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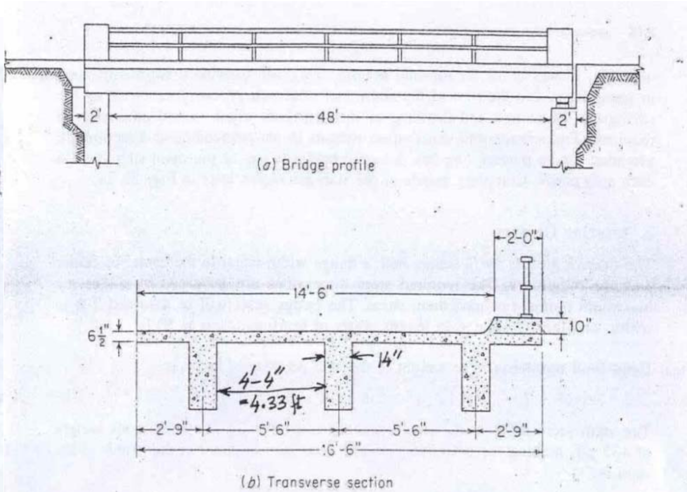
22.6 DESIGN OF A T BEAM OR DECK-GIRDER BRIDGE

a. Data and Specifications

A T-beam or deck-girder bridge similar to that shown in Fig. 22.3 is to be designed for the following conditions:

Clear span	48 ft
Clear width	29 ft
Live loading	HS20
Concrete strength f_c'	3000 psi
Grade 40 reinforcement	

The design is to meet the AASHTO specifications. The bridge will consist of six parallel girders supporting a floor slab, as shown in Fig. 22.19.



(a) Bridge profile

(b) Transverse section

b. Slab Design

Since the slab will be poured monolithically with the girders and will be fully continuous, the span will be taken as the clear distance between girders. Under the assumption that the girders will be 14 in. wide, the clear span will be 4 ft 4 in. For a total thickness of slab of 6 in. (including a $\frac{3}{4}$ in. wearing surface), with a 15 psf allowance for possible future protective covering, the total dead load is 90 psf. A coefficient of $\frac{1}{10}$ for positive and negative dead load moments will be assumed in the absence of definite specification values. The dead load positive and negative moments are

$$\frac{1}{10} \times 90 \times 4.33^2 = 169 \text{ ft-lb}$$

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e. Distribution of Wheel Loads on Concrete Slabs

The pertinent rules for the distribution of wheel loads on concrete slabs and some additional design requirements are as follows.

(1) Span lengths. For simple spans, the span length shall be the distance center to center of supports but shall not exceed clear span plus thickness of slab.

The following effective span lengths shall be used in calculating the distribution of loads and bending moments for slabs continuous over more than two supports:

Slabs monolithic with beam (without haunches):

S = clear span

Slabs supported on steel stringers:

S = distance between edges of flanges plus one half the stringer flange width

Slabs supported on timber stringers:

S = clear span plus one half the thickness of the stringer

(2) Edge distance of wheel load. In designing slabs, the centerline of the wheel load shall be assumed to be 1 ft from the face of the curb.

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(3) **Bending moment.** Bending moment per foot width of slab shall be calculated according to the methods given under cases 1 and 2 below. In both cases,

S = effective span length as defined under above heading, ft
E = width of slab over which wheel load is distributed, ft
P = load on one rear wheel of truck
P_{15} = 12,000 lb for H15 loading
P_{20} = 16,000 lb for H20 loading

Case 1: Main reinforcement perpendicular to traffic (spans 2 to 24 ft inclusive). The live load moment for simple spans shall be determined by the following formulas (impac not included):

HS20 loading:

$$\frac{S+2}{32}P_{20} = \text{moment, ft-lb per foot width of slab} \qquad (22.2a)$$

HS15 loading:

$$\frac{S+2}{32}P_{15} = \text{moment, ft-lb per foot width of slab} \qquad (22.2b)$$

In slabs continuous over three or more supports, a continuity factor of 0.8 shall be applied to the above formulas for both positive and negative moments.

