ProQuest Information and Learning
300 North Zeeb Road, Ann Arbor, MI 48106-1346 USA
800-521-0600

UMI®

LOADING SPECTRUM FOR HIGHWAY BRIDGES
AND
DAMAGE CRITERIA FOR CUMULATIVE LOADING

by

Chun-Kong Kwok

Submitted in partial fulfillment
of the requirements for the degree of
Master of Engineering

Department of Civil Engineering
School of Graduate Studies
University of Ottawa
Ottawa, Canada
July, 1972

ACKNOWLEDGEMENTS

The work presented in this thesis was carried out under the direction of Dr. N. J. Gardner, Department of Civil Engineering, University of Ottawa.

The author wishes to express his deep appreciation to Dr. N. J. Gardner, for his guidance, constant encouragement, inexhaustible patience during the periods of difficulty and invaluable suggestions made in the writing of this thesis.

For the encouragement during the writing of this thesis, the author extends his sincere appreciation to his wife Siu-Ling, and to Mrs. W. Storto for the typing of this thesis.

ABSTRACT

Bridges are presently designed to withstand the bending moments and shear forces produced by a standard design truck such as AASHO H20-S16. However, bridges are subjected to a large number of loads of different magnitude randomly ordered with respect to time. This thesis describes a literature summary to obtain a description of the magnitude of loads and number of occurrences (load spectrum) on highway bridges. Secondly, the behaviour of bridges under random fatigue load was investigated.

From the data obtained at weigh stations of the gross weight and axle loads of actual trucks, a general description of the load spectrum was obtained. By applying factors to allow for regional trends and local factors a design spectrum could be obtained for any particular bridge.

Literature concerning the damage criteria of bridges is very limited and no criteria has been found which satisfactorily summarizes the cumulative damage to bridge materials by random loads.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF ILLUSTRATIONS

CHAPTER 1

INTRODUCTION

The load capabilities of the hard surfaced roads frequently exceeded the carrying capabilities of trucks in the early days of automotive freight transport. Today, however, improved technology has provided the trucking industry with vehicles that have load carrying capabilities greatly in excess of the allowable loads permitted by law.

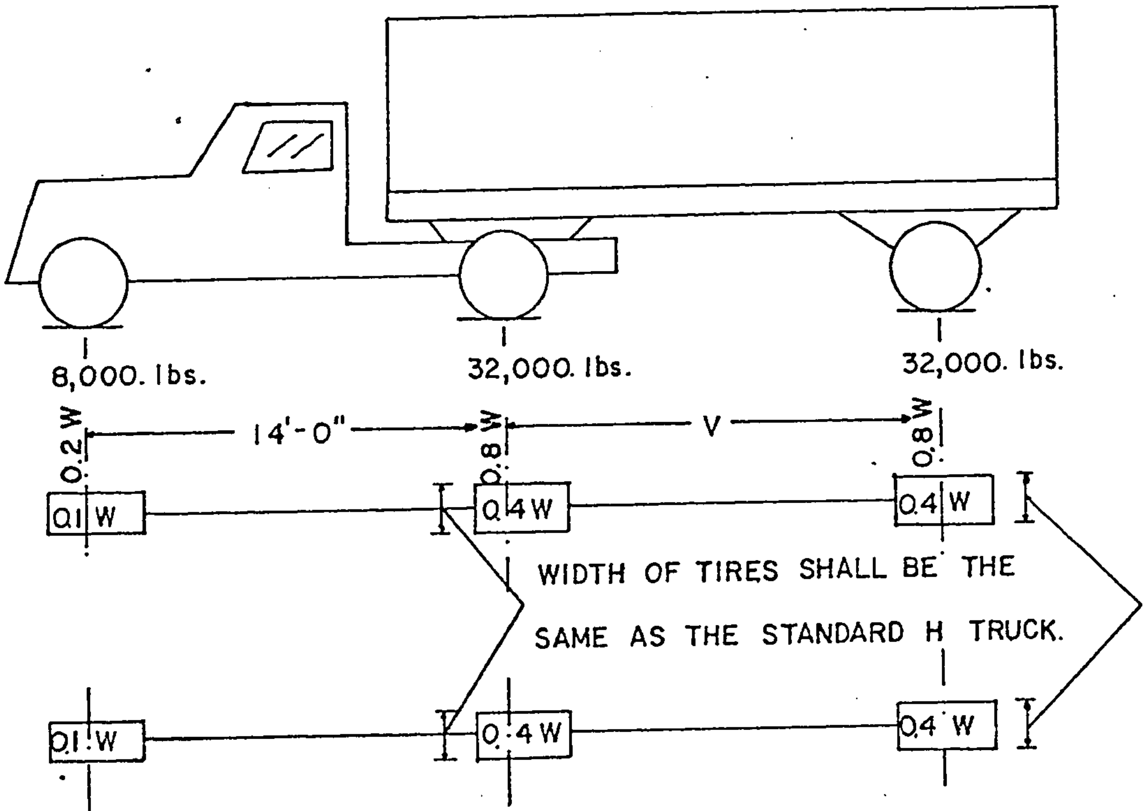
Engineers agree that the cost of highway provisions increases with each increase in maximum permissible axle loads and also with the anticipated number of occurrences of the various intensities of such loads. They are also in agreement that existing highways can be upgraded and that new roads with greater load carrying capacity could be built, but only at an increased cost. Easy or economical methods of upgrading bridges for either increased axle loads or gross load do not exist. This means that bridges, in general, have to be built originally to carry the maximum gross loads and wheel loads that will occur with consistent daily frequency over the anticipated life of the bridge. Any existing bridges of low load capacity generally must be replaced on a road system which has a considerable amount of daily truck traffic. Therefore, any liberalization of weight and size regulations must be gradual and evolutionary to avoid obsoleting vast mileages of highways and large numbers of bridges, either functionally or structurally, by legalizing excessively heavy vehicles.

Before the legislation can be changed it is necessary to know the distribution of vehicles (types) on existing structures and the loads they impose upon the structure. The permissible loads of vehicles are governed and limited by the provincial or state highway authorities. Generally, the AASHO standard vehicles HS and H are used. The limitation of the vehicle height, length, width and wheel-base are as shown in Figure 1 and Figure 2 [1]. Violation of vehicle limits is a problem for highway bridges; even though vehicles are controlled by weigh stations a significant percentage are still overweight.

Additionally, the life of the bridge is dependent upon the weight of the vehicle; number of load occurrences (fatigue); and vehicle arrival patterns which affect the fatigue life of the bridge (damage criteria). Consequently, bridges have to be designed for random fatigue loads.

The object of this thesis was to search the literature to determine the actual load spectrum for highway bridges under present legislation. Secondly, it was desired to determine if the combined load spectrum and the fatigue properties of the bridge (material) could be used to predict the life of the bridge.

H20-S16



WIDTH OF TIRES SHALL BE THE SAME AS THE STANDARD H TRUCK.

W = COMBINED WEIGHT ON THE FIRST TWO AXLES WHICH IS THE SAME AS FOR THE CORRESPONDING H TRUCK.

V = VARIABLE SPACING - 14 FEET TO 30 FEET INCLUSIVE SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM STRESSES.

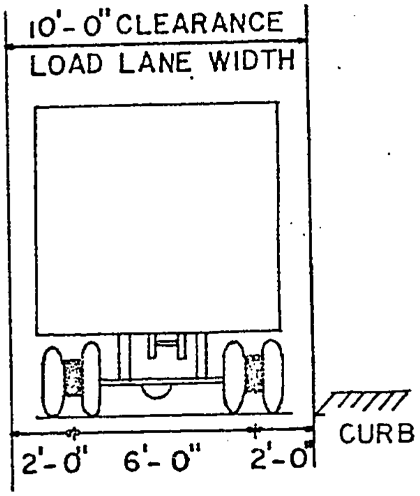


FIG. I STANDARD H-S TRUCKS

REF. NO. I.

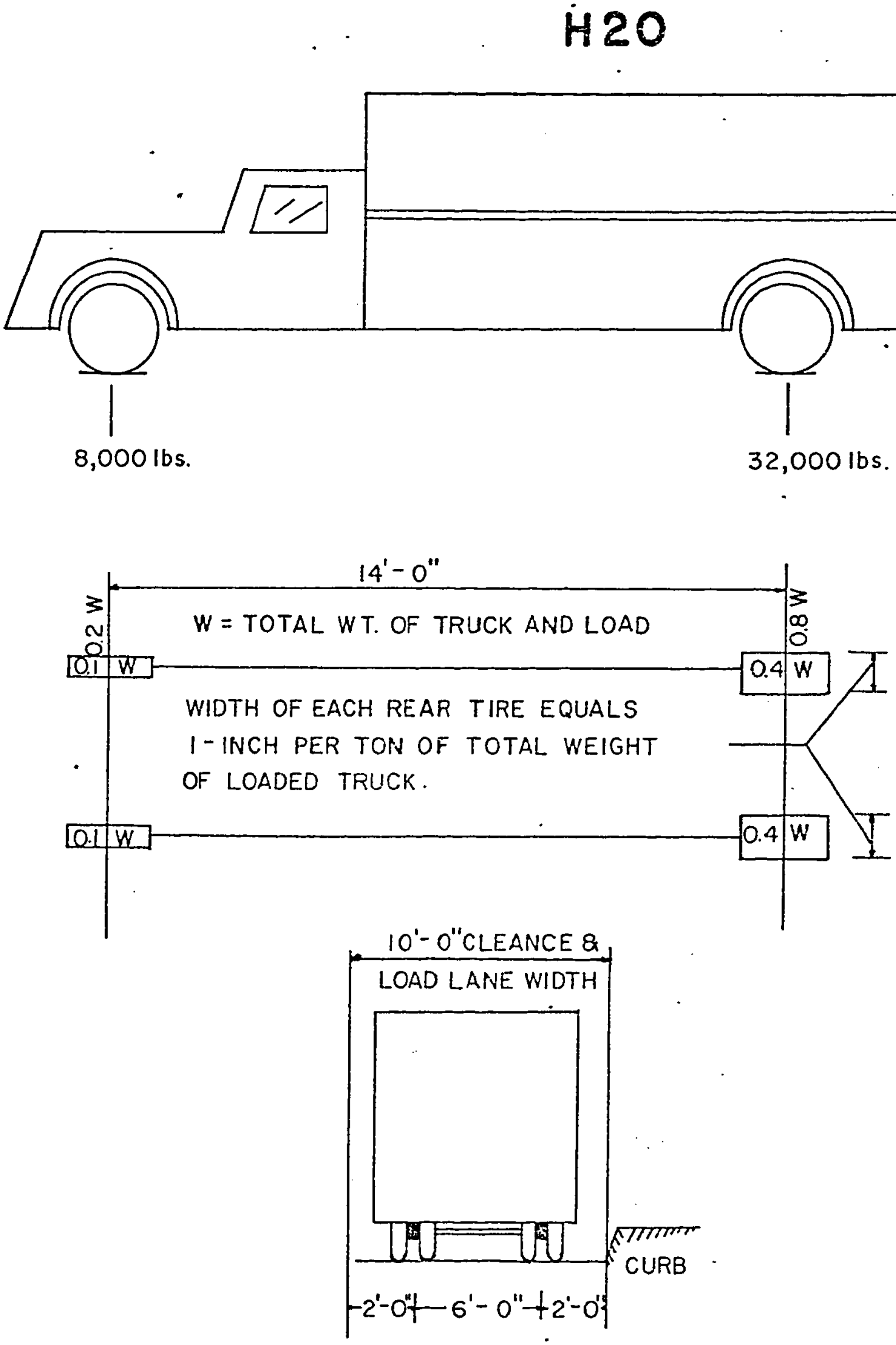


FIG. 2 STANDARD H TRUCK

REF. NO. 1.

CHAPTER 2

HISTORICAL REVIEW OF VEHICLES WEIGHT REGULATION

The American Association of State Highway Officials [2], which has in its membership representatives of all the State Highway agencies in the United States, has for many years represented the highway community in the development and recommendation of automotive vehicle weight laws. The goal of AASHO in recommending weight and size limitations for legislative consideration is to strike an optimum balance between the best and maximum use of the highway investment by obtaining a reasonable life expectancy of the facility.

In 1932, AASHO first adopted a policy on maximum dimensions, weights and sizes of vehicles. This policy permitted a single axle weight of 16,000 pounds. The allowable tandem axle and gross weight were both determined from the following formula:

$$W = c(b + 40) \qquad 1.1$$

where

W = the total weight in pounds.

c = a coefficient determined by individual states, with a minimum value of 700.

b = the distance between the first and last axle, in feet.

In 1944, the transport committee of AASHO gave initial consideration to a revision of the 1932 policy. This was after they had developed an interim substitute recommendation, dated May, 1942, that was applicable for the duration of the War Emergency. In 1946, a new policy was formulated (see Appendix A)[3]. This policy increased the permissible single axle load from 16,000 pounds to 18,000 pounds. A fixed value of 32,000 pounds for tandem axle load replaced the formula methods. While the formula was retained for determining the permissible gross weight, it was modified as shown below:

$$W = 1025(b + 24) - 3b^2 \qquad 1.2$$

In 1964, the AASHO increased the permissible single axle load from 18,000 pounds to 20,000 pounds but retained the 32,000 pounds permissible tandem load. The gross weight formula, was revised to include consideration of the number of axles, as follows:

$$W = 500(\frac{bN}{N-1} + 12N + 32) \qquad 1.3$$

where

N = the number of axles.

Within the past year, (1971) AASHO has considered increasing the tandem axle load from 32,000 pounds to 34,000 pounds, but this proposal was defeated by the member states. As a result, a revised 1968 AASHO policy has now been printed, which has not been changed from the 1964 version except in some minor respects. The increase in the allowable single axle load from 16,000 pounds in 1932 to the present 20,000 pounds value, and the increase in permissible gross weights reflected in the formulas are additional evidence of the overall trend in the liberalization of the vehicle weight laws.

In contrast to the AASHO proposal, the Ontario Department of Transportation and Communications [4,5] regulations encourages distribution of axle weights with regard to the moment and stresses induced in the bridge structures rather than the use of more axles. The maximum permissible weight W is therefore not directly a function of the base length b, as in the equation quoted above but of a modified base length, 'equivalent base length' B_m

$$B_m = kb \qquad 1.4$$

Where k is a coefficient between 0.9 and 1.75 depending on the distribution of axle weight. The following formula for the gross weight consists of the following second degree polynomial:

$$W = 20 + 2.07\, B_m - 0.0071\, B_m^2 \qquad 1.5$$

Since the 'equivalent base length' B_m varies directly as the distribution coefficient, this means that the distribution of axle weights is greatly dependent on k as indicated by Eq. 1.5. It will be evident that it is generally more favourable to shift the weights towards the ends of the base length b rather than towards the middle.

CHAPTER 3

EFFECTS OF VEHICLE PARAMETERS

With the growth of motor transport, the significance of many roads altered. Improvements to the roads were needed to suit the faster vehicles. It is inadequate to think of designing bridges to carry vehicles in the same way as roofs are designed to carry snow. The vehicle and bridge, and for that matter the rest of the highway, are in fact all components in a transportation/communication system, and all the components are subject to adjustment to produce optimum system performance. In this sense, legal limits on vehicle size and weight, and perhaps on speed, come to form with the highway bridge design specifications a unified though not necessarily coherent set of constraints on the total design problem. Optimum vehicle weights and sizes could be related logically to transportation economics and the economics of the requisite pavements and bridges and the pattern of taxation required for their provision and maintenance. It is for safety and economy that in addition to load limits, there be limits on the width, height and length of vehicles and their loads. The limitations required are mentioned below.

(i) A width limitation is required to fix pavement lane widths which determine the overall widths of roads and bridges.

(ii) Limitations in height are based on safety considerations and on existing overhead clearances.

(iii) The length of a vehicle, in conjunction with certain controlling dimensions lengthwise, such as wheel-base, and overhang, can affect the pavement width required on curves and also influences intersection layout. At some bridges, clearances between opposing vehicles are reduced and if vehicle widths were not limited it would be necessary to introduce one lane operation in many cases.

(iv) Wheel base distributions.

The wheel base is one of the factors controlling the vehicle weight. The wheel base limits range from 14 feet, 34 feet and 55 feet, for 3, 4, and 5 axles respectively. The vehicles with the shorter total wheel bases are designed to carry the heavier, more dense loads, such as steel plates and highway building materials; the vehicles with the longer wheel bases carrying lighter materials, such as furniture, tobacco, and hay.

(v) Distribution of trucks by type.

Most highways carry traffic composed of automobiles, buses, and trucks. This mixed traffic produces a very light average lane loading. Generally, the maximum proportion of automobiles to heavier vehicles is in the ratio of 4:1, [6] and will produce, under ordinary conditions, a lane load of between 150 pounds per linear feet and 250 pounds per linear feet when operating at a very slow or zero speed with an allowance of about eight feet between vehicles. The types of vehicles comprising the traffic on the various highways are quite varied, principally because of the individual provincial or state regulations on the size and weight of commercial carries. Vehicle weights are generally controlled by making trucks pass over loadometer stations. A record is kept of the gross vehicle weights, axle loads and truck types. From these records the distribution of truck types can be determined. However, in general truck designations are different in different provinces and this information can not be combined. In addition, the distribution of loads recorded at a loadometer station removed from a particular bridge site may not be indicative of the distribution of traffic on that bridge.

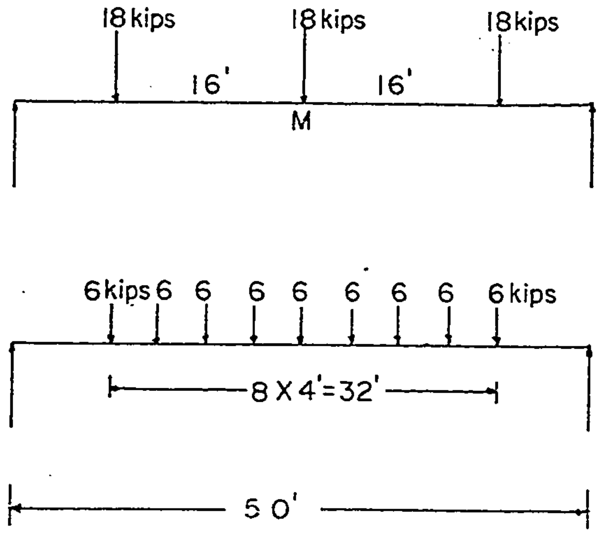
(vi) Distribution of Mean Gross Weight.

A statistical analysis can be performed on the loadometer data to test the interrelationships between gross weight, truck type, and loadometer station. The mean gross weight can be calculated for each truck type at each station. The sample size generally is adequate to provide a good estimate of the mean gross weight. Due to differences in mean gross weight between truck types and loadometer stations, different distributions have to be established for each test section.

Although mean gross-loads are reasonable measures of the central tendency of the gross-load distributions, mean gross-loads should not be input into a fatigue analysis because the fatigue load relationship is exponential. Usage of mean gross-loads rather than gross-load distributions would result in extremely low fatigue contributions because the mean of the loads does not give due consideration to the numbers of loads exceeding the mean.

(vii) Axleload Distribution.

Axleload distributions are a function of axle position, vehicle type, and bridge location. Although gross vehicle load is the most applicable loading parameter used in the fatigue analysis of large bridge structures, axleloads are often used for the fatigue analysis of certain bridge deck members. Different axleload groups with the same maximum permissible weight may generate very different moments, depending on the number and distribution of axle loads. In general, for the same bending moment, the maximum permissible weights must decrease as the number of axles increase, for the same base length. This can be verified by the following example:



Thus the bending moment due to 3 axles is:

$$M = 27 \times 25 - 18 \times 16$$
$$= 675 - 288$$
$$= 387 \text{ kips} \times \text{ft}$$

and the bending moment due to 9 axles is:

$$M = 27 \times 25 - 6(16 + 12 + 8 + 4)$$
$$= 675 - 6 \times 40$$
$$= 435 \text{ kips} \times \text{ft.}$$

From this example it is concluded that on the same span, and for the same bending moment, larger axle groups within the same base length must have a lower permissible weight [4].

In any vehicle, the groups of axles from axle 2 to and including the last axle of the vehicle govern the allowable vehicle weights. The load on axle 1 generally does not exceed 8,000 pounds. Thus, in a 5-axle vehicle, if all the various 2-axle, 3-axle, and 4-axle groups in axles 2 to 5 inclusive meet the requirements of the formula, axles 2 to 5 inclusive may weigh 72,000 pounds on a wheelbase of 48 feet, and the entire vehicle including axles 1 to 5 inclusive may weigh 80,000 pounds on a wheelbase of not less than 55 feet. All axle group loads shown in Table 1, however, are permissible for bridges having a design rating in excess of H 15 design. For visual comparison, these same permissible weights are shown graphically in Figure 3.

(viii) Weight Limitations.

Weight limitations usually draw a distinction between the loading transmitted through axles fitted with single wheels, axles fitted with dual wheels, and groups of axles. While weight limitations are intended primarily to limit the damage by vehicles to the road itself, they also reduce the tendency for vehicles to be loaded beyond the capacity of braking equipment and tires giving drivers more chance of controlling their vehicles in emergencies. The permissible gross weight of the vehicle as a whole is also dependent on the wheelbase of the vehicle, and might be affected by the spacing of the axles.

A comparison of the legal load for the American Association of State Highway Officials and the Ontario Traffic Act are listed in Appendix A and B. In Ontario, the Highway Traffic Act defines the maximum load per axle, and maximum load per unit. However, there is no restriction on the total number of units, or on the total weight of all units combined. There is nothing to prevent the use of vehicles made up of a large number of units together carrying an extremely large load (Figure 4). Although as far as it is known such vehicles have never been used, and are not likely to be used, it would be embarassing if they were: even the maximum load permitted in Ontario on a three-unit vehicle is much greater than would be permitted on a similar vehicle by model American loading regulations such as developed by the AASHO. It is interesting to compare the effects of H20-S16 design loads with the effects of maximum loads permitted by AASHO and Ontario regulations. Figure 5 shows the maximum bending moment per lane width produced in various spans by the different loads. It may be seen that the design load produces a considerably smaller effect than loads that would legally be permitted in Ontario [3].

