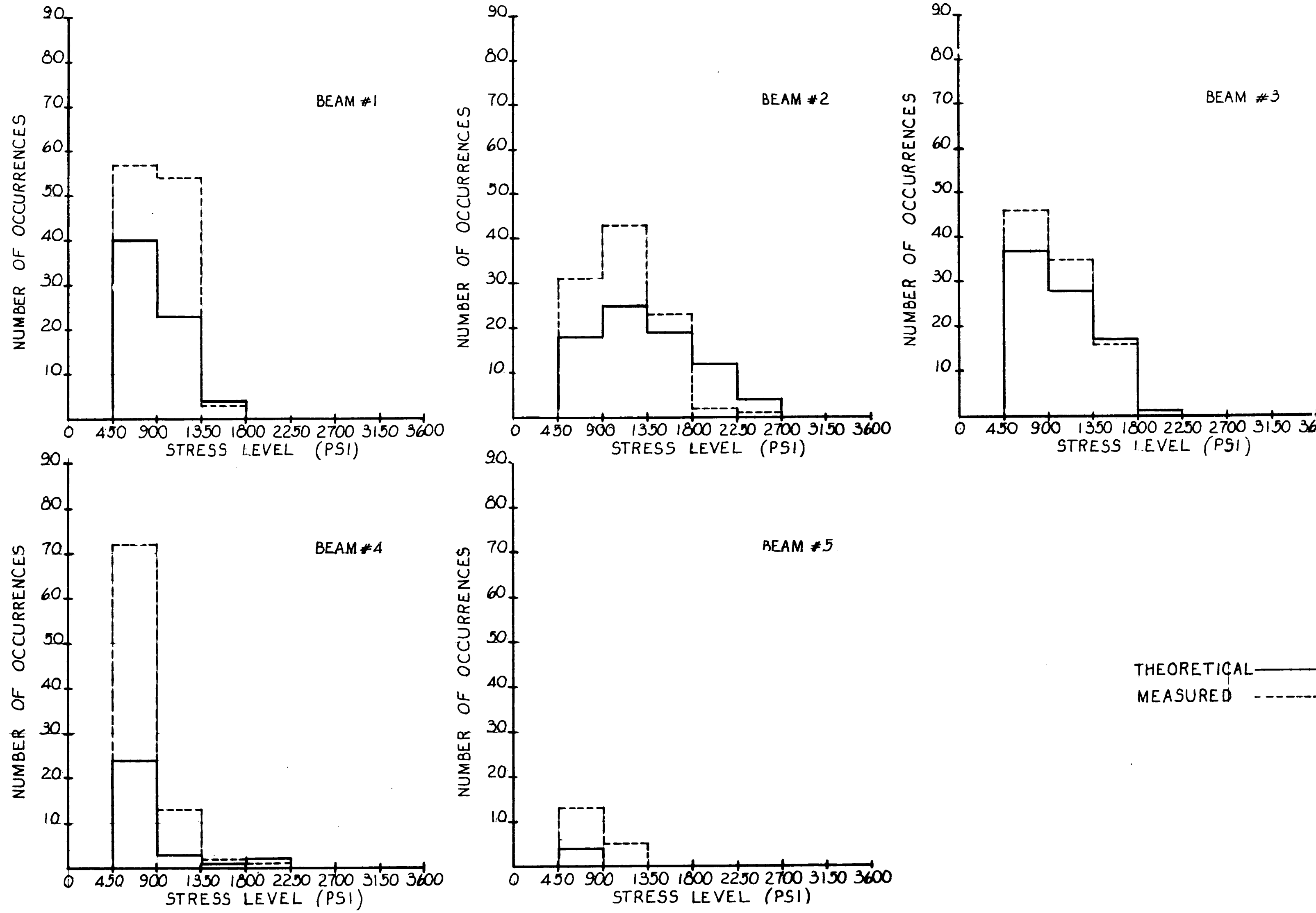


Figure 27. Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 1625 hours 9/9/68.

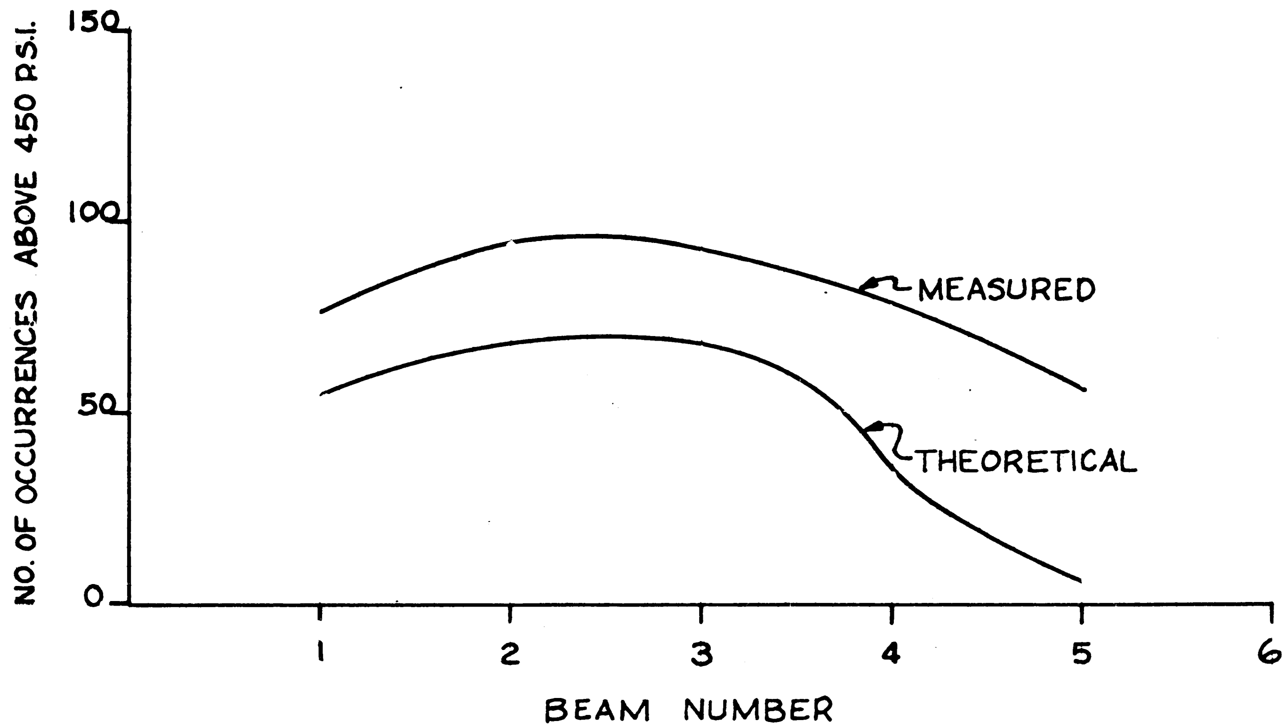


Figure 28. Comparison of theoretical and measured number of occurrences of midspan stress ranges over 450 psi, Rte. 95 bridge, 2157 hours, 9/6/68.

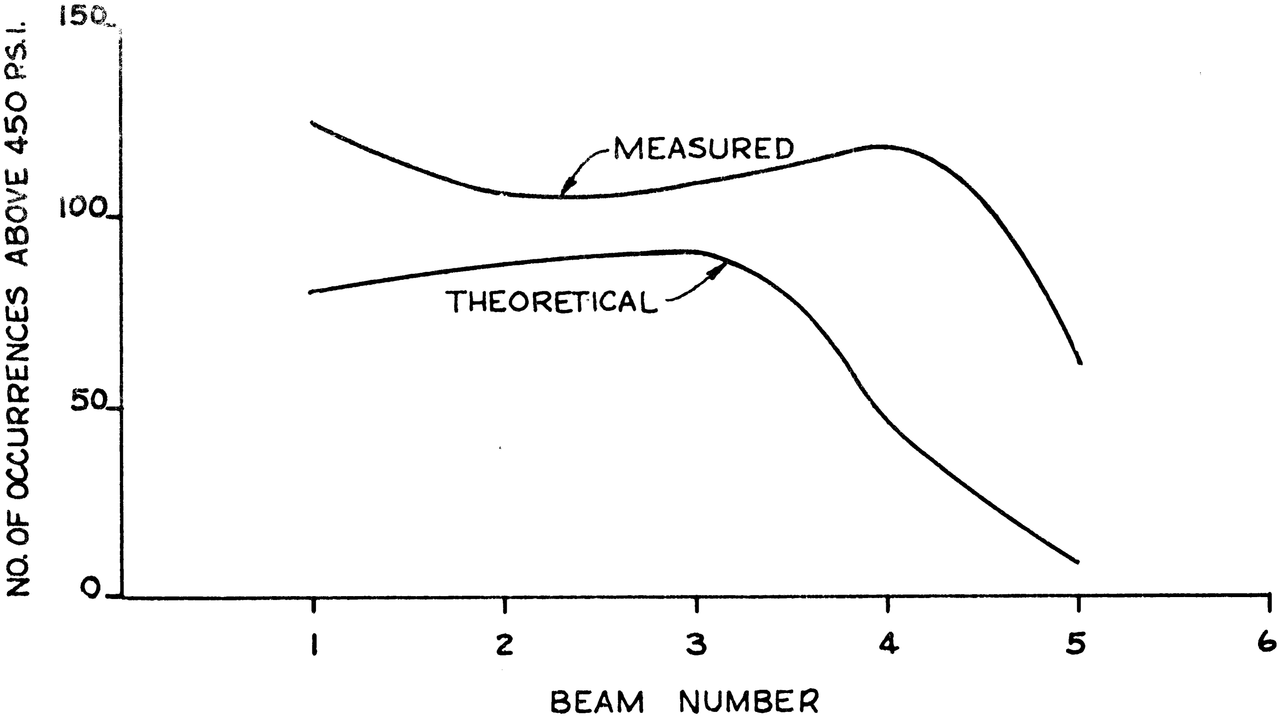


Figure 29. Comparison of theoretical and measured number of occurrences of midspan stress ranges over 450 psi, Rte. 95 bridge, 1645 hours, 9/8/68.

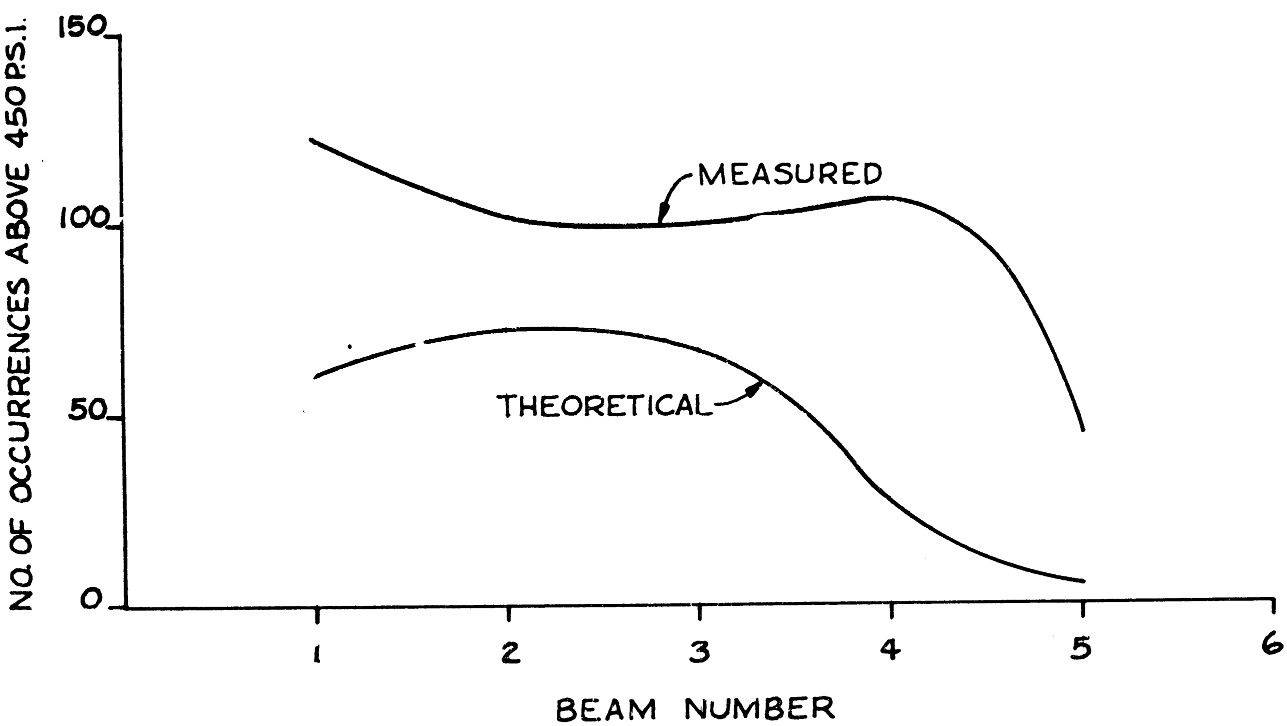


Figure 30. Comparison of theoretical and measured number of occurrences of midspan stress ranges over 450 psi, Rte. 95 bridge, 0854 hours, 9/9/68.