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For live load moment computations, case 1 of Sec. 22.3e applies:

$$0.80\ \frac{S+2}{32}\ P_{20} = \frac{0.80 \times 6.33 \times 16{,}000}{32} = 2530 \text{ ft-lb}$$

Since the loaded length is small, the impact coefficient is 0.30, and the impact moment is 760 ft-lb. The total positive and negative moments are 3459 ft-lb. Then

$$d = \sqrt{\frac{2 \times 3459 \times 12}{1200 \times 0.375 \times 0.866 \times 12}} = 4.19 \text{ in}$$

With the addition of 1 in. of protective concrete below the center of an assumed $\frac{3}{4}$ in. bar, and with the $\frac{3}{4}$ in. wearing surface (not considered structurally effective), the total thickness is 6.32 in. A $6\frac{1}{2}$ in. slab will be used, with an effective depth of 4.37 in. The dead load estimate need not be revised and

$$A_s = \frac{3459 \times 12}{20{,}000 \times 0.875 \times 4.37} = 0.54 \text{ in}^2$$

which is furnished by No. 6 bars 10 in. on centers. To avoid excessive bending of bars, and since short spans are involved, straight bars will be used in the slab,

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with No. 6 bars at 10 in. top and bottom. This will provide a surplus of steel in some areas, but the cost of the additional steel will probably be offset by the savings in fabrication and handling of the bent bars which would otherwise be required. Temperature and distribution stresses in the perpendicular direction are provided for by placing five No. 5 bars directly on top of the main slab steel in each slab panel. Complete details of the slab are shown later in Fig. 22.24.

c. Interior Girders

The interior girders are T beams with a flange width equal to the center-to-center distance of girders. The required stem dimensions are governed by either the maximum moment or maximum shear. The bridge seats will be assumed 2 ft in width, and the effective span length center of bearings taken as 50 ft.

Dead load moments. The weight of the slab per foot of beam is

$$96 \times 5.5 = 528 \text{ plf}$$

The stem section below the slab is assumed as 14 × 30 in, which adds weight of 437 plf, making the total 965 plf. The dead load moment at the center of the span is

$$M_d = \frac{1}{8} \times 965 \times 50^2 = 302{,}000 \text{ ft-lb}$$

To determine the points at which some of the horizontal steel may be cut off, it is necessary to compute the moment at some sections between the point of maximum moment and the support. At 10 ft from the support, the dead load moment is 192,800 ft-lb; at 20 ft, 289,000 ft-lb.

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(2) **Bending moment in stringers and longitudinal beams.** In calculating bending moments in longitudinal beams or stringers, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution shall be determined as follows:

(a) *Interior stringers.* Supporting concrete floors shall be designed for loads determined in accordance with the following table, in which S is the average spacing of stringers in feet:

Floor system	One traffic lane, fraction of a wheel load to each stringer	Two or more traffic lanes, fraction of a wheel load to each stringer
Concrete slab on steel I-beam stringers and prestressed concrete girders	$S/7.0$ (S_{max} = 10 ft)[a]	$S/5.5$ (S_{max} = 14 ft)[a]
Concrete slab on concrete stringers	$S/6.0$ (S_{max} = 6 ft)[a]	$S/5.0$ (S_{max} = 10 ft)[a]
Concrete box girder	$S/8.0$ (S_{max} = 12 ft)[a]	$S/7.0$ (S_{max} = 16 ft)[a]

[a] If S exceeds the value in parentheses, the load on each stringer shall be the reaction of the wheel loads, under the assumption that the flooring between the stringers acts as a simple beam.

(b) The live load supported by *outside stringers* shall be the reaction of the truck wheels, under the assumption that the flooring acts as a simple beam between stringers.

(c) The *combined capacity* of all the beams in a span shall not be less than the total live and dead load in the panel.

