Xalla Toone
1404143.

Lecture – 1

Date: ___ / ___ / _____

CE 316 / CE 324W, Concrete Structures Sessional-I.

Syllabus: ► lecture 1: Design of a Slab Bridge
► lecture 2 – 5: Design of a Balanced Cantilever Bridge
► lecture 6 – 10: Design of a Low Rise Building

Bridge: A bridge is a structure providing passage over an obstacle without closing the way beneath. The required passage may be for a road, a railway, pedestrians, a canal or a pipeline. The obstacle to be crossed may be a river, a road, railway or a valley. The communication route over the bridge may be a roadway, footpath, a cycle track, railway track or tramway.

Components of a bridge: The main parts of a bridge structure are-

- ► Decking, consisting of deck slab, girder, trusses, etc,
- ► Bearings for the decking
- ► Abutments and piers, viaduct
- ► Foundations for the abutments and the piers
- ► River training works, like revetment for slopes at abutments, aprons at
- ► bed level, etc.
- ► Approaches to the bridge to connect the bridge proper to the roads on either side and
- ► Handrails, guard stones, etc.

The components above the level of bearings are grouped as superstructure, while the parts below the bearing level are classed as substructure. The portion below the bed level of a river bridge is called the foundation. The components below the bearing and above the foundation are often referred as substructure.

Classification of Bridges: Bridges may be classified in many ways, as below;

(1) According to function as aqueduct (canal over a river), viaduct (road or railway over a valley), pedestrian, highway, railway, road-cum-rail or a pipeline bridge.

(2) According to the material of construction of superstructure as timber, masonry, iron, steel, reinforced concrete, prestressed concrete, composite or aluminium bridge.

(3) According to the form or type of superstructure as slab, beam, truss, arch, cable stayed or suspension bridge.

(4) According to the interspan relations as simple, continuous or cantilever bridge.

(5) According to the position of bridge floor relative to the superstructure as deck, through, half-through or suspended bridge.

(6) According to the method of connections of the different parts of the superstructure, particularly for steel construction, as pin-connected, riveted or welded bridge.

(7) According to the road level relative to the highest flood level of the river below, particularly for a highway bridge, as high-level or submersible bridge.

(8) According to the method of clearance for navigation as high-level, movable-bascule, movable-swing or transporter bridge.
(9) According to span length as culvert (less than 8 m), minor bridge (8 to 30 m), major bridge (above 30 m) or a long-span bridge (above 120 m).
(10) According to degree of redundancy as determinate or indeterminate bridge.
(11) According to the anticipated type of service and duration of use as permanent, temporary, military (pontoon, bailey) bridge.

Table 1: Span lengths for bridge structures

Type	Material	Span range		Maximum constructed (meter)
		meter	feet	
Slab	Concrete	0-12	0-40	
Beam	Concrete	12-210	40-700	Bendrof 208
	Steel	30-300	100-860	Rio-Niteroi 300
Truss	Steel	90-550	300-1800	Quebec 549
Arch rib	Concrete	90-130	300-1000	Gladesville 305
	Steel	120-370	400-1200	Fremont 382
Arch truss	Steel	240-520	800-1700	Bayonne 504
Cable stayed	Concrete	90-270	300-800	Wall 267
	Steel	90-350	300-1100	Duisburg 350, Great Belt 600
Suspension	Steel	300-1400	1000-4500	Verrazano 1298, Humber 1400

Design Specifations

(1) AASHTO specification: used in designing highway bridges
(2) AREA specification: used in designing railway bridges
[AASHTO: American Association of State Highway & Transportation Officials]
[AREA: American Railway Engineering Association]

Design Method

(1) AASHTO Service Load Design / AASHTO Allowable Stress Design
(2) AASHTO Load Factor Design

Loads on Bridges: The design of superstructure or for that matter any other component for a bridge, is based on a set of loading conditions which the component must withstand. These loads may vary depending on duration (permanent or temporary), direction of action, type of deformation, and nature of structural action (shear, bending, torsion, etc.).

- ►BS 5400 loads – United Kingdom
- ►Ontario Highway Bridge Design Code (OHBDC) – Canada
- ►American Association of State of Highway & Transportation Officials (AASHTO) – USA

1. Dead Load
2. Vehicle Load
3. Impact effect
4. Wind Loading
5. Longitudinal Forces
6. Centrifugal Forces
7. Buoyancy
8. Water Current Forces
9. Thermal Forces
10. Deformation & Horizontal Forces
11. Erection Stresses
12. Seismic Forces

1. Dead Load: The dead load on a superstructure is the aggregate weight of all superstructure elements (elements above bearings) such as the deck, wearing coat, railings, parapet, stiffeners and utilities. It will be seen in the design that the first step is to calculate the dead load of all the elements.