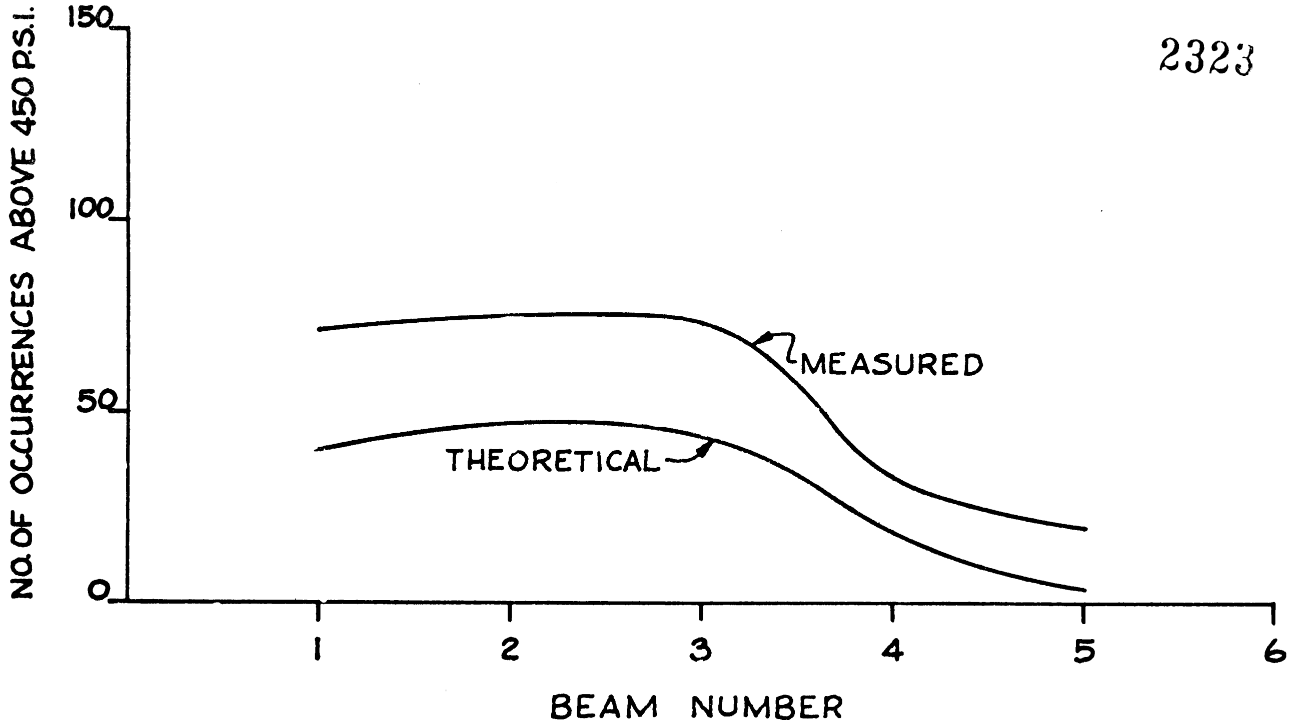


Figure 31. Comparison of theoretical and measured number of occurrences of midspan stress ranges over 450 psi, Rte. 95 bridge, 0958 hours, 9/9/68.

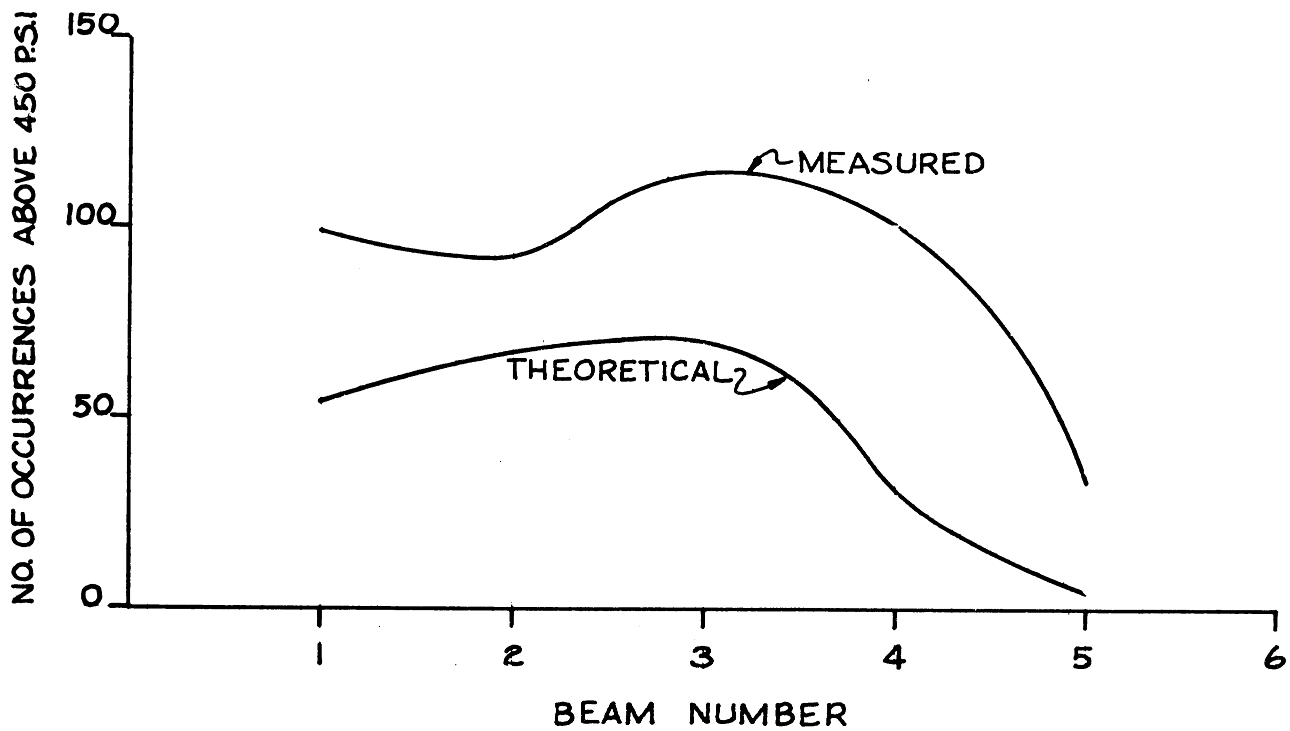


Figure 32. Comparison of theoretical and measured number of occurrences of midspan stress ranges over 450 psi, Rte. 95 bridge, 1102 hours, 9/9/68.

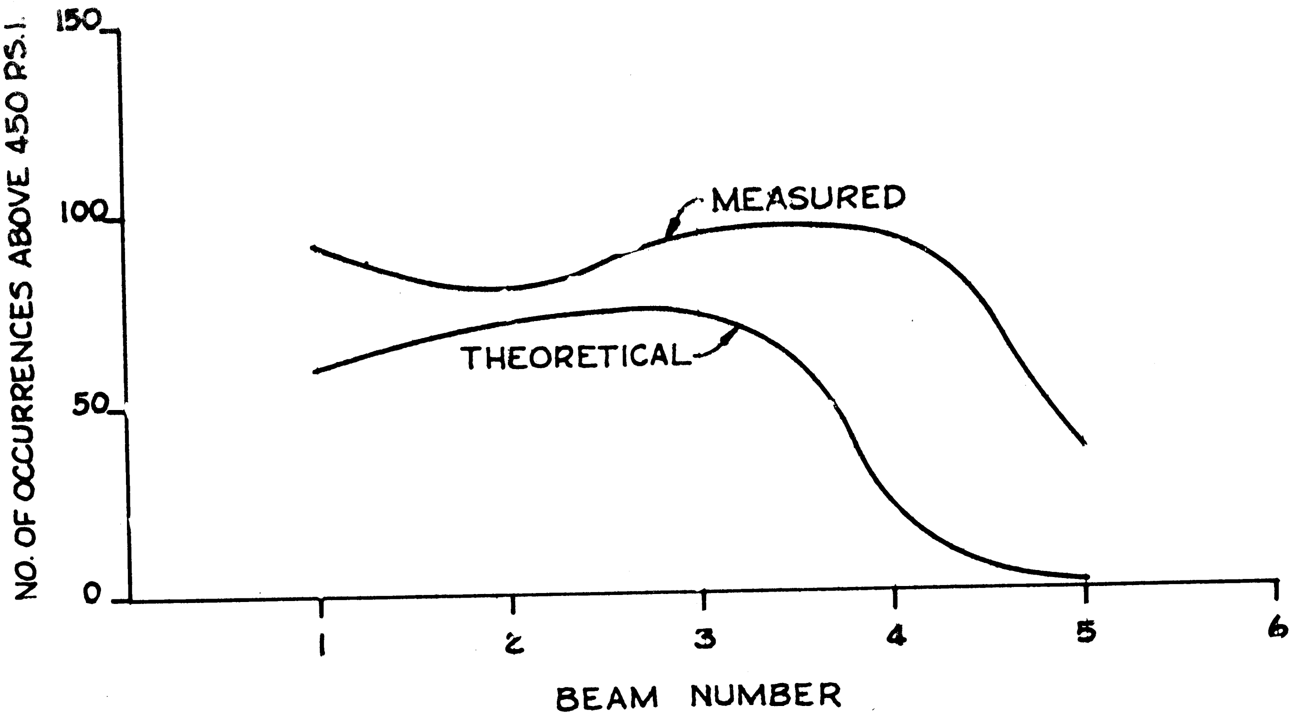


Figure 33. Comparison of theoretical and measured number of occurrences of midspan stress ranges over 450 psi, Rte. 95 bridge, 1207 hours, 9/9/68.

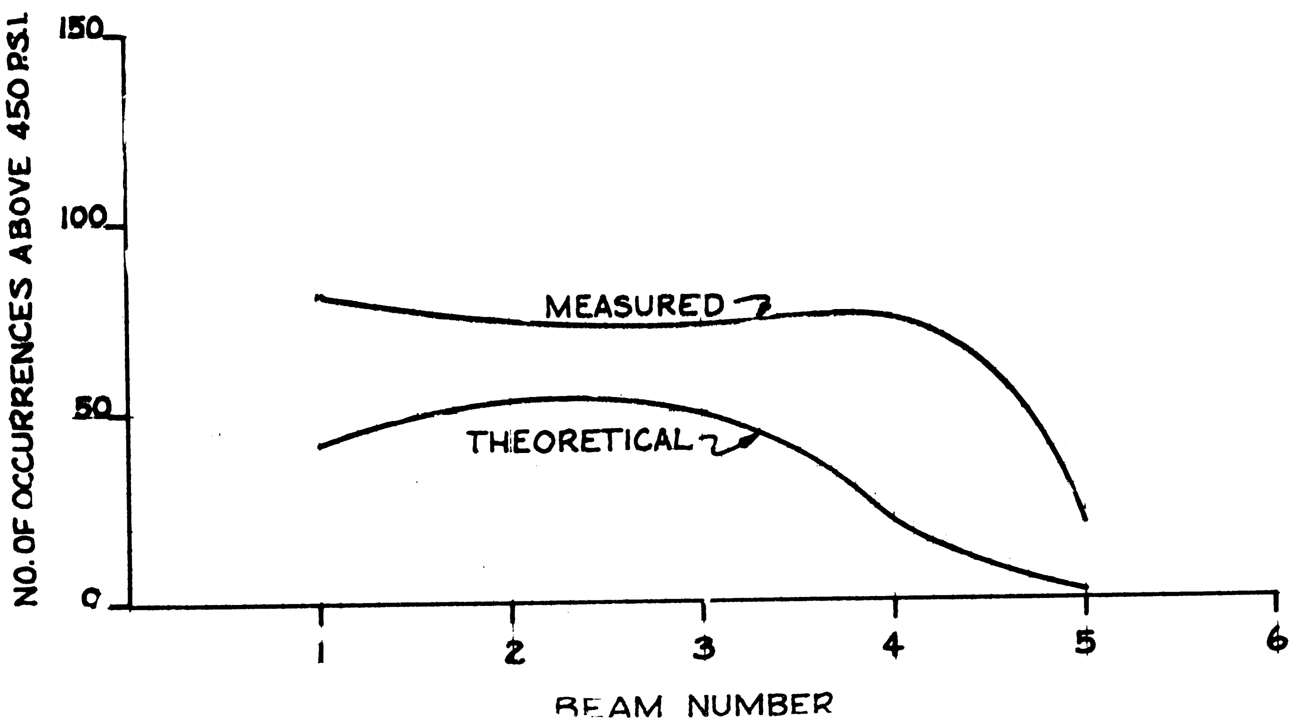


Figure 34. Comparison of theoretical and measured number of occurrences of midspan stress ranges over 450 psi, Rte. 95 bridge, 1312 hours, 9/9/68.

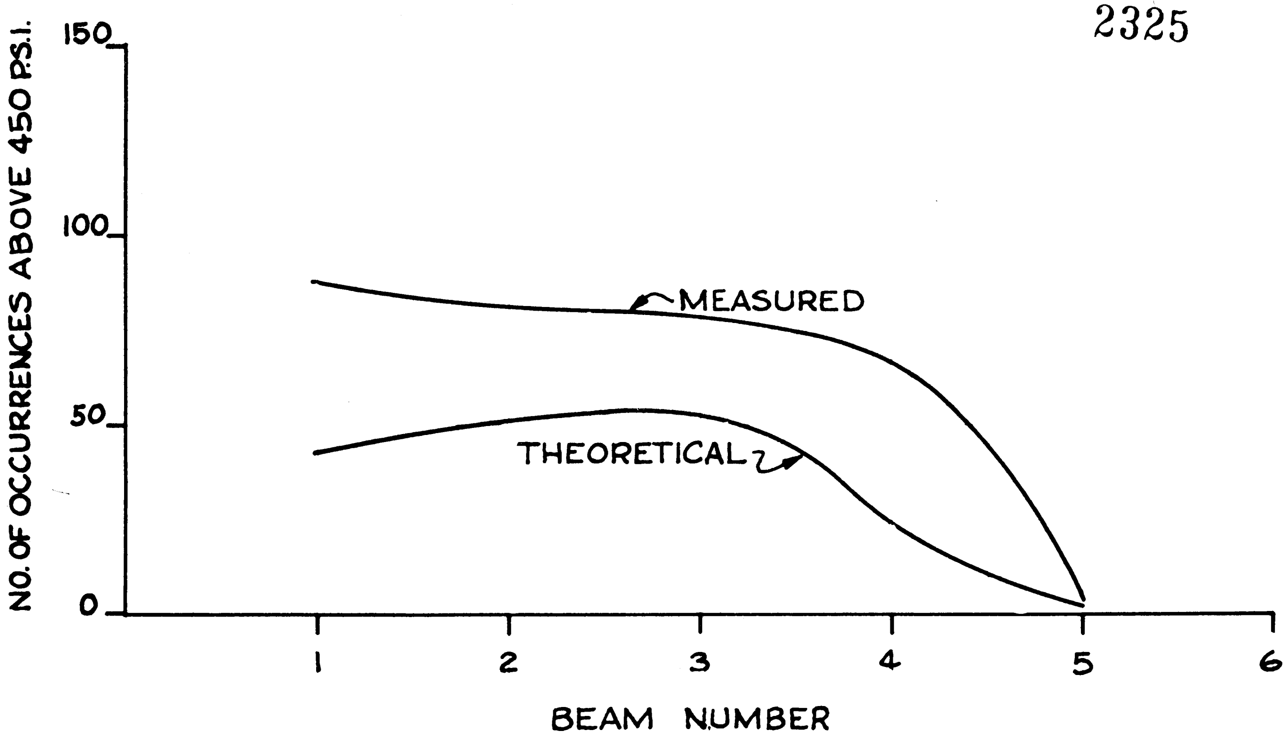


Figure 35. Comparison of theoretical and measured number of occurrences of midspan stress ranges over 450 psi, Rte. 95 bridge, 1416 hours, 9/9/68.

