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BRIDGE LOADINGS

SECTION 3

3.1.0 INTRODUCTION

This section of the bridge design manual illustrates the application of loads to some of the more common types of bridge structures. This section will cover the application of the live loads specified in the 1961 edition of the AASHO Standard Specifications for Highway Bridges, subsequent revisions, and Bridge Department modifications.

A short description of the different loadings will be presented, then examples of application of loads in simple and continuous spans will be shown.

3.1.1 Types of Loads

Reference is made to Section 2, Loads, in the AASHO Specifications, page 7.

Structures shall be proportioned for the following loads and forces when they exist:

Dead load
Live load
Impact or dynamic effect of the live load
Wind loads
Other forces, when they exist, as follows:
- Longitudinal force
- Centrifugal force
- Thermal forces
- Earth pressure
- Buoyancy
- Shrinkage stresses
- Rib shortening
- Erection stresses
- Ice and current pressure
- Earthquake stresses

We shall consider all of these, except dead loads, in this Section.

3.2.0 LIVE LOADS

At the present time all permanent bridges in the State Highway System are being designed for H20-S16-44 live load.

In addition to preparing plans for bridges in the State Highway System, the Bridge Department is at times requested to prepare bridge plans for counties, and to check bridge plans prepared by others for counties and municipalities. Some of these bridges are designed for live loads which are lighter than the H20-S16-44.

The discussion which follows will be limited to H20-S16-44 loads. The lighter AASHO loads are simply scaled down proportionally, except for deletion of the BPR Modification and addition of AASHO Article 1.2.4, Overload Provision.

3.2.1 H20-S16-44 Load

The H20-S16 truck represents a live load used for bridge design purposes only, to insure a certain minimum load carrying capacity. It represents a vast number and variety of actual truck types and loadings to which the bridge might be subjected under actual traffic conditions. When applying the H20-S16 truck to determine maximum stresses in a member, only *one truck per lane* is utilized.

The lane load represents an approximation of a truck train which consists of a 20-ton truck preceded and followed by 15-ton trucks. It is a simplified loading. The truck and lane loadings are shown graphically in Appendix 3-A. The table for maximum moment and shear in the Appendix shows that for simple spans one truck governs for moment in spans up to 145′, and the lane loading governs for longer spans.

In continuous spans one truck governs for negative moment in spans as short as 42 feet, and for positive moment in spans of about 110 feet, depending on ratios of adjacent span lengths.

3.2.2 B.P.R. Modification for Interstate Highways

The Bureau of Public Roads calls for a modification of the H20-S16 loading for bridges on Interstate Highways. This loading consists of two axles spaced four feet apart with each axle weighing 75 percent of the rear axle of the H20-S16 loading. Unless otherwise specified this alternative loading is used on all bridges in California.

This modification slightly increases live load moments in spans of 35 feet or less.

3.2.3 Application of The Load

The truck loadings described above are moved across the bridge, and as they move they generate changing moments, shears, and reactions in the bridge members. It is necessary to accumulate the maximum stresses resulting from these moving loads to properly design the members. At this point bridge design departs from the other fields of structural design. The problem of the heavy transitory load is unique to bridge practice. Extensive investigation and understanding are necessary to find the maximum stresses in all but the simplest structures.

All of the loads described above are loads that occupy one traffic lane. It is necessary to portion these loads into the deck slabs, girders, truss members, etc., according to some distribution method, and this method is provided in the design specifications.

Slabs are loaded by individual wheels, and our design specifications use plate theory to find maximum design moments in slabs from wheel loads. Standard designs are available for transverse deck slabs on girders and for longitudinally reinforced slab bridges.

Bridge girders, stringers and some floor beams are loaded by lines of wheel loads that roll along the deck. A wheel line is half of a truck load or lane load. The number of wheel lines assigned to each girder depends upon the girder spacing and the type of girder.

For concrete decks, typical interior girder distributions are as follows:

On timber stringers	$\frac{S}{5.0}$
On precast I-Beams, T-Beams, box sections, inverted U sections and steel I-Beam stringers	$\frac{S}{5.5}$
On concrete T-Beams	$\frac{S}{6.0}$
On concrete box girders	$\frac{S}{7.0}$

S = average stringer spacing in feet.

Exterior girders, widely spaced girders and floor beams have similar distributions, but have additional complications that are described at length in the specifications.

Bent caps, columns, and some floor beams are loaded by wheel line reactions, and it is necessary to determine how many wheel lines the member can carry. Article 1.2.6 of the AASHO Specifications offers a method of determining the number of lanes on a bridge, and with this method the lane width varies between 10 and 15 feet. Designers faced with future widening or variable width problems find this specification difficult to apply, and often decide to use 12-foot lanes that can be shifted laterally to produce maximum loadings.

When any member is loaded with three or more lanes, the live loading is reduced by a probability factor. This factor is described in Article 1.2.9 of the AASHO Specifications as follows:

	Percent
One or two lanes	100
Three lanes	90
Four lanes or more	75

3.2.4 Impact

Impact is applied to live load stresses in accordance with the following formula:

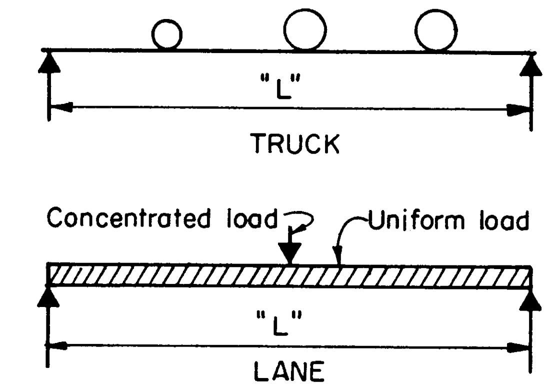
$$I = \frac{50}{L + 125} \text{ in which}$$

I = Impact Fraction (Maximum 30%)

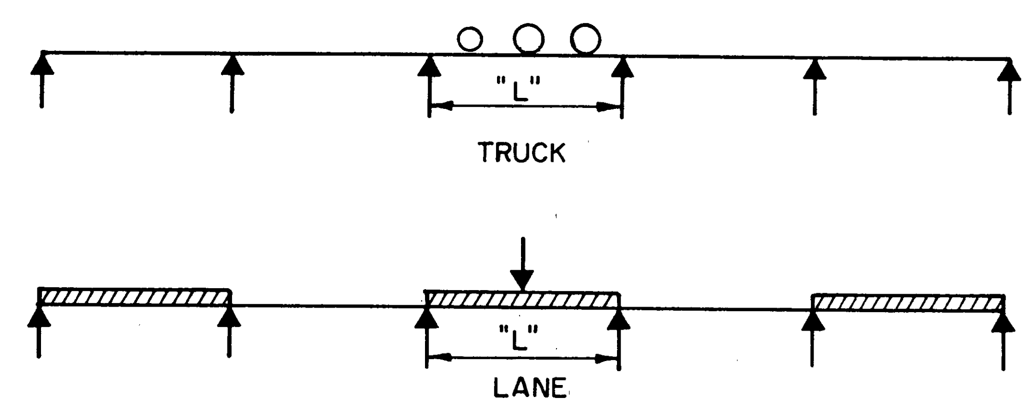
L = Length in feet of the portion of span which is loaded to produce the maximum stress in the member.

Following are some illustrations of the Loaded Length "L" in different types of structures for use in the Impact formula:

