

# I/6 FATIGUE

## INTRODUCTION

Mid-nineteenth century research in Germany demonstrated that progressive failure in metals can be caused by cyclic loads with magnitudes well below that of loads capable of producing distress in a static tension test. Failures of this type were called "fatigue failures," and bridge engineers of the time attempted to avoid them by designing connections subjected to stress reversals for an increased fictitious stress equal to the sum of the higher stress plus one-half of the lower stress. Provisions, based on this approach, were incorporated in the Highway Bridge Specifications issued by the American Association of State Highway Officials (AASHTO) in 1926; they remained in the Specifications until 1965.

In 1936, fatigue provisions for several basic types of welded joints were included in the First Edition of the Specifications for Welded Highway and Railway Bridges, published by the American Welding Society (AWS). Based primarily on fatigue tests conducted at the University of Illinois, these specifications gave allowable stresses for three different life categories, i.e., numbers of load repetitions. Shortly after publication, the AWS fatigue provisions were made part of the AASHTO Specifications by reference, and retained in this form until 1965. Thus, while detailed specifications were provided for certain basic welded joints, the AASHTO Specifications made no mention of other welded, riveted, and bolted details.

An important change took place in 1965, when AASHTO adopted comprehensive fatigue provisions, covering plain material as well as various types of welded, bolted, and riveted details. While similar in form to the earlier AWS provisions, they were modified to reflect the results of the more recent research that had been conducted at the University of Illinois, the Research Laboratory of the United States Steel Corporation (USS), and elsewhere.

Several large research projects on fatigue of bridge members were started at Lehigh University in 1966. The results of the first of these studies formed the basis for major revisions of fatigue provisions that were adopted in the 1974 Interim Specifications of AASHTO,\* and, with some modifications, are currently (1980) in effect.

During the years since 1974, the scope of fatigue-related investigations has broadened. At United States Steel, a major study of bridge member fatigue behavior under simulated traffic loadings was completed in 1978. Governmental agencies including several state highway departments and the Federal Highway and Transportation Administration, conducted field studies of

traffic loadings and service stresses. Other studies focused on crack growth, corrosion fatigue, actual fatigue failures in existing bridges, methods of improving fatigue strength, methods of repairing fatigue cracks, and statistical methods of designing against fatigue.

Despite the recent occurrence of some well-publicized cases, remarkably few fatigue failures have occurred in highway bridges. Compared with the large number of bridges in service, these occurrences remain quite rare. Nonetheless, designers should guard against complacency, since there are several reasons to expect more fatigue problems in the future: 1) growth in both magnitude and frequency of loads as the number of trucks and the truck sizes increase; 2) greater use of high-strength steels with concomitant higher stresses; 3) more frequent use of welded details and connections that may contain stress-raisers; and 4) improvements and refinements in design procedures that allow higher stresses.

The best way to avoid such problems in new bridges is to give appropriate consideration to fatigue behavior at an early design stage. Additionally, safety checks should be performed on the many bridges that were erected before the adoption of modern fatigue design specifications. Thus, it is important that the engineer have access to current information.

To provide such information in a comprehensive and clear manner, this chapter has been divided into four sections as follows:

### Section I FATIGUE LOADINGS

A summary of traffic characteristics and the resulting truck loadings that occur on highway bridges

### SECTION II FATIGUE STRESSES

A discussion of 1) fatigue stresses caused by the loadings, and 2) the factors required for calculating these stresses

### SECTION III FATIGUE BEHAVIOR

An explanation of the nature of fatigue and a discussion of factors affecting fatigue life

### SECTION IV FATIGUE DESIGN

A discussion of design methods to counter fatigue, including information on present and possible future specifications, good and bad details, and methods of improving the fatigue life of particular details.

\* On November 11, 1973, the American Association of State Highway Officials (AASHTO) changed its name to the American Association of State Highway and Transportation Officials (AASHTO).

## SECTION I

# Fatigue Loadings

## CONTENTS

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<b>1. INTRODUCTION</b>	<b>6.3</b>
<b>2. TRAFFIC CHARACTERISTICS</b>	<b>6.3</b>
Volume	6.3
Composition	6.4
Time Variation	6.4
Directional Distribution	6.5
Lane Distribution	6.5
<b>3. FATIGUE LOADING DATA</b>	<b>6.5</b>
Truck Volume	6.5
Truck Weights	6.6
Fatigue Design Truck	6.6
Truck Spacing	6.7
Lane Loading	6.8
Permit Vehicles	6.9
<b>4. SUMMARY</b>	<b>6.9</b>
<b>5. REFERENCES</b>	<b>6.10</b>
<b>6. TABLES</b>	<b>6.11</b>
<b>7. FIGURES</b>	<b>6.18</b>
<b>8. APPENDIX A</b>	<b>6.22</b>

## 1. INTRODUCTION

This section first discusses traffic characteristics and then develops fatigue loading data from these characteristics. Specifically, these characteristics include the following traffic factors: volume, composition, time variation, directional distribution, and lane distribution. This information is then used to develop data on fatigue loadings caused by trucks; this being the major source of fatigue in highway structures since passenger autos are too light to cause significant stresses.

The information on traffic volume and composition is used to develop design criteria for daily truck volumes on rural and urban highways. Variations in the weight of various truck types are given. When combined with traffic composition data, they provide truckweight dis-

tribution curves for typical rural and urban traffic. Additionally, as a convenient means of representing typical rural and urban truck traffic, fatigue design trucks are devised. Also given, are data that can be used to develop a fatigue design truck for any traffic composition described by the usual traffic survey data.

Since truck spacing affects fatigue design, traffic flow theory is used to estimate the percentages of spacings closer than various given amounts. In these calculations, data on such traffic factors as time variation and directional distribution are used. The discussion also evaluates the significance of lane loading and permit vehicles on the basis of available data.

## 2. TRAFFIC CHARACTERISTICS

There is a vast amount of available data on highway traffic characteristics in the United States, and these facts and figures are being supplemented at an accelerating rate. Most states make extensive traffic counts on a continuous schedule, and many maintain traffic maps that show traffic volume on various highways. All states participate in an annual survey of truck traffic that is coordinated and assimilated by the Office of Highway Planning of the Federal Highway and Transportation Administration (FHWA).

The FHWA and some states also conduct field measurements of loadings on bridges and the stresses caused by these loadings. Numerous special purpose studies are conducted by traffic, pavement, and bridge engineers. Unfortunately, this deluge of information on the characteristics of traffic, contains much that is either not directly applicable in defining traffic loadings on bridges or is not readily available to bridge engineers. Information that is applicable to bridge loadings is summarized below.

### VOLUME

Traffic volume is the number of vehicles passing a given point on a highway during a unit of time.

Traffic engineers are in general agreement that the maximum volume of hourly traffic that can be reasonably anticipated on a highway or bridge is about 2000 vehicles per hour per lane. However, under extremely rare conditions, volumes as high as 2400 can occur.<sup>1</sup> Hence, it follows that in a single 24-hour period the traffic on a four-lane highway could not be expected to exceed 192,000 vehicles. In actuality, even on heavily travelled highways, the daily traffic is always far less than this figure since the peak volume is never maintained throughout a 24-hour period.

A frequently used term for traffic volume is Annual Average Daily Traffic (AADT). This is the total traffic on a highway—in both directions—averaged over a one-year period. Tables I and II give typical AADT data from nationwide surveys. These data were obtained at a number of locations on the Interstate system in 1975,<sup>2</sup> and the same had been done for various major highways<sup>3</sup> in 1965. Table I presents the weighted average traffic count for total rural and urban Interstate system highways in each state. The mean values of AADT are respectively 10,728 for rural, and 44,174 for urban locations. The corresponding highest values for any state are 31,185 (rural) and 83,433 (urban).

Unfortunately, Table I does not indicate the number of lanes on which these data were based. Most rural Interstate highways have four lanes, but many urban Interstate highways have more than four lanes.

Table II gives the AADT on a per lane basis. As shown, the mean values of AADT were 9336 (rural) and 21,904 (urban) for four-lane highways, and 3308 (rural) and 8344 (urban) for two-lane highways.

## COMPOSITION

The composition or makeup of traffic is stated in terms of quantities for specific categories of vehicles expressed as a percentage of the total traffic volume.

The composition varies widely with the highway location and type. There are, however, typical compositions for various highway types, derived from nationwide surveys,<sup>4,5</sup> and these are given in Table III. Here, the vehicles are separated into two groups: 1) cars and light trucks, which include panel, pickup, and other 2-axle/4-wheel trucks and 2) buses and heavy trucks, which include 2-axle/6-wheel trucks, and various tractor/semi-trailer units having more than two axles.

In the first group, the average loaded weight of the heaviest vehicle type ("cars") is 8.8 kips<sup>4</sup> (4,000 kg); this weight is small enough to be neglected when designing bridges against fatigue. However, "trucks," as categorized in the second group, must be taken into account.

As defined, the figures of Table III show that in 1972, trucks comprised about 17% (rural Interstate), 11% (rural primary) and 7% (urban primary) of the total highway traffic. If light trucks are added, these percentages become 25, 24, and 18. Thus, when making fatigue calculations with truck-traffic data from any source, it is important to know whether the statistics include light trucks, such as pickup and panel trucks. The studies<sup>5</sup> also showed that approximately 2/3 of such trucks are loaded.

While total traffic volume is largest in urban areas, the *percentage* of trucks on urban highways is considerably less than it is on rural highways. This is so because the volume of cars (on urban highways) increases at a faster rate than that of trucks.

During the years from 1966 to 1972 the percentage of trucks on rural Interstate highways increased from 14.7% to 16.2%. At the same time, the percentage of 5-axle tractor/semi-trailer units increased at a considerably higher rate; in 1974 this type of vehicle made up 50% of truck traffic. Their average loaded weight also increased slightly during this period. While the percentage of trucks of this type was increasing, the percentage of 3- and 4-axle tractor/semi-trailers, that have a much smaller gross weight, was decreasing. Hence, the composition of truck traffic is moving toward a greater proportion of heavier vehicles.

## TIME VARIATION

The volume of traffic varies with time; the changes in magnitude reflect a response to economic and social transportation demands. These cyclic variations not

only relate to the hour of the day, but also to the day of the week, and the season.

Seasonal variations usually show higher traffic volumes during the summer. This is particularly true on rural highways. As an example, on three Connecticut highways in July and August, the average daily traffic (ADT) was higher than the annual average daily traffic (AADT), by the following percentages: 1) 80 to 100% on a rural recreational highway, 2) 30% on a main rural Interstate highway, and 3) 10% on a suburban state highway.<sup>1</sup>

Daily variations follow weekday/weekend cycles. Usually, the ADT remains about constant from Monday through Thursday and then it increases continuously from Friday through Sunday.<sup>1</sup> On rural highways, high traffic volumes normally occur during weekends, however, on urban highways the greater volume occurs on weekdays. The Sunday ADT on a Connecticut rural recreational highway was reported as 165% of the weekly ADT, whereas on a Nashville urban highway,<sup>1</sup> it was 75% of the weekly ADT.

Hourly variations are related to business hours, hence there is a large difference between weekdays and weekends. Figure 1 illustrates a typical weekday variation of car traffic for both an urban and a rural Interstate highway.<sup>5</sup> The variation for the urban highway shows typical peaks before and after normal working hours, while the rural traffic remained approximately constant during the day and then decreased to a minimum between 1 and 4 AM. This illustration shows traffic volume in *both* directions. At some locations, the peaks in one-directional traffic volume may be even higher than the two-directional peaks. This is a result of unequal directional distributions that are discussed later.

There is far less variation in the seasonal, daily, and hourly periods for trucks than there is for cars. This is particularly true for heavy tractor/semi-trailer units at rural locations. For example, Figure 2 shows the variation of heavy trucks (specifically, 5-axle tractor/semi-trailer) on an urban and a rural Interstate highway.<sup>5</sup> The truck traffic on the urban highway increases gradually from a minimum at about midnight to a maximum at about noon. On the rural Interstate highway the hourly variation is small. (Similar data from another source shows even less variation of heavy-truck volume on various rural highways.<sup>6</sup>)

The 30th highest hourly traffic volume (30HV) is often used in highway design. For a given type of highway, the ratio of 30 HV to AADT (expressed as a percentage) has been shown to vary within a fairly narrow range. For main rural highways, this ratio varies from 12% to 18% with an average value of 15%,<sup>1</sup> and the ratio for urban facilities will vary from 7% to 18% and has an average value of 11 percent. These ratios can be used for estimating peak-hour volumes from the AADT for a particular highway.

Prior to the advent of spiralling increases in petroleum prices, the national figure for vehicle-miles

traveled was consistently increasing at an annual rate of about 4.6%.<sup>3</sup> Hence, the assumed traffic growth for a given highway or bridge was usually taken as 5% per annum.

### DIRECTIONAL DISTRIBUTION

On most two-way highways, total traffic volumes are approximately equivalent in both directions over a one-year period. But, differences in directional distributions may occur during certain hours, days, and months of the year. Daily and monthly directional differences are important primarily on highways that carry heavy recreational traffic; except for such cases, directional distribution is usually balanced even over a one-day period. The most important hourly variations occur before and after work in urban areas outside of the central business district. In such areas, 60% to 80% of peak-hour traffic volume flows in one direction;<sup>1</sup> the average is 67%. Usually, most of the directional difference is in car traffic, but significant differences in truck traffic may also occur at some locations.

### LANE DISTRIBUTION

On multi-lane highways, the percentage of traffic that passes in each lane depends on the total traffic volume.<sup>6</sup> For four-lane highways, the percentage traveling in the shoulder lane decreases from about 98%, at a one-directional hourly traffic volume approaching zero, to about 55%, at an hourly volume of 1200. The percentage of the *truck* traffic that travels in the shoulder lane remains much more uniform; it varies from a value close to 100% to about 80% over this same range in traffic volume.

For a six-lane highway, the percentage of the total traffic that travels in the shoulder lane decreases as the hourly traffic volume increases; the percentage in the center lane first increases and then later it lessens; in the median lane the percentage steadily increases. But, here again, most truck traffic is in the shoulder lane. As the one-directional hourly traffic volume grows to 2200, truck traffic percentages in the individual lanes vary approximately as follows: shoulder lane 85 to 70, center lane 15 to 25, and median lane 0 to 5.

## 3. FATIGUE-LOADING DATA

Fatigue-loading data suitable for use in highway bridge design, are developed from the previously described traffic characteristics and certain other information. Trucks, as already noted, are the major cause of fatigue loadings on highway bridges; normally, the weight of cars is too small to create any significant stresses. This is the reason that almost all fatigue-loading data relate to trucks. Specifically, design data are developed on truck volume, truck weights, truck spacing, lane loading, and permit vehicles.

### TRUCK VOLUME

As defined by the Annual Average Daily Truck Traffic (AADTT), the volumes at various locations show considerable differences. This is caused by differences in both total truck volume (Tables I and II), and percentages of trucks in the traffic. Nonetheless, these data were used to develop design guidelines for situations where more specific information for the bridge site is unavailable.

The guidelines provide "design daily truck traffic" (DDTT) volume for bridges located either in rural or urban areas, and having any number of lanes. DDTT for bridges is used in designing every lane even though the truck volumes differ in the various lanes.

As has been noted, about 90% of truck traffic on four-lane highways is in the shoulder lane, and on six-lane highways about 75% is in the shoulder lane with the remainder using the adjacent lane. The portion of traffic in the adjacent lane causes stresses in the girders underneath the shoulder lane, but such stresses are of a smaller magnitude than if these vehicles had travelled the shoulder lane. (Traffic in the opposite direction is

considered far enough from the shoulder lane so that it may be disregarded.) Hence, for multi-lane bridges it is conservative to design the shoulder lane for the total traffic in one direction. Normally, this traffic volume is used in designing *all* girders — including those under the median lane that are subjected to lower truck traffic volumes. On two-lane bridges, traffic in the opposite direction is closer to the lane under consideration. For this reason, the DDTT is taken as the total traffic in both directions.

Analysis of Tables I, II, and III suggested that rational values of design daily truck traffic for major rural and urban highways could be obtained by multiplying the number of lanes by 400 and 600, respectively. To determine the validity of this simple rule in all cases, the AADT corresponding to the DDTT was calculated and compared with observed AADT values (Tables I and II). These calculations and comparisons are summarized in Table IV, the first line of which lists DDTT values calculated by the rule. The second line lists corresponding AADTT in both directions. In keeping with the DDTT, as defined, the AADTT values are equal to DDTT values for two-lane bridges and twice that for multi-lane bridges. Total traffic (AADT) was obtained from truck traffic (AADTT) by dividing by the assumed percentage of trucks in the total traffic. Two percentages were used: 1) a conservative, rounded value, and 2) an average value from Table III.

Resulting AADT values are listed on lines three and four (Table IV). For comparison, observed AADT values are given on the next three lines: the first gives mean values from Table I, the second gives mean values from Table II, and the third gives the 95% survival



value from Table II. This last value is an estimated AADT exceeded at only 5% of the highway locations in the United States.

The AADT values, from Table I, listed in Table IV assume six-lane highways at urban locations, and four-lane highways at rural locations.

For highways with four or more lanes, the AADT corresponding to the design daily truck traffic is close to the estimated 95% survival AADT. These data mean that the design daily truck traffic was exceeded only at about 5% of the highway locations in the nationwide survey summarized in Table II.

The AADT values corresponding to the design daily truck traffic for two-lane highways are between the observed mean value and the estimated 95% survival value. This seemingly lesser degree of conservatism is, however, counter-balanced by the fact that only half of the design daily truck traffic will actually pass in the lane under consideration; the rest will pass in the adjacent lane. The foregoing shows that the design truck traffic values given in Table IV are conservative for highways with any number of lanes.

## TRUCK WEIGHTS

Federal-aid amendments of 1974 increased the permissible weights of vehicles operating on Interstate highways to 20 kips (9,100 kg) for a single axle, 34 kips (15,000 kg) for a tandem axle, and 80 kips (36,400 kg) for the gross weight.<sup>7</sup> Legal truck weights in most states are similar to the permissible weights shown in Table V. At the end of 1972, the maximum legal gross weight for trucks was about 73 kips (33,000 kg) in most states.<sup>5</sup>

Some states permit small tolerances above the legal weights to take into account possible inaccuracies in the scales; a number of states modify the legal limits at certain times of the year as, for example, the spring thaw period. In most cases, states will grant weight and size increases for vehicles hauling certain commodities. All states will allow certain over-weight or over-size vehicles to use highways, if a permit is obtained; this aspect is discussed later.

Table VI presents average gross weights and axle loads, derived from nationwide surveys,<sup>4,5</sup> for the types of trucks previously discussed under the heading of Composition. (While buses were not weighed in these surveys, their usual weight range is from 7 to 35 kips<sup>1</sup> (3,000 to 16,000 kg.)) Measured ranges<sup>1</sup> of axle spacings for tractor/semi-trailer combinations are given in Table VII.

Figures 3 and 4 show probability-density curves of gross weight distribution for each truck type. These curves are normalized by expressing the gross weights as a percentage of the average gross weight for the given type of truck. The area beneath a curve between any two normalized weights, gives the fraction of trucks within that range. The distributions of weight are for the compositions of truck traffic observed in the surveys, which included the empty trucks that comprise about one-third of the truck traffic.<sup>5</sup>

The distribution of weights for each truck type is far from the statistically normal distribution. For the 5-axle trailer/semi-trailer units—the largest and single most important component of typical truck traffic—there are two peaks in the probability-density curve. One lies close to the maximum legal weight limit, and the other is at the empty weight for this truck type. All other truck types have only one major peak, and this is usually near the lower end of the range.

Composite gross-weight distribution (probability-density) curves for several truck traffic compositions given in Table III, are plotted in Figures 5 and 6. Data for these curves were obtained prior to 1973 when the maximum legal gross weight was 73 kips (33,000 kg) in most states.<sup>5</sup> Again, the area beneath a curve between any two weights, represents the fraction of weights within that range. The normalized weight distributions, given in Figures 3 and 4 for the various truck types, were used to calculate the composite distribution curves. A gross-weight distribution curve<sup>8</sup> for the 1970 nationwide loadometer survey is also plotted in Figure 6. This curve gives the distribution of weights above 20 kips (9,100 kg), while the other curves in Figures 5 and 6 give the distribution of weights for all trucks except light trucks—as defined earlier. All of the distribution curves in Figures 5 and 6 are similar and have three peaks. One peak is at about 28 kips (13,000 kg), and another smaller peak is at about 70 kips (32,000 kg). These peaks correspond to the empty and loaded weights for 5-axle trailer/semi-trailer units. The peaks are lower for urban than for rural traffic because the 5-axle trailer/semi-trailers make up a smaller percentage of total urban truck traffic. The third peak—at about 10 kips (4,500 kg)—corresponds to the empty weight of the 2-axle/6-wheel trucks. The third peak is highest for urban traffic because, in these locations, the percentage of such trucks is highest.

## FATIGUE DESIGN TRUCK

A given composition of truck traffic can be conveniently represented by a *fatigue design truck* with a gross weight selected so that a given number of passages of this truck would cause the same amount of fatigue damage as the same number of passages by trucks of different weights in the traffic. The gross weight of the fatigue design truck can be calculated from a histogram of truck weights<sup>9</sup> by the following formula:

$$W = \left[ \sum \alpha_i W_i^3 \right]^{1/3} \quad (1)$$

where:

$\alpha_i$  = the fraction of trucks within  
a given weight interval

$W_i$  = the midpoint of that interval

Gross weights for fatigue design trucks corresponding to the compositions of traffic in Figures 5 and 6 are given in Table VIII under the heading "From Nationwide Traffic Surveys." These weights range from 43.6

kips (19,800 kg) for urban primary highways to 51.5 kips (23,400 kg) for rural Interstate highways. Similar fatigue-design-truck gross weights were calculated from extensive data compiled in a recent study of bridge loadings<sup>10</sup> for metropolitan, urban and rural highways (Table VIII). These weights are in general agreement with those of the nationwide surveys.

Fatigue-design-truck gross weights have also been calculated using data obtained from several individual weighing stations. Published in studies of stresses in bridges,<sup>11,12,13,14</sup> these weights also appear in Table VIII. Most are within the range of values found in the nationwide surveys, but the highest is well above this average. This may have resulted from an unusual concentration of heavier trucks at that particular site. However, it may also reflect a tendency for weighing stations personnel to weigh only the larger vehicles while allowing the smaller to go unchecked.<sup>12</sup> On the other hand, drivers, knowing that their vehicles are overweight, are sometimes able to bypass weighing stations. Because of the possibility of such statistical biases, caution should be used in applying weighing-station records for fatigue design.

The preceding paragraphs explained how to calculate the gross weight of a fatigue-design truck when a composite gross-weight histogram for the entire truck traffic is known. Frequently, however, traffic surveys do not provide such histograms; instead, the surveys only give the percentages of various types of trucks and the *average* weight of each type.<sup>5</sup> If these data were used directly in Equation 1, the resulting fatigue design weight would be too low. For example, calculated in this way, the fatigue design weights in Table VIII would be about 10% lower.

The fatigue-design weight, however, can be calculated correctly from data of the percentages and average weights of various types of trucks by using the  $W'_F/W_a$  values in Table IX. Thus, the fatigue-design weight can be calculated for a particular highway location by using typical traffic survey data.  $W_a$  is the average weight of a given truck type.  $W'_F$  is the fatigue-design weight for that type of truck and was obtained by applying Equation 1 to the weight distribution curve for that type (Figure 3 or 4).

To calculate the fatigue-design weight for a particular highway location from typical traffic survey data:

- 1) Multiply the *average* gross weight of each type of truck (from the traffic survey) by the corresponding  $W'_F/W_a$  value from Table IX to get  $W'_F$  for that type of truck. If the categories of truck types in the traffic survey do not correspond to the six categories in Table IX they must be combined into these six categories.
- 2) Calculate the fatigue-design weight for the particular highway location from Equation 1; in this equation,  $\alpha_i$  is the percentage of trucks of a given type and  $W_i$  is the value of  $W'_F$  for that type.

The fraction of the total fatigue damage, caused by any component of the traffic, is equal to the  $\sum \alpha_i W_i^3$  for that

component divided by the summation for the total traffic. Percentages for traffic compositions from the nationwide surveys are given in Table IX and show that about 95 percent of the fatigue damage is caused by the 4- and 5-axle trucks.

## TRUCK SPACING

For light traffic volumes, the spacing of vehicles is accurately defined by a Poisson distribution<sup>1</sup> which gives the fraction of spacings less than a given value. Specifically,

$$F = 1 - e^{-\left(\frac{qd}{v}\right)} \quad (2)$$

in which  $F$  is the fraction of vehicles at a spacing (front-to-front) less than  $d$ ,  $v$  is the average speed of the traffic, and  $q$  is the one-directional traffic volume, usually for a one-hour period. This equation can be applied to all traffic in one direction regardless of the number of lanes.

The equation is normally considered accurate for traffic volumes below 500 vehicles per hour per lane. With higher traffic volumes on two-lane highways, the equation tends to underestimate the percentage of short spacings because the lack of passing opportunities causes bunching behind slower vehicles. Another limitation of the theory is that it assumes the minimum possible spacing to be zero although it must actually be equal to the average length of vehicle.

For four-lane highways, spacings can be zero so the second limitation does not apply. Also, the effect of the first limitation is greatly reduced. Consequently, the equation is accurate at much higher traffic volumes. For example, at a traffic volume of 1800 vehicles/hour, the distribution of headways (time between vehicles, or  $d/v$ ) predicted by the Poisson distribution, is very close to the measured distribution given in Figure 4.18 of Reference 1. However, the measured fraction of headways less than one second, was about 30 percent less than the predicted fraction.

Truck spacing alone rather than total vehicular spacing is the important factor in fatigue considerations. Normally,\* the distribution of truck spacings can be obtained from Equation 2 by using the truck traffic volume as  $q$ .<sup>11</sup> The average traffic speed ( $v$ ) is a function of the total traffic volume.<sup>1</sup> Hence, truck-spacing distribution is the function of total traffic volume as well as a percentage of trucks in that traffic. Truck spacings for traffic on a four-lane highway are given in Table X for both 10% and 20% trucks. (These percentages are representative of urban and rural Interstate highways, respectively.) Spacings are given for traffic volumes up to the capacity of the highway. At the highest volumes, actual spacings would probably differ somewhat from the values given in the table. For low traffic volumes, the spacings would also be valid for two-lane highways.

\* Steep uphill grades restrict trucks from passing freely, hence they tend to bunch and, as a result, behave as if they were on a two-lane highway.

Development of the information in Table X is detailed in Appendix A.

Distribution of truck spacings can also be related to the annual average daily traffic (AADT) by accounting for the variation in hourly volume that occurs as a result of daily, weekly, and seasonal traffic patterns. To establish this relationship, a histogram giving a typical percentage distribution of hourly one-directional traffic volumes for a year, would be valuable. Unfortunately, such information is not available. However, daily traffic patterns cause the largest variations in hourly traffic volume. Therefore, the daily patterns shown in Figures 1 and 2 were used to calculate truck spacings as a function of AADT.\*

Details of these calculations are given in Appendix A. Here, in brief, it was assumed that the distribution of hourly traffic volumes for the day applies throughout the year. This is not strictly true because the superposition of seasonal and other patterns on the daily pattern will cause variations in hourly traffic volume larger than the daily pattern alone. However, with the exception of highways with very large seasonal variations due to recreational traffic, these effects are not large. This is especially true because large variations in average hourly traffic volume have relatively small effects on the distribution of truck spacings.

Investigation of the effects of an unequal volume of car traffic in the two directions at peak hours (discussed in Appendix A) showed that this did not significantly alter the calculated truck-spacing distributions.

Curves giving the percentage of truck spacings less than 50, 100, 150 and 200 ft (15, 40, 46, 61 m) as a function of AADT were developed for two cases. Figure 7 gives such curves for four-lane rural highways and is based on the rural daily traffic patterns (Figs. 1 and 2)

Figure 8 shows similar curves for four-lane urban highways and is based on urban daily traffic patterns (Figs. 1 and 2). Both Figures 7 and 8 are also applicable to two-lane highways up to the indicated limiting values of AADT. Above these values the curves tend to underestimate the number of close spacings on two-lane highways. Four-lane highways on steep uphill grades should be treated as two-lane, because the truck passings are restricted.

The percentage of closely spaced trucks is quite small for normal AADT volumes. On four-lane rural highways with an AADT of 10,000 (the approximate mean value for *rural* Interstate highways) only about 1.3% of truck spacings are less than 100 ft (30 m). On four-lane urban highways with an AADT of 40,000 (the approximate mean value for *urban* Interstate highways — including those with more than four lanes) about 3.4% of truck spacings are less than 100 ft. For the same AADT, close truck spacing percentages are higher on rural highways than on urban highways because of the higher percentage of trucks in rural traffic. For example, at an AADT of 20,000 the percentage of

spacings less than 100 ft (30 m) was 2.6 for rural highways and 1.6 for urban highways. The 4- and 5-axle trucks that do most of the fatigue damage make up about 50% to 75% of the total truck traffic. The percentage of spacings between such trucks that are less than 100 ft is even less than the corresponding percentage for the total truck traffic.

Percentages given in Figure 8 for urban traffic are about 20% to 28% larger than would be calculated for a uniform distribution of traffic throughout each day. The percentages given in Figure 7 for rural traffic are almost identical to those for a uniform distribution of traffic. This indicates that variations in the time distribution of traffic volume do not have a large effect on the spacing percentages.

### LANE LOADING

Closely spaced traffic can be represented by a uniform lane loading calculated from the average vehicle spacing and weight. The closest possible spacing occurs when traffic is stopped.<sup>15</sup> However, because this condition rarely occurs on most bridges, it is generally inappropriate for fatigue design. Consequently, the following discussion is based on moving traffic. Quite complex and lengthy calculations would be needed to make detailed predictions of the frequency and magnitudes of lane loading occurrences. However, an indication of the expected magnitude and frequency can be obtained by considering a highway carrying 2000 vehicles per hour per lane. As noted, this volume is usually assumed as the maximum capacity of a highway, although higher volumes are possible under ideal conditions.

Traffic volume for the 30th highest hour of a year is usually about 15% of the AADT. Thus, a highway with an AADT of 13,300 vehicles per lane, or about 53,000 vehicles for four lanes, would be expected to have a traffic volume exceeding 2000 (15% of 13,300) vehicles per hour per lane only 30 times a year. As shown in Tables I and II, an AADT of 53,000 for a four-lane highway represents a very high traffic volume.

From Appendix Equations A6 and A2, the average speed for a traffic volume of 2000 vehicles per hour per lane is 30 mph and the corresponding average spacing (front-to-front of vehicles) in the lane is about 80 ft (24 m). The average spacing of trucks for this traffic volume (Table X) is normally far too great to allow trucks to be treated as part of a uniform lane loading. Instead, it would be reasonable to calculate lane loading for cars alone and to use one or two concentrated loads in combination with this lane loading to represent trucks.\*

If the average weight of a car\* is taken as 5 kips, the lane loading for a traffic volume of 2000 vehicles per hour per lane is 62.5 lb/ft (93 kg/m). This is only about 1/10 of the AASHTO<sup>16</sup> HS20 lane loading of 640 lb/ft (950 kg/m), which is based on a lane of closely spaced,

\* Total traffic in both directions.

\* As defined earlier, 2-axle/4-wheel trucks, such as panel and pickup trucks, are included in the car category.



heavily loaded trucks such as may occur when traffic is stopped. While this extreme condition is appropriate for static design, it would occur so rarely (perhaps never), that it would not affect fatigue design.

To summarize, lane loading is a negligible factor in the fatigue design of highway bridges except under an unusual circumstance that will cause truck bunching, such as a traffic light at the end of a bridge, or a steep grade.

### PERMIT VEHICLES

Earlier, it was stated that vehicles whose weight and size exceed legal limits may be allowed on highways under permits issued by state regulatory agencies. Policies for issuing permits vary widely among the states.<sup>17,18</sup>

Truck permits are issued by most states on both single-trip and multiple-trip bases. The multiple-trip permits usually allow an unlimited number of trips within a specified time period, ordinarily one, three, six or 12 months. Almost every state will routinely issue permits for vehicles that do not exceed size and weight limits established by the regulatory agency. Special considerations such as route analysis are required before a state will issue permits for vehicles that exceed these limits. The number and spacing of axles for vehicles that exceed these weight limits are usually greater than for normal trucks so that the deleterious effect of a greater gross weight is reduced. Moreover, special travel restrictions are often imposed to lessen these trucks' effects on bridges. In Maryland,<sup>19</sup> for example, the following restrictions are imposed: 1) no other vehicles are permitted on the bridge during the permit vehicle's transit; 2) the vehicle's speed is kept down to a crawl; and 3) the vehicle must travel down the center of the bridge in line with the main girders.

The gross weight and individual axle weight limits set by most states for routine permits, are listed in Table V. Gross weights range up to 150 kips (68,000 kg) and average about 104 kips (47,000 kg). The axle weights range up to 29 kips (13,000 kg) and average about 23 kips (10,000 kg). The Heavy and Specialized Carriers Conference recommends<sup>20</sup> weights slightly higher for routine permits: 100 kips (45,000 kg) gross weight for 5-axle vehicles, 120 kips (54,000 kg) gross

weight for 6-axle vehicles, and 24 kips (11,000 kg) axle weight. The California Department of Transportation has developed a series of P loads to represent overweight permit vehicles.<sup>7</sup> On many of this state's highways, routine permits are issued for vehicles closely resembling the P-9 truck, which has a gross weight of 218 kips (99,000 kg) distributed over one single axle and four tandem axles spaced at 18-ft (5.5 m) intervals.

The number of single- and multiple-trip permits issued by various states in 1977 are listed in Table V.<sup>18</sup> About 90% of the permits are for single trips, 4% are one-month multiple-trip permits, 4% are one-year multiple-trip permits, and 2% are multiple-trip permits of other durations. About 30% of the single-trip permits and 50% of the multiple-trip permits are for overweight or combined overweight and oversize. Since 1966, the number of both single-trip and multiple-trip permits increased by about 65%. In 1966, about a third of the permits were for mobile homes, which were oversize only, and another third were for construction equipment, half of which were overweight.<sup>17</sup> Also in 1966, about 80% of the overweight permits were for gross weights less than 100 kips.<sup>17</sup>

No data are available on the percentage of permit vehicles in the truck traffic or on the frequency of occurrence of permit vehicles on a typical bridge. The preceding figures suggest that permit vehicles with gross weights exceeding 100 kips (45,000 kg) represent a very small percentage of typical truck traffic; only a third of the permits issued are for overweight vehicles, and of these only a small proportion exceed 100 kips.

Gross weights of trucks in the nationwide surveys cited earlier ranged up to 100 kips and presumably included permit vehicles. Furthermore, it has been shown<sup>21,22</sup> that a few heavy overloads have a beneficial effect on fatigue resistance because they cause favorable residual stresses at stress raisers. Because permit vehicles with gross weights less than 100 kips have been included in the fatigue loadings previously discussed, and because the percentage of permit vehicles with gross weights exceeding 100 kips is very small, special consideration need not be given to the effects of permit vehicles on fatigue behavior except in unusual circumstances.

## 4. SUMMARY

Traffic characteristics of highway bridges and the resulting fatigue loadings were discussed. Data from nationwide studies show that *total* traffic volumes vary widely with respect to: 1) location, 2) time of day, 3) day of the week, and 4) the season. Truck volume, on the other hand, is much more uniform. Normally, at rural locations, less than 20% of the traffic are trucks (not including panel, pickup, and other 2-axle/4-wheel vehicles, and at urban locations less than 10% are trucks.

Analysis of these data showed that reasonable design values of daily truck traffic volume can be obtained by multiplying the number of lanes by 400 (for major rural highways), or 600 (for major urban highways).

Distribution of truck weights for typical traffic compositions is given. These distributions are used to compute the weights of fatigue design trucks that serve as convenient representations of typical truck traffic. These weights are about 45 and 50 kips (20,000 and

23,000 kg) for rural and urban traffic respectively. About 95% of fatigue damage from typical traffic is caused by 4- and 5-axle trucks.

The number of closely spaced trucks is very small for normal AADT volumes. For example, on four-lane rural highways with an AADT of 10,000—the approximate mean value for rural Interstate highways—only about 1.3% of the truck spacings are less than 100 ft. (30 m). Average truck spacing for such traffic is not close enough to treat the trucks as a uniform lane loading.

Even at peak traffic volumes, they are not spaced close enough to be treated as a uniform lane load. (While, at such volumes, cars may be spaced close enough to be treated as a uniform lane loading, the resulting magnitude is very small.) Thus, it may be concluded that lane loading can be disregarded in the fatigue design of highway bridges unless there are unusual circumstances. Similarly, special consideration need not be given to the effects of permit vehicles on fatigue behavior except where unusual conditions occur.

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## 6. TABLES

**TABLE I**  
**Annual Average Daily Traffic\* on Interstate Highways**

Region	State	AADT		Region	State	AADT	
		Rural	Urban			Rural	Urban
New England	Connecticut	28,250	53,122	West North Central	Iowa	8,746	18,245
	Maine	8,051	15,071		Kansas	6,251	20,488
	Massachusetts	19,416	35,388		Minnesota	6,842	34,347
	New Hampshire	9,420	20,744		Missouri	12,155	47,792
	Rhode Island	19,621	44,417		Nebraska	7,835	33,177
	Vermont	5,550	11,700		North Dakota	3,512	6,777
	Average*	12,089	40,242		South Dakota	4,772	7,435
Middle Atlantic				East South Central	Average	7,537	33,056
	New Jersey	21,901	53,383		Alabama	8,460	24,088
	New York	10,044	42,531		Kentucky	14,461	45,852
	Pennsylvania	13,293	37,764		Mississippi	7,122	17,071
South Atlantic (North)	Average	13,160	43,519	West South Central	Tennessee	15,821	52,443
	Delaware	—	47,735		Average	11,939	36,027
	Dist. of Col.	—	55,643		Arkansas	12,138	23,720
	Maryland	20,737	69,665		Louisiana	11,009	31,529
	Virginia	17,553	38,330		Oklahoma	9,970	30,919
	West Virginia	7,758	22,717		Texas	9,621	40,609
South Atlantic (South)	Average	15,367	50,843	Mountain	Average	10,206	37,071
	Florida	14,160	38,384		Arizona	7,496	39,550
	Georgia	16,061	52,161		Colorado	7,517	44,108
	North Carolina	14,411	29,649		Idaho	—	—
	South Carolina	10,025	26,075		Montana	2,952	4,904
East North Central	Average	13,851	39,307		Nevada	5,040	27,746
	Illinois	10,316	54,536		New Mexico	6,062	20,935
	Indiana	14,881	40,210		Utah	6,468	31,859
	Michigan	15,262	43,036		Wyoming	3,704	5,050
	Ohio	17,974	41,553		Average	5,624	30,039
	Wisconsin	15,533	44,706	Pacific	California	14,185	83,433
	Average	14,348	44,597		Oregon	9,704	25,691
	Alaska	—	—		Washington	12,709	47,717
	Hawaii	31,185	69,512		Average	12,847	69,761
	Average	31,185	69,512				

\*Total traffic volume in all lanes in both directions from the 1975 Traveled-Way Traffic Map published by the Federal Highway Administration. All figures are weighted averages. The weighted averages for the entire United States are 10,728 for rural highways and 44,174 for urban highways.

**TABLE II**  
**Annual Average Daily Traffic per Lane for Various Classifications of Highways\***

Classification**	Number of Highway Locations	Observed		Calculated***	
		Mean	Max.	Median	95% Survival
Rural					
Freeway (fully controlled access)	63	1,985	7,648	1,864	3,877
Expressway (partially controlled access)	19	3,072	7,173	2,885	6,000
Four Lane (uncontrolled access)	44	2,515	8,069	2,362	4,912
Combined	126	2,334	8,069	2,192	4,558
Two Lane	238	1,654	5,515	1,553	3,230
Urban					
Freeway (fully controlled access)	51	6,853	21,318	6,435	13,384
Expressway (partially controlled access)	18	5,270	12,358	4,949	10,292
Four Lane (uncontrolled access)	54	4,244	7,895	3,985	8,289
Combined	123	5,476	21,318	5,142	10,695
Two Lane	54	4,172	9,940	3,918	8,148

\*Data from the 1965 Highway Capacity Manual.

\*\*All highways except those identified as two-lane highways had more than two lanes. All except one of the rural freeways and expressways had four lanes. The urban freeways and expressways were approximately evenly divided between four and six lanes.

\*\*\*Calculated from an assumed Rayleigh distribution defined as  $p = ve^{-1/2v^2}$  in which p is the probability density and v is the nondimensional traffic volume. For this distribution, median/mean = 0.939 and 95% survival/mean = 1.954. The observed mean was multiplied by these ratios to get the median and 95% survival values.

**TABLE III**  
**Composition of Traffic**

Vehicle			Percent of All Traffic						Percent of Trucks**	
			Rural Interstate*		Rural Primary*		Urban Primary*			Combined**
Category	Type	Axles/Wheels	1966	1972	1966	1972	1966	1972	1974	1974
Cars and light trucks	Cars		79.3	74.7	75.7	76.1	84.5	82.2	74.3	
	Panel/pickup		4.9	8.3	9.1	11.9	7.6	10.0	12.0	
	Single unit	2 axle/4 wheel	0.6	0.4	0.7	0.6	0.9	0.6		
				84.8	83.4	85.5	88.6	93.0	92.8	86.3
Heavy trucks	Buses		0.5	0.4	0.4	0.4	0.6	0.6	0.4	2.9
	Single unit	2 axle/6 wheel	2.7	2.7	3.7	3.3	3.0	2.7	3.4	24.8
	Single unit	3 axle	0.5	0.5	0.8	0.8	0.4	0.5	0.7	5.1
	Tractor-semitrailer	3 axle†	1.1	0.6	0.8	0.4	0.5	0.3	0.5	3.7
	Tractor-semitrailer	4 axle†	3.6	2.0	2.6	1.3	0.8	0.9	1.6	11.7
	Tractor-semitrailer	5 axle†	6.1	9.4	4.4	4.8	1.0	1.8	6.6	48.2
	All other***		0.7	1.0	1.8	0.4	0.7	0.4	0.5	3.6
			15.2	16.6	14.6	11.4	7.0	7.2	13.7	100.0

\*Nationwide data for the particular type of highway.<sup>6</sup>

\*\*Nationwide data that includes data for various types of highways.<sup>5</sup>

\*\*\*Primary 3-axle truck/2-axle trailer units and 2-axle tractor/1-axle semi-trailer/2-axle trailer units.

†Most of the 3-axle units are 2S1 units, most of the 4-axle units are 2S2 units, and most of the 5-axle units are 3S2 units. The number before the S indicates the number of axles in the tractor and the number after the S indicates the number of axles in the semi-trailer.

**TABLE IV**  
**Truck Volume Data for Major Highways**

Highway Location	Parameter	Lanes				Notes
		2	4	6	8	
Rural	Design Daily Truck Traffic	800	1,600	2,400	—	1
	Corresponding AADTT	800	3,200	4,800	—	2
	Corresponding AADT (20% trucks)	4,000	16,000	24,000	—	2
	Corresponding AADT (17% trucks)	4,700	18,800	28,200	—	2
	Observed Mean AADT From Table I	—	10,700	—	—	2, 3
	Observed Mean AADT From Table II	3,300	9,300	14,000	—	2, 4
	95% Survival AADT From Table II	6,500	18,200	27,300	—	2, 5
Urban	Design Daily Truck Traffic	1,200	2,400	3,600	4,800	1
	Corresponding AADTT	1,200	4,800	7,200	9,600	2
	Corresponding AADT (10% trucks)	12,000	48,000	72,000	96,000	2
	Corresponding AADT (7% trucks)	17,100	68,600	102,900	137,100	2
	Observed Mean AADT From Table I	—	—	44,200	—	2, 3
	Observed Mean AADT From Table II	8,300	22,000	32,900	43,800	2, 4
	95% Survival AADT From Table II	16,300	42,800	64,200	85,600	2, 5

**Notes:**

- (1) Total daily truck volume in both lanes of a two-lane bridge and all lanes in one direction on bridges with more than two lanes.
- (2) The AADT and AADTT include all lanes in both directions.
- (3) This is the mean for all states.
- (4) This is the mean for various highway locations and for different highway classifications.
- (5) 95% survival value estimated as indicated in Table II. This estimated AADT value is exceeded only at 5% of United States locations.



**TABLE V**  
**Weight Limits and Permits Issued**

State	Gross Weight, Kips		Single-Axle Weight, Kips		Tandem-Axle Legal Weight, Kips	Number of Permits Issued in 1977			
	Legal	Routine Permit	Legal	Routine Permit		Single Trip		Multiple Trip	
						Total	Overweight**	Total	Overweight**
Alabama	84.0	80.0	20.0	22.0	40.0	31,295	5,417	1,674	0
Arizona	80.0	—	20.0	—	34.0	45,907	5,669	8,412	1,929
Arkansas	73.3	108.0	18.0	25.0	32.0	87,531	21,351	0	0
California	80.0	T	20.0	28.0	34.0	99,286	—	6,700	—
Colorado	85.0*	100.0	20.0	—	36.0	55,235	14,106	6,069	3,351
Connecticut	73.0	122.0	22.4	—	36.0	—	—	—	—
Delaware	80.0	90.0	20.0	—	40.0	19,826	4,081	350	350
Distr. of Columbia	73.3*	—	22.0	—	38.0	1,479	—	747	—
Florida	80.0	100.0	20.0	22.0	40.0	77,113	18,806	8,204	5,648
Georgia	80.0	100.0	18.0	25.0	36.0	48,856	—	3,651	—
Idaho	105.5	131.9	20.0	22.5	34.0	22,907	19,259	4,044	3,439
Illinois	73.3	88.0	18.0	20.0	32.0	121,130	42,400	11,400	4,555
Indiana	73.3	104.0	18.0	28.0	32.0	100,261	—	11,215	—
Iowa	73.3	75.0	18.0	—	32.0	48,319	—	9,789	—
Kansas	85.5	95.0	20.0	22.0	34.0	60,715	60,715	0	0
Kentucky	82.0	96.0	20.0	24.0	34.0	56,190	28,900	730	396
Louisiana	80.0	114.0	20.0	24.0	34.0	140,364	35,861	300	0
Maine	80.0	130.0	22.0	—	38.0	14,453	9,403	2,960	1,915
Maryland	73.3	90.0	22.4	30.0	40.0	—	—	—	—
Massachusetts	80.0	120.0	22.4	—	36.0	21,250	—	9,750	—
Michigan	80.0	132.0	20.0	—	34.0	76,507	—	12,308	—
Minnesota	80.0	84.0	20.0	20.0	34.0	43,339	7,489	10,605	1,989
Mississippi	73.3	109.0	18.0	—	32.0	46,369	13,129	727	0
Missouri	73.3	86.0	18.0	19.0	32.0	75,032	5,812	4,089	—
Montana	76.8	105.5	18.0	20.0	32.0	—	11,259	—	—
Nebraska	95.0*	106.0	20.0	—	34.0	37,233	24,919	629	629
Nevada	84.0*	129.0	20.0	18.0	34.0	7,205	2,306	2,140	852
New Hampshire	80.0	120.0	22.4	—	36.0	12,586	2,976	565	565
New Jersey	80.0	—	22.4	—	34.0	—	—	—	—
New Mexico	86.4	115.0	21.6	—	34.3	41,390	6,375	3,379	340
New York	80.0	110.0	22.4	—	36.0	45,395	11,604	23,194	20,724
North Carolina	76.0	94.5	19.0	25.0	—	29,914	13,188	7,150	1,607
North Dakota	80.0	87.0	20.0	20.0	34.0	40,146	12,197	0	0
Ohio	80.0*	T	20.0	29.0	34.0	97,792	29,283	1,381	0
Oklahoma	90.0*	82.0	20.0	20.0	34.0	—	—	—	—
Oregon	80.0	96.0	20.0	21.5	34.0	22,196	—	4,690	—
Pennsylvania	73.3	150.0	22.4	27.0	36.0	302,446	40,681	422	79
Rhode Island	80.0	100.0	22.4	—	36.0	2,022	—	40	—
South Carolina	80.0	90.0	20.0	—	36.0	5,229	—	30,367	—
South Dakota	95.0*	90.0	20.0	20.0	34.0	—	—	—	—
Tennessee	73.3	84.0	18.0	18.0	32.0	58,721	—	6,530	—
Texas	80.0	100.0	20.0	—	34.0	336,646	—	22,599	—
Utah	80.0	—	20.0	—	34.0	38,053	11,587	25,507	19,884
Vermont	80.0	100.0	22.4	—	36.0	5,715	745	764	405
Virginia	76.0	102.5	20.0	24.0	34.0	28,175	6,950	10,375	1,975
Washington	80.0	—	20.0	22.0	34.0	121,784	72,709	25,220	9,473
West Virginia	80.0	110.0	20.0	22.0	34.0	105,648	17,665	0	0
Wisconsin	80.0	110.0	20.0	25.0	34.0	44,288	16,946	24,076	0
Wyoming	101.0	135.0	20.0	22.5	36.0	69,684	36,933	704	0
						2,745,632	662,300	303,416	80,105

\*A smaller gross weight applies to 5-axle trucks.

\*\*Includes permits for overweight alone and combined overweight and oversize.

T indicates that a table or formula is used to obtain the maximum gross weight for routine permits. 1 kip = 453.6 kg

**TABLE VI**  
**Average Gross Weights and Axle Loads for Various Types of Trucks**

Type	Vehicle		Gross Weight, kips			1972 Axle Loads,** kips				
	Axles/Wheels	Loading*	1966**	1972**	1974***	A	B	C	D	E
Panel/pickup	2 axle	E	4.3	4.6	4.9	2.5	2.1			
		L	5.3	5.7	5.9	2.7	3.0			
		C	4.7	5.0	5.6	2.6	2.4			
Single unit	2 axle/4 wheel	E	5.4	5.9	6.5	3.0	2.9			
		L	6.8	7.5	8.8	3.3	4.2			
		C	6.3	6.9	8.0	3.2	3.7			
Single unit	2 axle/6 wheel	E	9.6	10.4	10.9	4.6	5.8			
		L	15.3	15.8	15.7	5.5	10.3			
		C	13.1	13.8	14.1	5.2	8.6			
Single unit	3 axle	E	18.1	19.2	20.1	7.7	6.0	5.5		
		L	36.1	38.9	38.2	10.7	14.3	13.9		
		C	27.1	29.5	32.2	9.3	10.3	9.9		
Tractor/ semi-trailer	3 axle	E	20.7	22.1	22.9	6.8	8.3	6.9		
		L	32.3	30.8	30.5	7.3	12.4	11.0		
		C	28.3	27.8	28.0	7.2	11.0	9.6		
Tractor/ semi-trailer	4 axle	E	24.4	26.1	26.7	7.3	8.4	5.1	5.2	
		L	47.0	45.4	43.2	8.3	15.1	11.0	11.1	
		C	38.7	38.8	37.7	7.9	12.8	9.0	9.1	
Tractor/ semi-trailer	5 axle	E	29.2	30.2	30.7	8.2	6.5	6.0	4.6	4.9
		L	61.7	62.3	62.0	9.2	13.7	13.2	13.0	13.2
		C	51.4	51.4	51.6	8.9	11.2	10.8	10.1	10.4
Tractor/ trailer†	3 axle/2 axle	E	26.7	28.3	—	8.3	5.8	5.5	4.5	4.2
		L	68.4	70.3	—	10.3	14.8	14.4	15.4	15.3
		C	53.3	54.4	—	9.6	11.4	11.1	11.3	11.1
Tractor/ semi-trailer- trailer†	2 axle/1 axle/ 2 axle	E	29.8	31.7	—	8.2	7.7	5.5	5.3	5.0
		L	63.4	63.9	—	9.2	15.4	14.3	12.6	12.4
		C	55.0	58.2	—	9.0	14.0	12.7	11.3	11.1

\* E = empty, L = loaded, C = combined empty and loaded.

\*\* Nationwide data taken on rural primary highways.<sup>6</sup>

\*\*\* Nationwide data taken on various types of highways;<sup>5</sup> combined calculated by assuming 1/3 of the trucks are empty.

† These units are the primary components of the "all other" category in Table III.

1 kip = 453.6 kg

**TABLE VII**  
**Axle Spacings for Tractor/Semi-trailer Units\***

Axles	Value*	Unit			
		3-axle	4-axle	5-axle	5-axle Spread Tandem
AB	Lowest	8	8	8	9
	Modal Range	11 to 12	11 to 12	10 to 11	10 to 11
	Highest	16	18	17	15
	% Within Modal Range	40.7 %	33.6 %	43.1 %	36.4 %
BC	Lowest	15	11	3	3
	Modal Range	31 to 32	23 to 24	4 to 5	4 to 5
	Highest	38	33	6	5
	% Within Modal Range	11.1 %	18.3 %	96.7 %	94.5 %
CD	Lowest		3	8	6
	Modal Range		4 to 5	27 to 28	22 to 23
	Highest		8	over 38	25
	% Within Modal Range		91.2 %	23.3 %	23.6 %
DE	Lowest			3	8
	Modal Range			4 to 5	9 to 10
	Highest			8	25
	% Within Modal Range			91.2 %	72.7 %

\*Spacings in feet. The modal range is the 1-foot (0.3 m) spacing increment that occurs most frequently. The % within the modal range is the percent of spacings within this 1-foot range. The percent distribution of spacings is given in Reference 2.  
1 kip = 453.6 kg

**TABLE VIII**  
**Gross Weights of Fatigue Design Trucks for Various Compositions of Traffic**

Reference	Highway Type	Date	Gross Weight, kips
<b>From Nationwide Traffic Surveys</b>			
9	Combined	1970	50.8
6	Rural Interstate	1972	51.5
6	Rural Primary	1972	47.2
6	Urban Primary	1972	43.6
5	Combined	1974	49.0
<b>From A Study Including 16 Locations</b>			
11	Metropolitan	P1974*	43.4
11	Urban	P1974	49.6
11	Rural	P1974	48.3
<b>From Individual Weighing Stations</b>			
12	Rural Interstate	P1974	57.4
13	Urban Interstate	1969	49.0
14	Rural Interstate	1969	52.4
14	Rural Interstate	1969	44.6
15	Rural Interstate	P1972	49.4
15	Rural Interstate	P1972	46.8
15	Rural Interstate	P1972	48.1
15	Rural Interstate	P1972	48.3
15	Rural Primary	P1972	50.3
15	Rural Primary	P1972	48.9

\*P means publication date; the year during which the data were taken was not given.  
1 kip = 453.6 kg

**TABLE IX**  
**Fatigue Parameters for Different Types of Trucks**

Truck			Percentage of Fatigue Damage			
			1972 Rural Interstate	1972 Rural Primary	1972 Urban Primary	1974 Combined
Type	Axles/Wheels	$W_F/W_a^*$				
Single Unit	2 axle/6 wheel	1.153	0.4	1.1	2.0	1.0
Single Unit	3 axle	1.212	1.1	3.2	4.3	2.7
Tractor-semitrailer	3 axle	1.058	0.7	0.9	1.4	0.8
Tractor-semitrailer	4 axle	1.097	6.9	8.6	12.6	7.2
Tractor-semitrailer	5 axle	1.114	79.7	77.7	61.7	80.3
Other**		1.114	11.1	8.5	18.0	8.0
Total for 4- and 5-axle trucks			97.7	94.8	92.3	95.5

\*  $W_F$  is the fatigue-design weight of a given type of truck; and  $W_a$  is the average gross weight of that type of truck.

\*\* Primarily 3-axle tractor/2-axle trailer units and 2-axle tractor/1-axle semitrailer/2-axle trailer units.

**TABLE X**  
**Truck Spacings for Four-Lane Highways**

Traffic*			10% Trucks							20% Trucks						
			Truck Volume, trucks/hr	Ave. Spacing ft	% Truck Spacings less than				Truck Volume, trucks/hr	Ave. Spacing ft	% Truck Spacings less than					
					50'	100'	150'	200'			50'	100'	150'	200'		
262	59	1192	26	11917	0.4	0.8	1.2	1.7	52	5959	0.8	1.7	2.5	3.3		
516	58	595	52	5948	0.8	1.7	2.5	3.3	103	2974	1.7	3.3	4.9	6.5		
760	57	397	76	3969	1.2	2.5	3.7	4.9	152	1984	2.5	4.9	7.3	9.6		
996	56	298	100	2975	1.7	3.3	4.9	6.5	199	1488	3.3	6.5	9.6	12.6		
1440	54	198	144	1984	2.5	4.9	7.3	9.6	288	922	4.9	9.6	14.0	18.2		
1849	52	149	185	1488	3.3	6.5	9.6	12.6	370	744	6.5	12.6	18.2	23.6		
2222	50	119	222	1191	4.1	8.0	11.8	15.5	444	595	8.0	15.5	22.2	28.5		
3000	45	79	300	794	6.1	11.8	17.2	22.3	600	397	11.8	22.2	31.5	39.6		
3556	40	60	356	595	8.0	15.5	22.3	28.5	711	298	15.5	28.5	39.6	48.9		
4000	30	40	400	397	11.8	22.3	31.5	39.6	800	198	22.3	39.6	53.0	63.5		

\* Total traffic in both lanes in one direction.