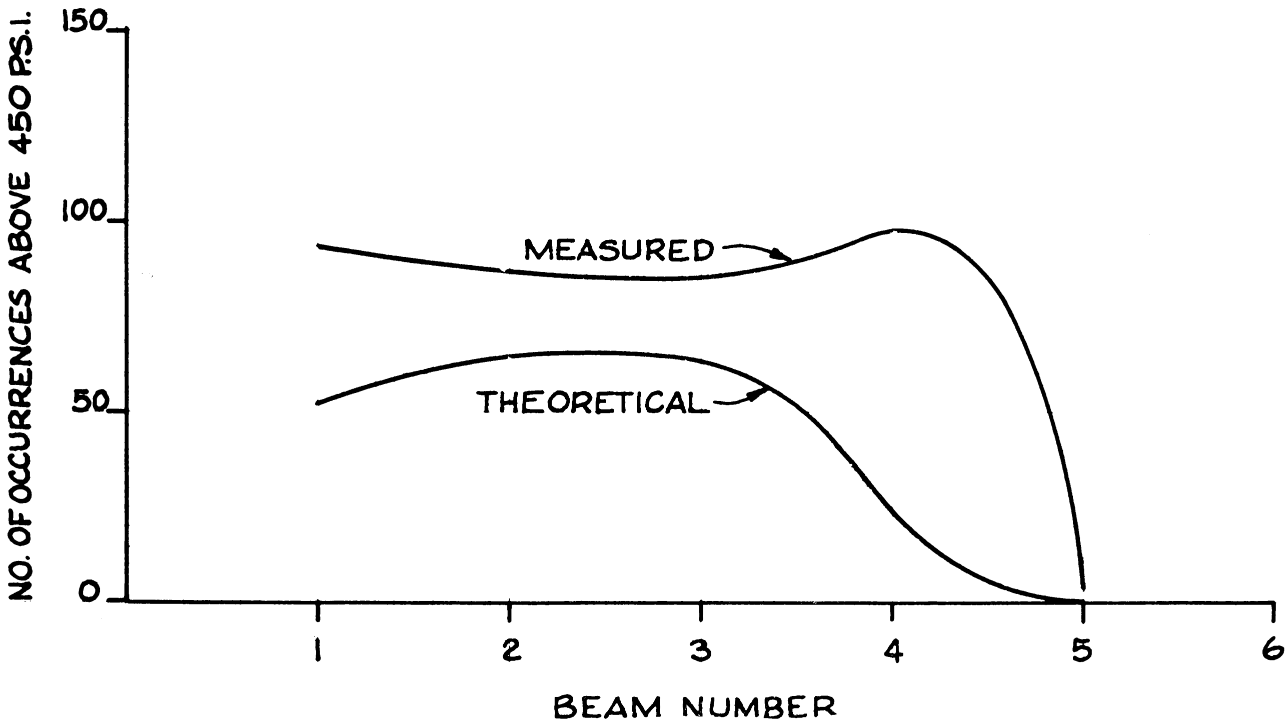


Figure 36. Comparison of theoretical and measured number of occurrences of midspan stress ranges over 450 psi, Rte. 95 bridge, 1520 hours, 9/9/68.

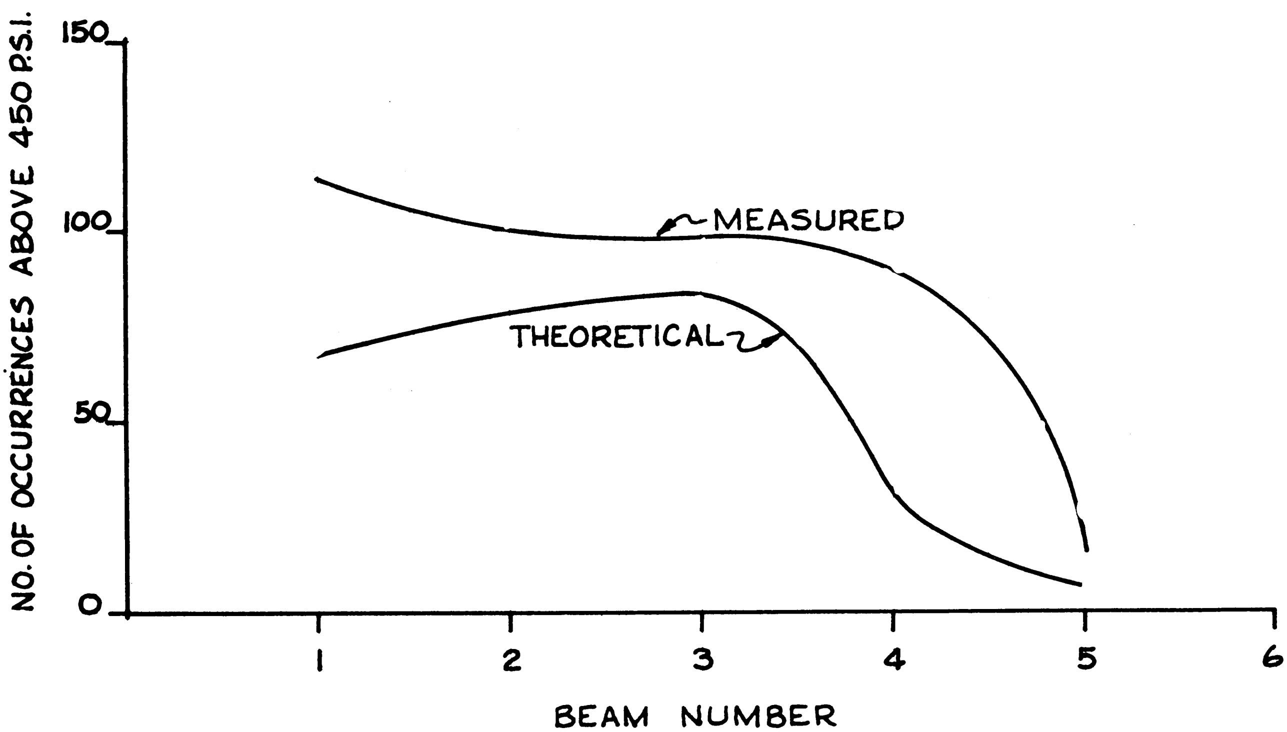


Figure 37. Comparison of theoretical and measured number of occurrences ot midspan stress ranges over 450 psi, Rte. 95 bridge, 1625 hours, 9/9/68.

ROUTE 81 BRIDGE TEST

General Description

The second structure tested, the bridge carrying the northbound lane of Route 81 over Cedar Creek, presents contrasts to the Route 95 bridge both in design type and traffic volume. The Route 81 bridge is composed of five 60 foot prestressed concrete beam spans, one of which was instrumented, and one 85 foot prestressed concrete beam span. The structure is located on a slight horizontal curve at the low point of a vertical curve. The instrumented span is skewed at approximately 13°-33'. The structure is generally typical of many prestressed concrete bridges throughout Virginia.

The average daily traffic volume and the percentage of trucks and buses, as published by the Virginia Department of Highways for the years 1968-1970, are shown in Table III.[3] The traffic volume at this relatively rural site is much less than that at the Route 95 test location.

TABLE III

AVERAGE DAILY TRAFFIC VOLUMES RTE. 81

From Rte. 11 N. of Strasburg to Rte. 627 E. of Middletown
(From reference 3)

		1968		1969*		1970	
		No.	%	No.	%	No.	%
	A.D.T. — all vehicles	3,255	100	3,422	100	3,692	100
Truck Type	2D (4-6 tires)	335	10.3	355	10.4	400	10.8
Truck Type	3 (6-10 tires)	12	0.4	8	0.2	75	2.0
Truck Type	2S-1, 2S-2, and 3S-2 (combined)	700	21.5	750	21.9	750	20.3
	Total Trucks	1,047	32.2	1,113	32.5	1,225	33.1
	Buses	8	0.2	10	0.3	18	0.5

*Year of Test

Details of Test

Test Span

The instrumented span on the Route 81 bridge was a prestressed concrete beam span, 58'-3" in length, center to center of bearings. As shown in Figure 38, the 8 inch thick concrete deck is supported on five AASHO Type III prestressed beams, detailed in Figure 39. The beams are numbered 1-5 from the right side of the bridge facing in the direction of traffic flow. Cast in place concrete diaphragms are located on the skew angle at midspan and over the bearings. The 30 foot clear roadway is divided into two traffic lanes, numbered from the right as before.

Gage Locations

Ten wire, resistance type strain gages were mounted on the surfaces of the concrete beams at midspan, in the positions shown in Figure 38. Type A93 concrete gages were used.

Results

Traffic Data

The Route 81 test ran continuously from 11:00 a.m. on Friday, September 5, 1969, to 11:00 p.m. on Monday, September 8, 1969, a period of 84 hours, but approximately 12 hours of sampling were lost due to equipment malfunction. Approximately 2,616 trucks crossed the bridge during 69 sampling periods, and weighing station records were completed on 2,276, or 87% of the vehicles.

Figure 40 shows the percentage of the trucks in each of eight ranges for both traffic lanes and the bridge as a whole. As in the case of the Route 95 bridge, the majority of the trucks, almost 98% at this site, crossed the structure in the right-hand lane. The Route 81 traffic differs from that at the Route 95 site in that the population distribution is skewed toward a higher percentage of heavy trucks.

Stress Data

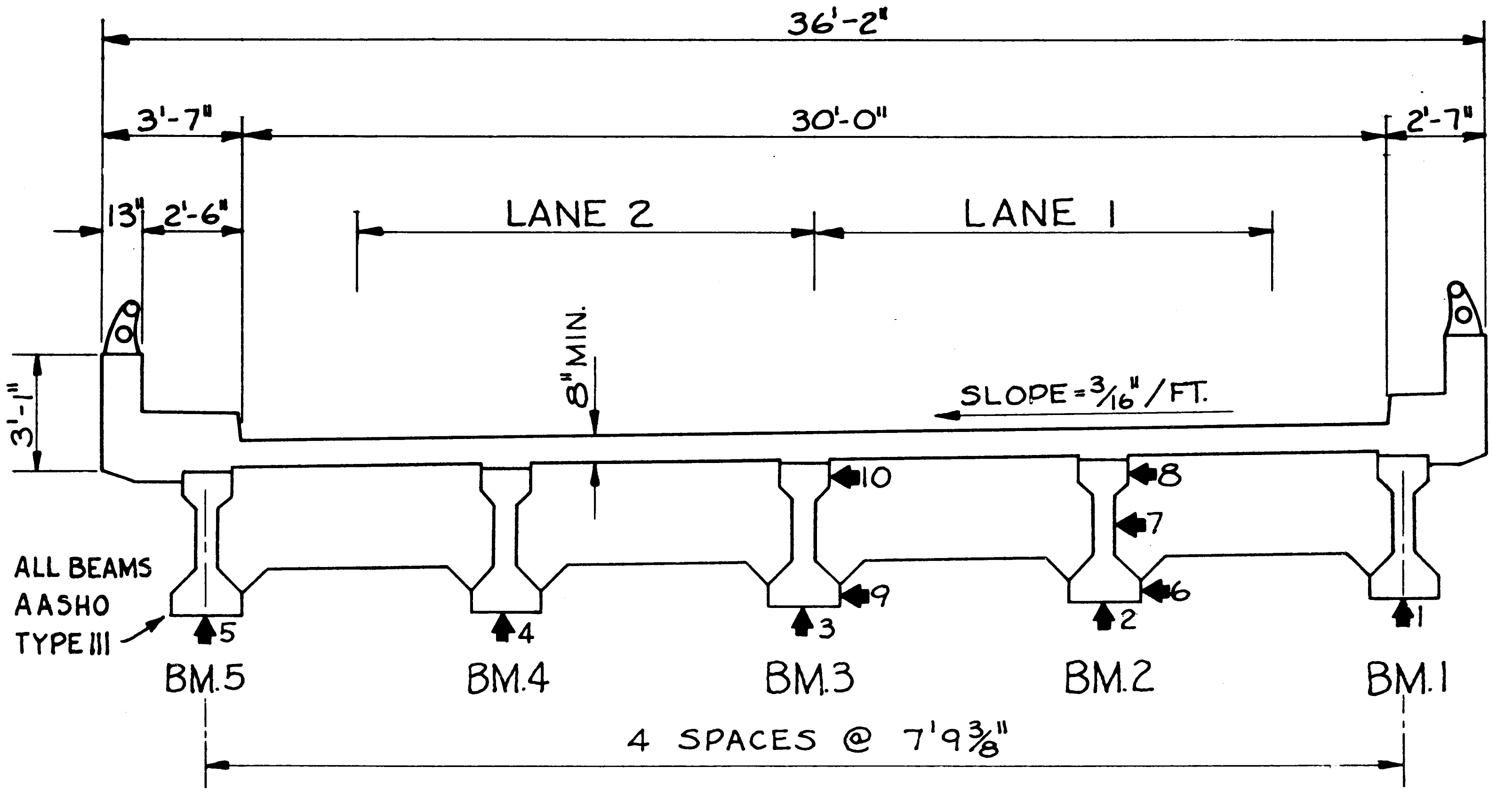
The number of occurrences at each strain range are tabulated in Table IV and shown graphically in Figures 41-50. The stresses shown are based on the use of a modulus of elasticity of 4.34×10^6 psi for the prestressed concrete, obtained through the A.C.I. formula(6).

$$E = W^{1.5} \ (33) \sqrt{f'_c}$$

where W is the weight of the concrete per cubic foot, assumed to be 150 lb., and,

f'_c is the compressive strength of the concrete, assumed to be 5,000 psi.