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Live load moments. The absolute maximum live load moment will occur with an HS20 truck on the bridge in the position† shown in Fig. 22.20. With the distribution of loads as specified in Sec. 22.3f, each interior girder must support $5.50/5.0 = 1.10$ wheel loads per wheel. Therefore the load from the rear wheel is $1.1 \times 16{,}000 = 17{,}600$ lb, and that from the front wheel is $1.1 \times 4000 = 4400$ lb:

$$R_L = \left[17{,}600(36.6 + 22.6) + 4400 \times 8.6\right]\frac{1}{50} = 21{,}600 \text{ lb}$$

$$M_{\max} = 21{,}600 \times 27.4 - 17{,}600 \times 14 = 346{,}000 \text{ ft-lb}$$

Ten feet from the left support, the maximum live load moment occurs with the rear trailer wheels at that point and the front wheels 38 ft from the left support. With this position of the loads,

$$R_L = \left[17{,}600(40 + 26) + 4400 \times 12\right]\frac{1}{50} = 24{,}400 \text{ lb}$$

$$M_{10} = 24{,}400 \times 10 = 244{,}000 \text{ ft-lb}$$

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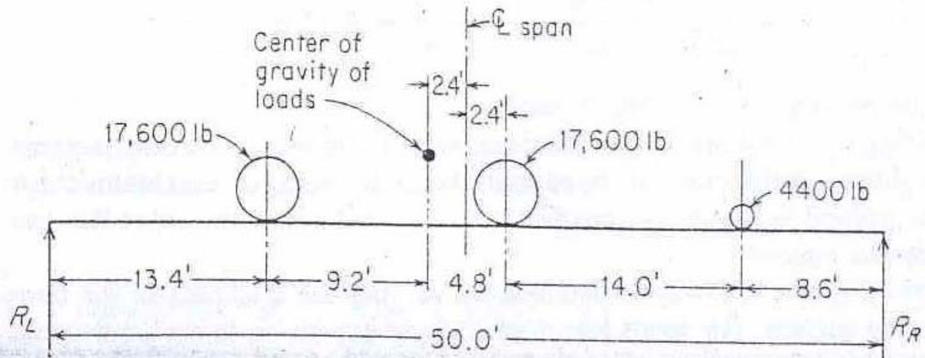

FIGURE 22.20
Position of wheel loads for maximum live load moment.

Similarly,

$$M_{20} = 330{,}000 \text{ ft-lb}$$

Impact moments. In computing impact moments for a girder such as this, it is usual to consider the whole span as the loaded length. The impact coefficient is therefore $50/(50 + 125) = 0.285$, and the impact moments are as follows:

$$M_{max} = 98{,}700 \text{ ft-lb}$$

$$M_{20} = 94{,}100 \text{ ft-lb}$$

$$M_{10} = 69{,}600 \text{ ft-lb}$$

Maximum total moments. The sum of the maximum dead load, live load, and impact moments is 746,000 ft-lb. The total moment at the 20 and 10 ft points are 713,000 and 506,400 ft-lb respectively.

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Dead load shears. The maximum dead load shear at the end of the beam is $965 \times 25 = 24{,}100$ lb. Ten feet from the support, the shear is 14,450 lb; 20 ft from the support, it is 4800 lb.

Live load shears. The absolute maximum shear occurs with the truck on the span in the position shown in Fig. 22.21. The maximum live load shear is 30,600 lb.

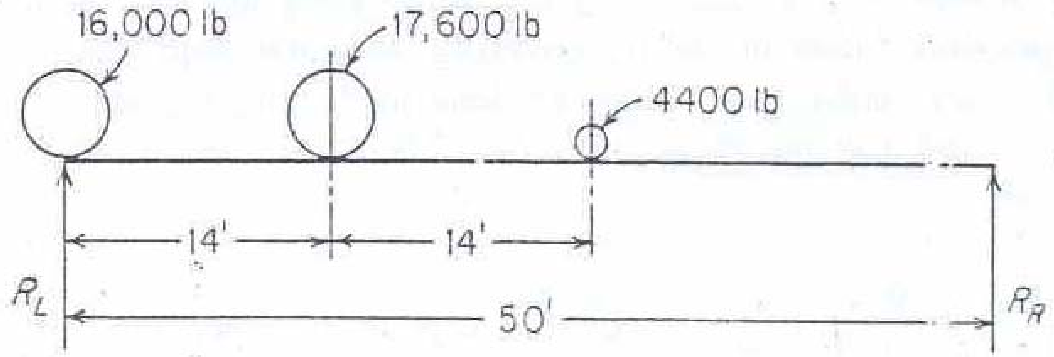

FIGURE 22.21
Position of wheel loads for maximum live load shear.