1 ft = 0.3048 m

1 kip = 1.609 kilometer/hr

## 7. FIGURES

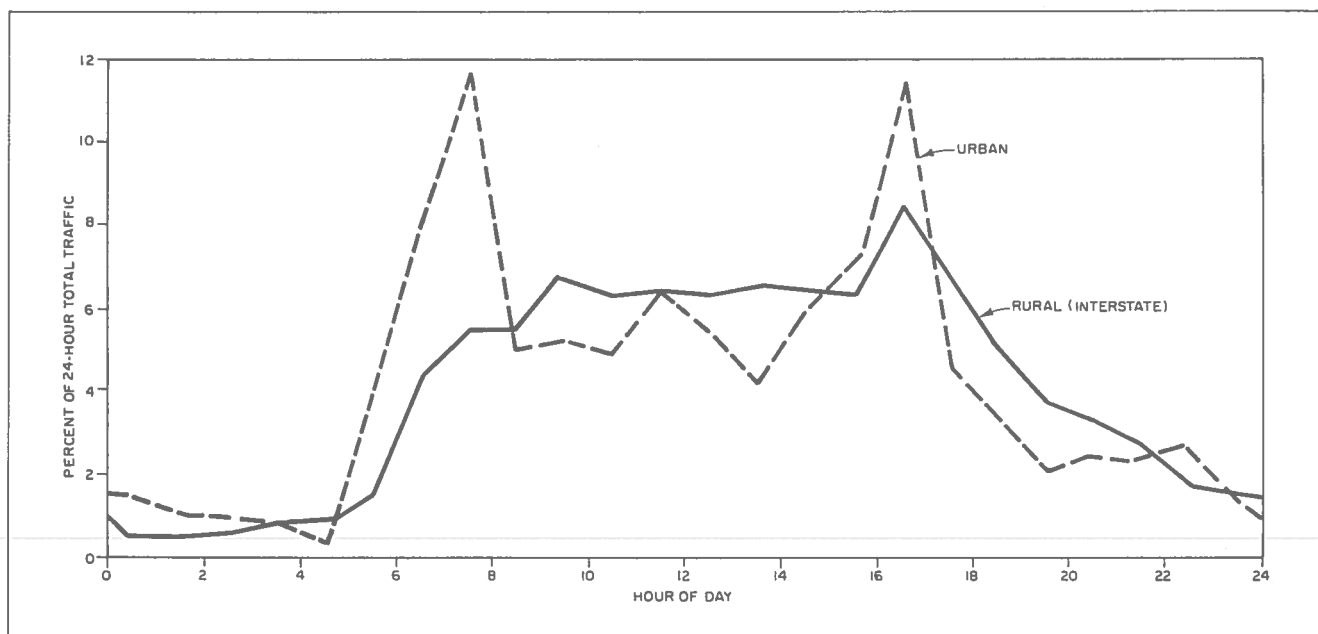


Figure 1. Hourly variation of *car* traffic in a 24-hour period.

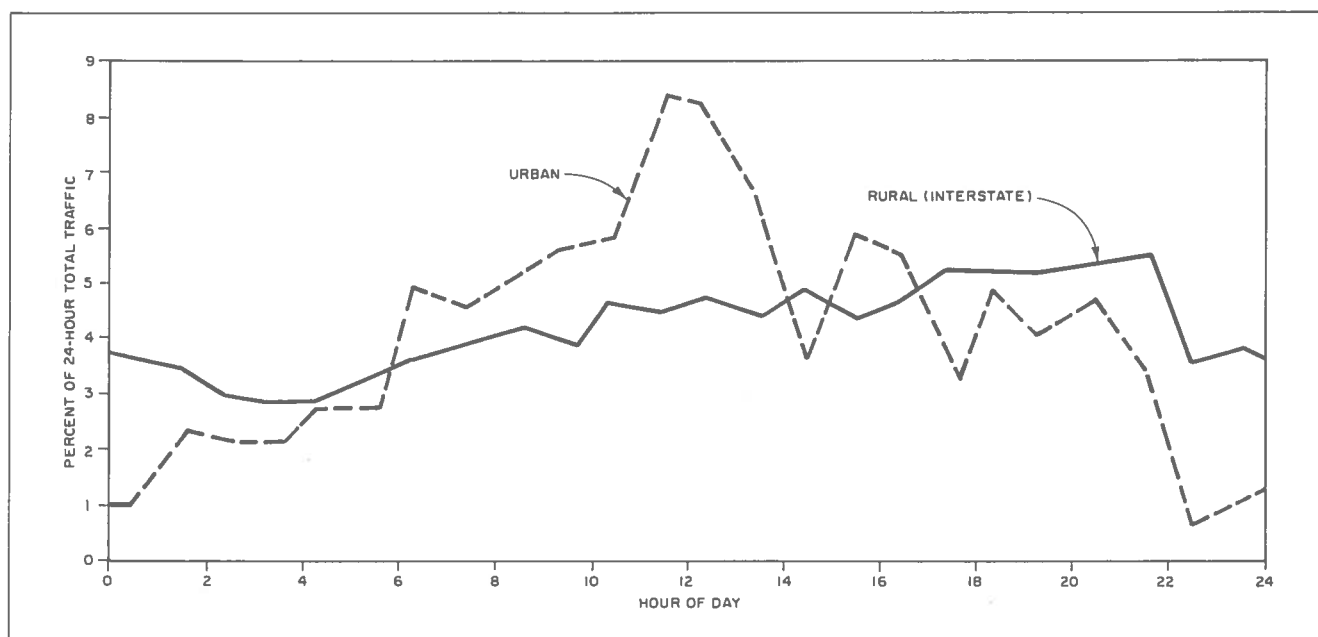
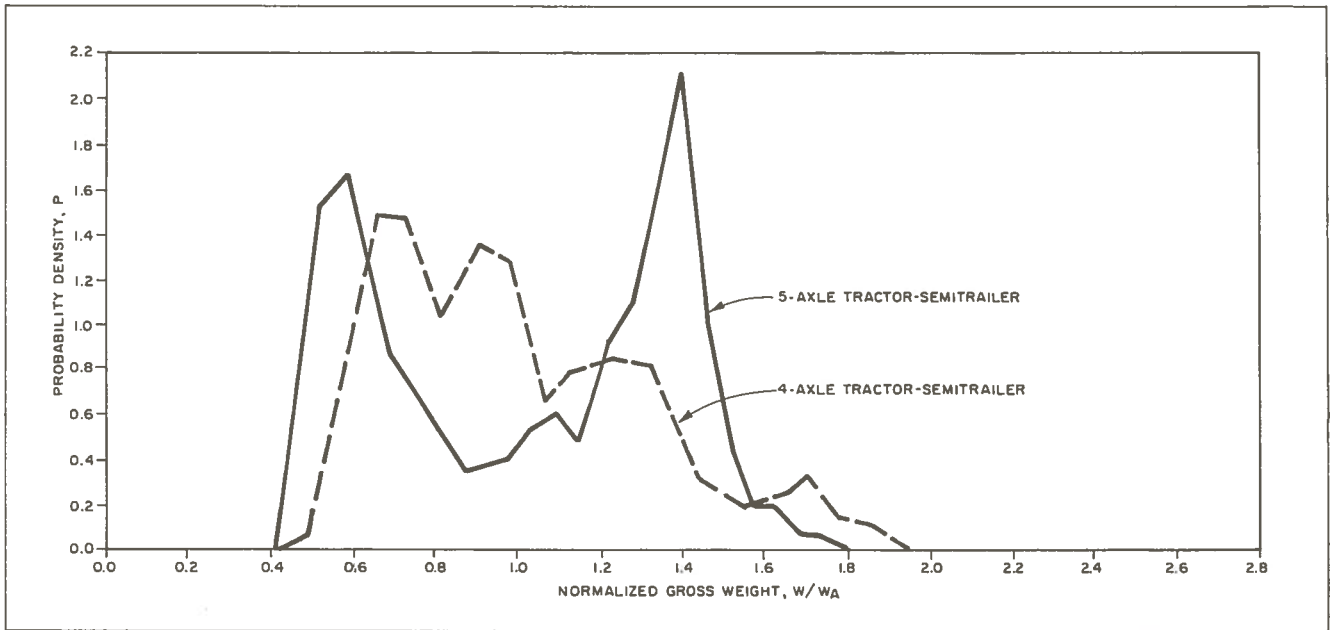
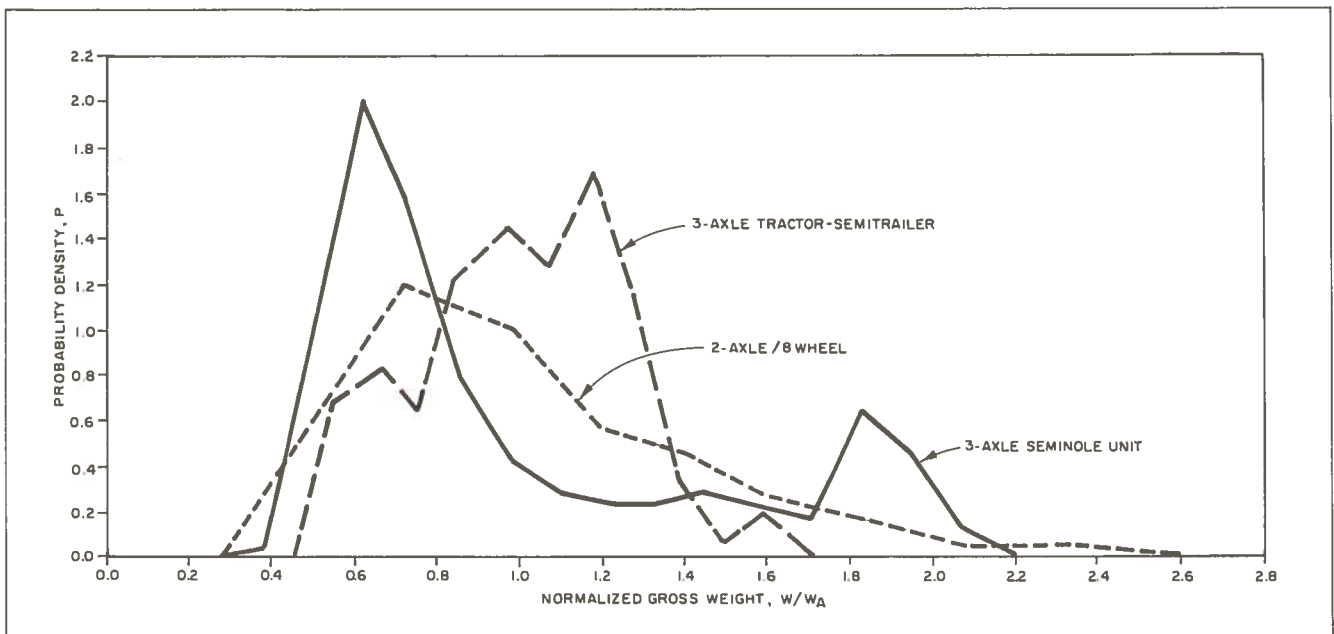


Figure 2. Hourly variation of *truck* traffic in a 24-hour period.

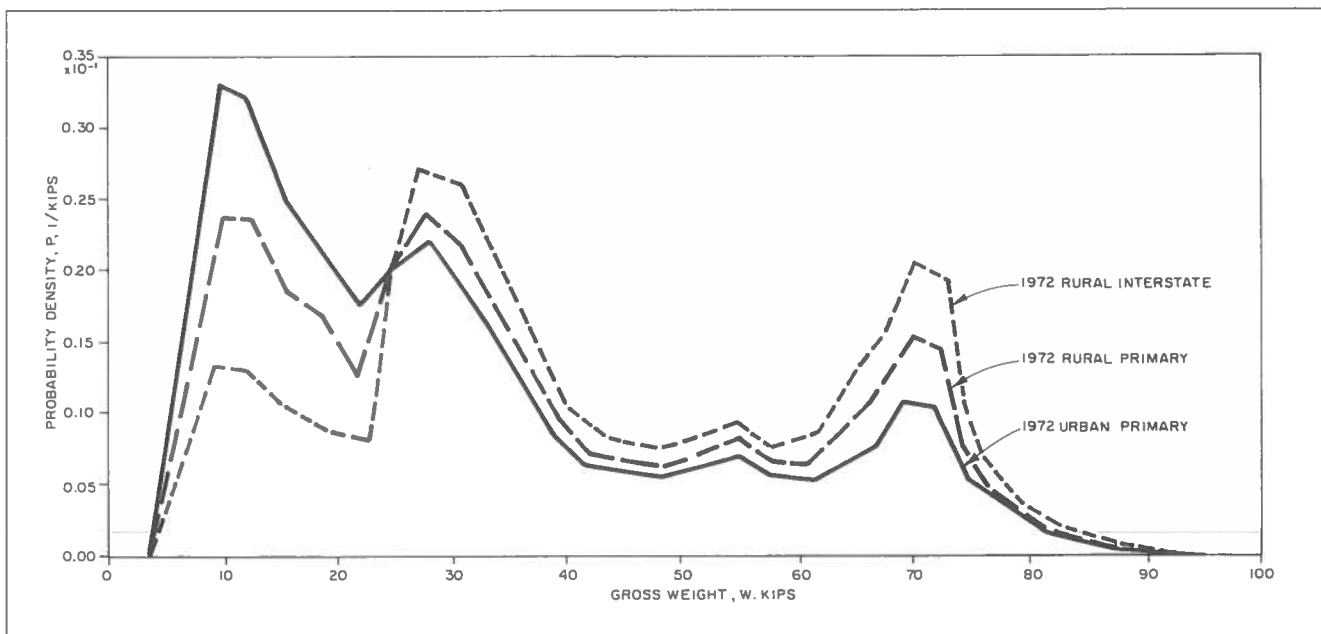




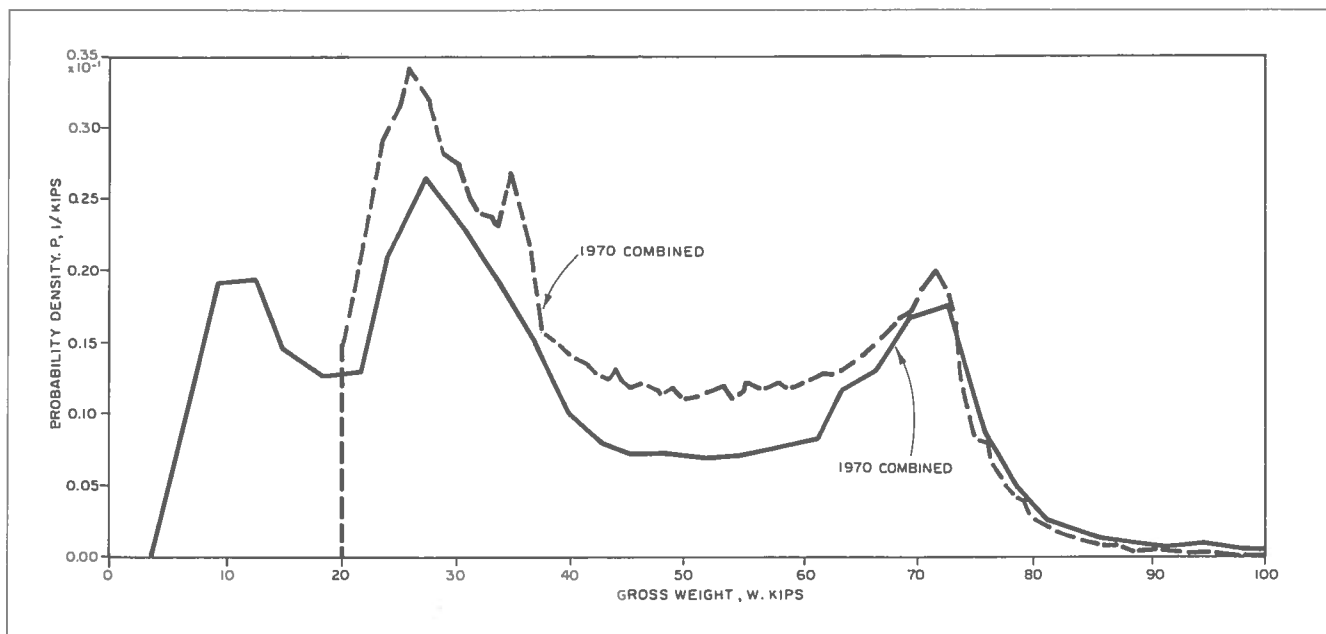
**Figure 3.** Gross-weight distribution curves for truck types: 4-axle tractor/semi-trailer and 5-axle tractor/semi-trailer.



**Figure 4.** Gross-weight distribution curves for three truck types: 2-axle/8-wheel, 3-axle tractor/semi-trailer, and 3-axle single unit.



**Figure 5.** Gross-weight distribution curves for various types of traffic during 1972.



**Figure 6.** Gross-weight distribution curves for combined traffic during 1970 and 1974.

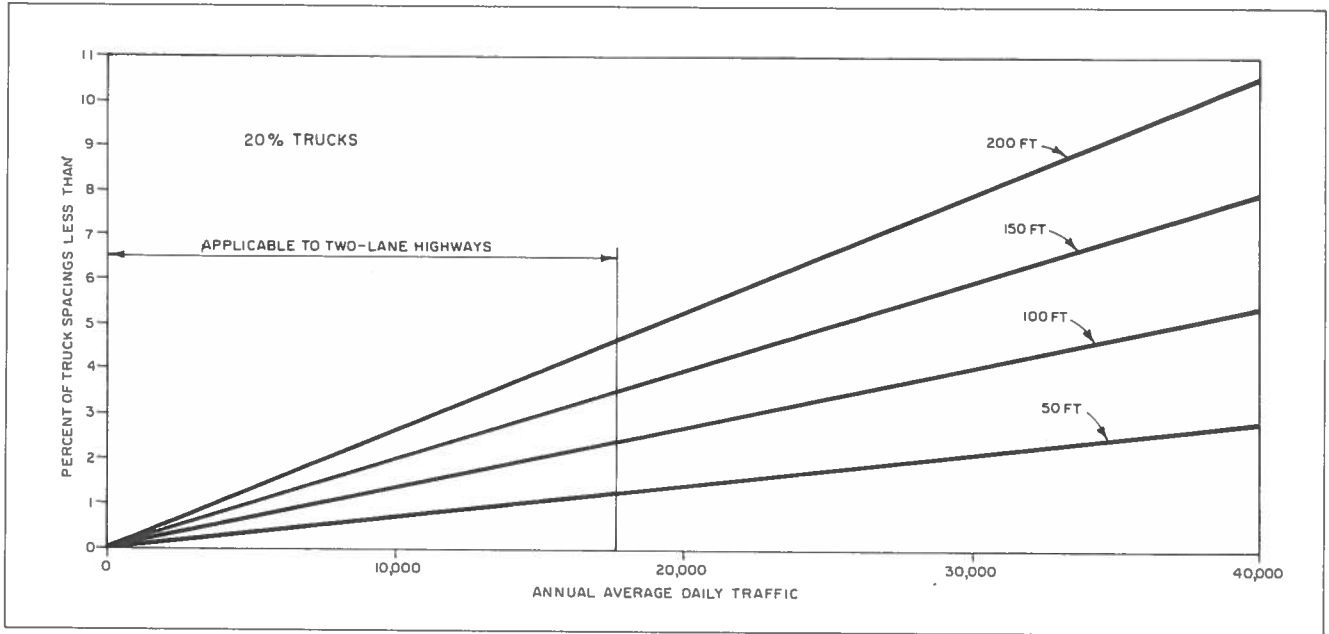


Figure 7. Truck spacings for four-lane *rural* highways — based on AADT with 20% trucks.

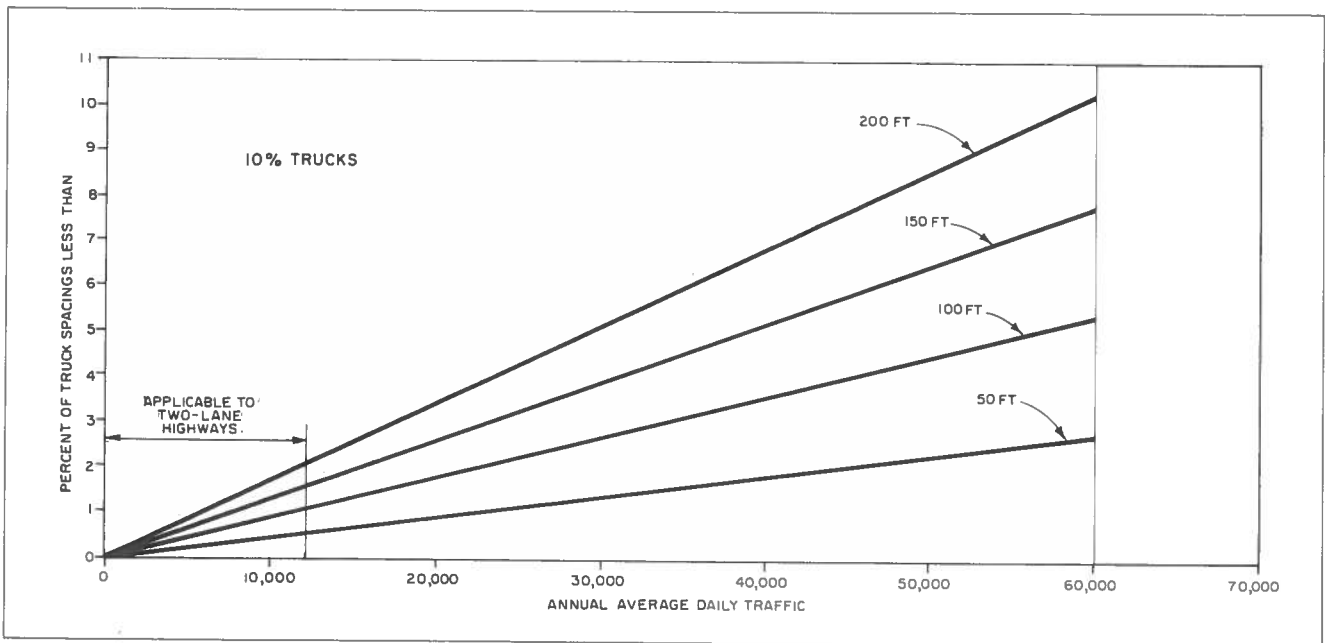


Figure 8. Truck spacings for four-lane *urban* highways — based on AADT with 10% trucks.

## 8. APPENDIX A

### TRUCK SPACINGS

For steady-state traffic, average headway (time interval between arrival of successive vehicles) is<sup>1</sup>

$$h_{ave} = \frac{1}{q} \quad (A-1)$$

where  $q$  is the traffic volume in one direction. Average spacing of vehicles,  $d_{ave}$ , is

$$d_{ave} = v h_{ave} = \frac{v}{q} \quad (A-2)$$

The fraction of headways less than a time,  $t$ , is given by<sup>1</sup>

$$F = 1 - e^{-qt} \quad (A-3)$$

Furthermore,

$$t = \frac{d}{v} \quad (A-4)$$

where  $d$  is the spacing of vehicles (front to front) and  $v$  is the average speed. Therefore, the fraction of spacings less than  $d$  is given by

$$F = 1 - e^{-\left(\frac{qd}{v}\right)} \quad (A-5)$$

The units of  $q$ ,  $t$ ,  $d$ , and  $v$  must, of course, be consistent.

According to traffic flow theory,<sup>1</sup> the traffic volume is related to average speed by

$$\frac{q}{q_{max}} = 4 \left[ \frac{v}{v_{max}} - \left( \frac{v}{v_{max}} \right)^2 \right] \quad (A-6)$$

In this equation,  $q_{max}$  is the maximum traffic volume and is assumed to be 4000 vehicles/hour, which is the normal capacity for two lanes of traffic. Maximum average speed,  $v_{max}$ , is assumed to be 60 mph—the speed when traffic is very light.

Data in Table X were calculated as follows: First, the average speed of traffic ( $v$ ) for a given volume was calculated from Equation A-6. The truck volume corresponding to this total volume is

$$q_T = P_T q \quad (A-7)$$

where  $P_T$  is the fraction of vehicles that are trucks. The fraction of truck spacings less than a given distance was calculated by applying Equation A-5 to the truck traffic alone. Thus, values of  $q_T$ ,  $v$ , and  $d$  were inserted into Equation A-5 to get  $F$ . Average spacings for the trucks or total traffic were obtained from Equation A-2.

To confirm the assumption that Equation A-5 can be applied to truck traffic alone, the theoretical distribution of truck spacings calculated in this way was compared with the distributions observed on nine bridges.<sup>11</sup> Specifically, curves giving the percentage of spacings less than 50, 100, 150, and 200 ft (15, 30, 46, 61 m) as a function of hourly truck volume were calculated

from Equation A-5. Corresponding observed percentages, for truck volumes ranging from 60 to 250 trucks/hour, agreed reasonably well with these theoretical curves. Furthermore, the observed percentages were generally less than the theoretical percentages for spacings of 50 and 100 ft (15 and 30 m). Thus, the theory was conservative for these data.

Curves in Figures 7 and 8 were obtained by making the above calculations for each hour in the day and taking weighted averages of these values. First, volumes of car traffic,  $q_c$ , and truck traffic,  $q_T$ , were computed from

$$q_T = \left( \frac{AADT}{2} \right) P_T P_T \quad (A-8)$$

$$q_c = \left( \frac{AADT}{2} \right) (1 - P_T) p_c \quad (A-9)$$

where  $P_T$  and  $p_c$  are the fractions of total daily truck and car traffic, respectively, in the hour under consideration. Values of these fractions were obtained from Figures 1 and 2. The total traffic volume is

$$q = q_T + q_c \quad (A-10)$$

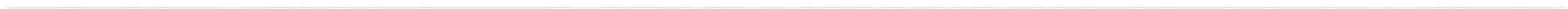
Limiting AADT values—below which the curves are applicable to two-lane highways—were selected so that  $q$  for the highest hour does not exceed 560 vehicles/hour in one direction. The average values of  $q$  corresponding to these limiting values are 333 and 250 vehicles/hour for rural and urban highways, respectively.

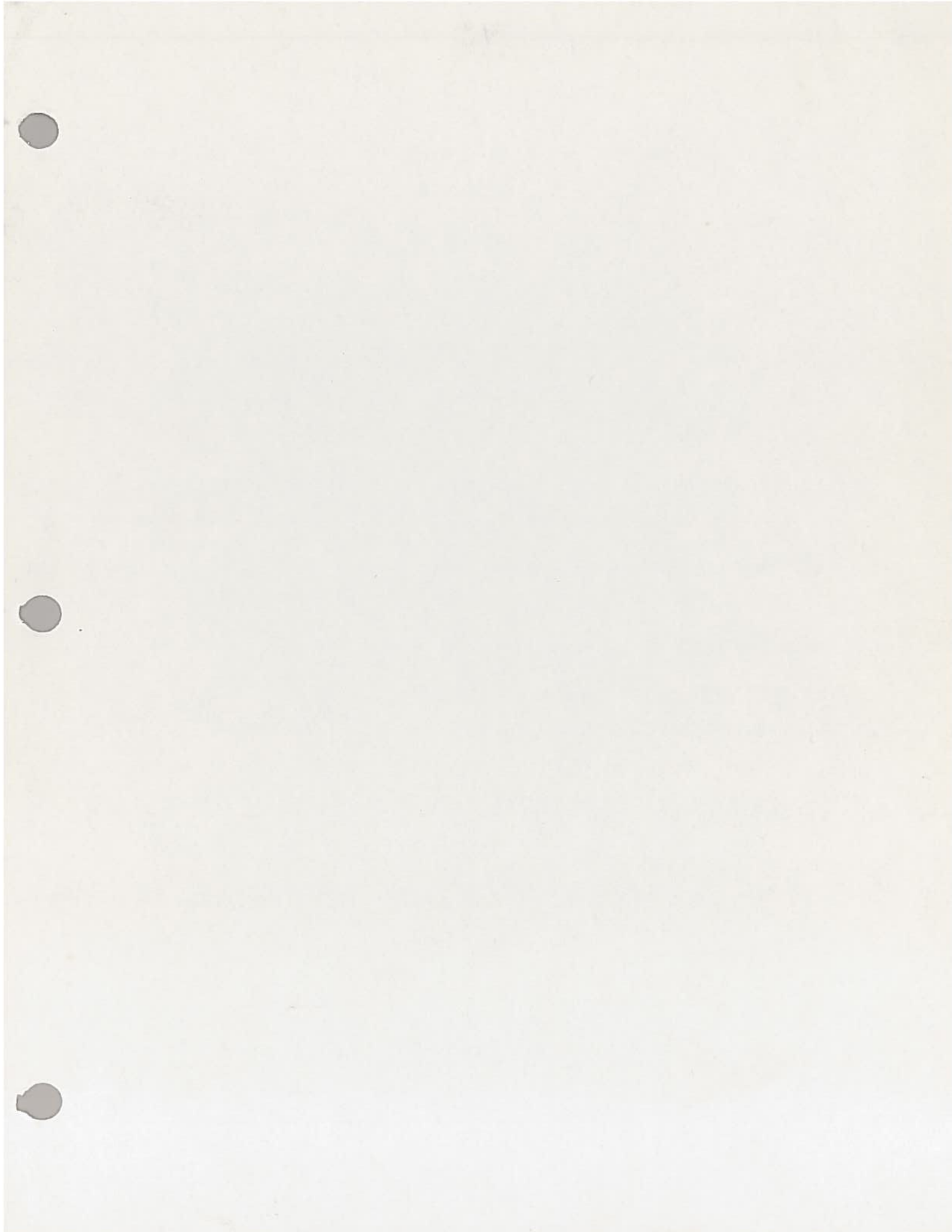
Equations A-8 and A-9 are based on the assumption that the AADT is equal in both directions. As discussed in the text, this is generally true within a one-day period, but frequently it does not hold for periods of peak traffic in urban areas where a 70/30 division is more typical. Therefore, the effect on truck spacings of this unequal distribution of peak traffic was investigated for the four-lane urban highways that were considered earlier. Specifically, it was assumed that 70% of car traffic between 6:30 and 8:30 AM is in the direction under consideration. It was also assumed that 30% of car traffic between 3:30 and 5:30 PM is in this direction. These time periods correspond to the peak hours in Figure 1. The total volume of car traffic in these four hours was not changed and the total volume in the two lanes for the day was assumed to be one-half of the AADT. The distribution of car traffic in the remaining time and the distribution of truck traffic throughout the day were not changed. For these conditions, the percentages of truck spacings less than given amounts differed from those in Figure 8 by less than 1%.

**PLEASE NOTE:**

Sections 2, 3 and 4 of this Chapter are in preparation and will be sent to you when they are completed.









## SECTION II

# Fatigue Stresses

## CONTENTS

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<b>1. INTRODUCTION</b>	<b>6.24</b>		
<b>2. OBSERVED STRESS SPECTRA</b>	<b>6.24</b>		
Single Truck Passage	6.24	b) Several Moving Loads of Constant Magnitude and Spacing	6.29
Multiple Truck Passages	6.24	c) Several Moving Spring Loads	6.29
Probability-Density Curves	6.25	d) Several Moving Spring Loads with Initial Oscillations	6.30
<b>3. CALCULATING STRESS SPECTRA     FROM LOADINGS</b>	<b>6.25</b>	e) Measured Impact Factors	6.30
Spacing and Weight Distribution of Wheel Loads for Fatigue Design Trucks	6.26	f) Design Approaches	6.31
Lateral Distribution Factor	6.26	Number of Stress Cycles Per Truck Passage	6.31
a) General Behavior	6.26	a) Simple-Span Girders	6.31
b) Number of Beams	6.26	b) Continuous-Span Girders	6.32
c) Position of Load	6.26	c) Cantilever (Suspended-Span) Girders	6.32
d) Position of Beam	6.26	d) Trusses	6.32
e) Relative Stiffness	6.27	e) Transverse Members	6.32
f) Other Parameters	6.27	Effect of Closely Spaced Trucks	6.33
g) Lateral Distribution Plots	6.27	<b>4. SUMMARY OF INFORMATION FOR         CALCULATING STRESS SPECTRA</b>	6.34
h) Lateral Distribution Chart	6.28	<b>5. CONCLUSIONS</b>	6.34
i) Measured Lateral Distribution-Factors	6.28	<b>6. REFERENCES</b>	6.34
Impact Factor	6.28	<b>7. TABLES</b>	6.36
a) Single Moving Load of Constant Magnitude	6.29	<b>8. FIGURES</b>	6.41

## 1. INTRODUCTION

This section describes stress spectra that were observed in existing bridges under real traffic conditions. This is followed by specific information on five factors that are needed to calculate stress spectra from loadings given in Section I: 1) the distribution of wheel loads for the fatigue design truck, 2) lateral-distribution factors for fatigue, 3) impact factors for fatigue,

4) the number of stress cycles per truck passage, and 5) the effects of closely spaced trucks.

As noted in the overall introduction to the Chapter, application of these data in a comprehensive fatigue-design procedure will be a part of the discussion in Section IV.

## 2. OBSERVED STRESS SPECTRA

### SINGLE TRUCK PASSAGE

A truck crossing a bridge produces one or more major stress cycles consistent with the predictions of influence lines, but with many smaller cycles superimposed.<sup>1,2</sup> For example, Figure 1 shows typical stress traces at mid-span for the passage of a single truck across different bridges. In every case these stress traces must be superimposed on the dead-load stress to determine the total stress. The smaller cycles superimposed on the major cycle (Fig. 1) result from vibrations.

The first two traces<sup>3,4</sup> are for stress in the main longitudinal members of simple-span bridges. The two peaks in the second trace are caused by independent effects of the two main truck axles crossing a relatively short span. The third and fourth traces<sup>5</sup> are for stress in the main longitudinal members of a three-span continuous bridge. The third trace is for the center span; the truck causes a large stress cycle as it crosses this span, and small cycles of the opposite sign as it crosses the two end spans. The fourth trace is for an end span; the truck causes a large stress cycle as it crosses this span, a smaller cycle of the opposite sign as it crosses the next span, and very little stress as it crosses the last span. The next two traces<sup>6,7</sup> are for stress in the main longitudinal members of cantilever (suspended span) bridges: the fifth trace is for a suspended span and the sixth trace is for an anchor span of a different bridge. Dynamic characteristics of this bridge type often result in large vibration stresses; this is particularly true immediately after the truck leaves the bridge.<sup>8</sup>

### MULTIPLE TRUCK PASSAGES

#### Histograms

The major stress parameter that affects fatigue behavior is the stress range, i.e., the difference between the peak and valley of a stress trace as shown in Figure 1. Frequency of occurrence of stress ranges with different magnitudes can be defined by histograms such as those in Figure 2. The height of each bar represents the percentage of stress ranges within a stress range interval defined by the bar width. Figure 2 illustrates three types of histograms obtained in field studies of bridges under normal traffic: unimodal<sup>6,7</sup> (one peak), bimodal<sup>5</sup> (two peaks), and continuously descending.<sup>9,10</sup>

Differences in histogram types are a result of the variety in data measuring and processing procedures. Many histograms merely include the major stress range for each vehicle passage and do not include vibrations or other wiggles in the curves. Usually, stress cycles are only counted for trucks since stresses created by passenger cars are very low. Defining what constitutes a *truck* will affect the histogram shape. If pickup trucks, vans, and other lightweight trucks are included in the term *truck*, the percentage of small stress ranges increases. For some histograms, a lower cut-off point is used, in which case, only stress ranges above the cut-off point are counted. When a cut-off point is not used, the histograms are usually unimodal or bimodal.<sup>9</sup> Most bimodal plots have less prominent second peaks than shown in Figure 2, and can be approximately represented by unimodal theoretical



curves. Frequently, if a cut-off point is used, the curves are continuously descending.<sup>9</sup> Where vibration stresses are included, they dominate the histogram and cause a continuously descending shape in which the major stress cycles represent only a very small percentage of the total.

Maximum stress ranges (excluding local stress raisers) as recorded in various studies are summarized in Table I. From each histogram in a study, a maximum stress range was obtained and the average and maximum values for each study are given in the table. Individual maximum values ranged from 0.8 to 10.5 ksi and averaged 4.3 ksi; only 8 percent exceeded 7 ksi.

As will be discussed in Section III, a variable-amplitude stress spectrum can be represented by an effective constant-amplitude stress range that causes the same fatigue damage. This type of effective stress range was calculated for each histogram, and both average and maximum values for each study are included in Table I. The individual values ranged from 0.5 to 4.9 ksi and averaged 1.8 ksi.

### Probability-Density Curves

The stress-range histogram for given data depends on the stress-range interval, or bar width, selected. However, stress-range histograms with different bar widths can be compared by converting them to probability-density curves (Fig. 3). To do this the height of each bar is divided by the width of that bar and plotted as the ordinate. The area under such a curve between any two values of the stress range corresponds to the percentage of occurrences within that interval.

A study of 51 sets of field measurements, taken from six different sources, showed that the frequency of occurrence of major stress cycles in typical bridges can be approximated by a two-parameter Rayleigh probability-density function.<sup>2,11</sup> While other probability-density functions have been proposed,<sup>11-15</sup> most of these are of greater complexity and involve more than two parameters.

The Rayleigh function, expressed as

$$p' = 1.011x'e^{-(1/2)(x')^2} \quad (1)$$

defines a family of probability-density curves of different shapes where

$e$  = the Napierian base (2.7183),  
 $p'$  = the non-dimensional probability density,  
 and  
 $x'$  = the non-dimensional stress range.

The non-dimensional parameters are defined by

$$x' = \frac{S_r - S_{rmin}}{S_{rd}} \text{ and } p' = pS_{rd} \quad (2)$$

where

$p$  = the probability density (units of 1/ksi),  
 $S_r$  = stress range (units of ksi),  
 $S_{rmin}$  = the lowest stress range in the spectrum,  
 and  
 $S_{rd}$  = a parameter equal to the modal stress range,  $S_{rm}$ , minus  $S_{rmin}$ .

Equation (1) is plotted in terms of the non-dimensional parameters  $p'$  and  $x'$  in Figure 3a, and in terms of the dimensional parameters  $p$  and  $S_r$  in Figure 3b.

As illustrated by Figure 3b, a particular curve from the family is defined by two parameters: 1) the modal stress range,  $S_{rm}$ , which corresponds to the peak of the curve, and 2) the parameter  $S_{rd}$ , which is a measure of the width of the curve, or the dispersion of the data. By changing  $S_{rm}$  the curve can be shifted sideways; and by changing  $S_{rd}$ , its width can be changed. Thus, a curve can be fit very closely to a great variety of actual frequency-of-occurrence data.

The variety of shapes is illustrated in Figure 3c, which shows four curves with the same value of  $S_{rm}$  but different values of  $S_{rd}/S_{rm}$ . Since the Rayleigh curves have a positive skew, the values of the median, mean, and root-mean cube (RMC) of the spectrum are to the right of the modal value by amounts shown in Figure 3b. The RMC is the cube root of the mean of the cubes of the individual values of  $x'$  or  $S_r$ .

## 3. CALCULATING STRESS SPECTRA FROM LOADINGS

As mentioned earlier, a single truck crossing a bridge causes one or more stress cycles. Stress cycles of different magnitudes, caused by weight variations of trucks in normal traffic, constitute the stress spectrum of a bridge. As discussed in Section I, a given composition of truck traffic can be represented by a *fatigue design truck* of "average" weight. Thus, the stress cycle caused by the passage of this truck represents the stress spectrum for that composition of traffic. Stress range for this cycle can be calculated by first placing the truck in the position that produces the maximum tensile stress at the location under consideration, and then

in the position that produces the maximum compressive stress. The algebraic difference is the stress range.

Several items of information are needed to calculate the stress range.<sup>16</sup> For calculating the bending moment, both the truck wheel spacing and the weight distribution to these wheels are needed. Factors for lateral distribution of the loads to main longitudinal beams or girders are also required for calculating the moment in each of those members. Impact factors must be applied. Information on the number of stress cycles per truck passage is needed for use in conjunction with the calculated effective stress range. The effect of

closely spaced trucks on stress spectra must be checked. All these factors are discussed in greater detail below.

## SPACING AND WEIGHT DISTRIBUTION OF WHEEL LOADS FOR FATIGUE DESIGN TRUCKS

Section I noted that 92 to 98 percent of fatigue damage in a highway bridge is caused by 4- and 5-axle trucks. It is reasonable, therefore, to base the spacing of axles for the fatigue design truck on the spacing for 4- and 5-axle trucks given in Table VII of Section I. On examination, these data show that the present (1981) AASHTO HS truck<sup>17</sup> satisfactorily represents the 4- and 5-axle trucks if the variable longitudinal spacing between the heavy axles is set at its upper limit of 30 ft. In this design truck, single axles replace sets of two closely spaced tandem axles in the actual 4- and 5-axle trucks (Fig. 4). The percentage of gross weight distributed to each axle in the fatigue design truck is the same as that in the HS truck. As Section I shows, 50 kips is a reasonable gross weight for the fatigue design truck. The transverse spacing between sets of wheels on an axle is 6 ft.

## LATERAL-DISTRIBUTION FACTOR

For calculating stresses caused in the longitudinal bridge members by a truck's passage across a bridge, it is necessary to know the fraction of the total truck moment carried by each member. This fraction of the total truck moment will be referred to here as the *lateral-distribution factor*.<sup>20</sup> There are several extensive analytical studies of lateral distribution in steel bridges<sup>18-22</sup> but they concentrate on developing factors for the worst possible static loading conditions—usually all lanes loaded simultaneously. Similarly, lateral-distribution factors specified by AASHTO<sup>17</sup> are also based on the worst possible static loading conditions, and are not generally appropriate for calculating the distribution for a single truck.

This prompted an extensive analytical study to develop simple lateral-distribution factors for a single truck. In this investigation, about 500 combinations of bridge configuration and load position were analyzed by finite-element techniques; these techniques were verified by comparing the results for one particular bridge with past experimental findings for that bridge.<sup>23</sup> The results of the study, which will be published in full in a follow-up paper, are summarized below.

### General Behavior

The slab is usually the main factor in the lateral distribution of load to the beams in a composite or non-composite steel bridge. Diaphragms have a minor effect<sup>18</sup> primarily because they are not closely spaced and are usually much more flexible than the longitudinal beams or girders. Full-depth cross-bracing is usually stiffer and may have a somewhat greater effect on

lateral distribution.<sup>22</sup> For simplicity, however, this contribution is conservatively neglected in the following development of lateral-distribution fatigue factors. (No investigation was made of the effects on lateral distribution of curvature and closer bracing spacing in curved-girder bridges.)

If the slab's longitudinal load distribution is neglected, as assumed by AASHTO,<sup>17</sup> the slab can be thought of as a plank spanning across the beams (Fig. 5). If the beams have equal stiffness and the plank is infinitely rigid, reaction between plank and each supporting beam is proportional to the beam's deflection. The sum of the reactions equals the applied force  $F$ . The reaction exerted by each beam and the corresponding lateral-distribution factors can be calculated from statics. The solid line in Figure 6 shows lateral-distribution factors calculated in this way for a six-beam simple-span bridge when the load is located over an exterior beam. If the plank is not infinitely rigid the beam deflections, and the lateral-distribution factors, differ from those for the rigid plank by amounts that depend on the relative stiffnesses of the beams and plank. A typical case is represented by the dashed line in Figure 6.

This illustration suggests that the lateral distribution to a beam in a bridge depends on four main factors: 1) the total number of beams, 2) the position of the load, 3) the lateral position of the beam under consideration with respect to the bridge centerline, and 4) the relative stiffness of the beams and slab. Each of these parameters is discussed below.

### Number of Beams

If slabs were infinitely stiff, the distribution factors would depend on the total number of beams since the load would be distributed to all of these beams. However, for actual bridges, the stiffness of both slab and beams is such that significant portions of the load are distributed to only three or four beams. Consequently, the total number of beams is *not* an important parameter in actual bridges.

### Position of the Load

For the infinitely stiff plank as postulated, the position of the load with respect to the centerline of the bridge would have a large effect, since the bridge cross-section would act as a rigid unit. The beams would share the load equally when the load is centered, but the outer beams would carry most of the load when the load is far from the center. For actual bridges, the position of the load is still important, but the load's position with respect to the beam under consideration is a more useful parameter than its position with respect to the bridge centerline. This is true because, in actual bridges, load is distributed primarily to only three or four beams.

### Position of the Beam

For the infinitely stiff plank the position of the beam, with respect to the bridge centerline, is very important because the distribution varies linearly across the

\* Lateral-distribution factors given by AASHTO<sup>17</sup> are fractions of a wheel load (both front and back), i.e., fractions of half the truck or lane load.

bridge. In an actual bridge, the position of the beam is also important, but again the relative position of the beam and the load is the most significant parameter. In terms of this parameter, all interior beams behave similarly, but the behavior of an exterior beam is quite different. Therefore, in subsequent discussions the distinction made is between interior and exterior beams.

### Relative Stiffness

The beams act like springs in supporting the plank. The equivalent spring constant is a function of the moment of inertia and span of the beams. The force in each spring, or beam, depends on the stiffness of the plank relative to this spring constant. Plank stiffness is defined by its moment of inertia and some characteristic length such as the beam spacing or plank length.

In actual bridges the stiffnesses of both the beams and the slab have been observed to vary within limits. The log-log plot in Figure 7 shows the variation of moment of inertia,  $I$ , with span,  $L$ , for a wide variety of typical composite and noncomposite simple and continuous beams in steel bridges.<sup>24-25,26</sup> For a given  $L$ ,  $I$  can be seen to vary by a factor of 10. The line drawn as the upper limit on beam stiffness is defined by the equation:

$$I = 3.3L^{2.3} \quad (3)$$

Slab stiffness varies within narrower limits. For example, where beams are spaced 8 ft apart, slab thickness is generally between 7 and 8 inches (7.5 inches average). Thus, the slab  $I$  varies only about 20 percent from the typical value. For other beam spacings, the effect of a change in the span of the slab tends to be offset by a change in its thickness so that the stiffness,  $(I/L)$  is not greatly changed. Thus, variation in beam stiffness has much more significance than variation in slab stiffness.

In the principal part of the finite-element study, therefore, a single typical beam spacing and slab thickness was used with several different bridge spans. Both upper and lower limits of  $I$  from Figure 7 were used with each span to include the variation in relative stiffness for actual bridges. Lateral-distribution factors, corresponding to the higher values of  $I$ , varied only slightly with span; the lateral-distribution factors corresponding to the lower  $I$  were somewhat lower, by amounts influenced by the span.

### Other Parameters

Other parameters were also investigated and all were found to have secondary importance: 1) diaphragms, 2) unequal beam stiffnesses, 3) variation of the beam stiffness along the beam, 4) longitudinal position of the loads, 5) beam spacing, and 6) type of bridge (simple or continuous).

### Lateral-Distribution Plots

The main effects discussed above are illustrated in Figures 8 and 9 for interior and exterior beams, respectively. These figures give the fraction of the total truck

moment that is carried by the beam under consideration as a function of the truck-position ratio.\*

Each curve on one of the figures represents a different case, but, for all cases the beam's moment of inertia corresponds to the upper limit defined in Figure 7. The upper-limit moment of inertia corresponds to the lowest relative stiffness of the slab compared with the beams, and highest lateral-distribution factors. The cases include: 1) 3-beam and 6-beam simple-span bridges with spans of 32, 64, and 160 ft, all with a beam spacing of 8 ft; 2) 6-beam simple-span bridges with a span of 64 ft and beam spacings of 6 and 10 ft, 3) 3-span continuous bridges with spans of 64 ft and 160 ft loaded near midspan, the distribution factors being determined both at midspan and at an interior support. Results for 8- and 10-beam bridges were almost identical with those for the 6-beam bridges.

The curves for all the different cases fall within a relatively narrow band, and the upper limit of this band provides conservative values of lateral-distribution factors for typical bridge configurations. As Figure 8 for interior beams illustrates, the factor peaks at about 0.5 with the truck centroid directly over the beam. There is a rapid decrease in the factor as the truck position is moved to either side of the beam.

Actually, however, on a bridge, each interior beam is within a distance equal to half the beam spacing from a truck in one of the traffic lanes. Thus, the truck-position ratio never exceeds 0.5 and a corresponding reduction in the distribution factor never exceeds 15% (Fig. 8). Therefore, it is convenient and conservative to use the peak factor of 0.5 for interior beams.

For the exterior beam, the factor is 0.9 when the truck centroid is directly over the beam but decreases rapidly as the truck is moved inward toward the bridge centerline (Fig. 9). When the position ratio exceeds 0.5 the distribution drops below 0.5 and has little practical significance since the factor for interior beams then controls.\*\*

Curves for interior and exterior beams have been found to apply to both positive and negative bending, and to both simple-span and continuous bridges. One exception is the negative bending at an interior support when a truck straddles the support, i.e., when wheels are positioned on both sides of the support. In this case, the peak factor for interior beams is about 0.6, but this is of little practical importance because a truck placed in this position does not produce much bending moment.

While lateral-distribution factors for beams having moments of inertia corresponding to the lower limit in Figure 7 follow the same trends as the upper-limit cases discussed above, their magnitudes are considerably lower. Also, the factors for the lower-limit stiffnesses are more sensitive to the span length. For beam

\*Truck-position ratio is defined as the lateral distance between the truck centroid and the beam, divided by the beam spacing.

\*\*The present AASHTO specifications<sup>17</sup> prohibit exterior beams from being smaller than the interior beams unless bridge widening is not anticipated.

stiffnesses between the limiting values, the distribution factors vary approximately exponentially with the beam stiffness. For interior beams, the variation of the peak lateral-distribution factor with the beam moment of inertia is shown in Figure 10 for several different spans. These curves were developed empirically from the finite-element study. Equations defining the curves are shown on the figure. In a similar way, the lateral-distribution factor for exterior beams varies with beam moment of inertia.

### Lateral-Distribution Chart

Applicable to bridges with more than two longitudinal beams, a convenient, conservative chart giving lateral-distribution factors for fatigue was developed from the preceding results and is given in Figure 11. The lateral-distribution factors,  $F_e$  for exterior beams, and  $F_i$  for any interior beam are shown as functions of the lane position ratio,  $P$ . (The lateral-distribution factor is the fraction of the total truck moment carried by the beam under consideration. The lane position ratio is the distance from the *exterior* beam to the centerline of the nearest outer lane, divided by the beam spacing. If the beam spacing is not uniform the spacing between the exterior beam and the first interior beam should be used.) If the outer-lane centerline lies, as usual, between the two exterior beams, the position ratio is positive. Otherwise, it is negative. The distance to the *lane* centerline is used as a parameter when considering fatigue because this is the most likely truck position.

The solid lines represent the upper limit of each lateral-distribution factor and can be used to make an initial fatigue check (Fig. 11). If the structural element under investigation is satisfactory for this factor, as is frequently the case, no further check is required. Otherwise, a second check can be made using a more precise but less conservative value (like the dashed line), which depends on the beam span,  $L$ , and the beam moment of inertia,  $I$ . [The span in which the truck (or its heaviest wheels) is positioned should be used in determining both  $L$  and  $I$ . For simple-span, continuous, and cantilever (suspended span) bridges, the span is the distance between adjacent piers. The moment of inertia, either composite or noncomposite, in the central positive-moment region should be used. If the moments of inertia of all the beams in the bridge cross-section are not the same, the greatest one should be used.]

The horizontal solid line represents the peak value of 0.5 from Figure 8. When  $P$  exceeds 0.5 the horizontal line applies to both interior and exterior beams, but below 0.5 it applies only to interior beams. [For most actual bridges  $P$  is greater than 0.5.] When  $P$  is less than 0.5, exterior beams are governed by the sloping solid line, which approximates the upper limit of the curves in Figure 9.

The level of the horizontal dashed line depends on the beam moment of inertia and span as defined in Figure 10. Again, above a limiting value of  $P$ , the horizontal

dashed line applies to both interior and exterior beams, but below this value the separate dashed lines apply. The factors from this chart are generally much less than the AASHTO<sup>17</sup> factors which are  $S/5.5$  and  $S/7$  (where  $S$  is the beam spacing in feet), based on *half* the truck moment.

### Measured Lateral-Distribution Factors

Table II gives typical lateral-distribution factors determined from field measurements on actual bridges. Determined only when one truck was on a bridge, these factors are, therefore, appropriate for fatigue. They were established with the assumption that a fraction of the total moment in each beam was proportional either to the measured deflection or to the measured flange strain for that beam. The factors are considered approximate because of the many uncertainties that exist when making this determination. These include the degree of composite action, the effect of skew, and the effects resulting from the assumption that each beam's moment is proportional to its deflection.

Data were obtained from eight different sources and cover simple-span, continuous, and cantilever (suspended span) bridges. [The key for the bridge and location numbers is given in a footnote to the table.] The factors were determined either at midspan or at an interior support. In Table II, the truck position ratio is the lateral distance from the *exterior* beam to the truck centroid divided by the beam spacing. It is similar to the lane position ratio, but defines the actual truck position during the test. The listed *Design Lateral-Distribution Factors* were determined from Figure 11 using the truck position ratios from the table as  $P$ .

The measured distribution factors ranged from 0.20 to 0.52; these are fractions of the full truck moment. Most were below 0.5 because most beam moments of inertia were well below the upper limit shown in Figure 7, and because most truck position ratios were above 0.5. The listed *Design Lateral-Distribution Factors* were generally in close agreement with the measured values.

The table's last column gives the value of the factor  $.5(S/5.5)$  for each bridge under consideration. This factor represents the AASHTO wheel load distribution factor of  $S/5.5$  divided by 2 to convert the AASHTO values to fractions of the *full* truck moment and thereby permit direct comparisons with the other factors which are based on full truck moment. AASHTO values are well above both the measured values and the design values from Figure 11. This shows that the  $S/5.5$  factor is overly conservative for fatigue design.

### IMPACT FACTOR

In calculating the stress range caused by the passage of a truck across the bridge, the truck is placed in positions that can cause maximum and minimum stresses at the location being investigated. An impact factor is then applied to these static stresses to account for dynamic effects. Already studied extensively,<sup>8,27-34</sup> these dynamic effects are discussed below.

### Single Moving Load of Constant Magnitude

A single load of constant magnitude moving across a simple beam at a velocity  $v$ , causes static and dynamic moments. Figure 12 shows the resulting static, dynamic, and total moments. Specifically, the illustration indicates the variation of midspan moment—that is proportional to stress—as a function of position and time ratios. The position ratio is the distance between the left support and the load divided by the span length. The time ratio is the time the load has been on the beam divided by the natural vibration period for the beam. This natural vibration period is the reciprocal of the lowest natural frequency of the beam. The total moment is composed of static and dynamic parts; the dynamic part divided by the peak static part is the impact factor.

The dynamic part of the midspan moment is expressed in the following equation:<sup>27</sup>

$$M_d = \frac{\pi^2 WL}{48} \left[ \frac{\alpha^2}{1-\alpha^2} \sin\left(\frac{2\pi\alpha t}{T}\right) - \frac{\alpha}{1-\alpha^2} \sin\left(\frac{2\pi t}{T}\right) \right] \quad (4)$$

in which  $\alpha$  equals the speed parameter (defined later),  $W$  equals the moving load weight,  $L$  equals the span,  $T$  is the bridge's vibration period, and  $t$  equals the time the load has been on the beam.  $M_d$  has two components: 1) the first corresponds to free vibration at the lowest natural frequency of the beam and is represented by the second term in the brackets, and 2) the second is a forced response similar to the static component, but much smaller, which is represented by the first term. The free vibration term dominates in most practical cases because  $\alpha$  usually has a value of about 0.1 (as discussed below). For practical cases vibrations in higher modes are negligible.

The maximum value of  $M_d$  occurs when both terms are positive and the sines are equal to unity. The impact factor,  $I$ , is then obtained by dividing this maximum value by the static moment  $WL/4$ . Thus,

$$I = \frac{\pi^2}{12} \left[ \frac{\alpha^2}{1-\alpha^2} + \frac{\alpha}{1-\alpha^2} \right] = \frac{\pi^2}{12} \left[ \frac{\alpha}{1-\alpha} \right] \quad (5)$$

While Equations (4) and (5) apply specifically to a single beam, they also provide reasonable approximations for the most heavily loaded beam in multi-beam bridges.<sup>27,28</sup>

The speed parameter is defined as:<sup>27</sup>

$$\alpha = \frac{vT}{2L} = \frac{v}{2fL} \quad (6)$$

in which  $v$  equals the velocity of the moving load and  $f$ , the reciprocal of  $T$ , equals the lowest natural frequency of the beam. For simple-span steel bridges the following approximate relationship applies:<sup>27</sup>

$$T = 0.0024L \quad (7)$$

in which  $L$  is expressed in feet and  $T$  is expressed in

seconds. This relationship was developed from  $T$  values measured on 31 simple-span steel rolled-beam, plate-girder, and truss bridges, and should not be applied to unusual bridges. Most of the measured values were within 20 percent of the values defined by Equation (7). Combining Equations (6) and (7) gives

$$\alpha \cong 0.0012v \quad (8)$$

wherein the units of  $v$  are in feet-per-second. For a speed of 60 mph,  $\alpha = 0.106$  and from Equation (5),  $I = 0.10$ .