CHAPTER 4

LOAD DISTRIBUTION

In 1953, Ivy and Lin [6] studied data derived from traffic studies of the lower deck of the San Francisco-Oakland Bay Bridge, in California (which is devoted entirely to heavy vehicular travel). The pattern of the traffic was measured during two 7-hour periods. The information recorded was as follows: (1) the time of departure from the tollgates; (2) classification of vehicles according to the number of axles; (3) the weight of the vehicle; and (4) the average speed of the vehicle during crossing.

The truck classifications, with diagrams of the typical vehicles for each type are listed in Figure 6. The wheelbase measurement for each typical vehicle is also shown. A typical pattern of truck loadings obtained from the survey is shown in Table 2. From this data it is interesting to note that the heaviest trucks did not generally follow one another. A loading of the heaviest trucks in regular pattern would be a rare natural occurrence or a planned, regulated event.

The actual maximum lane load obtained for the observed speed in the two-day record was only 145 pounds per linear foot of lane, which is much less than the 640 pounds per linear foot of lane specified by the AASHO. The average weight of vehicles using the lower deck of the San Francisco-Oakland Bay Bridge is known to be less than the average weight of trucks using the highways. Selecting heavy trucks weighed at loadometer stations have averaged about 20% heavier than the vehicles using the lower deck of the San Francisco-Oakland Bay Bridge.

The average length and weight of vehicles, and the total truck traffic recorded during the period of the survey is shown in Table 2. A total of 5,629 trucks passed through the tollgates during these two 7-hour periods, yielding data from which was derived an average truck weight of 14.6 kips and an average length of 23 feet. The average truck listed in Table 3 is a composite vehicle representing all of the trucks recorded during the survey. The lane loads found on the basis of the composite truck (shown in Table 4) are of interest because they apply only to extremely long spans. These lane loads would be the lower limits for any design live loading derived from the survey data. For shorter spans, the lane load would depend on the actual traffic pattern and would vary inversely with the loaded length.

In 1962, Stephenson and Noel [7] developed truck weight trends related to highway structures for the four regions of the United States; namely, the northeast, southeast, midwest, and west. These trends were developed using three points in time. For the first point, the loadometer data for 1942 and 1943 were combined. The 1948 and 1949 data were combined to form the second point in time, while the year 1954, alone, was used for the third point. Regional and national trends were developed for: (1) the distribution of traffic; (2) the distribution of freight carrying vehicles; (3) the distribution of heavy trucks; (4) the average axle load and axle spacings; (5) the average gross weight; and (6) the wheel-base length. It was recognized that truck traffic characteristics in the west were different from those in the east, a difference which would not be taken into account if national trends were used in more localized analyses. The data was collected at 500 to 600 loadometer stations distributed somewhat uniformly among the several states.

The contrasts between motor vehicle regulation in the eastern and western states are indicated in Table 5 which gives the composite limitations on heavy motor vehicle maximum size and weights for the years 1946 and 1958 as compared with AASHO Policy of 1946.

This data provides useful trends on a statewide basis but has one basic shortcoming. The combination of loadometer stations by highway system classification mark differences on individual routes and results in data that is not representative when evaluating the effect of truck loadings on a specific bridge in the highway system. Such information would be required to determine the effect of any proposed change in the vehicle weights on existing bridges.

Stephenson [8] assumed that 6,000 to 12,000 vehicles pass a bridge per day (speed approximately 40 miles per hour) which seems a rather low number in view of the increasing frequency of loading. On the assumption that heavy trucks amount to 5 percent of the total number of vehicles, amounting to 12,000 vehicles per day, half in each direction, this results in 6,000 x 0.05 x 365 x 50 = 5,500,000 heavy trucks loadings during the assumed useful bridge life of 50 years. In Table 6 particulars are given about "equivalent H-truck loading" for a span of 50 feet. (Note: the equivalent H-truck loadings provide a simple and effective index for comparing the bending stresses produced by a particular vehicle on a given span with those caused by a standard H-truck of specified weight on the same span. For example, if it were found that a particular heavy truck produced the same maximum bending moment on a 60-foot span as a standard H-truck weighing 45,000 pounds or 22.5 tons, it would be rated as an equivalent H 22.5 truck on a 60-foot span.).

Lynch [9] reported on loading frequencies and intensities on 21 bridges over the Ohio River during the years 1938-1966 and the following types of vehicles were considered: (1) passenger cars (72.34 and 82.3%); (2) buses (0.82 and 0.63%); (3) two-axle trucks (8.3 and 7.8%); (4) two-axle four-wheel trucks (7.9 and 4.8%); (5) three-axles single trucks (0.9 and 1.4%); (6) combination, 3-axle trucks (4.2 and 1.4%); (7) combination, 4-axle trucks (4.6 and 2.2%) and (8) combination, 5-axle trucks (1.0 and 0.4%). Classifications (1), (2) and (3) vehicles are four wheel vehicles. Type (4) vehicles have 4 wheels per axle and the same usually applies to the types (5) to (8). The first value given in the bracket for each type of vehicle gives the percentage in rural districts and the second value relates to urban districts. Thus this is a most valuable survey of an analysis with regard to vehicle types. Figure 8 summarizes the results from the 21 bridges by giving the variations of percentage of trucks with average daily crossing. Table 7 illustrates an analysis of this loading with regard to axle loads, giving the frequencies, showing that the entire truck loads amounted to approximately 10% of the entire number of vehicle loads.

Cudney [10] as part of his work on predicting the fatigue life of 8 highway bridges, recorded the frequency distribution of loads and distribution of truck types. A total of sixty-eight 6-hour sampling periods were taken on seven steel bridges and four six-hour periods on a prestressed concrete I-beam bridge. The truck type coding system utilized in this study is shown in Figure 9. The number of commercial vehicles that passed over the test span with respect to time was recorded and this data was used in conjunction with the traffic volume factor to estimate the average yearly commercial volume. The distribution of gross weight and truck types was obtained for bridges 1 and 2 where only the distribution of truck types was recorded for bridges 3-8.

Frequency distributions of gross vehicle load on bridge 1 and 2 for a six-hour time period on each of two days were collected. The maximum gross vehicle load for bridge 1 was 145 ± 5.5 kips, with an occurrence frequency of 1.8 percent. On bridge 2, the maximum vehicle load was 155 ± 5.5 kips occurring at a frequency of 1.6 percent. Maximum gross load, and corresponding frequencies with respect to vehicle type are given in Table 8. Approximately 85 percent of the 2,575 commercial vehicles crossing bridge 1 and 2,090 crossing bridge 2 were of the six vehicle types respresented in Figure 10.

In 1969, Galambos and Armstrong [11] chose a steel girder bridge with partial-length welded cover plates to investigate the loading history of highway bridges. It was a simple four-span, compositely designed bridge located in the northbound lane on Interstate 95 near Dumfries, Virginia. The site was chosen primarily because of the heavy truck traffic and the nearness to a truck weighing station. The bridge has a slight skew-angle, left forward at three degree 46 min. Only the northermost span of the four spans was tested.

The number of trucks was recorded for each month from January 1966 to December 1966 inclusive. The number of trucks counted during the test is shown in Table 9. The highest volume of truck traffic occurred during the summer months. Virginia vehicle regulations allows for a maximum gross truck weight of 70,000 pounds (there is an unofficial enforcement tolerance of 5 percent, which would raise the maximum gross weight to 73,500 pounds). From the data surveyed, the gross truck weight distribution was determined, as shown in Figure 11. The highest percentage of truck traffic weighed between 20 and 30 kips with only 5.6 percent being above 70,000 pounds. The truck traffic included two- to five-axle vehicles. The number and average gross weight of each axle group is given in Table 10.

Heins and Sartwell [12,13] discussed the problem briefly in connection with the Maryland State Roads Project on the New Vehicle Weight Law. Data from five bridges was used for correlation purposes with analytical programs and to provide some insight on vehicle loading patterns. Five bridges were tested in conjunction with the State's loadometer station survey, in order to obtain exact vehicle weights and dimension. Four of the bridges were monitored for a period of continuous twenty-four hours each. The fifth bridge was tested for a seven day period. The results from the monitoring of all of these five bridges are described in a series of histograms and tables.

Figure 13 presents the distribution of truck type determined in this survey. The first bridge, located on I-83 northbound has three simple supported spans of WF girders and a 7" reinforced concrete slab. The center span of length 47.0 feet was the test span.

The second bridge I-83 Southbound is parallel to the I-83 Northbound bridge. This bridge differs in the size of the girders, inclusive of cover plates, and is a composite structure. In order to examine the general trends of truck traffic, a classification count was conducted during a continuous seven day period. The survey was conducted from Sunday, October 27 through Saturday, November 2, 1968. Figure 14 describes the results for these weeks.

The third bridge used consisted of five, simple supported span. The test span is 38.0 feet long. The girders are composite and consist of 30 and 27 WF members with cover plates. Figure 15 describes a composite of the truck distributions for the entire 14.5 hour test period.

The fourth bridge consists of a 7" composite concrete deck supported by 36 WF 94 cover plated girders. The test span is 76'-7¼" long. Figure 16 lists the total distribution for the entire seven day classification counting period.

The fifth bridge consisted of three simple supported spans. The girders are WF sections with cover plates on the interior girders. The bridge has a 7" reinforced concrete slab with shear connectors along each girder. The north span length 41'-10", was the test span. Figure 17 presents the distribution of truck types during the entire week. During this period, approximately 65% of the entire truck traffic was made up of 2S2 and 3S2 type trucks.

In 1969, Heins and Sartwell [14] also presented results obtained from a three span continuous bridge located on U.S. 301 which was tested from July 10 through July 17, for approximately thirty hours. The distributions of truck types for the various test periods are shown in Figure 18. As can be observed, 3S2 type trucks constitute approximately 40% of the total truck traffic. The average daily traffic for this bridge was 10,500 vehicles. Truck traffic represents 17% of the total ADT, or 1,785 vehicles.

Unfortunately, the results of these various investigations cannot be summarized as the investigations were conducted in states with different specified vehicle capacities. However, the distribution of truck types usually has two peaks, one at the maximum specified load and one for small or medium capacity trucks. The distribution of gross vehicle weights is a unimodal type distribution with a peak at about 15 tons.

Increasing the maximum allowable vehicle weight will increase the magnitude of the loads at the higher end of the gross weight distribution curve.

CHAPTER 5

DAMAGE CRITERIA - THE PROBLEM OF FATIGUE

Fatigue is an old-age disease. Over the years a bridge may have been subjected to a large number of load occurrences, and of course, the older the bridge the more stress applications it has had and the more likely it may suffer from fatigue. Fatigue cracks will most likely be found where there is a high stress concentration such as at an abrupt change in geometry. These changes will be found at re-entrant cuts, holes and notches or at abrupt changes in stiffness. There can be no general criteria by which it can be said that one member may fail in fatigue and the other will not. The actual stress range and number of cyles are the important considerations. Members in which the stresses due to dead load are relatively small compared with those due to live load should receive the greatest attention with respect to fatigue, as fatigue failures are related mainly to the cycle component of load (live load).

Highway bridges are subjected to a great variety of forces, ranging from their own constant dead load, through slowly changing forces caused by material creep and temperature differentials, to an almost infinite variety of live loadings caused by moving vehicles. Most bridges are designed to carry a static load produced by a design truck and, in addition, certain empirical allowances are made for increased stresses caused by dynamic loads. These stresses are produced by the random loading on the bridge. Existing concrete bridges have been usually designed, under working load, either for zero tensile stress or a tensile stress up to $3\sqrt{f'_c}$ but not over 250 psi. Moreover, they must resist a static failure load, ensuring a load factor of 1.5 times the dead load plus 2.5 times the live load (including impact factor). Abeles [19] suggested that "there seems, however, no need to ensure a load factor to a static failure loading, if it were possible to design a highway bridge on the basis of the entire loading spectrum possible and probable during the expected bridge life with the varying intensities and frequencies." Thus, instead of considering two loading stages (working load and failure), it should suffice to base the design on the resistance to the cumulative effect of the fatigue loadings.

5.1 The Cumulative Effect of Fatigue Loading

Fatigue tests are generally carried out by cycling a specimen through a definite stress range until failure. In practice a bridge is subjected to many cycles of varying stress ranges. Consequently, some means of predicting the fatigue behaviour of members where the stress ranges vary and the order of application of the stress ranges is random. Miner's hypothesis [15] states that if an element is subjected to n_i stress cycles of a given range of which it can withstand N_i cycles then fatigue will occur when $\Sigma\frac{n_i}{N_i} = 1$. This is to say that the same fatigue resistance can be obtained for reduced numbers of cycles over greater loading ranges, provided that the sum of the proportions of applied cycles to the number of cycles that the member can sustain becomes one.

Cumulative fatigue failures have been discussed at the International Conference of Fatigue of Metals in London and New York, notably by Corten and Dalan [16] who summarized the causes of fatigue failure of metals as follows: (1) nucleation period to initiate permanent fatigue damage; (2) the number of damage nuclei (submicroscopic voids) increase with increase of stress; (3) damage at a given stress amplitude propagates with increase of cycles; (4) the rate of propagation of damage per cycle increases with increase of stress; (5) the total damage constituting failure is constant for all stress histories and (6) damage will continue to be propagated at stress levels that are lower than the minimum stress required to initiate damage. It was concluded that Miner's hypothesis is not adequate to handle the variety of cases with steel structures. However, no adequate theory exists at present.

5.2 Survey of Highway Bridges Fatigue Life

It is known a bridge is subjected to a random loading, which produces the stress history which controls the bridge life. If the total vehicles which pass the bridge per day or per year is known, then it is possible to predict how long a bridge can survive. For example, Stephenson (as mentioned previously) assumed 5 percent of the total number of the 6,000 to 12,000 vehicles which pass a bridge per day, will be heavy trucks and over the 50 years life of the bridge, a 50 foot span bridge would be subjected to 6,000 x 0.005 x 365 x 50 = 5,500,000 heavy truck loadings.

Cudney [10] proposed two Methods (Method 1 and Method 2) to predict the cumulative fatigue damage based from the fatigue curve. The fatigue curve represents an extrapolation beyond actual fatigue test values assuming that the stress range at 200 x 10^6 cycles is one-third the value at 2 x 10^6 cycles. Cudney calculated the fatigue life of 8 steel highway bridges using both methods and predicted the fatigue life to range from 530 years for bridge 8 by Method 2 to 105,000 for bridge 6 by Method 1. The results are shown in Table 11.

Galambas and Armstrong [11] tested four steel girder bridges in Dumfries, Virginia. The fatigue curve used was developed from the recommendations of the joint ASCE-AASHO Committee on Flexural Member. For rolled beams with cover plates using ASTM designation A36 steel, the Committee recommended a stress range value of 9,000 psi for fracture at 2×10^6 cycles. In House Document 35 [17] assumes that the stress range value at 200×10^6 cycles is equal to a third of the stress range value at 2×10^6 cycles. This assumption gives the stress range at 200×10^6 cycles of 3,000 psi. The stress range and cycles to failure are assumed to have a linear log-log relation as shown in Figure 19. The stress ranges recorded on the Dumfries bridge were not large enough to make a reliable estimate of its fatigue life. The highest stress range recorded was approximately 3,500 psi on beam 3, at midspan, and had an occurrence frequency of less than 0.01 percent. It is therefore impossible to make a prediction of the fatigue life of the bridge from these data.

Abeles [17] indicated that the fatigue resistance of prestressed concrete members over high stress ranges is appreciable and suggested that a satisfactory design solution against fatigue would be to ensure that the resultant stress at the tensile face remains compressive for millions of load cycles, whereas substantial nominal concrete tensile stresses and visible cracks may occur under a limited number of abnormal loadings, on the removal of which the crack completely close. Tests carried out at Duke University [18] have shown that more than 100,000 cycles can be safely sustained for a loading between 30 and 60 percent of SFL (Static Failure Load), with a load factor of 1.36 on the live load and almost 5,000 cycles for a loading between 30 and 80 percent SFL, with a load factor of 2.27 times the live load, when all strands were tensioned.

But by increasing the total area of steel, without increasing the prestressing force, fatigue resistance can be greatly increased. This seems to be advisable to apply to new designs in order to cope with the possibility of further increase in loading without increasing the cross section of the bridge beams.

The static ultimate load capacity of most prestressed concrete beams is controlled by the stress in the steel prestressing strands. Abeles stated that it is probable that the fatigue load capacity is also controlled by the steel strands and hence knowledge of the fatigue behaviour of the strand would define the fatigue behaviour of a prestressed concrete beam. However, the S-N curves and Goodman diagrams (as shown in Figure 20) for prestressing steel are available only up to 10×10^6 cycles, for which the stress range is $0.1\ f_s'$. It is assumed that for 500 million cycles the permissible stress range to avoid fatigue failure would be further reduced from $0.1\ f_s'$ to $0.05\ f_s'$. This would result in the concrete stresses still remaining in compression and relatively low. For loads which occur more frequently, but are of lower intensity, there should be a relatively large safety factor, the concrete remaining in compression.

It can be summarized that very little work has been carried out predicting the useful life of either steel or concrete bridge structures. It is necessary that the damage criteria for structural steel, reinforcing steel, prestressing steel and concrete be determined over low stress ranges. Subsequently for bridge members, the damage criteria should be modified to take account of member deterioration by chemical corrosion, aging effects, etc.

SUMMARY

Bridges are presently designed using a typical design truck such as H20-S16 developed by AASHO or the local legislating authority and using a static design philosophy together with an impact factor to allow for dynamic effects. However, bridges are subjected to dynamic loads of various intensities which occur on the bridge in a random manner. Materials or structural elements subjected to numbers of cycles of load are known to suffer from fatigue type failures. Basically, the problem falls into two main areas (1) determinations of the actual load distribution on bridges and relating it to the local permissible vehicle loads and (2) obtaining damage criteria for the various bridge materials under the probable stress states.

The distribution of loads on the various provincial or state highways can readily be obtained by analyzing the records at the various weighing or loadometer stations. Very little use has been made of the vast quantities of data available. However, from the published distributions it can be observed that trucks appear to be distributed mainly in two groups; small or medium size trucks or the largest size trucks permitted by legislation. The distribution of gross vehicle weights is the usual

unimodal distribution. However, local features, such as concentration of industry, should be taken into consideration when predicting the future traffic distribution on a proposed bridge. In general, the data exists to determine the load spectrum on any proposed or existing highway bridge. No 'damage criteria' theory exists which predicts the behaviour of structural materials under random loads. It is necessary that the damage criteria be established for concrete, structural steel, reinforcing steel and prestressing steel strand. Subsequently the useful life of highway bridges could be determined using the predicted load spectra and the appropriate damage criteria.

REFERENCES

1. CSA Standard S6-1966, "Design of Highway Bridges". Published May, 1966, by the Canadian Standards Association.
2. Morton, J. O., President, American Association of State Highway Officials, Statement presented in February 1968.
3. "The Evaluation of Highway Bridges", by D.T. Wright, Queen's University, Kingston, Ontario, Report No. Q6-1, November, 1956.
4. "Determining the Maximum Permissible Weights of Vehicles on Bridges" by F. W. Jung and A. A. Witechi, D.T.C. Report No. RR175, December, 1971.
5. "A Proposed Method of Regulating Vehicle Weights in Ontario" by M. D. Armstrong, F. W. Jung and W.A. Phang, D.H.O. Report No. RR166, September, 1970.
6. "Live Loading for Long-Span Highway Bridge" by R.J. Ivy and T. Y. Lin, ASCE Transactions, November 5, 1952, No.2708.
7. "Truck Weight Trends Related to Highway Structures" by H.K. Stephenson and J. S. Noel and A. D. Mayfield, Texas Transportation Institute, A and M College of Texas, July, 1962.
8. Stephenson, H. K., "Frequencies of Various Levels of Stress in Highway Bridges", Highway Research Board, Proceedings, Vol. 38, 1959.

9. Lynch, R. L., "Research of Traffic Loads on Bridges", Kentucky Dept. of Highways, Division of Research, Lexington, February, 1968.

10. "The Effects of Loadings on Bridge Life" by G. R. Cudney, Michigan Department of State Highways, HBR 251-260, 1968-1969.

11. "Loading History of Highway Bridge" by C.F. Galambos and W.L. Armstrong, Bureau of Public Roads, Federal Highway Administration, HBR 291-300, 1969.

12. "Tabulation of 24-hour Dynamic Strain Data on Four Simple Span Girder-Slab Bridge Structures" by C.P. Heins, Jr. and A.D. Sartwell, for Maryland State Roads Commission and U.S. Bureau of Public Roads, Report No. 29, June, 1969.

13. "Tabulation of Dynamic Strain Data on a Girder-Slab Bridge Structure During Seven Continuous Days" by A.D. Sartwell and C.P. Heins, Jr., for Maryland State Roads Commission and U.S. Bureau of Public Roads, Report No. 31, September, 1969.

14. "Tabulation of Dynamic Strain Data on a Three Span Continuous Bridge Structure" by A.D. Sartwell and C.P. Heins, Jr., for Maryland State Roads Commission and U.S. Bureau of Public Roads, Report No. 33, November, 1969.

15. Miner, M. A., "Cumulative Damage in Fatigue", Transactions 67, American Society of Mechanical Engineers, 1945.

16. Corten, H. T. and T. J. Dolan, "Cumulative Fatigue Damage", Conference on Fatigue of Metals, Institute of Mechanical Engineers, London, 1956.

17. Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal Aid System, 88th Congress, 2nd Session, U.S. Government Printing Office, Washington, D.C., House Document 354, August 19, 1964.

18. "The Expected Fatigue Life of Prestressed Concrete Highway Bridges as Related to the Expected Load Spectrum" by P. W. Abeles and E. I. Brown II, 2nd International Symposium on Concrete Bridge Design, Chicago, Illinois, April, 1969.