Table2: Unit weights of materials commonly used in highway bridges

Material	Unit weight (pcf)
Concrete	150
Steel	490
Cast Iron	450
Asphalt paving	140
Timber	35-60
Stone masonry	150-175
Aluminum	170

2. Live Load: The term live load means a load that moves along the length of the span. By this definition, a man walking on the bridge is also a live load. But a highway bridge is designed to withstand much more than just pedestrian loading. To give the designers the ability to accurately model the live load on a structure, hypothetical vehicles were evolved by AASHTO (American Association of State Highway and Transportation Officials). The loads are categorized based on their configuration and intensity.

3. Impact Effect: In order to account for the dynamic effects of the sudden loading of a vehicle on to a bridge structure, an impact factor is used as a multiplier for loads on certain structural elements. From the basic dynamics we know that a load that moves across a member introduces larger stresses than those caused by a standstill load. However, the basis of impact factors predicted by AASHTO is not fully known. It has been felt by the researchers that the impact factor is to a large extent depends on weight of the vehicle, its velocity, as well as the surface characteristics of the road. It is permitted to note that live load increases on account of the consideration of the impact effect.

AASHTO Live Loading

AASHTO Live Loading consists of:

(1) AASHTO design vehicle loading (consist of one truck)

(2) AASHTO design lane loading (consists of a series/train of trucks)

The present design vehicles contained in the AASHTO specifications were adopted in 1944. The loading consists of five loading classes

(1) AASHTO Truck/Wheel/Vehicle Loading (consist of one truck)

AASHTO Standard Trucks:

- H10-44 truck loading
- H15-44 truck loading
- H20-44 truck loading
- HS15-44 truck loading
- HS20-44 truck loading

H15-44 & H20-44: a single truck

HS15-44 & HS20-44: a single truck with a semi-trailer

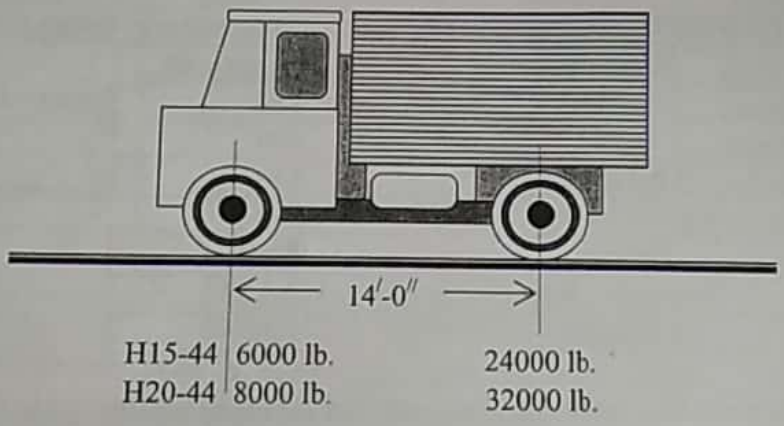


Figure 1: Standard H truck loading (axle load distribution)

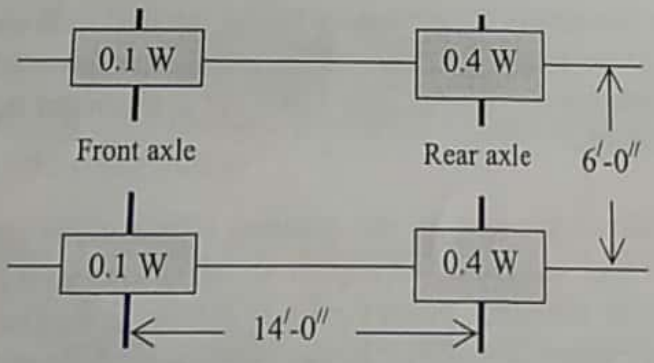


Figure 1: Standard H truck loading (wheel load distribution)