### Several Moving Loads of Constant Magnitude and Spacing

When several loads with fixed spacings move across a beam, the peak amplitude of the moment depends on the phase relationships. Two loads spaced at a distance  $s$  are in phase when  $s/v = nT$ , in which  $n$  equals any positive integer. If  $T$  is taken to be a function of the span as defined by Equation (7), the dynamic moment, and consequently the impact factor, are both functions of span. The impact factor is plotted in Figure 13 as a function of span for a set of loads corresponding to the fatigue design truck and also for a modified fatigue design truck in which the two main axles are each replaced by a set of two tandem axles spaced at 4 ft. This latter case is a better approximation of an actual 5-axle truck. The plots are all for a speed of 60 mph. Dynamic moments were calculated by applying Equation (4) to each of the individual loads and maintaining the proper phase relationships among these loads. Maximum amplitude for any time  $t$  was then divided by the maximum static moment for the set of loads to obtain the impact factor.

The impact factor for the single load (which has a constant value of 0.10 for any span, as discussed earlier) is also plotted in Figure 13. Curves for the other two cases undulate because of changing phase relationships. The fatigue design truck curve is below that of a single load, for spans over approximately 80 ft, but has several peaks above the single-load curve for shorter spans. However, the modified fatigue design truck only has one peak slightly above the single-load curve. Thus, single-load curves provide a reasonable approximation to the impact factor for the 4- and 5-axle trucks that cause most fatigue damage in bridges.

### Several Moving Spring Loads

The preceding discussion applies to loads that are constant in magnitude while moving across a beam or bridge. If a spring is placed between the load and the bridge, the force exerted by the spring varies as it crosses the bridge.<sup>27</sup> This variation in applied force will, in turn, affect the mid-span moment and the impact factor. The magnitude of the effect depends on three interrelated parameters: 1) the speed parameter, 2) the weight ratio (defined as the total weight of the truck divided by the bridge's total weight), and 3) the frequency ratio (defined as the lowest natural fre-

quency of the truck divided by the lowest natural frequency of the bridge).

In calculating the weight ratio of two-lane bridges, it is appropriate to include the weight for the total bridge width.<sup>27</sup> For wider bridges, it may be appropriate to include only a portion of the width, but no specific criterion has, as yet, been proposed. The minimum total weight of a 60-foot two-lane steel bridge is about 250 kips.<sup>27</sup> Dividing the 50-kip weight of the fatigue design truck by 250 kips gives a weight ratio of 0.2. Bridges with shorter spans will have a smaller weight, but only one main axle will be on these bridges when the static midspan moment peaks so that the weight ratio is not much greater. Wider bridges have a lower weight ratio. Therefore, for fatigue calculations, 0.2 to 0.3 is a reasonable upper limit for the weight ratio.

Natural frequencies of trucks range from about 3.0 to 4.5 Hz when the truck is acting on the tires alone (as happens when the springs bottom), and from about 1.7 to 2.9 Hz when the truck is acting on the tire/spring combination.<sup>27,35</sup> For steel bridges the natural frequencies range from about 2 to 20 Hz.<sup>27</sup> Thus, frequency ratios range from about 0.1 to 2.

When the speed parameter is less than 0.12 and the weight ratio is less than 0.3—the normal range of these parameters for steel bridges—the impact factors for spring loads were found to be about the same as those for constant loads, regardless of the magnitude of the frequency ratio.<sup>27</sup> For higher speed parameter values, the impact factors for spring loads are significantly higher.<sup>27</sup> The effect increases with increasing weight ratio and peaks at a frequency ratio near 0.5.<sup>27</sup> The effect of phase relationships among loads with fixed spacings is about the same for spring loads as for constant loads.

#### Several Moving Spring Loads with Initial Oscillations

In the preceding discussion it was assumed that the truck is moving smoothly without oscillations when it enters the bridge. In reality, however, most trucks are always oscillating to some extent, and those oscillations affect the vibration of the bridge. The magnitude of the effect is proportional to the amplitude of the initial oscillations and varies greatly with two phase relationships.<sup>27</sup> One is the relationship between the motion of the individual axles of the truck under initial oscillation as, for example, when one axle is moving up while another is moving down, or if they are moving together. The other relationship is between the initial truck oscillations and the bridge vibrations that would be caused by loads in motion without initial oscillations. Under actual traffic conditions, these two phase relationships occur randomly. The phase relationships corresponding to the axle spacings that were previously discussed also apply.

Rough pavements are the primary cause of initial oscillations in trucks. A relatively small bump can create a dynamic force that exceeds the static force corresponding to the vehicle weight, but this dynamic

force rapidly damps to a steady-state oscillation within a few cycles,<sup>27</sup> usually about 1.5. The amplitude for the truck's steady-state oscillations is limited by the peak value of the interleaf frictional force to a value of about 15 percent of the static force.<sup>27</sup> This value has been proposed<sup>27</sup> as a realistic representation of the initial truck oscillations in practice. It has also been proposed<sup>27</sup> that a conservative constant value at 0.15 be used to represent the effect of the initial oscillations with the most detrimental phase relationships.

Combining the dynamic effects caused by a smoothly moving truck (.10) with those caused by the initial oscillations (.15) gives an impact factor of 0.25. Because the contribution of the initial oscillations were based on the most unfavorable phase relationships, the impact factors for individual trucks in actual traffic should generally be less. Occasionally, much higher factors may be anticipated under unusual conditions such as a bump at a critical location; such conditions were not considered in the preceding discussion.

This sub-section, along with the three preceding, apply specifically to simple-span beams and bridges. However, limited studies<sup>27,28,30</sup> conclude that it is conservative to apply these criteria to continuous-span beams and bridges.

#### Measured Impact Factors

Measurements of impact factors in actual steel bridges seem to confirm the foregoing theoretical discussion. Impact factors approaching 100 percent have been reported in studies aimed at determining peak impact factors for nonfatigue design.<sup>31,33,34</sup> Generally, however, factors determined from stress traces for individual trucks in traffic, or for test trucks, are much lower. Similarly, impact factors calculated by comparing static and dynamic deflection measurements are usually much lower. Data from stress traces and deflection measurements are reviewed in the discussion that follows.

Impact factors for 37 different steel bridge spans, calculated from 197 stress traces, and derived from 10 different sources, are listed in Table III together with brief descriptions of each span. Each entry in the first column represents one bridge. If data are given for several spans of this bridge they are listed without a new entry in the first column.

Impact factors were calculated in the following way: on the stress traces, the dynamic effects appear as vibrations superimposed on the static curve (crawl curve), illustrated in Figure 12 and in the drawing accompanying Table III. Usually, these vibrations are at the natural frequency of the bridge and persist after the truck has left the bridge. Hence, dynamic effects can be identified and used to calculate the impact factor as shown in the drawing. Because several values must be scaled from small traces, and interpretation is often involved in the process, the resulting impact factors are approximate. The impact factor gives the dynamic effect on the peak stress. The dynamic effect on the stress range was also calculated as shown in the drawing and



is listed in Table III under the heading Stress-Range Impact Factor.

For fatigue considerations the average value, rather than the maximum for each span, is most important. The highest average value of the impact factor for any span was 29 percent, and the average value exceeded 20 percent in only 3 out of 37 spans. Of these, two were in the same bridge, and each consisted of only one reading. The highest average value of the stress-range impact factor was 54 percent, but the average value exceeded 31 percent in only 3 out of 37 spans. Two of these were in the same bridge that had the high impact factors mentioned above. The third was in a cantilever (suspended span) bridge—a type known to be susceptible to vibrations.<sup>8</sup> This bridge also showed very high vibrations after the truck left the bridge.

Table IV lists impact factors calculated from static and dynamic deflection measurements on 16 different steel bridge spans. Specifically, the impact factor was taken as the dynamic deflection created by a test truck divided by the static (or crawl) deflection caused by the same truck minus 1. Data for 30 of the spans are from a study on bridge vibrations that gave only the “maximum” impact factor for each span. The data for the other 6 spans include both average and maximum values for several different loading cases (position, speed, and weight of the truck); in all cases, the impact factors were only for the most heavily loaded beam. Impact factors for lightly loaded beams were often of a higher percentage, but represented much smaller actual deflections. Each entry in the second column represents one bridge.

The results are similar to those from the stress traces. The highest maximum impact factor for any of the 30 spans in the first study is 46 percent, but the next highest value is only 28 percent. For any of the 6 other spans listed in Table IV, the highest average value is 28 percent. Cantilever bridges tended to have higher impact factors than other types, and the text of the first study states that, “Eight of the nine cantilever-type bridges appeared susceptible to larger amplitudes of vibration.”<sup>8</sup>

### Design Approaches

Although impact has been studied extensively, AASHTO<sup>17,34</sup> continues to use the following empirical formula:

$$I = \frac{50}{L + 125} \text{ but not more than } 0.3 \quad (9)$$

in which  $L$  is the span in feet. This formula gives values decreasing from 0.3 as the span increases above 41.7 ft. Outside the U.S., highway bridge specifications in most countries require higher impact factor values.<sup>34,36</sup> For example, Ontario recently adopted a specification that includes impact factors ranging from 0.30 to 0.55 depending on the natural frequency of the bridge.<sup>37</sup> These factors are applied to the peak live-load stresses in both fatigue and nonfatigue design. The intention is to ac-

count for infrequently occurring extreme conditions.

However, impact factors for fatigue conditions should be based on typical or average conditions rather than extreme situations. Note that if the impact factor is only applied to the peak live-load stress, it must be large enough to account for the total effect on the stress range. For simple-span bridges the theoretical increase in stress range is twice the vibration amplitudes since they decrease the minimum stress while increasing the maximum stress. For continuous-span bridges, the peak tensile and compressive stresses occur when the truck is at different positions.

If each peak is increased by an appropriate amount, the total increase in stress range is correctly accounted for. The preceding theoretical and experimental information suggests that the impact factors given by the present AASHTO formula [Equation (9)] are fairly typical and are, therefore, generally appropriate for fatigue calculations for simple-span and continuous bridges.

This conclusion does not apply, however, to cantilever (suspended-span) girder bridges. For such structures, large vibration stresses can occur—particularly after the truck has left the bridge.<sup>8</sup> Consequently, in the discussion following (Number of Stress Cycles Per Truck Passage) it is suggested that such bridges be designed for several cycles per truck passage. Data in Tables III and IV suggest that the average impact factors for most, *but not all*, cantilever bridges are within AASHTO requirements.<sup>17</sup>

### NUMBER OF STRESS CYCLES PER TRUCK PASSAGE

#### Simple-Span Girders

The theoretical variation of mid-span moment as a fatigue design truck (Fig. 4) moves slowly across a simple-span girder bridge is illustrated in Figure 14. Actual variation of stress in a bridge subjected to traffic loading is similar, although the curve is more rounded and may have small vibration stresses superimposed as a result of dynamic effects. The effects of these vibration stresses are discussed later. For spans exceeding 60 ft, a single cycle occurs. For spans under 30 ft—the spacing of the main axles—two major stress cycles occur. A secondary cycle corresponding to the front axle is superimposed on one of these major cycles. Where the span is between 30 and 60 ft, a complex cycle with two major peaks occurs. The depth of the valley between the two peaks increases as the span decreases.

A complex cycle with one or more valleys causes more fatigue damage than a single simple cycle of the same amplitude. Thus, a complex cycle is equivalent to more than one simple cycle. A detailed explanation of how to calculate the equivalent number of cycles for any complex cycle will be presented in Section III, FATIGUE BEHAVIOR. Briefly, each secondary cycle superimposed on the main cycle is treated as a separate cycle that is equivalent to less than one main cycle. Specifically, it is equivalent to a fraction equal to the cube of the ratio of the ranges for the main and secon-

dary cycles. The equivalent number of cycles corresponding to a complex cycle is equal to 1.0 plus the fractions for all secondary cycles. Conservatively, this method neglects the effects of a fatigue limit.

This procedure was used to calculate the equivalent cycles caused by the fatigue design truck passing across simple-span bridges of different lengths. The results are plotted as the upper-most curve in Figure 15. (For improved clarity, different scales are used for spans above and below 40 ft.) Specifically, the equivalent cycles of moment are plotted at midspan, but plots at other locations follow similar trends. In simple spans exceeding 60 ft, equivalent cycles equal 1.0. As the span decreases below 60 ft, the equivalent cycles increase. The equivalent cycles equal 1.1 at a span of 40 ft, and then increase rapidly to 2.0 as the span decreases below 40 ft. Therefore, it is convenient to use a value of 2 for simple spans less than 40 ft and 1 for longer spans.

Vibration stresses superimposed on the static or crawl curve, discussed above, have two effects: 1) they increase both the maximum stress and the stress range of the complex stress cycle, and 2) they add wiggles to this cycle. The first effect is covered by the impact factor. To show that the second effect is negligible, it will be evaluated by considering the stress traces used to study impact factors. The main parameter controlling this effect is the vibration ratio—defined as the stress range of the superimposed vibration divided by the stress range of the complex (main) cycle. Table III lists the vibration ratios determined from available stress traces. For simple-span bridges the average value is 12 percent. Each vibration cycle of this magnitude is equivalent to only 0.0017 main stress cycles. Only one out of 39 traces had a vibration ratio exceeding 20 percent—a vibration cycle that is equivalent to 0.008 main stress cycles. Thus, this second effect of vibration stress can normally be neglected.

### Continuous-Span Girders

The variation of moment as a fatigue design truck moves slowly across a continuous girder bridge is more complicated, and often results in a complex cycle with several peaks of different heights. Determination of the complex cycle shapes for continuous bridges of two or three equal spans were made with the use of influence lines.<sup>38</sup> In each case, the shape was established for several different span lengths. The method previously described was then used to calculate the corresponding equivalent cycles, and the results are given in Figure 15. Curves for continuous bridges with a greater number of spans or with unequal spans are similar to those in Figure 15.

Curves for the equivalent cycles of midspan moment are similar to those for simple-span bridges, and can be conveniently represented by two equivalent cycles for spans below 40 ft and one cycle for longer spans. The curves for the equivalent cycles of moment at an interior support can also be conveniently represented by two equivalent cycles for spans below 40 ft. At spans

above 60 ft, however, the equivalent-cycle curves steadily increase toward a theoretical maximum value at an infinite span. This maximum value is 2.0 for two equal spans and lesser values for unequal spans and for more than two spans. Since the maximum value for two equal spans does not exceed 1.5 until the span exceeds about 270 ft, 1.5 equivalent cycles could be conveniently used above 40 ft. This value applies to moments at points on the girder within about 0.1 of the span on both sides of any interior support; elsewhere a value of 1.0 equivalent cycle applies.

The effect of the vibration stresses can again be shown to be negligible by considering the vibration ratios in Table III. For continuous-span bridges the average value is 14 percent, which corresponds to only 0.0027 equivalent cycles. Only five out of 92 traces had a vibration ratio exceeding 23 percent, which corresponds to 0.0122 equivalent cycles.

### Cantilever (Suspended Span) Girders

The vibration characteristics of cantilever (suspended span) girders may cause several major stress cycles for each passage of a truck (Fig. 1). Experimental measurements summarized in Table II show that the magnitude and number of such vibration cycles vary considerably, even for a particular bridge. It appears, however, that it would be conservative to use 10 equivalent cycles for such bridges, unless contrary information is available for the particular bridge under consideration.

### Trusses

The variation of force in a particular truss member as the fatigue design truck passes across the bridge can be determined from the influence line for that member. Corresponding equivalent cycles can be calculated by the method discussed above. For simple-span trusses, the force in the chord members varies from zero to a maximum and back to zero, but the forces in some web members reverse from tension to compression (or the opposite). In both cases, however, the member is only subjected to one cycle unless the span is very short.

### Transverse Members

Truck loads are usually imposed on transverse members, such as floor beams, through longitudinal members that frame into them. These members may be either simply supported or continuous over the transverse member. The variation of moment in the transverse member as the fatigue design truck moves across the bridge can be determined from the influence line for the reaction of the longitudinal members on the transverse member. This was done for simple and continuous longitudinal members of various spans. The corresponding equivalent cycles were then calculated.

When the longitudinal span exceeds 30 ft, the equivalent cycles for transverse members are very close to 1.0 for both simple and continuous longitudinal members. (This limiting span is half the limiting span for simple-span girders because the transverse member is af-



ected by wheel loads on the longitudinal members on both sides.) The equivalent cycles increase above 1.0 as the span decreases below 30 ft. However, at a span of 20 ft, the equivalent cycles are only 1.11 for two simple spans, 1.02 for two continuous spans, and 1.03 for four continuous spans. The equivalent cycles increase rapidly to 2.0 at shorter spans. Therefore, the equivalent cycles for a transverse member can be conveniently taken as 2.0 when the longitudinal members have a span less than 20 ft and as 1.0 when they have a longer span.

### EFFECT OF CLOSELY SPACED TRUCKS

When two closely spaced trucks cross a bridge they cause a complex stress cycle similar to those discussed above. The corresponding equivalent cycles are plotted in Figure 16 as a function of the truck spacing for several different simple spans. Two fatigue design trucks were used to generate the data, but the second truck is in the same lane as the first when the front-to-front spacing exceeds 50 ft (6 ft front to rear) and the second truck is in the adjacent lane for smaller spacings. It is assumed that, when the second truck is in the adjacent lane, it distributes only 0.3 of its weight to the lane in which the first truck travels. (In effect, the second truck weighs 0.3 as much as the first truck in this case.) Actual lateral distribution varies with many factors, but 0.3 is a reasonable value for this purpose.

The typical variation of equivalent cycles shown in Figure 16 is explained by considering the 180-ft span. When the spacing exceeds 224 ft the clear distance between trucks exceeds the span, and the two trucks behave independently. Thus, each causes one cycle. As the spacing is reduced below 224 ft, the two trucks produce a single cycle with two peaks, each with an amplitude equal to that for an individual truck. The valley between the two peaks gets progressively smaller as the spacing decreases, and reaches a minimum value equal to about 20 percent of the peak amplitude at a spacing near 120 ft. As a result, the equivalent cycles per truck passage decrease from 1 to 0.504. (It is clear that the limiting value would be 0.5; this could occur if the two trucks caused one cycle with the same amplitude as that for a single truck and with no valley.)

As spacing decreases below 120 ft, the equivalent cycles increase rapidly to a value of about 2.6 at a spacing of 50 ft. This rapid increase occurs because the peak amplitude of the cycle for the two trucks is greater than the amplitude for an individual truck. The theoretical upper limit for the equivalent cycles — not reached in this example — is 4. At spacings below 50 ft the second truck is assumed to move into the adjacent lane and weighs, in effect, only 0.3 times as much. When the spacing is zero, the equivalent cycles are equal to 1.1 for all spans since the two trucks produce a single cycle instead of two smaller individual cycles. The peak in the curve for a 30-ft span occurring at a spacing of 30 ft results because the center axle of the rear truck is directly beside the rear axle of the front

truck. Thus, for any given span length, certain spacings are beneficial while others are detrimental.

The effect of closely spaced trucks on the total life of a simple-span bridge was calculated by combining data from Figure 16 with the frequency of truck spacings of different magnitudes as shown in Figures 7 and 8 of Section I. In these calculations, there were two major assumptions. *First*, it was assumed that all trucks were of the same size and corresponded to the fatigue design truck. Therefore, the equivalent cycles for any two closely spaced trucks could be obtained from Figure 16. Actually, of course, trucks vary in size and, normally, any two closely spaced trucks would be of different size. However, to include this variation in the calculations would have made them quite complicated. Furthermore, this is a reasonable assumption because the weight of the fatigue design truck was selected to account for the variety of truck weights in actual traffic. *Second*, it was assumed that each set of two closely spaced trucks acts independently; i.e., the set under consideration is widely separated from all other sets. Usually, but not always, this would be true because only a small fraction of the spacings is small enough to affect most bridges.

When the effect of closely spaced trucks on the total life for a given span was calculated, the spacing range that affects this span was divided into 20-ft intervals. The fraction of spacings within each interval was determined from Figures 7 and 8 of Section I. The fraction of trucks involved is twice the fraction of spacings. Equivalent cycles for each of these trucks was assumed to be the value corresponding to the midpoint of the spacing interval and was determined from Figure 16. The weighted average of the equivalent cycles for all intervals was then calculated to represent the total traffic. Trucks spaced far enough apart to act independently — comprising most of the truck traffic — were included in this calculation of the weighted average.

Calculations were made for two cases that have about the same distribution of truck spacings: 1) four-lane rural highways with an annual average daily traffic (AADT) of 40,000 and 2) four-lane urban highways with an AADT of 60,000. Both cases represent very heavy traffic — roughly 4 and 3 times the observed mean values for rural and urban highways, respectively. For this traffic the number of trucks close enough to interact varies from 9 to 31 percent as the span varied from 30 to 240 ft. For lighter traffic, corresponding to the observed mean values of AADT for the two types of highways, these percentages would be only one-fourth and one-third as great since the percentages are approximately proportional to the AADT.

The calculations showed that by including the effects of closely spaced trucks there were slight increases in fatigue life as compared with the life for the same truck traffic but with all trucks acting independently. For the heavy traffic previously mentioned, life increases ranged from 0.1 to 1.4 percent for simple spans ranging from 30 to 240 ft. For lower traffic volumes these in-

creases would be correspondingly less. The effect is very small because some of the close spacings are beneficial and others are detrimental (Fig. 16). While

similar calculations were not made for continuous-span bridges, they would be expected also to show only a small effect of closely spaced trucks.

## 4. SUMMARY OF INFORMATION FOR CALCULATING STRESS SPECTRA

The data needed for calculating stress spectra from loadings, given in the preceding discussion, are sum-

marized in Table V.

## 5. CONCLUSIONS

Stress spectra caused by traffic loading have been observed in many bridges. A single truck moving across a bridge produces one or more major stress cycles with many smaller cycles superimposed. Occurrence frequency of major stress cycles of different magnitudes can be defined by a histogram in which each bar height represents the percentage of cycles within a stress interval that is represented by the bar width.

Three different types of histograms have been observed: 1) unimodal (one peak), 2) bimodal (two peaks), and 3) continuously descending. Differences in data acquisition and processing procedures produce different histogram types. The maximum stress ranges (maximum stress minus minimum stress in a cycle) observed in various studies ranged from 0.8 to 10.5 ksi and averaged 4.3 ksi; only 8 percent exceeded 7 ksi. These observed stress ranges excluded the effects of local stress raisers in accord with present fatigue design

methods.

As discussed in Section I, a given composition of truck traffic can be represented by a fatigue design truck of "average" weight. The stress range caused by this truck moving across a bridge can be used to calculate the bridge's fatigue life. The stress range is calculated by first placing the truck in the position that will cause the maximum tensile stress at the location being considered, then in the position that causes the maximum compressive stress.

The data, presented in this report, that are needed for this calculation, are 1) axle spacings for the fatigue design truck, 2) lateral-distribution factors for fatigue, 3) impact factors for fatigue, 4) number of stress cycles per truck passage, and 5) effects of closely spaced trucks. Application of this data in a comprehensive fatigue-design procedure will be discussed in Section IV, FATIGUE DESIGN.

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## 7. TABLES

**TABLE I**  
**Measured Stress Ranges Caused by Traffic**

Reference	State	No. of Bridges	No. of Histograms	Maximum Stress Range, ksi		Effective Stress Range, ksi	
				Avg.	Max.	Avg.	Max.
3, 9	Alabama	3	12	6.5	7.5	2.2	2.8
9, 10	Connecticut	2	20	3.9	6.0	1.5	2.1
5	Illinois*	2	17	5.3	9.0	2.4	4.9
39	Louisiana	6	62	3.7	6.0	2.1	2.9
9, 40-42	Maryland	6	13	3.6	6.0	1.4	2.4
6, 9	Michigan	7	14	5.5	7.2	2.0	3.0
9, 43	Minnesota	1	10	4.3	5.8	1.6	2.0
44	Ohio	10	49	4.2	6.5	1.3	2.3
9, 45	Pennsylvania*	1**	6	6.1	10.5	3.2	4.4
9, 46	Tennessee	2	4	3.8	4.0	1.6	1.7
9, 47	Virginia	1	8	3.2	4.5	1.4	2.1
		total=41	total=215	Avg.=4.3***		Avg.=1.8***	

\*Traffic includes heavy trucks only.

\*\*Bridge was a riveted girder bridge opened to traffic in 1953.

\*\*\*Average weighted according to number of histograms.

**TABLE II**  
**Measured Lateral-Distribution Factors for Single Trucks**

Reference	Bridge and Location*	Type	Factor Determined		Truck Position Ratio	Max. Distribution Factor		AASHTO .5 (S/5.5)
			At	From		Measured	Design	
5	1.1	3-Span	Midspan	Deflections	1.13	0.35	0.36	0.68
5	1.1	3-Span	Midspan	Deflections	2.00	0.32	0.36	0.68
5	2.1	2-Span	Midspan	Deflections	0.86	0.38	0.36	0.50
5	2.1	2-Span	Midspan	Deflections	2.00	0.28	0.33	0.50
50	1.1	5-Span	Midspan	Stress	1.00	0.42	0.46	0.73
50	1.1	5-Span	Midspan	Stress	3.00	0.48	0.46	0.73
50	1.2	5-Span	Support	Stress	1.00	0.40	0.42	0.73
50	1.2	5-Span	Support	Stress	3.00	0.33	0.42	0.73
44/51	1.1	3-Span	Midspan	Stress	**	0.24	**	0.67
44/51	2.1	3-Span	Midspan	Stress	**	0.33	**	0.86
44/51	3.1	3-Span	Midspan	Stress	**	0.26	**	0.72
44/51	4.1	3-Span	Midspan	Stress	**	0.27	**	0.67
44/51	5.1	3-Span	Midspan	Stress	**	0.22	**	0.72
44/51	6.1	3-Span	Midspan	Stress	**	0.27	**	0.72
44/51	7.1	4-Span	Midspan	Stress	**	0.32	**	0.72
44/51	8.1	3-Span	Midspan	Stress	**	0.28	**	0.68
44/51	9.1	3-Span	Midspan	Stress	**	0.23	**	0.67
19/23	1.1	Simple***	Midspan	Deflection	0.38	0.52	0.60	0.88
19/23	1.1	Simple***	Midspan	Deflection	0.69	0.41	0.42	0.88
19/23	1.1	Simple***	Midspan	Deflection	1.31	0.39	0.47	0.88
49	1.1	3-Span Cantilever	Midspan	Stress	0.70	0.37	0.42	0.53
49	1.1	3-Span Cantilever	Midspan	Stress	2.00	0.27	0.34	0.53
49	1.2	3-Span Cantilever	Support	Stress	0.70	0.35	0.42	0.53
49	1.2	3-Span Cantilever	Support	Stress	2.00	0.29	0.34	0.53
49	1.3	3-Span Cantilever	Midspan	Stress	0.70	0.38	0.42	0.53
49	1.3	3-Span Cantilever	Midspan	Stress	2.00	0.31	0.33	0.53
49	2.1	3-Span Cantilever	Midspan	Stress	0.70	0.39	0.42	0.54
49	2.1	3-Span Cantilever	Midspan	Stress	2.00	0.26	0.37	0.54
49	2.2	3-Span Cantilever	Support	Stress	0.70	0.42	0.42	0.54
49	2.2	3-Span Cantilever	Support	Stress	2.00	0.27	0.37	0.54
49	2.3	3-Span Cantilever	Midspan	Stress	0.70	0.45	0.42	0.54
49	2.3	3-Span Cantilever	Midspan	Stress	2.00	0.26	0.36	0.54
52	1.1	4-Span	Midspan	Stress	0.50	0.46	0.50	0.76
52	1.1	4-Span	Midspan	Stress	1.50	0.31	0.40	0.76
52	2.1	3-Span	Midspan	Stress	0.50	0.47	0.50	0.67
52	2.1	3-Span	Midspan	Stress	1.50	0.28	0.39	0.67
43	1.1	3-Span	Midspan	Stress	0.43	0.41	0.56	0.64
43	1.1	3-Span	Midspan	Stress	1.00	0.36	0.39	0.64
6/7	1.1	3-Span Cantilever	Midspan	Stress	1.00	0.41	0.50	0.94
6/7	1.1	3-Span Cantilever	Midspan	Stress	1.50	0.34	0.50	0.94
6/7	2.1	3-Span Cantilever	Midspan	Stress	1.50	0.26	0.41	0.56
6/7	2.1	3-Span Cantilever	Midspan	Stress	3.50	0.21	0.41	0.56
6/7	3.1	Simple	Midspan	Stress	2.50	0.28	0.42	0.55
6/7	3.1	Simple	Midspan	Stress	3.50	0.27	0.42	0.55

\*Bridge number is listed before the decimal point; location number (cross-section at which distribution factor was determined) is listed after the decimal point.

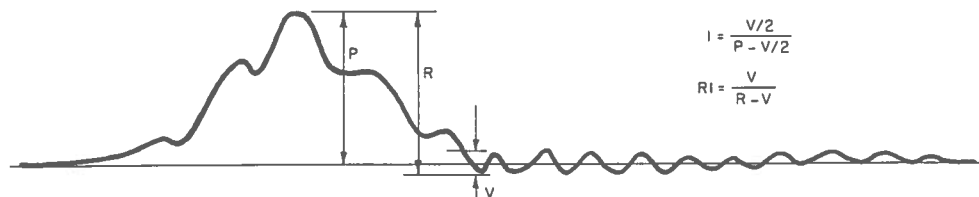
\*\*Available information insufficient to permit calculation of this value.

\*\*\*Exterior girders smaller than interior girders.

**TABLE III**  
**Impact Factors and Vibration Ratios Determined from Stress Traces for Steel Bridges**

Bridge Type	Span ft	No. Traces	Impact Factor, I		Stress-Range Impact Factor, RI		Vibration Ratio, V/R		Source
			Avg., %	Max., %	Avg., %	Max., %	Avg., %	Max., %	
SS/RB	47	4	8	14	12	22	10	18	4
	62	2	6	9	12	16	10	14	4
SS/RB	72	11	8	13	14	25	12	20	6
SS/WG	75	2	6	7	12	14	11	12	44
SS/RB	78	12	6	10	12	20	10	17	6
SS/RB	80	4	9	11	18	24	15	18	3
SS/RB	114	4	13	24	24	44	18	30	10
Averages			Avg. = 8	13	15	24	12	18	
CS/RB	36	1	27	27	53	53	34	34	44
	45	1	29	29	54	54	36	36	44
CS/RB	37	1	7	7	12	12	11	11	44
	47	2	13	14	22	24	18	19	44
CS/RB	41	1	5	5	7	7	7	7	44
	66	3	4	5	7	8	6	8	44
CS/RB	42	5	10	14	19	27	16	21	42
	52	10	6	12	12	24	10	20	42
CS/RB	43	21	7	23	12	36	10	26	5
CS/RB	44	1	3	3	5	5	4	4	44
	63	3	6	6	10	11	9	10	44
CS/RB	48	3	5	9	8	14	8	12	44
CS/RB	48	1	3	3	5	5	5	5	44
	60	3	5	6	9	10	8	9	44
CS/RB	49	2	24	31	31	36	23	26	44
CS/RB	64	1	10	10	16	16	14	14	44
	80	2	12	14	22	26	18	21	44
CS/WG	68	1	20	20	20	20	17	17	44
	114	2	12	20	22	38	17	27	44
CS/RB	75	4	7	7	10	11	9	10	5
	97	16	9	21	15	29	12	23	5
CS/RG	144	5	9	16	10	18	9	15	45
CS/WG	145	3	12	14	20	23	17	18	48
Averages			11	14	17	22	14	17	
C/RB	54S	4	7	10	14	21	12	18	49
	61A	2	11	12	19	21	16	17	49
C/RB	59S	12	5	9	9	20	8	17	6
C/RB	66A	12	3	5	6	8	5	8	6
C/RB	80S	12	17	30	49	100	37	78	6
C/WG	96A	12	7	10	12	18	11	15	6
C/WG	129A	12	15	20	27	36	21	27	6
Averages			9	14	19	32	16	26	

SS = simple span  
RB = rolled beam  
A = anchor span  
CS = continuous span  
WG = welded plate girder  
S = suspended span  
C = cantilever  
RG = riveted plate girder



**TABLE IV**  
**Impact Factors Determined from Static and**  
**Dynamic Deflection Measurements for Steel Bridges**

Reference	Bridge Type	Span, ft	No. Cases	Impact Factor	
				Avg.	Max.
8	SS/RB	45			0
		49			2
8	SS/RB	55			11
		56			16
8	SS/RB	61			6
		61			6
		61			9
8	SS/RB	64			15
		65			14
		65			46
		65			16
				Average 13	
8	CS/RB	43			8
		81			10
	CS/WG	69			13
				Average 11	
8	C/RB	50A			9
		65S			7
8	C/RB	57A			24
		61S			22
	C/RB	66A			24
		69S			24
	C/RB	69A			28
	C/WG	74A			12
	C/WG	74A			27
		47S			16
	C/RB	74A			18
		58S			16
	C/RB	74A			12
		58S			8
		75A			4
	C/WG	97A			12
				Average 16	
49	C/RB	61A	67*	11	35
		54S	69*	8	18
49	C/RB	105A	24*	7	13
		68S	62*	9	22
			Average 9		22
50	CS/RB	50	6	20	45
		65	5	28	52
			Average 24		48

SS=simple span  
RB=rolled beam  
A=anchor span  
CS=continuous span  
WG=welded plate girder  
S=suspended span  
C=cantilever

\*Impact factors determined for the heaviest-loaded beam only.

**TABLE V**  
**Information Needed for Calculating Stress Spectra from Loadings**

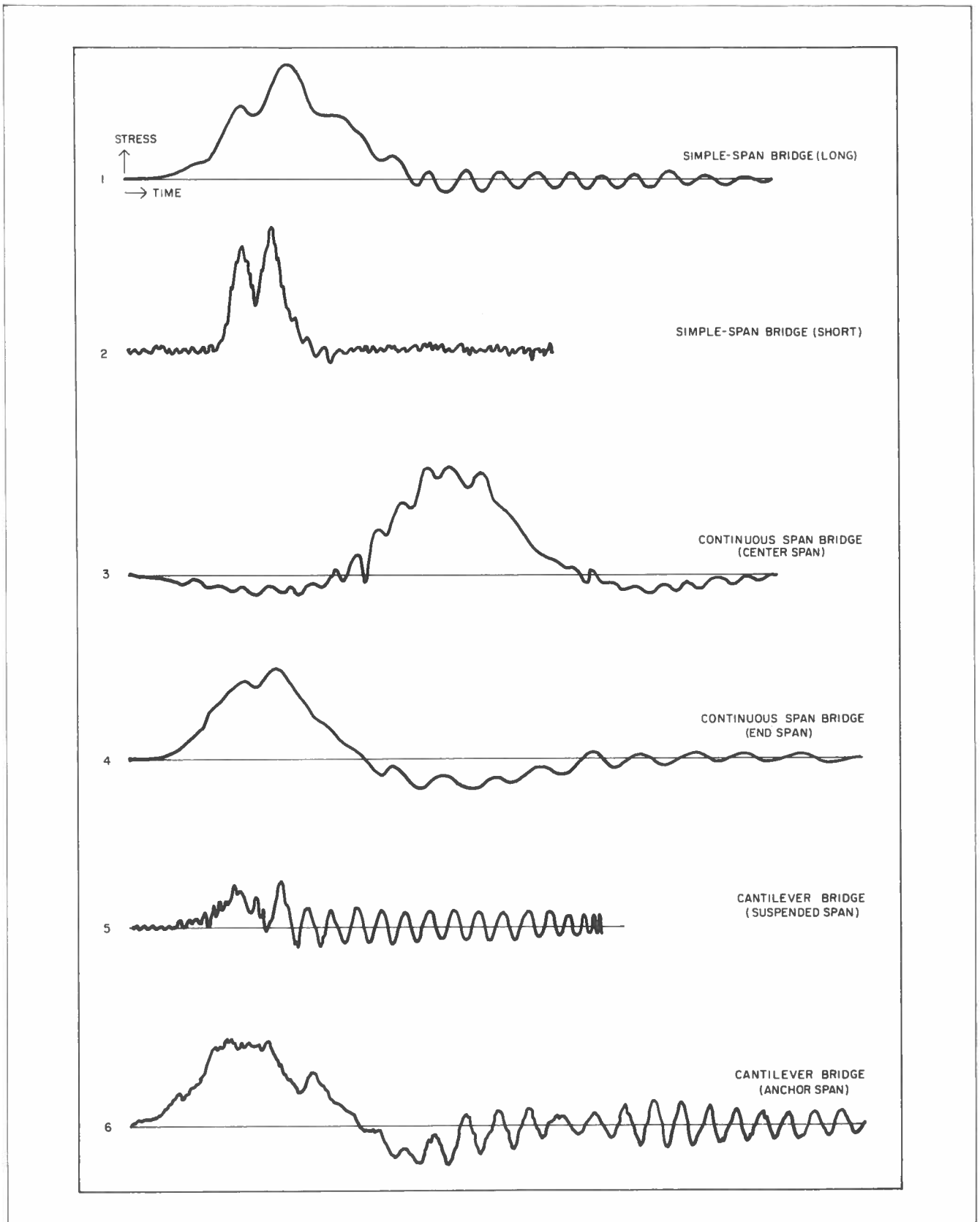
1. Spacing and Weight Distribution of Wheel Loads for Fatigue Design Trucks	Use fatigue design truck illustrated in Figure 4. This is similar to the AASHTO HS truck except that the suggested truck has its spacing between main axles fixed at 30 ft and its gross weight is 50 kips.
2. Lateral Distribution Factor	Use chart in Figure 11 to select lateral-distribution factors (fraction of truck moment carried by a beam) for interior and exterior beams as a function of the position of the outer traffic lane. An upper-limit value from the chart, usually 0.5, can be used to make an initial fatigue check. If this check is unsatisfactory, the chart's more specific lower values can be used where appropriate. These values are functions of the span, and moment of inertia of the longitudinal members, as well as the lane position.
3. Impact Factor	Use AASHTO equation to determine impact factors for simple or continuous girder bridges. When calculating stress range for continuous bridges, apply the impact factor to both positive and negative moments. The AASHTO factors may not be conservative for certain cantilever (suspended-span) girder bridges, but alternative factor determination methods are not available.
4. Number of Stress Cycles Per Truck Passage	<p>Use the appropriate number of stress cycles per truck passage as follows:</p> <p><i>Main Longitudinal Members</i></p> <p>Simple-span girders:</p> <p style="padding-left: 40px;">above 40-ft span, 1.0</p> <p style="padding-left: 40px;">below 40-ft span, 2.0</p> <p>Continuous-span girders near interior support:*</p> <p style="padding-left: 40px;">above 40-ft span, 1.5</p> <p style="padding-left: 40px;">below 40-ft span, 1.0</p> <p>Continuous-span girders elsewhere:</p> <p style="padding-left: 40px;">above 40-ft span, 1.0</p> <p style="padding-left: 40px;">below 40-ft span, 2.0</p> <p>Cantilever (suspended-span) girders, 10.0**</p> <p>Trusses, 1.0</p> <p><i>Transverse Members</i></p> <p style="padding-left: 40px;">above 20-ft spacing, 1.0</p> <p style="padding-left: 40px;">below 20-ft spacing, 2.0</p>
5. Effect of Closely Spaced Trucks	Except under certain conditions that tend to bunch trucks, the effect of closely-spaced trucks is negligible. These conditions include a traffic signal or a steep grade on a two-lane road.

\*Within a distance equal to 0.1 of the span on each side of the support.

\*\*Unless specific contrary information is available.



## 8. FIGURES



**Figure 1.** Typical stress traces for the passage of a single truck across various types of bridges.

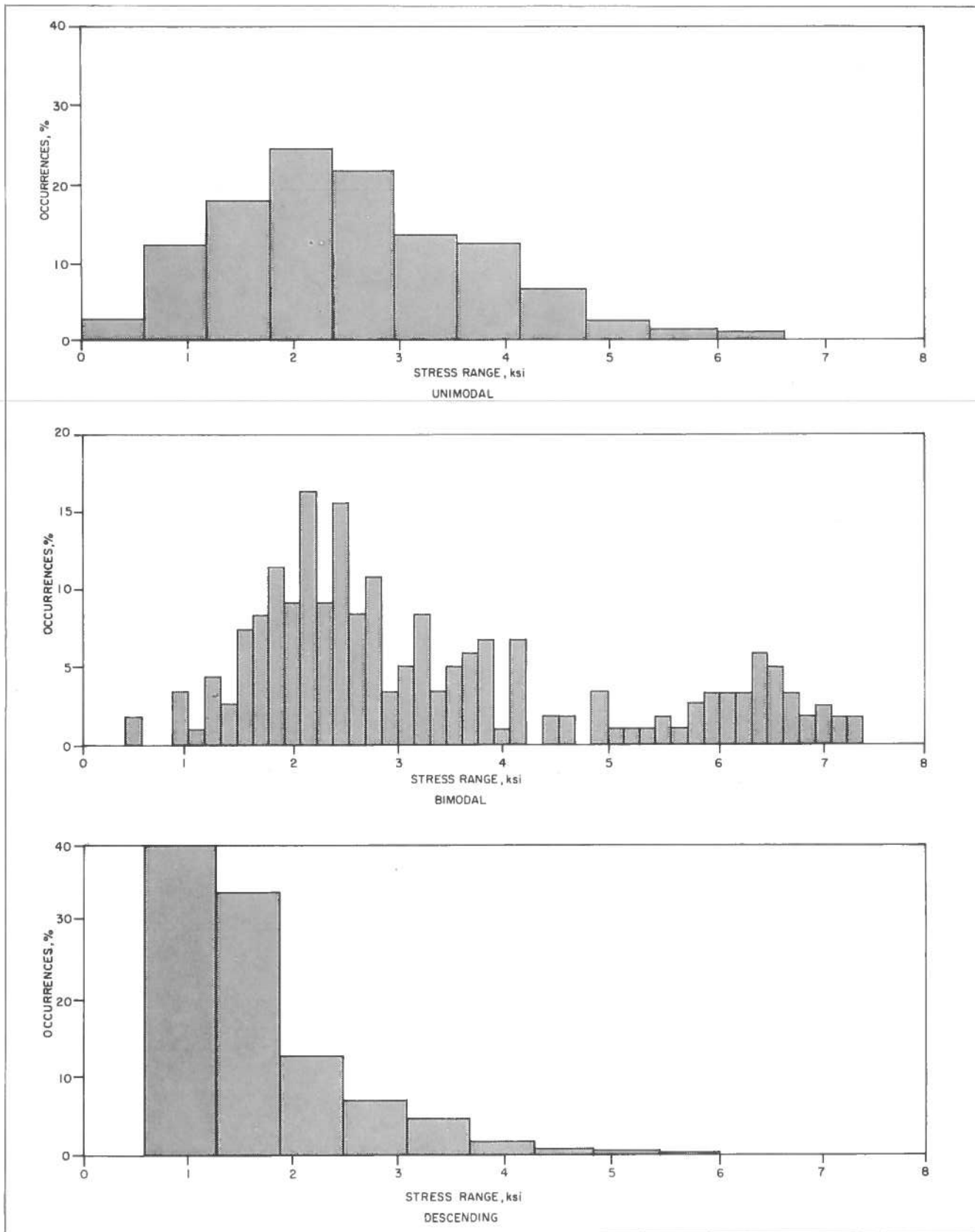
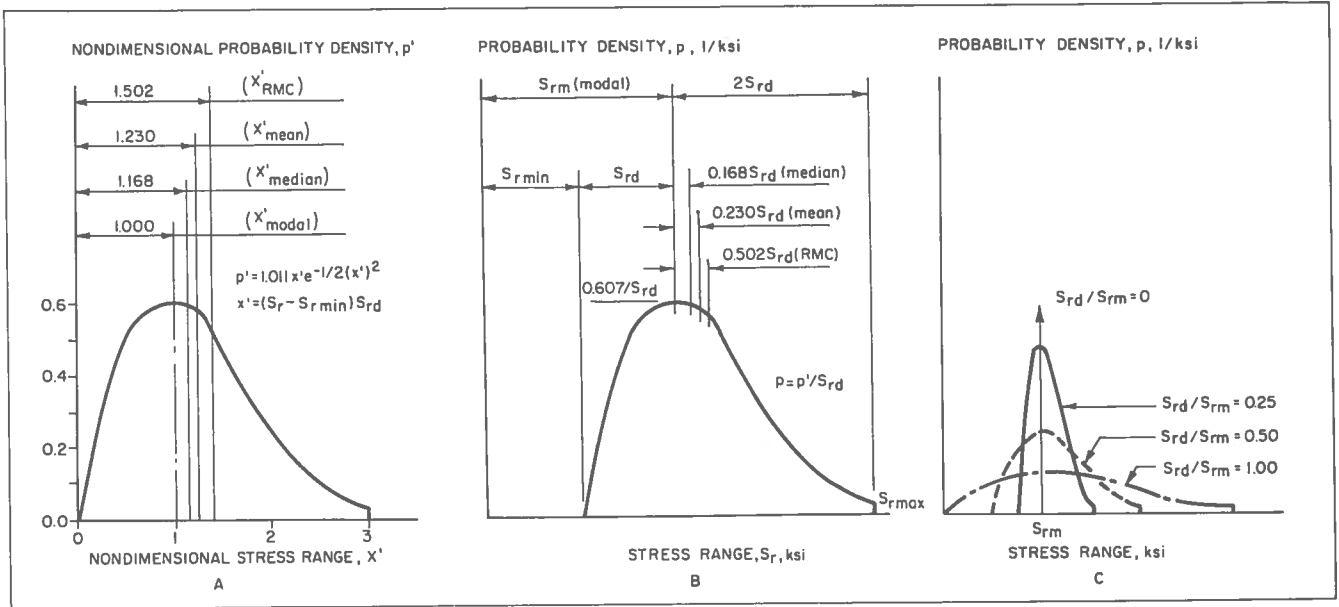
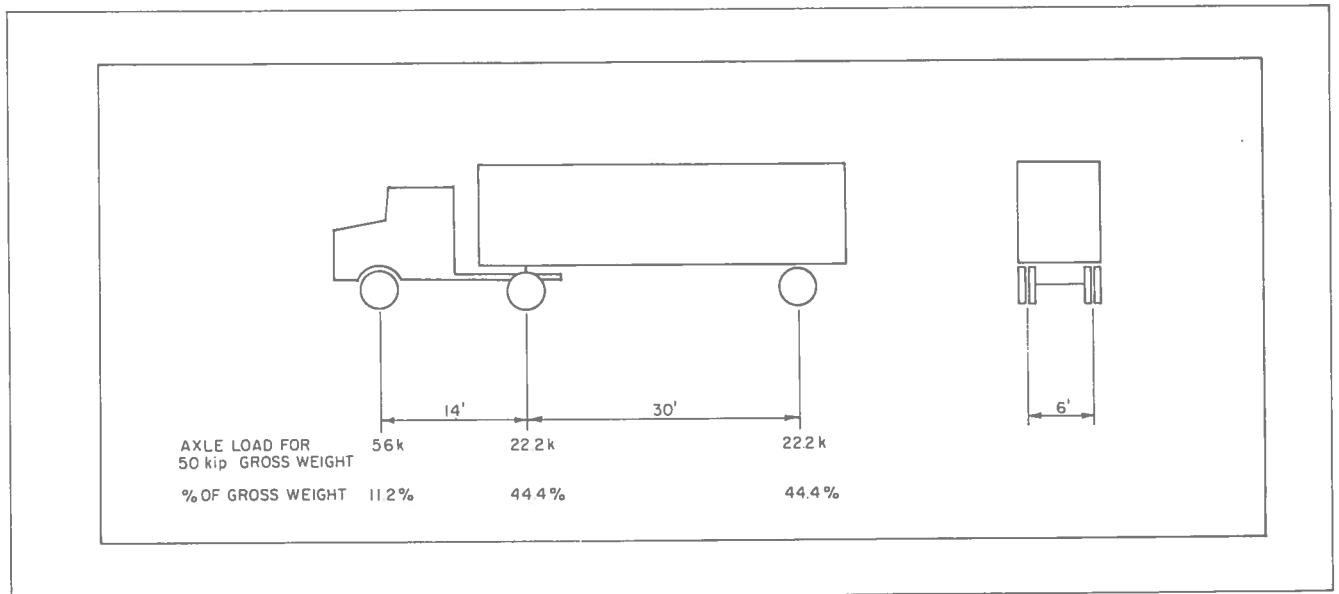


Figure 2. Typical stress-range histograms.



**Figure 3.** Characteristics of Rayleigh probability curves.  
Conversion factor: 1 ksi = 6.895 MPa.



**Figure 4.** Fatigue design truck.

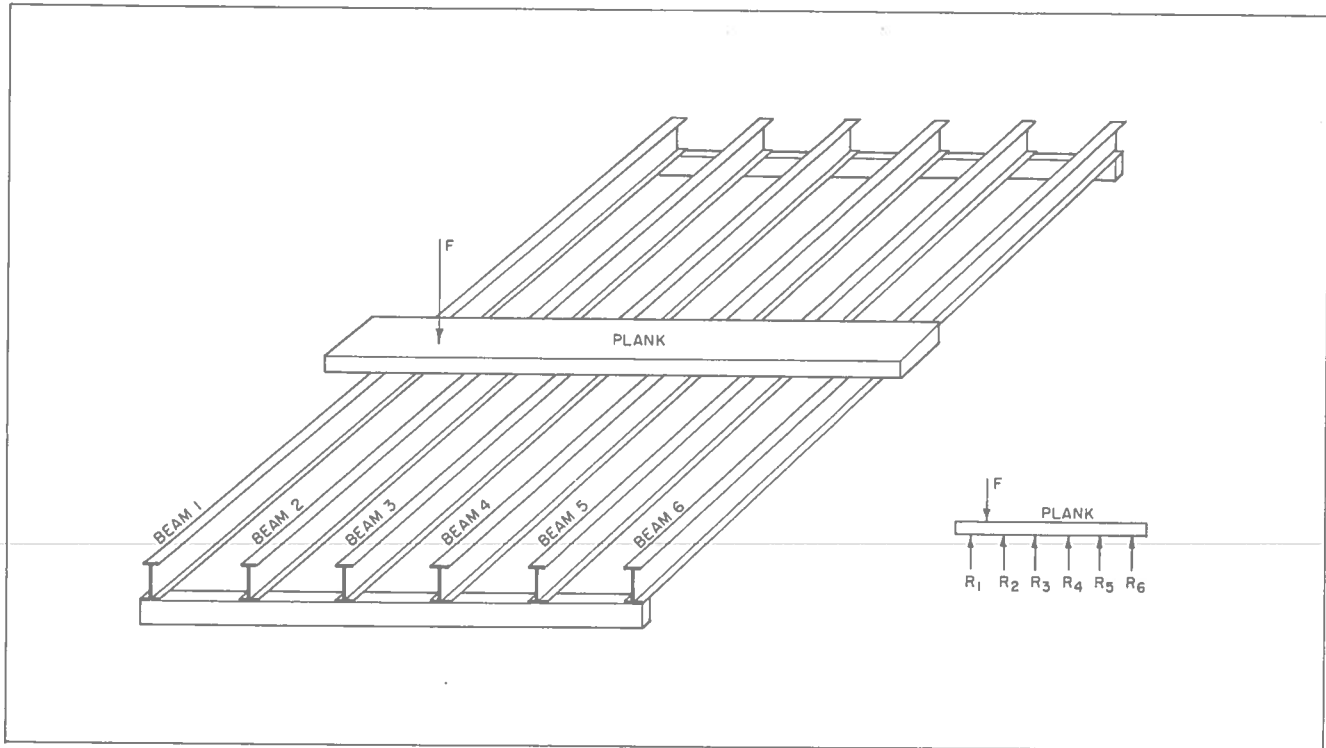


Figure 5. A plank supported on beams.