19. Abeles, D. W., E. I. Brown, II and J. M. Slepetz, "Fatigue Resistance of Partially Prestressed Concrete Beams to Large Range Loading", International Association for Bridge and Structural Engineering, Prepared Discussion, Theme IVb, 8th Congress, New York, 1968.

20. "Statistical Analysis of Heavy Truck Traffic for the State of Maryland" by C.A. Pryor, Jr., C.P. Heins, Jr., and C.T.G. Looney, for Maryland State Roads Commision and U.S. Bureau of Public Roads, June, 1967.

21. "Stress Histories of Highway Bridges" by R. Gene and M. Cudney, Journal of the Structural Division, p.2725, December, 1968.

22. The AASHO Road Test, "History and Description of Project", Highway Research Board, Special Report 61A, 1961.

23. The AASHO Road Test, "Bridge Research", Highway Research Board, Special Report 61D, 1962.

TABLES

Table 1

Permissible Axle Group Loads in Pounds by Formula $W=500(\frac{Nb}{N-1}+12N+32)$

Wh. Base	Number of Axles - N							
B-Feet	2	3	4	5	6	7	8	9
4	32,000							
5	33,000							
6	34,000							
7	35,000							
8	36,000	40,000						
9		40,750						
10		41,500						
11		42,250						
12		43,000	48,000					
13		43,750	48,670					
14		44,500	49,330					
15		45,250	50,000					
16		46,000	50,670	56,000				
17		46,750	51,330	56,630				
18		47,500	52,000	57,250				
19		48,250	52,670	57,880				
20		49,000	53,330	58,500	64,000			
21		49,750	54,000	59,130	64,600			
22		50,500	54,670	59,750	65,200			
23		51,250	55,330	60,380	65,800			
24		52,000	56,000	61,000	66,400	72,000		
25		52,750	56,670	61,630	67,000	72,580		
26		53,500	57,330	62,250	67,600	73,170		
27		54,000	58,000	62,880	68,200	73,750		
28			58,670	63,500	68,800	74,330	80,000	
29			59,330	64,130	69,400	74,920	80,570	
30			60,000	64,750	70,000	75,500	81,140	

Note: Axle group loads for wheelbase lengths above the horizontal dashed lines are not permissible for bridges of H15 design. All tabulated axle-group loads are permissible for bridges having design ratings in excess of H15 design.

Table 1 - continued

31			60,670	65,380	70,600	76,080	81,710	
32			61,330	66,000	71,200	76,670	82,290	88,000
33			62,000	66,630	71,800	77,250	82,860	88,560
34			62,670	67,250	72,400	77,830	83,430	89,130
35			63,330	67,880	73,000	78,420	84,000	89,690
36			64,000	68,500	73,600	79,000	84,570	90,250
37			64,670	69,130	74,200	79,580	85,140	90,810
38			65,330	69,750	74,800	80,170	85,710	91,380
39			66,000	70,380	75,400	80,750	86,290	91,940
40			66,670	71,000	76,000	81,330	86,860	92,500
41			67,330	71,630	76,600	81,920	87,430	93,060
42			68,000	72,250	77,200	82,500	88,000	93,630
43			68,670	72,880	77,800	83,080	88,570	94,190
44			69,330	73,500	78,400	83,670	89,140	94,750
45			70,000	74,130	79,000	84,250	89,710	95,310
46			70,670	74,750	79,600	84,830	90,290	95,880
47			71,330	75,380	80,200	85,420	90,880	96,440
48			72,000	76,000	80,800	86,000	91,430	97,000
49				76,630	81,400	86,580	92,010	97,560
50				77,250	82,000	87,270	92,570	98,130
51				77,880	82,600	87,750	93,140	98,690
52	36,000 lb.maximum irrespective of wheelbase length	54,000 lb.maximum irrespective of wheelbase length	72,000 lb.maximum irrespective of wheelbase length	78,500	83,200	88,330	93,710	99,250
53				79,139	83,800	88,920	94,290	99,810
54				79,750	84,400	89,500	94,860	100,380
55				80,380	85,000	90,080	95,430	100,940
56				81,000	85,600	90,670	96,000	101,500
57				81,630	86,200	91,250	96,570	102,060
58				82,250	86,800	91,830	97,140	102,630
59				82,880	87,400	92,420	97,710	103,190
60				83,500	88,000	93,000	98,290	103,750

Ref. No. 4

Table 2

Typical Pattern

Typical Axle Type	Departure Time Hour	Min	Sec	Truck Wt in pounds	Avg.Speed MPH
2	8	34	25	3,400	-
2-S1	8	34	25	28,500	25.0
2	8	34	30	26,900	30.5
2-S1	8	34	40	20,000	31.0
2	8	34	55	26,500	-
3-S2	8	35	00	56,000	26.1

Ref. No. 6

Table 3

Average Weight and Length of Truck

Truck Type	Avg.wt (in lb)	No.of Truck	Total wt
2	4,604	2,429	11,184
2	11,022	1,322	14,571
2	23,553	608	14,320
2-S1	24,568	367	9,017
2-S2	32,909	248	8,161
3-S2	44,088	209	9,214
3	17,547	177	3,106
2-S1-2	45,263	159	6,292
Others	48,264	130	6,274
Total		5,629	83,139
Avg.	14,592		

Ref. No. 6

Table 4

Lane Capacities and Loads

Speed (mph)	Following Distance	Lane Capacity veh/hr	Lane Loads lb/ft
0	30.9	0	472
5	39.7	660	365
10	50.9	1,040	285
15	64.7	1,220	225
20	80.9	1,305	180
25	99.7	1,320	145
30	120.7	1,310	120
35	144.7	1,275	100
40	170.9	1,240	85

Ref. No. 6

Table 5

A Comparison of Eastern with Western Heavy Motor Vehicle Operation

	1946		1958		1946
	Eastern	Western	Eastern	Western	AASHO Policy
			Length of Vehicle in Feet		
Single Unit	38.7	36.4	40.0	37.5	35.0
Truck Tractor Semi-trailer	47.1	55.9	48.8	59.2	50.0
Other Combinations	49.5	58.6	54.0	60.8	60.0
			Axle Loads in Pounds		
Single	20,622	18,000	22,313	18,915	18,000
Tandem	32,250	29,475	36,140	33,194	32,000
			Practical Max. GVW, Pounds		
Two-Axle Truck	29,767	26,083	30,285	26,877	26,000
Three-Axle Truck	40,917	38,375	43,783	40,717	40,000
3-Axle Truck Tractor Semitrailer	45,883	43,500	51,562	46,062	44,000
4-Axle Truck Tractor Semitrailer	47,158	52,708	61,433	59,813	55,470
5-Axle Truck Tractor Semitrailer	47,313	62,400	61,991	71,737	61,490
Other Combinations	53,886	69,442	65,591	75,772	71,900

Ref. No. 7

Table 6

Loading Spectrum for 50' Bridge

X	Fr	Fe	X	Fr	Fe
8	0.01	500	24	0.72	39,600
9	0.16	8,800	25	0.52	28,600
10	0.67	36,850	26	0.20	11,000
11	2.75	151,250	27	0.08	4,400
12	4.90	269,500	28	0.05	2,750
13	7.14	392,700	29	0.03	1,650
14	6.90	379,500	30	0.01	550
15	7.59	417,450	31	0.03	1,650
16	8.59	472,450	32	0.02	1,100
17	11.17	614,350	33	0.01	550
18	12.99	711,450	34	-	-
19	14.08	774,400	35	-	-
20	11.85	657,750	36	0.02	1,100
21	4.97	273,350	37	0.01	550
22	3.39	186,450	38	-	-
23	1.13	62,150	39	0.01	550

Ref.No. 8

X = equivalent H-loading in short tons.

Fr = relative frequency in % (probability in %).

Fe = entire frequency (Fe = 5,500,000 loadings).

(See also Figure 7)

Table 7

Loading Spectrum for 8 Bridges over Ohio River

Maximum Axle Load K	Fr %	Fex10^3	Fex10^6	Distribution of Loading	
< 9	94.6914	284,0742		Cars	85.14%
9-11	1.1881	3,564		Buses	0.57%
11-13	0.9241	2,772	293.591	2-axle trucks (4W)	4.69%
13-15	1.0603	3,181		3-axle trucks (6W)	4.82%
15-17	1.1584	3,475		3-axle single trucks	0.24%
17-19	0.7932	2,379			
19-21	0.1529	459	6.313	3-axle comb. trucks	1.30%
21-23	0.0206	62			
23-25	0.0069	20		4-axle comb. trucks	2.91%
25-27	0.0025	7.5			
27-29	0.0013	4	0.0135	5-axle comb. trucks	0.33%
>29	0.0007	2			
					100%

Fr = relative frequency in percent
Fe = entire frequency (300 million loading)
W = wheel

Ref. No. 9

Table 8

Maximum Gross Load and Frequencies

with Respect to Vehicle Type

Bridge	Vehicle Type	Maximum Gross Load (kips)	Maximum Gross Load Frequency (%)
1	2D	22.5±2.5	16.7
	2S2L	37.0±3.0	25.0
	2S1	45.0±4.0	44.1
	2S2	69.0±3.0	1.8
	3S2	70.5±2.5	3.9
	3S2L	77.5±2.5	7.1
	Others	142.0±6.0	12.5
2	2D	19.5±2.0	9.5
	2S1	32.5±3.0	23.5
	2S2L	43.5±3.0	20.0
	2S2	52.0±2.5	9.8
	3S2L	73.0±4.5	5.9
	3S2	74.5±3.0	11.1
	Others	154.0±8.0	10.1

Ref. No. 10

Table 9

Number of Trucks Weighed at Dumfries Weighing Station, Route I-95 Northbound

Month	Year 1966	Year 1967	Month	Year 1966	Year 1967
Jan.	41,459	36,678	Aug.	58,605	60,123
Feb.	42,149	42,823	Sept.	49,385	49,265
Mar.	49,028	50,840	Oct.	57,406	54,346
Apr.	53,841	43,954	Nov.	44,129	51,886
May	51,846	50,497	Dec.	26,409	45,355
June	58,322	60,176			
July	50,629	47,776	Total	583,208	593,719

Ref. No. 11

Table 10

Truck Type and Weight Distribution

Truck Type	No. of Trucks	% of Total Trucks	Avg. G. W(kips)
Two-axle	223	19.9	12.5
Three-axle	157	14.0	28.3
Four-axle	332	29.7	37.7
Five-axle	407	36.4	45.6

Ref. No. 11

Table 11

Tabulation of Fatigue Life Factors

Bridge	Annual Commercial Traffic Volume	Method 1 Stress Range (σ_r) Ksi	Frequency Range (±25) percent	Damage Cycle Range	Log N*	N	Damage Factor Range $\frac{\text{Damage Cycle}}{\text{N}}$	Annual Damage Range ΣDamage Factor	Fatigue Life Range, yr $\frac{1}{\text{Annual Damage}}$	Total Damage Cycle Range
		1	2	3	4	5	6	7	8	9
1	192,000	6.3	0.69 1.51	1,325 2.900	8.04	10.96×10^7	1.21×10^{-5} 2.64	2.19×10^{-5} 4.66	45,700 21,600	2,955 6,260
		5.7	0.85 1.75	1,630 3,360	8.22	16.60	0.29 2.62			
2	150,000	5.1	0.06 0.54	90 810	7.49	3.09	0.29 2.62	3.80 10.31	26,300 9,700	3,045 6,855
		4.5	0.33 1.07	495 1,605	7.72	5.25	0.94 3.05			
		3.9	1.64 2.96	2,460 4,440	7.98	9.55	2.57 4.64			

*Method 1

$\text{Log } N = 10.45 - 4.18 \log \sigma_r$ (Bridges 2,3,5,6 and 7)

$\text{Log } N = 11.38 - 4.18 \log \sigma_r$ (Bridges 1 and 8)

Ref. No. 10

Table 11 - continued

		Method 1								
		1	2	3	4	5	6	7	8	9
3	860,000	5.7	0.00 0.21	0 1,805	7.29	1.95	0.00 9.26	22.32 61.95	4,480 1,610	15,570 34,365
		5.1	0.10 0.50	860 4,300	7.49	3.09	2.78 13.92			
		4.5	0.56 1.24	4,820 10,660	7.72	5.25	9.18 20.31			
		3.9	1.15 2.05	9,890 17,600	7.98	9.55	10.36 18.46			
5	222,000	4.5	0.10 0.50	220 1,110	7.72	5.25	0.42 2.11	3.67 7.68	27,200 13,000	3,330 6,440
		3.9	1.40 2.40	3,110 5,330	7.98	9.55	3.25 5.57			
6	413,000	3.9	0.00 0.22	0 910	7.98	9.55	0.00 0.45	0.00 0.95	∞ 105,000	0 910
7	291,000	5.1	0.00 0.30	0 875	7.49	3.09	0.00 2.83	2.03 14.15	49,100 7,070	1,660 9,120
		4.5	0.12 1.08	350 3,145	7.72	5.25	0.66 5.99			
		3.9	0.45 1.75	1,310 5,100	7.98	9.55	1.37 5.33			
8	293,000	6.9	0.18 1.22	530 3,575	7.87	7.41	0.71 4.82	1.03 8.25	97,400 12,100	1,060 8,555
		6.3	0.00 0.48	0 1,405	8.04	10.96	0.00 1.28			
		5.7	0.18 1.22	530 3,575	8.22	16.60	0.32 2.15			

Table 11 - continued

		Method 2								
		1	2	3	4	5	6	7	8	9
1	192,000	6.3	0.69 1.51	1,325 3,900	8.28	$19.10\text{x}10^7$	$6.92\text{x}10^{-6}$ 15.15	$6.92\text{x}10^6$ 15.15	144,000 66,000	1,325 2,900
2	150,000	5.1	0.06 0.54	90 810	7.96	9.94	0.91 8.13	17.67 43.81	56,600 22,800	3,045 6,855
		4.5	0.33 1.07	495 1,605	8.12	13.12	3.76 12.21			
		3.9	1.64 2.96	2,460 4,440	8.28	18.88	13.00 23.48			
3	860,000	5.7	0.00 0.21	0 1,805	7.80	6.35	0.00 28.43	97.74 246.35	10,200 4,060	15,570 34,365
		5.1	0.10 0.50	860 4,300	7.96	9.94	8.65 43.26			
		4.5	0.56 1.24	4,820 10,660	8.12	13.12	36.71 81.28			
		3.9	1.15 2.05	9,890 17,600	8.28	18.88	52.38 93.38			
5	222,000	4.5	0.10 0.50	220 1,110	8.12	13.12	1.69 8.45	18.12 36.64	55,200 27,300	3,330 6,440
		3.9	1.40 2.40	3,110 5,330	8.28	18.88	16.44 28.19			

Method 2

Ref. No. 10

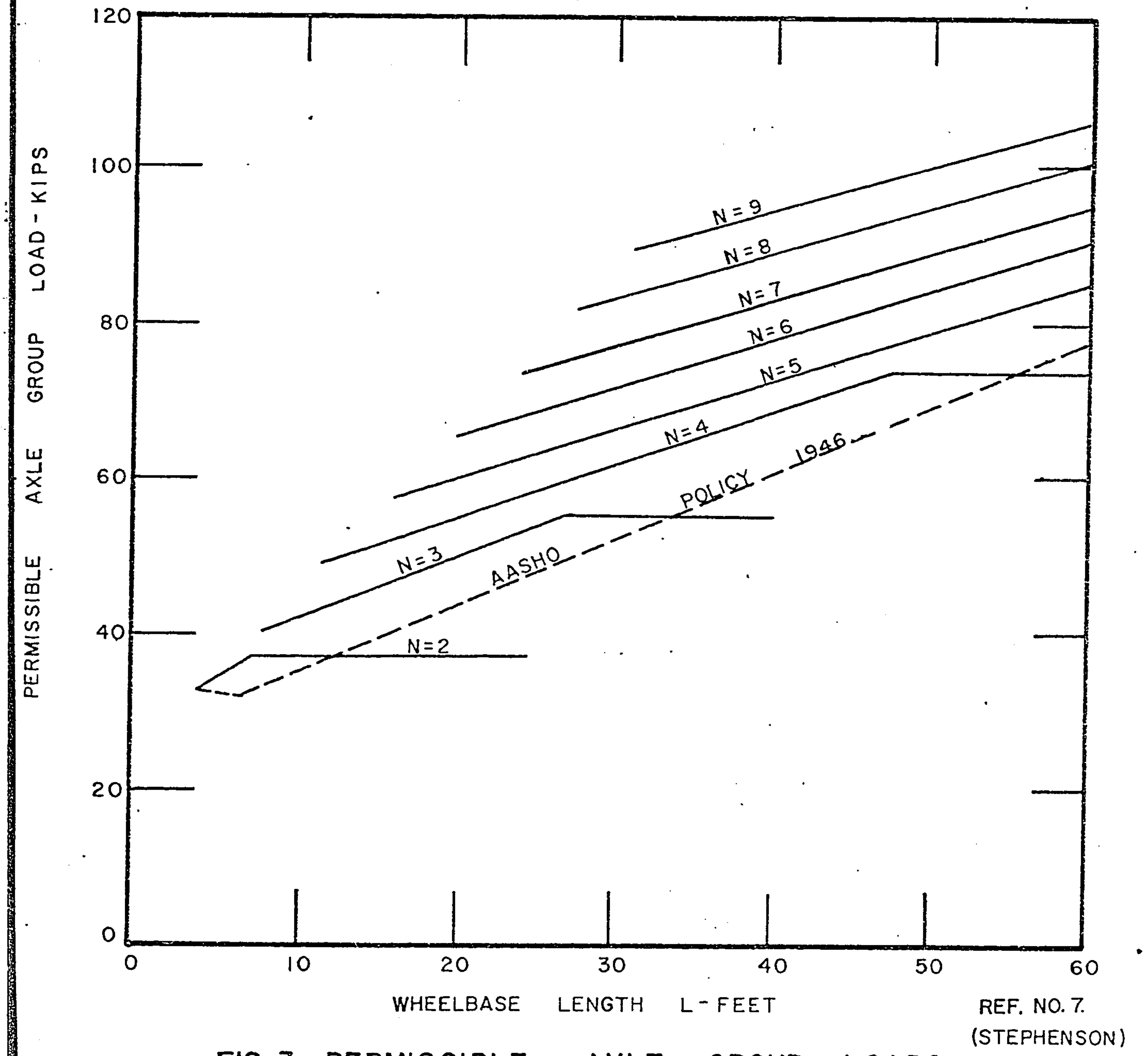
$\text{Log } N = 9.30 - 0.263\sigma_r$ (Bridges 2, 3, 5, 6 and 7)

$\text{Log } N = 9.30 - 0.162\ \sigma_r$ (Bridges 1 and 8)

Table 11 - continued

		Method 2								
		1	2	3	4	5	6	7	8	9
6	413,000	3.9	0.00 0.22	0 910	8.28	18.88	0.00 4.82	0.00 4.82	∞ 208,000	0 910
7	291,000	5.1	0.00 0.30	0 875	7.96	9.94	0.00 8.78	9.59 59.71	104,000 16,800	1,660 9,120
		4.5	0.12 1.08	350 3,145	8.12	13.12	2.66 23.96			
		3.9	0.45 1.75	1,310 5,100	8.28	18.88	6.93 26.97			
8	293,000	6.9	0.18 1.22	530 3,575	8.18	15.28	3.45 23.37	3.45 30.73	290,000 32,600	530 4,980
		6.3	0.00 0.48	0 1,405	8.28	19.10	0.00 7.36			

ILLUSTRATIONS

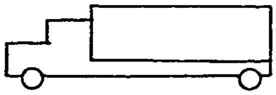


REF. NO. 7.
(STEPHENSON)

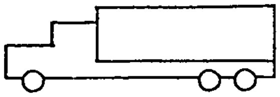
FIG. 3 PERMISSIBLE AXLE GROUP LOADS

NOTE: AXLE-GROUP LOADS FOR WHEELBASE LENGTHS LESS THAN 14', 34', AND 55' FOR NUMBER OF AXLES N=3, 4, AND 5, RESPECTIVELY ARE NOT PERMISSIBLE FOR BRIDGES OF H 15 DESIGN. ALL AXLE-GROUP LOAD SHOWN, HOWEVER, ARE PERISSIBLE FOR BRIDGES HAVING DESIGN RATINGS IN EXCESS OF H 15 DESIGN.

TRUCKS

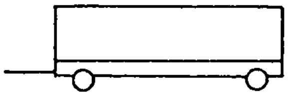


28,000 LB.

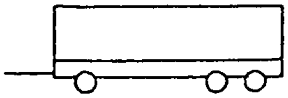


38,000 LB.

TRAILERS

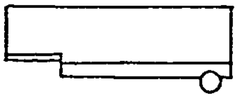


28,000 LB.

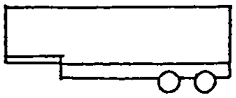


38,000 LB.

SEMI- TRAILERS



18,000 LB.



30,000 LB.

UNITS

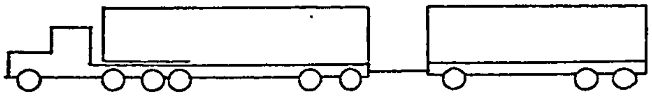
SINGLE UNITS AS SHOWN

MAXIMUM AXLE LOAD

18,000 lb/axle (SINGLE)

16,000 lb/axle (DUAL)

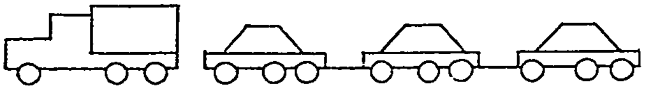
TRACTOR, SEMI- TRAILER AND TRAILER.



38,000 lb. 30,000 lb. 38,000 lb.

TOTAL 106,000 lb.

TRAIN TYPE VEHICLE PERMISSIBLE BY LAW OF ONTARIO.



38,000 lb. 38,000 lb. 38,000 lb. 38,000 lb.