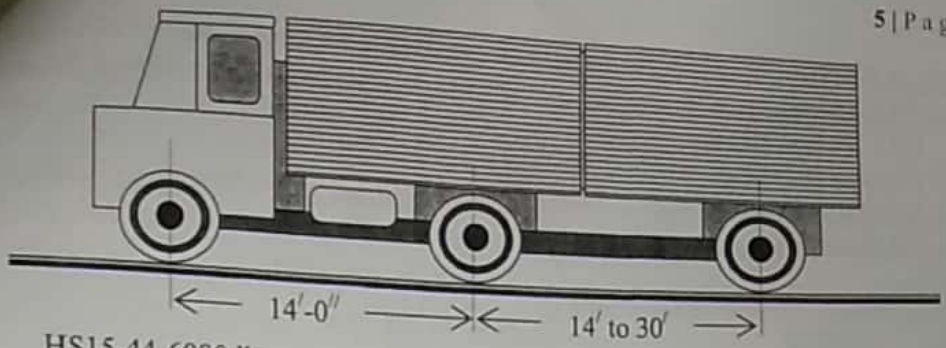


Figure 2: Standard HS truck loading (axle load distribution)

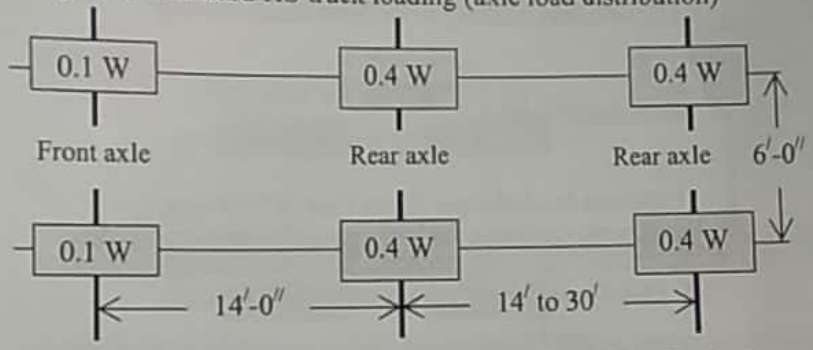
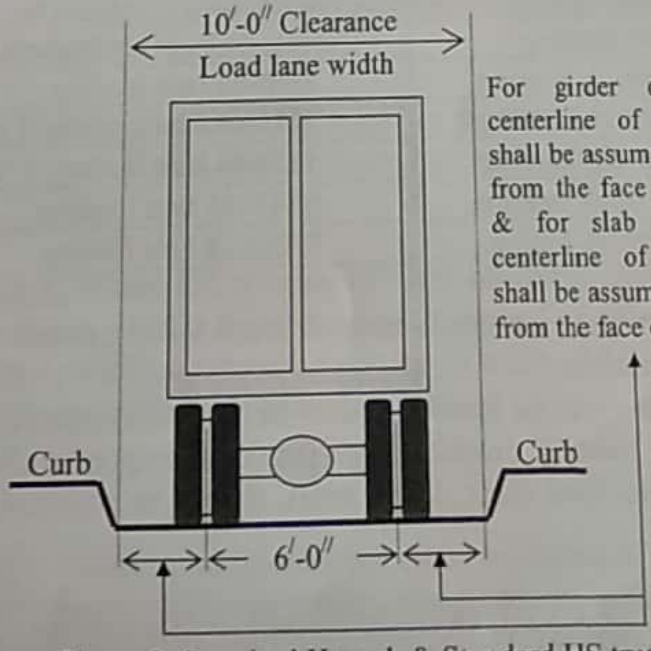


Figure 2: Standard HS truck loading (wheel load distribution)

In addition, the specification contains the equivalent lane loadings to replace truck loadings when they produce greater stresses than a single truck. Before the introduction of 1944 specifications the design line load consisted of a basic H truck, preceded and followed by a train of trucks that totaled three-quarters of the weight of the basic truck. In 1944, the HS truck was developed and equivalent lane loading took the place of the train of trucks.