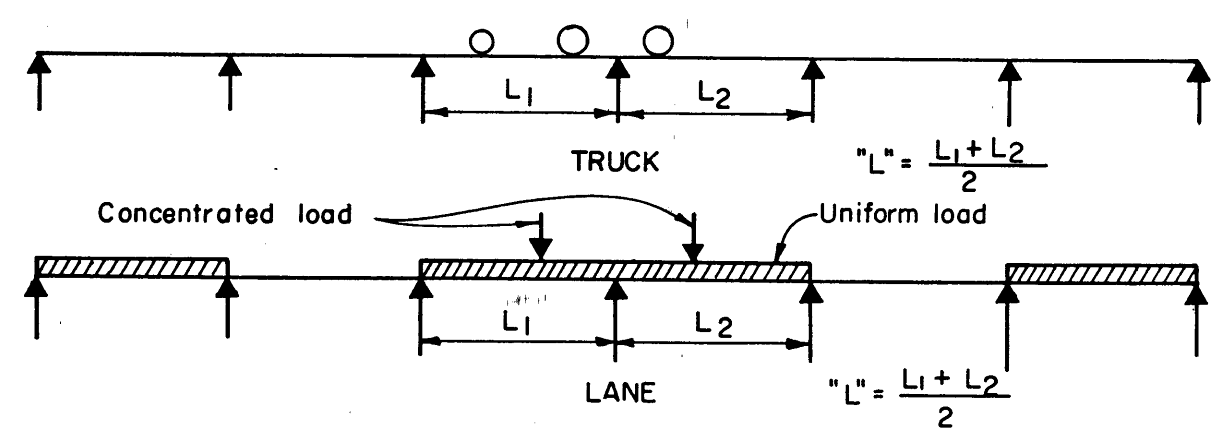
All Moments in Simple Spans



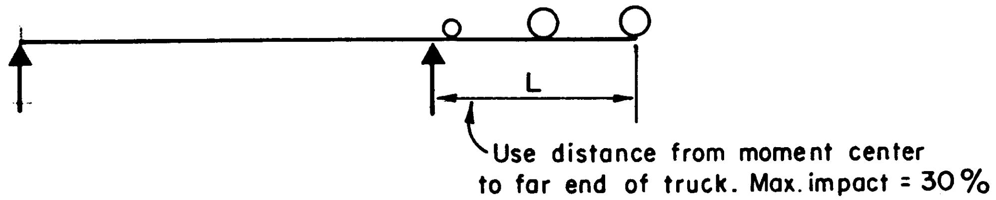
Positive Moments in Continuous Spans



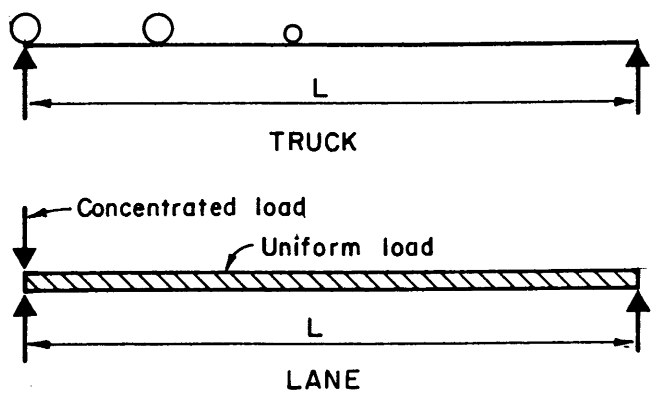
Negative Moments in Continuous Spans



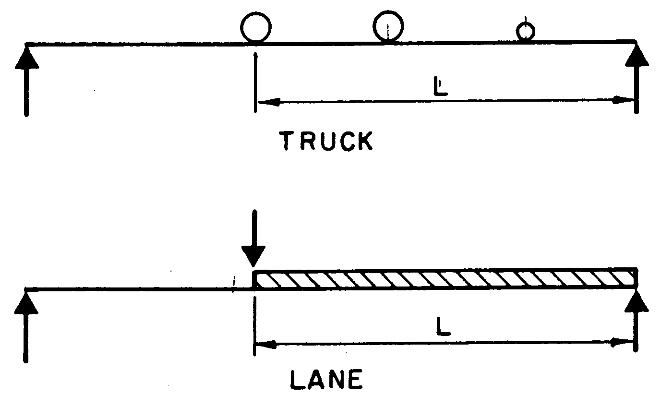
Moment in Cantilever Arms



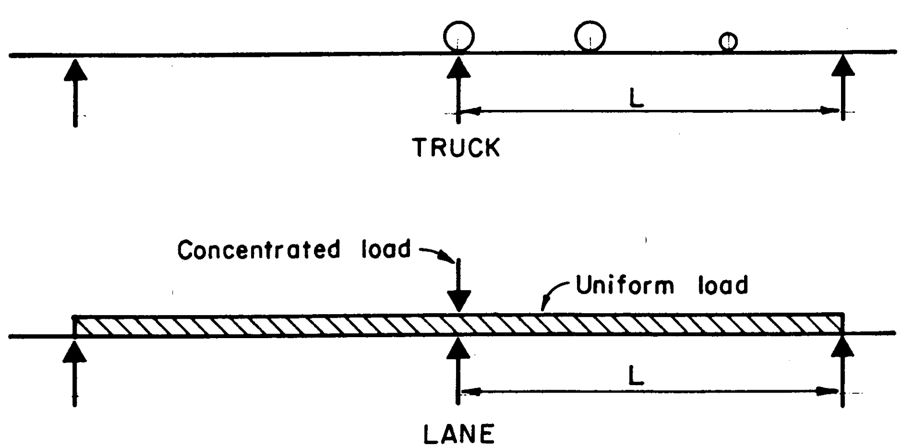
Shear at Support in Simple Spans



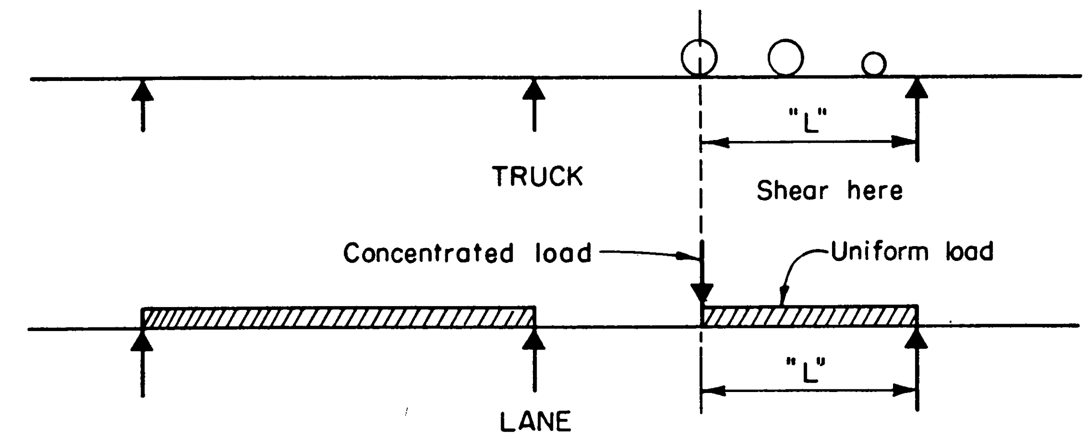
Shear within Span, Simple Spans



Shear at Support, Continuous Spans



Shear Within Span, Continuous Spans



3.3.0 OTHER LOADS

Following is a discussion of the application of loads to structures other than dead load, live load and impact.

3.3.1 Wind Loads

The basic wind load is a 100 mph wind which produces 75 psf on arches and trusses, 50 psf on girders and beams, and 40 psf on substructures.

This force is applied in a variety of ways depending on whether one has live loads present on the structure, whether one is designing superstructure or substructure, and whether the structure is ordinary or unusual.

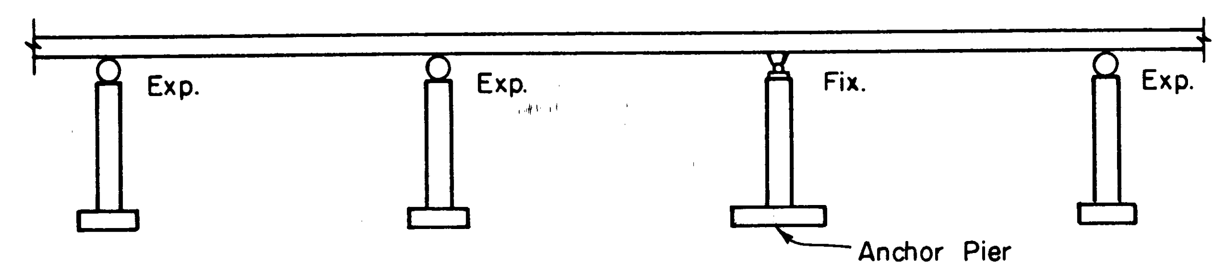
Loads on the superstructure are always applied to the area seen in elevation view. Loads on the substructure can be applied to elevation or transverse views, or skew angles in between.

Following are some examples of design in which the effect of wind loads should be investigated:

1. Structures on High Piers or Bents. Investigate its effect on the piers or bents, both laterally and longitudinally. Investigate its effect on the design of footings. The limiting height of pier where wind should be investigated varies with the length of spans and physical makeup of the structure.

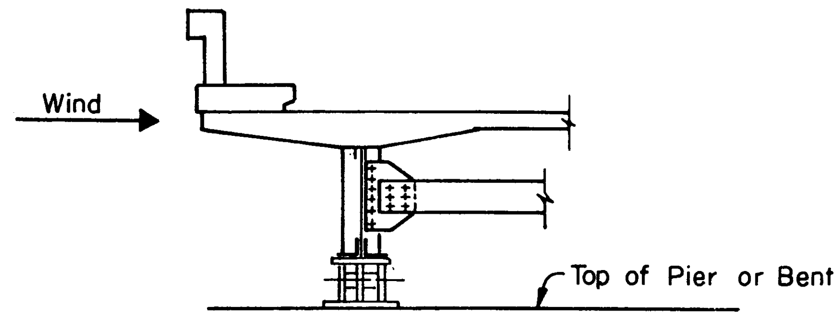
2. Anchor Piers or Bents. These would be affected by longitudinal wind as well as lateral.

3. Exterior legs of rigid frame bents where lateral wind in combination with the other forces might affect the design. For example, on the leeward leg of a rigid frame bent, the forces of wind, and temperature-increase, combined with the dead, live and impact loads could govern.



When utilizing wind loads in continuous structures, consideration should be given to the rigidity of the deck and its ability to transfer wind loads to abutments which might be considerably stiffer than the bents themselves. In these cases the abutments must be designed to support these lateral loads.

4. End diaphragms over piers or bents in plate girder spans.



In a plate girder span, the wind is transmitted to the pier via the deck and end diaphragm. The deck acts as a very stiff distributing agent in carrying the wind loads to the piers. When designing end diaphragms, the wind should be applied at the top of the plate girder. The moment center for the design of connections can be taken about the bearing pin or the bottom of the plate girder, as the judgment of the designer dictates.

AASHO states that wind on live load shall be applied six feet (6′) above the floor. This statement is best applied to the design of high piers. When designing for wind in rigid frame bents and plate girder diaphragms, it should be applied at the deck level. The wind is transmitted through the tires to the deck, and the amount of additional vertical load on the leeward side of the superstructure due to overturning effect of wind on the live load would be negligible.

3.3.2 Thermal Forces

We have modified the AASHO specification to better fit local climates as follows:

"Provision shall be made for stresses and movements resulting from variations in temperature. The temperature rise and fall depend upon location and will usually be in one of the ranges shown below."

Air Temperature Range	STEEL	CONCRETE
Extreme: 120° F Certain mountain and desert locations	Rise & Fall 60° F Movement/Unit Length .00039	Rise & Fall 40° F Movement/Unit Length .00024
Moderate: 100° F Interior Valleys and most mountain locations	Rise & Fall 50° F Movement/Unit Length .00033	Rise & Fall 35° F Movement/Unit Length .00021
Mild: 80° F Coastal Areas, Los Angeles, and San Francisco Bay Area	Rise & Fall 40° F Movement/Unit Length .00026	Rise & Fall 30° F Movement/Unit Length .00018

3.3.3 Longitudinal Force

AASHO states that provision shall be made for the effect of a longitudinal force of 5 percent of the live load in all lanes, using lane loads, with concentrated load for moment, and no impact. The reductions in load intensity of Article 1.2.9 apply. This force acts 6 feet above the floor. The force assumed is limited to traffic that is headed in the same direction.

Tractive force is combined with the other forces or loads of Groups III and VI. The piers or bents are the structure elements most affected. Occasionally, in rigid frame structures where the bents are very stiff, the moments produced by tractive force added to other forces might affect the design of the superstructure.

The application of the force 6 feet above the roadway results in one truck axle being loaded heavier than the other, and this small load does not change the girder moments very much. We are more concerned with the tractive force as a shear on the column tops.

3.3.4 Shrinkage

Shrinkage is important in arches, where rib shrinkage produces rib and column moments, and prestressed girders, where shrinkage produces loss of stressing force.

In other structures the temperature fall from the Thermal Forces specification provides adequately for shrinkage stresses.

3.3.5 Buoyancy

Whenever a portion of a structure will be submerged, the effects of buoyancy shall be considered in the design. In most of our small structures, its effects are unimportant and no economical advantage can be realized in the footing design. In large structures, however, its effects should be taken into account in the design of footings and piers. Uplift in piles is limited to an intermittent value of 40 percent of the allowable design load.

3.3.6 Earth Pressure

Some modifications of the AASHO Specifications regarding earth pressure have been adopted by the Bridge Department.

An equivalent fluid pressure of 36 lbs./sq.ft. per foot of height is used rather than the 30 lb. minimum as given in AASHO.

The 36 lb. equivalent fluid pressure is used in the design of the following structural elements:

(1) Toe pressure or toe piles in retaining walls and abutments.

(2) Bending or shear in retaining walls and abutments.