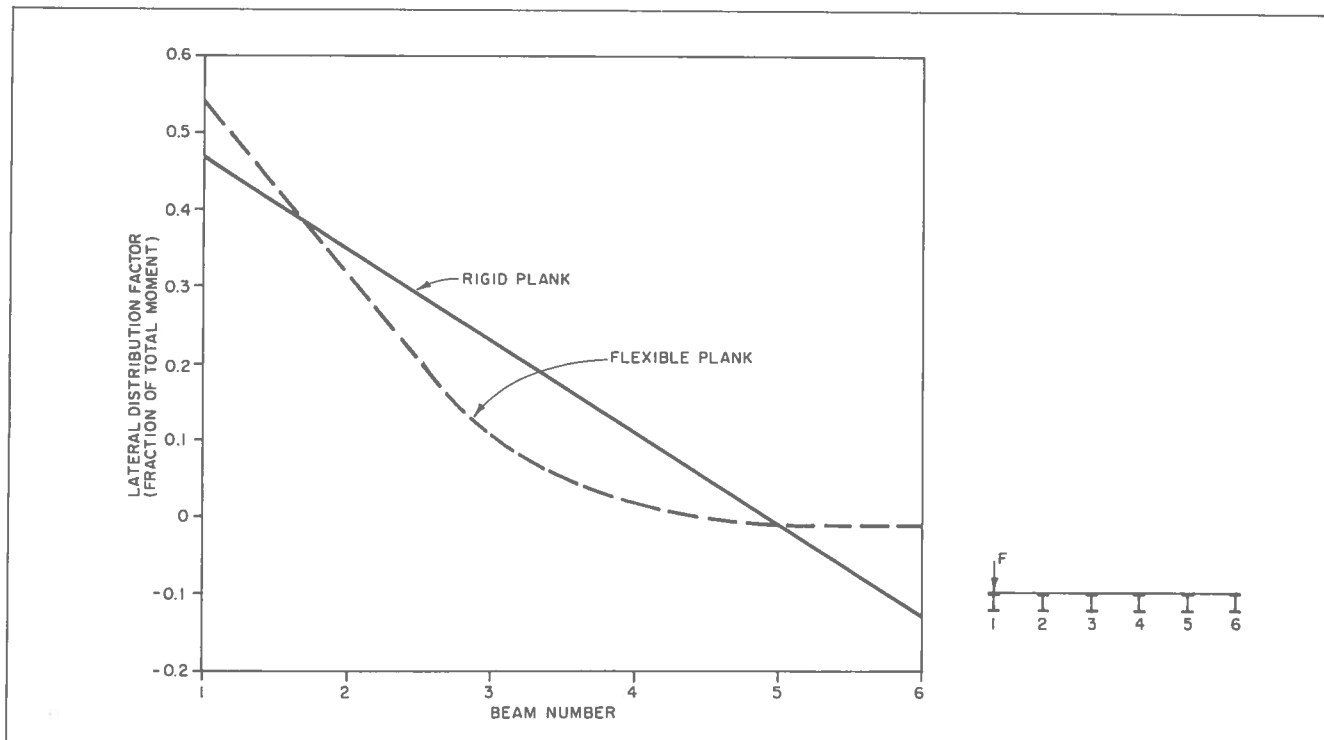
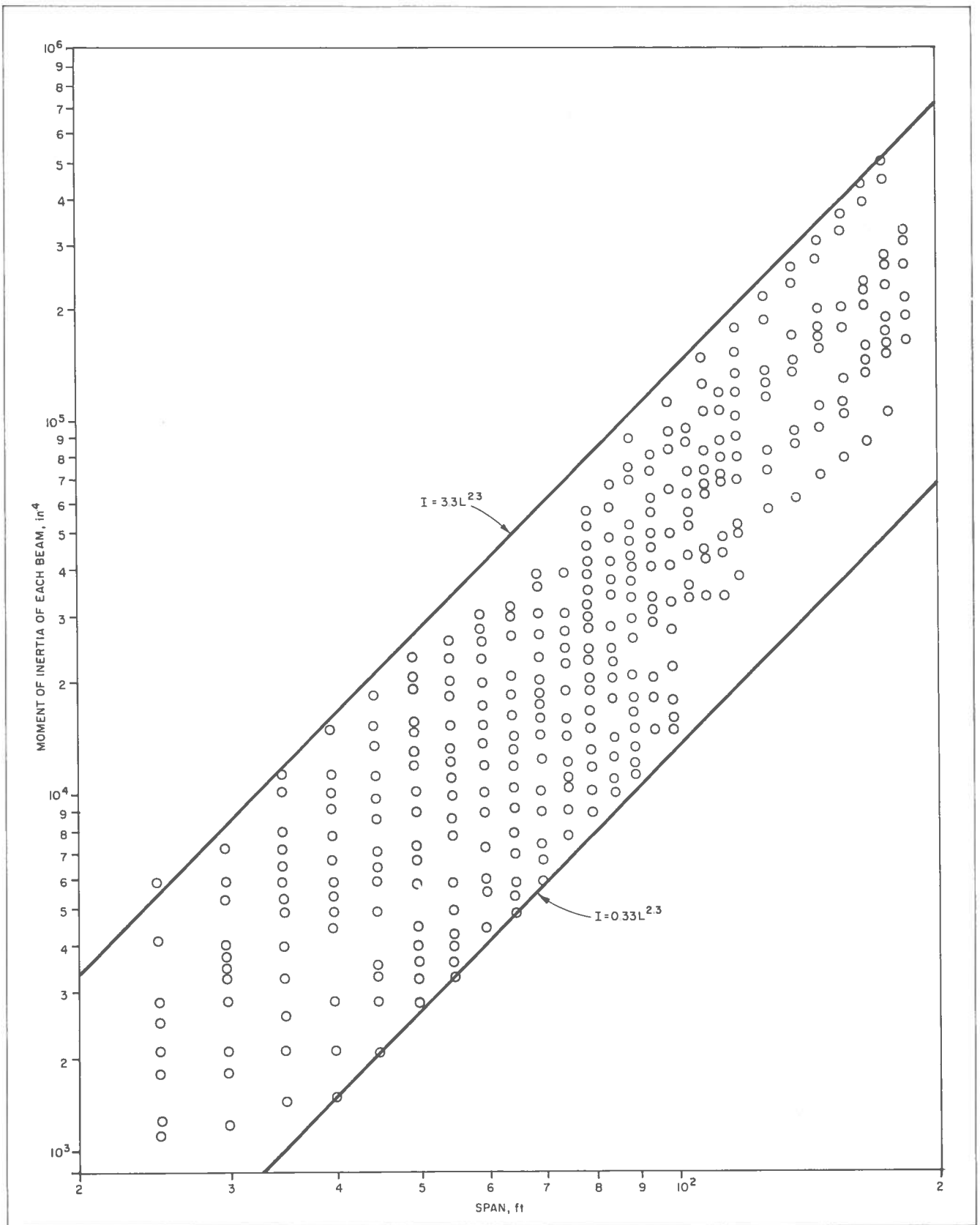
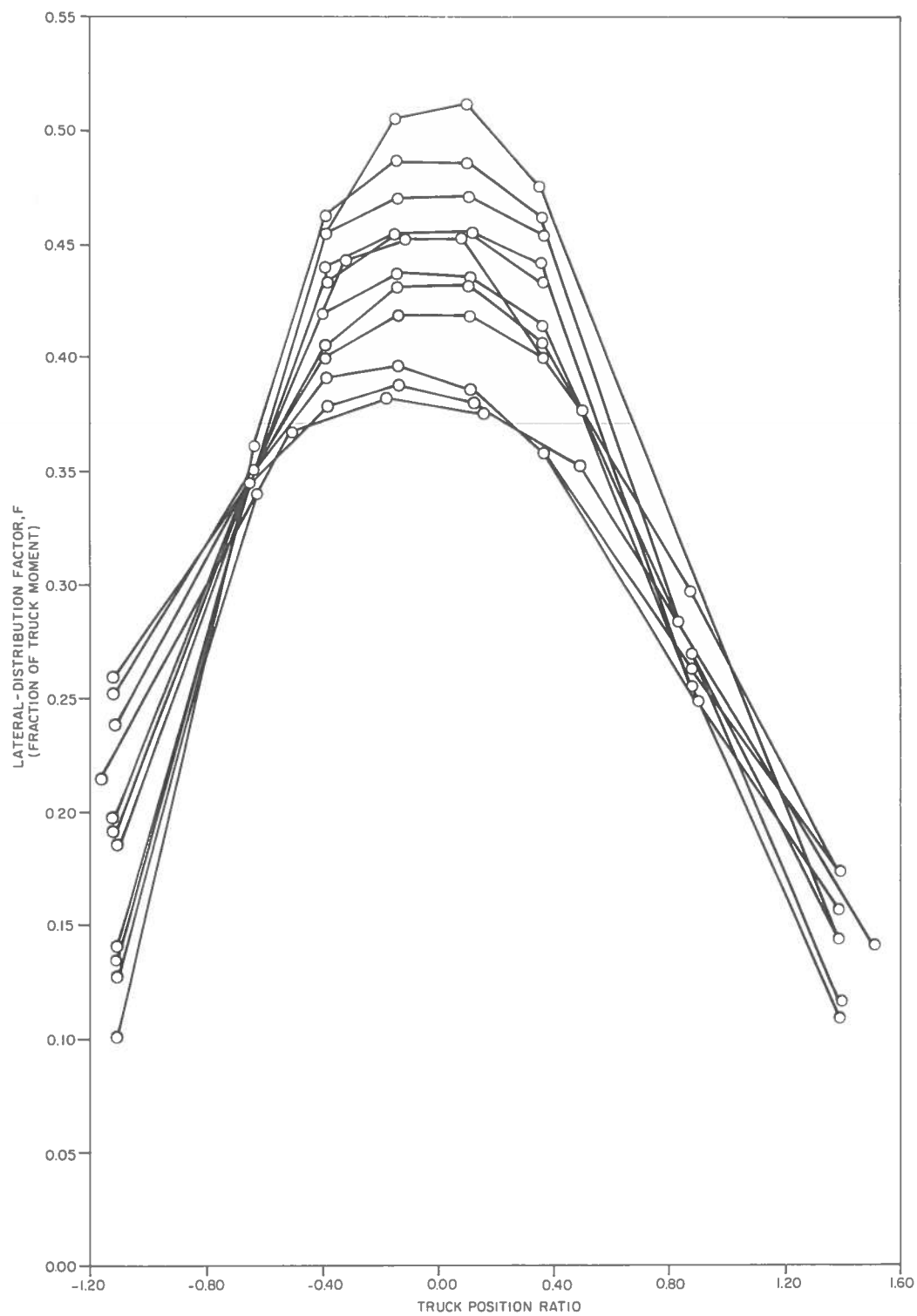


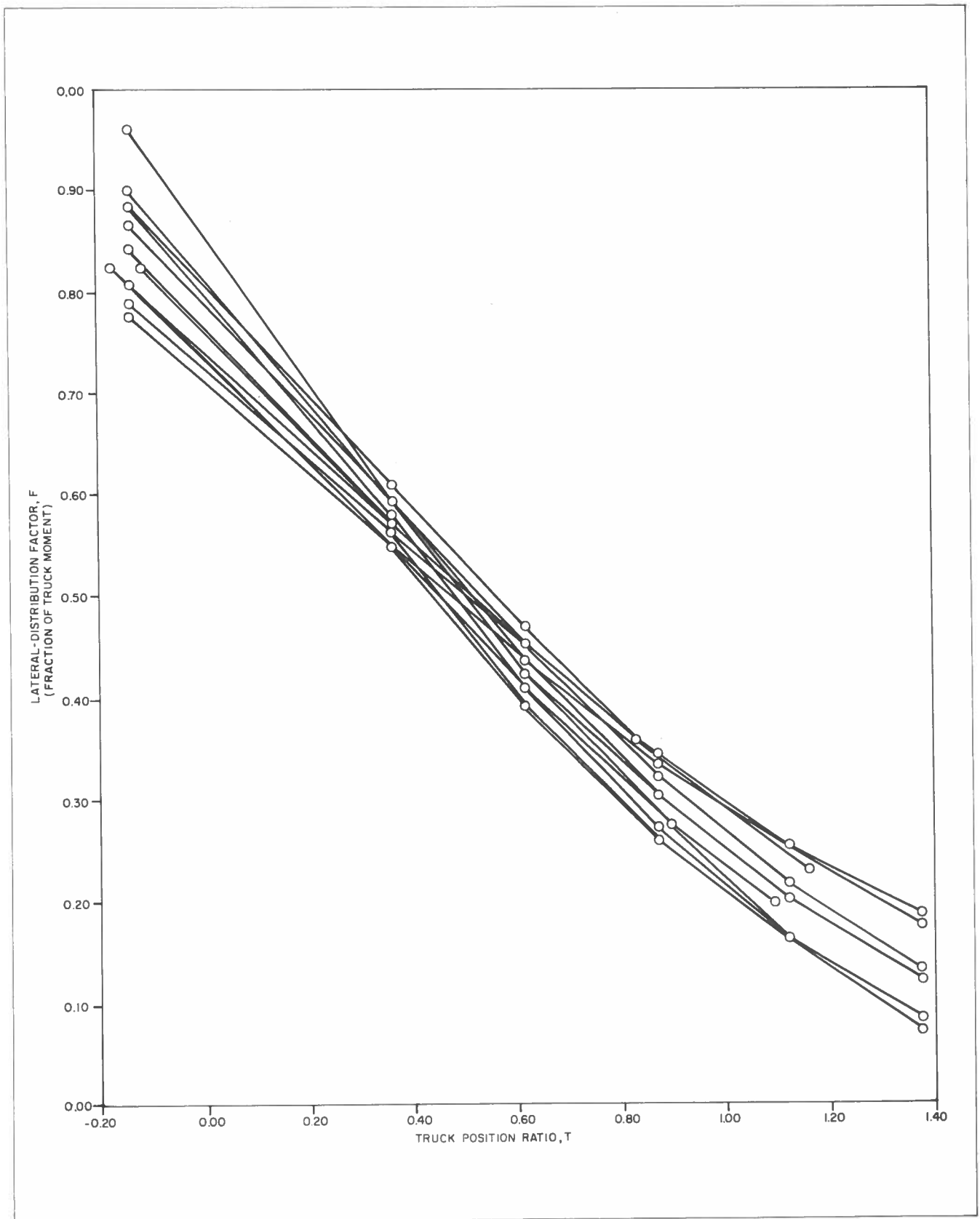
Figure 6. Lateral distribution for a plank on beams.



**Figure 7.** Variation of moment of inertia with span for steel bridges.



**Figure 8.** Lateral distribution factor as a function of truck position ratio for interior beams.



**Figure 9.** Lateral distribution factor as a function of truck position ratio for exterior beams.

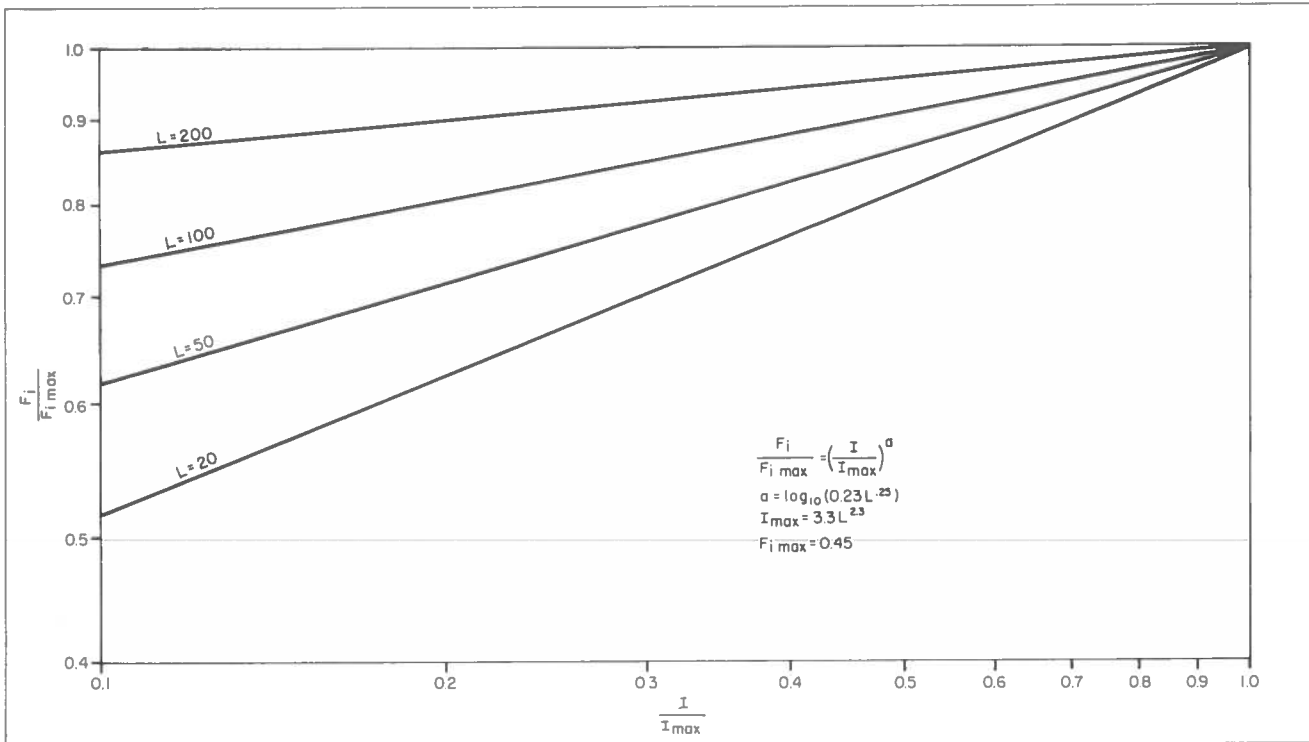


Figure 10. Distribution factor relations for interior beams.

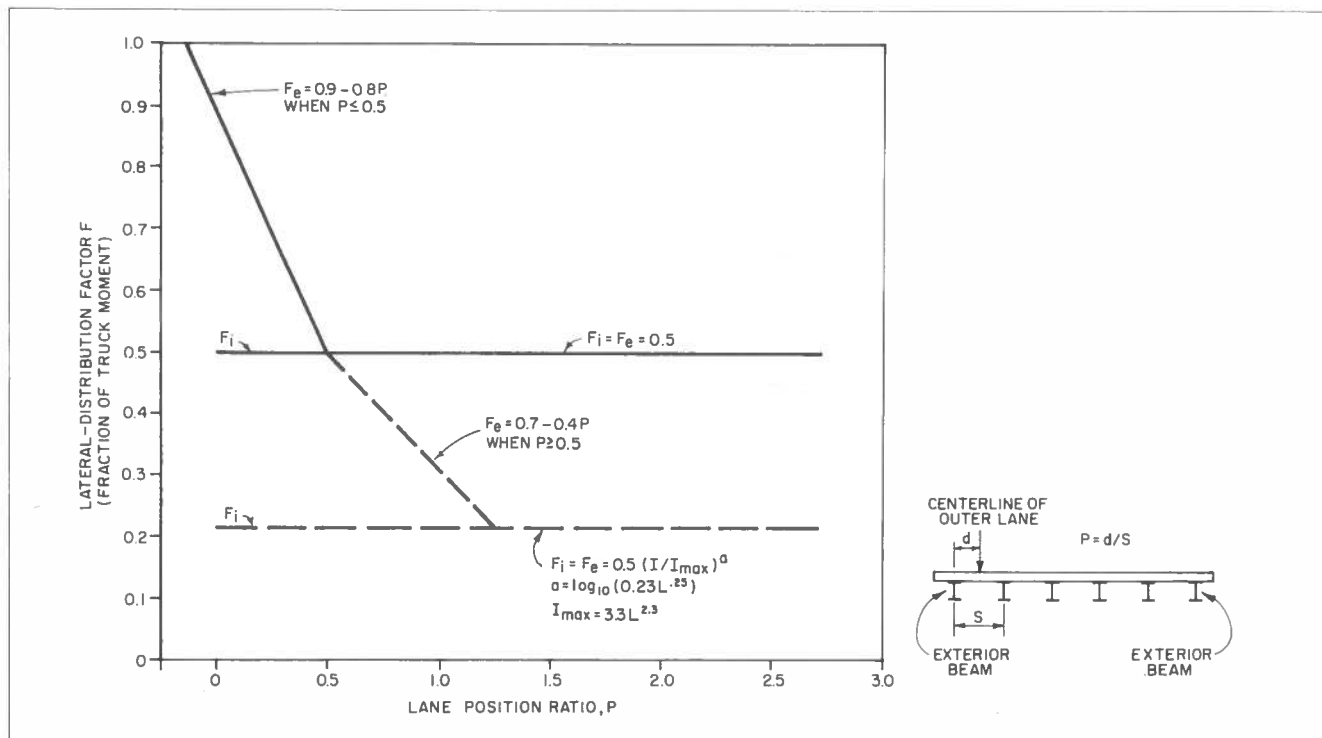
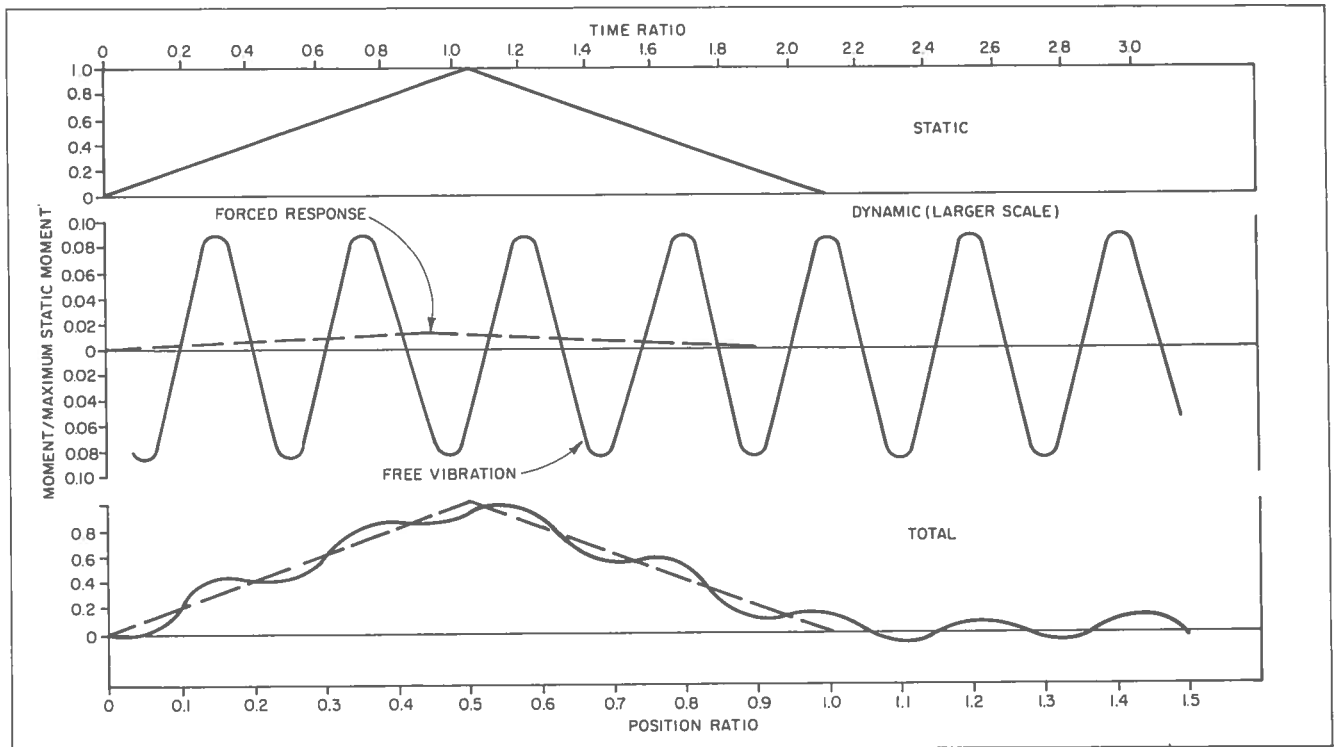
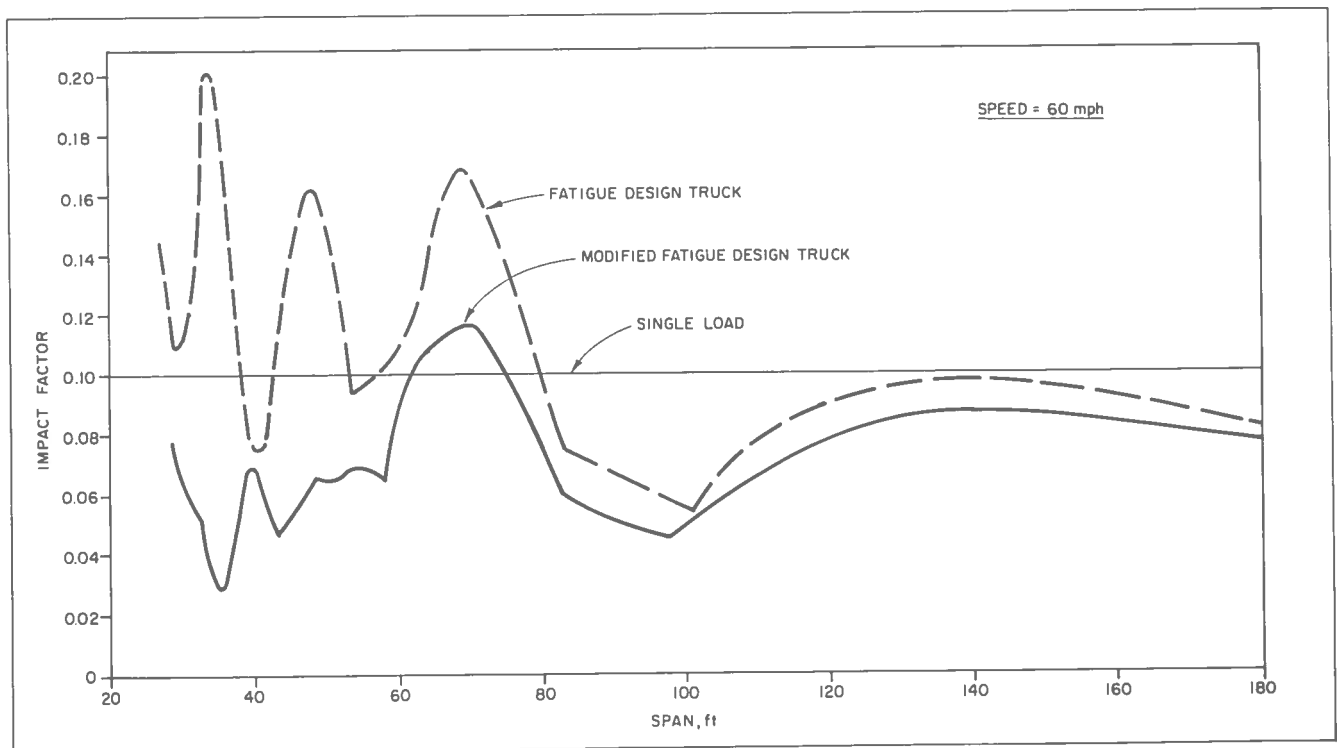


Figure 11. Lateral distribution chart for fatigue design.

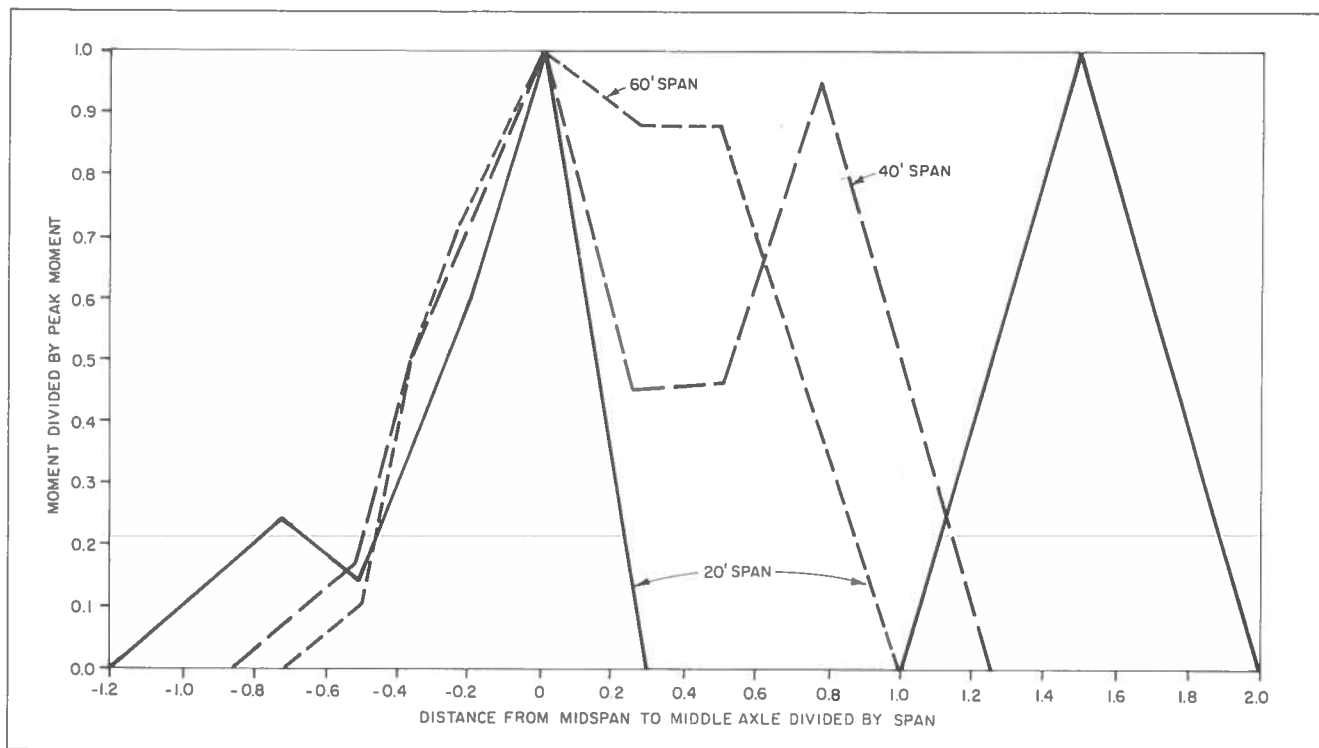




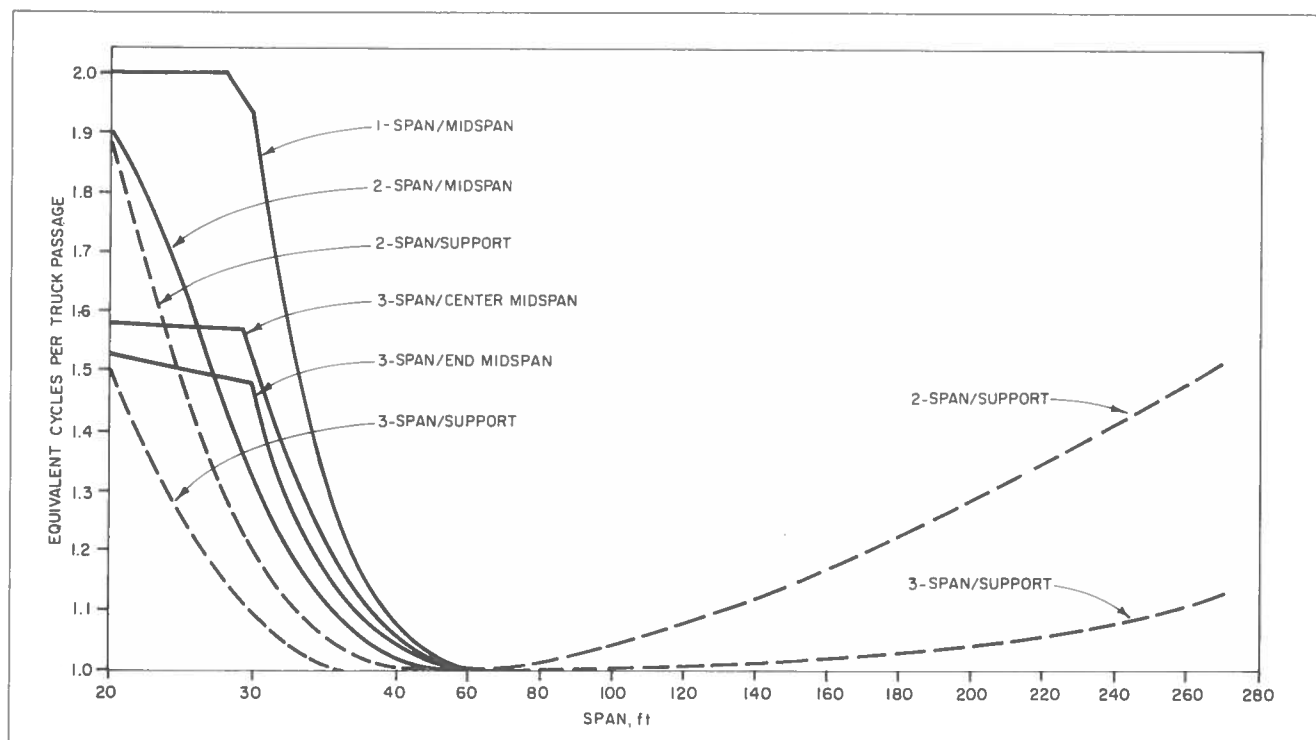
**Figure 12.** Midspan moment caused by a load moving across a simple beam.



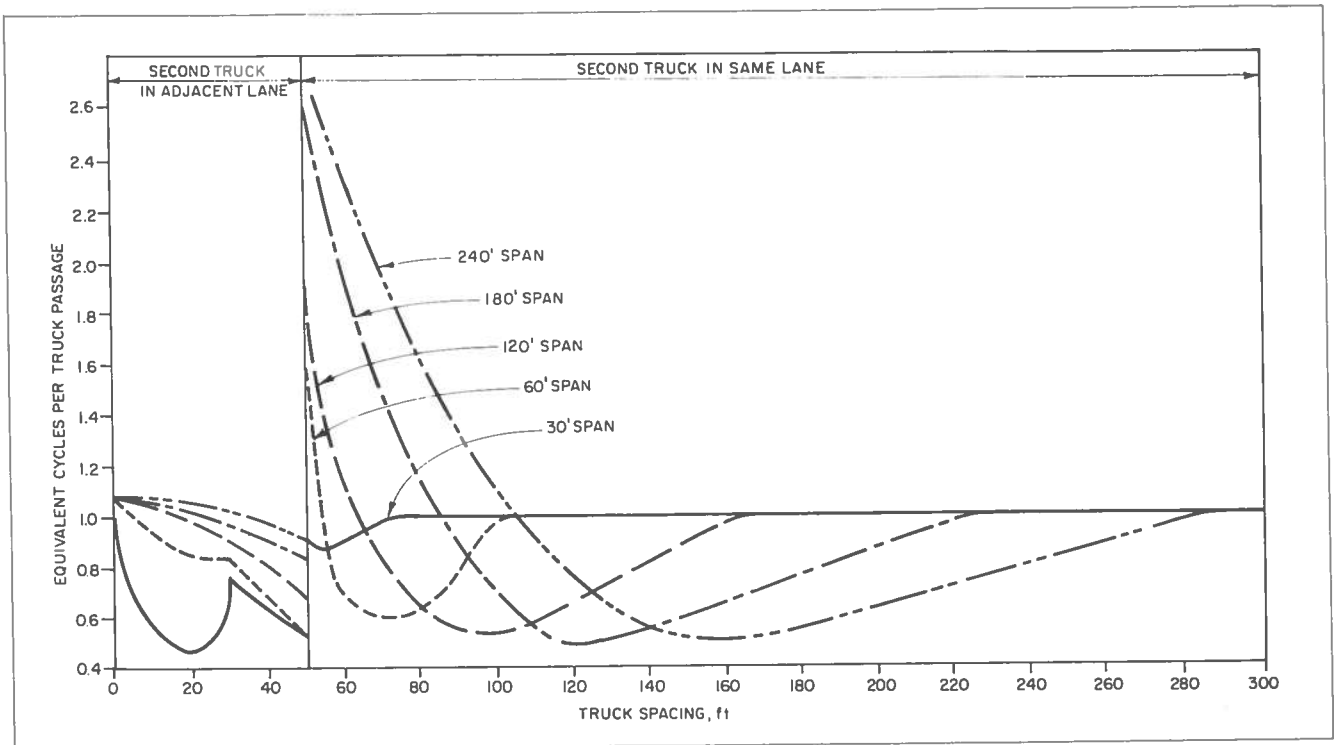
**Figure 13.** Theoretical impact factor as a function of span.



**Figure 14.** Midspan moment as a function of truck position for simple-span bridges.



**Figure 15.** Equivalent cycles per truck passage as a function of span for longitudinal members.



**Figure 16.** Interaction of two trucks on simple spans of various lengths.



## SECTION III

# Fatigue Behavior

## CONTENTS

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1. INTRODUCTION	6.54	b) Attachments	6.59
2. NATURE OF FATIGUE	6.54	c) Surface Conditions	6.59
—Mechanism of Fatigue	6.54	—Material	6.60
—SN Curve	6.54	a) Tensile Strength	6.60
—Scatter	6.55	b) Yield Strength	6.60
—Variable-Amplitude Loading	6.55	c) Fracture Toughness	6.60
a) Effective Constant-Amplitude Stress Range	6.55	d) Notch Sensitivity	6.60
b) Equivalent Number of Cycles	6.56	—Environment	6.60
3. FACTORS AFFECTING FATIGUE BEHAVIOR	6.56	a) Corrosion Fatigue	6.60
—Stress Spectrum	6.56	b) Fretting Fatigue	6.61
a) Stress Range	6.57	—Fabricated Members	6.61
b) Minimum, Maximum or Mean Stress	6.57	4. SUMMARY	6.61
c) Residual Stresses	6.57	5. REFERENCES	6.62
d) Multi-axial Stresses	6.58	6. TABLE	6.63
e) Periodic Overloads	6.59	—Fatigue Behavior Under Periodic Overload	6.63
—Detail Geometry	6.59	7. FIGURES	6.63
a) Notches, Holes, and Protrusions	6.59		

## 1. INTRODUCTION

This section reviews present-day concepts concerning the nature of fatigue and discusses the facts that affect fatigue life. In examining the nature of fatigue, a description is given of 1) the initiation and growth of cracks to final failure, 2) methods of presenting con-

stant- and variable-amplitude fatigue data, 3) scatter in such data, and 4) various design approaches. Finally, an analysis is presented of the factors affecting fatigue life; this includes stress spectrum, detail geometry, material and environment.

## 2. NATURE OF FATIGUE

### MECHANISM OF FATIGUE

Fatigue failures are caused by the initiation and/or growth of cracks when cyclic stresses are applied.<sup>3,4</sup> If the cyclic stresses are below a certain value called the fatigue limit, cracks do not initiate or those present do not grow. If the cyclic stresses are above the fatigue limit, cracks initiate after a certain number of cycles and then grow at a progressively increasing rate until complete fracture occurs (Fig. 1). Thus, a large portion of the total life occurs when the crack size is small.

The number of cycles required to initiate the crack decreases as the magnitude of the cyclic stress increases. Also, the crack grows faster if the cyclic stresses are higher. After the crack reaches a certain size, growth is very fast, as illustrated by the near vertical portion of the curves in Figure 1. Very little life remains at that stage even if the crack merely covers a small portion of the cross-sectional area. Final failure occurs when the net area of the cracked section is small enough so that a single application of the cyclic load can cause complete rupture.

Almost all bridge members contain stress raisers that create local regions of higher stress. For members without welds or sudden changes in geometry, surface roughness acts as the stress raiser. For welded details, such as a coverplate end or a stiffener, sudden changes in geometry interrupt the smooth flow of stresses and act as stress raisers. In addition, small imperfections such as slag inclusions and undercuts often occur at the toe of the weld where the stress concentration caused by geometry and residual stresses caused by welding

are greatest.<sup>5,6,7</sup> Thus, the stress raising effect of the imperfection is superimposed on that of the geometry. Fillet welds parallel to the direction of stress do not interrupt the smooth flow of stresses but usually contain internal imperfections, such as gas pockets<sup>6</sup> and surface roughness, that act as stress raisers. In many instances, these imperfections are crack-like, and the entire fatigue life is spent in crack growth. After cracks initiate, they become the most severe stress raisers and cause very high stresses in a very small area around the crack tip. The magnitude of these stresses and the size of the affected area both increase rapidly as the size of the crack increases. This is the process which causes the crack growth to accelerate exponentially (Fig. 1).

### SN CURVE

Fatigue behavior can be conveniently represented by an SN curve in which the total number of cycles to failure,  $N$ , is plotted against a cyclic stress parameter,  $S$ , such as the stress amplitude or stress range. For steel bridge applications, it is customary to represent the SN relationship by two straight lines on a log-log plot as illustrated by the heavy solid lines in Figure 2a. The horizontal line corresponds to the fatigue limit. A slightly curved line is sometimes used instead of the sloping straight line to represent the finite-life portion of the SN curve. However, this more complex representation is rarely justified for bridge applications. A semi-log plot of  $S$  against  $\log N$  is sometimes used, but the log-log plot is preferred because it is simpler to

represent mathematically and usually fits experimental data slightly better.<sup>8</sup>

The mathematical characteristics of the SN curve are shown in Figure 2. On the log plot, the sloping solid line defined by

$$\log N = \log A - B \log S \quad (1)$$

in which B is the reciprocal of the slope and log A is the x intercept. The y intercept is (log A)/B. Equation 1 can also be expressed as

$$N = \frac{A}{S^B} \quad (2)$$

the corresponding natural plot is shown in Figure 2b.

### SCATTER

If a series of fatigue tests is performed on the same type of specimen under the same cyclic stresses, the resulting fatigue lives will be scattered over a wide range. This scatter can be adequately represented by a statistical log-normal distribution (probability density) curve;<sup>4,9,10</sup> that is, the *logarithms* of the lives are distributed normally. Similarly, the logarithms of the deviations of the individual log N values from their mean value are also distributed normally (Fig. 3). Other distribution curves with slightly different shapes are sometimes used to define the scatter in fatigue data.<sup>4,10</sup> The area under the distribution curve between any two values of log N - (log N)<sub>mean</sub> corresponds to the percentage of occurrences within that interval. About 95 percent of the log-lives are within a range twice the standard deviation on both sides of the mean life. This range is cross-hatched in Figure 3. The standard deviation is the square root of the sum of the squares of the deviations of the individual log-lives from the mean life log-life divided by the number of data.\*<sup>11</sup>

If a series of fatigue tests is performed on the same type of specimen but under different cyclic stresses, the resulting fatigue lives will be scattered about a mean, or best fit, line similar to the (heavy solid) SN curve in Figure 2. The square root of the sum of the squares of the deviations from this line divided by the number of data\* is called the standard error of estimate, E, and is analogous to the standard deviation.<sup>11</sup> Scatter bands (shown as dashed lines) drawn at about twice the standard error of estimate on both sides of the mean line as shown in Figure 2 include about 95 percent of the data. Such scatter bands are often referred to rather loosely as 95-percent confidence limits; an explanation of the more precise definitions of limits used by statisticians is beyond the scope of this publication but is readily available elsewhere.<sup>8,10,11</sup>

To give some indication of the amount of scatter occurring typically in fatigue tests of bridge-type de-

tails, 28 sets of data from two sources<sup>8,12</sup> were investigated. The mean value of log<sub>10</sub>E (or the standard error of the estimate of log<sub>10</sub>N) for these sets was 0.125, and the standard deviation of log<sub>10</sub>E was 0.052. The lower plot in Figure 3 is for log<sub>10</sub>E = 0.125 and represents typical scatter in the lives of specimens tested under the same cyclic stresses. The upper plot is for log<sub>10</sub>E = 0.125 + 2(0.052) and represents scatter that is rarely exceeded. Thus, the lower limit of the scatter band for bridge-type details is typically about ½ the mean line, and rarely lower than ⅓ the mean line. The upper limit of the scatter band is typically about 2 times, and rarely more than 3 times, the mean line. The corresponding scatter factor (upper limit divided by lower limit) for the life of a bridge detail is typically about 4 and rarely exceeds 9. This is a relatively low scatter for fatigue results since factors of 100 are not uncommon in some types of fatigue tests.<sup>4</sup> Scatter tends to decrease as the severity of stress raisers in the specimen increases.<sup>4</sup>

### VARIABLE-AMPLITUDE LOADING

Constant-amplitude loading, frequently used in laboratory fatigue tests, consists of a series of identical cycles. Variable-amplitude loading, which occurs widely in practice, consists of a series of cycles with different magnitudes, usually applied in a random sequence. The magnitude of each individual cycle is defined by a stress parameter such as stress amplitude or stress range. Stress range is used in subsequent discussions because it is the best parameter for bridge applications. The occurrence frequency of stress ranges of different orders of magnitude can be defined either by a histogram or by a probability-density curve as discussed in the second section of this series.<sup>2</sup>

Different forms of variable-amplitude loading are illustrated in Figure 4. One of the least complicated forms (Fig. 4a) consists of simple individual cycles superimposed on a constant base. The individual cycles can either be interrupted by pauses or continuously connected. Each individual cycle continuously increases from a valley to the next peak and then continuously decreases to the next valley. This form of loading has been used in tests of simulated bridge members.<sup>8</sup> A somewhat more complicated form (Fig. 4b) consists of repeated complex cycles that have secondary reversals between the main peaks and valleys. The individual complex cycles are of different sizes (stress ranges) but are geometrically similar. This form is typical of truck loadings on bridges;<sup>2</sup> each truck passage causes one complex cycle. The most complicated form of variable-amplitude loading (Fig. 4c) is a continuous complex variation that is not repeated.

#### Effective Constant-Amplitude Stress Range

Frequently, it is convenient to represent a variable-amplitude stress-range spectrum by an effective constant-amplitude stress range selected on the basis that a given number of its cycles causes the same fatigue damage as an equal number of variable-amplitude cycles.<sup>8</sup> The effective stress range can be calculated from the histogram defining the variable-amplitude spec-

\*If the number, n, of data is small, it is appropriate to divide by n - 1.<sup>11</sup>

trum by the following equation<sup>8</sup>

$$S_{re} = \left( \sum \alpha_i S_{ri}^B \right)^{\frac{1}{B}} \quad (3)$$

in which  $S_{ri}$  is the midwidth of the  $i$ th interval,  $\alpha_i$  is the frequency of occurrence for that interval, and  $B$  is the reciprocal of the slope of the log SN curve as shown in Figure 2. For most structural details,  $B$  is 3.0. Equation 3 conservatively neglects any effects of fatigue limit on the effective stress range.<sup>8</sup>

#### Equivalent Number of Cycles

Convenient representation of a complex cycle can be made with an equivalent number of simple cycles having the same stress range as the complex cycle. This is particularly helpful for bridge applications since each truck passage produces one complex cycle and this can then be represented by an equivalent number of simple cycles.<sup>2</sup> The equivalent number of cycles can be calculated by the method described as follows:

The secondary reversals included in a complex cycle, such as illustrated in Figure 4b cause the same amount of fatigue damage as individual simple cycles of the same size. Therefore, the complex cycle is first decomposed into several individual cycles of different sizes as illustrated in Figure 5. These cycles of different sizes are then represented by an equivalent number of cycles of the same size.

The method of decomposing the complex cycle, illustrated in Figure 5 is similar to the rain-flow method described elsewhere.<sup>4,13</sup> The complex cycle begins and ends at the same stress—the base stress. The complex cycle consists of three stages. Stage 1 is between the beginning and the maximum or minimum stress for the cycle, whichever is closer. Stage 2 is between the maximum and minimum stresses for the cycle. Stage 3 is between the end and the maximum or minimum stress, whichever is closer. If the entire curve is above or below the base stress, Stage 2 does not exist.

The complex cycle can be decomposed into a primary cycle and one or more cycles of a higher order (secondary, tertiary, etc.)

Within each stage, the primary cycle follows a path without reversal (Fig. 5). When the path of the complex cycle reverses within a stage, the primary cycle moves

horizontally to intersect this path at a later point. This horizontal line is the base stress for the next higher order cycle.

The algebraic difference between the maximum and minimum stresses is the stress range for both the complex cycle and the primary cycle. Each higher order cycle begins and ends at the base stress for that higher order cycle. Each higher cycle has one peak or valley, and the algebraic difference between this peak or valley and the base stress for that cycle is the stress range for the cycle. A cycle of any order may include higher order cycles.

Fatigue damage is proportional to  $S_r^B$  as indicated by Equation 2. For example, for  $B = 3$ , the fatigue life at a stress range of 25 ksi is 8 times the life at a stress range of 50 ksi. Thus, each higher order cycle does  $(S_{ri}/S_{rp})^B$  times as much damage as the primary cycle and the equivalent number of primary cycles,  $N_e$ , for a complex cycle is

$$N_e = 1 + \left( \frac{S_{r1}}{S_{rp}} \right)^B + \left( \frac{S_{r2}}{S_{rp}} \right)^B \dots \left( \frac{S_{ri}}{S_{rp}} \right)^B \quad (4)$$

in which  $S_{rp}$  is the stress range for the primary cycle and  $S_{ri}$  is the stress range for a higher order cycle. Equation 4 conservatively neglects any effects of fatigue limit on the equivalent number of cycles.

In calculating the equivalent number of cycles it was assumed that stress range is the only governing stress parameter. This is an accepted assumption for bridge applications,<sup>12</sup> but not always for other applications, such as transportation equipment. Consequently, the base stresses for the higher order cycles are listed in Figure 5 even though they were not used in calculating the equivalent number of cycles.

Any complex stress variation of finite length can be decomposed and represented by an equivalent number of cycles. However, for applications involving a long continuous variation (Fig. 4c), it is not helpful to use the concept of an equivalent number of cycles. Instead, it is more convenient to work with the individual decomposed cycles or to represent the entire spectrum by an effective constant-amplitude stress range.

## 3. FACTORS AFFECTING FATIGUE BEHAVIOR

### STRESS SPECTRUM

An individual stress cycle can be defined in several different ways (Fig. 6). Two parameters are needed: one to define the cyclic variation and the other to define the superimposed steady stress. Either the stress range or the alternating stress can be used to define the cyclic variation, but for structural applications stress range is most commonly used. The minimum, maximum, or mean stress can be used to define the steady stress, but for bridge applications minimum stress is most common. Alternatively, the stress cycle can be

defined by the stress ratio and any one of the other stress parameters shown in Figure 6.

The effects of these cyclic stress parameters on the fatigue behavior of a specimen or detail subjected to a continuous repetition of similar cycles (constant-amplitude loading) is discussed later in this section. The shape and wave length of the cycles have little effect on the fatigue life. Similarly, the testing speed in cycles per minute has little effect under normal conditions. Also, time delays, or rest periods, between the application of individual cycles have little effect under normal



conditions.

Repeated loadings involving cycles of different sizes (variable-amplitude loading) or any complex variation of stress with time can be related to constant-amplitude fatigue behavior by the methods discussed earlier in this section. Therefore, it is not necessary to discuss most of the variable-amplitude parameters here. However, one such parameter, the sequence of individual cycles, does need discussion. The methods of relating variable- and constant-amplitude fatigue behavior, discussed earlier, assumed a random sequence such as occurs normally in practice. However, for certain variable-amplitude spectrums, the sequence of different cycles can have an effect. For example, it has been reported that "the application of a large number of cycles of stress just under the fatigue limit of the member, followed by stresses which are repeatedly increased in small increments, has been found to produce a substantial increase in fatigue resistance."<sup>14</sup> This is called *coaxing*. A second case in which sequence can have an effect involves periodic overloads discussed later in this section.

### Stress Range

Since fatigue is a cyclic phenomenon, the magnitude of the cyclic variation as defined by  $S_r$  is the most important stress parameter. The effect of this parameter on fatigue life is usually represented by an SN curve similar to that shown in Figure 2. A steady-stress parameter ( $S_{min}$  or  $S_{mean}$ ) is normally held constant, while  $S_r$  is varied so that the SN curve is for a particular value of  $S_{min}$  or  $S_{mean}$ . Alternatively,  $S_{max}$  is often varied while  $R$  is held constant to develop an SN curve. But this second approach has the disadvantage of intertwining the effects of the cyclic-variation and steady-stress parameters so that neither is clearly defined.

### Minimum, Maximum and Mean Stress

These steady-stress parameters,  $S_{min}$  or  $S_{mean}$ , have a much smaller effect on fatigue life than the cyclic-variation parameter,  $S_r$ . For fatigue design of bridges these parameters are usually neglected, but in other applications they may have a significant effect.<sup>15</sup> A discussion of these effects is useful to show the interrelationship of several stress parameters.

These effects can be conveniently illustrated by the fatigue charts in Figures 7 and 8. Figure 7 is for as-received plates of A441 steel.<sup>16</sup> Figure 8 is for longitudinal fillet welds, such as flange/web connections of the same steel.<sup>17</sup> The fatigue charts for other details are similar. Two life lines are shown in each figure, one for 100,000 cycles and the other for the fatigue limit. Each point on a life line defines the stress conditions that will cause a fatigue failure in that number of cycles. Two interconnected grid systems are used to define these stress conditions. The horizontal/vertical system defines the algebraic maximum and minimum stresses in the cycle. The diagonal grid system defines the alternating and mean stresses for the cycle. Each line radiating from the origin represents a particular stress ratio. The left diagonal axis, which is the alternating stress axis, represents a stress ratio of  $-1$ . The maximum and

minimum stresses at any point along this line are equal in magnitude but opposite in sign. The right diagonal axis, which is the mean stress axis, represents a stress ratio of  $+1$ . Any point along this line represents the static strength of the detail since the maximum and minimum cyclic stresses are the same. Horizontal lines representing the yield and ultimate tensile stresses, and vertical lines representing the compressive yield stress, are also shown.

The effect of mean stress on the magnitude of the stress range that will cause failure in a given number of cycles is shown by the diagonal grid in Figures 7 and 8. In both figures, the alternating stress (or stress range) decreases as the mean stress increases and eventually becomes zero at the static strength (ultimate tensile stress) of the detail. However, the rate of decrease is greater for the as-received plates than for the fillet welds. In fact, there is very little decrease for the fillet welds until the stress ratio exceeds one fourth. One possible reason for this difference is discussed in the section on residual stresses. Another reason is that crack initiation takes a larger portion of the total life for the as-received plates than for the fillet welds. Crack initiation is affected by mean stress but crack propagation is not.

In the region where mean stress is highest and hence has the greatest effect, the maximum stresses exceed the yield stress.\* This region is of little practical importance because the static rather than fatigue stresses govern practical designs. Fatigue tests of simulated bridge details within a practical range of stress parameters showed little effect of mean stress for most details.<sup>12</sup> Therefore, current structural specifications neglect the effects of mean stress and use stress range as the only stress parameter which greatly simplifies the specifications.<sup>18,19,20</sup>

Fatigue failures do not occur unless a part of the stress cycle is in tension. Therefore, the life lines become asymptotic to the minimum stress axis as shown in Figures 7 and 8. Thus, compressive mean stresses have a large beneficial effect if they are sufficiently great to cause the entire cycle to be compressive. In practice, however, caution should be exercised before dismissing the possibility of fatigue failures in regions of nominal compressive stresses because unexpected tensile stress components often occur as a result of the presence of residual stresses or local triaxial stress conditions.

### Residual Stresses

Residual stresses may have a significant effect on the fatigue life of a structure in low-stress or long-life exposures. They are internal stresses that are in equilibrium within a member and are a result of fabrication and manufacturing processes — such as welding and forming — that cause portions of the member to yield. They can also be caused by local yielding at stress

\*Fatigue stresses above the yield stress can be obtained with plate specimens; strain hardening occurs as required to reach these stresses.

raisers during loading. They remain constant after they are formed unless subsequent higher service loadings cause further local yielding. Therefore, they act as a mean stress superimposed on the applied stresses, and shift the cyclic stresses (Fig. 9). This shift creates the same effect as that caused by an applied mean stress of the same magnitude, provided the combined residual and applied stresses are equal to or less than the yield stress throughout the member.

Residual stresses have a large beneficial effect if they shift the cyclic stresses so that all, or almost all, of the cycle is in the compression region as shown by the first (starting from the left) example in Figure 9. The residual stresses can have a detrimental effect if they shift the cyclic stresses from the compression region into the tension region as shown by the second example. The residual stresses can also have a detrimental effect if they shift the cyclic stresses upward within the tension region, and a beneficial effect if they shift the cyclic stresses downward within this region. Residual stresses that merely shift the cyclic stresses upward or downward within the compression region have no effect because fatigue crack propagation does not occur under these conditions. These shifts within either the tension or compression regions are illustrated by the third and fourth examples in Figure 9.

Shot-peening, cold rolling, and surface hardening are processes designed to create high compressive residual stresses<sup>4,15</sup> in a thin surface layer, and very low balancing tensile residual stresses through the rest of the thickness. These compressive residual stresses are often large enough to shift the applied cyclic stresses into the compression region on the surface of the member. Since fatigue cracks usually initiate from the surface, peening, cold rolling, and surface hardening can greatly improve fatigue life<sup>4,15</sup> in some applications.

Welding or gas cutting a plate usually produces high longitudinal (in the direction of the weld or cut) tensile residual stress in a small area around the weld or cut, and smaller balancing compressive stresses elsewhere in the plate.<sup>21</sup> The compressive stresses will improve the fatigue life if they occur at the location where cracks initiate and if they are large enough to shift the cyclic stresses into the compression region. An example of such a beneficial effect occurred in a study of simulated bridge members under variable-amplitude loading.<sup>8</sup> Beam specimens with partial-length cover plates were fabricated first by welding the cover plates to the flange plates, followed by welding the flange plates to a web. The flange/web welds caused compressive residual stresses at the ends of the longitudinal cover-plate/flange welds where fatigue cracks usually initiate. Similar beams were fabricated by placing the cover-plate/flange welds last so that compressive residual stresses did not develop at their ends. At low applied stresses the first set of specimens had longer fatigue lives than the second set, but at higher stresses there was little difference.

The high tensile residual stresses in the weld and adjacent areas can cause fatigue cracking when the

applied cyclic stresses are in the compression region. This may not be seriously detrimental because the cracks usually do not propagate outside the region of tensile residual stresses. However, caution should be used before dismissing such cracks as unimportant because, at times, they do propagate through other regions where the residual stresses are thought to be compressive.<sup>8,12</sup> Presumably, this occurs because there is an unexpected tensile stress component in some direction as the result of triaxial applied or residual stresses. Such triaxial stresses occur around notches and point loads.

As a result of the yielding behavior illustrated in Figure 10, the actual cyclic stresses in the regions of high tensile residual stresses usually vary from a maximum equal to the yield stress to a minimum set by the applied stress range (unless the applied mean stress is compressive and greater than the amplitude of the applied cyclic stresses). The applied cyclic stresses are shifted so that the maximum cyclic stresses equal the yield stress as illustrated by the last two examples in Figure 9. This means that the magnitude of the applied mean stress has no effect on the actual cyclic stresses near the weld unless it is compressive and greater than the amplitude of the applied cyclic stresses. Consequently, the applied mean stress has little effect on the fatigue life of longitudinal fillet weld specimens.

#### Multi-axial Stresses

Cyclic bi-axial and tri-axial stress variations, such as occur for combined bending and shear in a beam web, can be conveniently represented by an equivalent uni-axial cyclic stress.\* This stress can be calculated by one of the following *theories of failure* used to predict yielding or fracture under static multi-axial stress conditions: 1) maximum principal stress theory, 2) maximum shear stress theory, or 3) von Mises distortion energy theory. Of these, the von Mises theory provided the best results for unnotched specimens.<sup>15</sup> According to this theory, the failure parameter is:

$$S_e = \sqrt{0.5 [(S_1 - S_2)^2 + (S_2 - S_3)^2 + (S_3 - S_1)^2]} \quad (5)$$

in which  $S_1$ ,  $S_2$ , and  $S_3$  are the principal stresses. For fatigue applications these principal stresses could represent any of the stress parameters ( $S_r$ ,  $S_a$ ,  $S_{min}$ , etc.) used to define a stress cycle. (For bi-axial stresses Equation 5 reduces to

$$S_e = \sqrt{S_1^2 + S_2^2 - S_1 S_2} \quad (6)$$

For combined axial stress and shear such as occurs in the web of a beam

$$S_e = \sqrt{S^2 + 3\tau^2} \quad (7)$$

in which  $\tau$  is the shear stress.)

\*Ordinarily, bi-axial or tri-axial stress conditions need not be considered in fatigue design for highway bridges.<sup>20</sup>

The maximum principal stress theory gives results that are not much different from those of the von Mises theory for bi-axial principal stresses of the same sign, but gives considerably higher equivalent stresses for principal stresses of opposite signs. For details involving attachments or similar stress raisers, the tensile stress component perpendicular to the attachment appears to be the most reasonable combined stress parameter.

#### Periodic Overloads

It has been shown that a few periodic overloads that greatly exceed the level of the predominant cyclic loads can cause large increases in fatigue life.<sup>15,22</sup> This is particularly true if the specimen or detail includes a stress raiser. The tensile overload causes yielding at the stress raiser and leaves compressive residual stresses after it is removed.<sup>15</sup> This compressive residual stress improves the fatigue life as discussed previously. Once a crack has initiated, the overload can cause compressive residual stresses at the crack tip that retard crack propagation.

The effect of overloads on the fatigue life of an A514 steel beam with stiffeners<sup>22</sup> is summarized in Table I. Sets of two identical beams were tested under the same cyclic loading, but one beam was overloaded and the other was not. All except one of the overloaded beams were runouts (no failure), while the other beams had an average life of 635,000 cycles.

#### DETAIL GEOMETRY

The geometry of a detail has a major influence on fatigue behavior. Notches, holes, protrusions, attachments, and other sharp changes in geometry disturb the smooth flow of the stresses and cause local areas of high stress that reduce fatigue life. The main geometric parameters that control the amount of the reduction are discussed in this section.

##### Notches, Holes, and Protrusions

The stress concentrations caused by notches, holes, and protrusions reduce fatigue strength by a factor  $K_f$ , that is analogous to the theoretical stress concentration factor,  $K_t$ .<sup>4,15</sup> Specifically,  $K_f$  is defined as the ratio of the fatigue strength corresponding to a given number of cycles for an unnotched specimen to the fatigue strength corresponding to the same number of cycles for a notched specimen of the same material.  $K_f$  is called the fatigue-strength reduction factor.

$K_f$  depends on 1) the theoretical stress concentration factor, 2) the stress gradient or amount of material subjected to the peak stress, 3) the number of cycles, and 4) the material.<sup>4,15</sup>  $K_f$  is much less than  $K_t$  for sharp notches, and similar abrupt changes in geometry, because the high stresses caused by such notches decrease rapidly and affect only a small amount of material around the notch. For example, a sharp notch may have a  $K_t$  above 10 but a  $K_f$  below 1.5. For less abrupt changes in geometry,  $K_f$  is much closer to  $K_t$ . For example,  $K_f$  can approach the  $K_t$  value of 3.0 for a large circular hole. For a small circular hole,  $K_t$  is still 3.0 but  $K_f$  may be considerably less because less ma-

terial is affected when the hole is small.