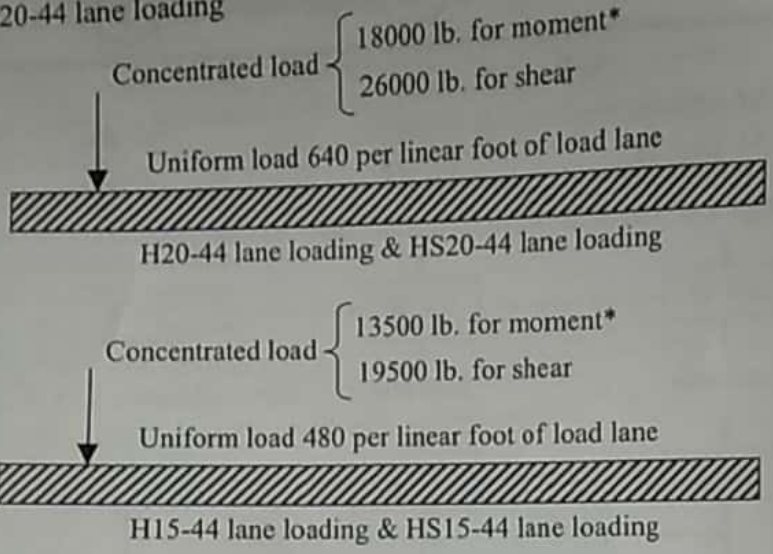


For girder design the centerline of the wheel shall be assumed to be 1 ft from the face of the curb & for slab design the centerline of the wheel shall be assumed to be 2 ft from the face of the curb.

Figure 3: Standard H truck & Standard HS truck

(2) AASHTO Lane Loading (consists of a series/train of trucks)

H10-44 lane loading
H15-44 lane loading
H20-44 lane loading
HS15-44 lane loading
HS20-44 lane loading



* For continuous spans, another concentrated load of equal load of equal weight shall be placed in the other span in the series in such a position as to produce maximum negative moment.

Figure 3: Equivalent H lane loading & HS lane loading

From above we see that every truck loading has corresponding equivalent lane loading.

Table: truck loading & corresponding lane loading (AASHTO)

Truck loading	Corresponding lane loading
H10-44 truck loading	H10-44 lane loading
H15-44 truck loading	H15-44 lane loading
H20-44 truck loading	H20-44 lane loading
HS15-44 truck loading	HS15-44 lane loading
HS20-44 truck loading	HS20-44 lane loading

Now, the question arises which one should be used (truck loading or lane loading) for live load stress computation.

In actual cases, these two types of loading should be separately placed over the bridge span. Which type loading produce large structural responses/effects (i.e. which gives maximum live load shear & moment), should be considered in the design.

When bridge span is small >> AASHTO Truck/Wheel Loading governs
When bridge span is large >> AASHTO Lane Loading governs

The type of loading used, whether lane loading or truck loading and whether the spans are simple or continuous, shall be the loading which produced the maximum stress.

Abnormal loading/Military loading: In war times, military trunk loading will pass over the bridge. So, a series of trunk move over the bridge should be considered in design.

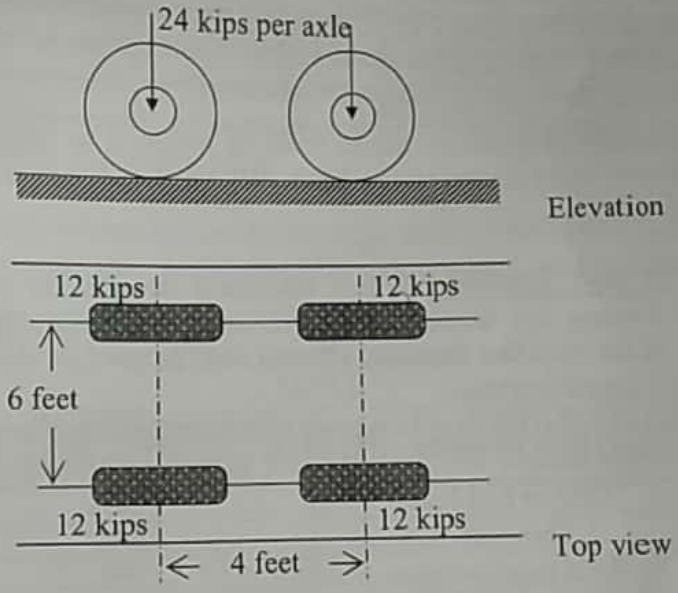


Figure 4: Special interstate highway loading

Bridge supporting interstate highways shall designed for HS 20-44 loading or alternate military loading of two axles 4 feet (1.219 m) apart with each axle weighing 24,000 lb (128kN), whichever produces the greater stress (see figure 4). This special loading will govern the design of simply supported structures up to spans of about 37 feet (11.28 m).

Minimum loading: For trunk highways, or for the other highways, or which may carry, heavy truck traffic, the minimum live load shall be the HS15-44 designated herein.