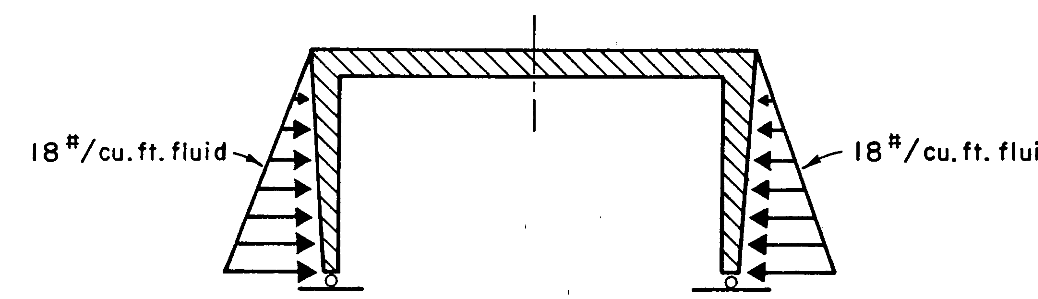
(3) Sliding of spread footings or the determination of lateral loads in piles.

(4) Bending or shear in walls of rigid frame structures.

For the design of rear piles in retaining walls or abutments, an equivalent fluid pressure of 27 lbs./sq.ft. per foot of height is used.

In no instance is zero earth pressure to be used in the design of any element of a retaining wall or abutment.

For rigid frames a maximum of one-half of the moment caused by lateral earth pressure may be used to reduce the positive moment in the beams or slabs. For this allowable reduction in moment the 18 lbs. equivalent fluid pressure is used without surcharge. This loading is shown below.



LONGITUDINAL SECTION — RIGID FRAME

3.3.7 Earthquake

Seismic force should be applied horizontally along the axes of the substructure. It is applied to all elements of the structure in which it would have an effect. For example, in the design of a footing, the coefficient for earthquake would be applied to all elements of the structure above it. The coefficient varies with different types of structures and periods of vibrations.

Application of earthquake force to design falls under Group VII, in which earthquake force is combined with dead load only. For the combination of dead load and earthquake, the allowable basic stresses are increased 33⅓ percent.

Earthquake forces should be investigated under the following conditions. These are typical examples only, and are not a complete list of cases.

1. Design of footings and tall piers.
2. Design of columns and footings in multiple column bents.
3. Slab or girder spans on pile bents. The pile section should be checked for direct load and bending at a point below ground line and at top of the pile.
4. Anchor piers on continuous structures.

3.3.8 Uplift

In continuous structures, a double live load in the first interior span can sometimes produce uplift at the abutment, and this should be investigated.

3.3.9 Centrifugal Forces

Centrifugal forces are significant in the design of bridges having small curve radii. They are also significant in the design of tall bridges having relatively large radii.

These forces act as shears in girder end frames and as loads at tops of columns. Again, the 6 foot vertical dimension between the point of load application and the floor is unimportant.

3.4.0 Combining Loads

The loads described in Article 3.2.1 through Article 3.3.9 are combined in nine different groups as shown below:

		Percentage of unit stress
Group	I = D + L + I + E + B + SF + CF	100%
Group	II = D + E + B + SF + W	125%
Group	III = Group I + LF + F + 30% W + WL	125%
Group	IV = Group I + R + S + T	125%
Group	V = Group II + R + S + T	140%
Group	VI = Group III + R + S + T	140%
Group	VII = D + E + B + SF + EQ	133⅓%
Group	VIII = Group I + ICE	140%
Group	XI = Group II + ICE	150%

D = Dead Load
L = Live Load
I = Live Load Impact
E = Earth Pressure
B = Buoyancy
W = Wind Load on Structure
WL = Wind Load on Live Load
F = Longitudinal Force due to Friction
CF = Centrifugal Force
LF = Longitudinal Force from Live Load
R = Rib Shortening
S = Shrinkage
T = Temperature
EQ = Earthquake
SF = Stream Flow Pressure
ICE = Ice Pressure

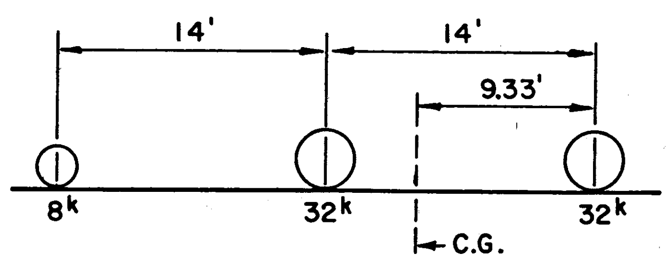
Stress increases are not allowed for members that carry wind stresses only.

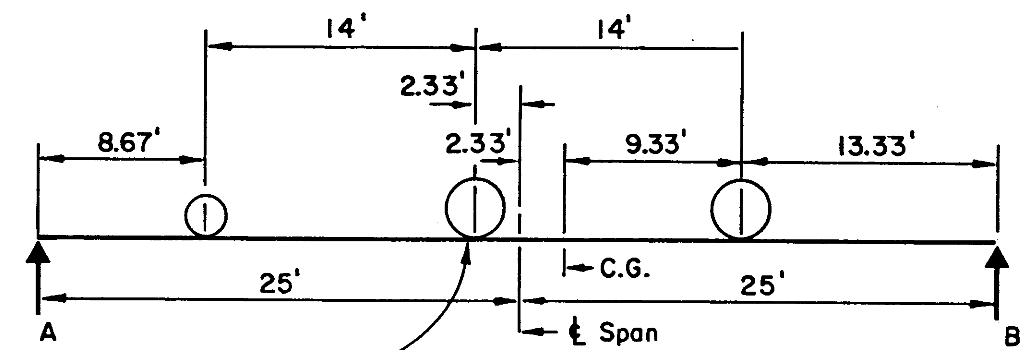
Different groups control the design of different parts of the structure, and it is often necessary to tabulate loads and allowable stress increases to determine the controlling loads on members such as truss diagonals or bent columns. It is, of course, not necessary to investigate all the loadings for each bridge. It is often evident by inspection that only a few loadings are likely to control the design of any single type of structure.

3.5.0 APPLICATION OF LIVE LOAD FOR MOMENTS IN SIMPLE SPANS

The maximum live load moment in a simple span is determined as follows: As an example take a 50′ span with H20-S16 truck loading.

The maximum moment is produced when the center of gravity of the loads and the nearest load are bisected by the center of the span.





The maximum moment is under the rear axle of the truck

(Slide rule computations)

$$\text{Reaction at A} = \frac{(13.33' + 27.33')}{50'} \times 32^k + \frac{41.33'}{50'} \times 8'' = 32.7^k$$

$$\text{Moment} = 32.7^k \times 22.67' - 8^k \times 14' = 628'^k$$

Referring to the table of simple span moments in the Appendix, the moment is given as $627.9'^k$

3.5.1 Envelope Curve of Maximum Moment

This is a curve which is drawn by combining the moments for dead load and live load with moments produced by any other loads or forces. It is a curve which indicates the maximum moment produced at any point in a member. It is used for determining the cut-off points for reinforcing bars in concrete members, or points of flange size change on plate girders.

3.5.2 Envelope Curve of Maximum Moments in a Simple Span

Construction of the envelope curve for simple spans can be accomplished in two ways. The first involves the placing of the truck in several positions on the span and computing the live load moment at several points. These values are then combined with the dead load moments to get the envelope curve. This is a laborious method, and is used by digital computers.

The second method involves an approximation, and is widely used in hand computations. If a single concentrated load were placed at successive points on the span and the moment computed at each point, the curve passed through the points would be a second degree parabola.

Passing a truck across the span produces a curve which closely follows a parabola. A deviation from the parabola occurs in some spans due to the necessity for changing the direction of the truck. In others, all of the wheels cannot be placed on the span to produce maximum moment. However, for practical purposes the combined dead load, live load and impact curve can be plotted as a second degree parabola.

As an example, we will return to the 50 foot simple span and compute dead load and live load moments for one wheel line:

1. Compute the dead load moment at the center of the span DLM = 0.125 WL^2 for uniform load.

2. From the Table of Live Load Moments in Appendix 3-B the maximum live load moment for a 50′ span is 627.8 $ft.^k$ per lane.

This maximum moment is located 2.33 feet from the centerline of span, but by assuming it to be at centerline of span no appreciable error will be introduced.

Then:

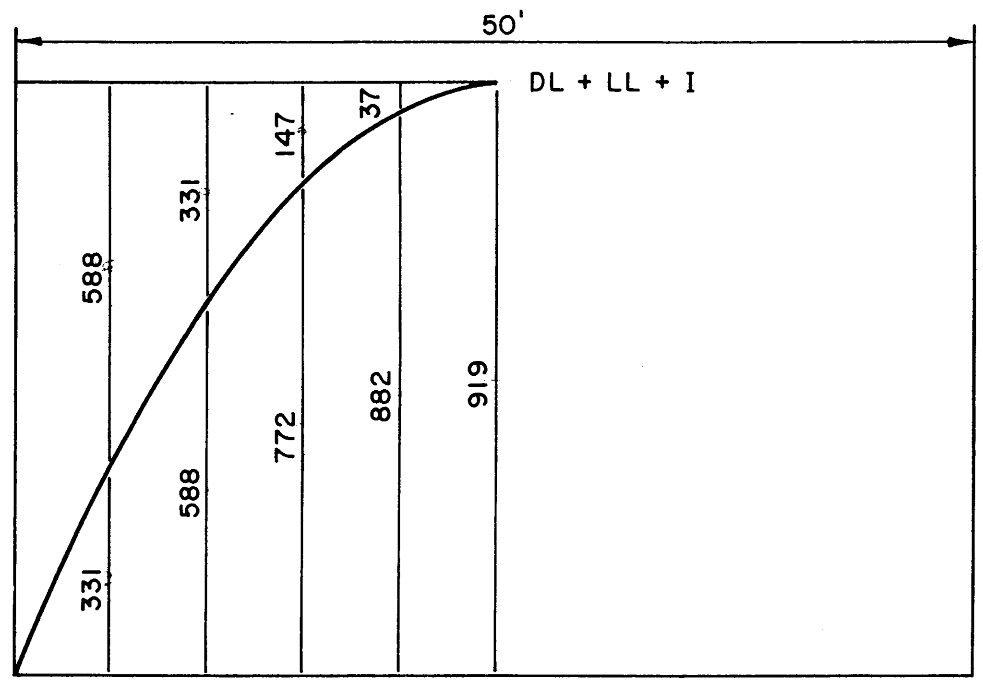
$$DLM = .125 \times 1.0 \times \overline{50}^2 = 313.\ ft.^k$$

$$LL + I\ (1.5\text{ wheel lines}) =$$

$$\frac{1.5}{2} \times 628 \times 1.286 = 606$$

$$\text{Total} \ldots\ldots 919\ ft.^k$$

The envelope curve is then plotted by passing a second degree curve through the midpoint moment of 919 ft.k Comparison of this curve with the exact one would show that they agree within practical limits.