For a given notch, hole or protrusion,  $K_f$  is usually greatest at long lives (large number of cycles). Stress concentrations have a smaller effect at shorter lives because the peak stresses are reduced by yielding.<sup>15</sup> The closer the nominal stresses are to the yield stress, the smaller the effect of a stress concentration on fatigue strength. Therefore, the SN curve for a notched specimen approaches that for a smooth specimen as the life decreases.<sup>4</sup> The effect of the type of steel on  $K_f$  is discussed later under the heading of Notch Sensitivity.

#### Attachments

Any plate welded, bolted or riveted to a longer main plate or beam will cause stress concentrations when the main plate or beam is loaded. Such attachments are illustrated in Figure 11. If the attachment is oriented parallel with the stress in the main plate, the stress concentrations occur at the ends of the attachment. If the attachment is oriented perpendicular to this stress, the stress concentrations occur along the transverse edge of the attachment. These stress concentrations reduce the fatigue strength of the main plate by an amount that depends on length (or width) of the attachment in the direction of the stress.<sup>23,24,25</sup> The length is important because it affects the amount of stress that is transferred from the main plate into the attachment. At a certain length the stress in the attachment becomes equal to the stress in the main plate and a further increase in length has no effect. The detrimental effect of longitudinal attachments on fatigue life can be reduced by using a concave transition radius at the ends and properly grinding the welds at those locations.<sup>23</sup>

#### Surface Conditions

Surface roughness causes stress concentrations that control the fatigue strength of plates or shapes with mill, weathered, or flame-cut surfaces. As a result, the fatigue strength of such plates is less than that of plates with machined and polished surfaces. The scatter in fatigue results also tends to be greater for such plates and shapes than for welded details.

Fatigue tests of weathering steels, such as ASTM A588 steel, after atmospheric exposure times of up to 11 years, show that the weathering reduces the mean fatigue strength compared with that of an as-received mill surface.<sup>33</sup> This reduction in mean fatigue strength is caused by the increased surface roughness that occurs during the first 2 or 3 years of exposure. The surface roughness stabilizes after this time interval and the resulting stress concentration is less severe than that of the surface roughness caused by good-quality flame cutting.<sup>26</sup> Thus, the fatigue strength of weathered plates is above that of flame-cut surfaces.<sup>26</sup> Weathering has a lesser effect on welded details than on plain plates because the fatigue strength of such details is usually controlled by imperfections that are more severe than the surface roughness caused by weathering.<sup>26</sup> Thus, it can be concluded that the present AASHTO fatigue-design curves<sup>18</sup> are equally applicable to predicting the fatigue behavior of both weath-

ered and un-weathered bridge-steel components.<sup>26</sup>

The fatigue strength of plates and details can be improved by various surface treatments including 1) machining, grinding, and polishing, 2) electroplating, 3) shot peening, 4) cold-rolling, 5) surface hardening by flame or induction heating, and 6) carburizing or nitriding.<sup>15</sup> The first two methods reduce surface roughness and the last four induce compressive surface stresses.

## MATERIAL

### Tensile Strength

The basic fatigue strength of various steels as determined from polished specimens depends on their ultimate tensile strength. For completely reversed loading ( $R = -1$ ) the alternating-stress (one-half the applied stress range) fatigue limit is generally about one-half of the tensile strength if determined from plane or rotating beam specimens, and about one-third of the tensile strength if determined from axially loaded specimens.<sup>14,27</sup> The bending specimens have a higher fatigue limit than for plates with polished surfaces, because of their stress gradient. The effect of tensile strength decreases with increasing severity of details. For plates with mill surfaces, the effects of tensile strength are considerably smaller than for plates with polished surfaces, but are still significant, especially at shorter lives where the fatigue strength approaches the tensile strength.<sup>28</sup> For groove-welded plates, the effect is small<sup>28</sup> and for more severe details it is negligible except at very short lives. Fatigue tests of welded simulated bridge details within a practical range of stress parameters showed little effect of type (strength) of steel.<sup>12</sup> Therefore, current structural specifications<sup>18,19,20</sup> give the same allowable fatigue stresses for all steels.

### Yield Strength

The basic fatigue strength of a steel, as measured by polished specimen tests, does not appear to be directly related to its yield strength. However, there is an indirect relationship between the two, because yield strength tends to increase with tensile strength, which is related to fatigue strength as discussed earlier. Moreover, yield strength can have an influence on the effects of stress raisers, residual stresses, and mean stress on fatigue behavior as discussed earlier. Generally, yielding tends to reduce the detrimental effects of these factors. Consequently, a lower yield strength can sometimes be beneficial.

### Fracture Toughness

The basic fatigue strength of a steel as measured by polished-specimen fatigue tests is not directly related to its fracture toughness as measured by Charpy V-Notch or  $K_{Ic}$  tests.

The possibility that a reduction in the fatigue life of an actual bridge would be caused by low fracture toughness is small. Past studies<sup>29</sup> have shown that unless fracture toughness is very low a large crack is required to cause a fracture even if the bridge is subjected to a severe combination of low temperature and

high traffic load. The remaining fatigue life with such a crack present is small since most of the fatigue life is expended when the fatigue crack is small. Consequently, use of a steel with a higher fracture toughness would have little effect on the service life of the bridge.

### Notch Sensitivity

As mentioned earlier, the effect of a stress concentration on fatigue strength depends on the material. The notch sensitivity index,  $q$ , is frequently used to define this dependence.<sup>4,15</sup> It is determined from the equation

$$q = \frac{K_f - 1}{K_t - 1} \quad (8)$$

by testing notched and unnotched fatigue specimens of the same material to get  $K_f$ . Unfortunately, there is no standard specimen nor test procedure for determining  $q$ . Therefore, it can be determined from any type of notched specimen and the values obtained from different specimens can be quite different. The value of  $q$  also varies with the number of cycles at which  $K_f$  is defined. Nevertheless, the notch sensitivity index provides a useful indication of the fatigue behavior of various steels when stress raisers are present, especially if the same type of specimen is used to evaluate all steels.

The value of  $q$  varies from 0 to 1 as  $K_f$  approaches  $K_t$ , that is, as the effect of the stress raiser on the fatigue strength increases. Structural steels have a low notch sensitivity and ultra-high-strength steels have a high notch sensitivity.

## ENVIRONMENT

Corrosion is the process of oxidation of surface layers by the environment. If corrosion is permitted to cause a sufficient reduction in the cross-sectional area of a member, it will increase fatigue stresses and thereby reduce fatigue life.

Stress corrosion cracking is cracking of a steel caused by the environment when static tensile stress is present. If such cracking occurred it would reduce the fatigue life under subsequent cyclic loading. However, available data show that distilled water or salt water do not cause such cracking in bridge steels.<sup>30</sup>

### Corrosion Fatigue

Corrosion fatigue refers to the combined action of corrosion and cyclic loading, which produces detrimental effects greater than either acting alone. The weathering of steel surfaces, which was discussed earlier, does not involve such combined action and, therefore, is not considered to be corrosion fatigue. Although many substances can cause corrosion fatigue, the ones of most interest for structural applications involve combinations of water and salt (sodium chloride). De-icing practices and ocean spray are common sources of such environments in bridges.

Because of the large number of variables that affect corrosion-fatigue behavior and the complex interaction of these variables, it is very difficult to provide quantitative apriori predictions of corrosion-fatigue behaviors for various materials and environments. Such

behaviors must be established by specimen testing of the material and environment of interest. Such tests have been conducted for bridge steels in distilled water and 3.5-percent sodium-chloride solution.<sup>30</sup> The data show little effect of these environments on the corrosion-fatigue crack-propagation behavior for these steels and that the corrosion-fatigue behavior for fabricated bridge components should be essentially the same as their fatigue behavior.<sup>30</sup>

#### **Fretting Fatigue**

Fretting fatigue can occur when two metal surfaces in contact are subjected to small repetitive sliding movements. These movements cause the initiation of surface cracks that eventually may grow into ordinary fatigue cracks. Fretting can greatly reduce the fatigue life of a member or cause a fatigue failure that would not otherwise occur. The mechanism of fretting is very complex and apparently involves both mechanical and chemical action.<sup>4,15</sup> Fretting produces a powder consisting of oxides of the metals in contact; for steels in contact, the powder is rust. The powder provides a warning that fretting is occurring.

There are insufficient quantitative data available on fretting to permit accurate fatigue design calculations. However, the effects of some important factors are known.<sup>4,15</sup> As the contact pressure increases, the fretting fatigue life decreases to a minimum value, and remains close to that minimum value as the pressure becomes high enough to prevent sliding movements.<sup>4,15</sup> Corrosive environments tend to reduce fretting life. High-hardness, higher-strength steels appear to be more susceptible to fretting than lower-strength structural steels.

Fretting can be prevented or minimized by 1) inserting a soft material (such as wood, brass, or copper)

between the contact surfaces, 2) applying a surface treatment to induce compressive residual stresses in the contact surfaces, 3) preventing relative movement by high contact pressure, keyways, adhesion, or other devices, and 4) reducing the cyclic stresses that propagate the fretting cracks. Lubrication by oils or greases normally provide only a small improvement.<sup>4</sup>

#### **FABRICATED MEMBERS**

Fabricated members utilize various types of structural details that can be categorized according to the severity of the combination of factors that affect the fatigue behavior of such details. The main factors are the detail geometry and the initial imperfections and residual stresses that are likely to be present. The effect that the combination of factors has on each type of detail can be determined by fatigue tests on that type. Different types of details with approximately the same fatigue strengths can be conveniently included in one fatigue category.

Variations in the geometry, imperfections, and residual stress that occur within each type of detail cause scatter in the fatigue results for that type. Nevertheless, fatigue tests of typical details provide a more accurate and convenient method of determining the fatigue behavior of fabricated members than trying to assess the effect of each pertinent factor (geometry, imperfections, etc.) individually from small specimen data. Therefore, current structural design specifications<sup>18,19,20</sup> utilize this detail-category approach. The application of the approach to highway bridges, and the basis for the AASHTO specifications<sup>18</sup> that cover such bridges, will be discussed in detail in the final section of the fatigue chapter of the *U.S. Steel Highway Structures Design Handbook*.

## **4. SUMMARY**

Fatigue failures are caused by the initiation and/or growth of cracks when cyclic stresses are applied. Fatigue behavior can be conveniently defined by an SN curve consisting of two straight lines on a log-log plot: a horizontal line representing the fatigue limit and a sloping line representing finite-life behavior. Considerable scatter of fatigue results typically occurs. Variable-amplitude fatigue behavior can be conveniently related to constant-amplitude behavior by an effective stress parameter and/or an equivalent number of cycles.

Stress spectrum and detail type are the main parameters affecting fatigue life. Stress range is the main stress parameter; minimum, mean, or maximum stress

has a much smaller effect. Residual stresses have a large effect only if they shift the applied cyclic stresses from a compression region into a tension region or vice versa. A small number of periodic overloads can have a beneficial effect on fatigue behavior. The effect of stress raisers on fatigue behavior depends on the theoretical stress concentration factor, the stress gradient, the number of cycles, and the notch sensitivity of the material. Type of steel has only a secondary influence on fatigue behavior of most structural details. Under certain conditions corrosion fatigue or fretting fatigue can cause large detrimental effects.

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## 6. TABLE

**TABLE I**  
**Fatigue Behavior Under Periodic Overload**

Test Set	Overload	Cycles to Failure
1	none	522,000
	periodic	1,213,000 NF
2	none	570,000
	periodic	4,900,000 NF
3	none	727,000
	periodic	2,648,000 NF
4	none	634,000
	1st cycle	1,240,000
5	none	723,000
	1st cycle	4,200,000 NF

- 1) cyclic stresses: max. 35 ksi; min. 2 ksi
- 2) overload stress: 100 ksi
- 3) frequency of periodic overloads: every 100,000 cycles
- 4) specimen: A514 steel beam with stiffener
- 5) source: reference 29
- 6) NF indicates no failure

## 7. FIGURES

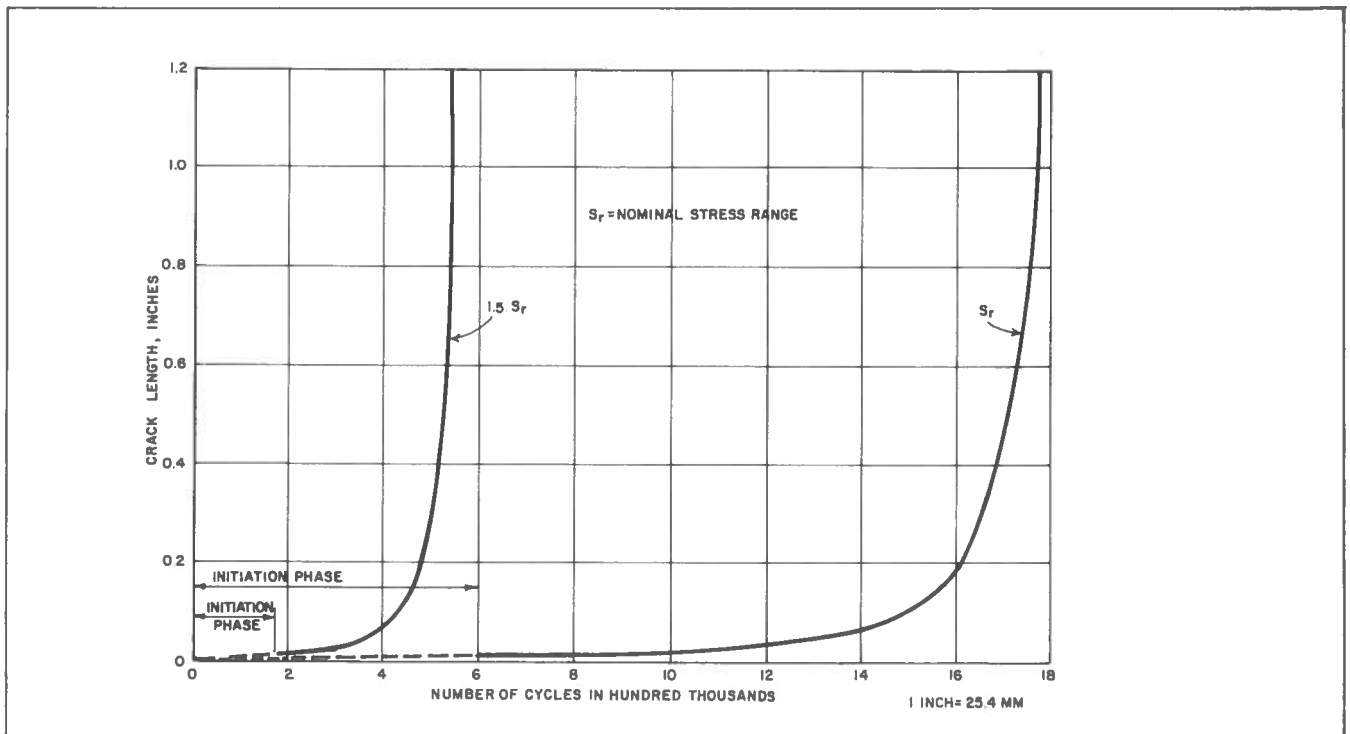


Figure 1. Typical crack-growth curves.

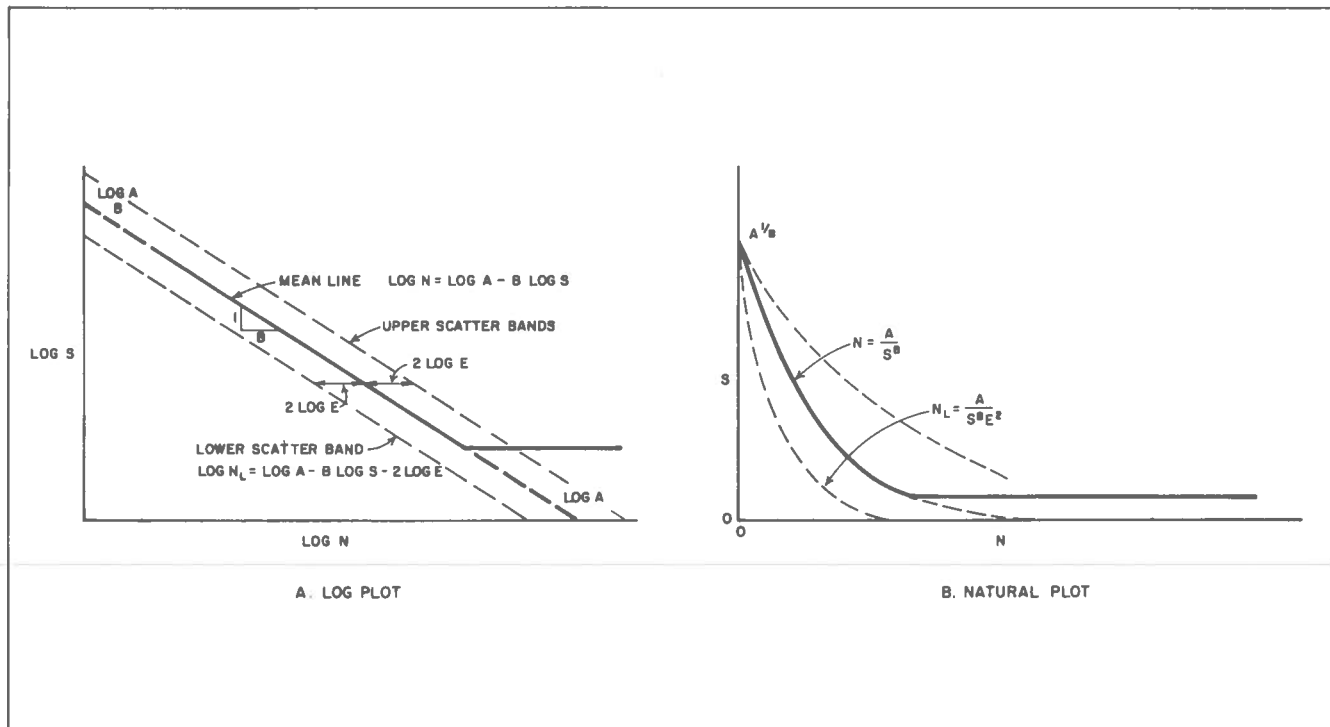


Figure 2. SN relationships.

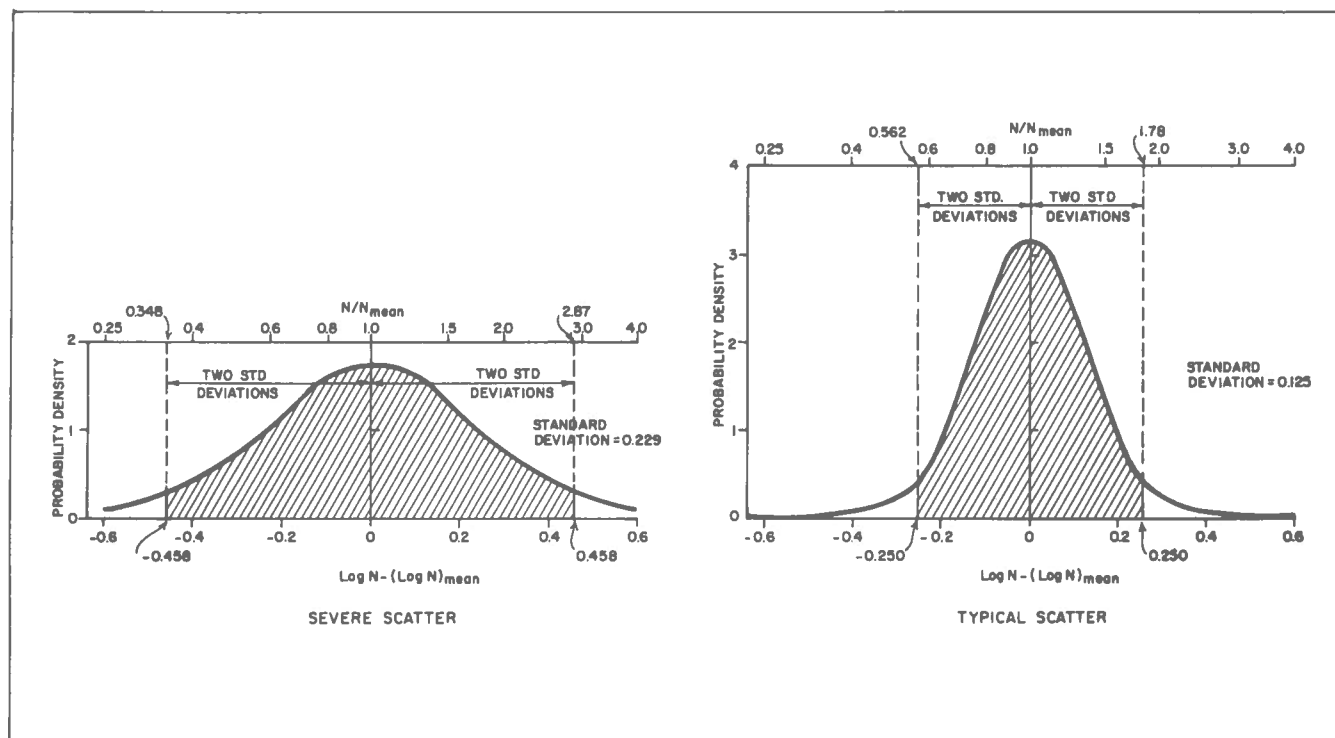


Figure 3. Scatter in fatigue data for bridge details.



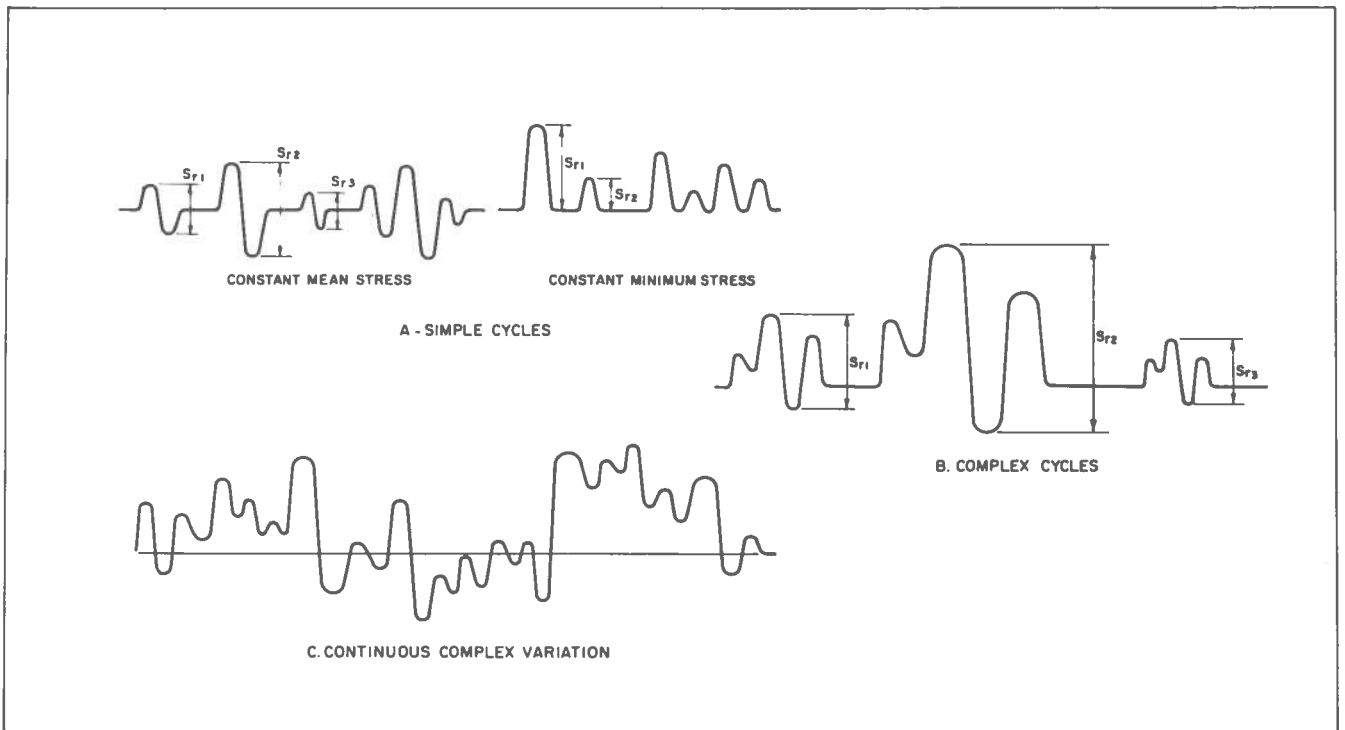


Figure 4. Types of variable-amplitude cycles.

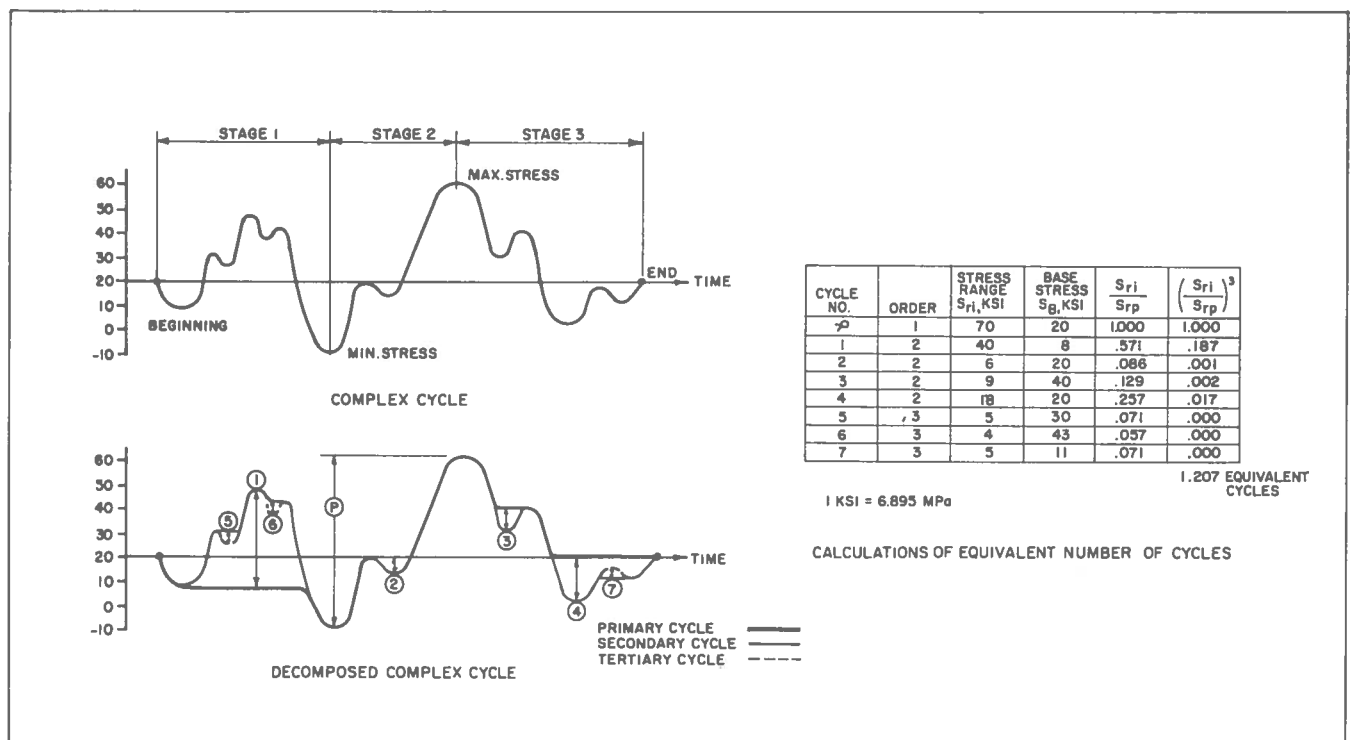


Figure 5. Calculations of equivalent number of cycles.

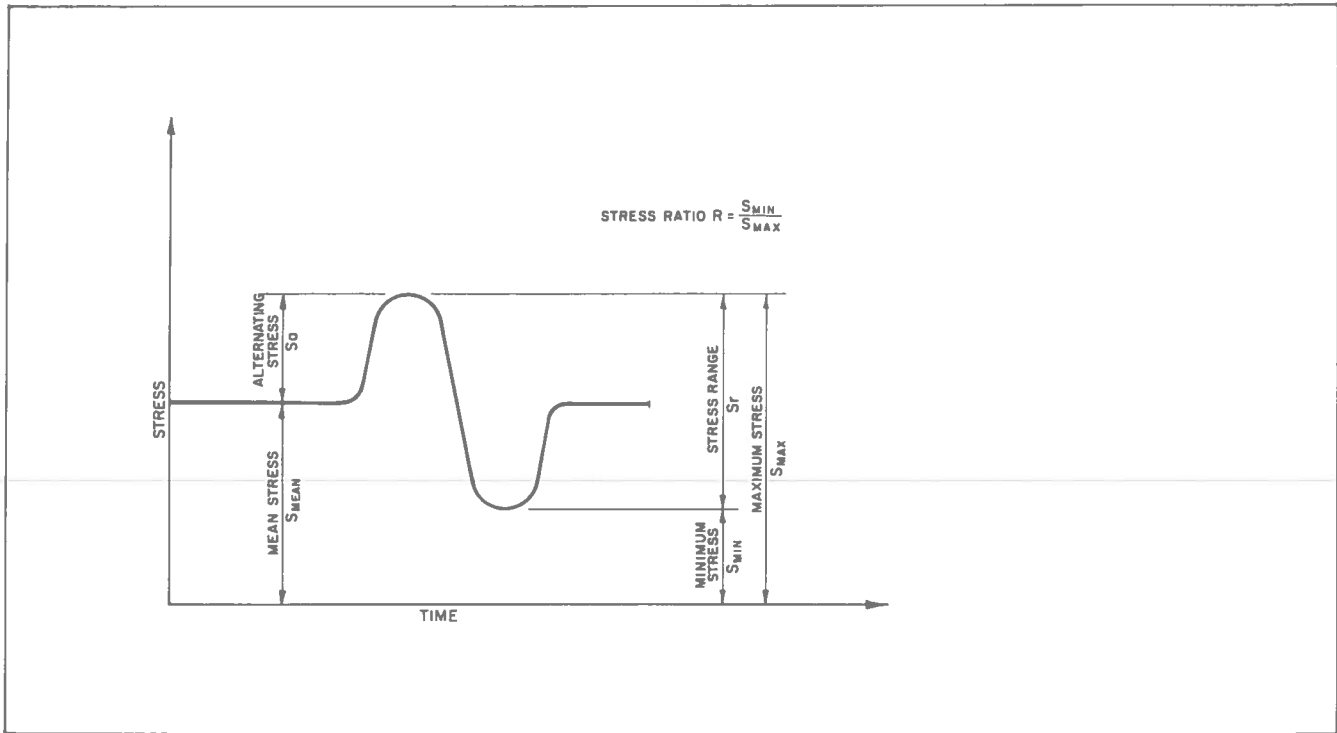


Figure 6. Parameters defining an individual stress cycle.

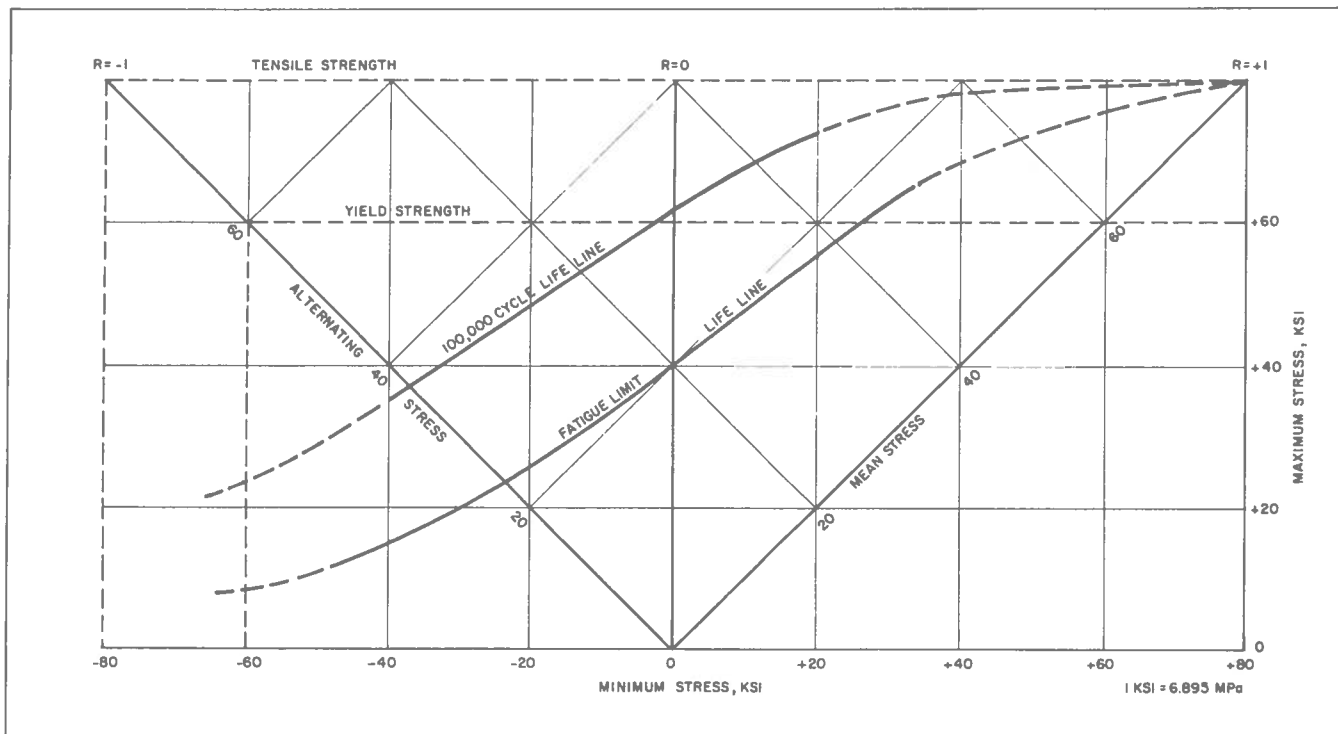


Figure 7. Fatigue chart for as-received plates of A441 steel.

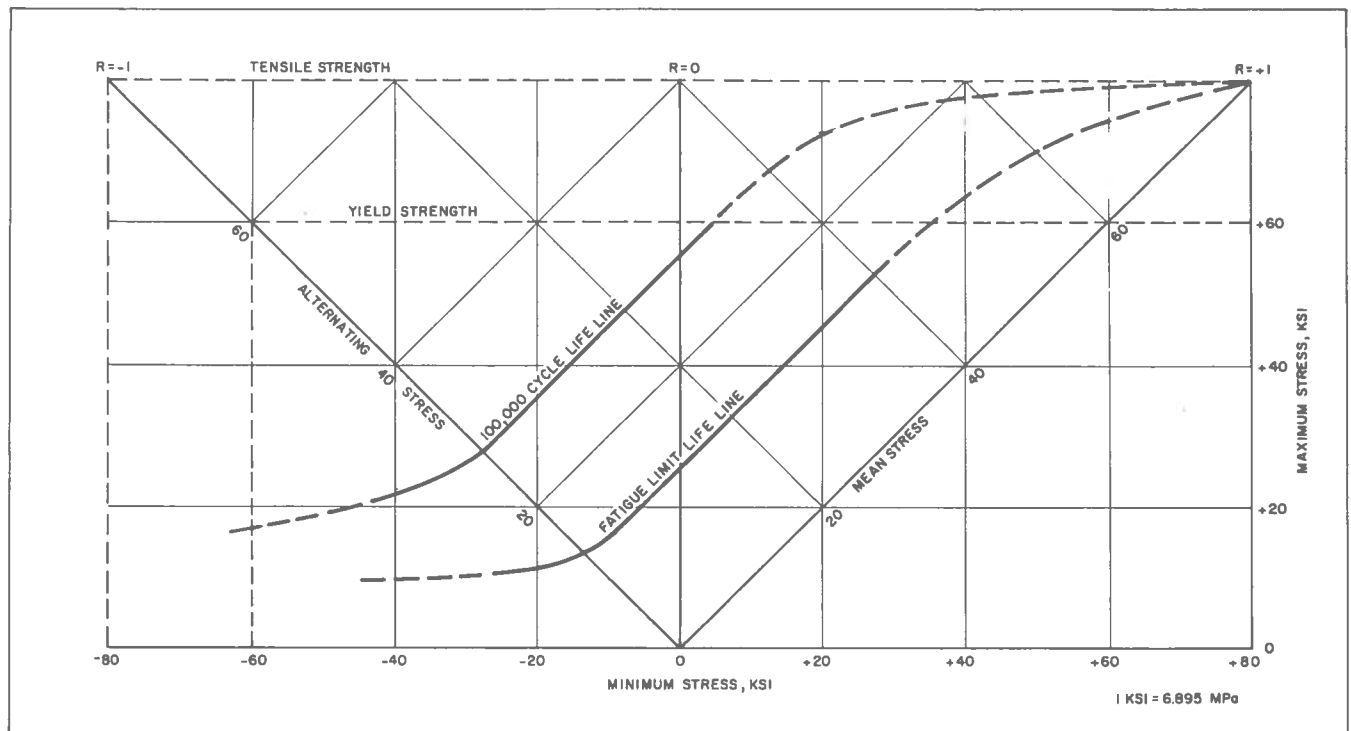


Figure 8. Fatigue chart for longitudinal fillet welds on A441 steel plate.

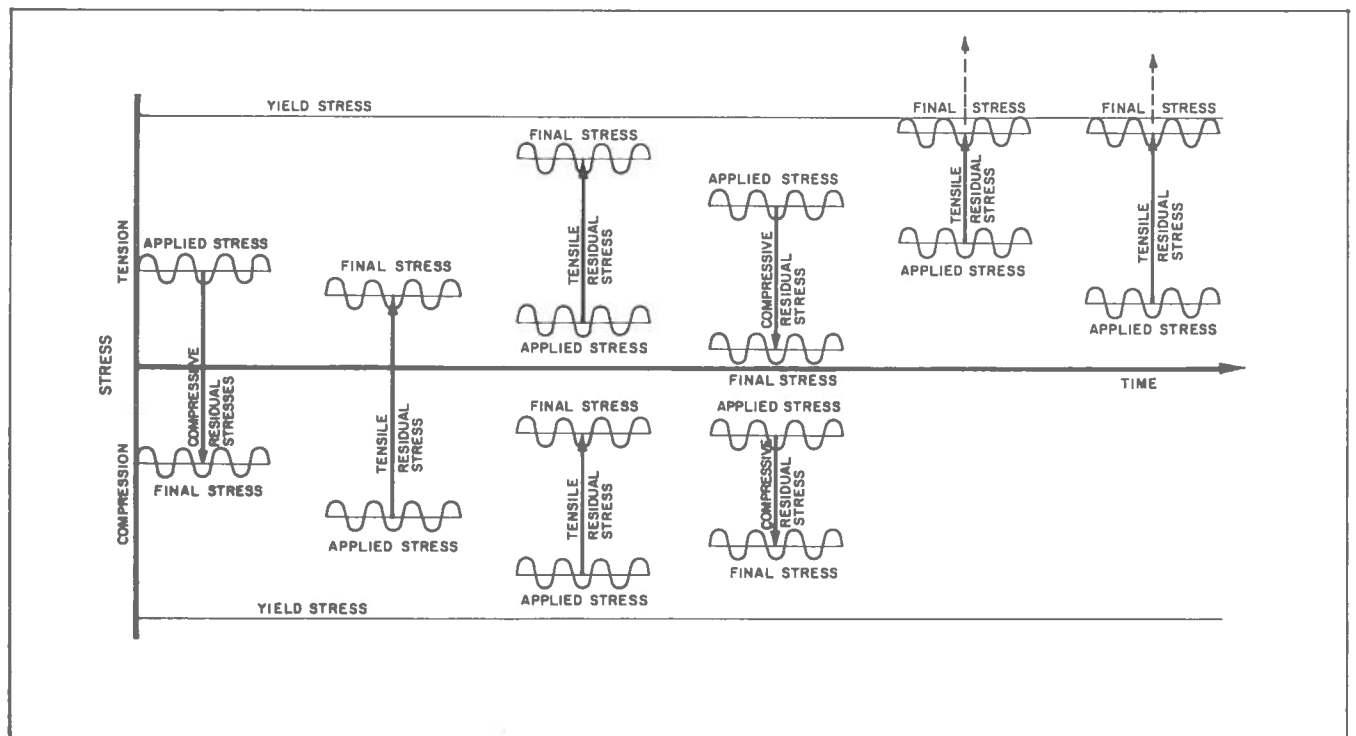


Figure 9. Shifts in cyclic stresses caused by residual stresses.

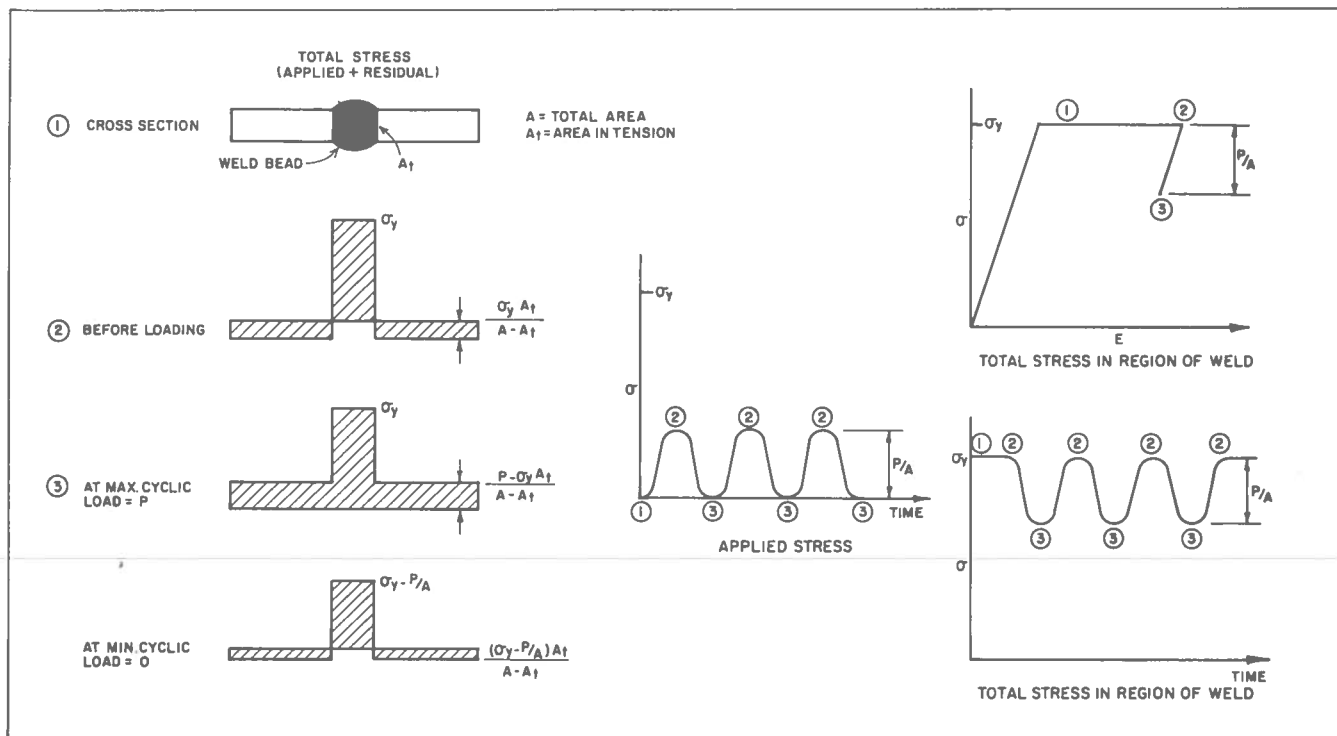


Figure 10. Shift in cyclic stresses due to tensile residual stresses in weld.

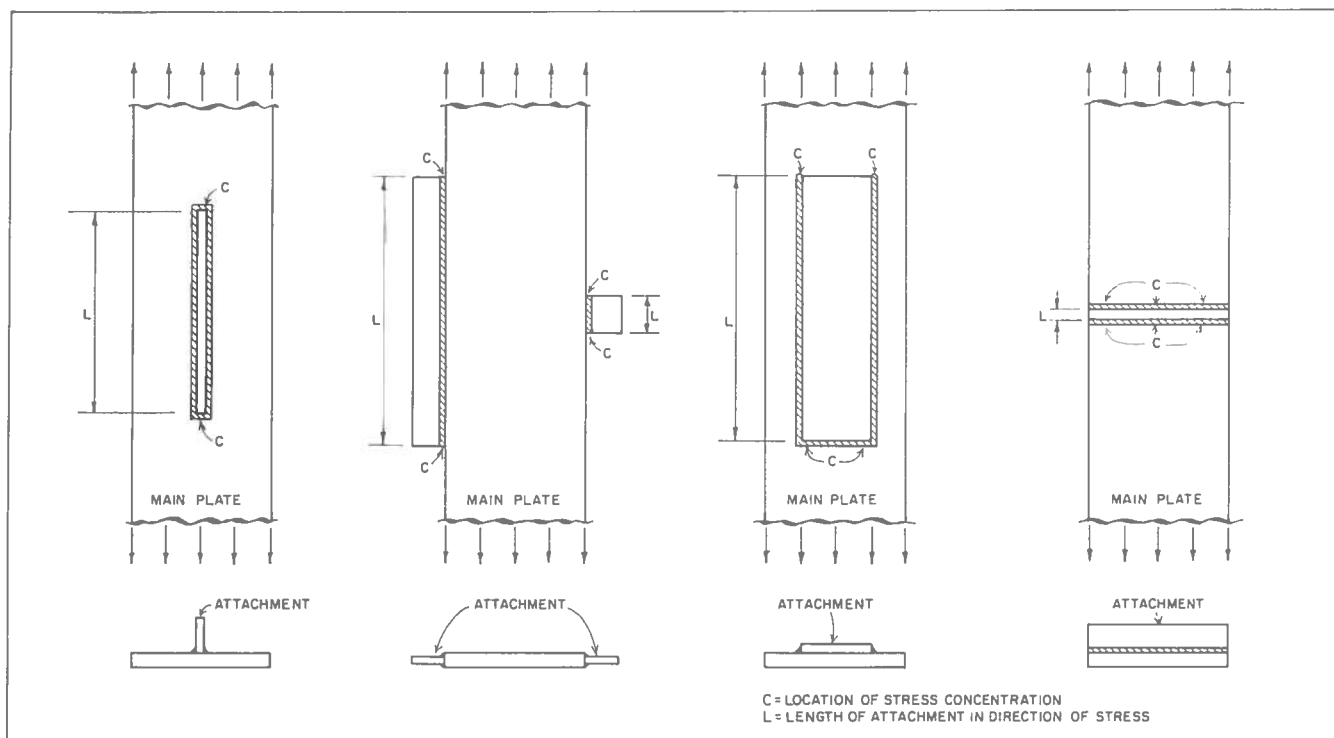


Figure 11. Typical attachments.

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## ERRATA

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**Highway Structures Design Handbook**  
Vol. I, Chap. 6 FATIGUE, Section IV Fatigue Design.

(Insert this page immediately preceding Section IV of this chapter.)

**Page I/6.79:** Equation (2), in the left hand column should read

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

In the sentence immediately following this equation,  
tabular reference should read  
Table IV

**Page I/6.101:** The equation shown in Figure 3 (upper right) should read

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

**Page I/6.103:** Caption under upper left-hand detail of Figure 7 should read

BENDING APPLIED TO WEB

**Page I/6.105:** On Figure 11, upper right-hand detail of longitudinal stiffener splices should note a 6" radius on the stiffener. Stiffener shown in Detail 13 below should also indicate a 6" radius.



## SECTION IV

# Fatigue Design

## CONTENTS

<b>1. INTRODUCTION</b>	<b>6.70</b>	—Details Not Covered by AASHTO	<b>6.82</b>
—Design Approaches	6.70	a) Cope Holes	6.82
<b>2. EXPERIMENTAL BASIS FOR AASHTO DETAIL CATEGORIES</b>	<b>6.71</b>	b) Cruciform Joints	6.83
—Category A	6.71	c) Bolted Attachments	6.83
—Category B	6.72	—Secondary Bending	6.83
—Category C	6.73	a) Partial End Fixity	6.84
—Category D	6.74	b) Member Distortions	6.84
—Category E	6.74	—Good and Bad Designs for Common Details	6.85
—Category E'	6.75	a) Stiffeners	6.85
—Category F	6.75	b) Splices	6.87
<b>3. BRIDGE DESIGN AND RATING METHODS</b>	<b>6.76</b>	c) Cover Plates	6.87
—Present AASHTO Methods		d) Diaphragms, Simple Cross Beams, and Brackets	6.87
a) Allowable Stress Range	6.76	e) Cross Frames	6.88
b) Calculated Stress Range	6.76	f) Lateral Bracing	6.88
c) Design-Life Categories	6.77	g) Continuous Cross Beams	6.89
d) Rating of Bridges	6.77	h) Trusses	6.90
e) Comparison with Actual Behavior	6.77	—Treatments That Improve Fatigue Performance	6.90
—Proposed New Methods	6.78	a) Peening	6.90
a) Fatigue-Design Truck	6.78	b) TIG Remelting	6.90
b) Design Stress Range	6.78	c) Grinding	6.90
c) Allowable Stress Range	6.79	d) Spot Heating	6.91
d) Design Life	6.79	—Repair of Fatigue Cracks	6.91
e) Rating	6.80	a) Grinding	6.91
<b>4. DESIGN AND FABRICATION PRINCIPLES</b>	<b>6.80</b>	b) Peening	6.91
—Fabrication Practice Related to Fatigue of Non-Fracture-Critical Members	6.80	c) TIG Remelting	6.91
a) Plate Surfaces and Edges	6.80	d) Rewelding	6.91
b) Groove Welds	6.81	e) Drilling Holes	6.91
c) Prohibited Welds	6.81	f) Replacing Rivets	6.92
d) Workmanship	6.81	<b>5. SUMMARY</b>	<b>6.92</b>
e) Bolted Joints	6.82	<b>6. REFERENCES</b>	<b>6.92</b>
f) Contract Plans	6.82	<b>7. TABLES</b>	<b>6.95</b>
—Fabrication Practice Related to Fatigue of Fracture-Critical Members	6.82	<b>8. FIGURES</b>	<b>6.100</b>
		<b>9. APPENDIX</b>	<b>6.110</b>

# 1. INTRODUCTION

This section briefly discusses several approaches currently used to design against fatigue, and indicates why the stress-life detail-category approach is preferred for bridge design. Also described are current AASHTO procedures for design and rating of highway bridges, that are based on the stress-life detail-category approach, and the experimental basis for the AASHTO detail categories. The section also presents a proposed new method that more realistically approximates conditions in actual highway bridges.

The second part of this section discusses fabrication practices related to fatigue and good and bad designs for critical details such as longitudinal-stiffener ends, cover-plate ends, splices, and connections. The effects of secondary bending in initiating fatigue cracks is explained. Methods of improving the fatigue performance of new members and of repairing fatigue damage to existing members are described. Such methods as peening, grinding, preloading, TIG remelting, crack-arrest holes, and rewelding are included.

## DESIGN APPROACHES

Four fatigue-design approaches are currently in widespread use: 1) the fracture-mechanics approach that is used in a variety of applications, 2) the strain-life approach, widely used in automotive applications, 3) the stress-life reduction-factor approach, widely used in machine design, and 4) the stress-life detail-category approach, generally used for structural applications, such as bridges and buildings.

In the fracture-mechanics approach, the fatigue life of a member is predicted from crack-initiation and crack-propagation data obtained in tests on small specimens.<sup>1</sup> The stress conditions in a small region at the tip of a sharp crack, which change as the crack grows, control fatigue behavior. Several critical assumptions must be made in calculating these stress conditions in fabricated members: 1) the size of any crack, or crack-like imperfection, present before loading, 2) the effect of residual stresses at various stages in the crack growth, and 3) the exact stress concentration factor at various points in the region of the crack due to detail geometry.<sup>2,3</sup> Because of these assumptions, and other uncertainties pertaining to the basic initiation and propagation data, it is difficult to predict accurately the fatigue behavior of fabricated members by the fracture-mechanics approach.<sup>2,3</sup>

In the strain-life approach, the fatigue life of a member is predicted from strain vs life data obtained in fatigue tests of small unnotched specimens by relating these data to the cyclic plastic strains at a critical location (usually at a notch) in the member.<sup>4,5</sup> Strain, rather than stress, is used as the design parameter because the fatigue strain at the critical location (notch) often exceeds the yield strain even though the nominal fatigue stresses are well below the yield stress. Laborious calculations, or direct measurements, are required to trace the variation of strain at a notch subjected to complex variable-amplitude loadings. Furthermore, assumptions and uncertainties similar to those mentioned with regard to the fracture-mechanics approach occur in determining the strain variation and predicting the fatigue life.<sup>4,5</sup> Therefore, it is difficult to predict accurately the fatigue behavior of fabricated members by the strain-life approach.

In the stress-life reduction-factor approach, a fatigue-strength reduction factor is applied to stress-life unnotched small-specimen data to predict the life of a notched specimen or member.<sup>6,7</sup> This approach uses stresses to define fatigue behavior even when yielding has occurred at the notch. The reduction factor varies with the size and type of notch, number of cycles, mean stress level, and material.<sup>6,7</sup> Therefore, selection of the proper reduction factor is critical in accurately predicting the fatigue behavior of a fabricated member by this method.

In the stress-life detail-categories approach, typical structural details are grouped according to severity, and allowable stresses corresponding to various design lives are given for each group. The allowable stresses for each group are obtained by performing fatigue tests on specimens that simulate each type of detail in the group. Thus, the stress concentrations, residual stresses, initial flaws, and material variability (especially near welds) that cause uncertainties in the first three approaches are included directly in the tests. Of course, these quantities vary among individual specimens contributing to the scatter of results. However, this scatter is considered in the design process by using the lower limit of the scatter band. Consequently, the fourth approach is more convenient than the first three for predicting the fatigue behavior of fabricated members and is preferred for structural applications.<sup>8,9,10</sup>



## 2. EXPERIMENTAL BASIS FOR AASHTO DETAIL CATEGORIES

The fatigue provisions of the present AASHTO specifications<sup>8</sup> are based on the stress-life detail-categories design approach. The experimental basis for the selection of detail categories and the assignment of allowable stresses to each category are described in the following paragraphs. The allowable stresses are based primarily on an extensive program of constant-amplitude fatigue tests of simulated bridge members conducted at Lehigh University<sup>11</sup> through<sup>17</sup> and are supported by variable amplitude tests conducted at the U.S. Steel Corporation Research Laboratory<sup>2</sup> and by earlier constant-amplitude tests conducted at the University of Illinois,<sup>18,19</sup> U.S. Steel Corporation,<sup>19,20</sup> and elsewhere.<sup>19</sup>

AASHTO has classified bridge members into seven detail categories as described in Table I<sup>8</sup> and illustrated in Figure 1. The fatigue behavior for each category is conservatively represented by the corresponding SN curve in Figure 2. The stress parameter for these curves is stress range; the effects of minimum or mean stress were shown to be small and are neglected.<sup>11,16</sup> Tests of simulated bridge members<sup>2,11</sup> showed that the type (strength) of steel has little effect on fatigue behavior within a practical range of stress parameters. Therefore, the SN curves in Figure 2 apply to all structural steels used in bridges.

The sloping portions of six of the SN curves are parallel and spaced approximately equally. The reciprocal of the slope for these curves, and the stress exponent in the exponential SN relationship given in Equation 2 of Section III, is 3.0. Tests have shown that this value is appropriate for most fabricated members.<sup>2,11</sup> The seventh SN curve, which applies only for shear stress on the throat of a fillet weld, has a corresponding stress exponent of 5.86. The sloping portion of each curve represents the approximate lower 95 percent confidence limit for 95 percent survival as derived from test results for the most severe detail in that category.<sup>16,17</sup> An explanation of such a confidence limit is given in Section III of this chapter.

The horizontal portion of each curve represents the best estimate that could be made of the fatigue limit for the most severe detail in that category. This estimate was based on experimental data supplemented by crack-growth threshold calculations.<sup>16</sup> Further experimental studies are being conducted to better establish the fatigue limits for various details.

Tensile residual stresses in welded members can cause fatigue cracking in regions that are subjected only to applied compressive stresses as discussed in Section III. These cracks, however, usually stop growing after they propagate into adjacent regions of zero or compressive residual stresses.<sup>11,17</sup> Consequently, the AASHTO fatigue provisions<sup>8</sup> do not impose stress-range limitations where the stresses are always compressive. In practice, however, caution should be used

before dismissing the possibility of fatigue failures in regions of nominal compressive stresses because unexpected tensile stress components often occur as a result of residual stresses or local triaxial stress conditions.<sup>2</sup> Such triaxial conditions occur around notches and concentrated loads.

Each of the seven AASHTO detail categories is briefly discussed below. The individual details included in each category are identified and important factors that influence the fatigue strength of each are described. These factors include imperfections normally present in all structural members as a result of manufacturing and fabrication processes.<sup>3,17</sup> Such imperfections are very difficult to eliminate, and efforts to remove them may cause worse conditions than were originally present.<sup>3,17</sup> Since the simulated members that were tested to establish allowable fatigue stresses contained such imperfections, the effects of these imperfections have been adequately accounted for.