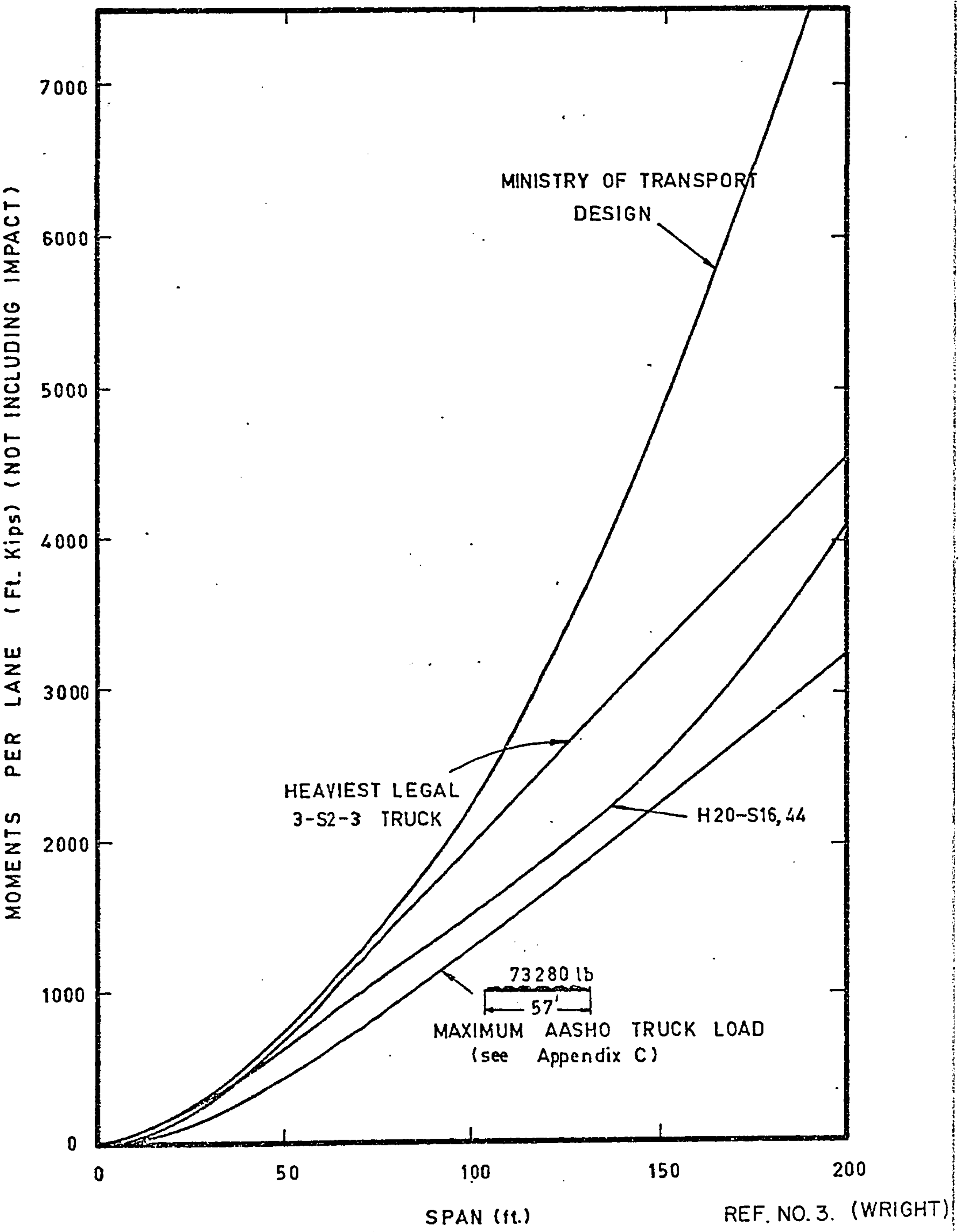
TOTAL 152,000 lb.

COMBINATIONS

COMBINATIONS PERMISSIBLE IF LESS THAN 50' LONG.

FIG. 4 LEGAL LOAD IN ONTARIO

REF. NO. 3.



REF. NO. 3. (WRIGHT)

FIG. 5 MAXIMUM BENDING MOMENTS FOR VARIOUS LOADS FOR SPANS UP TO 200 Ft.

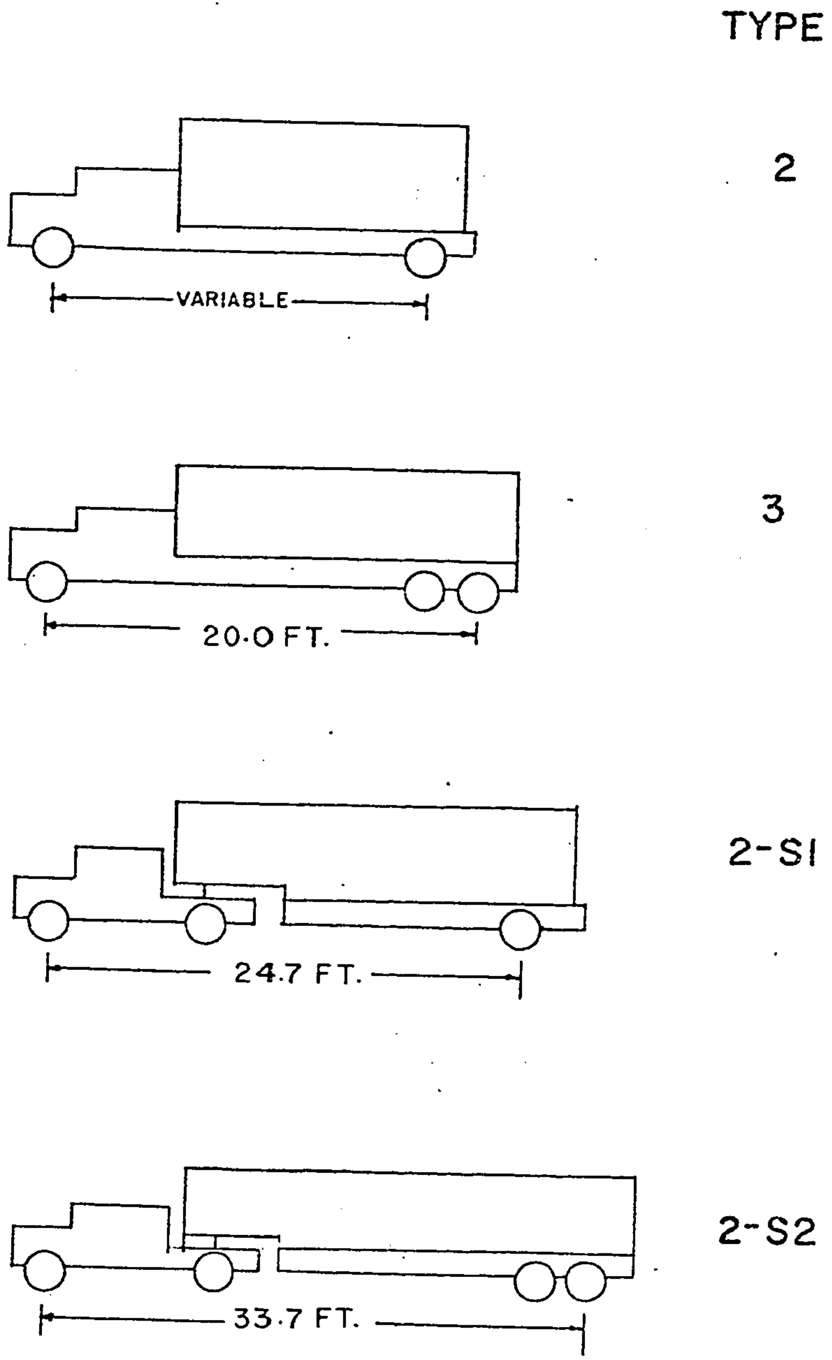


FIG. 6 IDENTIFICATION OF TRUCK TYPES

CALIFORNIA REF. NO. 6.

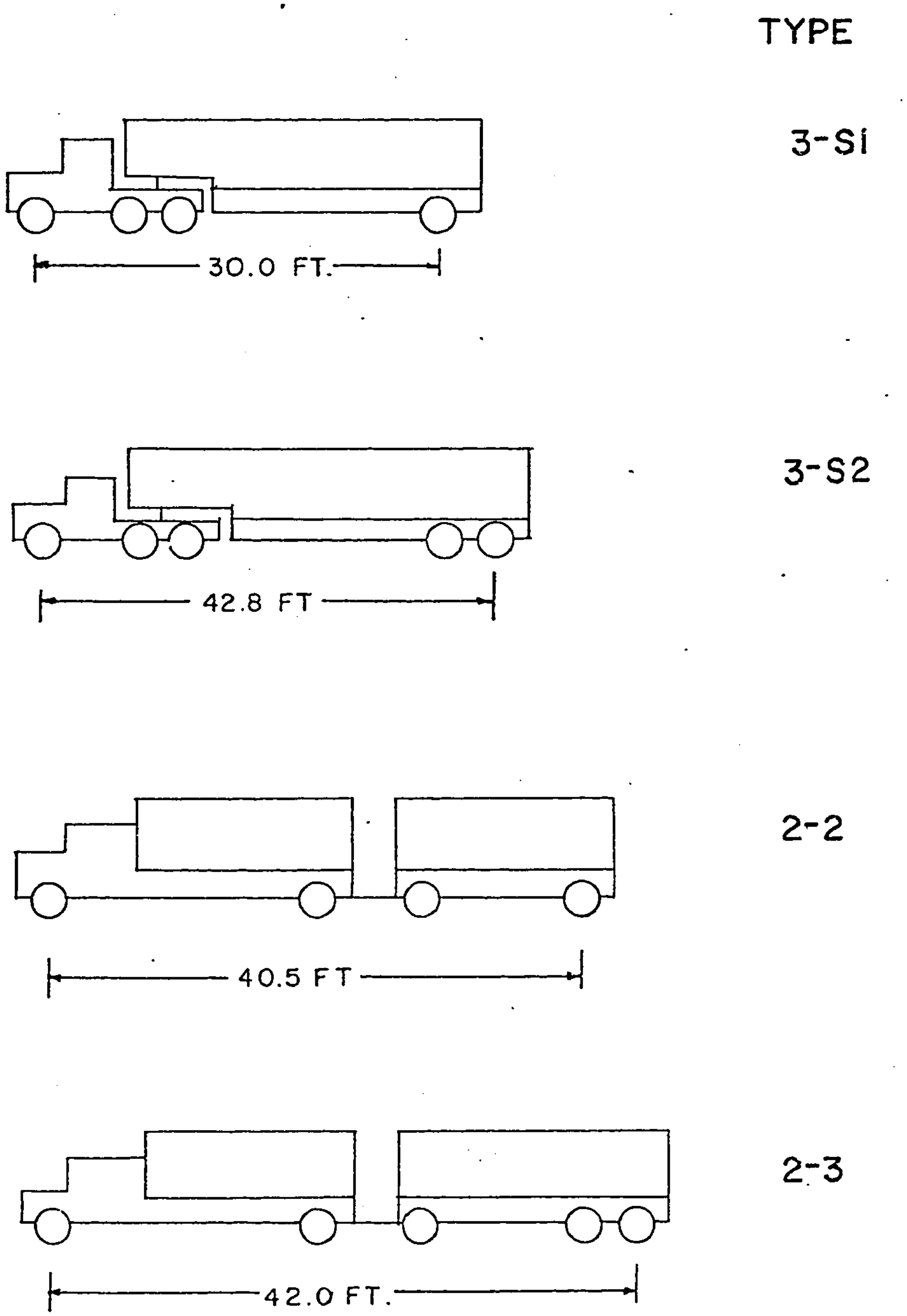


FIG. 6 IDENTIFICATION OF TRUCK TYPES

(CONT.)

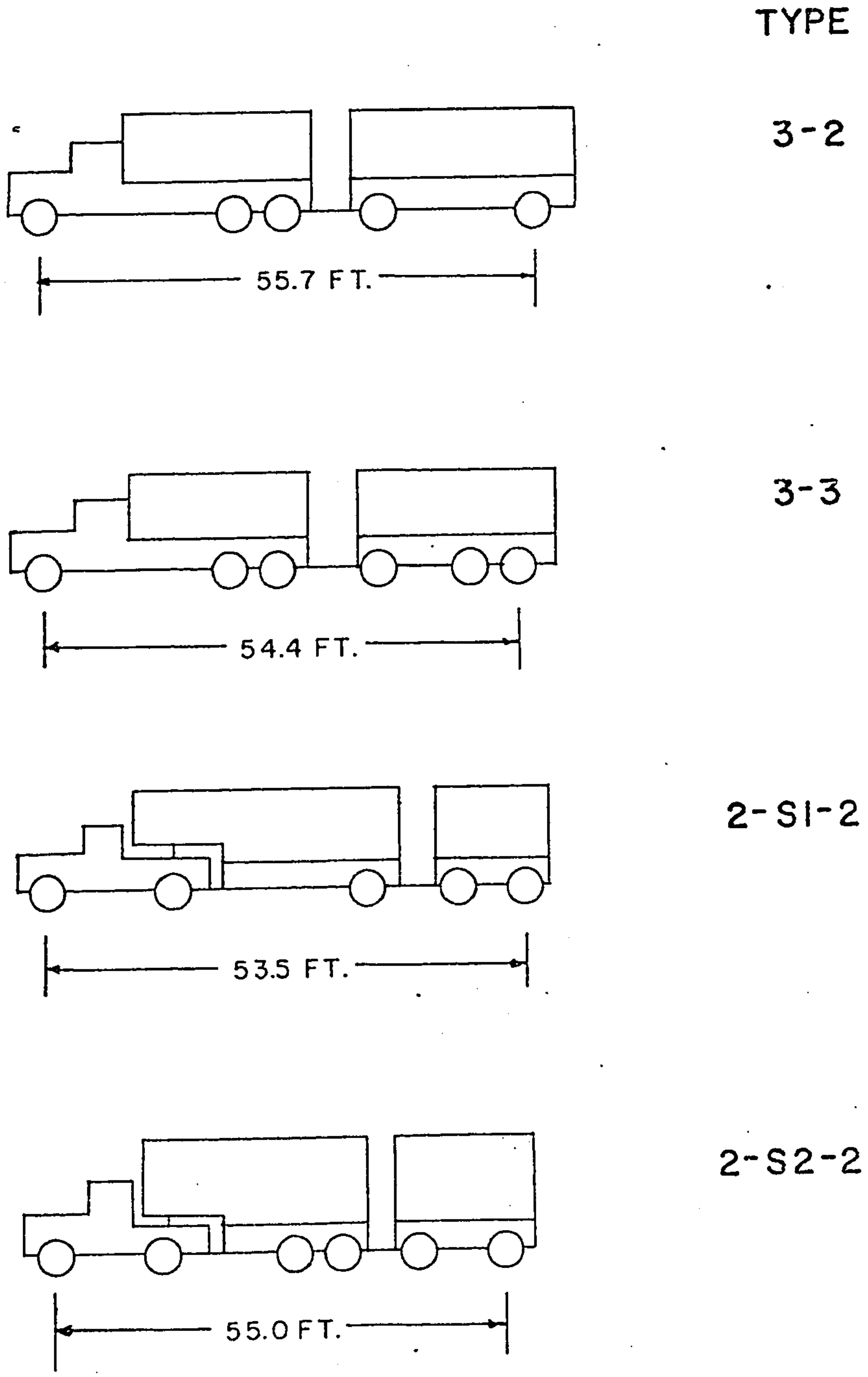


FIG. 6 IDENTIFICATION OF TRUCK TYPES
(CONT.)

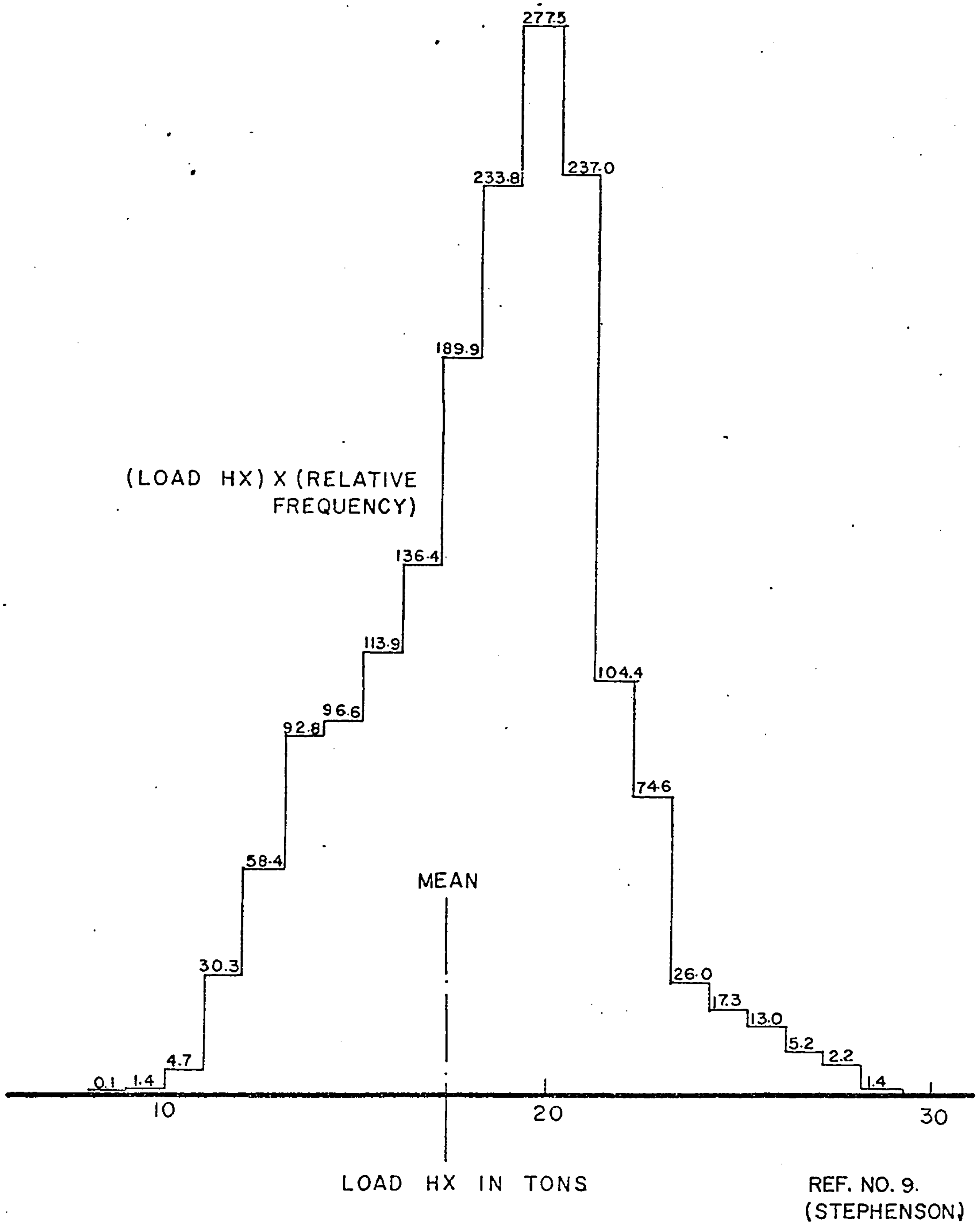
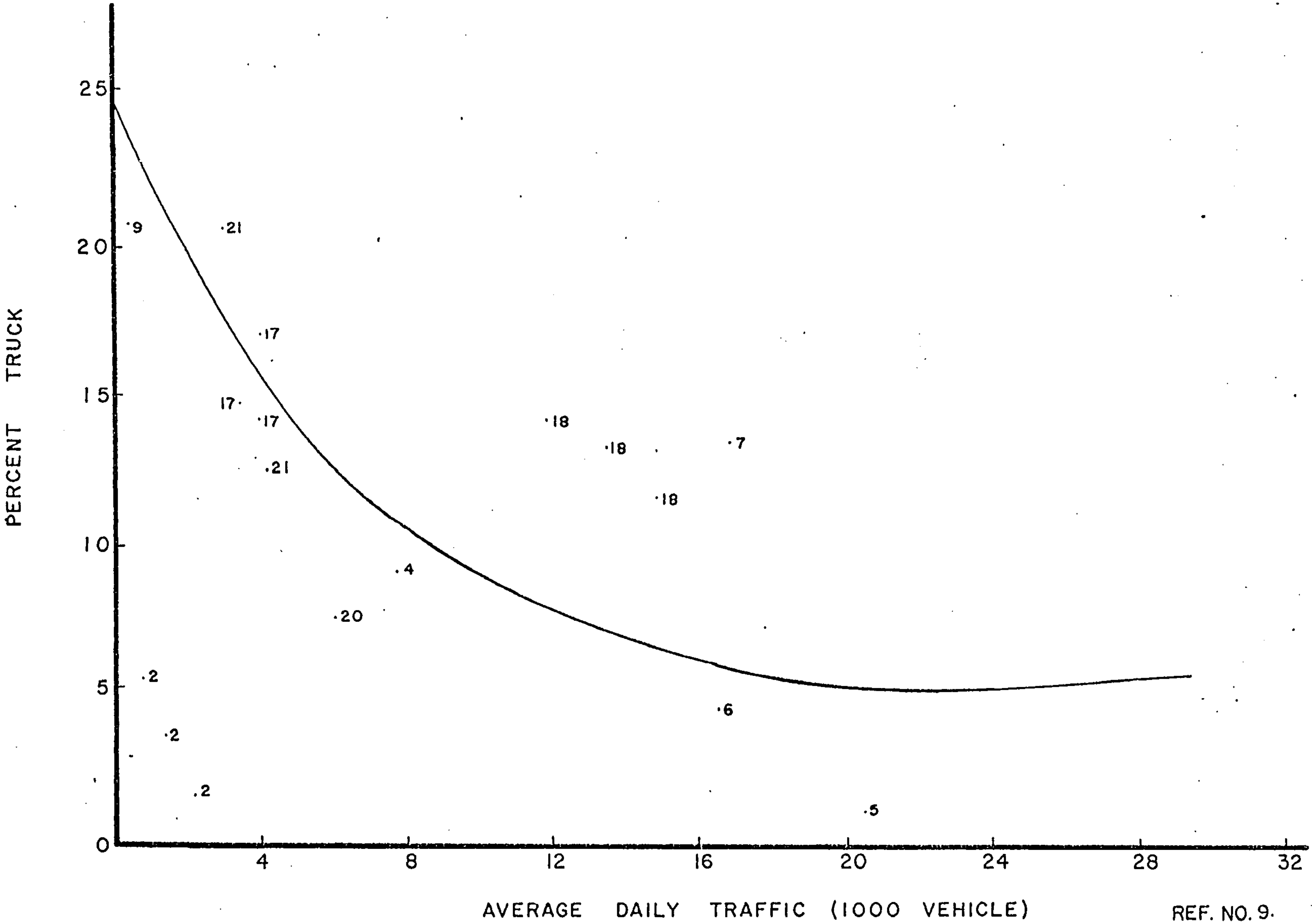