DL + LL + I MOMENT

3.6.0 APPLICATION OF LIVE LOAD FOR MOMENTS IN CONTINUOUS SPANS

On continuous structures it is not always obvious by inspection how the loads should be placed to produce maximum conditions. A great deal of guesswork can be eliminated in the placing of live loads for maximum moment, shear, or reactions by the use of influence lines. It would not be practical to construct influence lines for all structures. However, there are several sets of them which are available to the employees of the Bridge Department, and one can quickly learn to sketch influence lines freehand with sufficient accuracy to determine loading points.

The loading tables in Appendix 3-B show maximum moments, shears, and reactions for H20-S16 loads in continuous spans with various end conditions. These tables are useful in moment distribution by hand methods, but they cannot replace a basic understanding of load application methods.

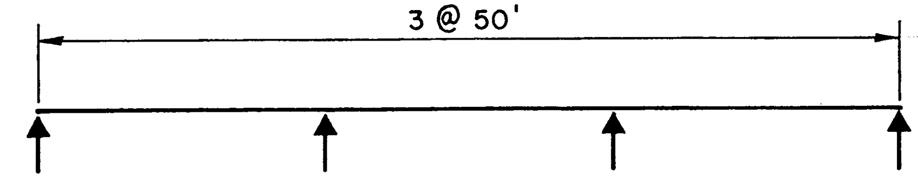
It is difficult to visualize the way truck loadings actually generate a moment envelope when using tables, so we shall go through a step by step manual development of a moment envelope in the succeeding sections.

3.6.1 Envelope Curve for Positive Moment in an End Span of Three Continuous Spans

Let us take an actual case and plot the envelope curve of positive moment for the end span of three continuous simply supported spans shown in Figure 3.1. This will be done to indicate that the maximum positive moment is near the 0.4 point of the span. The moments tabulated below for the several points on the span are based on a girder dead load of 2.0 kips per lin. ft. and 1.5 wheel lines per girder, plus 28.5 percent impact. Live load is H20-S16-44.

FIGURE 3-1

ENVELOPE CURVE OF POSITIVE MOMENT IN END SPAN



(Uniform Moment of Inertia)

TABLE 3-1

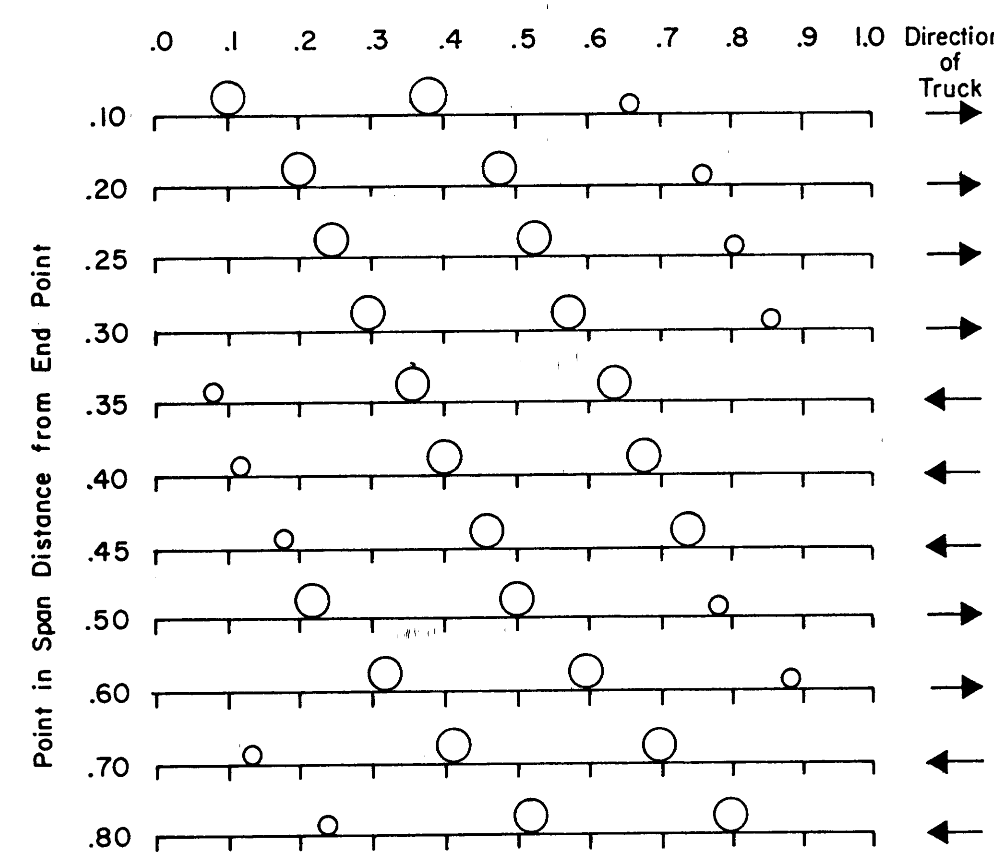
Point in span	*DLM (Ftk)*	*LLM (Ftk)*	*28.5% Impact*	*Total (Ftk)*
.1	175	177	50	402
.2	300	292	83	675
.25	344	323	92	759
.30	375	347	99	821
.35	394	365	104	863
.40	400	370	106	876
.45	394	365	104	863
.50	375	358	102	835
.60	300	326	93	719
.70	175	250	71	496
.80	0	147	42	189

From this tabulation it can be seen that the maximum moment is at the 0.4 point of the span. For practical purposes the truck could be placed anywhere from the 0.35 to the 0.45 point to obtain the maximum positive moment, without appreciable error.

FIGURE 3-2

CHART SHOWING TRUCK POSITIONS FOR DETERMINING POSITIVE MOMENT IN THE END SPAN, OF THREE CONTINUOUS SPANS

(Uniform I)



For the conditions of dead load and live load indicated, the preceding tabulation of moments in Table 3-1 gives the maximum positive moment that can be obtained at each of the points shown. A curve passed through these points constitutes an envelope curve of positive moment.

It would be impractical and time consuming to make a moment distribution for each position of live load in the span as shown in Figure 3-2. The construction of this envelope curve can be simplified by the application of several known factors.

1. The ordinate of maximum moment is located near the 0.4 point.
2. The curve of dead load moments is a second degree parabola.
3. The curve of live load moments is for all practical purposes a second degree parabola.

Thus, by computing the dead and live load moments at the 0.4 and 0.8 points in the span, the envelope curve can be constructed.

The procedure to follow is this:

1. Make a moment distribution for dead load and construct the dead load moment curve as in Figure 3-3.
2. Place the truck in position for positive moment at the 0.4 point as in Figure 3-3.1.

FIGURE 3-3

DEAD LOAD MOMENT

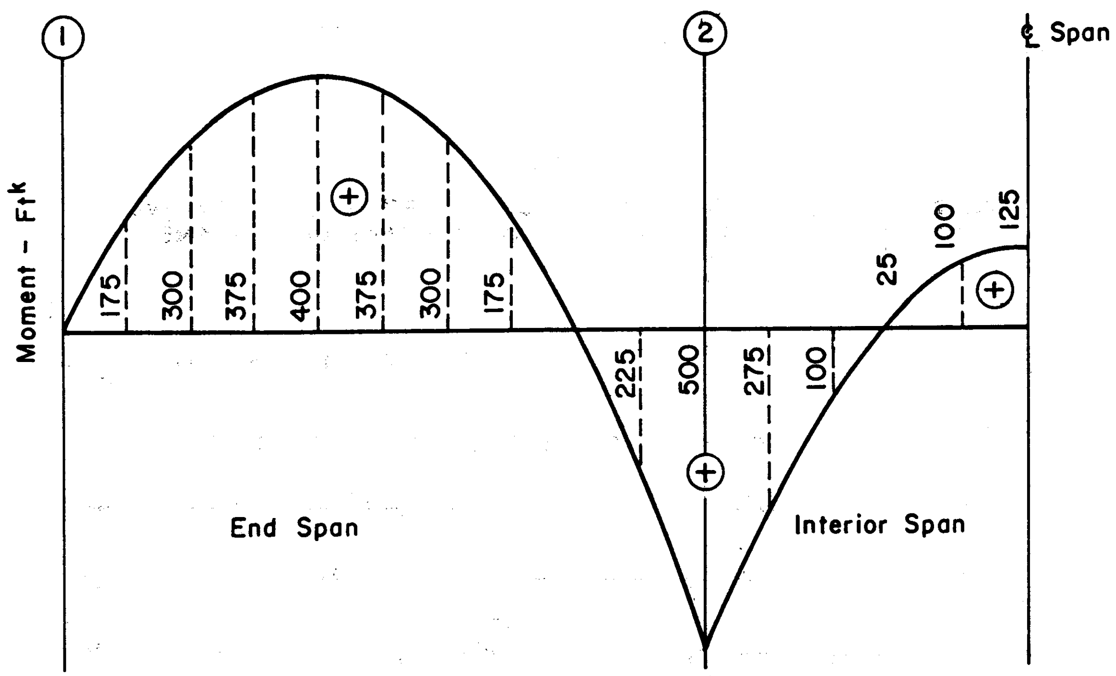
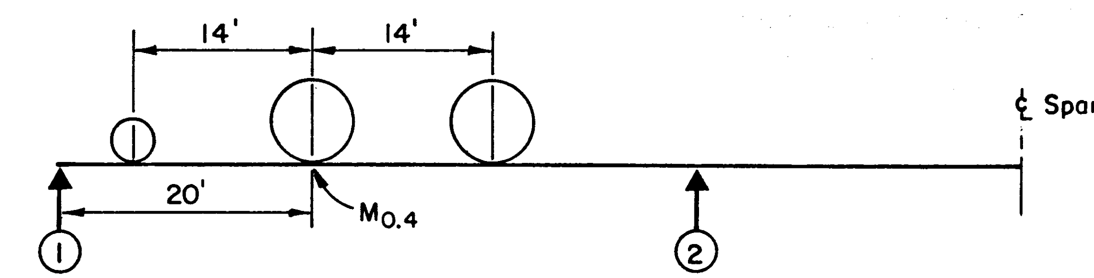


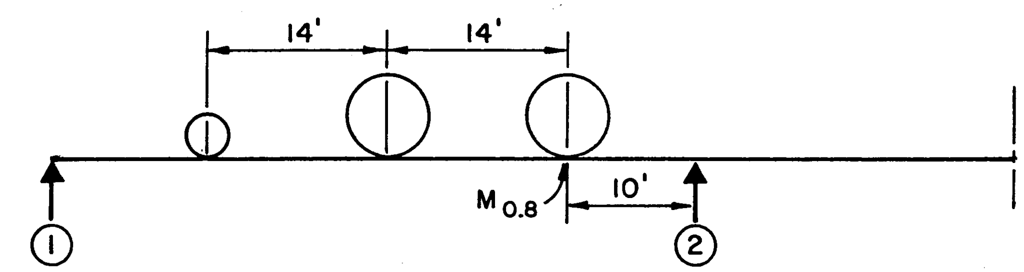
FIGURE 3-3.1



The net positive moment for 1.5 wheel lines including 28.5 percent impact is 876 ft.[k].