The minimum test level was set at 10.1 micro-inches per inch of strain, or 43.8 psi.



NOTE: ◀ INDICATES LOCATION OF STRAIN GAGE HAVING NUMBER SHOWN.

Figure 38. Details of instrumented span, Route 81 bridge.

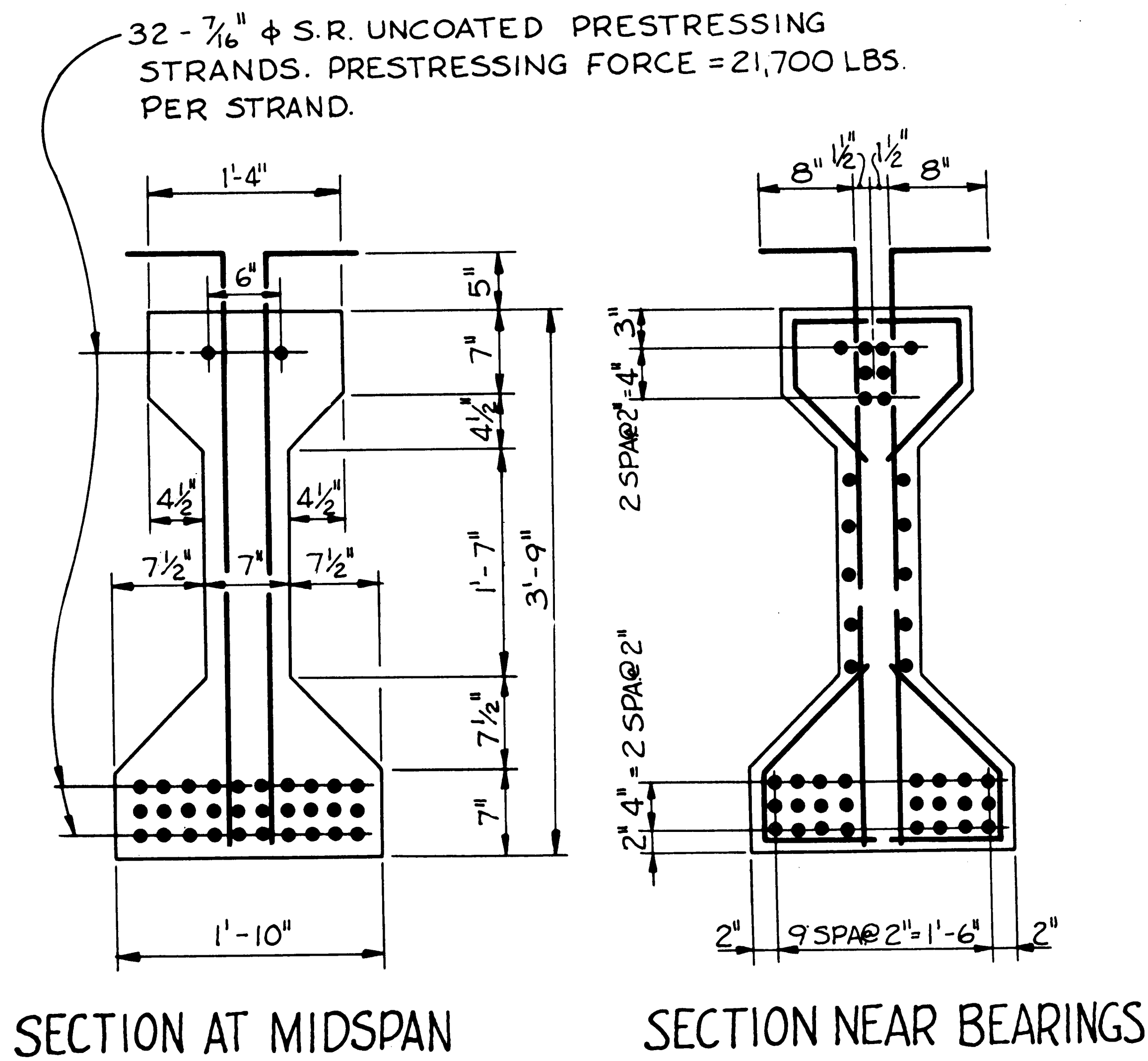
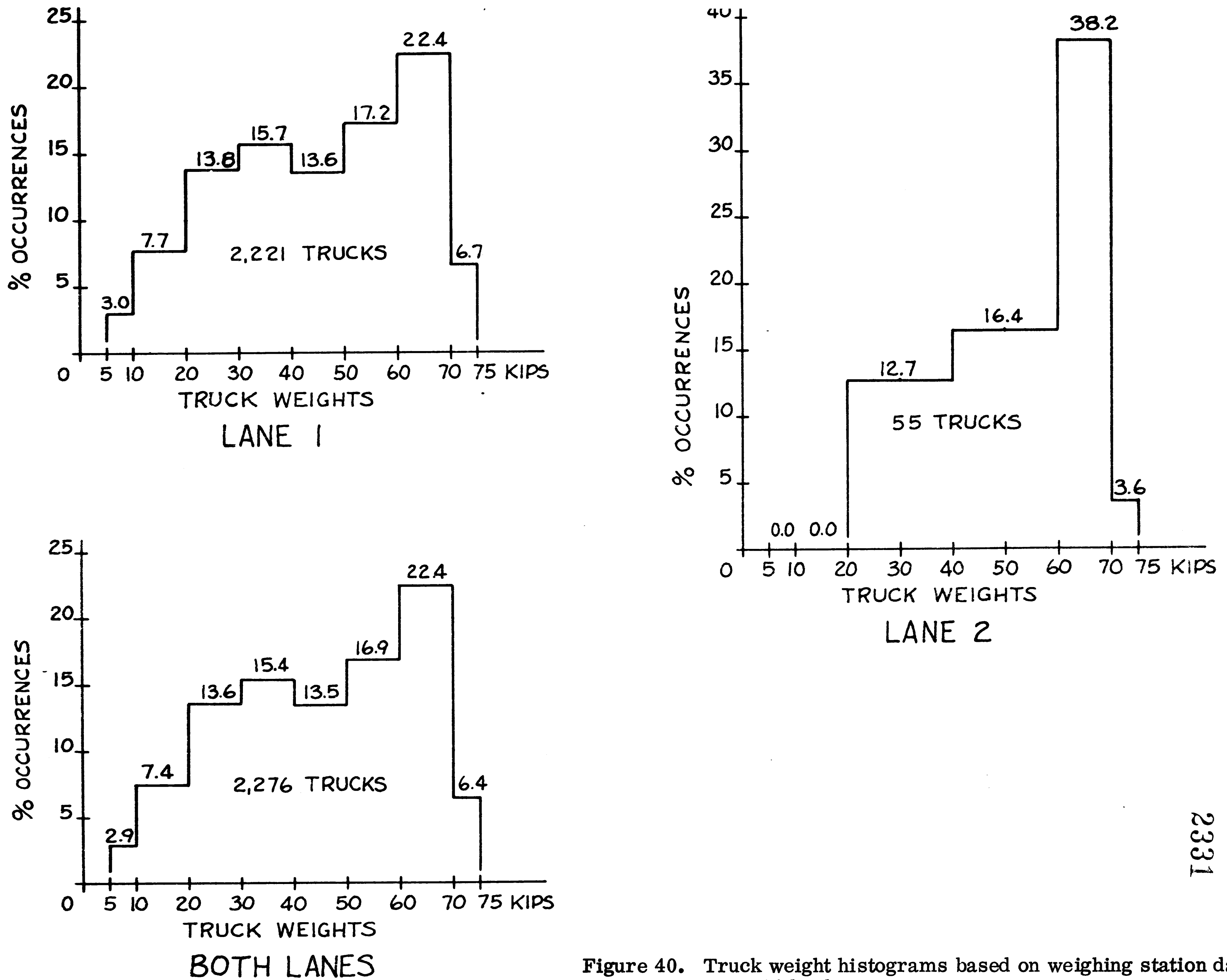


Figure 39. Details of AASHO Type III beams, Route 81 bridge.



2331

Figure 40. Truck weight histograms based on weighing station data, Route 81 bridge.

TABLE IV

TOTAL NUMBER OF OCCURRENCES AT EACH STRAIN LEVEL, ROUTE 81 BRIDGE
(Note: Position of gages is shown in Figure 37.)

Strain Level	Strain μ -, in./in.	Stress, psi	Gage Number 1	2	3	4	5	6	7	8	9	10
0												
			0	0	0	0	0	0	0	0	0	0
1	90.9	394.2										
			0	0	0	0	0	0	0	0	0	0
2	80.8	350.4										
			0	0	0	0	0	0	0	0	0	0
3	70.7	306.6										
			2	1	0	0	0	0	0	0	0	0
4	60.6	262.8										
			2	7	3	1	0	2	0	0	0	0
5	50.5	219.0										
			2	18	7	0	0	8	1	0	2	0
6	40.4	175.2										
			68	226	84	29	2	145	2	1	28	16
7	30.3	131.4										
			887	950	1,069	91	16	1,120	6	5	789	53
8	20.2	87.6										
			1,963	992	1,237	1,013	233	1,365	213	31	1,433	69
9	10.1	43.8										
	TOTAL		2,924	2,194	2,400	1,134	251	2,640	222	37	2,252	138

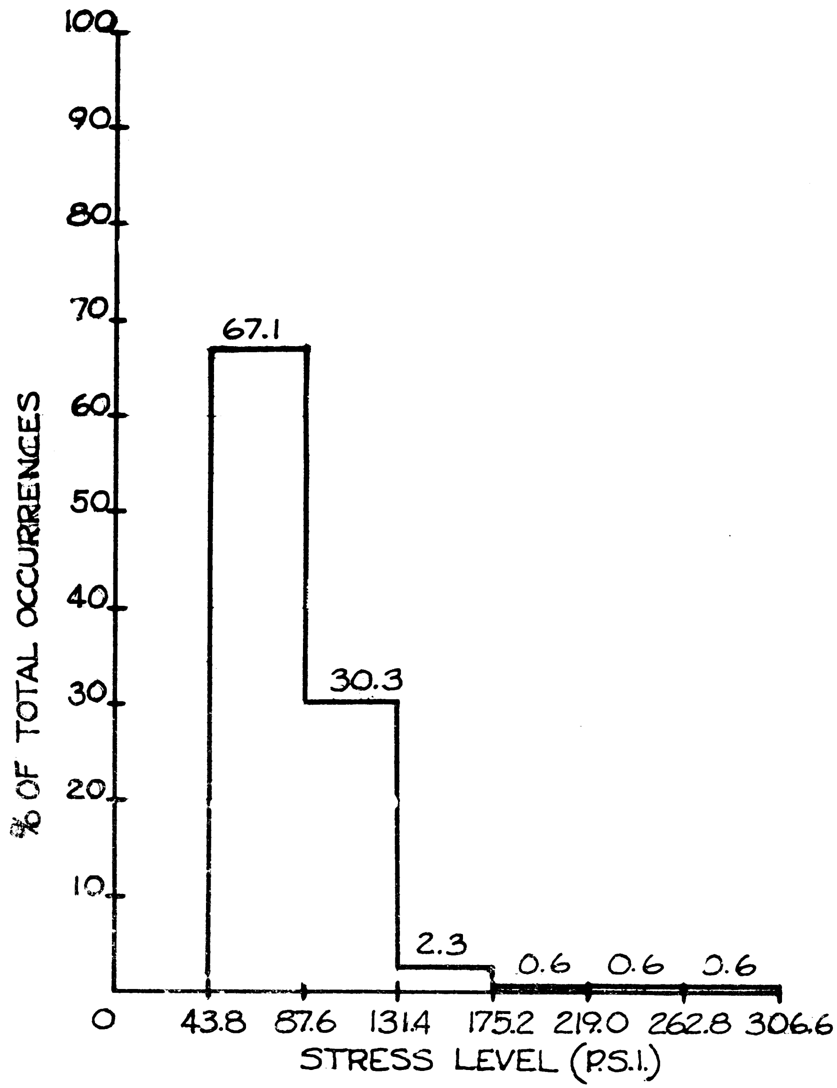


Figure 41. Stress histogram, gage #1, midspan beam 1.

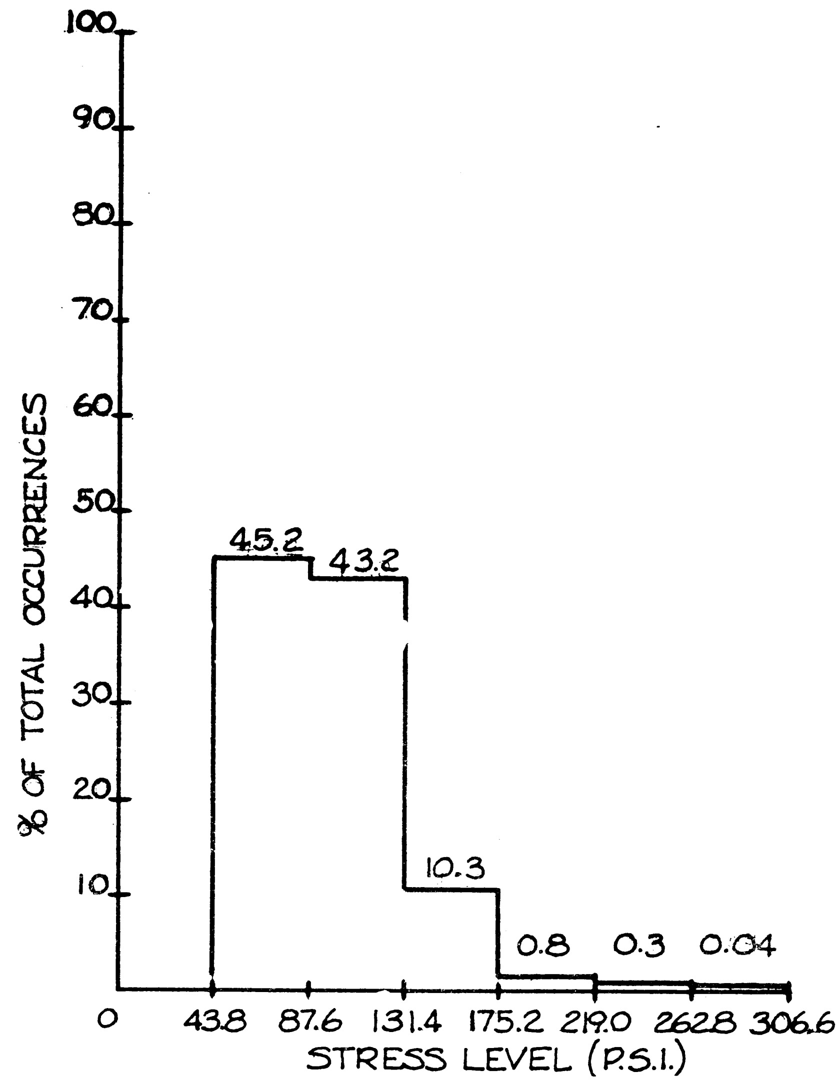


Figure 42. Stress histogram, gage #2, midspan beam 2.