Traffic lanes: Where the spacing of main supporting members exceeds 6.5 feet for timber floors or 10.5 feet for concrete or steel grid floors, the lane loading or standard trucks shall be assumed to occupy a width of 10 feet. These loads shall be placed in design traffic lanes having a width of

$$W = \frac{W_c}{N}$$

Where, W_C = Roadway width between curbs exclusive of median strips.
N = Number of design traffic lanes as shown in the following table.
W = Width of design traffic lane (AASHTO standard lane width = 10 feet)

W_C (in feet)	N
$20<W_C\leq30$	2
$30<W_C\leq42$	3
$42<W_C\leq54$	4
$54<W_C\leq66$	5
$66<W_C\leq78$	6
$78<W_C\leq90$	7
$90<W_C\leq102$	8
$102<W_C\leq114$	9
$114<W_C\leq126$	10

The lane loadings or standard trucks shall be assumed to occupy any position within their individual design traffic lanes (W) which will produce the maximum stress.

Reduction of load intensity: Where maximum stresses are produced in any member by loading any number of traffic lanes simultaneously, the following percentages of the resultant live load stresses shall be used in view of improbable coincident maximum loading:

No. of lanes	Reduction of live load stresses (percent)
One or two lanes	100%
Three lanes	90%
Four lanes or more	75%

The reduction in intensity of floor beam loads shall be determined as in the case of main trusses or girders, using the width of roadway which must be loaded to produce maximum stresses in the floor beam.

Note that the combined design load capacity of all the beams in a bridge span shall not be less than that required to support the total live and dead loads of the span.

AASHTO Impact Loading (Dynamic effect of vehicles)

It is well-known that a vehicle moving across a bridge at a normal rate of speed produces greater stresses than a vehicle that remains in static position on the structure. This increment in stress can be called the dynamic effect. The terminology for the dynamic effect, among bridge designers and bridge design specifications, is impact.

An attempt to make an analytical study of the dynamic effect of a particular vehicle crossing over a specified bridge would lead to many complications because of the quantities that could be evaluated only approximately. Because it is difficult to make a confident analysis, the procedure has been assigning an approximate quantity for the dynamic effect. This quantity is determined by a simple equation given in the AASHTO specifications for determining the "impact factor" (IF) & "impact coefficient" (IC):

$IF \text{ or } IC = \left(\frac{50}{125+L}\right) \le 0.30$ Where, L is the loaded length in feet	$IF \text{ or } IC = \left(\frac{15.24}{38+L}\right) \le 0.30$ Where, L is the loaded length in meters

The stresses due to impact are calculated by multiplying the load stress by the value of IF/IC obtained from above equation. They are then added directly to the live load stresses to obtain the total stresses due to vehicle loads. The quantity L is defined as the loaded length of the structure, a value that is more specifically the length of structure that would be loaded to produce maximum stress if the live load consisted of uniformly distributed load.

The maximum value of the impact factor suggested by AASHTO is 0.30. This corresponds to a simple span 41.70 feet. All shorter spans use the value 0.30 for impact.

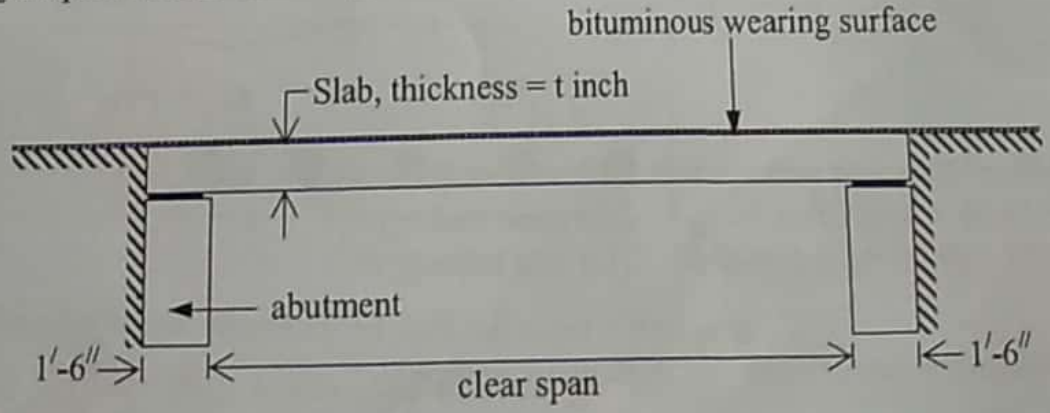
$$IF \text{ or } IC = \left(\frac{50}{125+L}\right) \le 0.30$$