3. Place the truck in position for positive moment at the 0.8 point.

FIGURE 3-3.2



The net positive moment for 1.5 wheel lines including 28.5 percent impact is 189 ft.[k].

4. Plot the ordinates for dead load plus live load and impact, at the 0.4 and 0.8 points. Then, using the ordinate at the 0.4 point as the vertex, pass a parabola through zero at support 1, and through the ordinate at the 0.8 point.

A comparison of the results obtained by this method and the previous tabulated values in Table 3-1 shows, respectively, at the 0.2 point, 660 ft.[k] against 675 ft.[k]; and at the 0.6 point, 705 ft.[k] against 719 ft.[k].

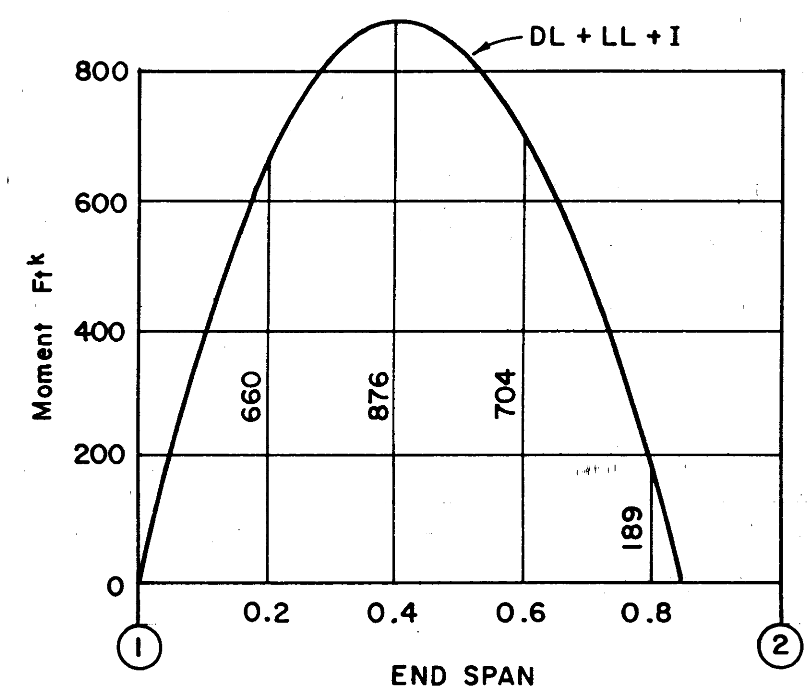


FIGURE 3-4

ENVELOPE CURVE FOR POSITIVE MOMENT

Values shown are slide rule computations

3.6.2 Envelope Curve for Negative Moment Over 2nd Support of Three Continuous Spans

Maximum Negative moment over the support is produced by the equivalent lane load. The structure is loaded as in Figure 3-5.

Moments produced by the equivalent loading are slightly greater than those produced by the H20-S16 truck. It is difficult to predict when truck or lane loadings will control for negative moment, since this depends on the structure geometry. Figure 3-6 shows this relationship for equal lengths of end and interior spans.

FIGURE 3-5

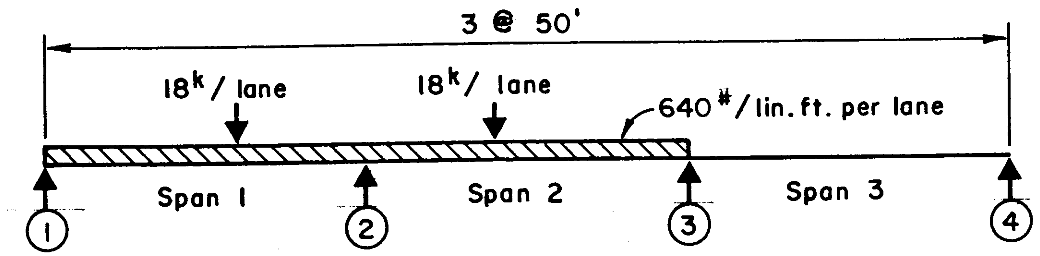
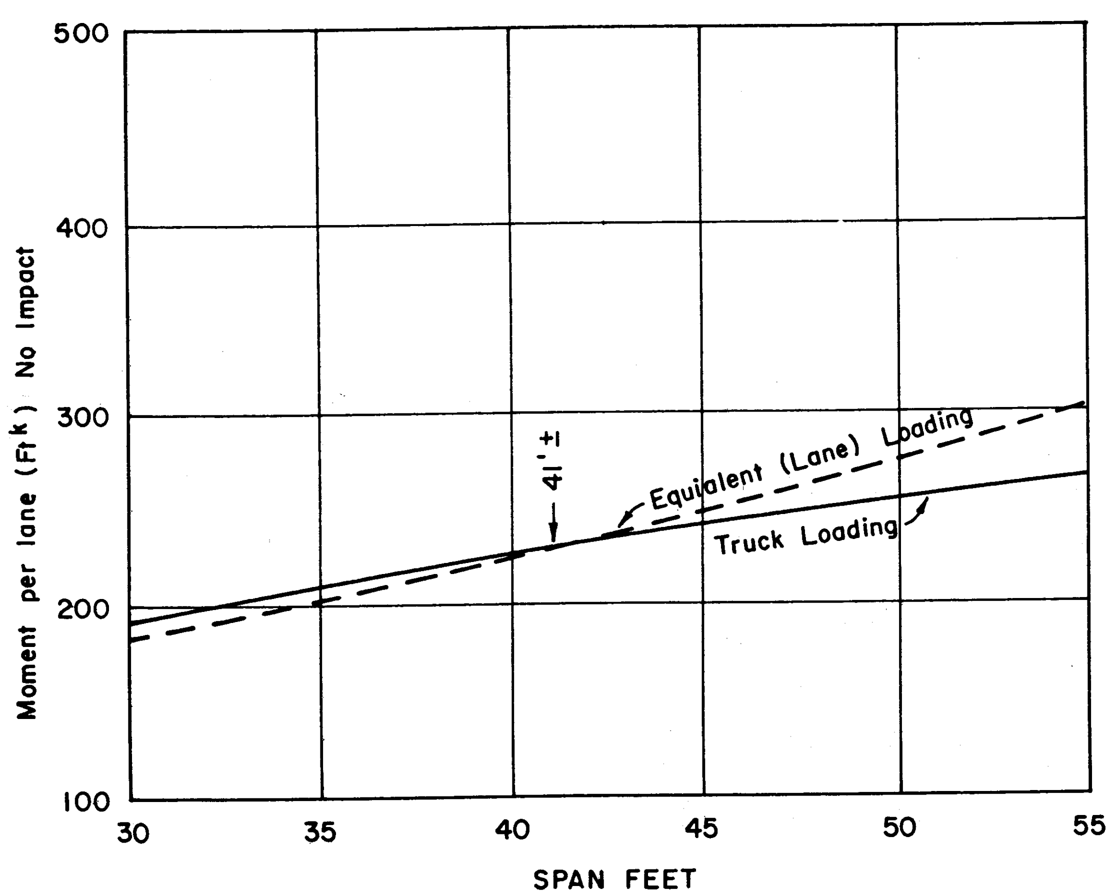


FIGURE 3-6



The value of the live load negative moment over support 2 in Figure 3-5 is 338 ft.ᵏ. This is for 1.5 wheel lines and includes 28.5 percent impact.

Maximum live load negative moment over Span 1 is produced by loading Span 2. The load position for this moment is shown in Figure 3-7. The negative moment in unloaded spans is frequently overlooked by beginners.

The value of the live load plus impact moment over support 2 for this loading is 230 ft.ᵏ. Moment is plotted in Figure 3-7.1.

FIGURE 3-7

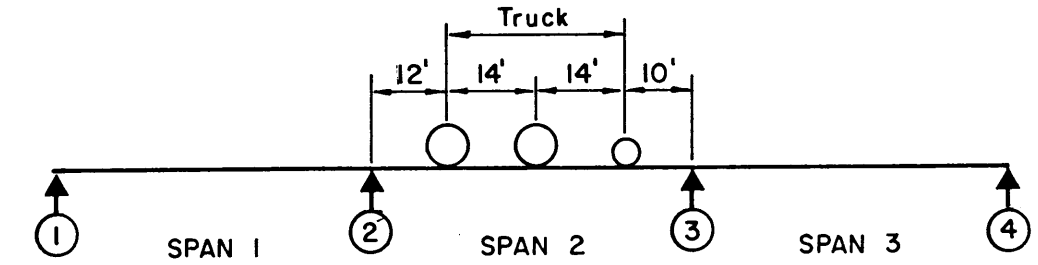
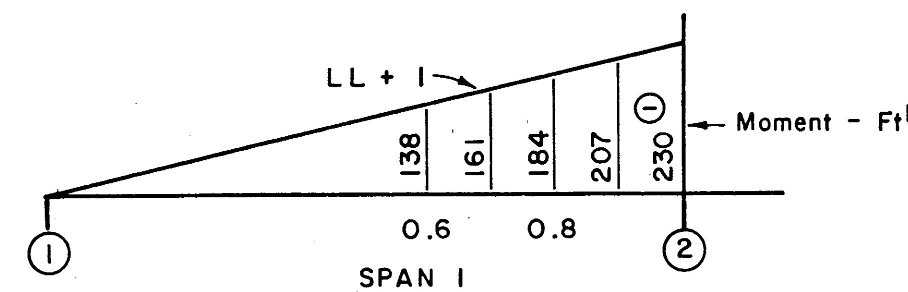
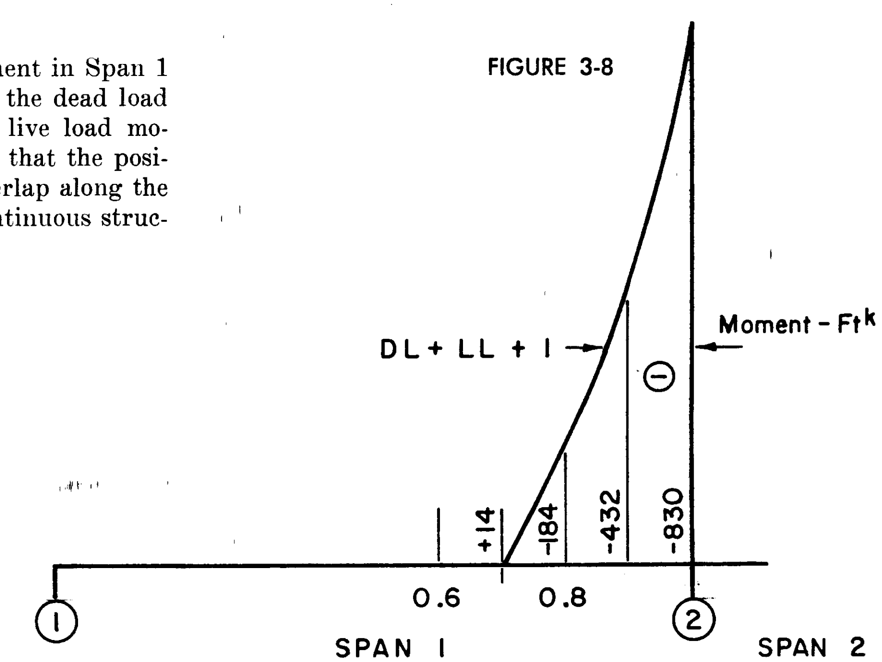


FIGURE 3-7.1



The envelope curve for negative moment in Span 1 is obtained by combining the values of the dead load moment curve in Figure 3-3 with the live load moments of Figure 3-7.1. It will be found that the positive and negative moment envelopes overlap along the base line. This is a characteristic of continuous structures.