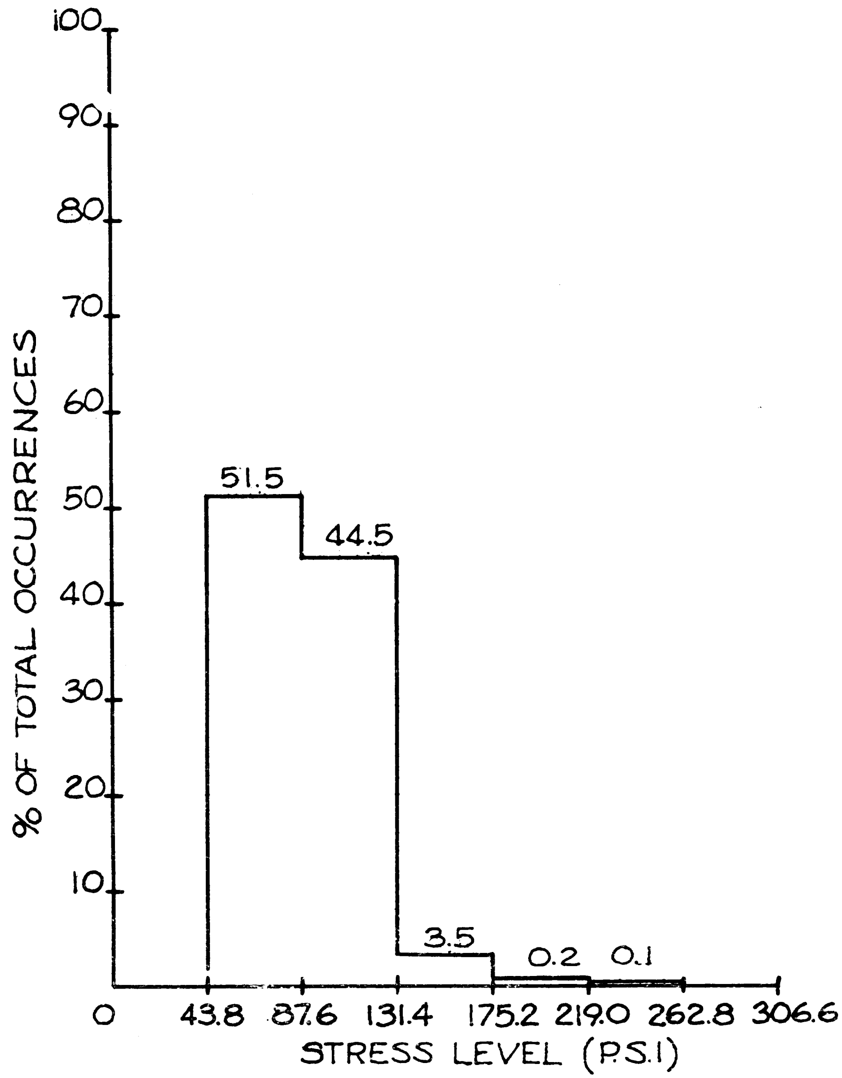


Figure 43. Stress histogram, gage #3, midspan beam 3.

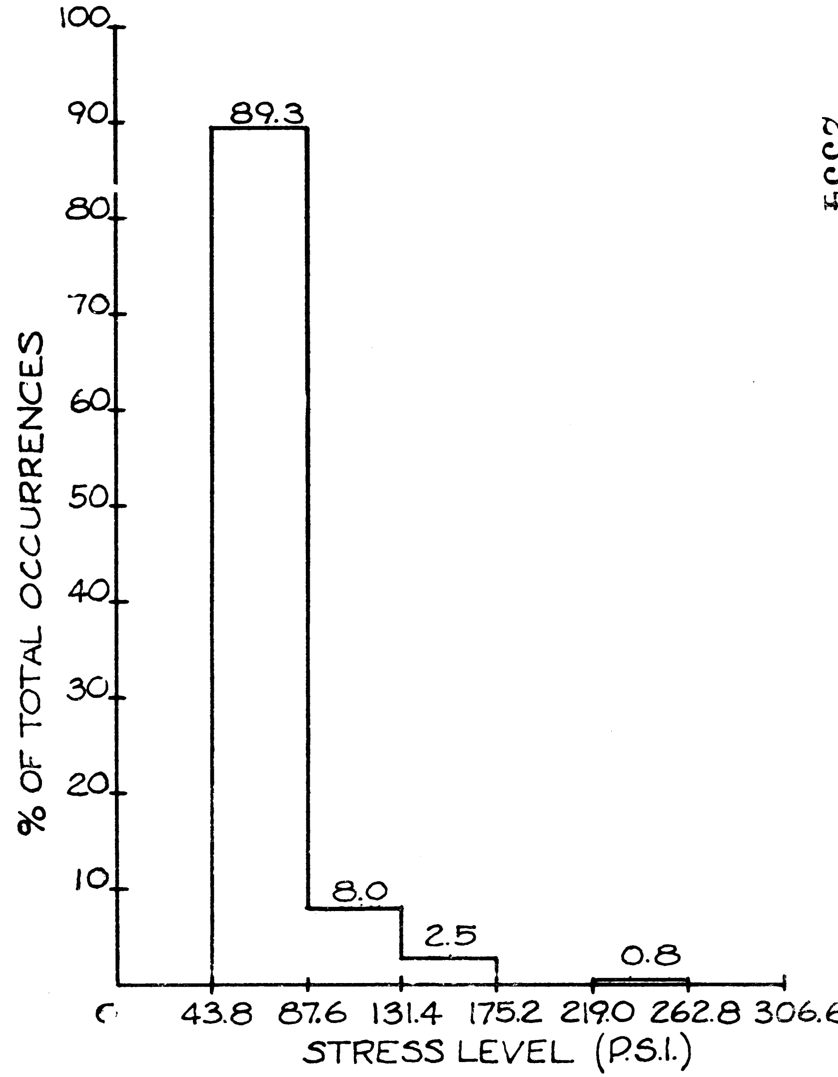


Figure 44. Stress histogram, gage #4, midspan beam 4.

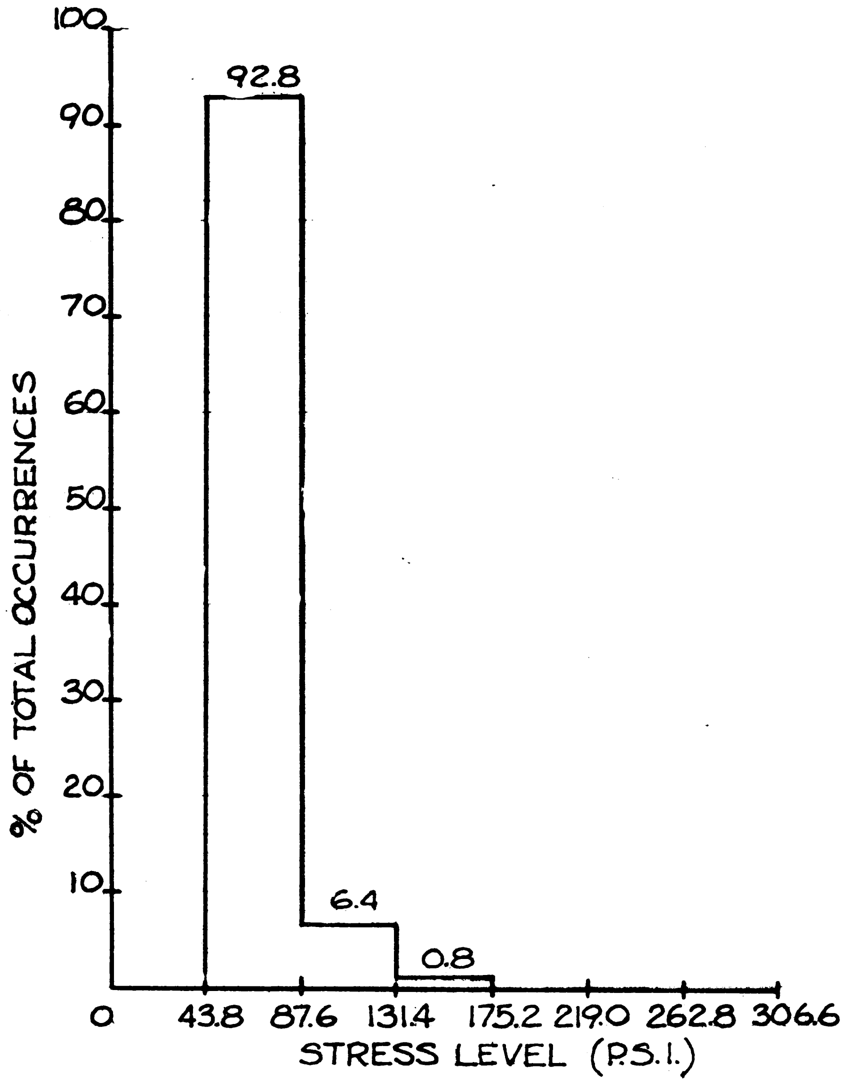


Figure 45. Stress histogram, gage #5, midspan beam 5.

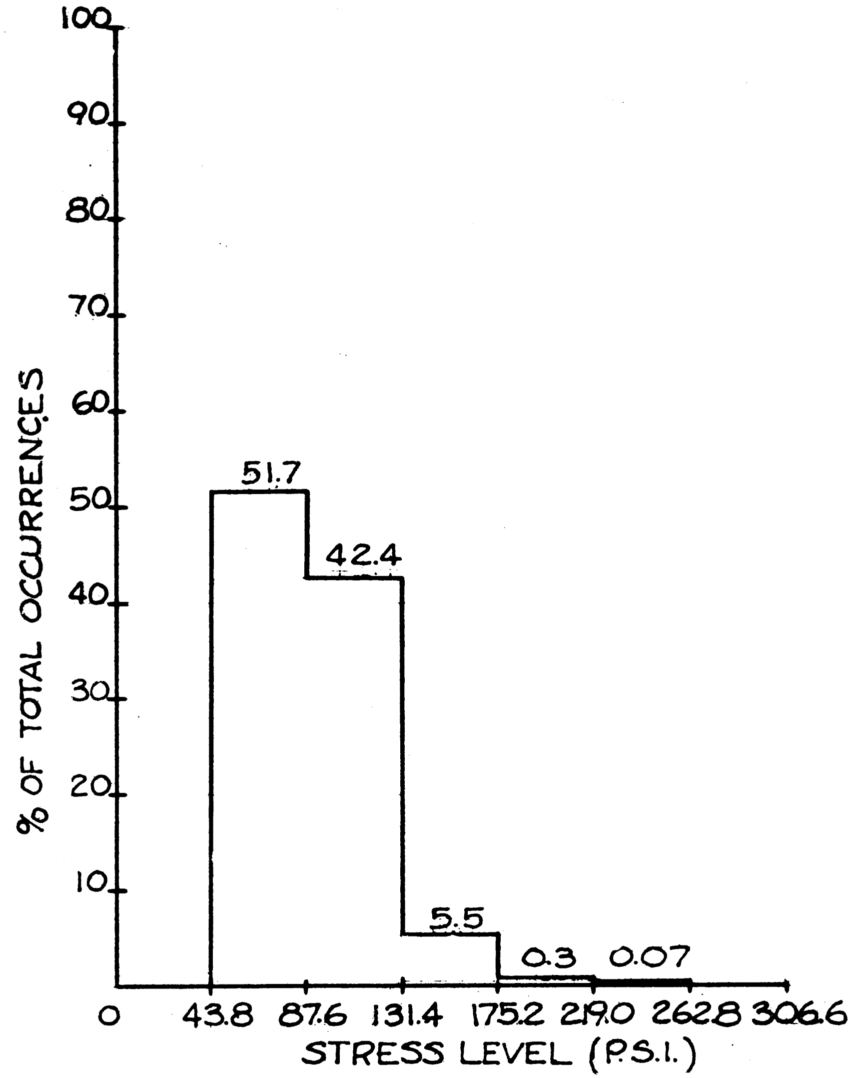


Figure 46. Stress histogram, gage #6, side of bottom flange, beam 2.