FIG. 7 LOAD FREQUENCY HISTOGRAM



REF. NO. 9.
(LYNCH)

FIG. 8 LOAD FREQUENCY OF 21 BRIDGES OVER THE OHIO RIVER

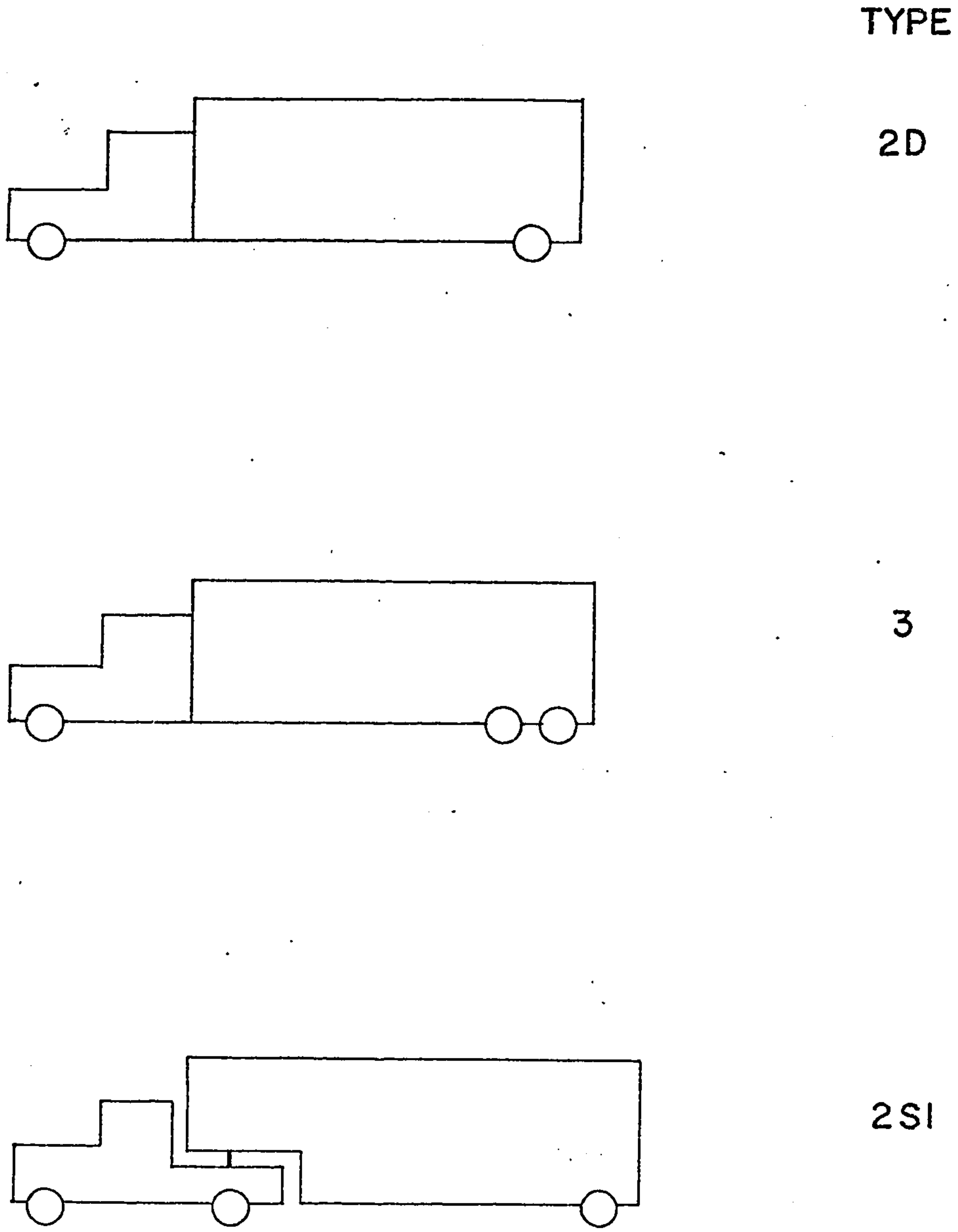


FIG. 9 IDENTIFICATION OF TRUCK TYPES

MICHIGAN REF. NO. 10.

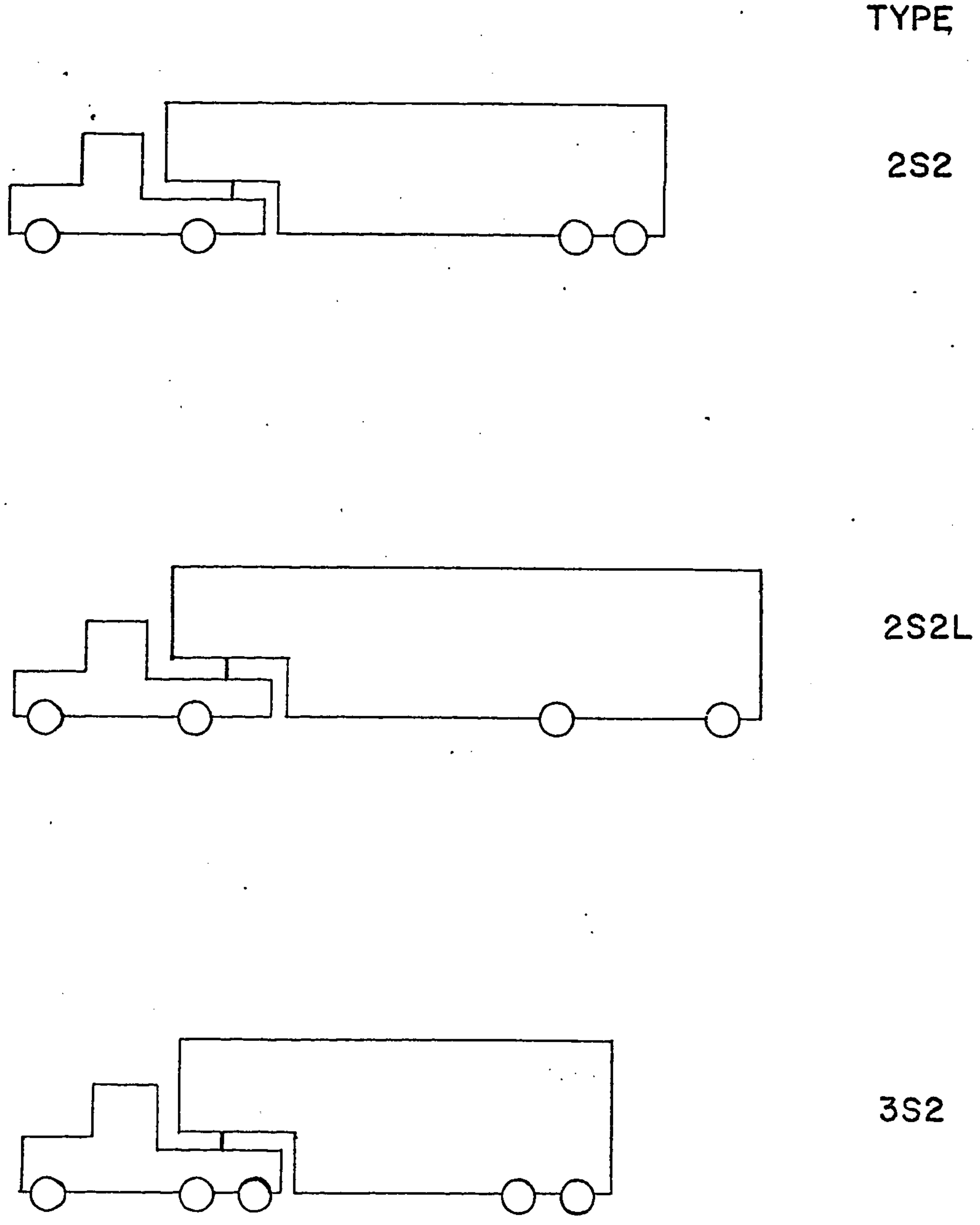
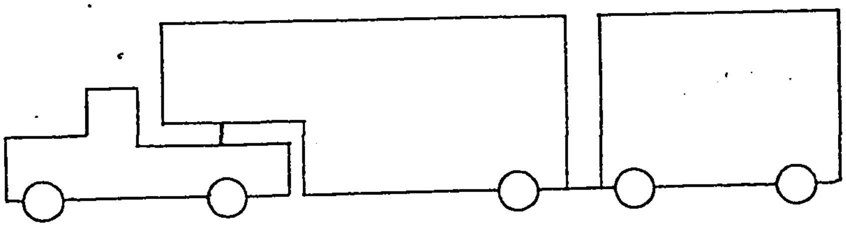
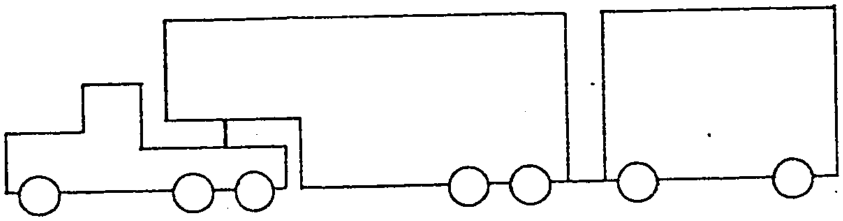


FIG. 9 IDENTIFICATION OF TRUCK TYPES
(CONT.)

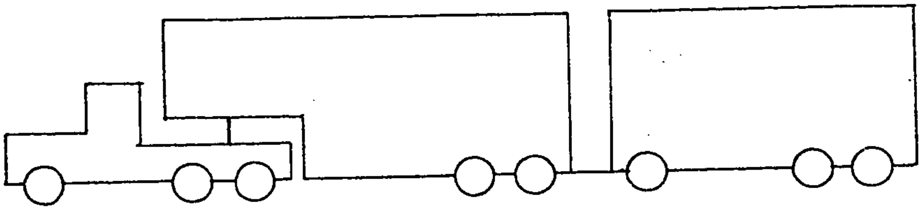
TYPE



2S1-2



3S2-2



3S2-3

FIG. 9 IDENTIFICATION OF TRUCK TYPES

(CONT.)

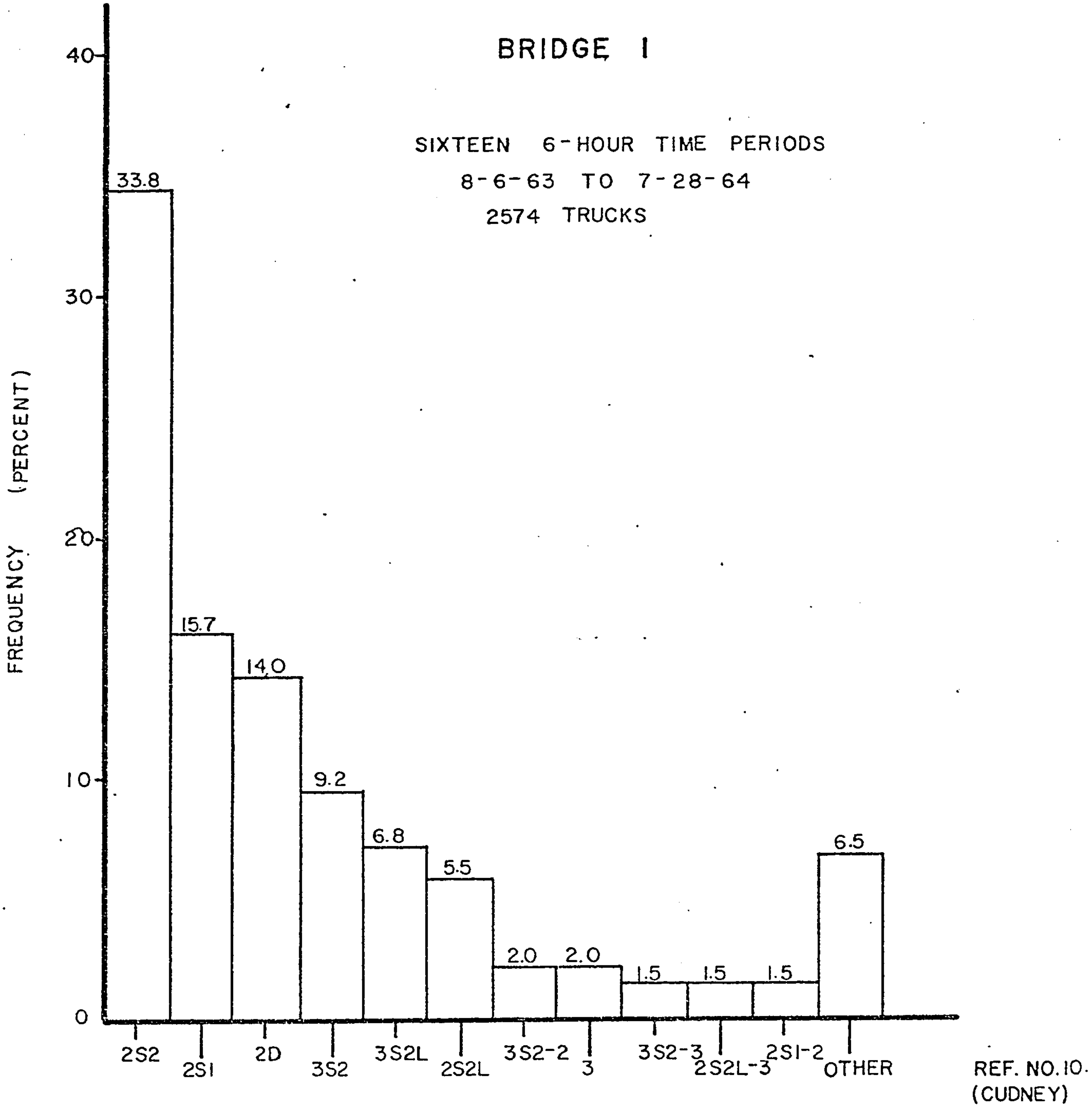


FIG. 10 FREQUENCY DISTRIBUTION OF THE ELEVEN TRUCK TYPES

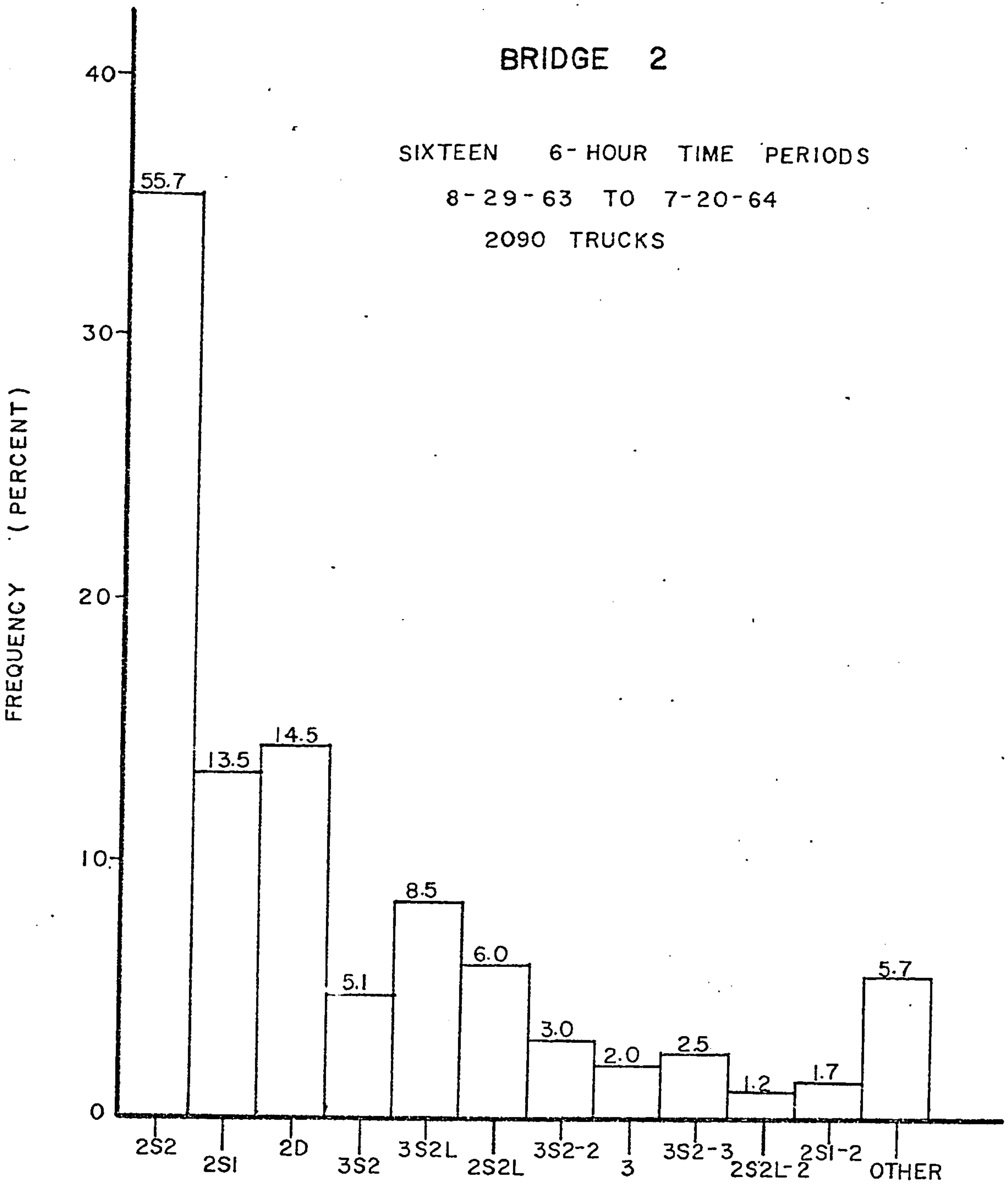


FIG.10 FREQUENCY DISTRIBUTION OF THE
(CONT.) ELEVEN TRUCK TYPES

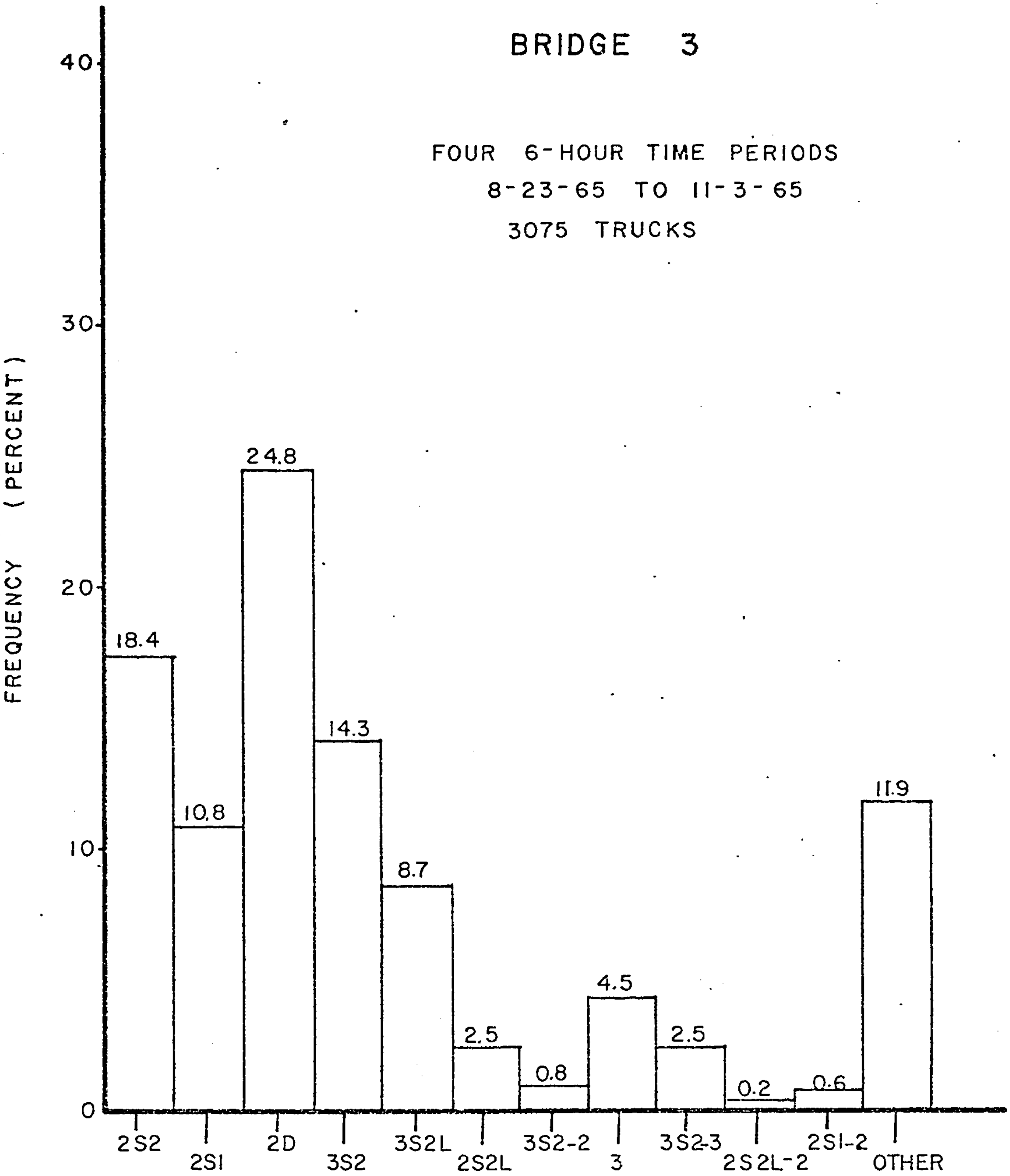


FIG. 10 FREQUENCY DISTRIBUTION OF THE ELEVEN TRUCK TYPES
(CONT.)