FIGURE 3-8



The envelope curve for negative moment in Span 2 is obtained by loading Span 1 as shown in Figure 3-9.

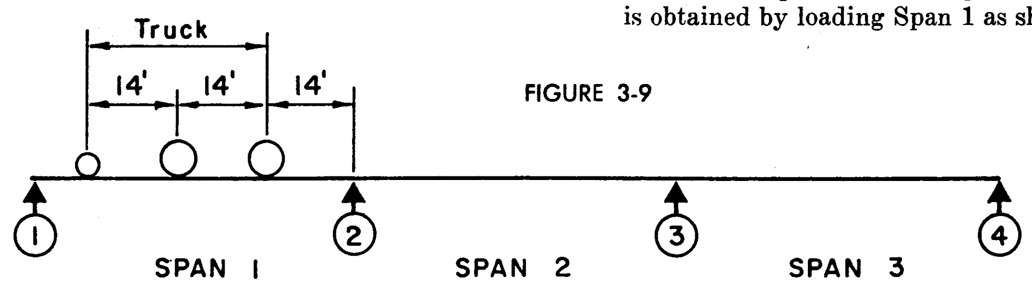


FIGURE 3-9

The moment in Span 2 for this loading is plotted in Figure 3-9.1.

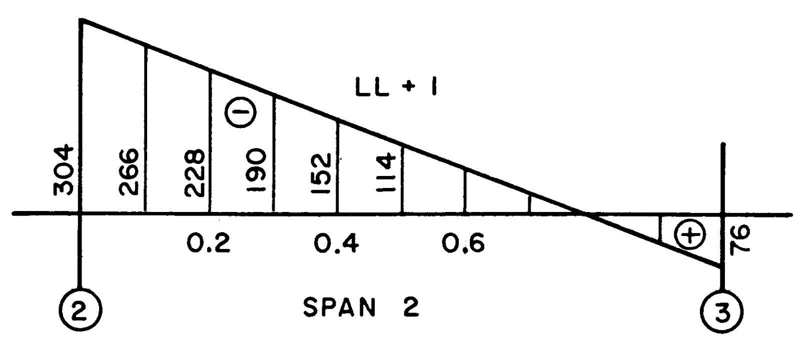


FIGURE 3-9.1

Combining the dead load moments in Figure 3-3 with the moments in Figure 3-9.1, the envelope curve of Figure 3-10 is obtained.

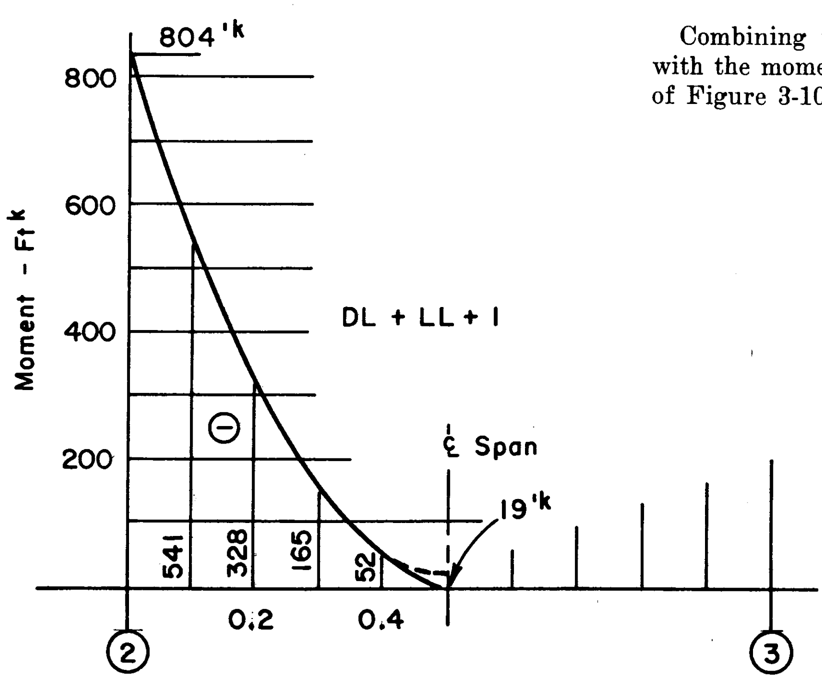


FIGURE 3-10

Actually a further refinement could be obtained by loading Spans 1 and 3 with equivalent loading as shown in Figure 3-11.

FIGURE 3-11

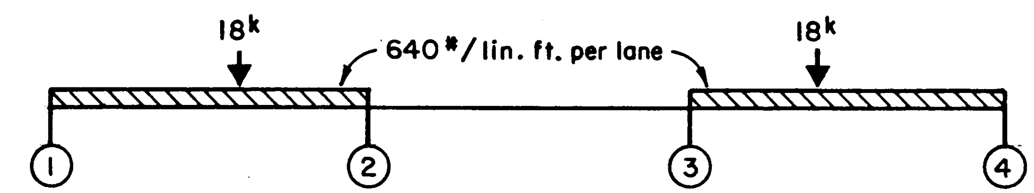
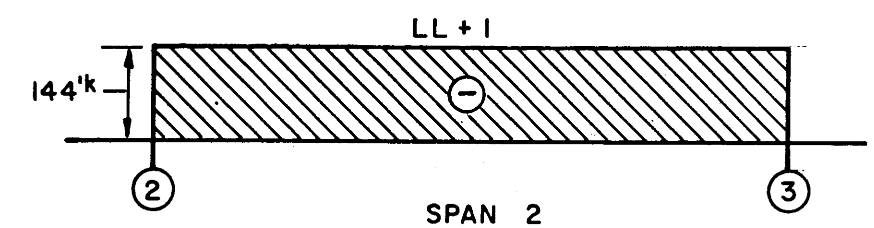


FIGURE 3-11.1



The live load moment in Span 2 produced by this loading is as shown in Figure 3-11.1.

This loading would modify the envelope curve of Figure 3-10 by the dotted curve shown between the 0.4 and 0.5 point of Span 2. It would be of no practical value in determining cutoff of negative bars in a concrete span, because the value falls below the resisting moment of the longitudinal bars which would normally be carried through.

3.6.3 Envelope Curve for Positive Moment in the Interior Span of Three Continuous Spans

It is necessary to compute the moments at the 0.2 and 0.5 points in the span for combined dead load and live load and then pass a 2d degree parabola through them.

Positions of truck for live load moment at the 0.2 and 0.5 points are shown in Figures 3-12 and 3-13 respectively.

FIGURE 3-12

POSITION OF TRUCK FOR + $M_{0.2}$ IN SPAN 2

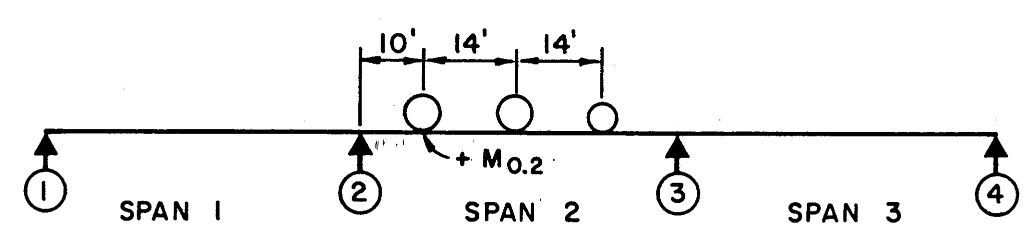


FIGURE 3-12.1

MOMENT DIAGRAM FOR LOAD POSITION OF FIGURE 3-12

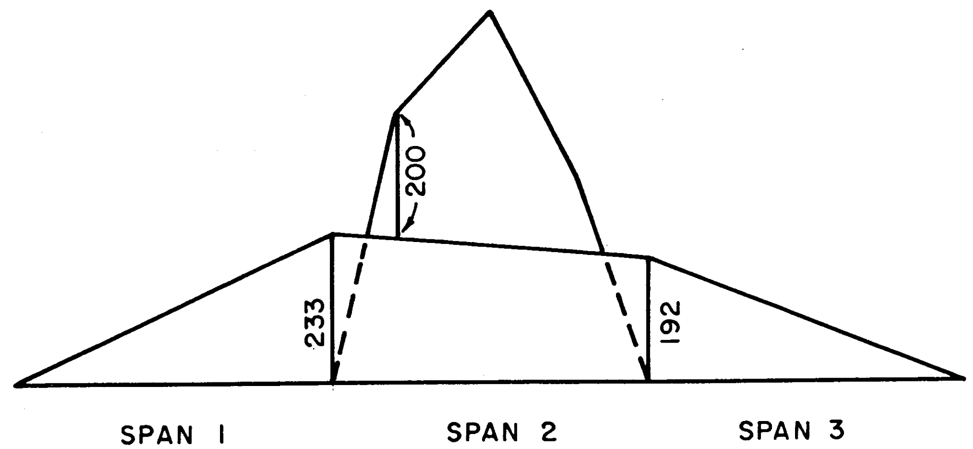


FIGURE 3-13

POSITION OF TRUCK FOR + $M_{0.5}$ IN SPAN 2

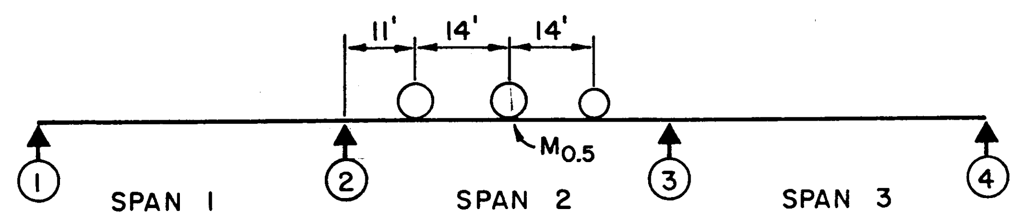
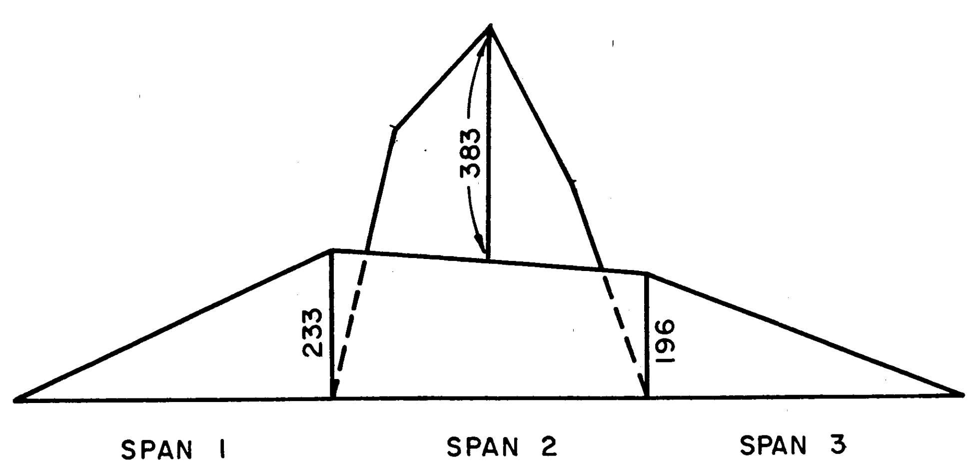