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The maximum shears at the 10, 20, and 25 ft points are 24,300, 16,400, and 12,700 lb respectively.

Impact shears. For loaded lengths of 50, 40, 30, and 25 ft, the impact shears are as follows:

End shear	8700 lb
10 ft section	6900 lb
20 ft section	4700 lb
Midspan section	3600 lb

Total shears. The total shears are as follows:

End shear	63,400 lb
10 ft section	45,650 lb
20 ft section	25,900 lb
Midspan section	16,300 lb

Determination of cross section and steel area. According to the AASHTO specification, the nominal shear stress at service load is to be calculated from $v = V/b_w d$, where b_w = web width. In beams with web reinforcement the nominal shear stress v may not exceed $4.95\sqrt{f'_c}$. In sizing the web area for the present design a maximum shear of $2.95\sqrt{f'_c} = 162$ psi will be used. In this case,

$$b_w d = \frac{V}{v} = \frac{63,400}{162} = 391 \text{ in}^2$$

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If b_w is taken as 14 in., the required d is 28.0 in. If three rows of No. 11 bars are assumed, with 2 in. clear between rows and 2.5 in. clear below the bottom row to allow for stirrups and concrete protection, a total depth of 34.56 in. is obtained. A 36 in. total depth will be used, resulting in an effective depth of 29.4 in. The depth of the stem below the slab is then $36 - 6\frac{1}{2} = 29\frac{1}{2}$ in. This is so close to the assumed value that the dead load moments need not be revised. The required tensile steel area is then

$$A_s = \frac{M}{f_s(d - t/2)} = \frac{746{,}700 \times 12}{20{,}000(29.4 - 5.75/2)} = 16.90 \text{ in}^2$$

which will be furnished by 11 No. 11 bars.

A check of the maximum stress in the concrete shows a maximum compression of 1190 psi, practically equal to the allowable stress of 1200 psi.

The lower four tensile bars, somewhat more than one third the area, will be extended into the support. The upper two layers will be cut off 15 diameters or one twentieth of the span beyond the point at which they are no longer required, in accordance with the specification. The critical extension in this case is one twentieth of the span, or 30 in. Cutoff points are found graphically as in Fig. 22.22.

Note that extra stirrups must be provided along each terminated bar over a distance from their termination point equal to $0.75d = 22$ in. The excess stirrup area required is $A_v = 60b_w s/f_y$, and the spacing is not to exceed $d/8\beta_b$, where