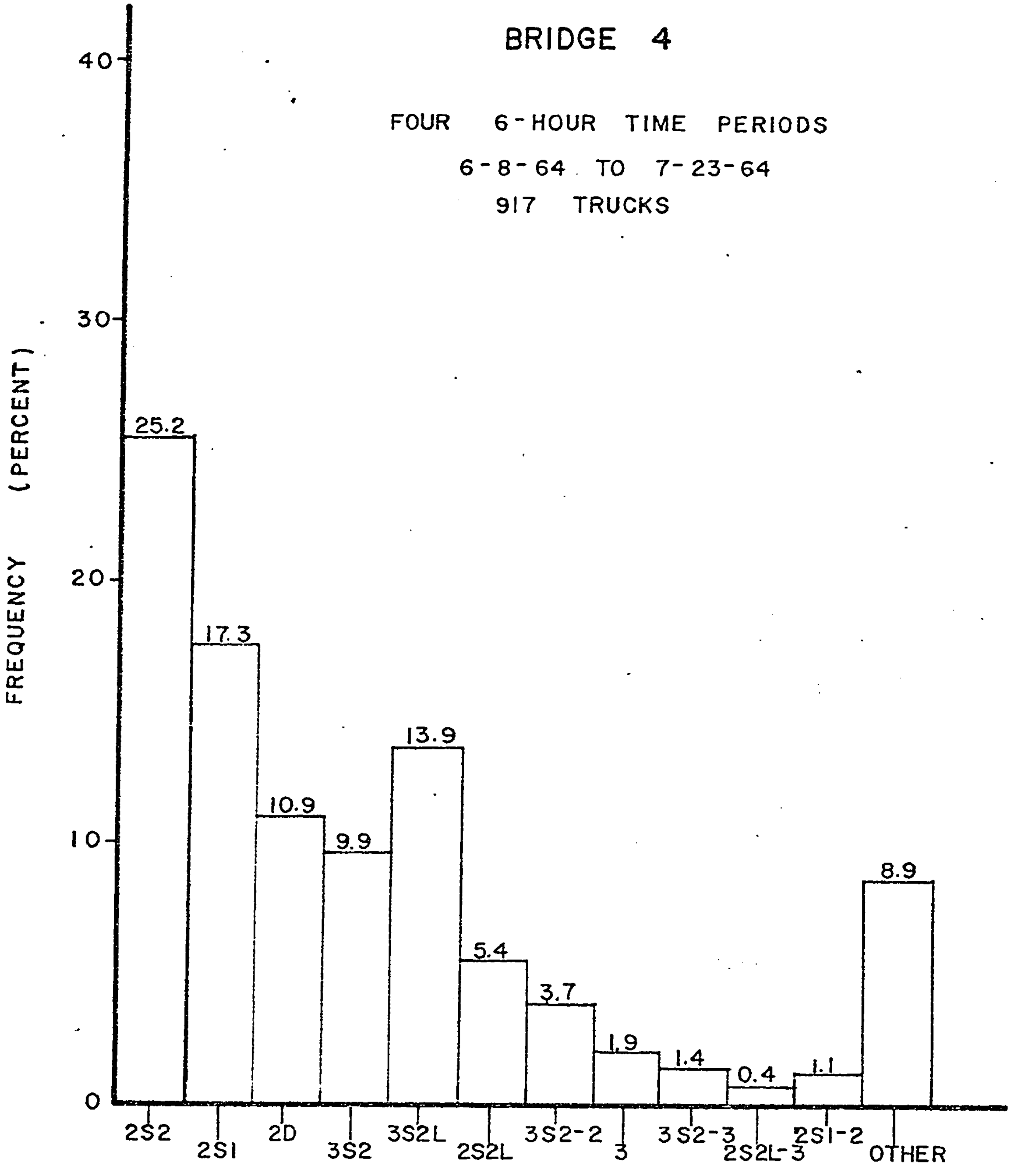


FIG.10 FREQUENCY DISTRIBUTION OF THE ELEVEN TRUCK TYPES
(CONT.)

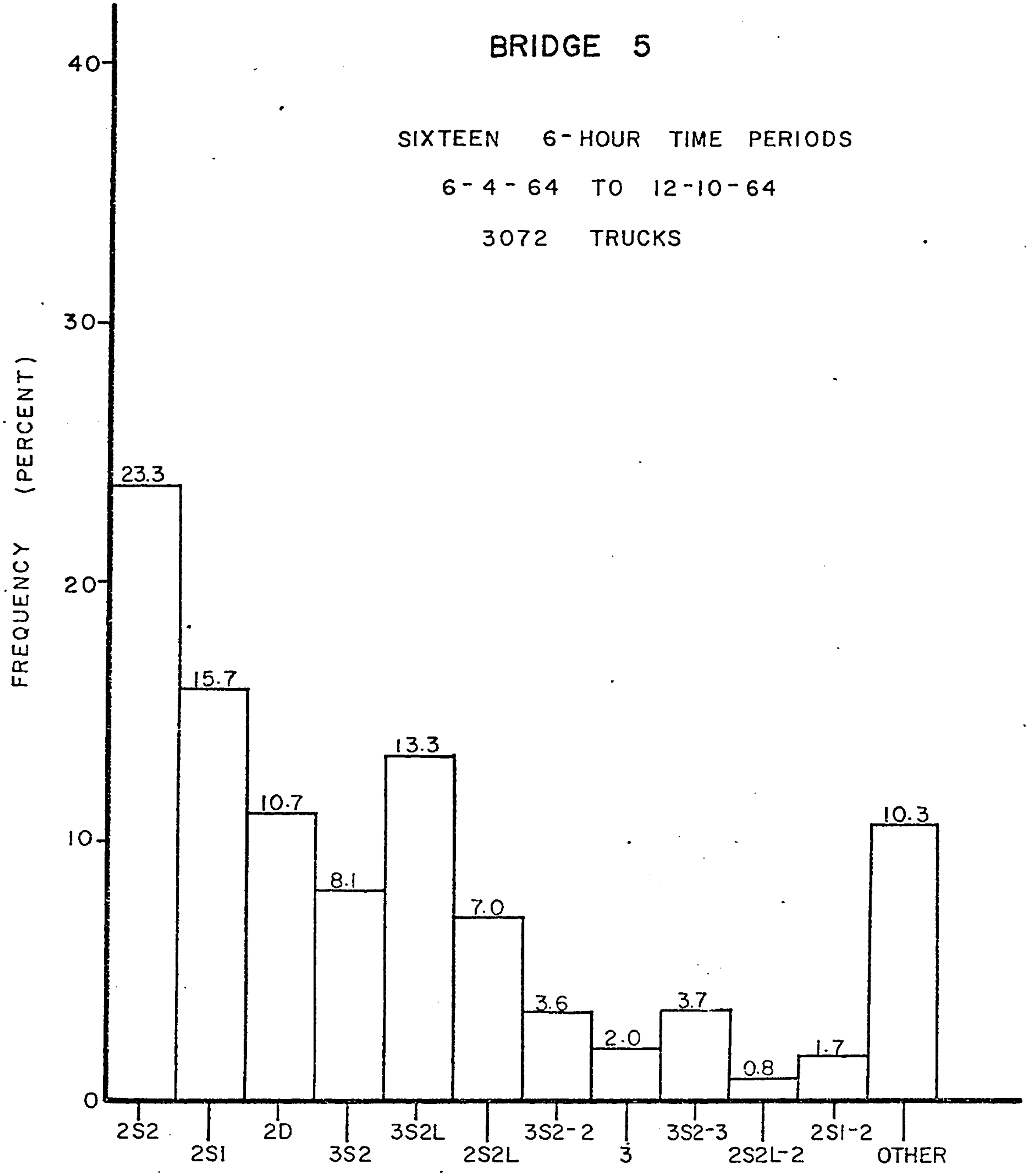


FIG.10 FREQUENCY DISTRIBUTION OF THE
(CONT.) ELEVEN TRUCK TYPES

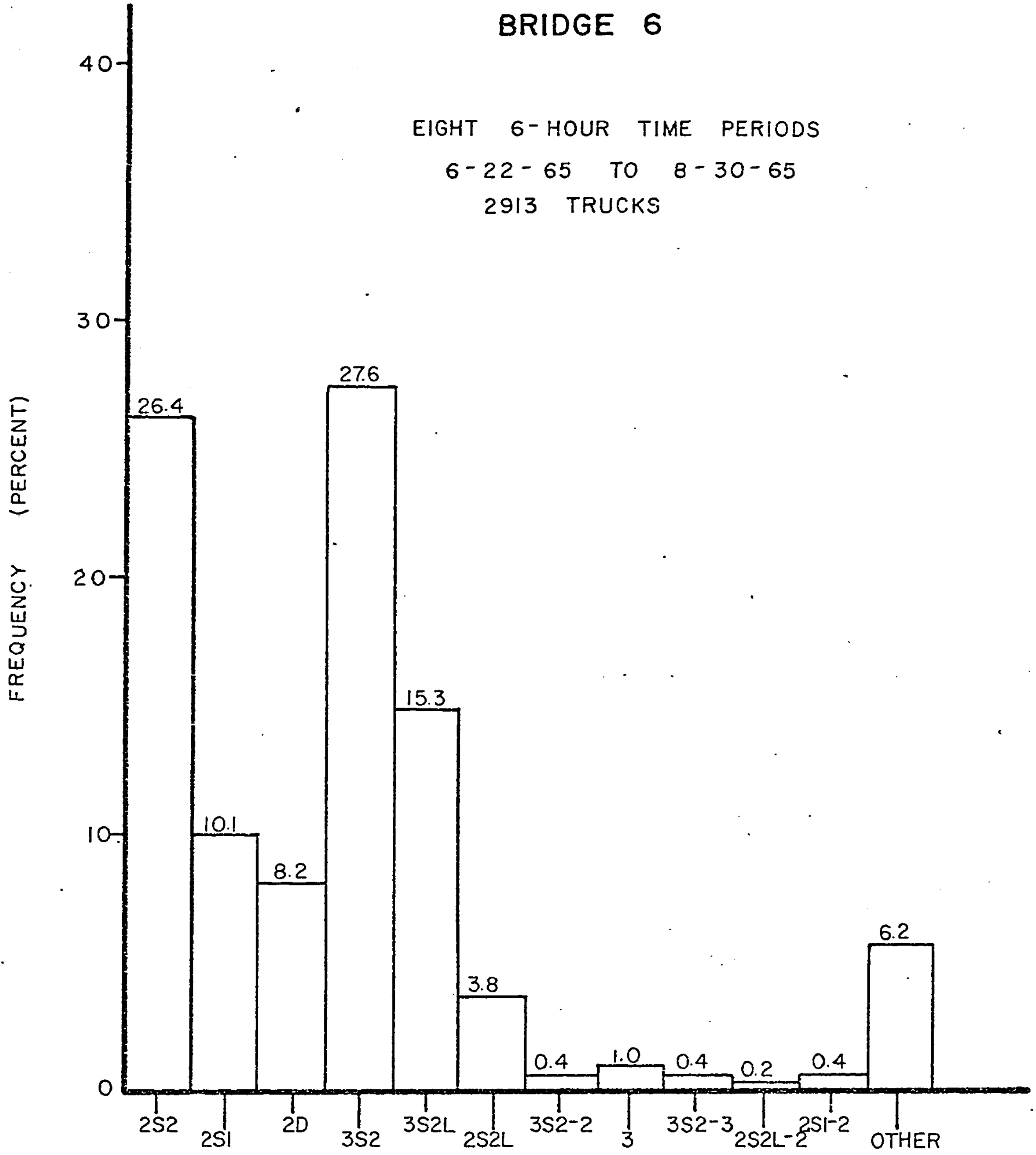


FIG. 10 FREQUENCY DISTRIBUTION OF THE
(CONT.) ELEVEN TRUCK TYPES

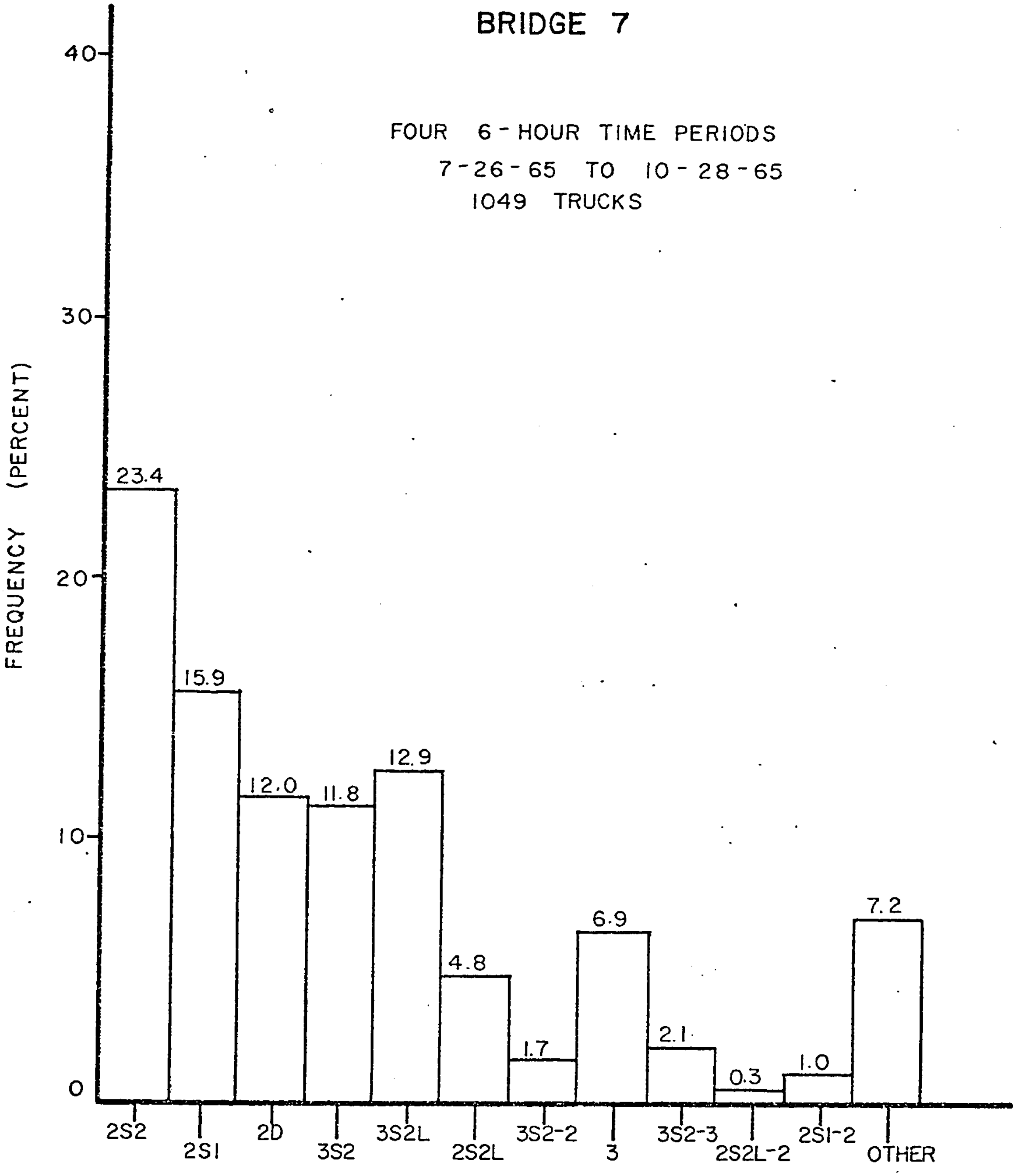


FIG. 10 FREQUENCY DISTRIBUTION OF THE ELEVEN TRUCK TYPES
(CONT.)