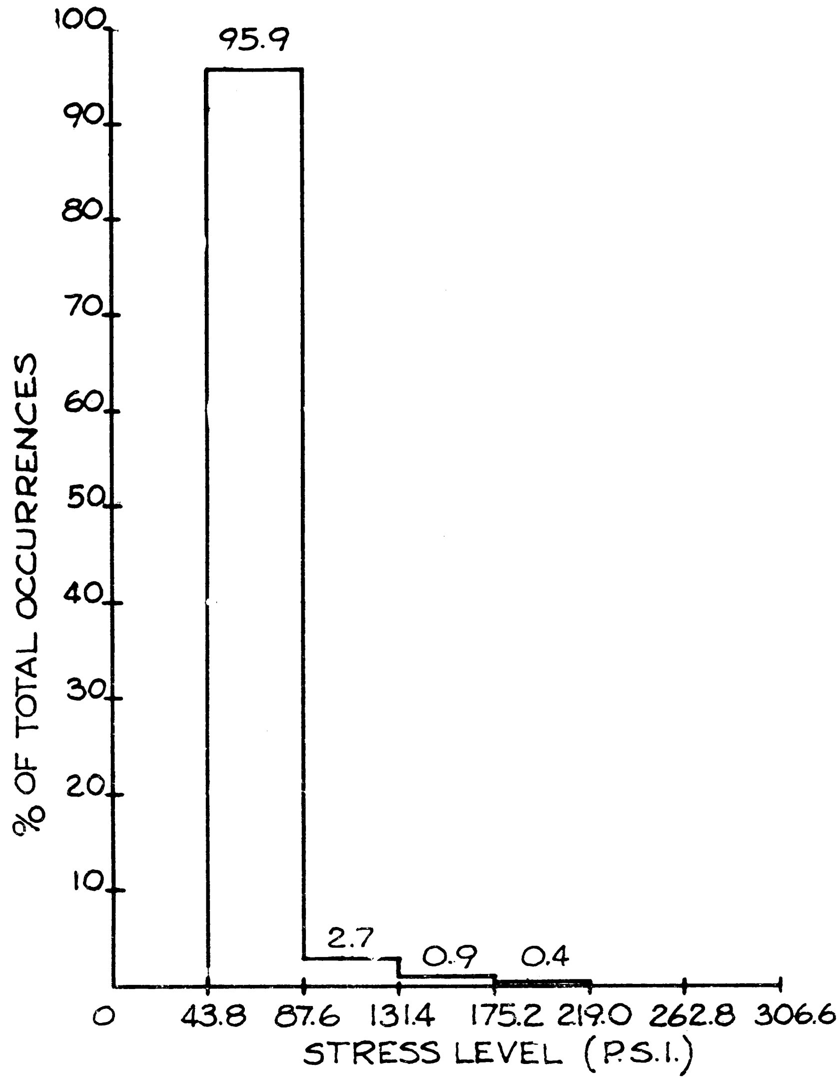


Figure 47. Stress histogram, gage #7, side of beam 2 at mid-depth.

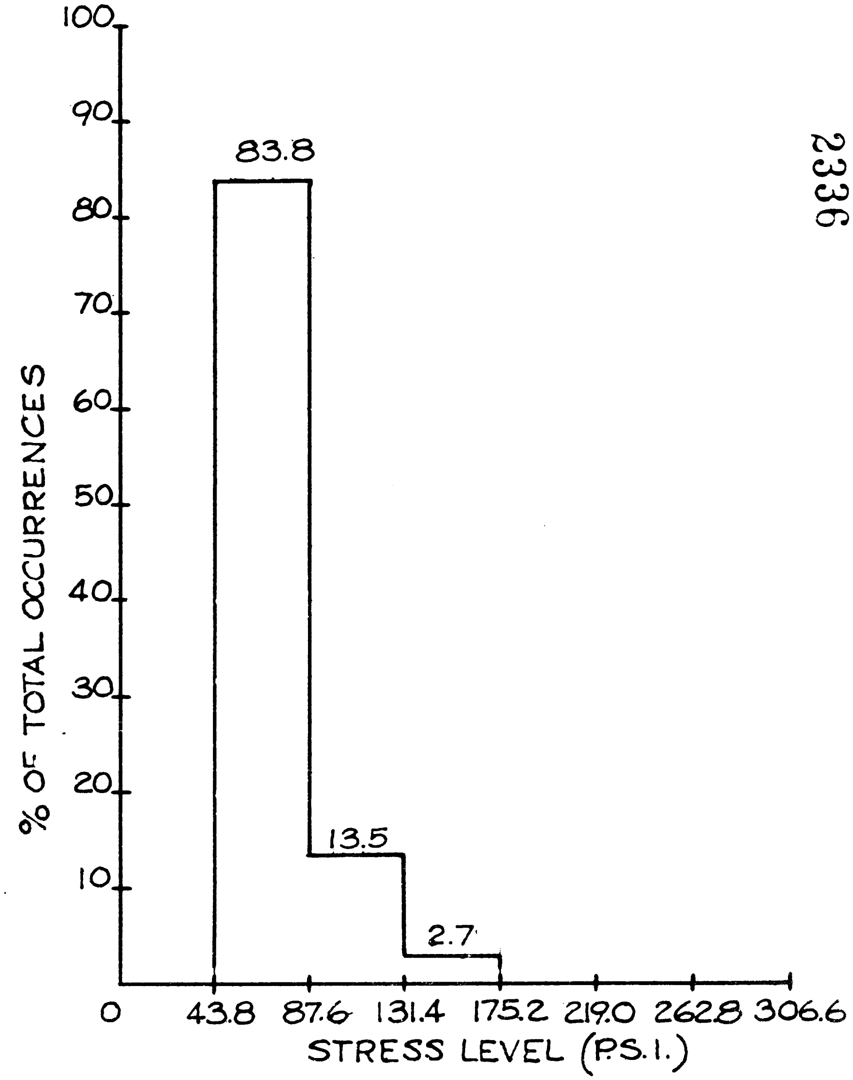


Figure 48. Stress histogram, gage #8, side of top flange, beam 2.

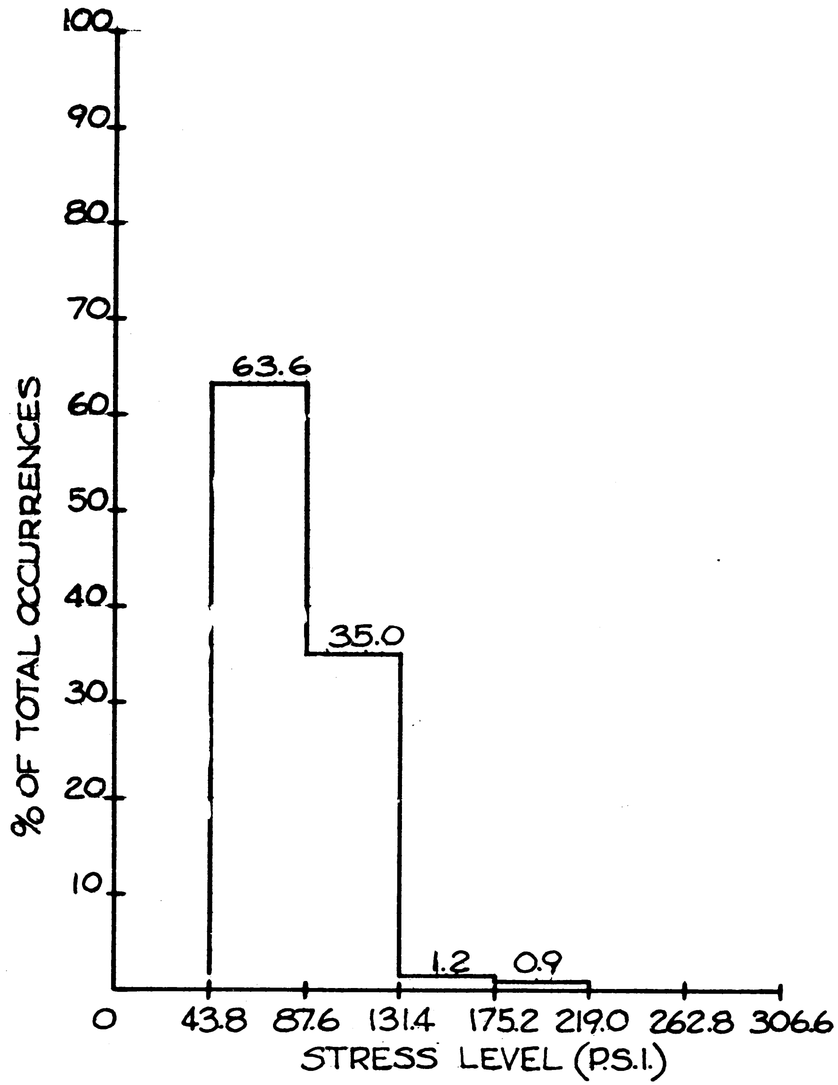


Figure 49. Stress histogram, gage #9, side of bottom flange, beam 3.

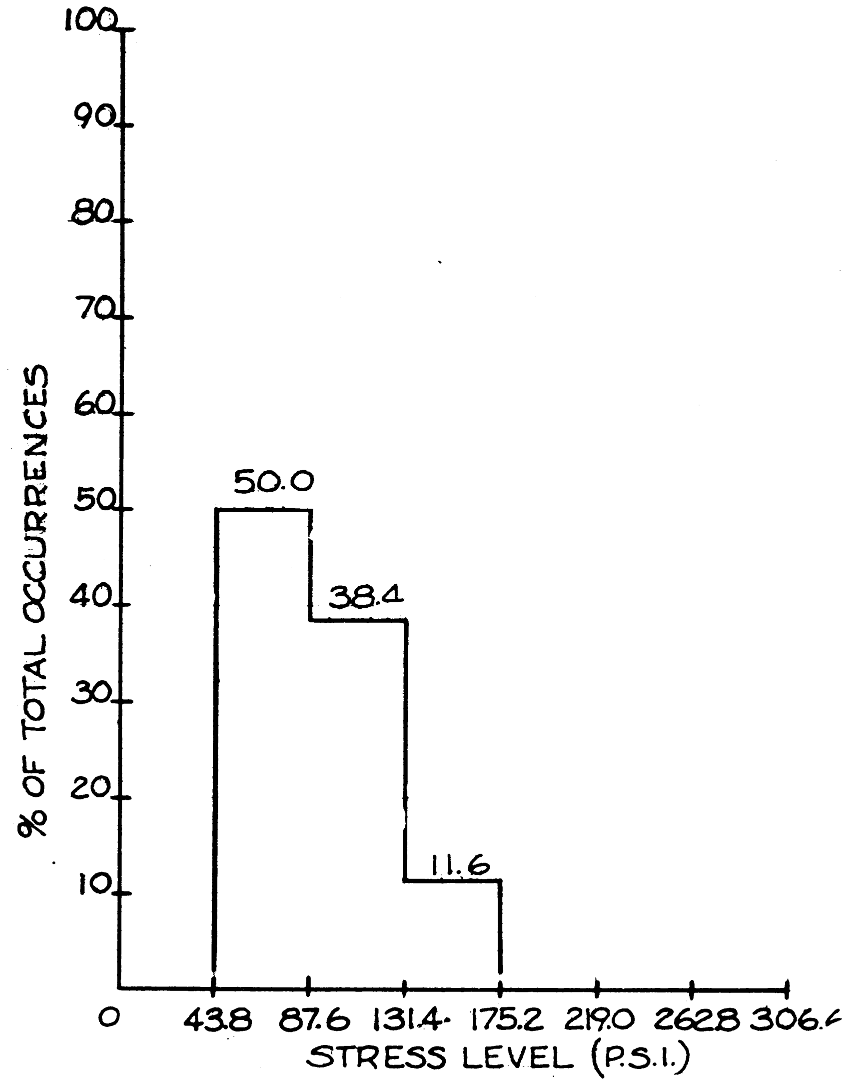


Figure 50. Stress histogram, gage #10, side of top flange, beam 3.

The data in Table IV indicate that beam 2, which is located directly beneath the right-hand lane, is subject to the greatest number of occurrences of the higher stress ranges, those above 130 psi. Likewise beams 4 and 5 under the left-hand lane, seldom used by trucks, have significantly fewer occurrences of ranges above 130 psi. This sensitivity to the path of the vehicles was also evident in the case of the Route 95 bridge. Beams 1 and 3 were more equitably loaded on the Route 81 bridge than on the Route 95 structure, possibly due to the action of the stiff cast-in-place concrete diaphragms in distributing the load.

The gages located on the side surfaces of beams 2 and 3, gages 6 through 10, show the expected decrease in stress as the gage position approaches the location of the neutral axis. The data for gages 6-10 also tend to verify the values obtained at the midspan lower flange gages.

The histograms in Figures 41-50 indicate that the bulk of the strain ranges recorded at any of the instrumented points on the girders were below 130 psi, and the highest ranges, 3 occurrences recorded in beams 1 and 2, were below 300 psi. Fatigue is, therefore, unlikely to present any problems on this bridge under currently allowable service loads.

Theoretical Correlation Study

The analytical approach utilized for the Route 95 bridge was also applied to the Route 81 structure, with only slight modifications. As before, the maximum moment due to the individual trucks was distributed to the girders by means of factors developed through the application of finite difference theory in conjunction with a representative three axle truck loading. In the case of the prestressed structure, however, it was necessary to include the stiffness contributions of the cast-in-place concrete diaphragms.

The midspan lower flange stress levels predicted by the analytical method were compared with stress ranges measured during three selected periods of two or three hours each. The comparison is made in a series of histograms, Figures 51-53, showing the number of occurrences in each of the stress levels. While the theoretical results verify the magnitude of the measured stresses, the analytical approach consistently predicts a greater number of events in the higher stress ranges, those of 130 psi and above.

Curves showing the total number of occurrences above the minimum test level for each of the six beams are shown for the three sampling periods in Figures 54-56. The total number of occurrences above the minimum test level is reasonably the same, indicating that, as would be expected, the effects of impact and secondary stress ranges due to dynamic loading are less pronounced in the case of the relatively massive concrete superstructure. This is verified by the total number of stress events, Table IV, caused by the 2,616 trucks crossing the structure during testing. In some instances the average is less than one event above 43.8 psi per vehicle.