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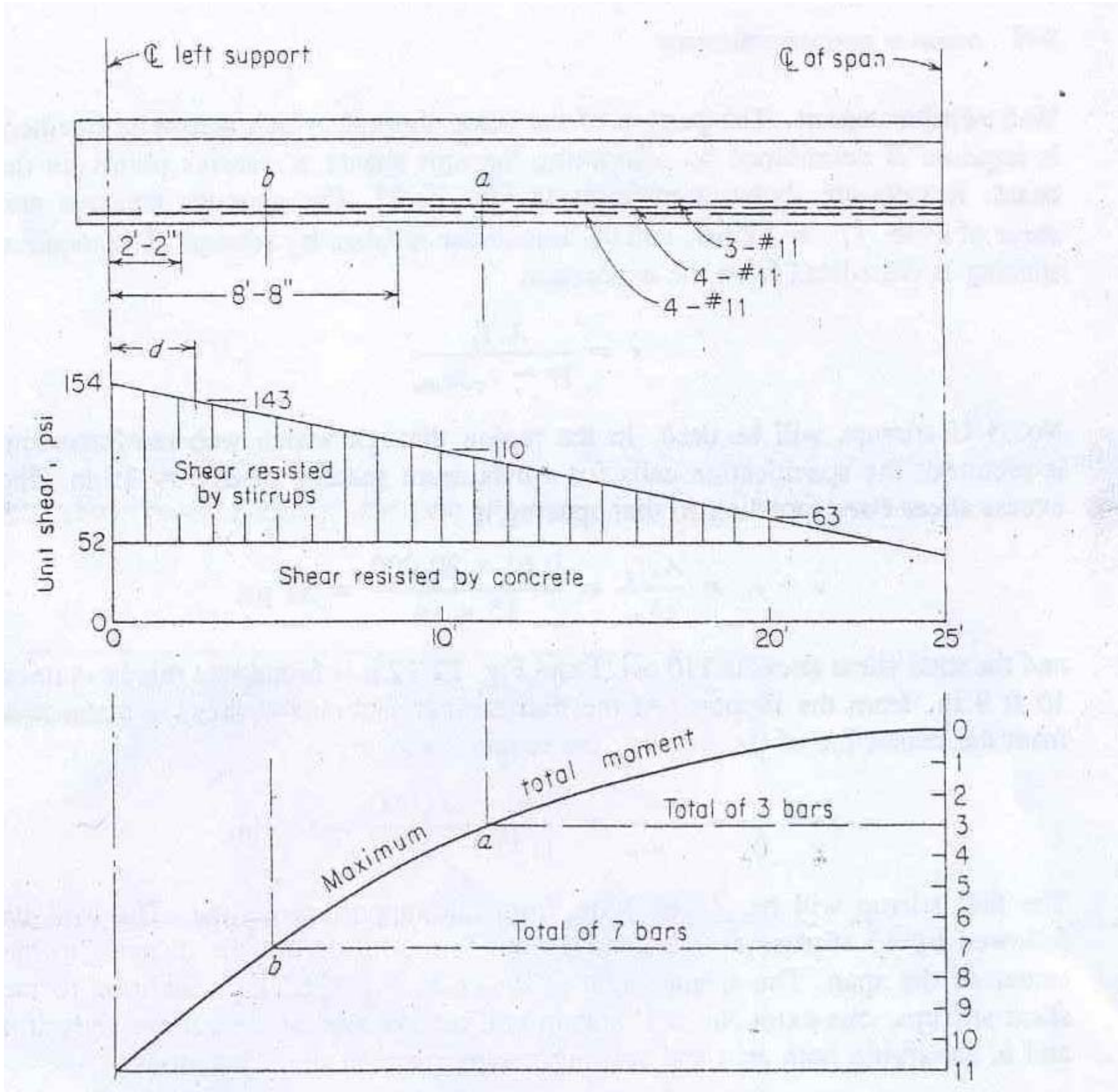

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β_b is the ratio of the area of reinforcement cut off to the total area of reinforcement at the section. In the present case, this gives maximum spacings of 13.50 and 7.35 in. at locations *a* and *b* respectively (see Fig. 22.22). The selection of these special stirrups will be deferred pending design of the shear stirrups.

All bars must be extended a full development length, 46 in. in the present case, past locations of maximum stress. Comparison with the selected cutoff points of Fig. 22.22 confirms that this requirement is met for all bars. In addition, the specification requires that at simple supports the development length should not exceed $M/V + l_a$, where M is the computed moment capacity of the remaining tensile bars, V is the maximum shear force at the section, and l_a is the extension of the bars beyond the centerline of the support. For the four No. 11 bars remaining, which have an effective depth of 32.8 in., $M = 312{,}000$ ft-lb. For $l_a = 9$ in., the computed maximum length is 68 in., well above the development length of 46 in. for the No. 11 bars.