FIGURE 3-13.1

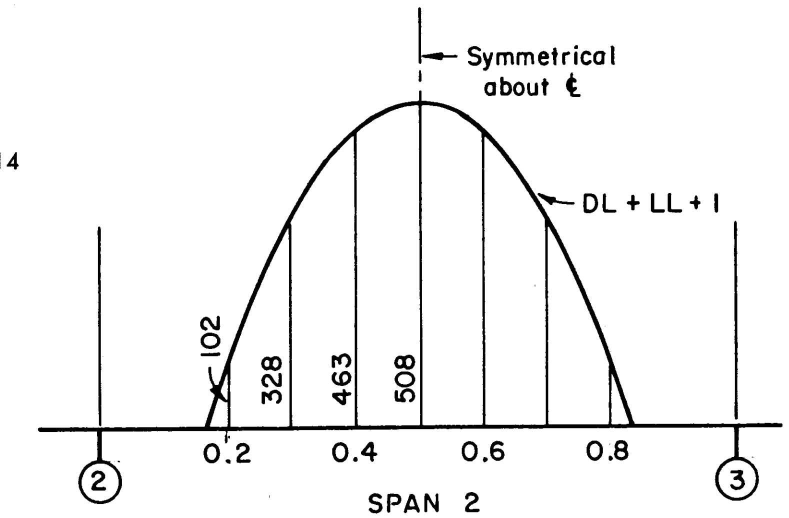
MOMENT DIAGRAM FOR LOAD POSITION OF FIGURE 3-13



The LL+I positive moments at the 0.2 and 0.5 points are 202 ft.[k] and 383 ft.[k] respectively.

Combining these moments with the dead load moments of Figure 3-3 and passing a parabola through them gives the envelope curve of Figure 3-14 for DL+LL+I.

FIGURE 3-14



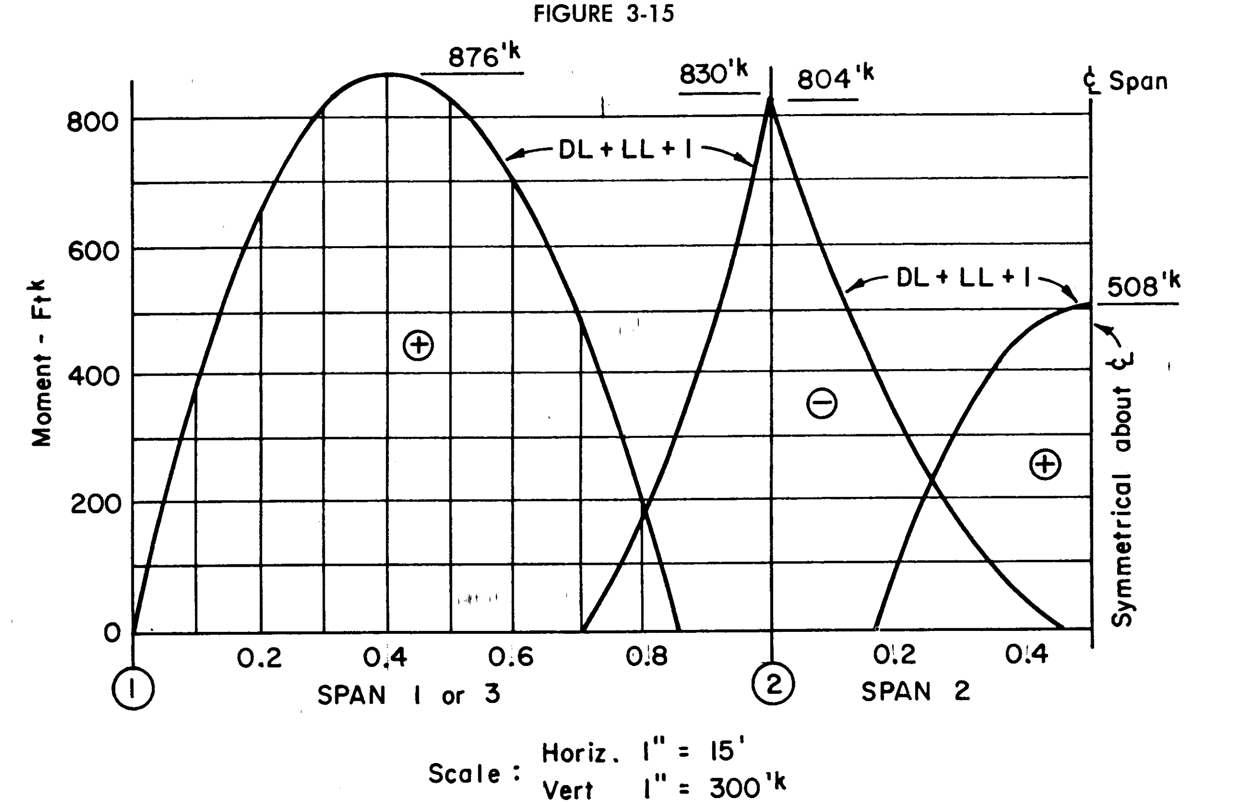
3.6.4 Complete Moment Curve for Three Continuous Spans

A complete moment curve can now be constructed for the three span continuous structure by combining the moment curves of Figures 3-4, 3-8, 3-10, and 3-14.

Both positive and negative moments are drawn above the zero line.

Some designers prefer to plot the positive moment above the zero line and the negative below. This is a matter of personal preference; both methods have merits.

FIGURE 3-15



3.7.0 APPLICATION OF LIVE LOAD TO CONTINUOUS SPANS FOR DETERMINATION OF MAXIMUM SHEARS

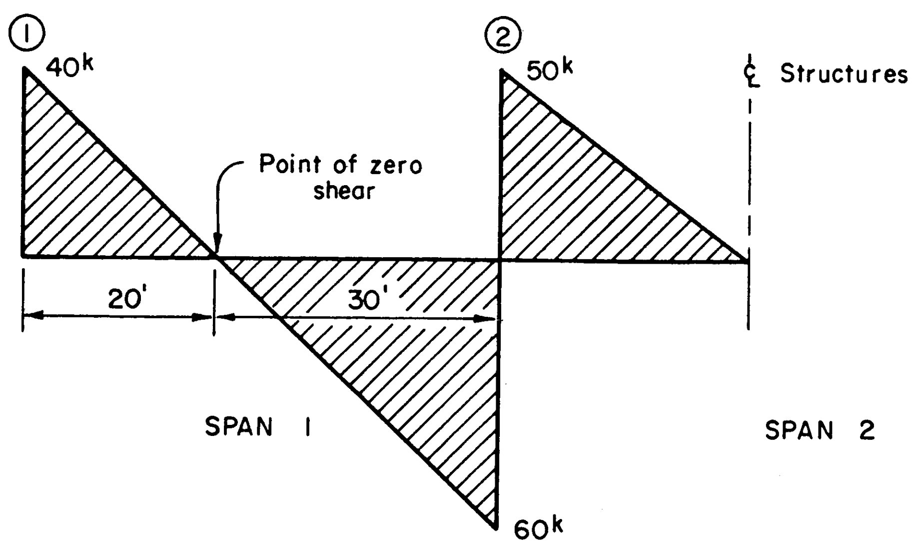
3.7.1 Shear in End Span

The same three-span continuous structure is utilized here to illustrate the placing of loads in continuous spans for determination of live load shear. Dead load and live load are as stipulated in Article 3.6.1.

Figure 3-16 below shows the dead load shear curve for a uniform load of 2.0 kips per lin. ft. over the three spans.

FIGURE 3-16

DEAD LOAD SHEAR



It can be found that the maximum live load shear at Support 1 is governed by the truck loading. If a truck were passed across Span 1 such that it would be in proper position at each point to produce maximum shear the live load shear curve would be as shown by the dotted lines in Figure 3-17 below. This is for 1.5 wheel lines.

For a practical utilization of the curve of maximum shear it is necessary to determine the LL+I shear at only three points; at Support 1, at the 0.4 point which is 20′ from the left support, and left of Support 2. These points are then connected by straight lines as indicated by the solid lines in Figure 3-17.

The variation between the actual shear curve and the straight line method is of no practical concern as other empirical assumptions introduce a greater error.

Following are the load positions for determining LL+I shears at the points mentioned above. These shears can be determined by the method of Moment Distribution; however, a short cut method of shear determination has been developed and is shown in Appendix 3-C, "Live Load Shear Design Data."