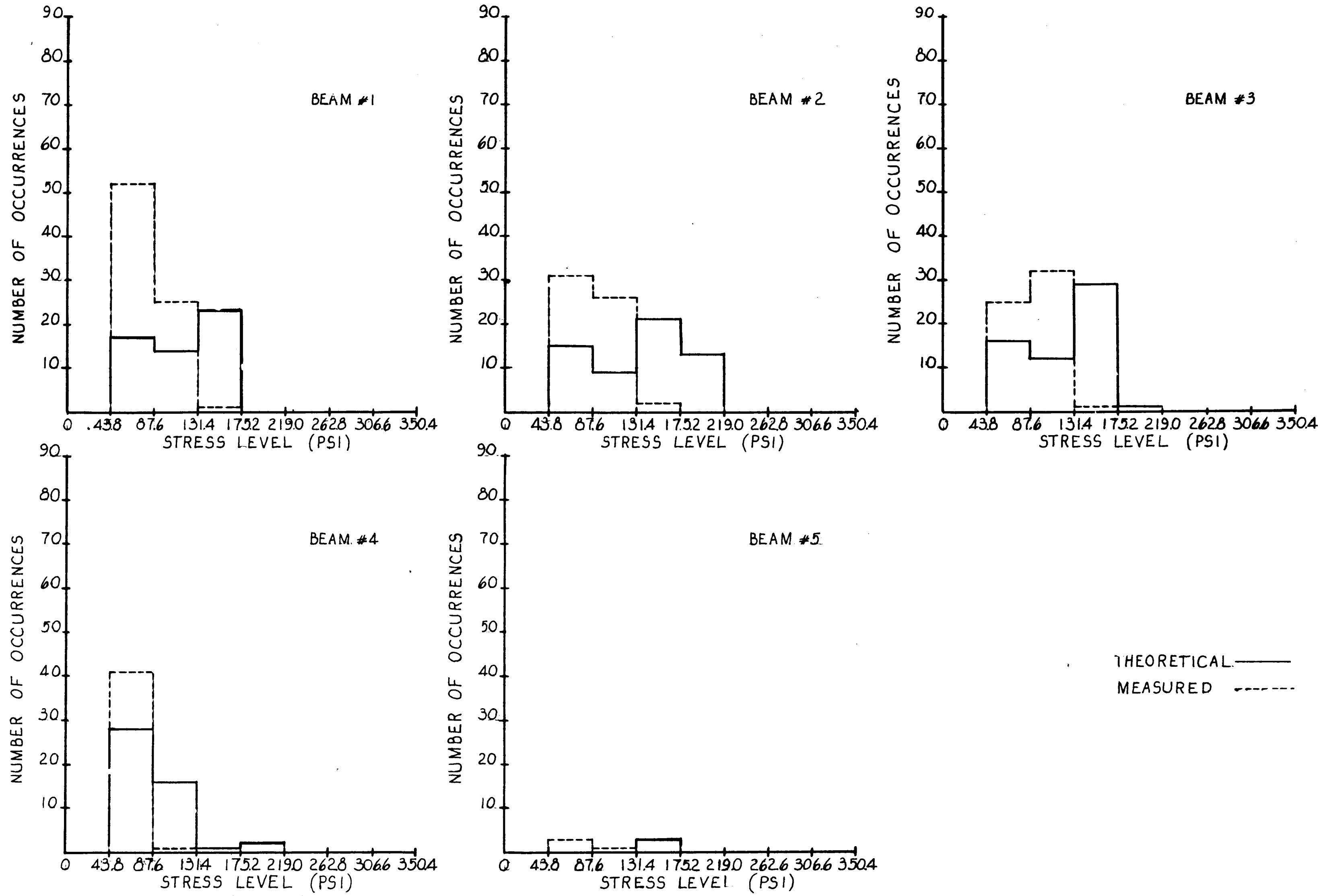


Figure 51. Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 1418 9/7/69.

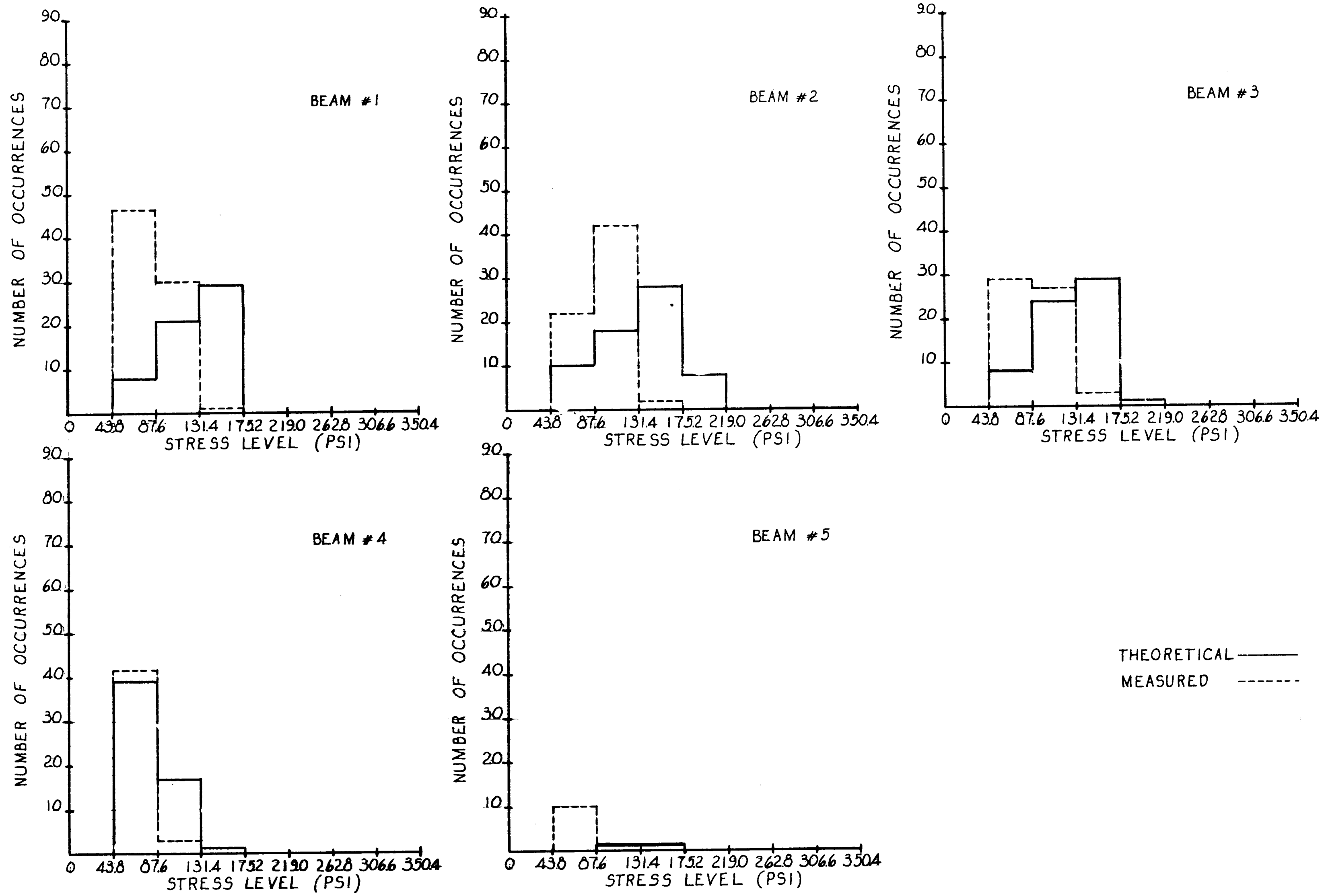


Figure 52. Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 1200 hours 9/7/69.

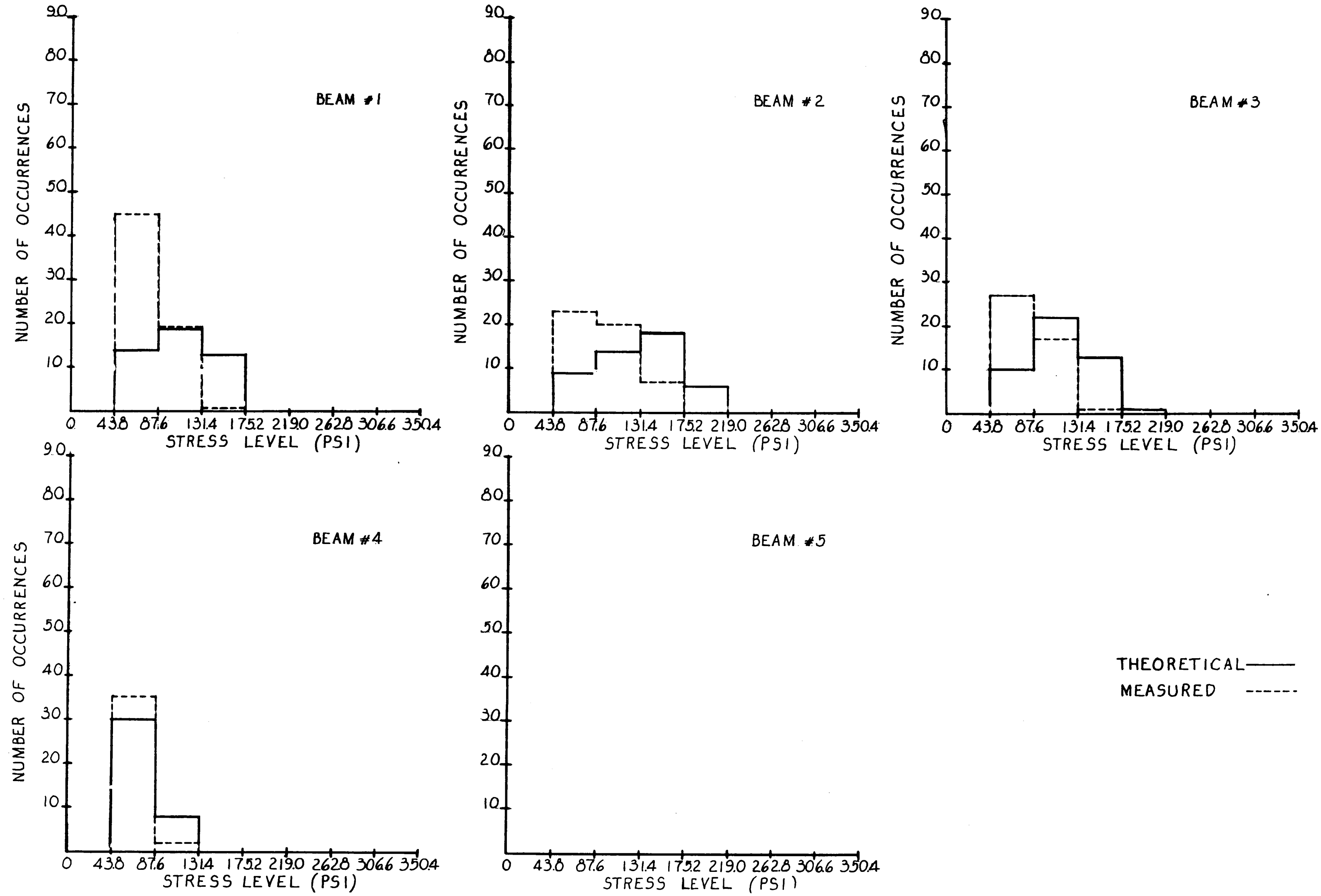


Figure 53. Comparison of theoretical and measured number of occurrences at each stress level, midspan gages, 0858 hours 9/7/69.

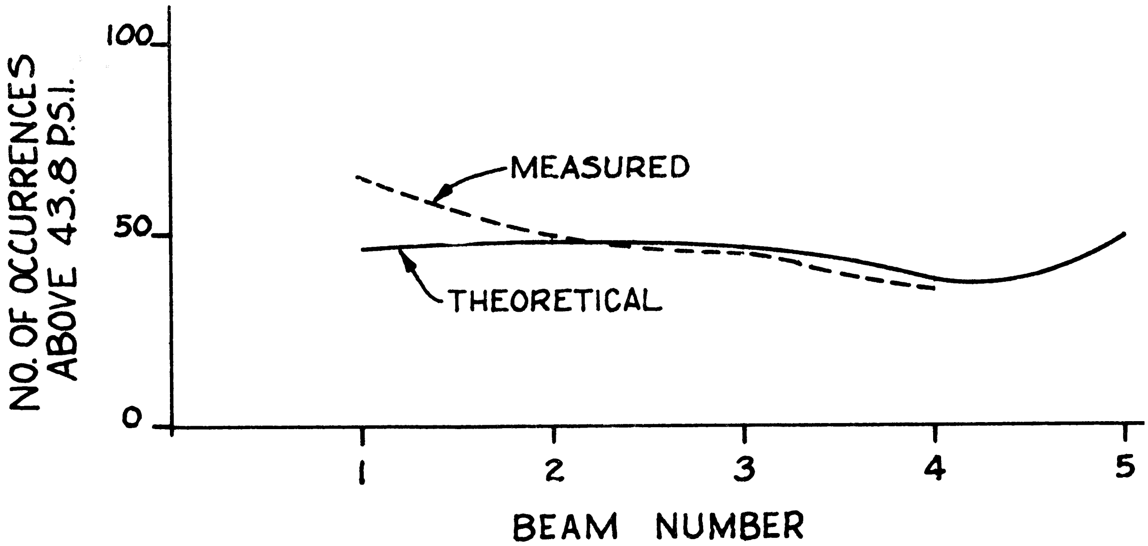


Figure 54. Comparison of theoretical and measured number of occurrences of midspan stress ranges over 43.8 psi, Rte. 81 bridge, 0858–1105 hours, 9/7/69.

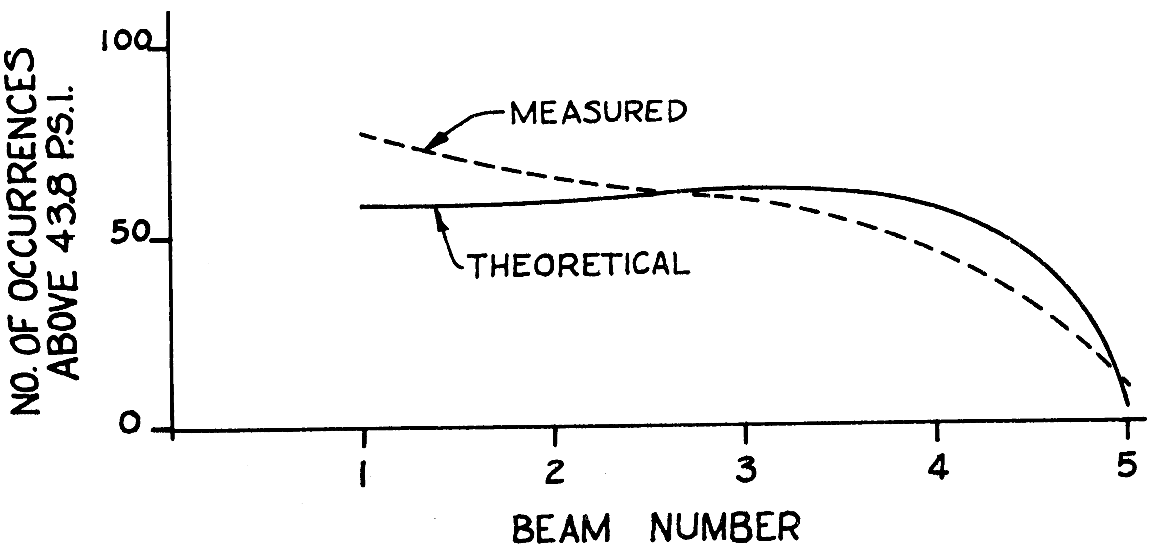


Figure 55. Comparison of theoretical and measured number of occurrences of midspan stress ranges over 43.8 psi, Rte. 81 bridge, 1209–1314 hours, 9/7/69.