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Web reinforcement. The portion of the beam through which web reinforcement is required is determined by computing the unit shears at various points on the beam. Results are shown graphically in Fig. 22.22. The concrete resists a unit shear of $0.95\sqrt{f'_c} = 52$ psi, and the remainder is taken by stirrups. The required spacing is calculated from the expression

$$s = \frac{A_v f_s}{(v - v_c)b_w}$$

No. 5 U stirrups will be used. In the region through which web reinforcement is required, the specification calls for a maximum spacing of $d/2 = 15$ in. The excess shear corresponding to that spacing is

$$v - v_c = \frac{A_v f_s}{s b_w} = \frac{0.61 \times 20{,}000}{15 \times 14} = 58 \text{ psi}$$

and the total shear stress is 110 psi. From Fig. 22.22 it is found that this is attained 10 ft 9 in. from the support. At the first critical section for shear, a distance d from the centerline of the support, the required spacing is

$$s = \frac{A_v f_s}{(v - v_c)b_w} = \frac{0.61 \times 20{,}000}{(143 - 52)14} = 9.6 \text{ in.}$$

The first stirrup will be placed 3 in. from the support centerline. This will be followed by 13 stirrups at 9 in. and 12 at 15 in., filling out the distance to the center of the span. The arrangement is shown in Fig. 22.24. In addition to the shear stirrups, one extra No. 5 U stirrup will be included at the cut bar ends at a and b, satisfying both area and spacing requirements at those locations.

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d. Exterior Girders

The exterior girders are identical in cross section with the interior girders, because the raised curb section will be poured separately and cannot be counted on to participate in carrying loads.

Moments. In addition to the 965 plf dead load obtained for the interior girders, the safety curb adds 250 plf, producing a total dead load of 1215 plf and a maximum dead load moment at the center of the span of 380,000 ft-lb. A portion of each wheel load that rests on the exterior slab panel is supported by the exterior girder. That portion is obtained by placing the wheels as close to the curb as the clearance diagram will permit and treating the exterior slab panel as a simple beam. The position is shown in Fig. 22.23, and the proportion of the load is $4.25/5.50 = 0.773$. The longitudinal position of the load which will produce the absolute maximum bending moment is the same as for the interior girders. The absolute maximum live load moment can therefore be obtained by direct proportion:

$$M_{max} = \frac{0.773}{1.10} \times 346{,}000 = 243{,}000 \text{ ft-lb}$$

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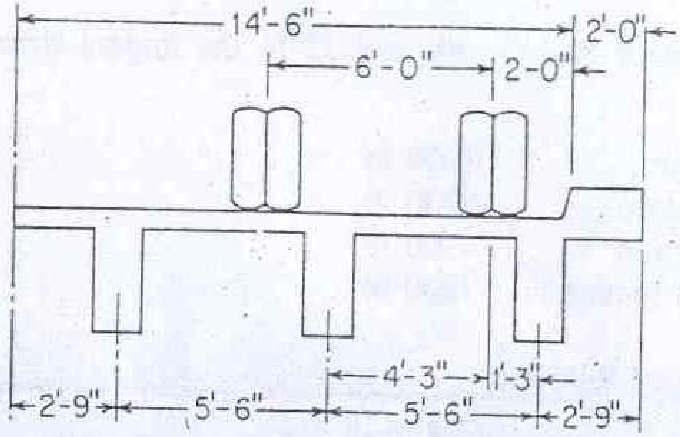

FIGURE 22.23
Lateral position of wheel loads for design of exterior girders.

The impact moment is $0.285 \times 243{,}000 = 69{,}300$ ft-lb, and the total maximum moment is 692,300 ft-lb.

Shears. The maximum dead load shear is

$$V_D = 1215 \times 25 = 30{,}400 \text{ lb}$$

The maximum live load shear is proportional to the maximum live load shear in the interior girders:

$$V_L = \frac{0.773}{1.10} \times 30{,}600 = 21{,}500 \text{ lb}$$

The impact shear at the support is 6100 lb. The total shear at the support is then 58,000 lb. Shears at other points are found similarly.