### CATEGORY A

Category A provides the highest fatigue strength attainable for bridge members. It includes plates and shapes with mill surfaces or good-quality flame-cut edges, that is, edges with an average surface roughness<sup>21</sup> not exceeding<sup>1000</sup> micro-inches and no severe gouges. Although the AASHTO fatigue provisions do not mention weathered surfaces, a recent study<sup>22</sup> concluded that it is appropriate to include weathering-steel plates and shapes with such surfaces in Category A. The study also indicated that weathering of various types of fabricated details does not change their category classification. Category A does not include plates with sharp nicks or gouges due to handling; such nicks and gouges should be repaired in accordance with ASTM<sup>23</sup> and AASHTO<sup>24</sup> specifications.

The allowable SN curve for Category A approximates the lower 95-percent confidence limit from tests conducted at Lehigh University<sup>11</sup> on rolled shapes of A36 and A441 steels.<sup>25</sup> This curve also conservatively represents data from tests conducted at Lehigh<sup>12</sup> on A514 steel rolled shapes and data from elsewhere on rolled plates and shapes of various steels.<sup>12</sup> The fatigue strength of the rolled plates and shapes was controlled by normal surface imperfections and roughness. Because of the variability in the severity of the imperfections and roughness, the scatter in fatigue results for rolled plates and shapes is greater than the scatter for fabricated details. Tests conducted at Lehigh University<sup>11</sup> on welded shapes with good-quality flame-cut edges showed that the failure of such shapes initiated in the flange/web fillet welds rather than at the flame-cut edges. Therefore, good-quality flame-cut edges were included in Category A

## CATEGORY B

Category B includes 1) continuous longitudinal\* fillet or groove welds, 2) full-penetration transverse\* groove welds ground flush, 3) tapered splices with the weld reinforcement removed, 4) 24-inch radius curved transitions for flange plates or groove-welded attachments, and 5) bolted joints. Specific requirements for some of these details are given in Table I.

The allowable SN curve for Category B approximates the lower 95-percent confidence limit from tests conducted at Lehigh University on welded shapes without stiffeners, attachments, or other geometric discontinuities.<sup>11</sup> Shapes of A36, A441, and A514 steels<sup>25</sup> were included. Longitudinal-fillet-weld data from other sources are conservatively represented by this SN curve.<sup>11,12</sup> These data include the variable-amplitude results from a study conducted at the U.S. Steel Research Laboratory.

Since no significant geometric stress raisers were present in the welded shapes, cracking usually initiated at normal internal imperfections, such as gas pockets, in the flange/web fillet welds.<sup>11</sup> Generally, the critical imperfections occur near tack welds, stop/start positions, or weld repairs, that are permitted in bridge fabrication.<sup>24</sup> Incomplete penetration in continuous longitudinal fillet welds does not affect their fatigue strength because it results in a discontinuity whose plane is parallel with the direction of stress. Such discontinuities do not act as stress raisers and hence have little effect on fatigue behavior.<sup>13</sup> Terminations or interruptions of longitudinal fillet welds do act as stress raisers and are covered in other detail categories. Manual fillet welds tend to have a lower fatigue strength than automatic or semi-automatic fillet welds, but are included in Category B if they are properly made.<sup>11</sup>

Longitudinal groove welds are similar to fillet welds and, therefore, are included in Category B.<sup>13</sup> The assignment to Category B of full-penetration transverse groove welds ground flush is based primarily on fatigue tests conducted at Lehigh University on welded beams with groove-welded flange splices.<sup>11</sup> The results for these beams were similar to the results for welded beams without splices. The flange splices had a 1 to 2½ taper in width and were nondestructively inspected to establish weld soundness. Other data on full-penetration groove-welded joints in plates showed that their fatigue strength is controlled by normal internal imperfections, and approaches that of plates with mill surfaces provided that the weld reinforcement is properly ground flush and the weld soundness is established by non-destructive inspection.<sup>20</sup> Consequently, to be included in Category B, transverse groove welds must be nondestructively inspected to establish weld soundness.

Joints with a 1 to 2½ taper in either width or thick-

ness were included in Category B primarily on the basis of the Lehigh tests mentioned in the preceding paragraph.<sup>11</sup> When these data were interpreted, no tests had been conducted at Lehigh on tapers in thickness,<sup>12</sup> but a few available data from the University of Illinois<sup>26</sup> were generally above the allowable SN curve for Category B. More recent studies suggest that a 1 to 2½ taper in thickness is more severe than a similar taper in width because the stress concentration for the thickness taper is about 2.0,<sup>27,28</sup> while the stress concentration factor for the width taper is only 1.1.<sup>11</sup> Nevertheless, the fatigue results from a few recent tests performed at Lehigh on welded girders with a 1 to 2½ thickness taper were above the allowable SN curve for Category B.

Because the original Lehigh Tests<sup>11</sup> of welded beams with tapered flange splices showed that the results for A514 steel were below the allowable SN curve for Category B, width tapers in A514- or A517-steel<sup>25</sup> plates are prohibited in AASHTO Article 10.18.5.5 on splices.<sup>8</sup> Curved width transitions, which provided a fatigue strength above the Category B allowable SN curve,<sup>11</sup> are permitted with A514 steel, however.<sup>8</sup> AASHTO gives no special provisions for 1 to 2½ thickness tapers with A514-steel plates, but the above discussion suggests that it would be appropriate to use a fatigue category lower than B for this case.

The welded beams with tapered flange splices that were tested at Lehigh<sup>11</sup> also had full-penetration groove-welded flange splices with a curved width transition. The radius of the curve was 24 inches. The transverse groove welds were ground flush and non-destructively inspected to establish weld soundness. The splices with curved width transitions had about the same fatigue strength as the tapered splices if the results for A514 steel were eliminated from both sets of data.<sup>11</sup> Therefore, 24-inch-radius curved transitions for flange splices, and attachments or gusset plates that are joined to members by full-penetration longitudinal groove welds, are included in Category B. The ends of the longitudinal groove welds must be ground smooth. For attachments with curved transitions of any radius, the same detail category applies to both longitudinal stresses in the main member and transverse stresses in the attachment.

Twenty-four-inch curved transitions for attachments (Fig. 1) joined to members by longitudinal fillet welds were originally included in Category C.<sup>13,17</sup> but were moved to a lower category\* when later studies showed that there is a high probability of subsurface discontinuities occurring at the roots of the fillet welds in the transition radius near the point of tangency.<sup>14</sup> Such subsurface discontinuities cannot be readily detected by nondestructive inspection. Similar root imperfections could occur in partial-penetration groove welds. Therefore, curved transitions associated with such

\*As used here, *transverse* is defined as the direction perpendicular to the main stress in the member, and *longitudinal* is the direction parallel to the main stress.

\* Lower category means a category with a lower allowable SN curve.

welds should be treated in the same way as curved transitions associated with fillet welds; this is not clear from the wording of the specifications.<sup>8</sup>

The inclusion of high-strength bolted joints in Category B was based on existing data from several sources.<sup>29</sup> These data are specifically for symmetric butt splices (double-lap joints) of two types: 1) slip-resistant (friction) joints and 2) bearing joints. In slip-resistant joints, most of the force is transferred by friction between the contacting surfaces and fatigue cracks usually initiate at the surface in the gross section due to fretting.<sup>29</sup> Therefore, fatigue behavior is controlled by gross-section stresses. In bearing joints, a large portion of the force is transferred by bearing and shear of the bolts, and fatigue cracks usually initiate at holes in the net section.<sup>29</sup> Therefore, fatigue behavior is controlled by net-section stresses. The available fatigue data are conservatively represented by the allowable SN curve for Category B when the gross-section stresses are used for slip-resistant joints and the net-section stresses are used for bearing joints.<sup>29</sup> Consequently, the gross-section and net-section stresses, respectively, must be used in applying the AASHTO specifications<sup>8</sup> to slip-resistant and bearing joints.

The scatter in the available data for bolted-joints is larger than for most other details because of variations in joint configuration, hole fabrication methods, bolt-tightening techniques, contact-surface conditions, cyclic loading conditions, and other factors. For both types of bolted joints, higher clamping forces, greater coefficients of friction for contact surfaces, and stress reversals (rather than cycles without reversals) tend to result in higher fatigue strengths.<sup>29</sup> For bearing joints, drilled, or subpunched and reamed, holes tend to provide higher fatigue strengths than punched holes.<sup>13</sup>

Category B is not intended to apply to 1) single-lap joints, 2) joints subjected to prying action, or 3) joints subjected to direct tension perpendicular to the plate surfaces.<sup>29</sup> Significant bending stresses develop in the plates of such joints, but only membrane stresses occur in symmetric butt splices. A truss joint in which an I or box section is connected by gusset plates on two sides can be considered a symmetric butt splice and included in Category B even though the bolts joining each gusset plate are in single shear.

### CATEGORY C

Category C includes 1) transverse stiffeners or attachments, 2) full-penetration transverse groove welds with the reinforcement not removed, 3) 6-inch-radius curved transitions for groove-welded attachments, and 4) stud-type shear connectors. Specific requirements for some of these details are given in Table I. The allowable fatigue limit for transverse stiffeners is slightly higher than that for the other details in this category.

The allowable SN curve for Category C approximates the lower 95-percent confidence limit from tests conducted at Lehigh University on welded beams and

girders with transverse stiffeners that were welded to either the web alone or to the web and tension flange.<sup>12</sup> Appropriate data on transverse stiffeners from other sources also fall above this SN curve.<sup>12</sup> Fatigue cracks initiated at the toe of the fillet weld at the end of the stiffener if the stiffener was not welded to the flange, and at the toe of the stiffener/flange fillet weld if it was.<sup>12</sup> The crack initiation was influenced by the stress concentration factor of 2.2 to 4 at these locations<sup>27</sup> and sometimes also by the normal weld imperfections. The fatigue strengths of the two types of stiffeners—those welded to the flange and those not—were about the same. When lateral bracing was attached to the stiffeners it did not reduce their fatigue strength.<sup>12</sup>

Bending stress at the crack-initiation location is the stress parameter for the experimental data on stiffeners and for the corresponding allowable SN curve. It was concluded that the presence of shear in combination with this bending stress need not be considered in normal bridge designs because of the shear/moment relationships that exist in such bridges.<sup>12</sup> However, if abnormally high shear stress occurs in combination with abnormally low bending stress at the stiffener, the maximum principal tensile stress should be used as the stress parameter in conjunction with the allowable SN curve.

Transverse attachments, i.e., plates fillet welded to the flange or web perpendicular to the stress, are equivalent to transverse stiffeners and belong in Category C. In fact, any attachment less than 2-inches wide in the direction of stress is similar to a transverse stiffener and is included in Category C.

Test data<sup>18,19,20</sup> show that weld reinforcement causes a stress concentration that reduces the fatigue strength of full-penetration transverse groove welds. For reinforcement with a 60-degree angle at the toe of the weld, the stress concentration factor was reported to range from 1.3 to 1.8.<sup>27,30</sup> Available fatigue data showed that Category C is appropriate for such welds, provided that their soundness has been established by nondestructive inspection.<sup>12</sup> One to 2½ tapered transitions in width or thickness are permitted in conjunction with such transverse groove welds because the limited data available for such joints<sup>27</sup> were generally above the allowable SN curve for Category C.

If an attachment plate is groove-welded to the edge of the flange of a member, a high stress concentration occurs at the edge of the attachment and reduces the fatigue strength of the member. If a curved transition is provided at the edge of the attachment as illustrated in Case 14 of Figure 1, the stress concentration and the fatigue strength depend on the radius of the transition. As mentioned earlier, tests<sup>11</sup> showed that a 24-inch curve transition belongs in Category B; the 6-inch-radius transition was assigned to Category C on the basis of its higher stress concentration factor. Subsequent fatigue tests conducted at Lehigh on 6-inch and 2-inch transitions suggest that this was appropriate.<sup>14</sup> The ends of the longitudinal groove welds must be ground smooth. These welds should be full-penetration

welds, as previously discussed, although this is not specifically stated in the specifications.<sup>8</sup>

Stud-type shear connectors can be considered to be short attachments and, therefore, included in Category C. The results of fatigue tests on plates or beams with studs attached by arc or friction welding<sup>30</sup> confirm that it is appropriate to assign such details to Category C. Channel-type shear connectors fillet-welded transversely to the flange are also attachments. They are included in Category C if their width in the direction of the flange stress does not exceed 2 inches; otherwise, they are included in a lower category.

## CATEGORY D

Category D includes 1) 4-inch attachments, 2) 2-inch-radius curved transitions for groove or fillet welded attachments, and 3) riveted joints. Specific requirements for some of these details are given in Table I.

The allowable SN curve for Category D approximates the lower 95-percent confidence limit from tests conducted at Lehigh University on beams with 4-inch-long attachments fillet welded to the tension flange.<sup>12</sup> Appropriate data from other sources on fillet- or groove-welded attachments to plates or beams also fall above this allowable SN curve.<sup>12</sup> Two types of attachments were included in the Lehigh<sup>12</sup> data: 1) attachments with welds on longitudinal edges only and 2) attachments with welds on both longitudinal and transverse edges. The former type tended to have a slightly higher fatigue strength.<sup>12</sup> Fatigue cracks initiated at the toes of the transverse welds or the ends of the longitudinal welds, and were controlled by the stress concentrations at these locations.

Fatigue tests were also performed at Lehigh<sup>12</sup> on attachments of other lengths. The results varied with the length of the attachment. These results fell between those for a transverse stiffener, that can be considered equivalent to an attachment of minimum length (in the direction of stress), and those for a partial-length cover plate, which is equivalent to an attachment of maximum length. The fatigue strength decreased with increasing attachment length because more force is developed in longer attachments and this increases the stress concentration caused by the attachment. The 4-inch attachment provides a convenient category about midway between that for cover-plate ends and that for stiffeners. Category D, of course, is conservative for attachments shorter than 4 inches, except that when such shorter attachments exceed 12 times their thickness they are excluded from Category D.<sup>8,12</sup> This exception governs only attachment plates thinner than 0.333 inch.

Fillet- or groove-welded attachments of any length are included in Category D if they have a curved transition radius of not less than 2 inches and the weld ends are ground smooth.<sup>8</sup> Such transitions were assigned to Category D on the basis of the reduced stress concentration factor provided by the radius.<sup>13</sup> Subsequent fatigue tests<sup>14</sup> showed that it is conservative to include

groove-welded attachments with a 2-inch radius in Category D. In fact, data for groove-welded rectangular attachments in which the weld ends had been ground to a radius of 0.2 to 0.4 inch, fell within the allowable SN curve for Category D.

Fatigue data from the University of Alberta<sup>31</sup> on fillet-welded longitudinal attachments with 4-inch-radius transitions showed that such attachments "provided a fatigue resistance compatible with Category D."<sup>14</sup> On the basis of these and other data,<sup>32</sup> it was concluded that fillet-welded attachments with curved transitions of various radii above 2 inches should not be included in Categories B and C, as they formerly were,<sup>13,17</sup> but could be retained in Category D.<sup>14</sup>

Although riveting is no longer used to fabricate new bridges, many riveted bridges are currently (1985) in service. Consequently, AASHTO<sup>8</sup> includes riveted joints in its classification of details. Available fatigue data<sup>33,34,35</sup> on riveted symmetric butt splices (double-lap joints) shows that there is an exceptionally large amount of scatter for such data, and that the lower bound SN curve for the data has an unusually flat slope. Consequently, riveted joints have been assigned to the relatively low Category D. Net-section stress is the appropriate stress parameter for such joints because the clamping force provided by rivets is uncertain and cracking often occurs in the net section.

Many different types of riveted details are currently (1985) in service in existing bridges. For example, built-up girders, cover-plate ends, transverse stiffeners, truss gusset-plate details, and transverse attachments are common in riveted, as well as in welded, construction. Although these different types of riveted details may have different fatigue strength, they are all included in Category D.

## CATEGORY E

Category E includes 1) ends of cover plates fillet-welded to flanges not greater than 0.8 inch thick, 2) attachments longer than 4 inches, 3) intermittent longitudinal fillet welds, and 4) fillet-welded lap joints. Specific requirements for some of these details are given in Table I. Originally, this category was thought to represent the lower bound of fatigue strengths for fabricated bridge members,<sup>16,17</sup> but it was later discovered that lower strengths occur.<sup>13,36</sup>

The allowable SN curve for Category E approximates the lower 95-percent confidence limit from tests conducted at Lehigh University on beams with partial-length cover plates having ends that were either fillet-welded or unwelded.<sup>11</sup> Appropriate constant-amplitude fatigue data from other sources on cover-plate ends were shown to be consistent with this SN curve.<sup>12</sup> The results of subsequent variable-amplitude tests conducted at the U.S. Steel Research Laboratory<sup>2</sup> on cover-plate ends similar to those tested at Lehigh are also consistent with the allowable SN curve.

In the Lehigh tests,<sup>11</sup> the partial-length cover plates were welded to 1) rolled beams, 2) rolled beams with

full-length cover plates, or 3) welded beams. The test specimens included 1) cover plates that were both narrower and wider than the flange, 2) several different types (strengths) of steel, and 3) cover plates that were either 1½ or 2 times the flange thickness. AASHTO Article 10.13.3 presently (1985) limits the thickness of a single cover plate to 2 times the flange thickness and the total thickness of all cover plates to 2½ times the flange thickness. Fatigue cracks initiated at the toes of the transverse welds or the ends of the longitudinal welds and were controlled by the stress concentrations at these locations.

The Lehigh tests showed that, on cover plates wider than the flange, a transverse end weld is required to assure that the fatigue strength does not fall below Category E.<sup>11</sup> Therefore, in the previous edition of the AASHTO specifications, a transverse end weld was required when the cover plate was wider than the flange. This provision, however, is not included in the present edition.<sup>8</sup> For cover plates narrower than the flange, an end weld tends to reduce the fatigue life<sup>11</sup> and is not required.<sup>8</sup>

Variations in the end geometry of cover plates, including tapered and rounded ends, were shown to have little effect on fatigue strength.<sup>17,18,37</sup> Therefore, all such cover-plate ends are included in Category E.

As mentioned earlier, tests<sup>12</sup> showed that the fatigue strength of attachments decreases as attachment length increases, and is equal to the fatigue strength of a cover-plate end when the attachment length exceeds a certain value. This value has not been exactly established, but tests<sup>12</sup> showed it exceeds 8 inches and probably is about 16 to 20 inches. To avoid an excessive number of different categories, however, attachments with lengths exceeding 4 inches have been conservatively included in Category E. Similarly, attachments having curved transitions with a radius less than 2 inches are included in category E. Similarly, attachments having curved transitions with a radius less than 2 inches are included in Category E, even though they have a higher fatigue strength than attachments without a curved transition. The assignment to Category E of attachments with a length greater than 4 inches and a transition radius less than 2 inches applies to both fillet- and groove-welded attachments.

The ends of intermittent longitudinal fillet welds cause stress concentrations somewhat similar to, but probably lower in magnitude than, cover plate ends. Because of the lack of fatigue data on such welds they have been conservatively assigned to Category E.<sup>13</sup> Fillet-welded lap joints such as illustrated in Case 9 of Figure 1, are also similar to cover-plate ends and are included in Category E.

#### **CATEGORY E'**

Category E' includes 1) ends of cover plates fillet-welded to flanges greater than 0.8 inch thick and 2) girder flanges greater than 1-inch thick that pierce through the web of another girder and are fillet welded

to each side of that web. This category represents the lower bound of fatigue strengths for acceptable bridge details. Details with lower fatigue strengths exist but should not be used because such fatigue strengths are too low for most practical bridge applications.

The allowable SN curve for Category E' approximates the lower 95-percent confidence limit from tests conducted at Lehigh University on heavy beams with partial-length cover plates having ends that are either fillet welded or unwelded.<sup>13,28</sup> Most of the beams were W36x230 rolled shapes, which have a flange thickness of 1.26 inches. Most of the cover plates were 1.25-inches thick. Fatigue cracks initiated at the toes of the transverse welds or ends of the longitudinal welds and were controlled by the stress concentrations at these locations. Apparently, the stress concentrations are higher for these heavy beams than for the lighter beams included in Category E.

The AASHTO specifications<sup>8</sup> do not specifically mention girder flanges greater than 1-inch thick that pierce through the web of another girder and are fillet welded to each side of that web. However, the results of recent fatigue tests have led Lehigh University to recommend<sup>14</sup> that such details be included in Category E'. Similar details in which the piercing flange is less than 1-inch thick can be thought of as attachment plates and "treated as Category E connections."<sup>14</sup> Similar cross-girder details in which "seal" welds are placed on only one side of the pierced web have very low fatigue strengths and should not be used.<sup>13,14</sup>

Fatigue tests<sup>14</sup> of plates attached to flange surfaces only by transverse fillet welds showed that such details have fatigue strengths well below the allowable SN curve for Category E'. Therefore, the AASHTO specification<sup>8</sup> states that "Gusset plates attached to girder flanges with only transverse fillet welds, [are] not recommended." In the tests, fatigue cracking usually initiated at the root of the weld and severed the plate from the flange.<sup>14</sup> In one test, however, the attachment also caused fatigue cracking in the flange.

#### **CATEGORY F**

Category F includes only shear stress on the throat of fillet welds. It applies to continuous or intermittent longitudinal or transverse fillet welds. The allowable SN curve for this category was developed<sup>13</sup> from fatigue tests conducted at the University of Illinois<sup>38</sup> during the early 1940s on small fillet welded plate specimens in which the welds were subjected to high shear stresses. Shear stresses, as defined in Category F, usually do not control the fatigue strength of fillet welded details;<sup>13</sup> instead, the fatigue strength of such details is usually controlled by axial or bending stresses at the toe of transverse welds or ends of longitudinal welds as defined in other categories. The allowable SN curve for Category F has a slope different from that of the SN curves for other categories, but it has been suggested that the same slope could be used.<sup>39</sup> Future studies have been suggested to verify this.<sup>13</sup>

### 3. BRIDGE DESIGN AND RATING METHODS

#### PRESENT AASHTO METHODS

The present (1985) AASHTO methods for both the design<sup>8</sup> and rating<sup>40</sup> of highway bridges are similar. For each of the detail categories discussed previously, the specifications<sup>8</sup> give allowable stress ranges for each of four design-life categories. The appropriate design-life category for a particular bridge and type of loading (truck or lane) is defined in tables given in the specifications.<sup>8</sup> The calculated stress range for any detail must be less than the allowable stress range for that detail. These present AASHTO methods are discussed in more detail below.

#### Allowable Stress Range

Instead of giving allowable SN curves such as those in Figure 2, AASHTO<sup>8</sup> gives specific values of allowable stress range corresponding to four different design-life categories or points on these curves: 1) 100,000 cycles, 2) 500,000 cycles, 3) 2,000,000 cycles and 4) over 2,000,000 cycles. Stresses for this last category are taken at the fatigue limit or horizontal portion of the curve.

Specific values of the allowable stress ranges given by AASHTO for redundant and nonredundant load path structures are listed in Table II. The values for redundant structures correspond exactly to the allowable SN curves in Figure 2 and are intended for use with structures that have multiple load paths, such as multi-girder bridges or multi-element eye bars. For such structures, fracture of a single element does not cause collapse of the structure.

The allowable stress ranges for nonredundant structures are intended to provide greater safety for those structures that could collapse as a result of the fracture of a single element. Such structures<sup>8</sup> include "flange and web plates in one or two girder bridges, main one-element truss members, hanger plates, and caps at single or two column bents." The allowable stress ranges for each of the first three design-life categories was obtained by using the redundant allowable stress range from the next longer design-life category.<sup>17</sup> The allowable stress ranges for the last design-life category (over 2,000,000 cycles) are generally the same as, or a few ksi lower than, the corresponding values for redundant structures. Category E' details are not permitted with nonredundant structures because allowable stresses derived as discussed above would be too low for practical usage.<sup>13</sup>

#### Calculated Stress Range

Generally, only traffic or wind loadings need be considered in calculating the stress range. Main (longitudinal) load carrying members must be checked for two traffic loadings: 1) truck loading and 2) lane loading. Transverse members and details subjected to wheel loads must be checked for truck loading but not lane loading. Wind bracing members must, of course, be checked for wind loading. The stress range calculated for each applicable loading must be less than the cor-

responding allowable stress range unless the calculated stresses are always compressive.

At any location along a beam or girder, the stress range equals the sum of the absolute values of the maximum positive and negative live-load (plus impact) moments divided by the section modulus. Positive moment is defined as moment producing tension in the bottom flange. The maximum live-load moment of either sign is obtained by placing the truck or lane load in the critical position for moment of that sign in accordance with AASHTO<sup>8</sup> Article 3.11. Lane load is placed on all portions of the member where it contributes to the desired moment and omitted elsewhere.

Dead load does not affect the stress range since it does not cause stress variations. Nevertheless, dead load can affect fatigue calculations because a fatigue check is required only if a portion of the stress cycle is in tension. For continuous-span beams or girders, negative dead-load moments near interior supports are usually larger than the positive live-load moments in this region. Therefore, the bottom portions of the girder are always in compression and need not be fatigue checked. Similarly, the positive dead-load moments near midspans are usually larger than the negative live-load moments in this region. Therefore, the top portions of the girder are always in compression and need not be fatigue checked. In the lengths of the beam or girder between these two regions, tension stresses occur in both the top and bottom portions. Therefore, both portions must be fatigue checked. Dead load does not affect fatigue calculations for simple-span beams and girders because both the live- and dead-load moments are always positive. Consequently, the top portion of the girder is always in compression and need not be fatigue checked.

Chord members in simple- or continuous-span trusses are subjected to stresses similar to those in the flanges of girders, and need to be checked for fatigue in the same way. Web members (diagonals and verticals) in trusses are usually subjected to reversals as the truck passes from one side of the member to the other. Such members must be checked for fatigue.

Generally, the same loadings, lateral-distributions factors, and impact factors are used in calculating the stress range for the fatigue check as are used in the normal strength design. As a result, the calculated stress ranges generally exceed the stress ranges that actually occur in bridges as will be discussed in more detail later.

For one design-life category, namely that for over 2,000,000 cycles applied to longitudinal members, special calculation procedures are used in the fatigue check. Specifically, the stress range is calculated by placing a single truck on the bridge and distributing its weight to the girders as designated in AASHTO Article 3.23 for one traffic lane loading.<sup>8</sup> This results in the use of a lateral-distribution factor of  $S/7$  instead of  $S/5.5$ ,



where  $S$  is the average girder spacing in feet. This procedure recognizes that most fatigue stresses are caused by single trucks as discussed in Sections I and II.

### Design-Life Categories

The appropriate design-life category for a particular bridge member subjected to traffic loadings can be determined from Table III.<sup>8</sup> Case I was recently adopted to provide for extremely heavy truck traffic<sup>17</sup> such as that which caused fatigue cracking in the Yellow Mill Pond Bridge.<sup>36</sup> Cases II and III were retained from earlier versions of the AASHTO specifications because "no fatigue problems have been experienced with bridges in these categories."<sup>17</sup>

The AASHTO specifications do not define which types of trucks should be included in the Average Daily Truck Traffic (ADTT). Presumably, panel, pickup, and other 2-axle/4-wheel trucks should be excluded since they cause little fatigue damage (Section I).

The meaning of the particular design lives specified in Table III is obscure. As discussed later in more detail, these lives do not represent either the total number of trucks expected to pass over the bridge during its life, nor the number of 72-kip trucks expected to pass during this period. Instead, they are artificial numbers corresponding to allowable stress ranges that are expected to provide reasonable designs and that are consistent with a life expectancy of 60 to 70 years.<sup>17</sup>

AASHTO specifies that the design lives in the table should be used "unless traffic and loadometer surveys or other considerations indicate otherwise." Because the specified design lives are artificial numbers, however, it is difficult to relate them to such surveys. Some guidance for doing this is given in Reference 17.

AASHTO<sup>8</sup> specifies that 100,000 cycles of maximum wind loading should be used in the fatigue check unless available information indicates otherwise. Again this design life is an artificial number intended to provide a reasonable design.

### Rating of Bridges

To assure the continuing safety of existing bridges, such bridges are periodically inspected and safety rated by procedures given in the AASHTO maintenance manual.<sup>40</sup> The manual permits the rating of steel bridges by either the Load Factor Method or the Working Stress Method. By either method, each bridge is rated at two load levels that provide different factors of safety against static (nonfatigue) failure: 1) the Operating Rating, corresponding to the maximum load permitted to cross the bridge, and 2) the Inventory Rating, corresponding to the load permitted to cross the bridge on a routine basis.

In the Load Factor Method, fatigue strength does not affect the Operating Rating, but is one of three criteria affecting the Inventory Rating. The rating procedure yields the maximum load (or percentage of the rating vehicle used by a particular state) permitted for fatigue. This maximum load is based on the most critical fatigue detail in the bridge, and is calculated by multiplying the gross weight of the rating vehicle by the ratio of allowable stress range to the stress range

calculated for that vehicle. The methods of calculating the stress range are the same as those used for the design of new bridges. Fatigue is not mentioned in the Working Stress Method, but similar procedures can be used to get a maximum fatigue load for this method.

### Comparison with Actual Behavior

The AASHTO<sup>8</sup> fatigue-design procedures described above were developed before extensive information was available on the fatigue stresses that actually occur in bridges. Therefore, these procedures do not accurately reflect the actual conditions described in Sections I and II. The differences are illustrated in the examples in the next two paragraphs.

First, consider a relatively short simple-span girder bridge for which the fatigue design is governed by truck rather than lane loading. Assume that the average daily truck traffic is 2,000 and that each passage of a truck causes one loading cycle. This traffic will cause 36,500,000 loading cycles in 50 years — a reasonable minimum life for a bridge. According to the FHWA 1970 nationwide loadometer survey presented in Section I, 7.37% of these cycles or 2,670,000 cycles, would exceed a weight of 72 kips, which corresponds to the AASHTO HS20 design truck. The AASHTO specifications<sup>8</sup> require that main members of a bridge on a major highway with an average daily truck traffic of less than 2,500 be designed for 500,000 stress cycles caused by an HS20 truck (or for 100,000 cycles of lane loading, which was assumed not to govern). As discussed in Section II, field measurements have shown that the actual stress ranges in bridges are usually considerably less than would be calculated for the HS20 truck by present AASHTO procedures. Thus, the AASHTO design conditions for this example combine an artificially high fatigue stress range with an artificially low number of stress cycles to get a reasonable design.

Next, consider a continuous girder bridge with relatively long spans for which lane loading governs the AASHTO fatigue design. In designing this bridge, the loading would first be applied over certain portions of the bridge to obtain the maximum positive moment and then over different portions of the bridge to obtain the maximum negative moment. The sum of the two moments would be used to calculate the design stress range. This type of loading results in the worst possible positive and negative moments and, therefore, is appropriate for static design.

However, this type of loading, which results in large stress ranges, would occur very rarely, if ever. Therefore, it is overly conservative for use in determining the fatigue life of the bridge. Instead, the life would be mainly influenced by truck loadings. Thus, the AASHTO design conditions for this example are far from the conditions that actually affect the fatigue life of the bridge.

Next, consider the present method of including fatigue in the rating process. Not only does this method fail to reflect the actual fatigue conditions in bridges, but it also fails to provide the *type* of information needed to accurately assess the present condition and

estimated remaining life of a bridge. In fact, the meaning of the maximum permissible fatigue loading, which is presented as a percentage of the rating vehicle gross weight, is vague. How many additional repetitions (beyond those that have already been applied) of this loading can be tolerated? More important, how does this loading relate to the actual traffic, which consists of trucks of various weights? How can it be used to estimate the present amount of fatigue damage and the remaining life?

## PROPOSED NEW METHODS

Because the AASHTO specifications<sup>8</sup> do not accurately reflect the fatigue conditions that actually occur in bridges, several new methods of fatigue design have been proposed in recent years.<sup>41-45</sup> The intent of these new methods has been to utilize more realistic fatigue loading and stress conditions and/or to provide more uniform reliability factors. All of the new methods use the AASHTO detail categories and the fatigue strength information developed at Lehigh University.

One of the proposed new methods,<sup>39,41</sup> which was developed at the U. S. Steel Research Laboratory, is described in detail under subsequent headings. It accurately reflects actual fatigue conditions as described in Sections I and II, and can be conveniently applied to both the design and rating of bridges. It could be easily incorporated into specifications, and could be tailored to provide any desired degree of safety by simply changing the value of certain key parameters.

In addition to the proposed new methods mentioned above, a method involving modifications of the detail categories has been proposed in Europe. Specifically, the Lehigh fatigue data has been used by the European Convention for Constructural Steelwork\* (ECCS) as the basis for a proposed group of 15 detail classes. The allowable SN curves for these classes are equally spaced between the upper- and lower-bound AASHTO curves. The percentage difference between the allowable stress ranges for adjacent classes is about 12 percent compared with about 30 percent for the AASHTO detail categories. This appears to be a greater refinement in detail categories than is justified by the available fatigue data. Nevertheless, the ECCS proposal is being reviewed in this country.<sup>15</sup>

### Fatigue-Design Truck

The first step in the proposed new method<sup>39,41</sup> is to select the gross weight for a fatigue-design truck that represents the variable truck weights in actual traffic. This gross weight is less than that of an HS20 truck<sup>8</sup> because it represents an "average" condition for the actual traffic. Specifically, the weight is selected so that the fatigue damage caused by a given number of passages of this truck is the same as damage caused by an equal number of passages of different-sized trucks in actual traffic.

If the distribution of truck weights (percentages of trucks of different weight) for the traffic under consideration is known, the following formula<sup>2</sup> is used to calculate the gross weight of the fatigue-design truck:

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

In which  $\alpha_i$  is the fraction of trucks with a gross weight  $W_i$ . As discussed in Section I, a value for  $W_F$  of 50 kips would generally be appropriate if the actual weight distribution is not known. This weight is based on distributions from various nationwide surveys and may not be appropriate for locations where extremely heavy truck traffic is expected. It is the bridge designer's responsibility to identify such locations and to obtain the data needed to calculate an appropriate weight from Equation 1. This weight is one of the key parameters that could be changed by specification writers to provide any desired degree of safety.

The appropriate axle spacing and distribution of axle loads for the fatigue-design truck are shown in Figure 4 of Section II. As discussed in this section and elsewhere,<sup>47</sup> the spacing of 30 ft between main axles is appropriate because most fatigue damage is done by trucks with this spacing.

Lane loading generally need not be considered in fatigue design because the net effect of closely spaced trucks on fatigue is usually small as discussed in Sections I and II and elsewhere.<sup>47</sup> However, exceptions may exist at locations where special conditions tend to bunch trucks. These conditions may include a steep grade on a two-lane road or a traffic signal. For such situations the bridge designer must use his judgment based on a knowledge of local traffic conditions because no specific criteria for fatigue design are available. The discussion of traffic characteristics given in Section I might provide some help in interpreting local conditions.

### Design Stress Range

The next step is to calculate the design stress range caused by the passage of the fatigue-design truck across the bridge in the lane under consideration. First, the moment range is calculated by putting the truck in positions that cause the maximum positive and negative moments at the detail under consideration. Then a lateral-distribution factor is used to calculate the design stress range from the moment range. The lateral-distribution factors given in the AASHTO specifications<sup>8</sup> are based on severe conditions<sup>48</sup> that do not occur often as discussed earlier.

Therefore, the lateral-distribution chart<sup>48</sup> shown in Figure 11 of Section II is used in the new method unless a more refined lateral-distribution analysis is made. This chart, which is explained in detail in Section II, gives the fraction of the *total* truck moment that is carried by an exterior or interior beam or girder. It is based on a single truck at the center of the critical traffic lane since this is the appropriate condition for fatigue design. The lateral-distribution factor for any beam is a function of the lane position ratio,  $P$ , which is

\*The United States is represented in this international group.



the distance from the exterior beam to the center line of the outer traffic lane divided by the beam spacing. This parameter is important because the bridge cross section tends to act as a semi-rigid unit subject to twisting due to the eccentricity of the load.<sup>48</sup>

The solid lines represent the upper limit for the lateral-distribution factor, and can be used to make an initial conservative fatigue check. If this check is satisfactory, no further check is required. Otherwise, a second check can be made using the more precise, less conservative, dashed lines, which depend on the beam span and moment of inertia. When  $P$  exceeds 0.5, the solid horizontal line applies to both interior and exterior beams, but below 0.5 it applies only to interior beams. When  $P$  is less than 0.5 the exterior beams are governed by the solid sloping line.

As discussed in Section II, the impact factor defined in AASHTO Article 3.8<sup>8</sup> is generally appropriate for fatigue design and should be applied to both the positive and negative moments.<sup>49</sup> This factor may not always be conservative for cantilever (suspended span) girder bridges, but alternative factors are not available.<sup>49</sup>

In line with the present AASHTO fatigue specifications, no fatigue check is required if the stress due to combined dead, live and impact loading is compressive for all trucks in the traffic. This occurs when the dead-load compressive stress exceeds twice the live-load (plus impact) tensile stress caused by the fatigue-design truck because this truck has about  $\frac{1}{2}$  the weight of the largest truck in the traffic<sup>39</sup> as will be discussed in more detail later.

#### Allowable Stress Range

The next step is to compare the design stress range with the allowable SN curve for the detail under consideration. The AASHTO<sup>8</sup> detail categories are used. The sloping portions of the allowable SN curves, which are given in Figure 3, are generally identical with those in Figure 2 except that the slope of the curve for Category F has been made equal to that of the others as suggested in Reference 39. Therefore, all curves are defined by the equation

$$N = \frac{A}{F_{sr}} \quad (2)$$

Values of the constant  $A$  are given in Table III for the various detail categories.

The horizontal portions of the SN curves in Figure 3 are drawn at stress ranges equal to  $\frac{1}{2}$  the constant-amplitude fatigue limits, which correspond to the horizontal lines in Figure 2. Truck-weight distributions given in Section I, and other information,<sup>39</sup> indicate that the weight of the heaviest truck in normal traffic is typically about 100 kips, twice the weight of the fatigue-design truck. (Special-permit trucks with more axles could be heavier without causing higher fatigue stresses.) The level of these horizontal lines is one of the key parameters that could be changed to provide any degree of safety desired by specification writers.

When the design stress range is below the appropri-

ate horizontal line in Figure 3, all of the stress ranges in a spectrum corresponding to typical traffic are below the constant-amplitude fatigue limit. Therefore, the fatigue life is infinite and no further fatigue check is required. This would often be the case in practical designs. In exceptional cases, where the weight of the heaviest truck is more than twice that of the fatigue-design truck, the horizontal lines in Figure 3 should be lowered accordingly. The bridge designer is responsible for identifying and accounting for such conditions.

If a greater degree of safety for nonredundant structures is desired, this could be done by using lower allowable SN curves in a manner similar to that in the present AASHTO specifications.<sup>8</sup> It could also be done by requiring that the design stress ranges for all details be below the horizontal lines in Figure 3, or perhaps below a lower set of such lines.

#### Design Life

If the design stress range is above the appropriate horizontal line in Figure 3, the minimum life of the bridge must be estimated from

$$L = \frac{N}{365TP} \quad (3)$$

in which  $L$  is the life in years,  $N$  is the number of cycles determined from the allowable SN curve,  $T$  is the average daily truck traffic, and  $P$  is the number of stress cycles per passage of a truck. A minimum life of 50 years is proposed, but in some cases a higher value may be desirable. The minimum life is another value that could be changed by specification writers.

For bridges on two-lane highways,  $T$  is the total truck traffic volume in both directions and includes all trucks except panel, pickup, and other 2-axle/4-wheel trucks. For bridges on highways of more than two lanes,  $T$  is the total truck traffic in all lanes in one direction. In both cases, this is conservative because only part of the traffic actually travels in the lane under consideration. The rest travels in the adjacent lane or lanes and causes less fatigue damage than assumed.

If the expected truck-traffic volume is known, it should be used; otherwise, the Design Daily Truck Traffic values listed in Table IV of Section I would generally be appropriate, as discussed in that section. However, it is the bridge designer's responsibility to identify extreme conditions that would require the use of higher traffic volumes.

The design values listed in the table were developed from nationwide traffic surveys and represent very heavy traffic. For bridges on major rural highways, the proposed design value<sup>50</sup> is 400 times the number of lanes, that is, 800 for bridges on two-lane highways, 1600 for bridges on four-lane highways, and 2400 for bridges on six-lane highways. For bridges on two-lane highways, this value represents the traffic in both directions, and for highways of more than two lanes, the value represents the traffic in one direction. Thus, bridges on multi-lane highways are designed for higher traffic volumes per lane than bridges on two-lane highways. This is consistent with observed traffic volumes

from nationwide traffic surveys. For bridges on major urban highways, the proposed design value is 600 times the number of lanes.

Proposed<sup>47</sup> design values of  $P$ , the number of stress cycles per truck passage, are given in Table V of Section II and are explained in that section. Generally, the design value is 2 for very short spans that are affected by individual axle loads and 1 for longer spans, but there are exceptions. (Table V of Section II contains an error. The number of stress cycles per truck passage for continuous-span girders near interior supports should be 2.0 rather than 1.0 when the span is below 40 ft.)

#### Rating

The proposed new method is well suited to the evaluation or rating<sup>40</sup> of existing bridges because it provides a realistic evaluation of the remaining life and can account for future changes in traffic volume or truck-weight distributions. Generally, the procedures de-

scribed in the preceding paragraphs are used in evaluating existing bridges as well as in designing new bridges. Of course, estimates of past and future traffic volume are used to select an appropriate value of  $T$ .

The effect of changes in the volume or weight distribution of the truck traffic during the life of the bridge can be calculated from

$$\sum \frac{T_n L_n}{N_n} = \frac{1}{365P} \quad (4)$$

in which the subscript  $n$  denotes a period of constant volume and distribution of traffic. Any number of periods of constant traffic can be used.  $L$  for the last period is the unknown in Equation 4.

Fatigue considerations do not restrict the weight of the trucks that are permitted to cross the bridge. If this weight is restricted by other considerations, the restricted weight should be used in estimating the remaining fatigue life.

## 4. DESIGN AND FABRICATION PRINCIPLES

### FABRICATION PRACTICE RELATED TO FATIGUE OF NON-FRACTURE-CRITICAL MEMBERS

Normal fabrication practice for welded, bolted, or riveted steel highway bridges is specified in Division II, Section 10 of the AASHTO specifications.<sup>8</sup> In addition, fabrication practice for welded bridges is covered in more detail in the AWS specifications<sup>51</sup> as modified by AASHTO,<sup>24</sup> and the repairs permitted on steel plates before fabrication are described in an ASTM standard.<sup>23</sup> Several aspects of normal fabrication practice that relate to fatigue behavior are discussed in this article. Special fabrication procedures that are required for fracture-critical members<sup>52</sup> are discussed later under fracture-critical members.

#### Plate Surfaces and Edges

Injurious imperfections, such as surface or edge pits, nicks or gouges, should be repaired as specified by ASTM A6<sup>23</sup> before rolled plates or shapes are shipped from the steel producer to the fabricator. Similar imperfections that result from handling during fabrication should be repaired in the same way to assure that the material will provide a fatigue strength consistent with Category A.

The welding specifications<sup>24,51</sup> give extensive provisions defining when and how internal discontinuities exposed at cut edges must be repaired. These provisions apply to internal discontinuities that were present before the plate was cut and are parallel with the uncut plate surfaces. Discontinuities less than 1-inch long need not be repaired regardless of their depths. Longer discontinuities over 1/8-inch deep must be repaired by grinding or by gouging and welding. Discontinuities deeper than 1 inch are gouged and welded only

to a depth of 1 inch. The unwelded portion that is deeper than 1 inch, and any internal discontinuity that does not extend to within 1 inch of a cut edge, is permitted to remain unless the total area of all such discontinuities exceeds 4% of the plate surface area. If this limit is exceeded, the plate is rejected. Although they may be fairly large, the internal discontinuities permitted to remain have little effect on fatigue strength if the plate is stressed in its plane so that the discontinuity is parallel with the direction of stress. If the plate is stressed in the through-thickness direction, however, such discontinuities greatly reduce fatigue strength.

The welding specifications<sup>24,51</sup> require that the surface roughness of oxygen-cut (flame-cut) edges not exceed 1000 microinches<sup>21</sup> for plates up to 4-inches thick and 2000 microinches for thicker plates. Surface roughness exceeding these limits and occasional shallow notches or gouges must be repaired by proper machining or grinding.<sup>24,51</sup> Within certain depth limits, deeper notches can be repaired by welding.<sup>24,51</sup> Oxygen-cut edges with depths beyond these limits are unacceptable and cannot be repaired.<sup>24,51</sup> Corners of oxygen-cut edges on main stress-carrying members must be provided with 1/16-inch chamfers.<sup>24</sup> This removes shallow notches that tend to occur along such corners and can initiate fatigue cracks. Thus, fabrication practice is consistent with Category A for oxygen-cut plates up to 4-inches thick, but not for thicker plates. To assure that these thicker plates meet the fatigue-strength requirements for Category A, the engineer must specify on the contract drawings that oxygen-cut edges must be ground to a maximum surface roughness of 1000 microinches. Otherwise, the thicker plates could probably be included in Category B, although no fatigue

data are available to verify this.

Surface requirements for sheared edges are not mentioned in the fatigue or fabrication provisions of the AASHTO specifications,<sup>8,24</sup> but such edges should meet the same limitations on notches and surface roughness as oxygen-cut edges. To assure this, the engineer must include an appropriate note on the contract drawings.

### Groove Welds

Articles 2.5, 9.17, and 9.21 of the welding specifications<sup>24,51</sup> require that all transverse groove welds, except unstressed welds specifically identified on the drawings, have complete joint penetration and be fully inspected by radiographic or ultrasonic procedures. Thus, such welds qualify for Category B if the contract drawings show that the welds must be ground flush, and for Category C if not. To assure complete penetration, groove welds welded from one side must be provided with a steel backing (backup bar) or another type of backing demonstrated to be satisfactory by qualification tests, and groove welds welded from both sides must be back gouged.<sup>51</sup>

The designer should be cautious in permitting partial-penetration transverse groove welds in components intended for purely architectural purposes because such components may actually carry stress and cause fatigue cracking. For example, fatigue cracks have been caused by incomplete penetration in transverse groove-welded splices in longitudinal stiffeners in positive-moment regions where they were used for architectural reasons alone.<sup>17</sup>

The welding specifications<sup>24,51</sup> permit partial-penetration groove welds for longitudinal joints, such as flange/web joints in plate or box girders, and such welds need not be fully inspected by radiographic or ultrasonic procedures. Thus, fabrication practice for such welds is consistent with Category B.

The welding specifications<sup>24,51</sup> also permit partial-penetration longitudinal groove welds joining an attachment to a member. If such attachments are provided with a transition radius of 6 inches or more, and are intended for inclusion in Categories B or C, however, these groove welds should have complete joint penetration. To assure this, an appropriate note should be made on the plans.

The welding specifications require that steel backing (backup bars) used for transverse groove welds must be removed and that the joints must be ground smooth. This is necessary to avoid a stress concentration not anticipated in fatigue Categories B and C.

Steel backing used for longitudinal groove welds need not be removed, but is subject to the same fatigue provisions as the member.<sup>24,51</sup> The backing must be continuous or must be spliced by complete-penetration groove welds before being attached to the base metal. This is necessary to avoid the high stress concentration that would occur at a gap in a discontinuous backing.<sup>52</sup> Such a stress concentration is not anticipated in the fatigue provisions.<sup>8</sup> A similar stress raiser occurs where a backing bar is butted against a diaphragm or

other transverse plate.<sup>14</sup> Although this detail is not specifically prohibited in the welding specifications,<sup>24,51</sup> it should be avoided because it can cause early fatigue cracking.<sup>17,53</sup>

Welds used to attach steel backing to base metal in longitudinal joints must be continuous for the length of the backing.<sup>24</sup> This practice avoids the stress raisers that occur at the ends of intermittent welds.<sup>17</sup> However, stress raisers do occur at the ends of continuous welds used to attach backing for joints that do not extend the entire length of the member, such as those for longitudinal attachments. The effects of such stress raisers on fatigue behavior has not been investigated.

Glass tape backing can produce a very smooth surface,<sup>14</sup> but can be used instead of steel backing only if demonstrated as acceptable by weld-joint qualification procedures.<sup>24,51</sup> No fatigue data are available on joints made with such tape.<sup>14</sup>

Complete-penetration groove welds in prequalified T or corner joints must be reinforced by fillet welds,<sup>51</sup> which provide a better contour at the corners of the intersecting plates. These fillets improve the fatigue behavior of such joints in resisting lateral-bending or axial-tension stresses applied to the stem of the T or to either of the corner plates. As discussed later, lateral-bending stresses often occur due to secondary distortions, especially twisting of flanges.

### Prohibited Welds

According to the welding specifications,<sup>24,51</sup> no welds are permitted on the work except 1) welds detailed on the approved shop plans, 2) welds used in the repair of base metal or welds as authorized by the applicable codes,<sup>23,24</sup> and 3) other welds approved by the responsible engineer. This provision is necessary to avoid fatigue failures initiated by welds unknown to the engineer.

Certain types of welds, that have low fatigue strengths, are specifically prohibited in bridges:<sup>24,51</sup> 1) butt joints not fully welded throughout the cross section, 2) intermittent groove welds, 3) intermittent fillet welds except those used to attach stiffeners (presumably vertical stiffener only) to webs, and 4) plug and slot welds on primary tension members.

### Workmanship

The welding specifications<sup>24,51</sup> include several requirements for good workmanship that are very helpful in avoiding fatigue failures. Temporary welds, including tack welds, are subject to the same welding procedure requirements as final welds. Tack welds not incorporated into final welds must be removed in such a manner that the base metal is not nicked or undercut. Other temporary welds must be removed when required by the engineer and the surface must be finished flush. In bridge construction, extension bars and runoff plates must be removed, and the ends of the welds made smooth and flush with the edges of abutting parts. Cracks and blemishes caused by arc strikes must be ground to a smooth contour. The importance of properly repairing areas damaged by fabrication procedures, such as tack welding, is illustrated by the

results of a recent fatigue test on an electroslog-welded girder; early fatigue cracking initiated at a gouge caused by knocking off a tack-welded strongback.<sup>27</sup>

The welding specifications<sup>24,51</sup> require that abutting parts of butt joints be carefully aligned, but permit a misalignment not exceeding the lesser of  $\frac{1}{8}$  inch, or 10% of the thickness of the thinner part, when bending due to misalignment is restrained as it in butt joints of girder flanges. Fatigue tests have shown that misalignment has a large detrimental effect if the bending it causes is not restrained, but only a minor effect if it is restrained.<sup>54</sup>

### **Bolted Joints**

Only high-strength bolts of ASTM A325 or A490 steels<sup>25</sup> are permitted in bolted joints subjected to fatigue loadings.<sup>8</sup> Regardless of whether the bolts are used in slip-resistant (friction) or bearing joints they must be tightened sufficiently to produce a bolt tension at least 70% of the specified minimum tensile strength of the bolt.<sup>29</sup> The AASHTO construction provisions<sup>8</sup> give procedures for bolt tightening and inspection to assure that the required minimum bolt tension is achieved. In addition, AASHTO specifies<sup>8</sup> nine different classes of surface conditions that are acceptable for the contact surfaces of slip-resistant joints; the class must be shown on the plans for all slip-resistant joints. These requirements for bolt tension and contact surface conditions assure that slip-resistant joints develop sufficient friction to provide a fatigue strength consistent with Category B. Contact surfaces for bearing joints need not satisfy the requirements of these classes, but must be free of loose scale, burrs, dirt, and other foreign material not including paint.<sup>8</sup>

The construction provisions<sup>8</sup> require the use of sub-punched (or subdrilled) and reamed holes with certain fabrication procedures and joint details, but permit the use of full-size punched or drilled holes with others. In all cases, however, the holes must be clean cut without ragged edges or burrs, and must be properly aligned.<sup>8</sup> These provisions help to prevent premature fatigue cracks from initiating at the holes.

### **Contract Plans**

As indicated in the preceding paragraphs, many of the requirements for various fatigue detail categories are consistent with normal fabrication practice and need not be specified on the design or working drawings. Other requirements, including the following, must be specifically stated on the plans: 1) the removal of reinforcement by proper grinding for transverse groove welds intended for Category B, 2) use of complete-penetration longitudinal groove welds with attachments having transition radii of 6 inches or more and intended for Categories B or C, 3) grinding of the weld ends at transition radii for details in various categories, and 4) the finishing to a surface roughness not exceeding 1000 microinches<sup>21</sup> for oxygen-cut edges of plates thicker than 4 inches intended for Category A.

Although not required, notes pertaining to some specified fabrication requirements may help to prevent these requirements from being overlooked and possibly

causing fatigue failures. A few examples of such notes follow: 1) No welds are permitted except as shown on these plans or approved by the engineer, 2) all groove welds must have complete joint penetration except as shown on these drawings, 3) all tack welds not incorporated into final welds must be removed and the base metal ground flush.

## **FABRICATION PRACTICE RELATED TO FATIGUE OF FRACTURE-CRITICAL MEMBERS**

Steel tension members or tension components of members whose failure would be expected to result in collapse of the bridge are considered to be fracture critical. AASHTO requires special procedures for the fabrication and inspection of such members.<sup>52</sup> Special welding procedures are specified. Weld repairs of base metal at the producing mill are not permitted. Repair welding during fabrication generally requires prior approval by the engineer and is tightly controlled. Extensive nondestructive testing by qualified personnel is required.

These special fabrication and inspection procedures assure high-quality fabrication and thereby improve the safety of the member with respect to fatigue, as well as to brittle fracture. Furthermore, fracture-critical members are designed for the allowable stress ranges for nonredundant members which are well below those for redundant members. This combination of high-quality fabrication and low allowable stress ranges greatly reduces the risks of a fatigue failure in a fracture-critical member.

## **DETAILS NOT COVERED BY AASHTO**

Three simple details not specifically covered by AASHTO,<sup>8</sup> but used in more complex bridge details, are discussed in this section: cope holes, cruciform joints, and bolted attachments.

### **Cope Holes**

Cope holes are often used, at such locations as butt splices in beams, where fillet or groove welds intersect. The purpose of such holes is to provide access for welding and to avoid 1) possible weld imperfections at a location that is difficult to weld properly, and 2) high shrinkage strains that tend to occur at weld intersections. Fatigue cracks have initiated at weld intersections in highway bridges.<sup>55</sup> A cope hole can also prevent a crack that initiates in one weld from propagating into an adjacent perpendicular plate. For example, a cope hole in a longitudinal stiffener can prevent cracks that initiate at a welded joint between this stiffener and a transverse stiffener from propagating into the girder web.

AASHTO<sup>8</sup> does not specifically cover cope holes. If a cope interrupts a longitudinal fillet or groove weld, it might be classified as a weld end and included in Category E. However, available fatigue data indicate that this is too conservative. Specifically, the results of 136 tests<sup>18-19,26</sup> conducted at the University of Illinois on butt-spliced welded beams with cope hole interrupting the flange/web welds are consistent with Category C

(Fig. 4). The proposed European fatigue code<sup>46</sup> includes cope holes (interrupting longitudinal flange/web type welds) together with several AASHTO Category C details, such as transverse stiffeners and attachments (thicker than ½ inch) in its Class 72; the allowable SN curve for Class 72 is 20 percent below that for Category C.

Cope holes that interrupt longitudinal welds should have good-quality oxygen-cut or machined edges. Nicks should be repaired by grinding. To be consistent with the test results cited above, the cope holes should be semicircular and have a radius not exceeding 1 inch. Preferably, the cope hole should be cut before the longitudinal weld is made.

Cope holes that interrupt transverse welds, such as the fillet welds between a transverse stiffener and the tension flange, provide the beneficial effect of eliminating a weld intersection without producing a stress raiser in the longitudinal direction. Such cope holes can be conveniently made by chamfering a plate corner to produce a triangular hole. It has been recommended that the cope distance be 2 inches or 4 to 6 times the web (or main plate) thickness, whichever is larger.<sup>17</sup>

#### **Cruciform Joints**

Fillet- or groove-welded cruciform joints are not specifically covered by the AASHTO specifications,<sup>8</sup> but occur in several bridge details. In such a joint, the axial stress in a main plate is transmitted through an interrupting plate that is connected to the main plate by fillet or groove welds (Fig. 5a). This joint differs from a transverse-attachment, or stiffener, detail in which the main plate is not interrupted (Fig. 5b).

If complete-penetration groove welds are used in the cruciform joint, fatigue cracking initiates at the toe of the welds, and the fatigue behavior of the joint is similar to that of a transverse stiffener or attachment. Thus, it is appropriate to include this joint in Category C. The proposed European code<sup>46</sup> does specifically include such joints together with transverse stiffeners and attachments (thicker than ½ inch) in Class 72.

If fillet welds are used, however, a large notch perpendicular to the direction of stress is created by the lack of fusion. The controlling fatigue strength of the joint can be determined either by cracking initiated at the root of the weld as a result of the notch or by cracking across the throat of the weld.<sup>56</sup> Both types must be checked. The first type of cracking is related to axial stress in the loaded plates, and the fatigue strength of the joint may be considerably below that for Category C.<sup>56</sup> The proposed European code<sup>46</sup> includes this case in Class 57, which is slightly above Category E. The second type of cracking is controlled by shear in the welds as defined by AASHTO Category F. The slope of the SN curve for this category is different from that of the other categories. In contrast, the proposed European code<sup>46</sup> uses an SN curve that has the same slope as the others, but is 10 percent below the SN curve for AASHTO Category E'.