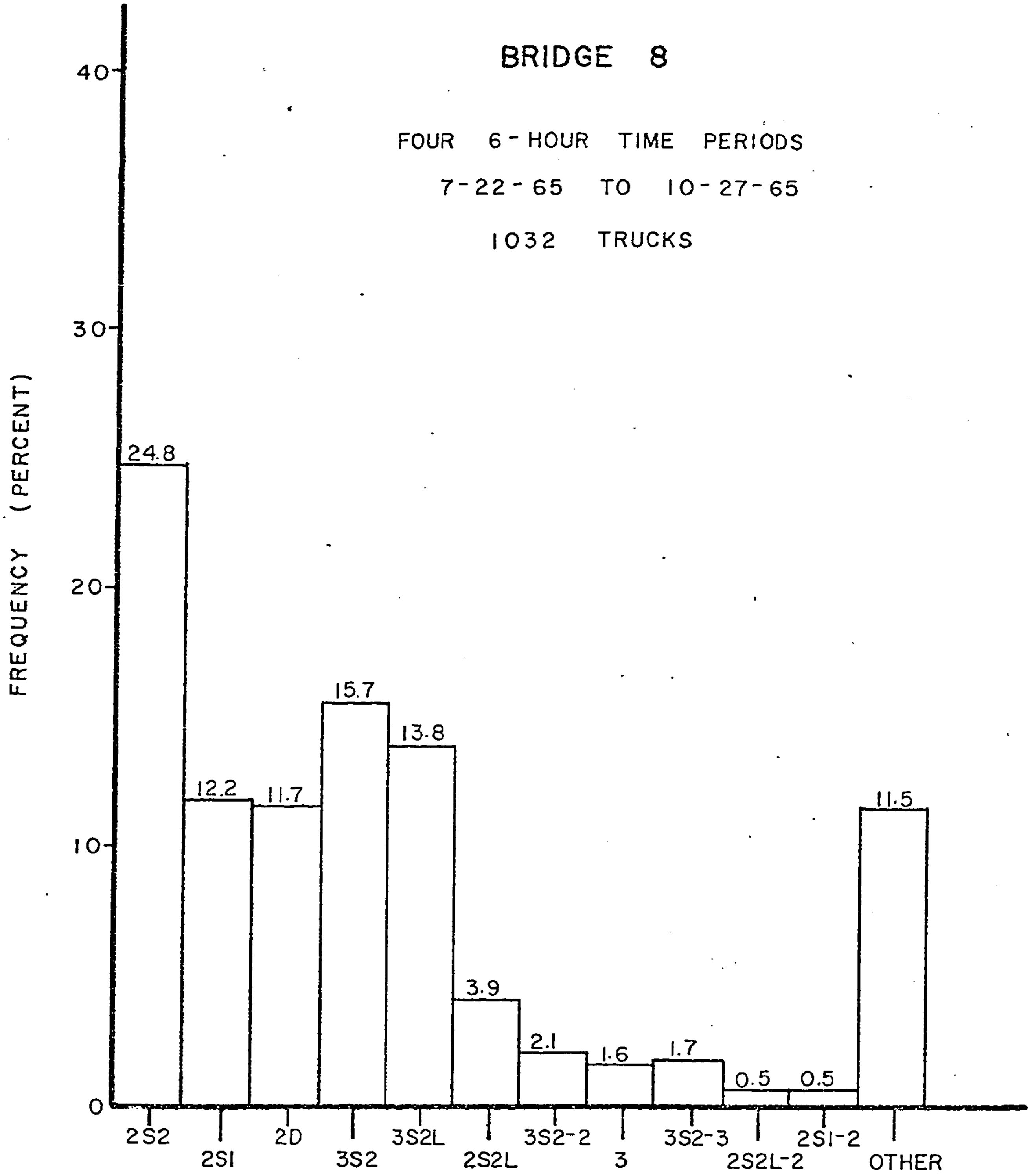


FIG.IO FREQUENCY DISTRIBUTION OF THE
(CONT.) ELEVEN TRUCK TYPES

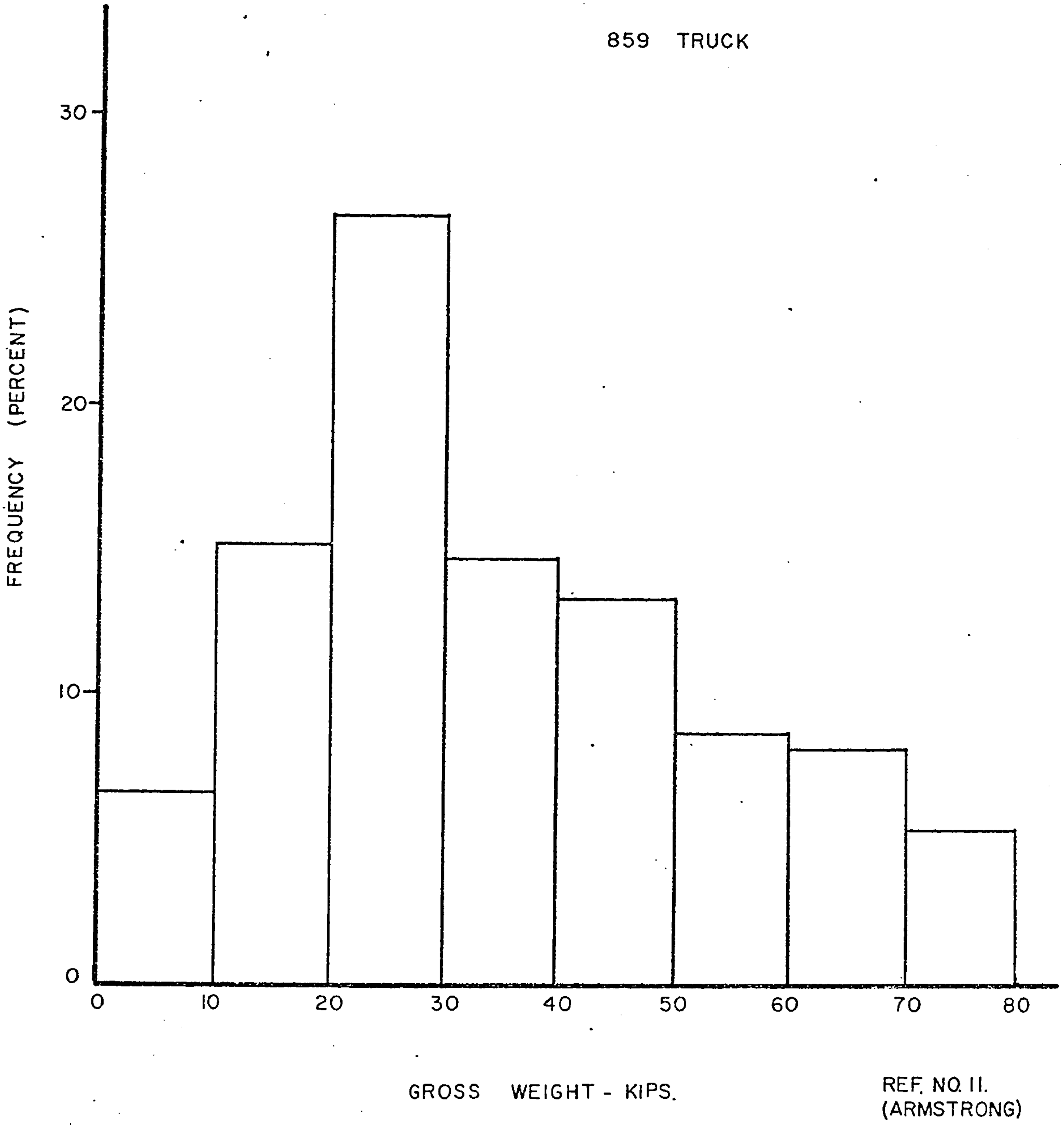


FIG. 11 FREQUENCY DISTRIBUTION OF TRUCK GROSS WEIGHTS

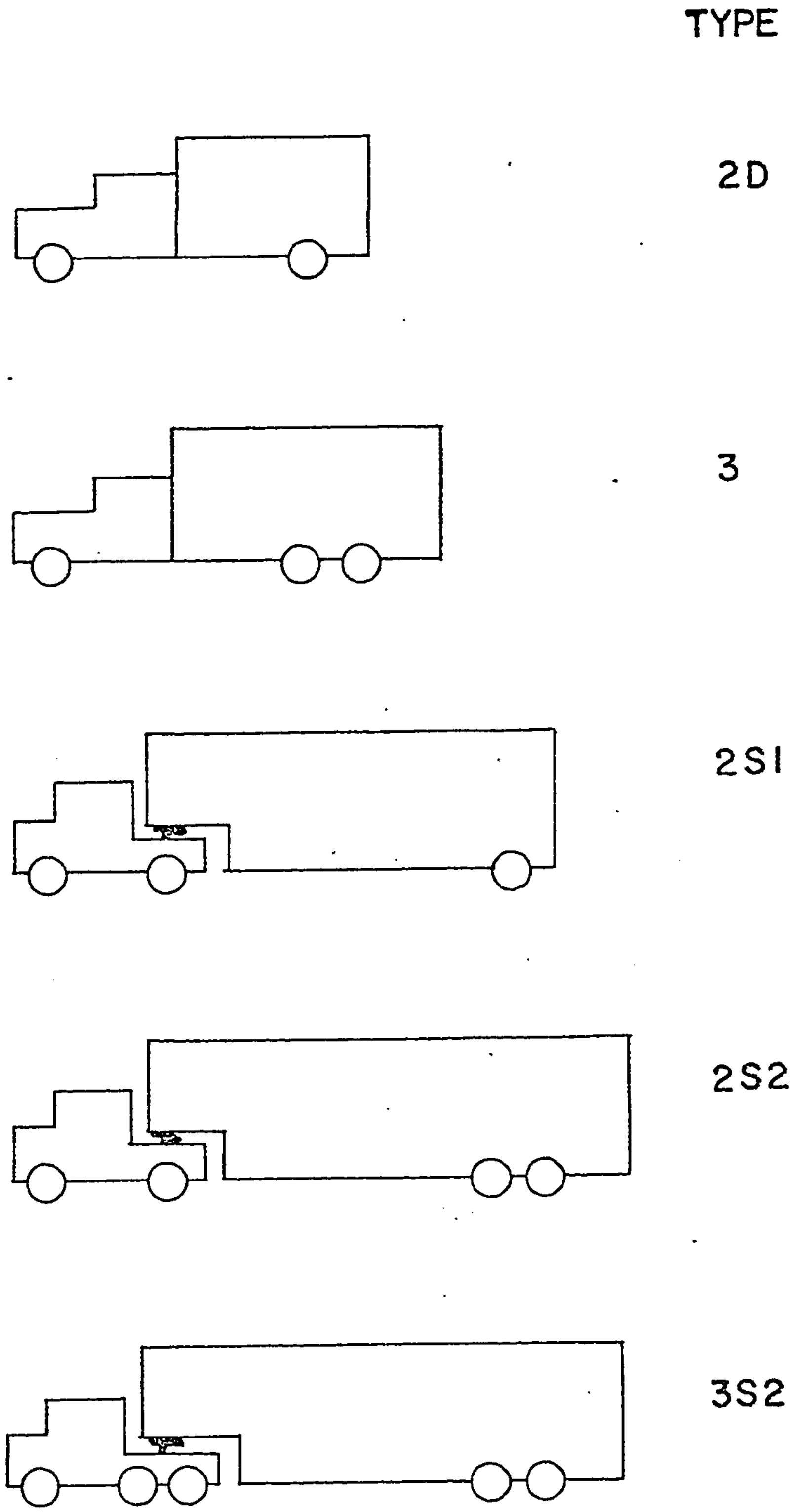


FIG.12 IDENTIFICATION OF TRUCK TYPES

MARYLAND REF. NO. 12.

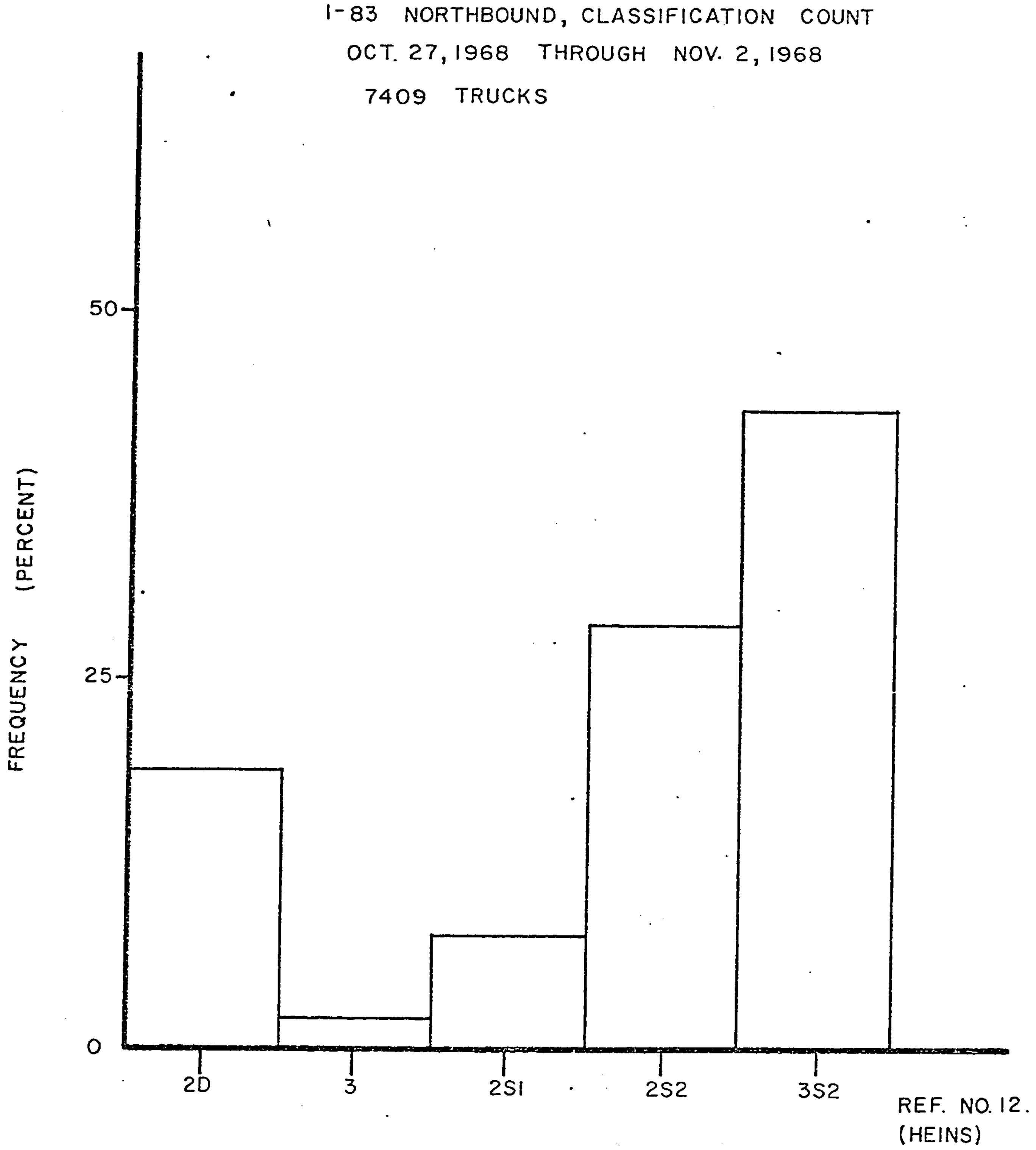
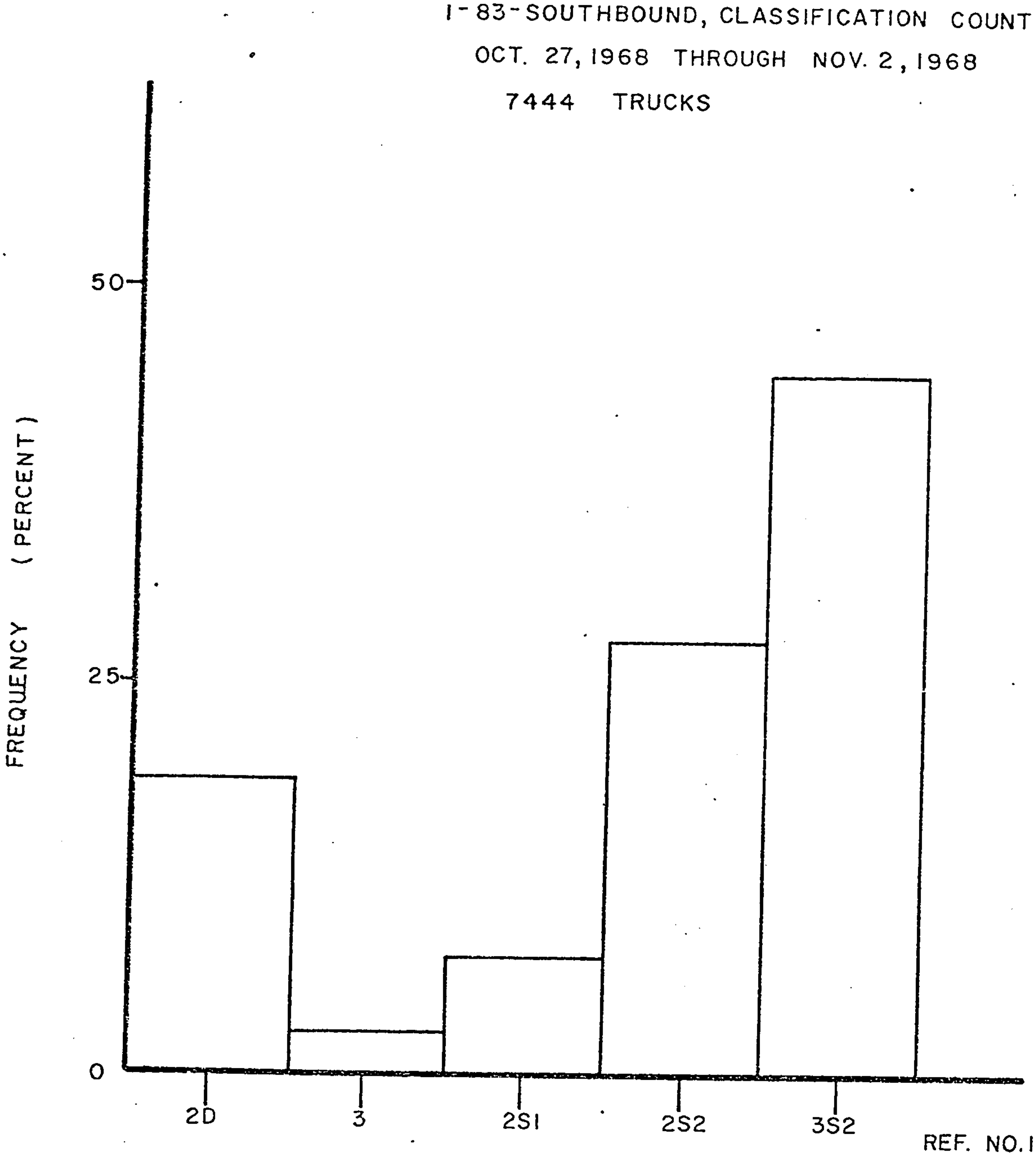
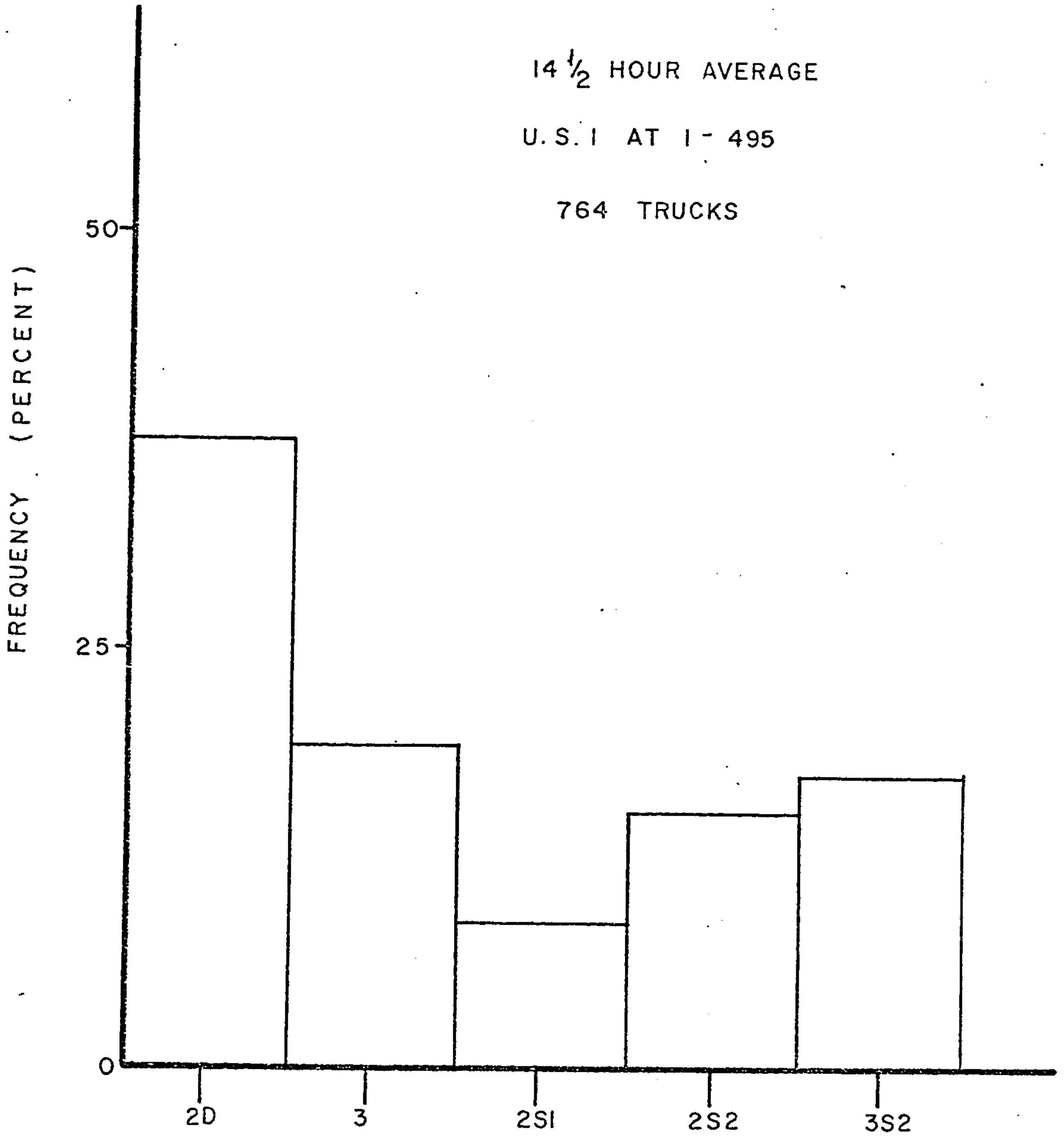


FIG.13 DISTRIBUTION OF TRUCK TYPES


I-83-SOUTHBOUND, CLASSIFICATION COUNT
OCT. 27, 1968 THROUGH NOV. 2, 1968
7444 TRUCKS

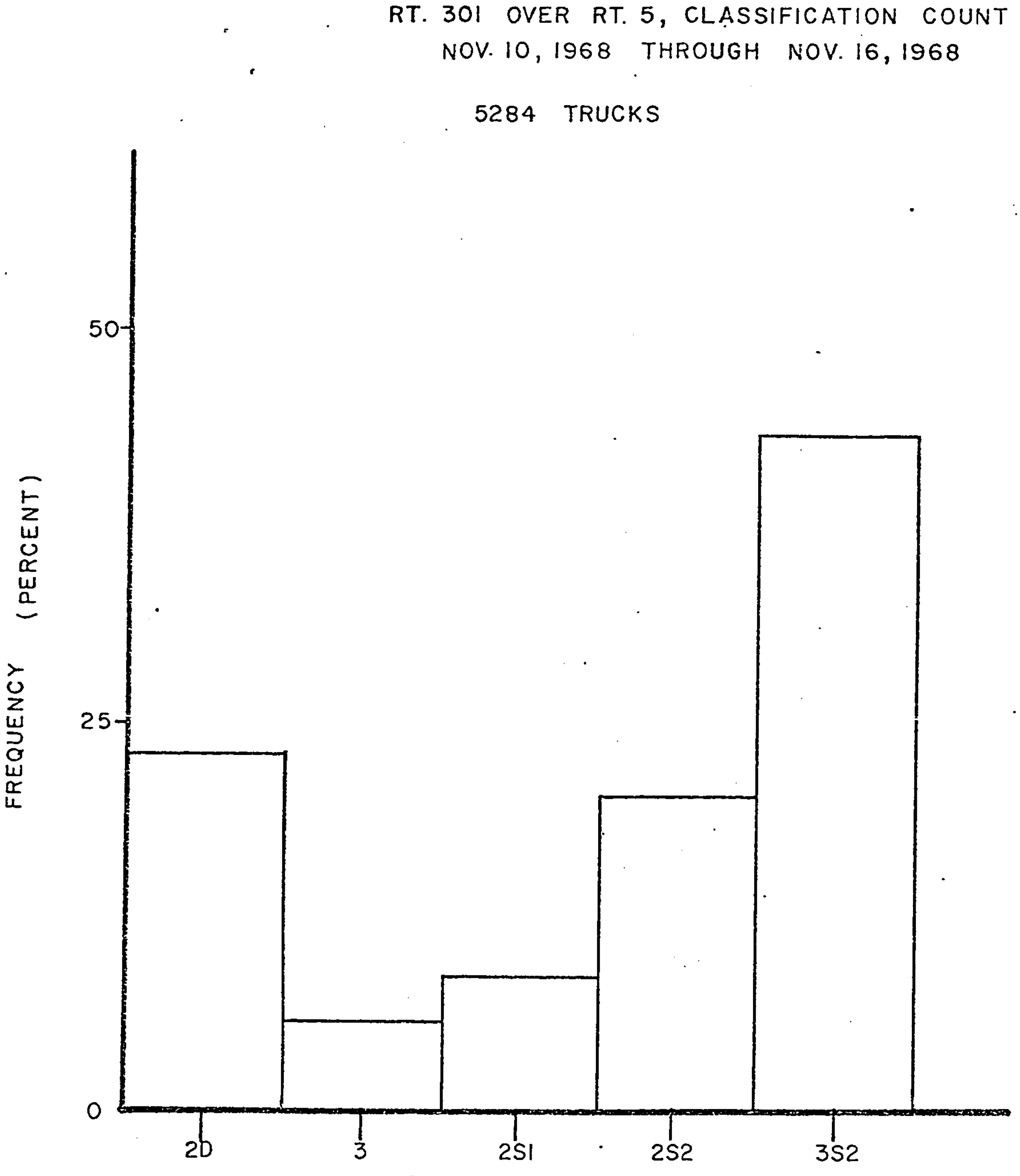
REF. NO. 12.
(HEINS)

FIG. 14 DISTRIBUTION OF TRUCK TYPES



REF. NO.12.
(HEINS)