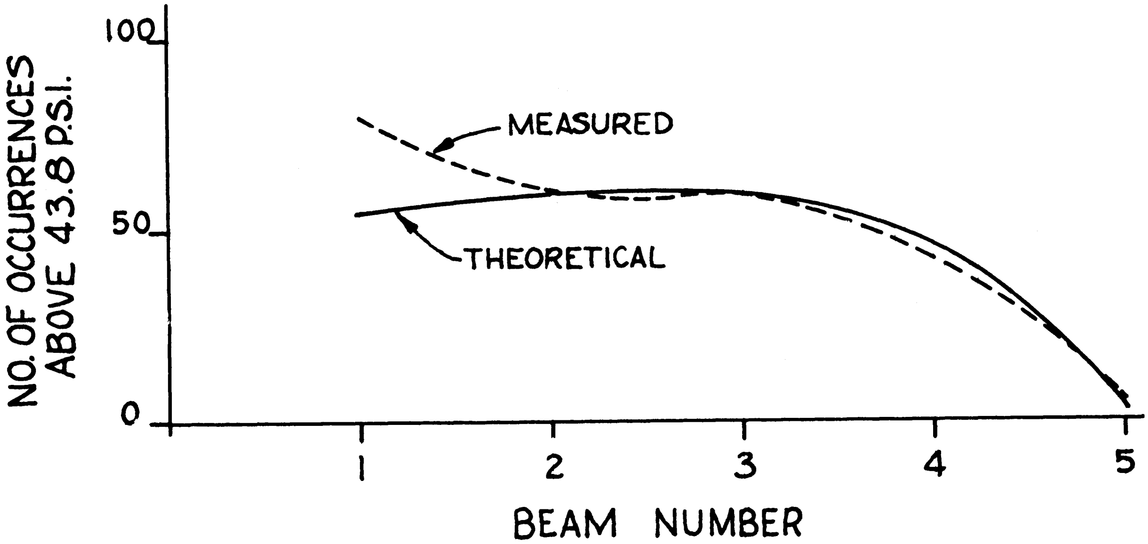


Figure 56. Comparison of theoretical and measured number of occurrences of midspan stress ranges over 43.8 psi, Rte. 81 bridge, 1418-1522 hours, 9/7/69.

The consistent difference in the predicted and measured events for beam 1 is again ascribed to the assumption of a high moment of inertia for the exterior beam.

In summary, the number of higher stress ranges predicted by the analytical method was consistently too high, which indicates an error, possibly in the selection of the modulus of elasticity, which is difficult to ascertain. Comparison of theoretical correlations for the Route 81 bridge with those of the Route 95 bridge, presented earlier, indicated that the effect of impact and the occurrence of secondary stress ranges in conjunction with the passage of a single vehicle are of greatest importance in the case of relatively light, flexible spans.

CHARACTERISTICS OF TRUCK POPULATIONS

The weighing station data obtained in both the Route 95 and Route 81 bridge tests were analyzed statistically through the use of an electronic digital computer to determine average axle and gross weights and average axle spacings. Only the gross weight data from the Route 95 test were considered valid, because of an inconsistency in recording the data for the individual axles, but the average axle weights and spacings obtained during the Route 81 test are considered representative. The magnitudes of the truck populations differ slightly from those discussed earlier, but the error is not considered to be serious.

The average axle weights and spacings obtained for each truck type during the Route 81 test are shown in Table V, which also shows the standard deviation of the population of values in each case. Similar data for the gross weights recorded during the Route 95 test are shown in Table VI. Statistical comparisons of gross weights recorded at each bridge based on the means and standard deviations shown in Tables V and VI indicated that the populations of 2D, 2S-2, and 3S-2 trucks and the combined values differed significantly at the 95 percent confidence level between the two sites. The comparisons were based on the assumption of a normal distribution, although the magnitude of the standard deviation relative to that of the mean indicates that the distributions may, in fact, be slightly skewed. The statistical difference is probably due to the fact that the large populations defined the mean quite accurately; the practical effect of differences between the two sets of data is considered insignificant.

The data indicate the relative importance from a design viewpoint of the 3S-2 truck, first, because it has significantly heavier axle and gross weights and, second, because it is the most prominent combination, accounting for more than 50 percent of the total truck population in each case. It can also be seen that the average axle spacings of the 3S-2 truck are generally similar to those of the other three axle combinations.

TABLE V

AVERAGE AXLE AND GROSS WEIGHTS AND AVERAGE AXLE SPACINGS (WITH STANDARD DEVIATIONS SHOWN IN PARENTHESES) FOR TRUCKS IN THE ROUTE 81 BRIDGE STUDY

Truck Type	2D	3	2S-1	2S-2	3S-1	3S-2	All Types
Number of Trucks	272	26	139	473	2	1,591	2,503
% of Population	10.9	1.0	5.6	18.9	0.1	63.6	100.0
Average Weights (kips)							
Front Axle	4.70 (2.23)	7.16 (2.27)	7.15 (1.33)	8.07 (1.25)	6.55 (0.21)	8.66 (1.00)	8.01 (1.77)
Mid Axle Group	8.39 (3.90)	15.19 (8.25)	11.54 (7.39)	13.20 (3.50)	22.70 (0.71)	23.40 (6.69)	19.10 (8.38)
Rear Axle Group	—	—	10.98 (8.30)	17.25 (6.23)	7.95 (0.78)	22.80 (7.46)	18.37 (10.12)
Gross Weight	13.08 (5.51)	22.35 (9.99)	29.67 (15.93)	38.51 (9.86)	37.20 (0.28)	54.86 (13.96)	45.48 (18.95)
Axle Spacings (feet)							
Front-Mid Axles	14.0 (4.46)	14.0 (3.05)	11.0 (1.30)	11.0 (3.64)	14.0 (1.41)	12.0 (2.83)	12.0 (3.24)
Mid-Rear Axles	—	—	29.0 (7.23)	27.0 (7.04)	20.0 (7.07)	30.0 (4.67)	26.0 (10.34)
Overall	14.0 (4.46)	14.0 (3.05)	40.0 (8.47)	39.0 (5.55)	34.0 (8.49)	42.0 (7.36)	38.0 (11.15)

TABLE VI

AVERAGE GROSS WEIGHTS (WITH STANDARD DEVIATIONS SHOWN IN PARENTHESES) FOR TRUCKS IN THE ROUTE 95 BRIDGE STUDY

Truck Type	2D	3	2S-1	2S-2	3S-1	3S-2	All Types
Number of Trucks	542	82	432	1,725	7	3,029	5,817
% of Population	9.3	1.4	7.4	29.7	0.1	52.1	100.0
Gross Weight	14.63 (5.44)	27.54 (12.32)	27.92 (7.69)	41.14 (10.55)	35.76 (11.60)	53.37 (14.90)	43.86 (17.59)

DISCUSSION OF RESULTS

The primary purpose of this study was to evaluate the stresses caused by service loadings on two highway bridges in Virginia. Such a study is now feasible through the use of the data acquisition system developed for the Federal Highway Administration. The equipment functioned satisfactorily and provided an indication of the service lives to be expected of the two instrumented structures under today's truck traffic.

The stresses recorded in the beams of the instrumented spans are considered acceptable, with respect to both the mangitude of a single loading and the effects of repeated loadings. The steel beam composite span on Route 95, in which the vast majority of the measured stress ranges were below 2,250 psi, can accommodate the expected increase in the volume of traffic under current load limits. It must be remembered, however, that fatigue considerations include both the frequency of the stress range and its magnitude. Should allowable loadings be increased significantly beyond the current maximum gross or axle weights, the fatigue life could become critical. The performance of the prestressed concrete beam span on Route 81 appears to indicate that such structures are not as prone to fatigue limitations of service life as are the less massive steel beam bridges. The current state of knowledge in this new area of research is such that care should be exercised in extrapolating the results of this limited study to other bridges. Continuous span bridges can be expected to exhibit different service life characteristics, as may simple span bridges designed under other philosophies in other states.

It should also be realized that while this study concentrated on the main supporting elements, strain ranges of comparable magnitude were recorded in the diaphragm and ranges of greater magnitude were measured in the deck reinforcement of the Route 95 bridge. The effect of stresses in these members on the service life of a structure deserves consideration.

An interesting aspect of this study was the sensitivity of the beams to the pattern of service loads. It is, of course, expected that the beams under the load are the most highly stressed, and this fact, coupled with the tendency of truck drivers to remain in the right-hand lane when traffic permits, results in more stress occurrences in the beams under the right-hand lanes and many less occurrences in the far beams. Future studies could concentrate on these critical beams, using more gages at selected points on a single member.

The effect of secondary stresses produced by a single vehicle proved to be more critical in the more flexible steel beam span on Rte. 95, at which an average of approximately 1.5 occurrences per vehicle were recorded. A maximum of 1.1 events per vehicle was recorded at the prestressed concrete span on Rte. 81, and less than one event above 43.8 psi per vehicle was recorded in the heavily loaded beam. Data obtained in this study indicate that the influence of secondary stress ranges on service life may be negligible in massive concrete structures, but the effect must be considered in flexible structures.

The sampling periods for both structures included in this study were in September, chosen for the practical reason that manpower was available at that time, and no attempt was made to determine the effect of monthly or seasonal traffic variations on either bridge. However, the sampling periods for both structures included Sunday night, a period of peak northbound truck traffic, and it is believed that an evaluation of monthly or seasonal variations in the volume of trucks would not have affected the significance of the results of the study. The average weights and axle spacings presented earlier are believed to be representative of heavy vehicles on Virginia's highways.

While certain refinements are in order, the analytical methodology utilized in this study does appear to present a valid approach to correlating truck characteristics and bridge response. The theory, which employs realistic and accepted methodology, did verify the magnitude of the strain ranges recorded in the field. It is possible that a refined analytical approach of this type, using an appropriate traffic loading input, could be applied to a structure which could then be checked experimentally if the computations indicated the existence of high stress ranges.

CONCLUSIONS

Several conclusions are warranted by the results of this study.

1. The steel beam composite span instrumented in this study is adequately designed for the magnitude and frequency of the truck traffic to which it is subjected. The stress ranges measured on the structure are such that an increase in traffic under current load limits can be safely accommodated. However, since fatigue life is a function of the number of loadings, which will increase, as well as the magnitude of the stress range, the results of this study should not be interpreted as a justification to raise allowable weight limits.

2. The low stress ranges measured in the prestressed concrete beam span instrumented in the Route 81 tests indicate that fatigue may not be a problem in massive prestressed concrete bridges of moderate span.

3. The analytical method utilized in this study verified the accuracy of the strains recorded by the data acquisition system used in the field. The equipment performed satisfactorily.

4. The effect of the secondary stress ranges produced by a single vehicle is important in the case of a relatively light, flexible structure such as a composite steel beam span. The effect may be negligible in the case of more massive structures such as the prestressed beam span tested in this study.

5. The sensitivity of a typical bridge structure to the position of an applied load, coupled with the tendency of truck drivers to remain in the right-hand lanes whenever possible, would allow the critical beam in a structure to be determined in many cases. Future studies could concentrate gages on this structural element.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the interest and cooperation of J. N. Clary, Bridge Engineer, Virginia Department of Highways, in the initiation of this study.

The contributions of William L. Armstrong, Ronald R. Salmons, and Harry R. Laatz of the Federal Highway Administration, and Marvin H. Hilton, William A. Carpenter, Vincent J. Valenti, and John J. Hagen of the Virginia Highway Research Council to the success of this research are greatly appreciated.

Completion of the field work required the assistance of many employees of the operating divisions of the Virginia Department of Highways, including E. L. Tidd, Jr., Associate Traffic Engineer; H. C. Crowder, Resident Engineer at Edinburg; and G. H. Roberts and J. S. Hodge, formerly Resident Engineers at Manassas. Particularly, the excellent cooperation of the crews of the Dumfries and Stephens City weighing stations was invaluable. The kindness of Carl G. Brown, who allowed access through his property adjoining the Cedar Creek bridge, is also appreciated.

The research was conducted under the general supervision of Jack H. Dillard, Director of the Virginia Highway Research Council; and Charles F. Scheffey, Director, Office of Research, and Ernest G. Wiles, Deputy Chief, Structures and Applied Mechanics Division, of the Federal Highway Administration.

REFERENCES

1. Galambos, C. F., and W. L. Armstrong, "Acquisition of Loading History Data on Highway Bridges", Public Roads, Volume 35, No. 8, June 1969, U. S. Department of Transportation, Washington, D. C., pp. 177-189.

2. Galambos, C. F., and C. P. Heins, Jr., "Loading History of Highway Bridges — Comparison of Stress Range Histograms", Paper presented to the 50th Annual Meeting of the Highway Research Board, Washington, D. C., January 1971.

3. "Average Daily Traffic Volumes on Interstate, Arterial, and Primary Routes", Commonwealth of Virginia, Department of Highways, 1968, 1969, and 1970.

4. "Report of Weighing Operations, December 31, 1970, Thirty-One Years", Virginia Department of Highways, Traffic and Planning Division, Richmond, Virginia, January 1971.

5. "Highway Laws of Virginia" (Reprint from the Code of Virginia and the 1970 Cumulative Supplement, issued by the Virginia Department of Highways), The Michie Company, Charlottesville, Virginia, 1970, Section 46.1-339.

6. A.C.I. Standard Building Code for Reinforced Concrete, A.C. 318-63, American Concrete Institute, Detroit, Michigan, 1963, Section 1102.