This type of joint involves through-thickness stresses in the interrupting plate. Such stresses are

undesirable because the plate may contain laminations that could cause early cracking due to weld shrinkage or fatigue stresses. However, if the interrupting plate is a narrow plate, such as a stiffener, any laminations that exist are likely to be exposed during cutting.

#### **Bolted Attachments**

While bolted attachments are not specifically mentioned in the AASHTO fatigue provisions, they are generally presumed to be covered by the provisions for bolted slip-resistant or bearing connections, that are based entirely on data for symmetric butt joints (double-lap joints). However, the behavior of high-strength bolted attachments differs from that of high-strength bolted butt joints in two ways.

First, the stresses in bolted attachments, as well as in welded attachments,<sup>53</sup> are controlled by compatibility requirements. Specifically, the elongation of the attachment (minus any slippage in the bolted detail) must equal the elongation in the main plate between the attachment ends. The stresses in the bolted butt joint, on the other hand, are not controlled by compatibility requirements. Instead, all of the force in the main plate must be transmitted through the splice plates.

Second, the bolted joint can be readily classified as either a slip-resistant or bearing joint, depending on how many bolts are used to carry the desired joint force, but the bolted attachment cannot be so readily classified as a slip-resistant or bearing detail with respect to the stress in the main plate. This is because the attachment is not designed to carry any specific force in that direction, and the force that actually is carried cannot be accurately computed. Therefore, it is somewhat confusing to apply the AASHTO<sup>8</sup> bolted-connection provisions to bolted attachments.

For bolted attachments, as well as welded attachments,<sup>53</sup> the force in the attachment, and the fatigue strength of the detail, depends on the rigidity of the attachment plate or angle. Rigid attachment plates or angles cause high forces and low fatigue strengths. However, microslippage that occurs between the attachments and main plates, especially at the attachment ends, greatly reduces the stress concentrations at these ends.<sup>29</sup> Consequently, bolted attachments have much higher fatigue strengths than similar welded attachments. Therefore, it is generally reasonable to assign bolted attachments to Category B, based on the nominal (gross section) longitudinal stress in the main (web or flange) plate. By using the nominal stress as the governing stress, the necessity for classifying the detail as a slip-resistant or bearing connection is eliminated. However, since various types of bolted attachments have not been fatigue tested, it would be prudent to minimize geometric stress concentrations, or use a lower fatigue category, for particularly severe bolted attachment details. For example, a curved transition could be provided for lateral-bracing gusset plates bolted to girder flanges.

#### **SECONDARY BENDING**

Secondary bending results from either 1) partial fixity

at beam or truss joints that are assumed to be pinned, or 2) distortions of various members of the bridge, especially bracing members.<sup>17</sup> Secondary bending stresses usually have little effect on the static strength of the bridge and are not calculated in the design. They can, however, cause fatigue cracking in either secondary bracing members or main members. In fact, many of the fatigue cracks that have occurred in actual bridges have resulted from secondary bending.

Since secondary bending stresses are not usually calculated, the normal AASHTO fatigue-design procedures<sup>8</sup> cannot be applied directly. Instead, the bridge design must be systematically reviewed, after the framing plan has been completed, to identify and correct potential fatigue problems due to secondary bending. To provide guidance for such a review, the relevant general principles are discussed in this section and applied to various specific details in the next section.

#### Partial End Fixity

The behavior of a typical "pinned" joint connecting a beam to a girder web is illustrated in Figure 6.<sup>29</sup> The moment/rotation relationship for the joint is given by the solid curved line, which is specifically for a simple web connection. (A double seat-angle connection to the beam flanges would provide a stiffer joint represented by the dashed curved line.) The relationship between the end moment and end rotation for the beam is represented by the solid straight line. The intersection of the straight and curved lines defines the actual end moment and rotation for the case under consideration. A stiffer connection type results in a higher end moment and lower rotation.

The end moment can cause fatigue cracking in 1) the connecting angle or plate, 2) the beam itself, or 3) the bolts (or weld) attaching the connecting angle to the girder web. Reducing joint stiffness improves fatigue behavior with respect to all three failure modes. This can be accomplished by using 1) the most flexible type of connection (simple web connection), 2) the smallest connecting angle thickness consistent with static design, and 3) the minimum number of bolts necessary to carry the shear.

Cracking in the connecting angle can be further minimized using a large gage for the outstanding leg (distance from angle corner to first row of rivets) in the top third of the beam; a minimum value of

$$g = \sqrt{\frac{Lt}{12}} \quad (5)$$

is recommended.<sup>17</sup> In this equation,  $g$  is the gage in the top third of the beam depth (and also in the bottom third if tensile stresses can develop in that region),  $L$  is the span in inches, and  $t$  is the thickness of the angle in inches.

Cracking in the beam itself usually occurs when the bending strength of the beam has been greatly reduced by coping the flanges to facilitate the connection. Therefore, avoiding such copes, or suitably strengthening coped beams, prevents such crackings.<sup>17</sup>

Fatigue failures of the bolts attaching the angle to the

girder web result from direct tension loads caused by the end moment. To avoid such failures, the bolts must be properly tightened since this reduces the variation of stress caused in the bolt by the cyclic tension loads.<sup>29</sup>

The end moments that develop due to partial fixity in "pinned" truss joints are similar to those in "pinned" beam joints. However, the joint rotations that must be accommodated in truss joints result from member shortening rather than lateral loading on the members and, therefore, are much smaller.

#### Member Distortions

Distortions of various members in a bridge can cause lateral bending of webs and gusset plates. Usually the lateral bending in the web results from twisting of the flange, lateral movement of the flange, or out-of-plane distortion of the web. Lateral bending in gusset plates usually results from out-of-plane movements imposed on these plates by the members connected to them. Several specific cases are illustrated in Figure 7; most of these are discussed in detail in Reference 17.

Cross bracing (and to a lesser extent, diaphragms) between adjacent girders cause out-of-plane movements in the girder webs when the girders deflect different amounts. Similarly, traffic loadings can cause lateral bracing members to impose out-of-plane movements on the horizontal gusset plates to which they are attached, even though such bracing is designed only to resist lateral buckling and/or wind loading. The out-of-plane movements caused by both types of bracing are usually much greater in curved and skewed bridges than in straight bridges. Vibration of lateral bracing excited by traffic loadings can also cause out-of-plane movements and fatigue cracking.<sup>17</sup> Horizontal traffic loadings, especially on curves, can cause lateral movements of the flanges of floor beams supporting the deck.

As illustrated in Figure 7, lateral forces or moments imposed at locations away from the girder supports can usually be accommodated by twisting of the cross section as a whole without the development of large lateral bending stresses in the web. At supports where twisting of the cross section is prevented, however, large lateral bending stresses can occur in the web, especially if a portion of the web is restrained by stiffeners, connection plates, or connection angles, so that all of the imposed rotation must be accommodated in a short length of the web. The magnitude of the stress varies inversely with the gap distance between the flange/web weld and the end of the stiffener, connection plate, or connection angle weld. Thus, lateral bending stresses in the web can be minimized by providing an adequate gap. A minimum gap of 4 inches has been recommended.<sup>17</sup> Alternatively, lateral bending stresses in the web can be eliminated by welding the stiffener or connection plate to the flange.

Similarly, the lateral bending stresses caused in bracing gusset plates were thought to vary inversely with the gap between the end of the bracing and the girder web or flange to which the gusset is attached. However, a recent finite-element study, and fatigue tests of web gusset plates, suggest that the lateral bending stresses



imposed by the bracing are small and are not greatly affected by the gap.<sup>14</sup>

The lateral-bending fatigue strength depends on the type of fillet provided at the intersection of the web and flange or of the gusset plate and web. For rolled shapes, which have a smooth generous fillet, the fatigue strength approaches that of Category A.<sup>17</sup> For fillet welded or complete-penetration groove welded flange/web or gusset/web joints, the fatigue strength is probably equal to that of Category C.<sup>17</sup> Partial-penetration groove welds, such as are used at the corners of box girders, usually do not have a fillet and often have a lack of fusion that is equivalent to a crack at the corner. Consequently, such joints have a low lateral-bending fatigue strength and are particularly susceptible to secondary bending problems.

### GOOD AND BAD DESIGNS FOR COMMON TYPES OF DETAILS

Table V identifies the particular details that must be checked for fatigue in five common types of beam and girder bridges. Typical cross sections and details for such bridge types are shown in Volume II of this Handbook and in other U. S. Steel publications.<sup>57,58</sup> Since plate girder bridges framed with cross beams and stringers are not covered in these publications, a typical cross section for such a bridge is shown in Figure 8.

Suggested good designs for the details used in these types of bridges are discussed in the following paragraphs. Poor designs for these details, and other details that have caused fatigue problems, are also discussed. For some types of details, several different designs, of increasing cost but decreasing fatigue severity, are suggested so that the designer can select the lowest cost detail consistent with his needs. Generally, a detail design should be chosen that provides sufficient fatigue strength so that the girder cross section need not be increased to accommodate fatigue.

The suggested designs are based on the current (1984) AASHTO fatigue provisions<sup>8</sup> supplemented by other current information. Judgment is necessarily involved in applying the AASHTO provisions to many of the details. Other details, such as cope holes, are not specifically covered by AASHTO<sup>8</sup>. In such cases, however, the basis for the suggested design is indicated. In the figures that illustrate suggested designs the noted fatigue categories relate to the effects of the detail on the main longitudinal members.

During recent years, several details have been reclassified into new AASHTO categories. For example, fillet-welded attachments with a transition radius exceeding 2 inches were recently moved to Category D from higher categories. Generally, the original classifications were based on judgment supported by limited test data on related details. Because of the complex nature of fatigue, such reclassifications are inevitable, and will probably continue in the future as more information is obtained. Similarly, some of the suggestions made herein may require modification as more information becomes available in the future.

### Stiffeners

Suggested stiffener details are shown in Figure 9. Transverse intermediate and bearing stiffeners should be attached to the girder web by fillet welds. Preferably, these welds should be continuous, but the specifications<sup>24,51</sup> permit intermittent welds for intermediate stiffeners. Bearing stiffeners should be tight fitted (milled) or complete-penetration welded to the bottom flange as specified in AASHTO<sup>8</sup> Article 10.34.6.1 to provide a satisfactory transfer of the bearing force into the stiffeners. Generally, bearing stiffeners are also fitted to the top flange. Intermediate stiffeners should be tight fitted to the compression flange as specified in AASHTO<sup>8</sup> Articles 10.34.4.6 and 10.48.5.5, but are generally stopped a short distance from the tension flange to avoid the cost of tight fitting. To minimize fabrication costs, the stiffeners are not welded to either flange unless they are used as attachments, which will be discussed later. To reduce the chances of fatigue cracks occurring at the ends of the stiffener welds due to twisting or lateral movements of the flanges during shipping or handling,<sup>17</sup> a gap of 4 to 6 times the web thickness should be provided between the end of the stiffener weld and the near edge of the flange/web welds as specified in AASHTO<sup>8</sup> Articles 10.34.4.9 and 10.48.5.5. Such a gap is needed even when the stiffener is "tight fitted" to the flange because the fitting tolerance<sup>51</sup> is sufficient to permit significant lateral bending of the web.<sup>17</sup> Transverse stiffeners have been widely fatigue tested and can be classified as Category C.

When longitudinal stiffeners are terminated in contraflexure regions that are subjected to stress reversals, the ends must be designed to resist fatigue. Four different longitudinal stiffener end details are suggested in Figure 9. Only the first of these stiffener details, the Category E detail, has been fatigue tested as such. The classification of the other three is based on the general category of fillet or groove welded attachments with transition radii. The last two details, which are classified as Categories C and B, require that a short distance near the end of the stiffener be complete-penetration groove welded rather than fillet welded. A suggested procedure for fabricating these two details is illustrated in Figure 10 and described below.

First, bevel the portion of the stiffener that will be groove welded as required by the welding specifications.<sup>51</sup> A notch will exist where the bevel is stopped at Location A in Figure 9. This notch should be ground or gouged to a smooth rounded contour to avoid weld flaws at that location. Thus, the end of the groove weld will be similar to the end of a weld repair and should not cause a fatigue problem.

Second, cut the end of the stiffener to the proper radius leaving enough material to permit a satisfactory groove weld where the cut edge becomes tangent to the web. Third, manually groove weld the end portion, starting at the end of the bevel and moving toward the end of the stiffener. Do not use a back-up bar; instead, back gouge as required by the AWS specifications<sup>51</sup> to

achieve a satisfactory complete-penetration groove weld.

Fourth, place submerged arc fillet welds on both sides of the stiffener for its entire length, including the end portions already groove welded. This will provide fillet weld reinforcement to the groove welds, as required<sup>51</sup> for prequalified T joints, and should result in a smaller stress raiser than would result if the fillet welds were stopped where the groove welds started.

Fifth, grind the weld and stiffener material to a smooth contour where the radiused stiffener end becomes tangent to the web. By groove welding this region before the fillet welds are placed, the lack of fusion and weld imperfections that would otherwise occur at this critical location<sup>14</sup> are avoided. A minor stress raiser is expected to occur where the groove weld ends, but is not expected to be critical because it is away from the geometric stress raiser at the end of the stiffener. Nevertheless, it would be desirable to perform fatigue tests on this detail design to verify its fatigue strength.

Figure 9 shows an experimental stiffener end detail consisting of a smooth round hole at the end of a fillet welded longitudinal stiffener. This detail would be easy and cheap to fabricate; the hole could be drilled before the stiffener is attached. The detail has not been used in practice or fatigue tested, but would probably qualify as a Category B or C detail. The hole should reduce the stress concentration at the end of the stiffener in the same way that a circular hole reduces the stress concentration at the end of an existing fatigue crack. As discussed under "Repair of Fatigue Cracks," a hole is widely used to arrest the growth of existing fatigue cracks.

Longitudinal stiffeners are sometimes shop spliced before they are attached to the web. Such splices should be complete-penetration groove welded and nondestructively inspected as shown in Figure 11. Such splices qualify for Category C and may be upgraded to Category B by grinding off the weld reinforcement.

As illustrated in Figure 11, longitudinal stiffeners generally should be terminated at both sides of bolted field splices in girder webs. (There is generally little need to make the stiffener continuous across the splice.) If this is done, the appropriate fatigue category for the detail depends on the stiffener end design as discussed earlier. However, if necessary to provide continuity, two plates can be bolted to both ends of the stiffener. Such a detail has not been fatigue tested, but presumably would qualify as a Category B double-lap bolted joint.

Most intersections between transverse and longitudinal stiffeners occur in compression regions and need not be designed to resist fatigue. However, some intersections occur near points of contraflexure and are subjected to stress reversals, so they must be designed to resist fatigue. Several designs for stiffener intersections are suggested in Figure 11. The simplest approach is to place the two types of stiffeners on opposite sides of the web as shown in Detail 1.

If the stiffeners are placed on the same side of the web, the longitudinal stiffener may be cut at its intersections with transverse stiffeners as specified in AASHTO<sup>8</sup> Article 10.34.5.4 without significantly reducing the effectiveness in resisting web buckling. The interruption in the longitudinal stiffener, however, causes a stress raiser that affects fatigue behavior even if the stiffener is not considered to be carrying stress. Therefore, the fatigue strength of the intersection will generally be controlled by the design chosen for the stiffener end as illustrated in Details 2 and 3.

Sometimes the transverse and longitudinal stiffeners are welded together as illustrated in Details 4 and 5. Detail 4 qualifies as a Category C detail and may be cheaper to fabricate than Detail 3, which requires the somewhat involved procedure discussed earlier. In Detail 4, the longitudinal stiffener is continuous and interrupts the transverse stiffener. However, the transverse stiffener is fillet welded to the longitudinal stiffener to assure that it is fully effective as a transverse stiffener. A chamfer is provided to avoid the intersection of these fillet welds with the fillet welds joining the transverse and longitudinal stiffeners to the web. This chamfer, and the lack of fusion in the fillet welds joining the transverse stiffener to the longitudinal stiffener, do not reduce the fatigue strength of the detail because of their orientation with respect to the longitudinal stresses in the web and stiffener. Although this type of detail has not been fatigue tested, it is essentially the same as a stiffener welded to the tension flange, and therefore, qualifies as a Category C detail.

In Detail 5, the longitudinal stiffener is interrupted by the transverse stiffener. If properly fabricated, this detail could qualify for Category C, but it has two important disadvantages when compared with Detail 4. First, it is probably more expensive to fabricate, because it requires complete-penetration nondestructively inspected groove welds, instead of fillet welds, joining the stiffeners. Complete-penetration groove welds are required because the stress in the longitudinal stiffener must be transferred through the transverse stiffener as discussed under "Cruciform Joints." Second there is less certainty about its fatigue strength, because the effect of the chamfers in the longitudinal stiffeners cannot be established precisely. (These chamfers are necessary to avoid the intersection of the groove welds with the fillet welds joining the longitudinal and transverse stiffeners to the web.) Fatigue cracks have initiated from such an intersection in an actual bridge.<sup>55</sup> Probably, these chamfers have a fatigue strength similar to that of the semicircular cope holes discussed earlier, but this has not been verified by fatigue tests.

Some of the stiffener details discussed previously apply to composite box girders as well as plate girders. Other stiffener details that apply only to composite box girders are suggested in Figure 12. The transverse web stiffener details shown in Figure 12 are similar to those for plate girders and qualify for Category C.

Bottom-flange stiffener arrangements are shown in



Details 3, 4, 5, and 6. AASHTO<sup>8</sup> Article 10.39.4.4.7 specifies that the transverse stiffeners need not be attached to the flange, but must be attached to the web and longitudinal stiffener in a manner sufficient to resist a specified force. Details 3, 4, and 5 are controlled by the transverse fillet welds joining the transverse stiffener to the longitudinal stiffener and to the web or web stiffener. Although none of the details have been fatigue tested, they clearly qualify for Category C unless the transverse stiffener is thicker than 2 inches. If significant secondary bending stresses were to occur in the girder webs as a result of distortions of the cross section, the ends of the welds attaching the transverse stiffeners to the webs would be Category E details. However, AASHTO<sup>8</sup> Article 10.39.3.2.1 indicates that such stresses generally need not be considered.

Detail 6, which has not been fatigue tested, is limited to Category D if the width of the transverse stiffener flange does not exceed 4 inches, and to Category E or lower if it does. The attachment of the transverse stiffener flange to the longitudinal stiffener flange is similar to the attachment of a plate to a girder flange by transverse welds<sup>14</sup> alone and should be avoided if these welds are spaced more than a few inches apart.

### Splices

Flange, web, and cover plates are often spliced to provide transitions in thickness and/or width. Such splices must be made before the plates are joined to other components to form built-up members.<sup>51</sup> Several such splices are illustrated in Figure 13. All have 1 to 2½ tapers as specified in AASHTO<sup>8</sup> Article 10.18.5.5, and qualify for Category B if the weld reinforcement is properly ground off and for Category C if it is not. Most of these types of splices have been adequately fatigue tested, but splices with a thickness taper and the weld reinforcement in place have been subjected to only very limited testing. Furthermore, poor weld contours in combination with the taper could cause higher than expected stress concentrations. Consequently, it is preferable to avoid such splices even though they are permitted by AASHTO.<sup>8</sup> A 24-inch-radius curve can be used instead of the taper for width transitions, but fabricators generally prefer the taper. However, the curved transition must be used with A514 steel.<sup>8</sup> Thickness tapers can be made by 1) chamfering the thicker plate, 2) sloping the weld surface, or 3) a combination of the two.<sup>51</sup> Chamfering, which is usually done by oxygen cutting, adds to joint preparation costs, but reduces the amount of weld metal required.

The weld intersection resulting from a spliced flange fillet welded to a web, has not caused fatigue problems in either laboratory fatigue tests or actual bridges. The inner surface of the splice weld should be ground smooth where the fillet welds cross the splice, but a cope hole should not be provided in the web. Similar intersections between web splice welds and longitudinal stiffener fillet welds are not expected to cause fatigue problems.

Welded butt splices may be used to join portions of long girders or beams either in the shop or field. Bolted

butt splices are used primarily in the field. Suggested designs for such splices are given in Figure 13. Detail 1 uses a semi-circular cope to avoid intersecting welds and to permit weld passes to be made on the under side of the flange. As discussed under "Cope Holes," the cope should be made before the welds, and should have a good-quality oxygen-cut or machined edge. As discussed earlier, this type of splice has been fatigue tested and shown to be consistent with Category C even though it is not specifically covered by AASHTO<sup>8</sup>. Detail 2 uses a circular hole, which may provide a lower stress concentration than the semicircular hole, but this type of splice has not been fatigue tested.

The bolted splice shown in Detail 3, is in Category B based on gross-section stresses if it is designed as a slip-resistant (friction) joint, and based on net-section stresses if it is designed as a bearing joint. Although few bolted beam or girder butt splices have been fatigue tested, such joints are similar to symmetric butt splices which have been widely fatigue tested.<sup>52</sup> If the flanges on the two sides of the joint have different widths, the wider flange should be tapered at a 1 to 2½ slope to meet the narrower flange.

### Cover Plates

Four designs for cover-plate ends are suggested in Figure 14. Details 1 and 2, which are typical of present practice, must be assigned to Category E' if the flange thickness exceeds 0.8 inch, and to Category E if it does not. Category E' details are not permitted in non-redundant bridge members. For Detail 1, the transverse end weld may be omitted if the development length from the theoretical end to the actual end is increased to 2 times the cover-plate width as specified in AASHTO Article 10.13.4. However, the end weld is required in Detail 2.

Details 3 and 4 are experimental details that are expected to provide a better fatigue strength. In Detail 3, the cover-plate end is chamfered to a 1 to 3 slope, fillet welded, and carefully ground to this same slope. The results of fatigue tests conducted at the University of Maryland<sup>59</sup> on 28 cover-plate ends of this design were consistent with Category C. Detail 4 is expected to provide a fatigue strength consistent with Category B, which includes bolted joints. The cover plate should extend a minimum of 1½ times its width beyond the theoretical end, but the longitudinal welds should be stopped at the theoretical end. The bolted end extension should be designed as a slip-resistant (friction) joint to carry the computed force in the cover plate at its theoretical end. Only two fatigue tests have been performed on this type of detail;<sup>60</sup> the results are consistent with Category B. On the basis of these two results the detail was used on a bridge in Holland.<sup>60</sup>

### Diaphragms, Simple Cross Beams, and Brackets

Suggested connection designs for diaphragms, simple cross beams, and brackets are shown in Figure 15. In the diaphragm connections in Details 1, 2, and 3, connection plates or angles are welded or bolted to the main-beam web, and are bolted to the diaphragm web. To minimize lateral bending stresses in the beam web, a

minimum gap of 4 inches should be provided between the connection plate or angle and the nearest flange, or the connection plate should be welded to the flange. The connection plates, of course, are similar to stiffeners and are Category C details with respect to the beams. Although Detail 2 has not been fatigue tested, it should qualify for Category B if the angle is attached to the beam web by properly tightened high-strength bolts as discussed earlier. The diaphragms in Detail 3 can be attached to the connection plates by fillet welds or bolts.

Details 4 and 5 show suggested connections of transverse floor beams to longitudinal girders when the deck is composite or noncomposite. In both cases, the floor beam is bolted to a transverse bearing or intermediate stiffener. In Detail 5, a short horizontal stiffener is fillet welded to the floor beam web to replace some of the bending strength lost by coping the top flange. Such a stiffener is not needed when the deck is composite with the floor beam because the deck strengthens the coped beam.<sup>17</sup>

A similar bolted connection between a transverse floor beam and longitudinal stringers is suggested in Detail 6. Because its bottom flange need not be coped, the stringer generally has adequate bending strength without a web stiffener even if the deck is not composite. To minimize the partial fixity at the end of the stringers, the gage distance to the first row of bolts should be consistent with Equation 5.

When the cross beams are attached to stiffeners, these stiffeners should be welded to both flanges to minimize lateral bending of the web. These welds should preferably be stopped short of the flange edges to avoid possible craters. Chamfers should be provided to avoid weld intersections.

Detail 7 shows stringers placed on top of a floor beam and bracket. The splice plate connecting the top flanges of the floor beam and bracket should not be connected to the girder flange; otherwise, relative movements of the girder flanges and stringers can cause high in-plane bending stresses and fatigue cracking in the splice plate.<sup>17</sup> Alternatively, such fatigue cracking can be prevented by framing the stringers into the floor beam and bracket and embedding the top flanges of the girder, floor beam, and bracket in the slab.<sup>17</sup>

### Cross Frames

Suggested designs for cross frames for plate-girder and composite box-girder bridges are shown in Figure 16. To minimize lateral bending stresses in a plate girder web, the stiffeners to which the cross frames are attached should be fillet welded to both flanges. As shown in Details 1 and 2 of Figure 16, these welds should preferably be stopped short of the flange edges to avoid possible craters at the edge. Chamfers should be provided at the top and bottom of the stiffener to accommodate the flange/web fillet welds and to avoid an intersection between these welds and the stiffener/web and stiffener/flange welds. The chamfer should be a minimum of 1 inch, but 2 inches would usually be preferable.

Many times, the stress in intermediate cross frames is assumed to be small and is not computed. Therefore, the connections of cross frame members to the stiffeners are usually not designed for fatigue and can be made by either high-strength bolts or fillet welds. However, intermediate cross frames in curved-girder bridges,<sup>61</sup> intermediate cross frames in substringer bridges, and end cross frames in all bridges are designed as main members and must be fatigue checked in the usual way. Consequently, such frames might require high-strength bolted connections.

Intermediate cross frames (or diaphragms) and struts are not required in composite box-girder bridges, according to AASHTO Articles 10.39.6.2 and 10.51.6, but they are often used to stiffen the cross section to facilitate handling and erection. If such cross frames are not removed, their effect on the fatigue strength of the box girders must be considered. After the slab has been placed, the box girder has a high torsional rigidity. Consequently, it was formerly believed that the stresses in the cross bracing members will be small, and the stiffeners to which they are attached need not be welded to the flanges. Later, it was demonstrated that fatigue cracks developed in the webs of some in-service box girder bridges at the toe of the fillet welds connecting stiffener to the web, at cross frame locations. Therefore, it is recommended that the web stiffeners be welded to the flanges at cross frame locations.

### Lateral Bracing

Until recently, AASHTO required that bottom-flange lateral bracing be used to carry wind loadings in plate girder bridges when the span length exceeded 125 ft. Now, however, AASHTO<sup>8</sup> does not specify any particular span length beyond which bottom-flange lateral bracing is required. Since lateral bracing attachments tend to be severe fatigue details, and have caused cracking in actual bridges, lateral bracing should be avoided unless definitely needed.

Several suggested lateral bracing attachments are shown in Figure 17. These attachments must be fatigue checked with respect to both the longitudinal stresses in the main member due to traffic loadings, and the transverse stresses applied to the gusset (attachment) plate by wind forces in the bracing. In addition, connections of the bracing members to the gusset plate must also be fatigue checked for wind loadings. These connections can be made either by bolting, which qualifies for Category B, or fillet welding, which qualifies for Category E. Since the wind can blow from either side of the bridge, the bracing connections and the transverse gusset plate stresses are subjected to complete reversals in two-girder bridges. Such reversals can cause relatively high applied stress ranges, but the allowable stress ranges are also relatively high because only 100,000 cycles of wind loading need be considered.<sup>8</sup> For multi-girder bridges, two lines of lateral bracing are usually provided. Each line carries wind from only one direction so that reversals do not occur.

From a fatigue standpoint, lateral bracing can gen-

erally be attached to the flange more effectively than to the web. Bolted and welded attachments to the flange are suggested in Details 1 and 2 of Figure 17. The bolted detail is provided with a transition radius to minimize the geometric stress raiser at the projecting edges of the gusset plate as suggested earlier. Also, as noted, this detail has not been fatigue tested.

Detail 2, which is complete-penetration groove welded and has a 6-inch transition radius, qualifies for Category C with respect to both longitudinal and transverse stresses. The weld must be non-destructively inspected if it is subjected to transverse stresses. As illustrated, the ends of the transition radii must be ground smooth, but the weld reinforcement that exists if both the gusset plate and flange are the same thickness need not be removed. If the gusset plate is thinner than the flange, the groove weld must be reinforced with fillets as required for prequalified groove-welded T joints.<sup>51</sup> This type of welded detail had been adequately fatigue tested.

Detail 2 could be upgraded to Category B for both longitudinal and transverse stresses if 1) the gusset plate is the same thickness as the flange, 2) the weld reinforcement is ground off, and 3) the transition radius is increased to 24 inches. On the other hand, Detail 2, must be downgraded to Category D for both longitudinal and transverse stresses if the gusset plate is thinner than the flange and is connected by fillet welds instead of groove welds. If no transition radius is provided, it must be further downgraded to Category E for the longitudinal stresses. None of these modified details, however, has been fatigue tested. A gusset plate fillet welded to the flange surface, as illustrated in Detail 3, has a very low fatigue strength and is not recommended.

Lateral bracing attachments to the web are usually made at transverse stiffener locations where cross frames are also attached. Details 4, 5, 6, and 7 illustrate such attachments, which may be controlled by either the ends of the gusset plate or the gusset/stiffener intersection. Gusset-plate ends are similar to stiffener ends. Consequently, the end detail necessary to achieve the desired fatigue category can be selected from Figure 9. However, since the transverse stiffener prevents the detail from achieving a classification higher than Category C, there is no advantage to choosing an end detail with a category higher than this.

Detail 4 utilizes a continuous gusset plate that interrupts the transverse stiffener, which is fillet welded to the gusset. Since this intersection is essentially the same as a transverse stiffener welded to a tension flange, there is little doubt that it qualifies for Category C. Similarly, the gusset-plate ends are expected to qualify for Category C as discussed under "Stiffeners." Therefore, Detail 4 is classified as a Category C detail although it has not been specifically fatigue tested.

Detail 5 utilizes an interrupted gusset plate that is fillet welded to both the web and transverse stiffener. The ends are provided with a 2-inch radius that is ground smooth near the point of tangency. Thus, the

ends are expected to fit into Category D. The attachment is slotted to fit over the stiffener and then fillet welded to it. As discussed under "Cruciform Joints," the classification of this type of fillet-welded joint, which is adversely affected by lack of fusion perpendicular to the direction of stress, is uncertain. Since the stress in this fillet-welded joint is probably less than the stress at the gusset-plate ends, however, the joint is not expected to be more critical than these ends. Similarly, the cope holes are not expected to be more critical than the ends. Therefore, Detail 5 is expected to be a Category D detail, but there is considerably more uncertainty about it than about Detail 4.

Recently, details with a cutout around the stiffener, like that in Detail 6, have been widely used in bridges, and have been fatigue tested.<sup>14</sup> However, the tested details had square ends that controlled the fatigue strength at values consistent with Category E. Consequently, the classification of the cutout itself has not been established. Probably, it is between that of a small semi-circular cope hole (Category C) and that of an interrupted fillet weld (Category E). Thus, Category D seems to be an appropriate classification for Detail 6, but uncertainty remains about this classification.

Detail 7 is a bolted attachment with a cutout around the stiffener. Although this detail may qualify as a Category B detail, it has been shown as Category C because of uncertainty about the effect of the cutout. It has not been fatigue tested.

#### Continuous Cross Beams

Intersections between longitudinal and transverse bending members sometimes must be made in such a way that bending continuity is provided in both directions. For example, intersections between longitudinal beams or girders and transverse rigid frames are sometimes made in this way. Such intersections are very complicated<sup>17</sup> and often have very low fatigue strengths. Furthermore, a large number of different intersection details are possible. In most cases, however, the flange of one member either passes through a cutout in the web of the other or is welded to both sides of the web.

In Detail 1 of Figure 18, which was proposed by the Federal Highway Administration,<sup>13</sup> cross beams are butted against opposite sides of the main girder web. The webs of the cross beams are attached to the web of the main girder by bolted connection angles. Flange splice plates are passed through cutouts in the girder web and are bolted to the top and bottom flanges of both cross beams to provide continuity in that direction.

The cutouts have rounded ends and good-quality oxygen-cut or machined edges to improve fatigue strength. Although most of the moment in the cross beams will pass through the bolted flange splices, some will pass through the web connections. Therefore, the bolts that pass through the girder web will be loaded in direct tension and are subject to prying action.<sup>29</sup> Fatigue criteria for such joints are given in Reference 29. Although this type of detail has not been fatigue tested, it is expected to qualify for Category B or C in

both directions.

Fatigue tests<sup>14</sup> have shown that the attachment of cross beam flanges to girder webs by groove welds or fillet welds, as illustrated in Detail 2, results in very low fatigue strengths (Category E or lower) with respect to the stresses in the girder. Cross beam flanges that are passed through a girder web, but are seal welded to that web as illustrated in Detail 3, have even lower fatigue strengths<sup>14</sup> and should not be used.

#### Trusses

A typical high-strength bolted truss joint is illustrated in Detail 1 of Figure 19. Gusset plates with appropriate curved transitions are provided on both sides of the member so that the joint behaves like the symmetric butt joints on which the AASHTO<sup>8</sup> fatigue provisions for bolted connections are based. Therefore, such joints qualify for Category B.

When a gusset plate is welded to one truss member and bolted to others, as illustrated in Detail 2, its fatigue behavior is similar to that described previously for lateral bracing attachments. The appropriate fatigue category, for the truss member to which the gusset plate is welded, depends on the transition radius and type of weld.

Attachment of the floor beams to the truss members involves many of the different types of attachments that have been classified by AASHTO<sup>8</sup> and discussed previously.

### TREATMENTS THAT IMPROVE FATIGUE PERFORMANCE

Many different treatments have been applied to various details in an attempt to improve their fatigue performance. A comprehensive assessment of such treatments is given in Reference 62. The application of several of the more practical treatments to new bridge members is discussed in this section. The application of some of these treatments, in the repair of cracked members in service, is discussed in the next section. Because these treatments are not presently recognized in the AASHTO specifications,<sup>8</sup> they cannot be used to qualify a detail that would not otherwise satisfy the specifications. However, they could be used to provide an extra margin of safety for key members, and to lengthen the actual fatigue life of the bridge.

#### Peening

Peening has been used successfully to improve fatigue strength in many different applications. To be effective, the peening should be uniform in intensity and coverage. Three types of peening have been used:<sup>62,63</sup> 1) shot peening<sup>64</sup> in which pellets are shot at the surface, 2) single point peening<sup>13,65</sup> in which a single ½-inch diameter rod is applied pneumatically and 3) multiple point peening<sup>65,66</sup> in which 0.08-inch-diameter rods are applied pneumatically. In all of these methods, the peening cold works the surface and causes a thin layer of compressive residual stresses that are balanced by tensile residual stresses below this layer. The peening also closes shallow surface imperfections or cracks.

The surface compressive residual stresses are

superimposed on the applied stresses and thereby improve the fatigue strength as discussed in Section III. The improvement is greatest when the applied tensile stresses, both constant and cyclic, are low enough so that the net cyclic stresses are always in compression. High tensile dead load stresses tend to reduce the effectiveness of peening unless it is done while the dead load stresses are being applied. The tensile residual stresses below the surface can be detrimental to crack extension behavior if the crack front resides or propagates into this region.

For bridge applications, peening is most often applied to the toes of transverse or longitudinal welds, and to groove welds with the reinforcement in place. For such details, increases in fatigue strength (at 2,000,000 cycles) of about 20 to 200 percent have been reported.<sup>62</sup> For fillet-welded details, the improvement that can be achieved by peening the toe is often limited by root cracking.<sup>13</sup> Root cracking is normally less critical than toe cracking but becomes more critical when the toe is improved. Peening requires a lesser degree of operator skill, and is generally cheaper, than the other treatments. Of the three peening methods, the single point method is generally preferable with respect to both cost and effectiveness.<sup>62</sup>

#### TIG Remelting

The gas tungsten arc (TIG) welding process can be used to remelt the toe of a previously deposited fillet weld and thereby eliminate shallow imperfections that often occur at that location. TIG remelting is generally regarded<sup>13,62</sup> as the most reliable treatment for improving the fatigue strength of fillet weld details, but requires greater operator skill and is more costly than peening. In fact, it is estimated that the cost of TIG remelting is about 3 times that of single point peening. Usually, it is necessary to remove mill scale by sand blasting before TIG remelting<sup>13</sup> and to use appropriate procedures<sup>13,67</sup> to help avoid weld craters at critical locations.

Since the improvement due to this treatment is caused by the removal of imperfections, it is not significantly affected by dead load stresses, but it may be limited by root cracking. Increases in fatigue strength (at 2,000,000 cycles) of 40 to 250 percent and 15 to 35 percent, respectively, have been reported<sup>62</sup> for transverse and longitudinal fillet welds. Fisher<sup>13</sup> indicated that fillet welded details, such as cover-plate ends, can be improved by one AASHTO detail category (from E to D, etc.) by TIG remelting.

#### Grinding

Grinding to a smooth contour is a requirement for several types of details covered in the AASHTO specifications.<sup>8</sup> These grinding requirements are discussed elsewhere; only grinding intended to improve other types of details, and not covered by AASHTO, is discussed here. Specifically, the discussion covers grinding to remove imperfections and to improve the contour at the toes of fillet welds.

Grinding can be done with either a rotary disc (typically a 4-inch disc with a 60-150 grit) or a conical burring

bit. Burr grinding is preferred for treating fillet weld toes because disc grinding tends to be erratic and can cause worse conditions than existed before grinding.<sup>62</sup> Some investigators used three successive 30-200 grit polishes after burr grinding to further improve the surface. The cost of burr grinding with and without polishing is estimated<sup>62</sup> to be 3 to 4 and 12 times, respectively, that of single point peening.

Fisher<sup>13</sup> applied burr grinding without subsequent polishing to cover-plate ends, and did not achieve a significant improvement in fatigue strength. Other investigators were able to achieve 40 to 200 percent improvements in the fatigue strengths of various fillet welded details by burr grinding either with or without polishing<sup>62</sup>. Because of its higher cost and less reliable results, however, burr grinding is a less attractive alternative than peening or TIG remelting for improving fillet welded details.

#### **Spot Heating**

As discussed in Section III, welding or flame cutting can cause compressive residual stresses at the end of a previously placed longitudinal fillet weld and thereby improve fatigue strength. A similar beneficial effect can be achieved by spot heating. As mentioned under Peening, the beneficial effects of compressive residual stresses are greatest when the applied tensile stresses are low enough so that the net cyclic stresses are always compressive.<sup>63,68</sup> Improvements of 70 to 200 percent in the fatigue strength (at 2,000,000 cycles) have been achieved with spot heating. However, data on this treatment are very limited, and detailed procedures are not available for applying the treatment to various detail geometries. Therefore, it is less attractive than the treatments discussed previously.

### **REPAIR OF FATIGUE CRACKS**

Selecting the best method for repairing a fatigue crack in a bridge member depends primarily on the crack's size and location. Large cracks may require major repairs such as replacing members, adding bracing to redistribute load, or adding bolted splice plates. In certain cases, it may be desirable to leave the crack unaltered and merely monitor its future growth.<sup>69</sup> Several different methods are available for repairing or arresting cracks that fall between these two extremes. These methods are discussed here.

#### **Grinding**

Steel producers are permitted<sup>23</sup> to remove surface or edge imperfections up to 1/8 inch deep by grinding without replacing the removed metal. Edge or surface fatigue cracks (probably initiated by a nick) not exceeding this depth can easily be repaired in the same way. In fact, even deeper edge cracks could be safely removed by grinding provided that the ground area is well faired with gentle changes in contour. Grind marks perpendicular to the direction of stress should be avoided. Fatigue cracks at the toe of a fillet weld are more difficult to remove successfully by grinding<sup>13</sup> because more abrupt changes in contour are required at the weld.

#### **Peening**

Single point peening has been used successfully to repair fatigue cracks up to 1/8-inch deep at the toe of a fillet weld.<sup>13</sup> It is a simple, effective, and economical way of making repairs, but should be used with caution unless there is reliable evidence to show that the cracks are not deeper than 1/8 inch and that the peening was uniform in coverage and severity to provide compressive stresses along the entire front of a 1/8-inch deep crack. Otherwise, a buried crack will remain and severely limit the remaining fatigue life.

#### **TIG Remelting**

Gas tungsten arc remelting has been shown<sup>13</sup> to be effective in removing cracks up to 3/16 inch deep at the toe of a fillet weld. Again, caution is needed to avoid buried cracks.

#### **Rewelding**

Larger fatigue cracks can often be repaired in the same way that unacceptable welds and internal imperfections are repaired during the original fabrication. The AWS specifications<sup>51</sup> cover such repairs. First, the crack is completely removed by air carbon arc gouging, oxygen gouging, chipping, grinding, or machining. It may sometimes be desirable to use dye-penetrant or magnetic-particle inspection to assure that the crack has been completely removed. Next the gouge is rewelded to its original contour. Subsequent grinding to a smooth contour may sometimes be desirable.

Generally this is the most reliable method of repairing a fatigue crack because the crack can be fully removed and the repaired region restored to its original condition. The repair will extend the remaining fatigue life of the detail, but will not always fully restore the original life because of the effects of accumulated cycles outside of the repaired region. Treatments such as peening, TIG remelting, and grinding can be used after rewelding to further extend the remaining life of the detail. Residual stresses caused by extensive rewelding on an existing bridge could affect the fatigue strength of adjacent details and should be considered in selecting an appropriate repair method. An NCHRP study (Project 12-27) is currently developing detailed guidelines for the weld repair of large cracks in existing bridges.

#### **Drilling Holes**

The growth of full-thickness fatigue cracks in steel plates can be arrested by drilling holes at the crack ends. This technique has been successfully used in many different applications including bridges.<sup>13-14,69</sup> The purpose of the hole is to reduce the very high stress intensity that occurs at the crack tip. Therefore, it is essential that the hole include the crack tip. Since the actual end of a fatigue crack is difficult to detect visually, it is suggested the the near edge of the hole be placed at the apparent crack end to assure that the actual end will be within the hole. Also, it is advisable to dye-penetrant inspect the hole surface to verify that the crack tip has been removed. Hole diameters between 0.5 and 1.0 inch have been used.<sup>13-14,69</sup>

The fatigue category for a circular hole in a plate

generally ranges from B to D depending on the smoothness of the hole edges.<sup>69</sup> A carefully reamed hole qualifies as Category B.<sup>69</sup> Since it is not important that the hole be precisely circular, hand filing can be used if needed to improve smoothness.

The fatigue strength of a crack with circular holes at both ends is less than that of a single circular hole and depends on the length between the outer edges of the holes. If this length is below a limiting value,  $L_{LIMIT}$ , further cracking will not occur. The following equation is proposed to define the limiting length

$$L_{LIMIT} = \frac{200}{S_r^2} \text{ for } L_{LIMIT} \geq 1 \text{ inch} \quad (6)$$

where  $L_{LIMIT}$  is in inches and  $S_r$  is the applied stress range in ksi. The actual maximum stress range occurring in the bridge, rather than an artificially high design value, should be used in this equation. The equation is derived in Appendix A from a stress-intensity threshold developed<sup>14</sup> from a rather limited number of data and therefore should be regarded as approximate. The equation implies a fatigue limit of 14 ksi for a single 1-inch-diameter hole; this is slightly below the fatigue

limit of 16 ksi for Category B.

High-strength bolts with washers can be placed in the drilled holes and tightened by the turn-of-nut method further to reduce the possibility of cracking.<sup>14,69</sup> This produces compressive stresses around the hole, but makes it more difficult to inspect for new cracks. The nonburr side of the washer should be placed against the plate to avoid cracks initiated by the burr<sup>69</sup>.

### Replacing Rivets

The fatigue life of riveted joints in existing bridges can be considerably extended by merely replacing some of the rivets with high strength bolts. The maximum extension can be achieved by replacing all rivets, and repairing all observed cracks in the joint plates. However, life extensions of 2 to 6 times can be obtained by merely replacing rivets at locations where cracks can be observed in the adjacent plate material.<sup>70</sup> With this approach, cracks in the plates need not be repaired unless they extend more than 1 inch beyond a rivet head. The bolts should be tightened by the turn-of-nut method as specified for bolted joints. Washers under the turning elements should be placed with the nonburr side against the plate.<sup>69</sup>

## 5. SUMMARY

Four currently used fatigue-design approaches are described briefly. It is concluded that the stress-life detail-category approach is most convenient for structural applications. In this approach, which is utilized in the AASHTO specifications, typical structural details are grouped according to severity, and allowable stresses corresponding to various design lives are given for each group. The experimental basis for the AASHTO detail categories is discussed in depth. Application of the AASHTO procedures in the design and rating of highway bridges is described, and a proposed new method that more accurately approximates conditions in actual bridges is presented.

Several fabrication practices affect fatigue performance; these practices are identified and discussed. Secondary bending can occur in bridges as a result either of partial end fixity of joints that are assumed to be pinned, or of distortions of various members, especially bracing members. Secondary bending stresses are usually not calculated in design, but can cause fatigue

cracking as discussed in this section.

Suggested good designs are presented for details for 1) stiffeners, 2) splices, 3) cover plates, 4) diaphragms, simple cross beams, and brackets, 5) cross frames, 6) lateral bracing, 7) continuous cross beams, and 8) trusses. Also, poor designs that have caused fatigue cracking are discussed. Some of the discussed details utilize cope holes, cruciform joints, and bolted attachments, that are not specifically covered by AASHTO. Consequently, information is presented on the fatigue strengths of these three simple details.

Several treatments that have been shown to be effective in improving the fatigue performance of new bridge members are described: 1) peening, 2) grinding, 3) TIG remelting, and 4) spot heating. Methods of repairing or arresting fatigue cracks in existing bridge members are also discussed. These include 1) grinding, 2) peening, 3) TIG remelting, 4) rewelding, 5) drilling holes, and 6) replacing rivets with high strength bolts.

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## 7. TABLES

TABLE I  
AASHTO DETAIL CATEGORIES

General Condition	Situation	Kind of Stress	Stress Category (See Table II)	Illustrative Example (See Figure 1)
Plain Material	Base metal with rolled or cleaned surfaces. Flame cut edges with ASA smoothness of 1,000 or less.	T or Rev <sup>a</sup>	A	1, 2
Built-Up Members	Base metal and weld metal in members without attachments, built-up plates, or shapes connected by continuous full or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied stress.	T or Rev	B	3, 4, 5, 7
	Calculated flexural stress at toe of transverse stiffener welds on girder webs or flanges.	T or Rev	C	6
	Base metal at end of partial length welded cover plates having square or tapered ends, with or without welds across the ends			
	(a) Flange thickness < 0.8 in. (b) Flange thickness > 0.8 in.	T or Rev T or Rev	E E'	7 7
Groove Welds	Base metal and weld metal at full penetration groove welded splices of rolled and welded sections having similar profiles when welds are ground flush and weld soundness established by nondestructive inspection.	T or Rev	B	8, 10, 14
	Base metal and weld metal in or adjacent to full penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no greater than 1 to 2½, with grinding in the direction of applied stress, and weld soundness established by nondestructive inspection.	T or Rev	B	11, 12
	Base metal and weld metal in or adjacent to full penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2½ when reinforcement is not removed and weld soundness is established by nondestructive inspection.	T or Rev	C	8, 10, 11, 12, 14
	Base metal at details attached by groove welds subject to longitudinal loading when the detail length, L, parallel to the line of stress is between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev	D	13
	Base metal at details attached by groove welds subject to longitudinal loading when the detail length, L, is greater than 12 times the plate thickness or greater than 4 inches long.	T or Rev	E	13
	Base metal at details attached by groove welds subjected to transverse and/or longitudinal loading regardless of detail length when weld soundness transverse to the direction of stress is established by nondestructive inspection.			
	(a) When provided with transition radius equal to or greater than 24 in. and weld end ground smooth	T or Rev	B	14
	(b) When provided with transition radius less than 24 in. but not less than 6 in. and weld end ground smooth	T or Rev	C	14
	(c) When provided with transition radius less than 6 in. but not less than 2 in. and weld end ground smooth	T or Rev	D	14
	(d) When provided with transition radius between 0 in. and 2 in.	T or Rev	E	14

**TABLE I (cont'd.)  
AASHTO DETAIL CATEGORIES**

<b>General Condition</b>	<b>Situation</b>	<b>Kind of Stress</b>	<b>Stress Category (See Table II)</b>	<b>Illustrative Example (See Figure 1)</b>
<b>Fillet<sup>b</sup> Welded Connections</b>	Base metal at intermittent fillet welds	T or Rev	E	—
	Base metal adjacent to fillet welded attachments with length L, in direction of stress less than 2 in. and stud-type shear connectors	T or Rev	C	13, 15, 16, 17
	Base metal at details attached by fillet welds with detail length, L, in direction of stress between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev	D	13, 15, 16
	Base metal at attachment details with detail length, L, in direction of stress (length of fillet weld) greater than 12 times the plate thickness or greater than 4 in.	T or Rev	E	7, 9, 13, 16
	Base metal at details attached by fillet welds regardless of length in direction of stress (shear stress on the throat of fillet welds governed by stress category F)			
	(a) When provided with transition radius equal to or greater than 2 in. and weld end ground smooth	T or Rev	D	14
<b>Mechanically Fastened Connections</b>	(b) When provided with transition radius between 0 and 2 in.	T or Rev	E	14
	Base metal at gross section of high-strength bolted slip resistant connections, except axially loaded joints which induce out-of-plane bending in connected material.	T or Rev	B	18
	Base metal at net section of high-strength bolted bearing-type connections	T or Rev	B	18
	Base metal at net section of riveted connections	T or Rev	D	18
<b>Fillet Welds</b>	Shear Stress on throat of fillet welds	Shear	F	9

<sup>a</sup>"T" signifies range in tensile stress only; "Rev" signifies a range of stress involving both tension and compression during a stress cycle.

<sup>b</sup>Gusset plates attached to girder flanges with only transverse fillet welds, not recommended.

**TABLE II**  
**AASHTO ALLOWABLE FATIGUE STRESS RANGES**

<b>Redundant Load Path Structures*</b>				
<b>Category See Table I</b>	<b>Allowable Range of Stress, <math>F_{sr}</math> (ksi)<sup>a</sup></b>			
	<b>For 100,000 Cycles</b>	<b>For 500,000 Cycles</b>	<b>For 2,000,000 Cycles</b>	<b>For 2,000,000 Cycles</b>
A	60	36	24	24
B	45	27.5	18	16
C	32	19	13	10
				12 <sup>b</sup>
D	27	16	10	7
E	21	12.5	8	5
E'	16	9.4	5.8	2.6
F	15	12	9	8
<b>Nonredundant Load Path Structures</b>				
<b>Category See Table I</b>	<b>Allowable Range of Stress, <math>F_{sr}</math> (ksi)<sup>a</sup></b>			
	<b>For 100,000 Cycles</b>	<b>For 500,000 Cycles</b>	<b>For 2,000,000 Cycles</b>	<b>For 2,000,000 Cycles</b>
A	36	24	24	24
B	27.5	18	16	16
C	19	13	10	9
			12 <sup>b</sup>	11 <sup>b</sup>
D	16	10	7	5
E <sup>c</sup>	12.5	8	5	2.5
F	12	9	8	7

\*Structure types with multi-load paths where a single fracture in a member cannot lead to the collapse. For example, a simply supported single span multi-beam bridge or a multi-element eye bar truss member has redundant load paths.

<sup>a</sup>The range of stress is defined as the algebraic difference between the maximum stress and the minimum stress. Tension stress is considered to have the opposite algebraic sign from compression stress.

<sup>b</sup>For transverse stiffener welds on girder webs or flanges.

<sup>c</sup>Partial length welded cover plates shall not be used on flanges more than 0.8 inches thick for nonredundant load path structures.

**TABLE III  
AASHTO DESIGN-LIFE CATEGORIES**

<b>Main (Longitudinal) Load Carrying Members</b>				
<b>Type of Road</b>	<b>Case</b>	<b>ADTT<sup>a</sup></b>	<b>Truck Loading</b>	<b>Lane Loading<sup>b</sup></b>
Freeways, Expressways, Major Highways, and Streets	I	2,500 or more	2,000,000 <sup>c</sup>	500,000
Freeways, Expressways, Major Highways, and Streets	II	less than 2,500	500,000	100,000
Other Highways and Streets not included in Case I or II	III		100,000	100,000
<b>Transverse Members and Details Subjected to Wheel Loads</b>				
<b>Type of Road</b>	<b>Case</b>	<b>ADTT<sup>a</sup></b>	<b>Truck Loading</b>	
Freeways, Expressways, Major Highways, and Streets	I	2,500 or more	over 2,000,000	
Freeways, Expressways, Major Highways, and Streets	II	less than 2,500	2,000,000	
Other Highways and Streets	III	—	500,000	

<sup>a</sup> Average Daily Truck Traffic (one direction).

<sup>b</sup> Longitudinal members should also be checked for truck loading.

<sup>c</sup> Members shall also be investigated for “over 2 million” stress cycles produced by placing a single truck on the bridge distributed to the girders as designated in Article 3.23.2 for one traffic lane loading.

**TABLE IV**  
**FATIGUE EQUATIONS AND CONSTANTS FOR PROPOSED NEW METHOD**

Category (1)	$F_{srL}$ in kips per square inch (2)	Constant A (3)
A	12	$240 \times 10^8$
B	8	$105 \times 10^8$
C (stiffeners)	6	$37 \times 10^8$
C (other attachments)	5	$37 \times 10^8$
D	3.5	$20 \times 10^8$
E	2.5	$10 \times 10^8$
E'	1.3	$4 \times 10^8$
F	4	$10 \times 10^8$

Note:  $N = A/F_{sr}^3$ ;  $N$  = estimated minimum number of stress cycles to failure;  $F_{sr}$  = design stress range based on  $W_F$ , in kips per square inch;  $F_{srL}$  = maximum allowable stress range for infinite fatigue life, in kips per square inch; A = constant listed herein.

**TABLE V**  
**DETAILS THAT MUST BE FATIGUE CHECKED**

Type of Detail	Type of Bridge				
	Rolled Beam	Plate Girder			Box Girder
		Multi Girder	Sub- stringers	Cross Beams	
Web/Flange Weld	—	A	A	A	A
Transverse Stiffener (Intermediate; Bearing)	NM/S	S	S	S	S
Longitudinal Stiffener (End; Intersection with Trans- verse Stiffener; Long. Weld)	—	S	S	S	S
Butt Splice	S	S	S	S	S
Cover Plate (End; Longitudinal Weld)	S	—	—	—	—
Bracket for Overhang or Side- walk (Attachment; Connections)	S	S	S	S	S
Diaphragm (Attachment)	A	—	—	—	S
Cross Frame (Attachment; Frame; Connections)	—	A	A	A	S
Cross Beam	—	—	—	A	—
Lateral Bracing (Attachment; Frame)	S	S	S	S	S
Shear Studs	NM/C	NM/C	NM/C	NM/C	NM/C

Symbols:

A = always; S = sometimes; NM = negative moment region only; C = composite.

Notes:

1) Attachment means that the effect of the attachment on the main member must be checked.

2) Frame means that the secondary frame itself must be checked.

3) Connections means that the connection between the secondary frame or member must be checked.