FIG.15 DISTRUTION OF TRUCK TYPES

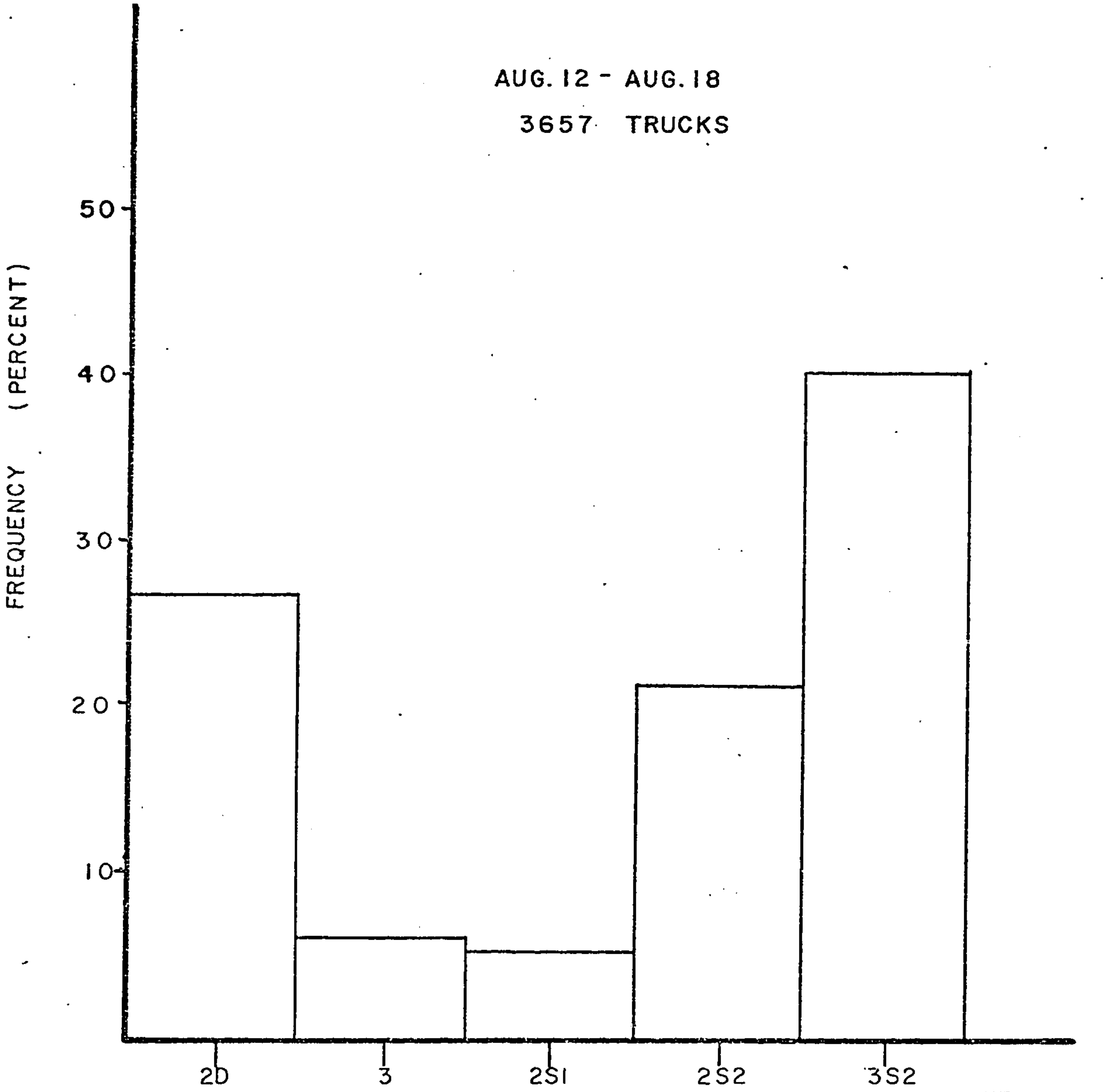


RT. 301 OVER RT. 5, CLASSIFICATION COUNT
NOV. 10, 1968 THROUGH NOV. 16, 1968

5284 TRUCKS

REF. NO. 12.
(HEINS)

FIG.16 DISTRIBUTION OF TRUCK TYPES



REF. NO. 13.
(HEINS)

FIG. 17 DISTRIBUTION OF TRUCK TYPES

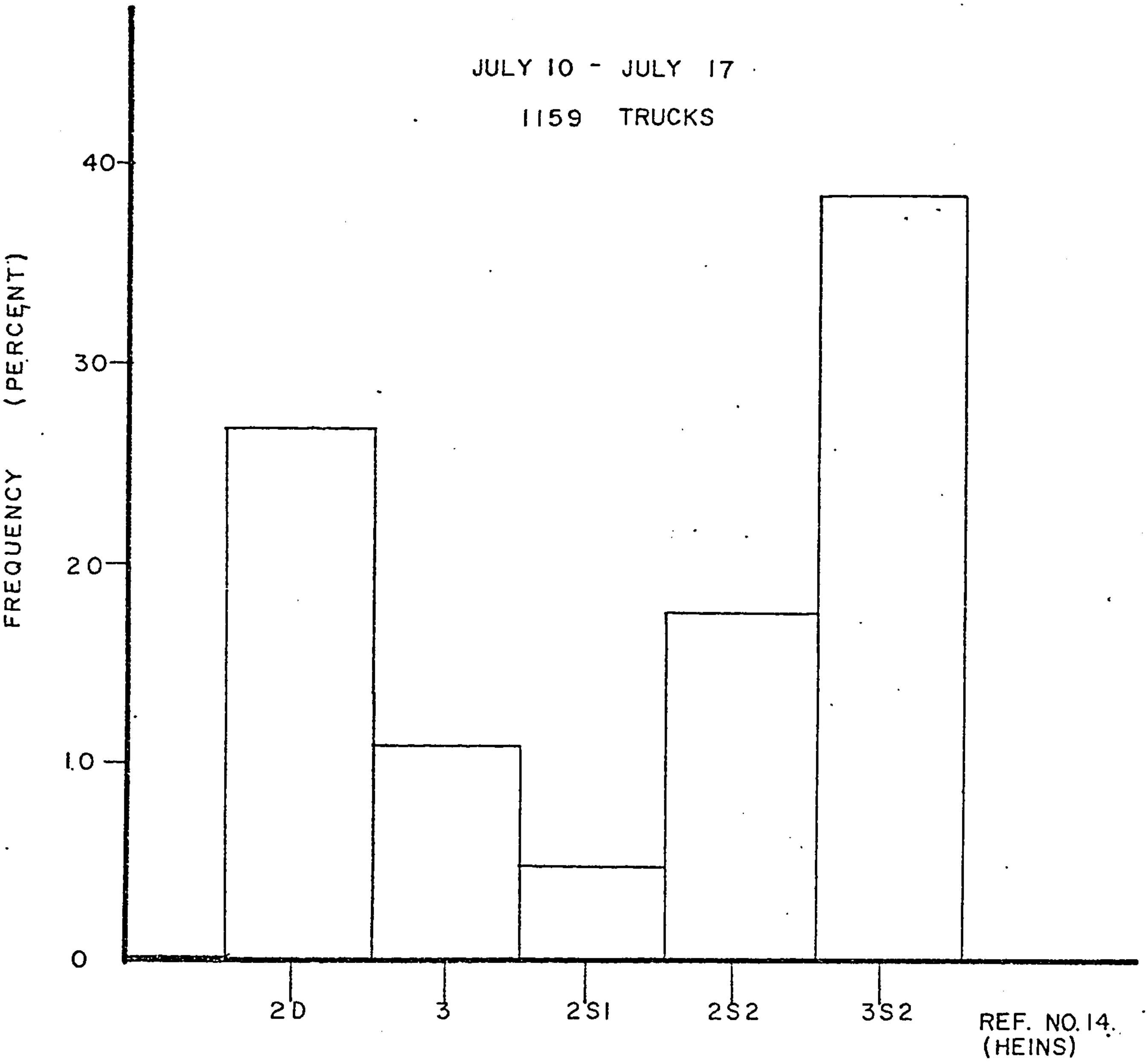
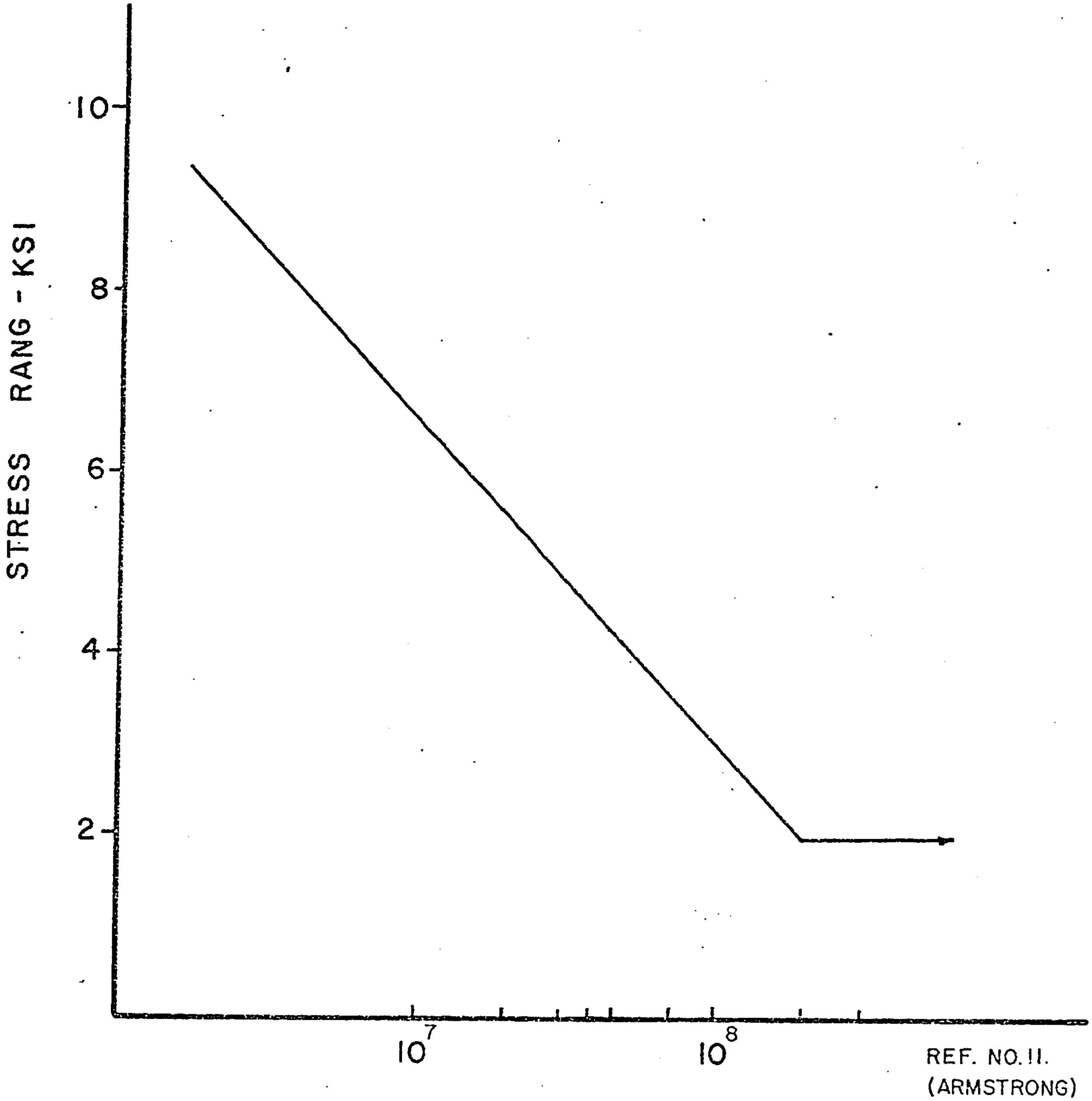
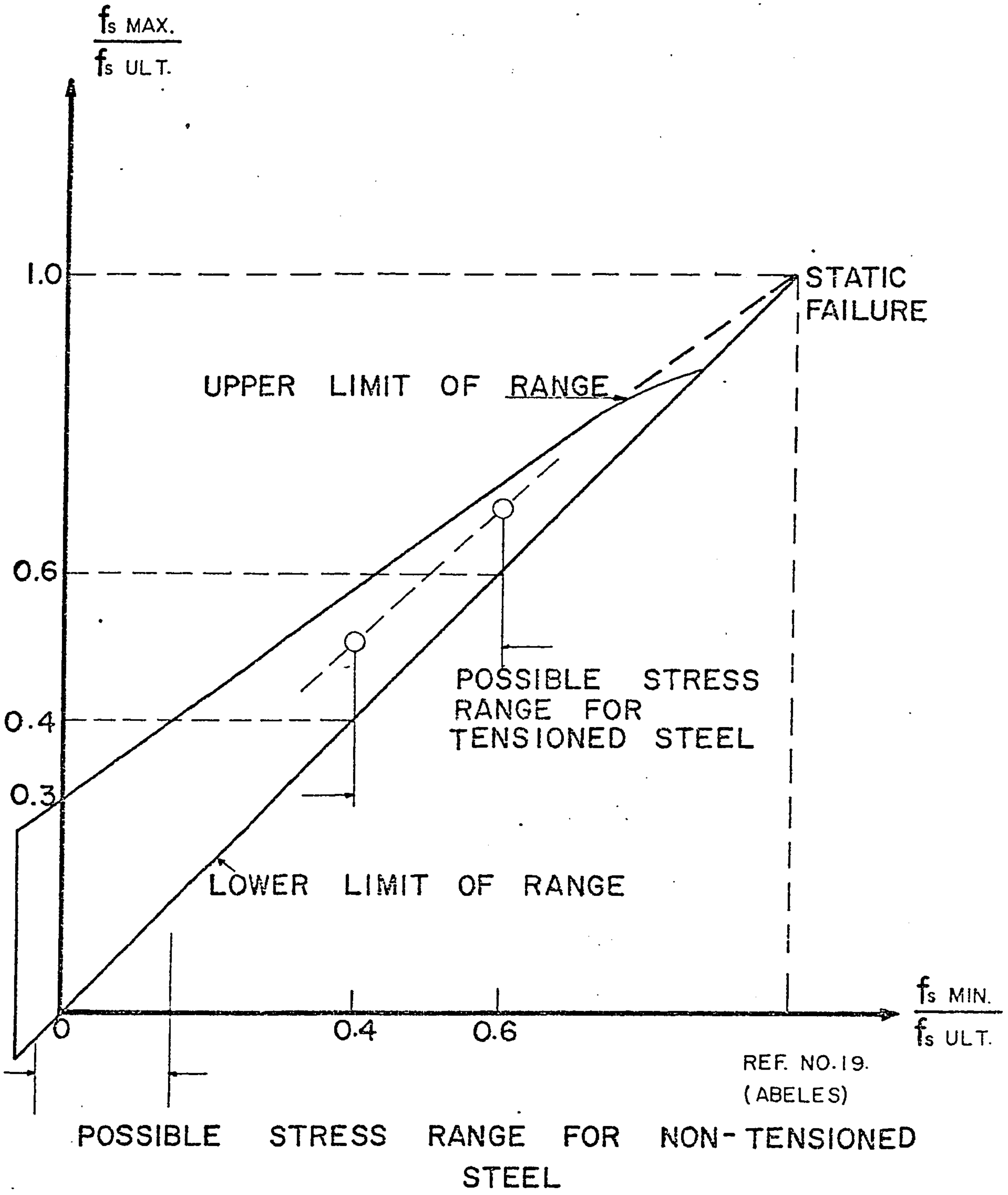


FIG.18 DISTRIBUTION OF TRUCK TYPES



REF. NO. 11.
(ARMSTRONG)

FIG. 19 ASSUMED S-N CURVE FOR BEAM WITH WELDED, PARTIAL-LENGTH COVER PLATES



REF. NO. 19. (ABELES)

FIG. 20 GOODMAN DIAGRAM

APPENDICES A and B

APPENDIX A

"Policy Concerning Maximum Dimensions, Weights and Speeds of Motor Vehicles to be Operated Over the Highways of the United States" of the AASHO (April 1, 1946).

A. Width

No vehicle unladen or with load, shall have a total outside width in excess of 96 inches. (Note: It is recognized that certain conditions inherent in the design of vehicles suggest the desirability of 102 inches as a standard of maximum width. The existence of numerous bridges and a large mileage of highways too narrow for the safe accommodation of vehicles of such width precludes the present adoption of the higher standard of width.)

B. Height

No vehicles, unladen or with load, shall exceed a height of 12 feet, 6 inches.

C. Length

(i) No single truck, unladen or with load, shall have an over-all length, inclusive of front and rear bumpers, in excess of 35 feet.

(ii) No single bus, unlade or with load, shall have an over-all length, inclusive of front and rear bumpers, in excess of 40 feet, provided that a bus in excess of 35 feet in over-all length shall not have less than 3 axles.

(iii) No combination of truck-tractor and semi-trailer, unladen or with load, shall have an over-all length, inclusive of front and rear bumpers, in excess of 50 feet.

(iv) No other combination of vehicles shall consist of more than two units, and no such combination of vehicles, unlade or with load, shall have an over-all length, inclusive of front and rear bumpers, in excess of 60 feet.

D. <u>Speed</u>

(i) Minimum speed. No motor vehicle shall be unnecessarily driven at such slow speed as to impede or block the normal and reasonable movement of traffic. Exception to this requirement shall be recognized when reduced speed is necessary for safe operation or when a vehicle or combination of vehicles is necessarily or in compliance with law or police direction proceeding at reduced speed.

(ii) Maximum speed. No truck shall be operated at a speed greater than 45 miles per hour. Passenger vehicles may be operated at such speeds as shall be consistent at all times with safety and the proper use of the roads.

(iii) Vehicles equipped with solid rubber or cushion tires shall be operated at a speed not in excess of 10 miles per hour.

E. <u>Permissible Loads</u>

(i) No axle shall carry a load in excess of 18,000 pounds. (Note: An axle load shall be defined as the total load transmitted to the road by all wheels whose centres may be included between two parallel transverse vertical planes 40 inches apart, extending across the full width of the vehicle.

(ii) No group of axles shall carry a load in pounds in excess of the value given in the following table corresponding to the distance in feet between the extreme axles of the group, measured longitudinally to the nearest foot.
(The table quoted expresses the following relations:

for $4 < x < 7$, $w = 32{,}000$ lb.

for $8 < x < 57$, $w = 24{,}600 + 1{,}025\,x - 3x^2$

where x is the distance in feet between the extremes of any group of axles. The maximum load permitted for $x = 57$ feet is 73,280 lbs.).

(iii) The maximum axle and axle-group loads recommended in paragraphs (i) and (ii) above are subject to reasonable reduction in the discretion of the appropriate highway authorities during the periods when road subgrades have been weakened by water saturation or other causes.

(iv) The operation of vehicles or combinations of vehicles having dimensions or weights in excess of the maximum limits herein recommended shall be permitted only if authorized by special certificates issued by an appropriate State authority.

Table 12

Permissible Loads as Recommended by AASHO Policy Adopted April 1, 1946

Distance L in feet between the extremes of any group of axles	Maximum load W in pounds carried on any group of axles		
4	32,000	31	53,490
5	32,000	32	54,330
6	32,000	33	55,160
7	32,000	34	55,980
8	32,610	35	56,800
9	33,580	36	57,610
10	34,550	37	58,420
11	35,510	38	59,220
12	36,470	39	60,010
13	37,420	40	60,800
14	38,360	41	61,580
15	39,300	42	62,360
16	40,230	43	63,130
17	41,160	44	63,890
18	42,080	45	64,650
19	42,990	46	65,400
20	43,900	47	66,150
21	44,800	48	66,890
22	45,700	49	67,620
23	46,590	50	68,350
24	47,470	51	69,070
25	48,350	52	69,790
26	49,220	53	70,500
27	50,090	54	71,200
28	50,950	55	71,900
29	51,800	56	72,590
30	52,650	57	73,280

Ref. No. 3

APPENDIX B

Excerpts from the "Ontario Highway Traffic Act" relating to Lengths and Weights of Vehicles Permitted.

Section 19:2

No vehicle, other than a public vehicle or a semi-trailer as defined in clause b of subsection 6 of section 37, including load and contents, shall exceed the length of 33 feet, and no combination of vehicles, including load and contents, coupled together shall exceed the total length of 50 feet.

Section 34:2

Unless a special permit has been issued pursuant to section 35, no vehicle having a gross weight in excess of the following shall be moved upon wheels, rollers or otherwise over or upon a Class A Highway:

a) The gross weight of a vehicle other than those mentioned in clauses b, c, and d shall not exceed 28,000 pounds and the weight upon one axle shall not exceed 18,000 pounds, and if axles are spaced less than eight feet apart the weight on one axle shall not exceed 14,000 pounds.

b) The gross weight of a vehicle of three axles so designed that under any loading conditions the ratio of the weight on the middle axle to the weight on the rear axle remains constant shall not exceed 38,000 pounds and the weight on one axle shall not exceed 16,000 pounds.

c) When a conversion-unit consisting of a single axle designed to convert a two-axle vehicle into a three-axle vehicle as described in clause b is used with or attached to a two-axle vehicle, the gross weight of such converted two-axle vehicle shall not exceed 38,000 pounds.

d) The gross weight of a vehicle equipped wholly or in part with non-pneumatic tires shall not exceed 16,000 pounds and the weight upon one axle shall not exceed 12,000 pounds.

e) The gross weight of a semi-trailer with two axles so designed that under any loading conditions the weight on both axles remains constant shall not exceed 30,000 pounds.

Section 34:3

Unless a special permit has been issued pursuant to section 35, no vehicle having a gross weight in excess of the following shall be moved upon wheels, rollers or otherwise over or upon a Class B Highway:

a) The gross weight of a vehicle shall not exceed 22,000 pounds and the weight upon one axle shall not exceed 16,000 pounds and if axles are spaced less than eight feet apart the weight on one axle shall not exceed 10,000 pounds. 1955, c.29, s.5.

Section 35:1

The municipal corporation or other authority having jurisdiction over the highway may, upon application in writing, grant a permit for the moving of heavy vehicles, loads, objects or structures, in excess of the limits prescribed by section 19 or 34.