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Determination of cross section and reinforcement. Once the shears and moments due to dead and live loads and impact are obtained, the design of the exterior girders would follow along the lines of that for the interior girders. The specification stipulates that in no case shall an exterior stringer have less carrying capacity than an interior stringer. This controls in the present design, and the exterior girders will duplicate the interior girders.

e. Miscellaneous Details

Diaphragms. A transverse diaphragm will be built between the girders at either end of the bridge. The chief function of these diaphragms is to furnish lateral support to the girders; with some abutment details, the diaphragms also serve to prevent the backfill from spilling out onto the bridge seats. A similar diaphragm will be built between the girders at midspan. Such intermediate diaphragms are required for all spans in excess of 40 ft and serve to ensure that all girders act together in resisting the loads.

Fixed bearing. A fixed bearing is provided at one end. Vertical dowels are placed in the breast wall of the abutment and are bent so as to project into the longitudinal beams or deck slab. Horizontal dowels are embedded in the deck and approach slabs. The diaphragm rests directly on the bridge seat.

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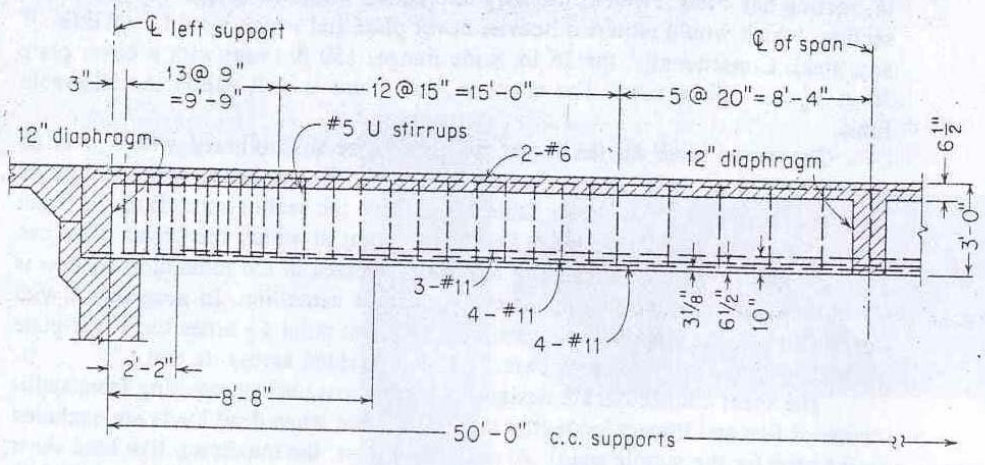
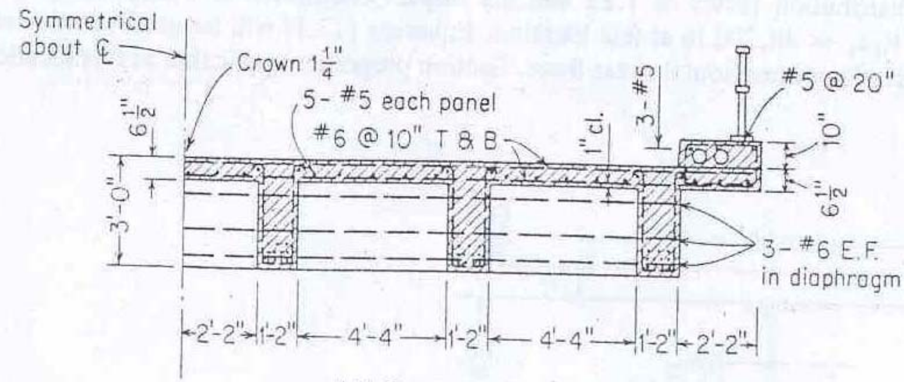
Expansion bearing. Elastomeric pads are provided at one end of the span. The bottom of the diaphragm is flush with the bottoms of the beams and is not in contact with the bridge seat.

Waterproofing and drainage. All joints will be filled with a mastic compound to prevent water from seeping through the joints. Removal of surface water will be accomplished by crowning the roadway $1\frac{1}{4}$ in. and pitching the gutters toward the ends by providing a camber of $2\frac{1}{2}$ in. at the center. Besides facilitating drainage, this camber serves to prevent the appearance of sag that would be evident if the girders were at the same level throughout the span.

Full details are shown in Fig. 22.24.



(a) Details of interior girder



(b) Cross section