1

2

3

4

5

6

7

8

9

10

11

12

13

14

15

16

17

18

2' Rad. A514 and A517

Groove or Fillet Weld

Diaph. Gusset

Squared End, Tapered or Wider than Flange

Category B

Category E'

Category B

Category B

Category E

Category E\*

Category F (in weld metal)

Category E\* (in base metal)

Category E\* (in base metal)

WELD CONDITION\*

WELD CONDITION*	CAT.
Unequal Thickness - Reinf. in Place	E
Unequal Thickness - Reinf. Removed	D
Equal Thickness - Reinf. in Place	C
Equal Thickness - Reinf. Removed	B

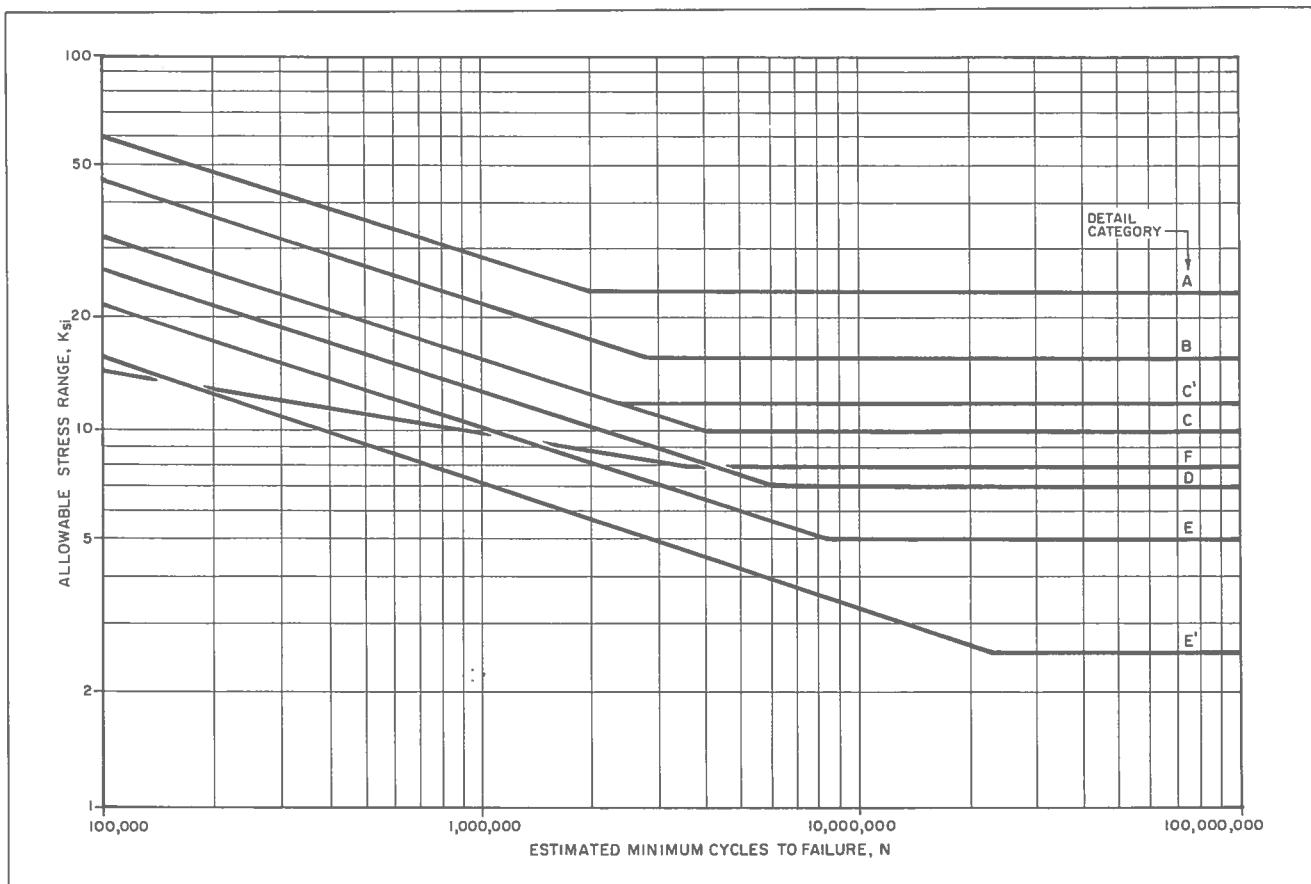
\* For transverse loading - check transition radius for possible lower category

R*	CAT.	
	Fil	Gr
$R \geq 24"$	D	B
$24" > R > 6"$	D	C
$6" > R > 2"$	D	D
$2" > R$	E	E

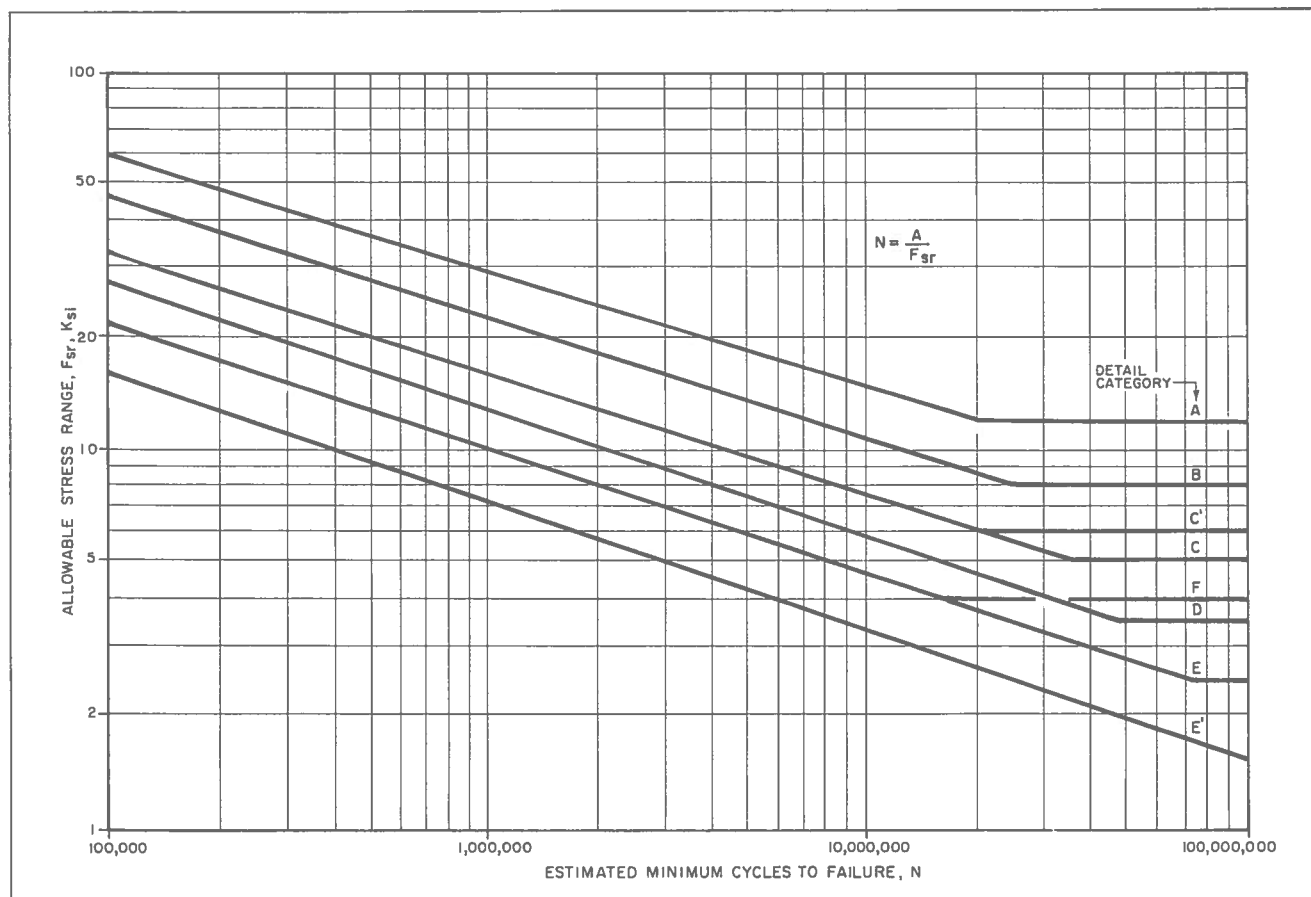
\* Also applies to transverse loading

\* At End of Weld, Has No Length

I/6.100



**Figure 2.** AASHTO allowable SN curves for redundant members.



**Figure 3.** Allowable SN curves for proposed new method.

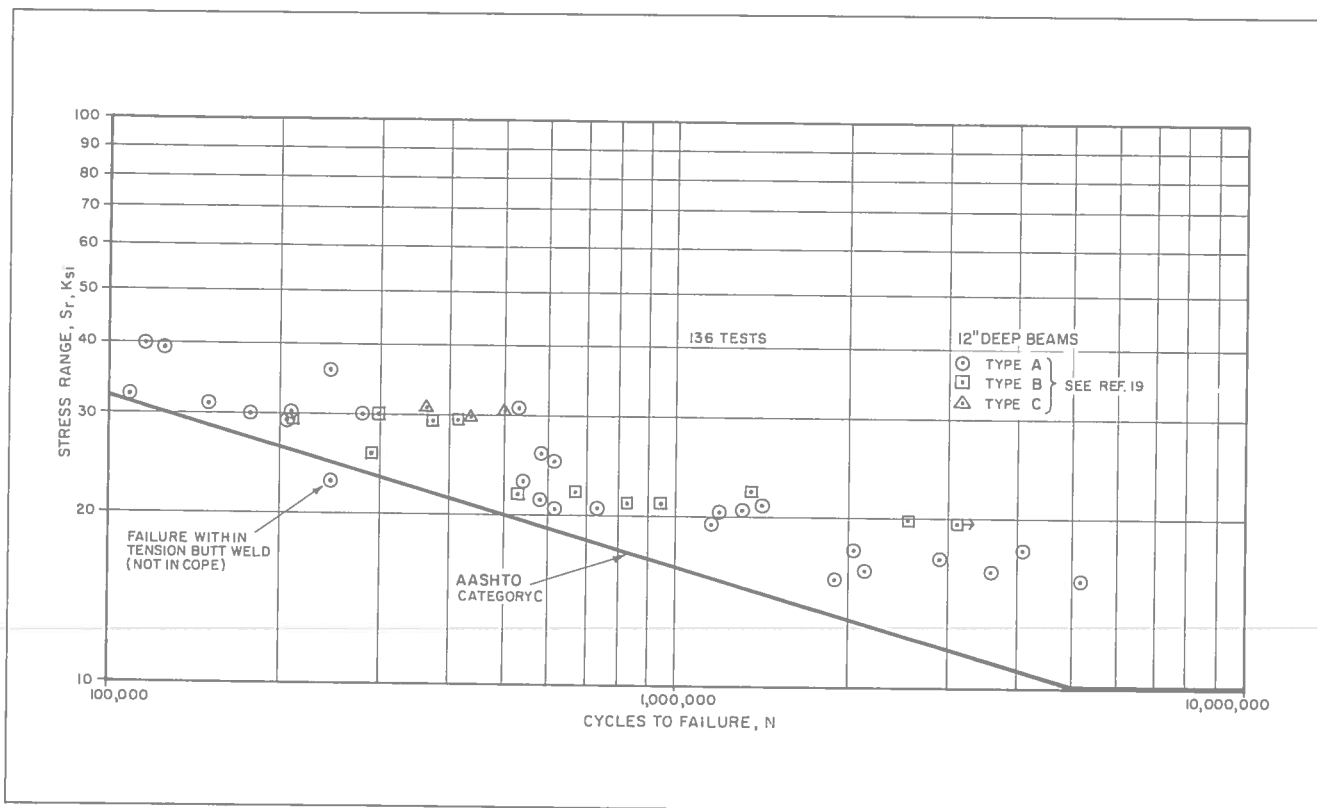


Figure 4. SN curve for cope holes.

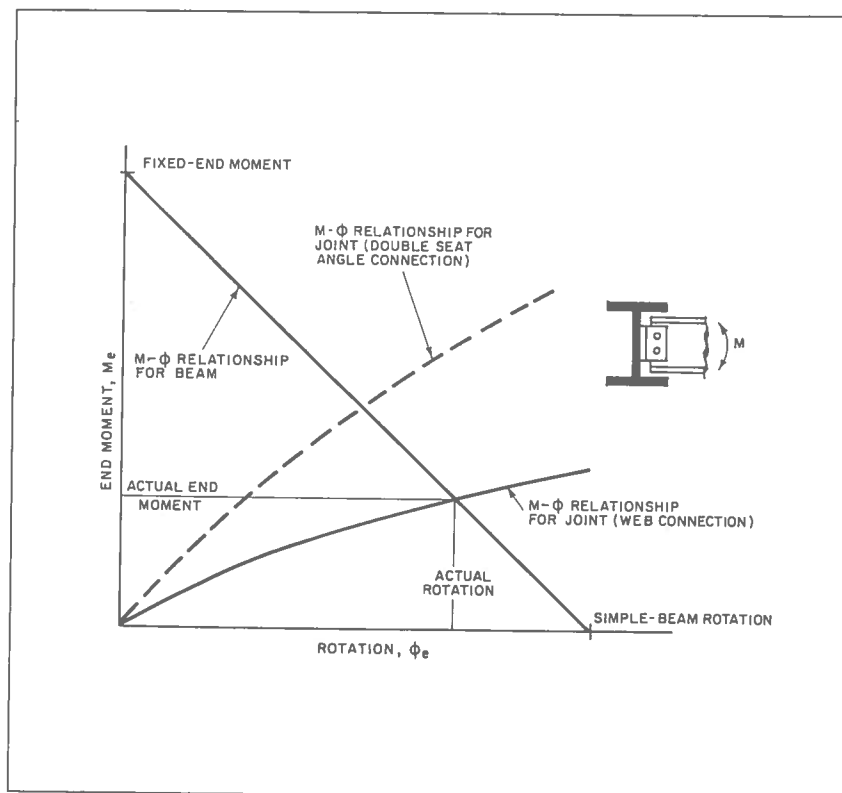
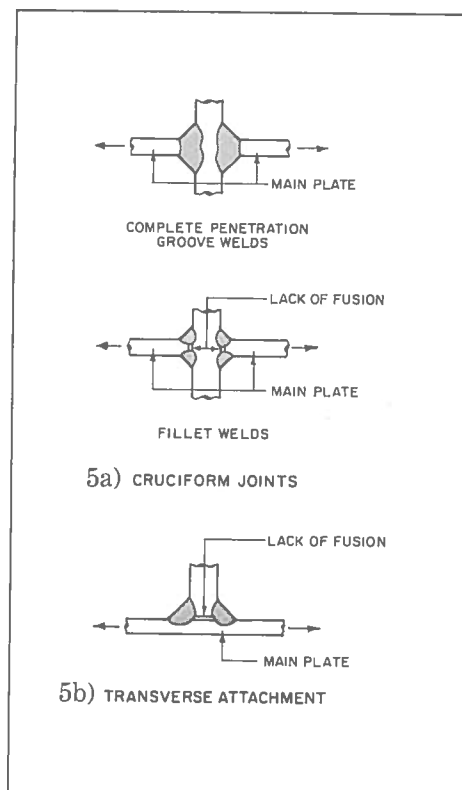
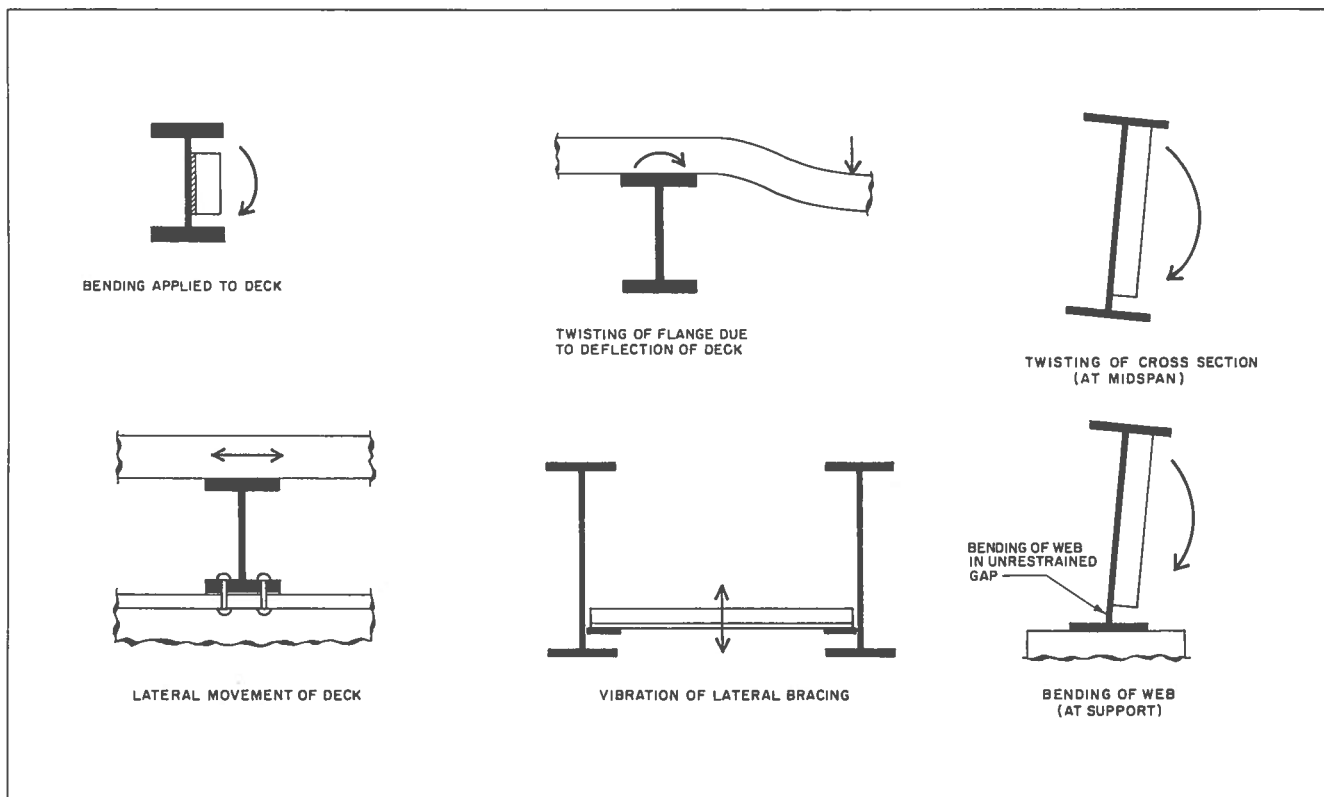


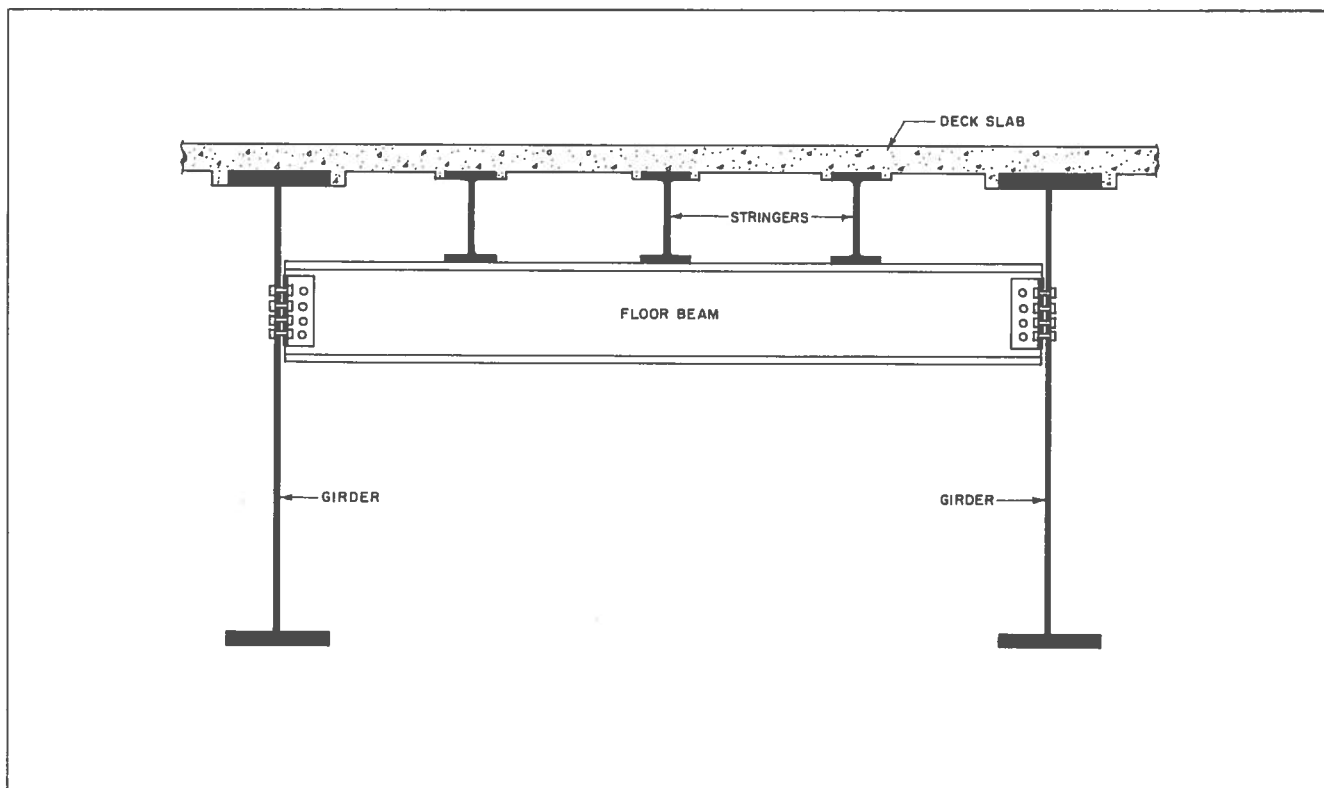
Figure 5. Comparison of cruciform joints with transverse attachments.

Figure 6. Behavior of simple-beam end connections.





**Figure 7.** Lateral bending of web.



**Figure 8.** Cross-section for plate girder bridge with floor beams and stringers.

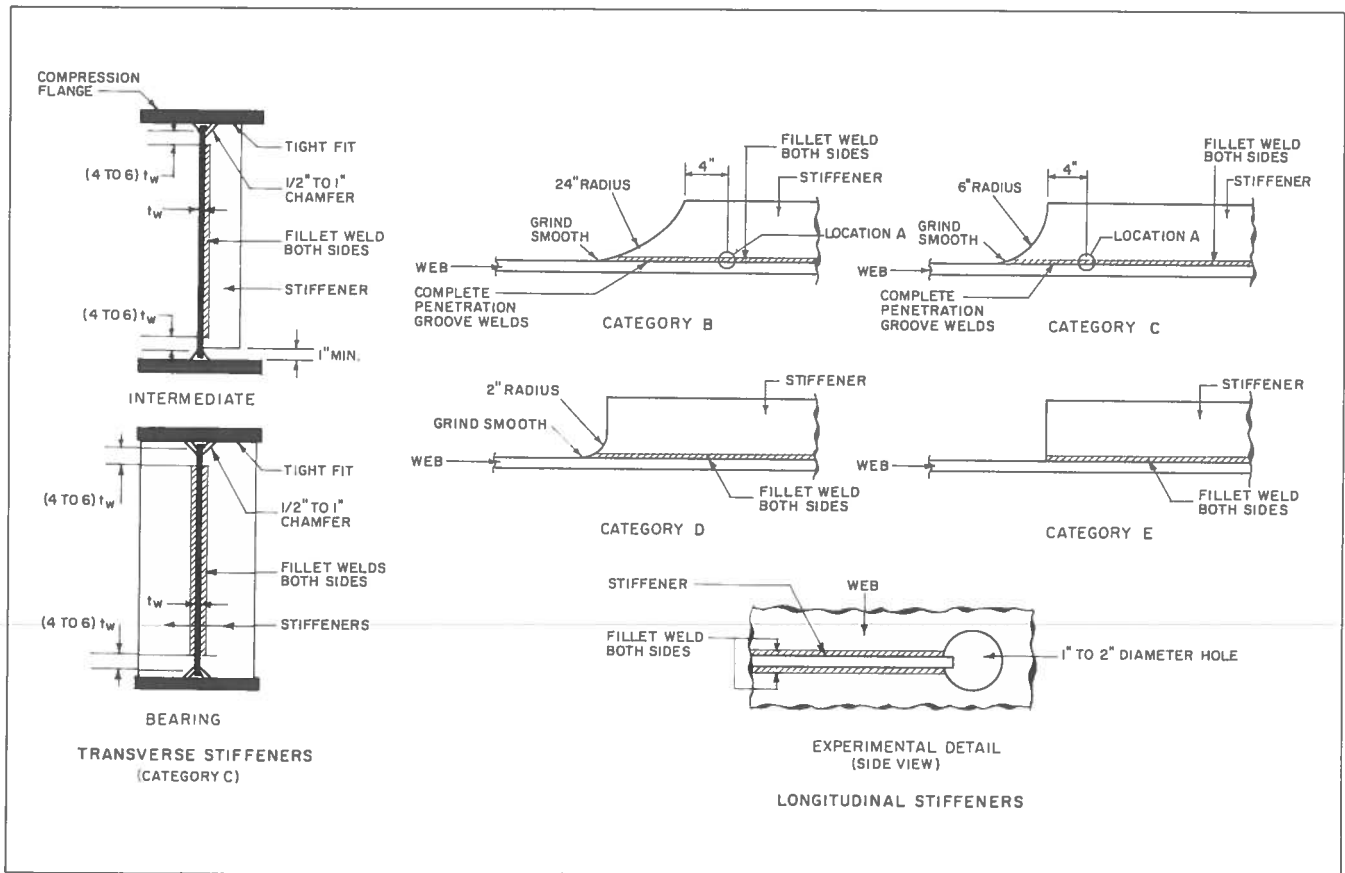


Figure 9. Plate girder stiffeners.

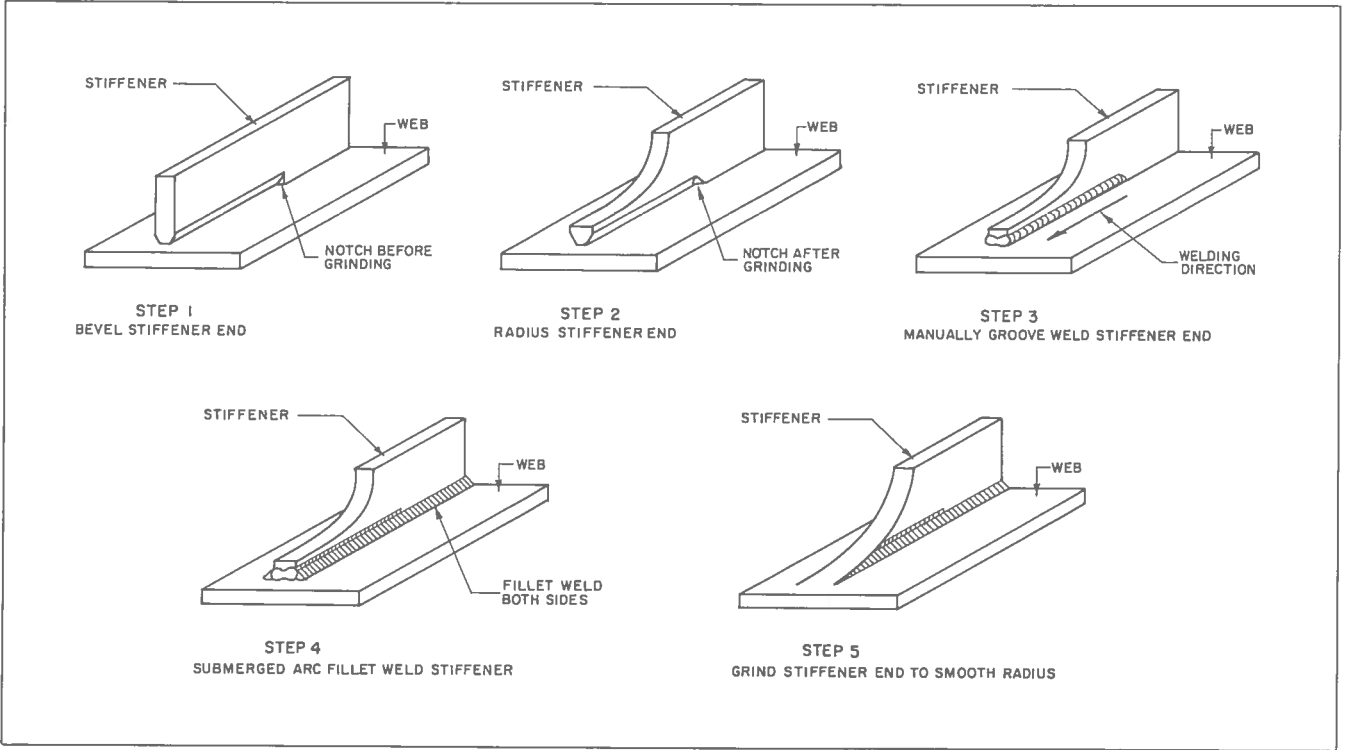


Figure 10. Fabrication procedure of Category C stiffener end.

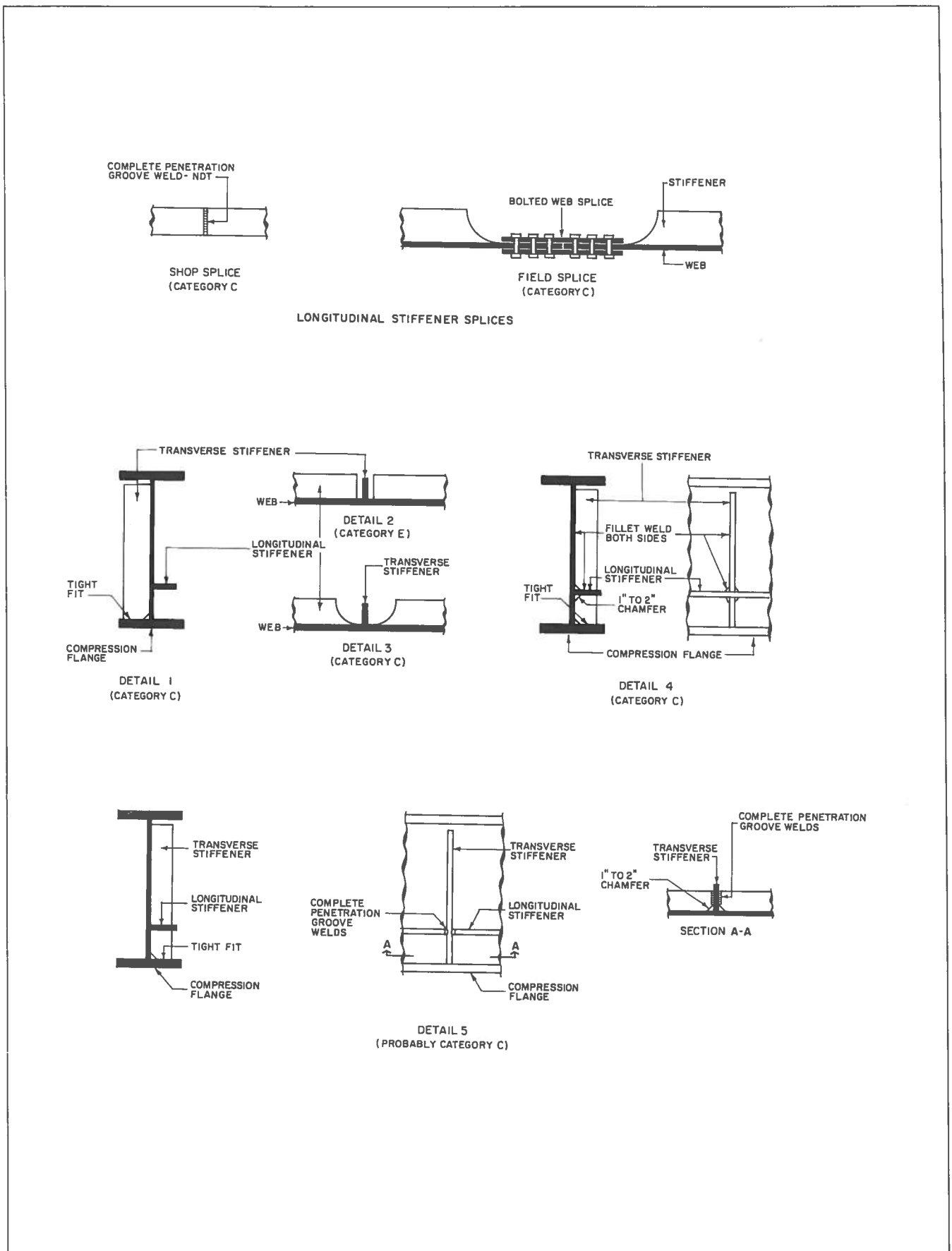


Figure 11. Stiffener splices and intersections.

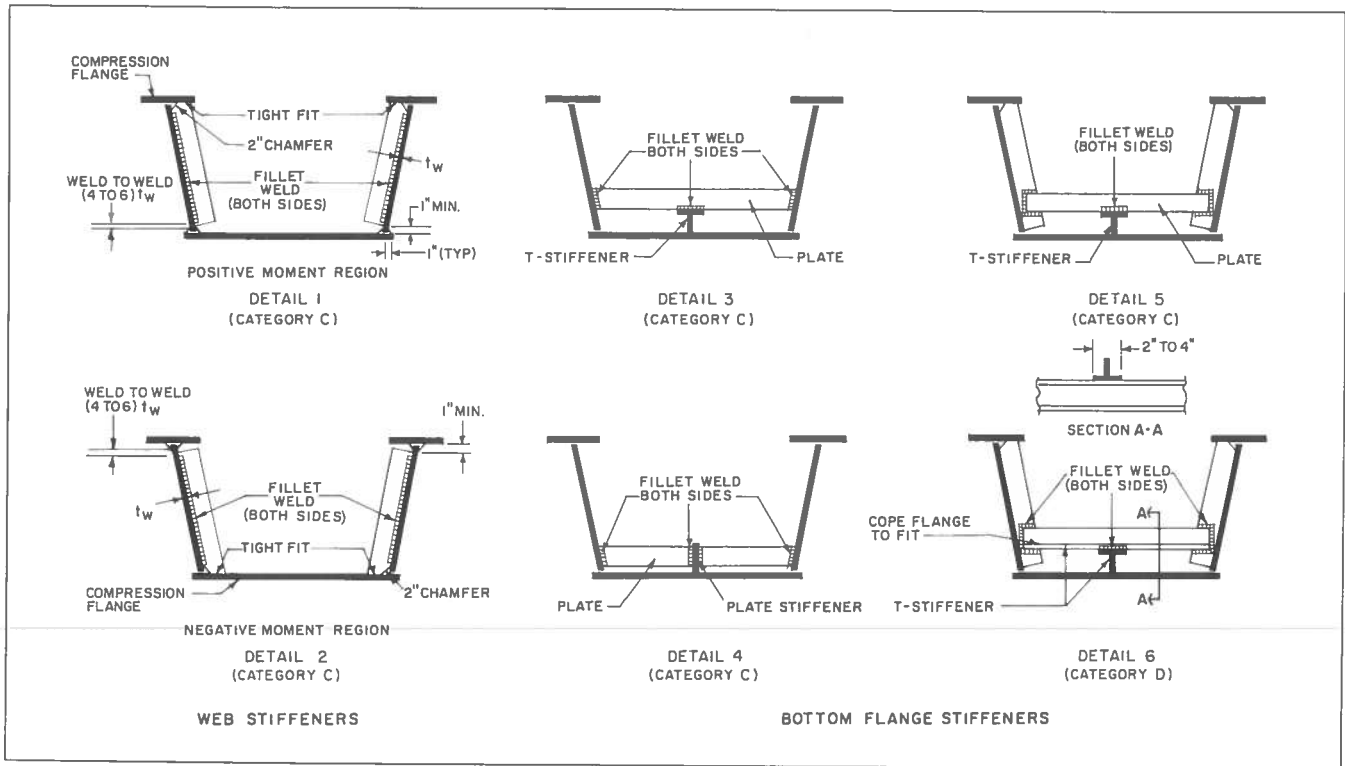


Figure 12. Composite box girder stiffeners.

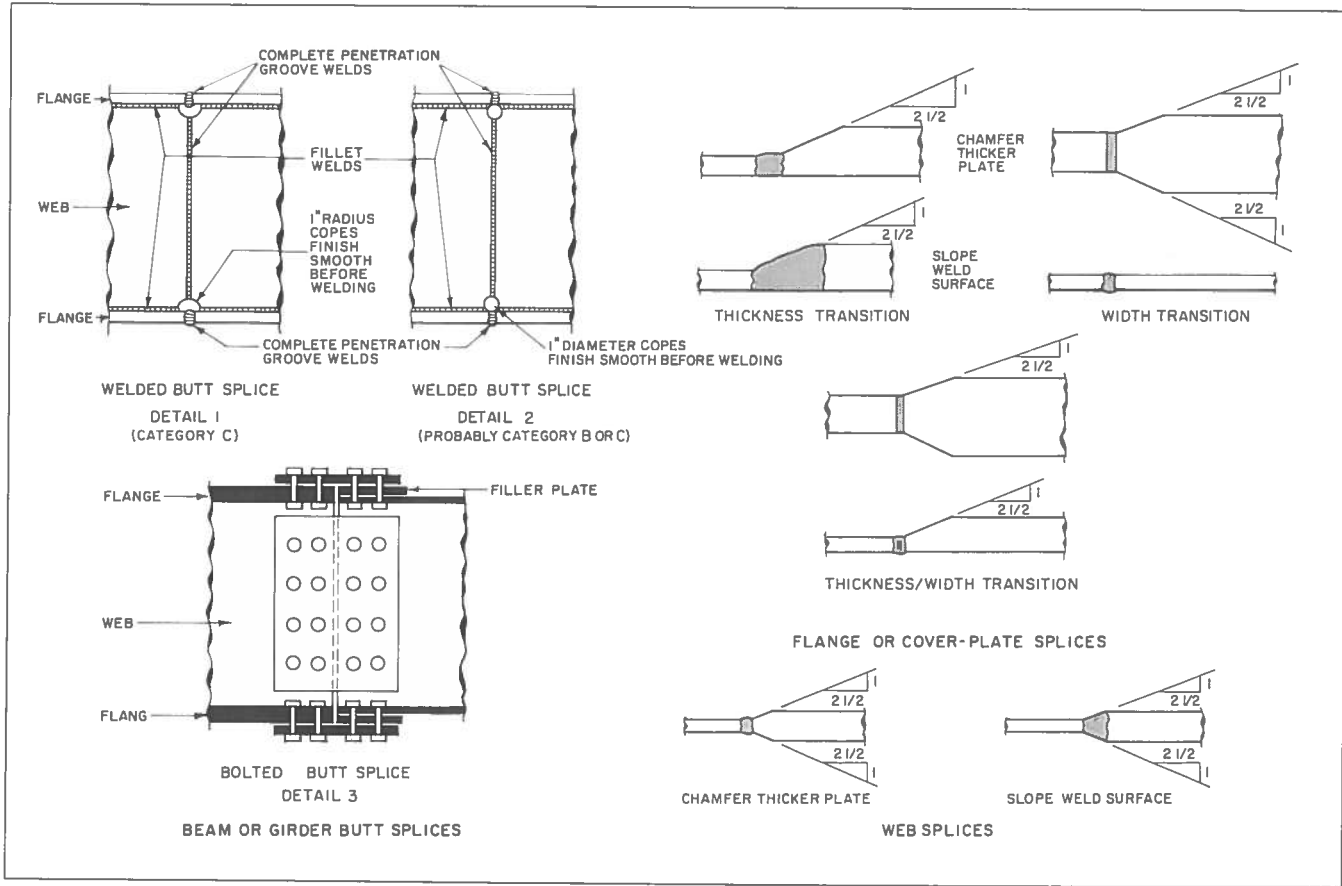


Figure 13. Splices.

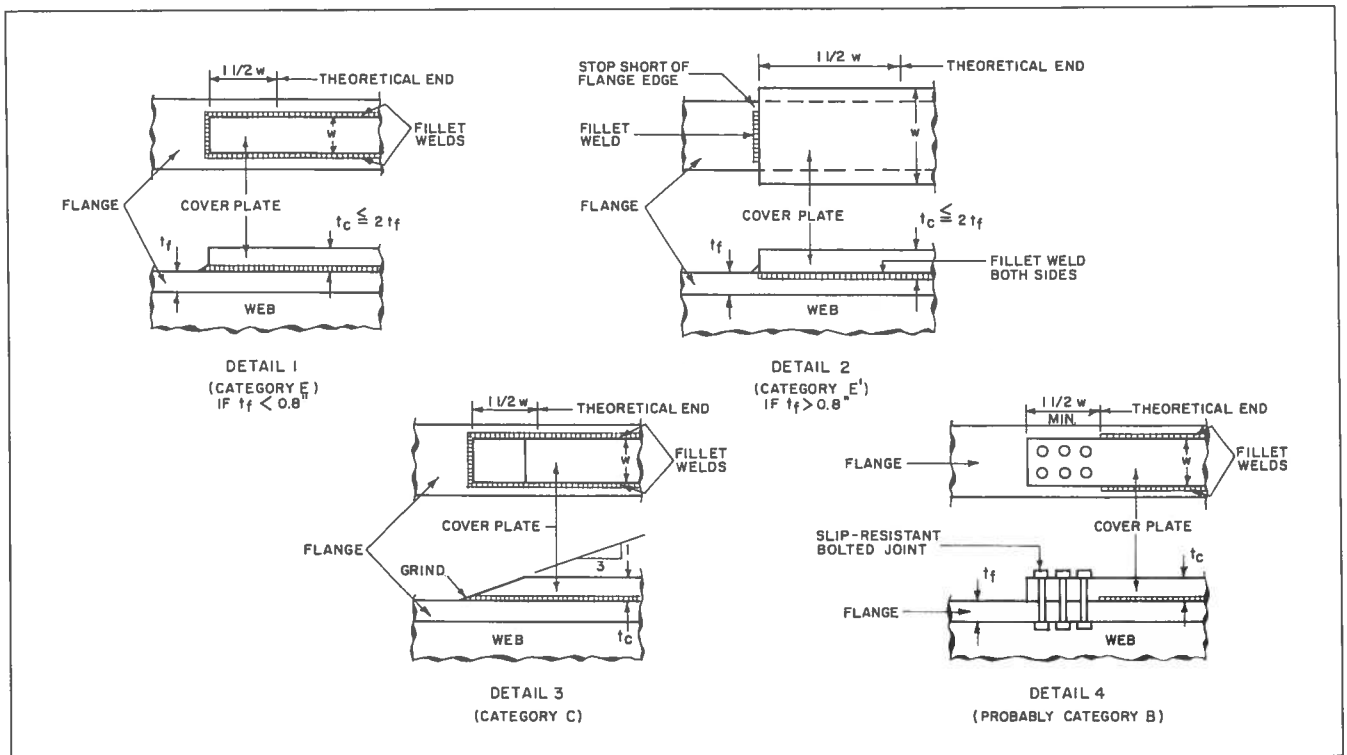


Figure 14. Cover plates.

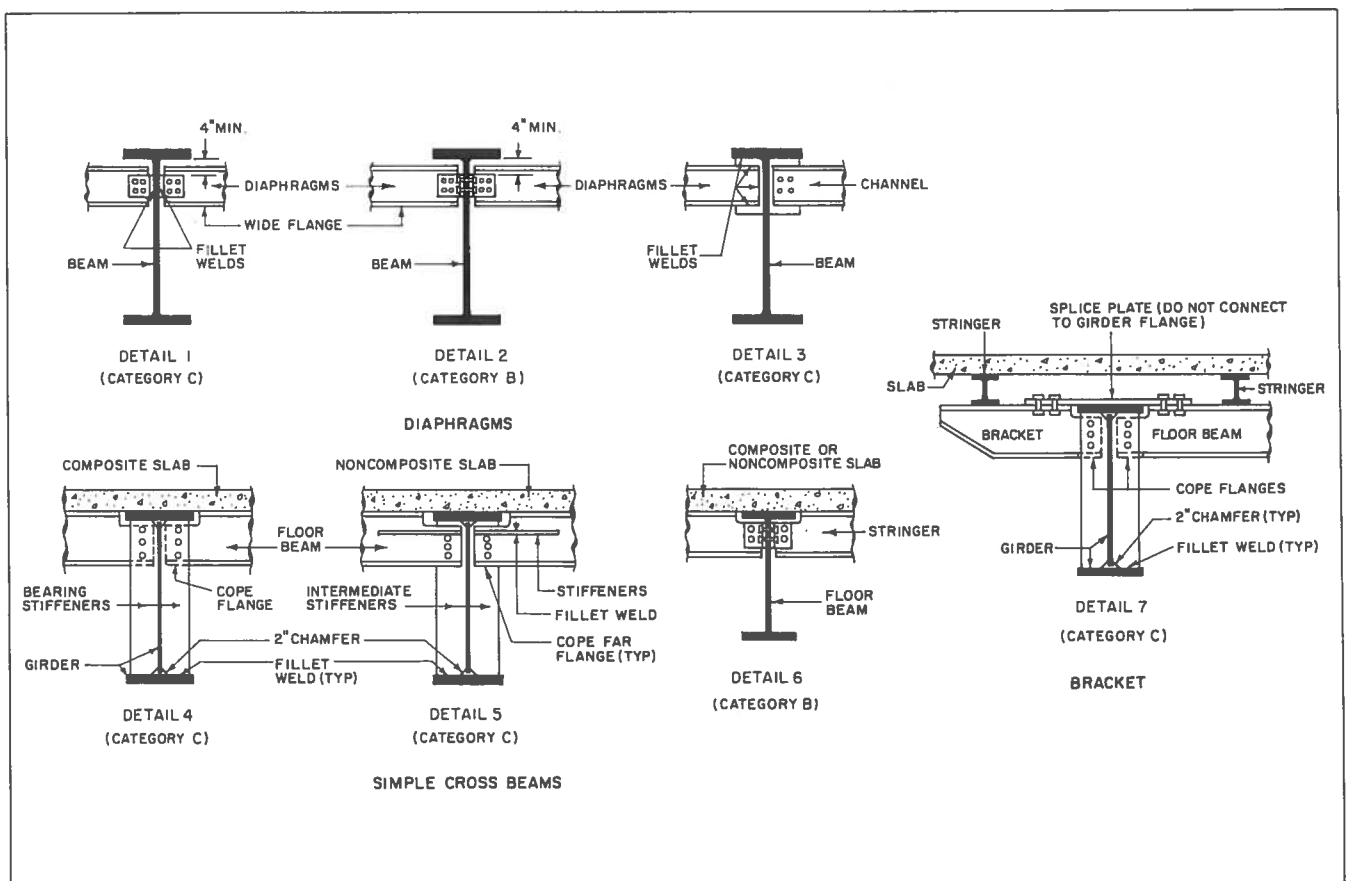


Figure 15. Diaphragms, simple cross beams, and brackets.

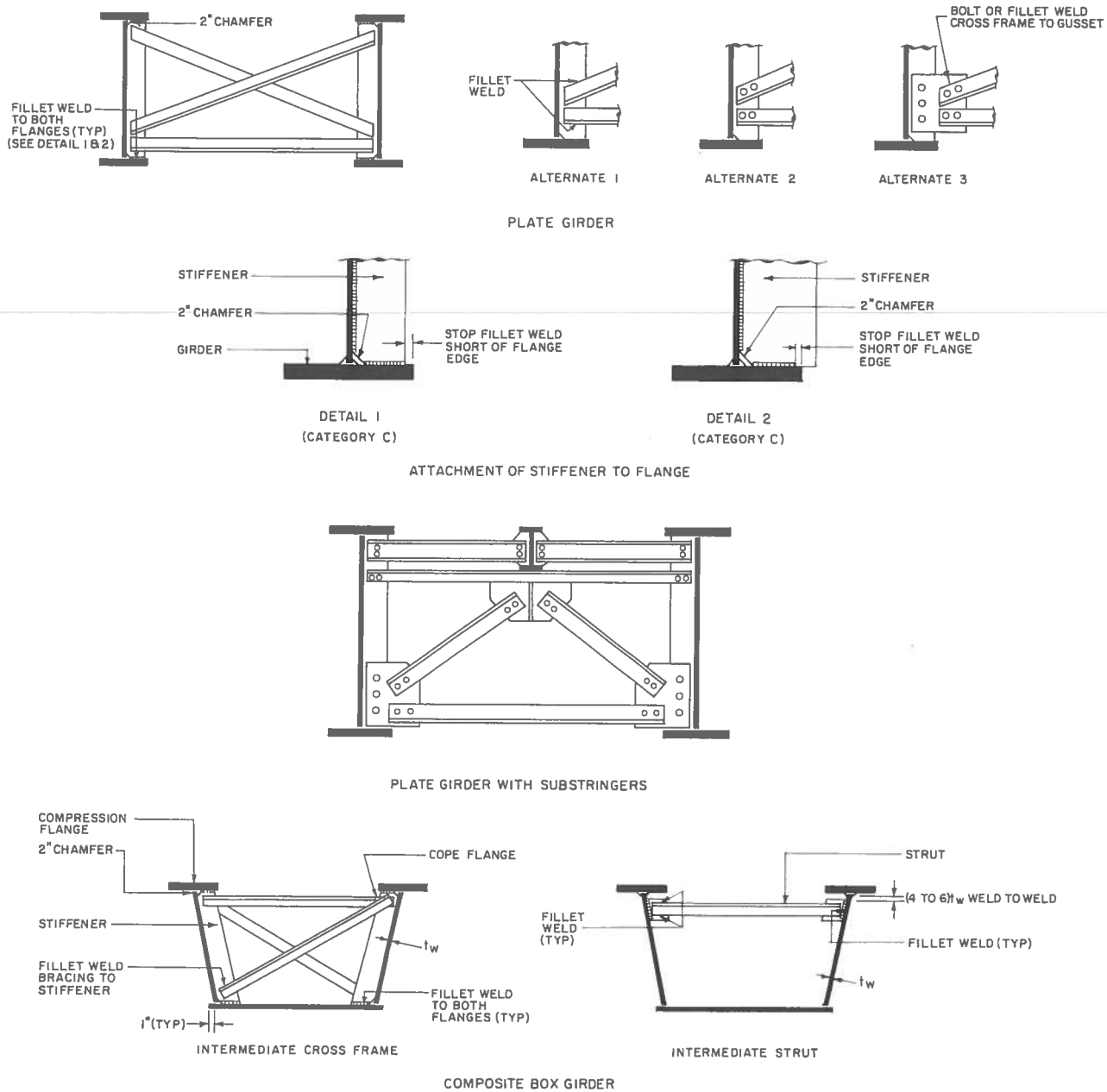


Figure 16. Cross frames.

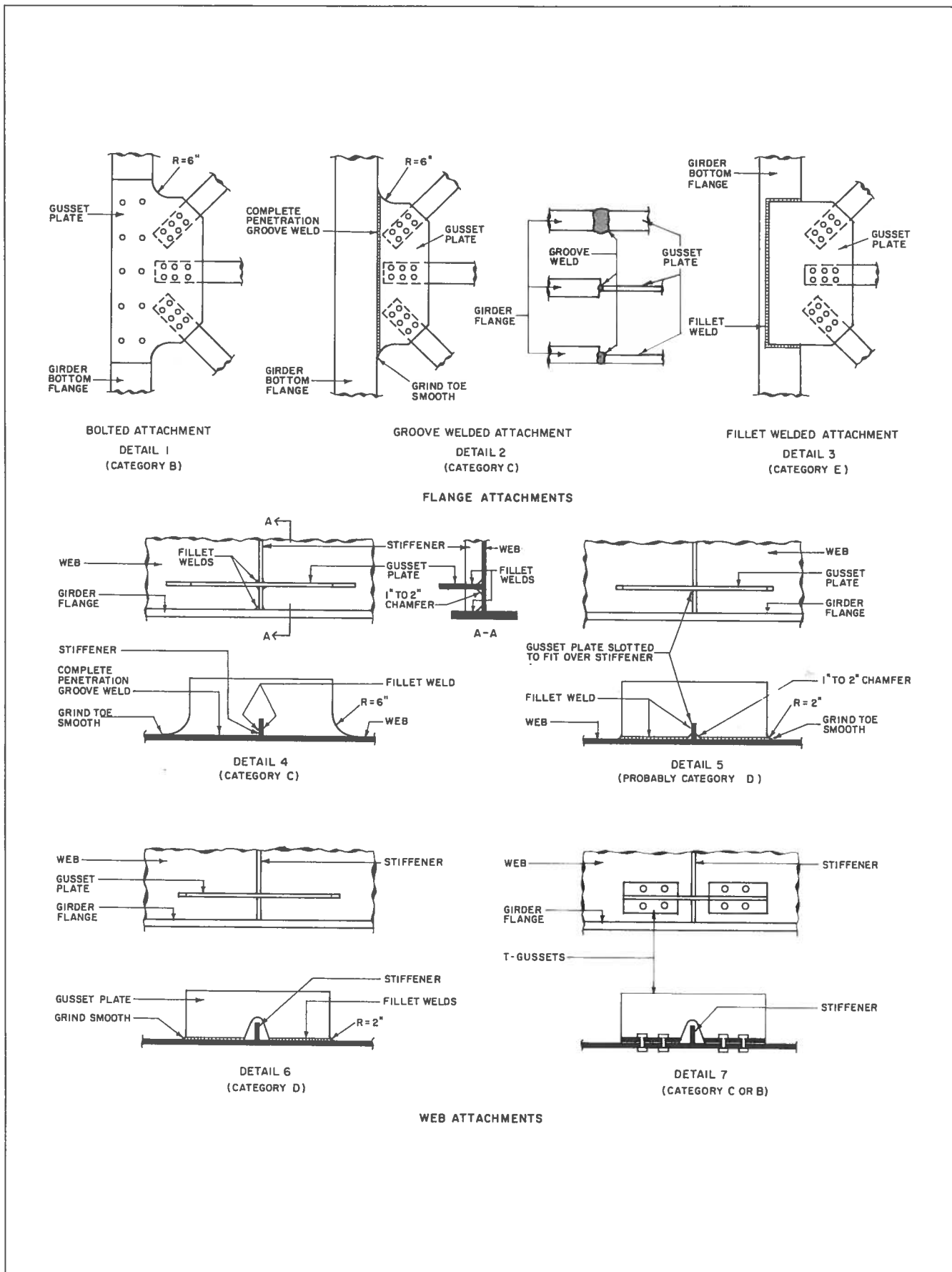


Figure 17. Lateral bracing attachments.

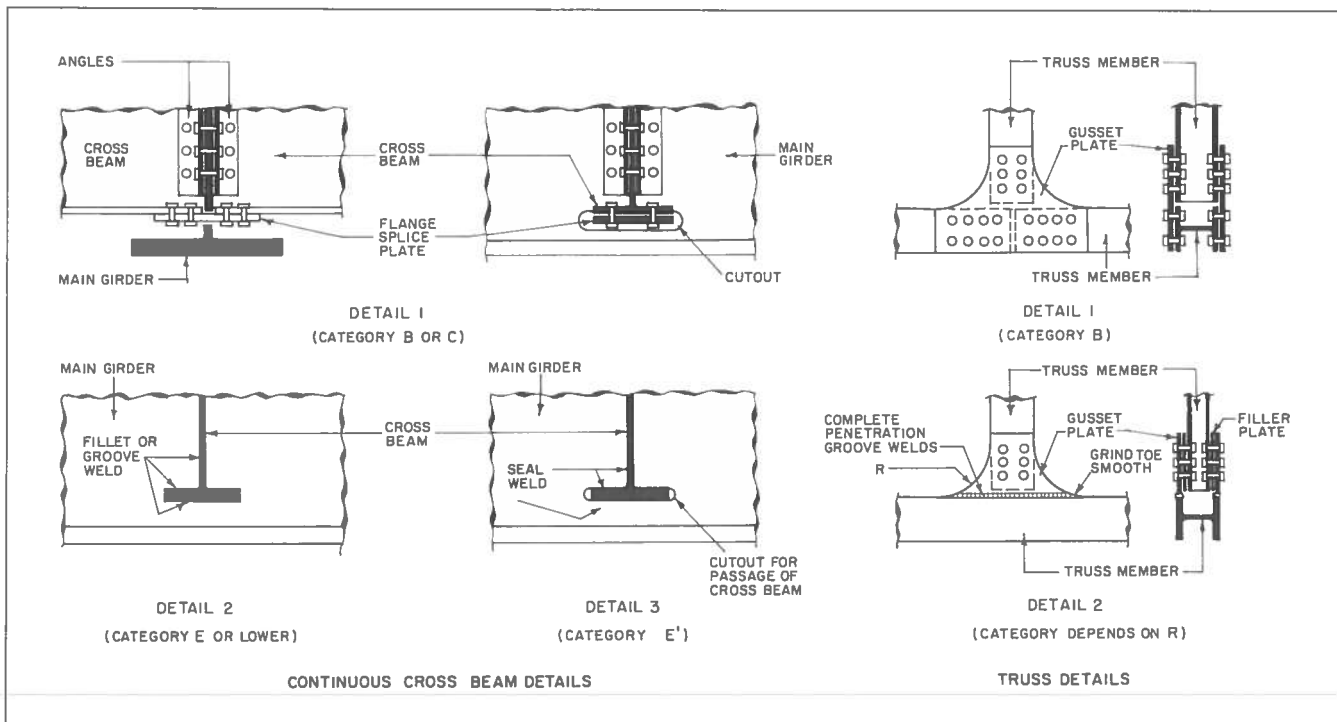


Figure 18. Continuous cross beam and truss details.

## 9. APPENDIX

### (A) CRITERION FOR CRACK ARREST HOLES

Further growth of an existing fatigue crack can be arrested by placing circular holes at both ends if the original crack length is not too large. A criterion for the limiting crack length, below which further cracking will not occur, is derived below from a stress-intensity criterion originally proposed by Barsom<sup>1,3,71</sup> and modified by Fisher<sup>14</sup> to account for roughness on the surface of a fabricated hole.

Let

- $a$  = one-half the length between outer edges of the two holes
- $L$  = length between outer edges of the two holes
- $r$  = radius of the holes
- $S_r$  = stress range
- $S_y$  = yield stress
- $\Delta K$  = stress-intensity range

$$\text{Modified criterion: } \frac{\Delta K}{\sqrt{r}} < 4 \sqrt{S_y}$$

$$\text{Definition of stress intensity: } \Delta K = S_r \sqrt{\pi a}$$

$$S_r \sqrt{\frac{\pi a}{r}} < 4 \sqrt{S_y}$$

$$S_r^2 \frac{\pi a}{r} < 16 S_y$$

$$a_{\text{LIMIT}} = \frac{16 S_y r}{\pi (S_r)^2}$$

Let

$$a = L/2, S_y = 36 \text{ ksi}, r = 0.5 \text{ inch (size of tested holes)}$$

$$L_{\text{LIMIT}} = \frac{16(36)(.5)}{\pi (S_r)^2}$$

$$L_{\text{LIMIT}} = \frac{183.3}{(S_r)^2}$$

$$\text{Round to } L_{\text{LIMIT}} = \frac{200}{(S_r)^2}$$

If this criterion is applied to a single 1-inch-diameter hole, the limiting stress range below which cracking would not occur (fatigue limit) is obtained by setting

$$1 = \frac{200}{(S_r)^2}$$

and solving for

$$S_r = \sqrt{200} = 14.1 \text{ ksi.}$$

This value is slightly below the fatigue limit of 16 ksi for a Category B detail and thus is reasonable for a single hole.