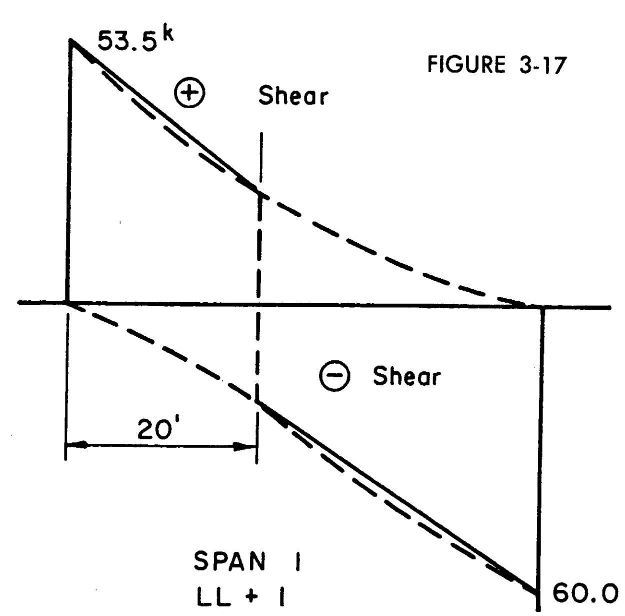


FIGURE 3-17

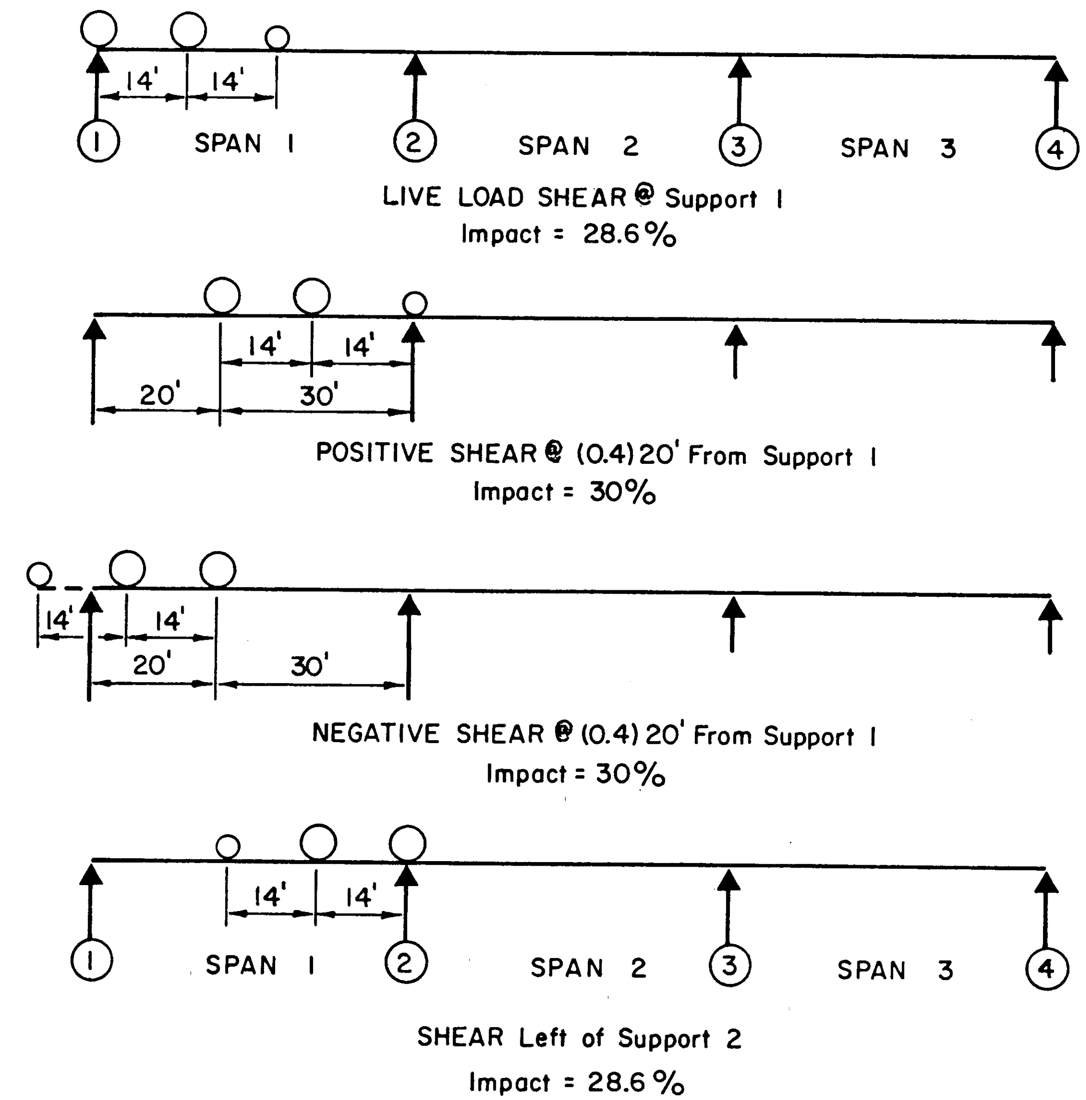
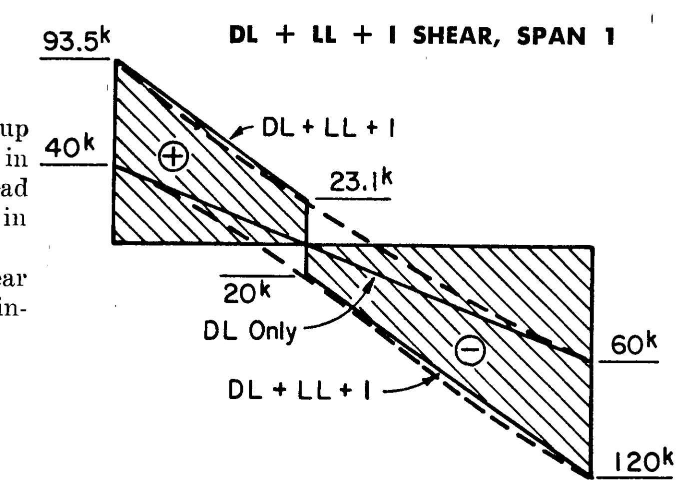


FIGURE 3-18

DL + LL + I SHEAR, SPAN 1

The shear diagram for use in determining stirrup spacing in concrete beams, or for stiffener spacing in plate girders is constructed by combining the dead load curve in Figure 3-16 with the LL + I curve in Figure 3-17. This curve is shown in Figure 3-18.

From this curve the designer can scale off the shear values at any convenient interval for use in determining stirrup spacing, etc.



Generally it is not necessary to use the whole curve for stirrup spacing. That portion near the point of zero dead load shear has values such that nominal stirrup spacing (limited by specification to a maximum of three-fourths the effective depth) can be used.

Subsequent sections of this course will cover the utilization of shear curves for the various types of structures.

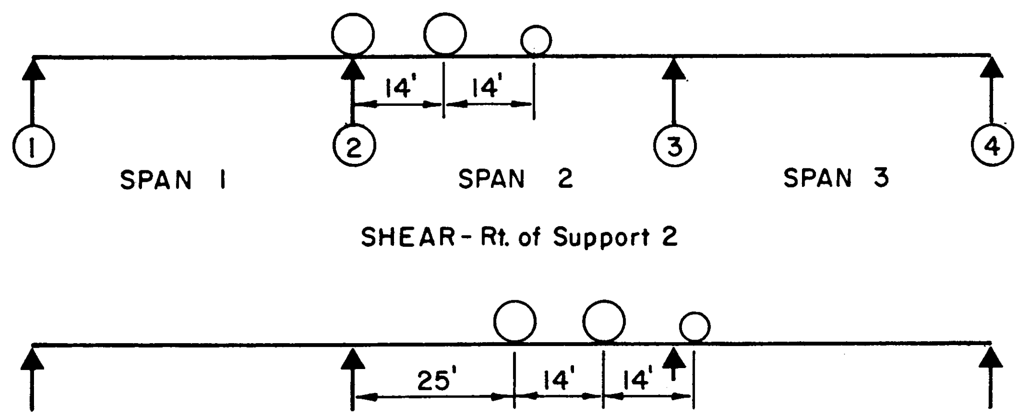
3.7.2 Shear in Center Span

In this structure the DL + LL + I shear curve for Span 2 is symmetrical about centerline of span, therefore, it is necessary to compute the shear at the support, and at the centerline only.

Following are the load positions for computing the required LL + I shears in Span 2. The truck governs.

FIGURE 3-19

LL + I SHEAR, SPAN 2



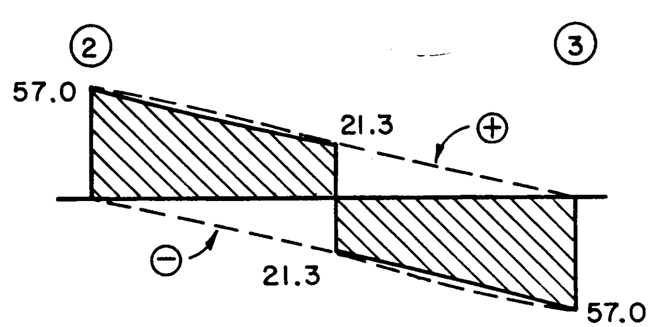
POSITIVE SHEAR - Center of Span 2

Note : Negative shear has the same value and is determined by placing the truck on the other end of the span

POSITIVE SHEAR—Center of Span 2

Note—Negative shear has the same value and is determined by placing the truck on the other end of the span.

FIGURE 3-20

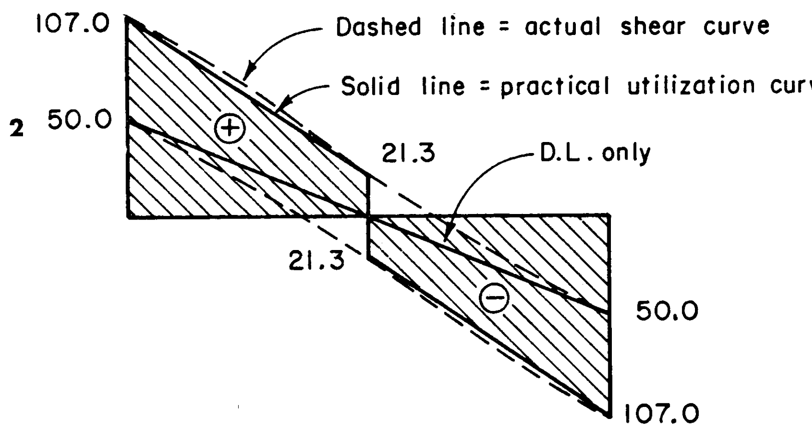


LL + I SHEAR DIAGRAM

The LL + I shear curve of Figure 3-20 is then combined with the dead load shear curve of Figure 3-16 to obtain the curve of maximum shear for Span 2, shown in Figure 3-21 below.

FIGURE 3-21

DL + LL + I SHEAR, SPAN 2



3.8.0 CONCLUSION OF SECTION 3

The data presented in this section cover one method of accomplishing the loading analysis. As a designer gains added experience in the handling of loads and placing them in structures for maximum conditions, he develops his own methods.

It has been assumed in the example covering the continuous structure that the reader has a knowledge of moment distribution or some other method which would enable him to make the necessary computations. Computing machinery can be used to do much of the mathematics of loadings where the volume of work warrants, but this machinery does not develop one's design judgment. A good knowledge of fundamentals forms the best foundation for understanding loadings.

Subsequent sections of this manual will cover in more detail the application of loads to specific structures.