Design of a Simply Supported Slab Bridge

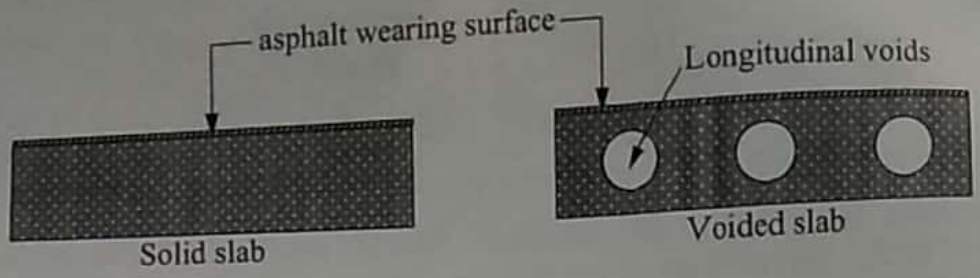
Design Data :

	Odd Roll No.	Even Roll No.
Clear Span	15 feet & onwards with 0.50 feet increments with roll number = 15′, 15.5′, 16′, 16.5′, ... & so on	
Clear Width	26 feet (2 lane)	24 feet (2 lane)
Wearing Surface	30 psf	30 psf
Live Loading	HS 20 (P_{20} = 16000 lb.)	HS 15 (P_{15} = 12000 lb.)
Concrete Compressive Strength (f'_c)	4000 psi	3000 psi
Reinforcing Steel	60 grade (f_y = 60 ksi)	40 grade (f_y = 40 ksi)

Design Method : AASHTO / ASD (American Association of State Highway & Transportation Officials – Allowable Stress Design)



Slab bridge (longitudinal section)



Design of Slab

(1) Effective span (S) = Clear span + bearing width of abutment = C/C distance between supports

(2) Effective span (S) = Clear span + slab thickness

Take minimum value of effective span (S) from the above two

Let, slab thickness, $t = 1.2\left(\frac{S_{feet}+10}{30}\right) =$ ..... inch (upper rounding to 1″ & $t \geq 8''$)

self weight of slab $= \frac{t}{12} \times 150$ psf = ... psf

wearing surface & protective coating = 25 psf

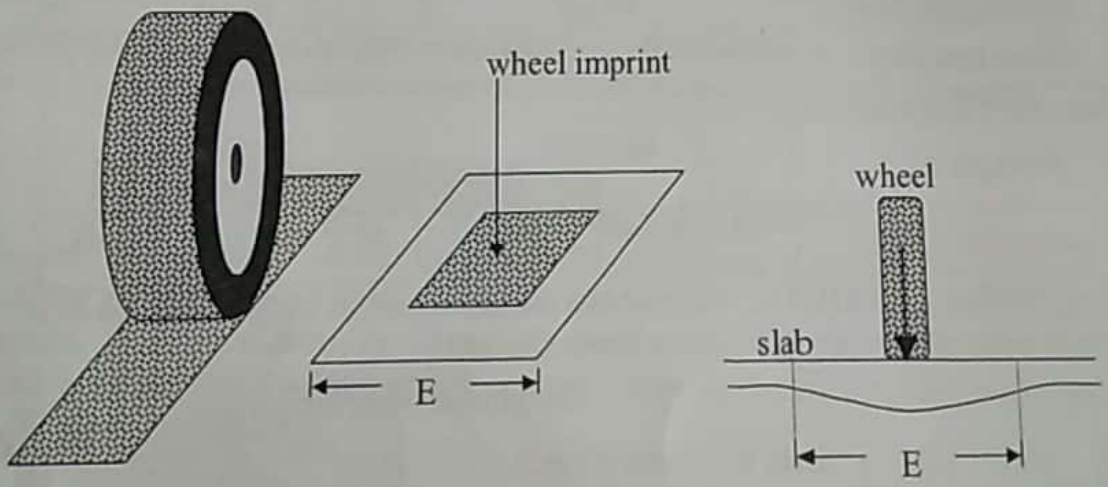
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total dead doad (w_{DL}) = ... psf

Dead load moment (DLM) $= +\frac{1}{8} w_{DL} S^2$ ft-lb/ft width of slab

Load on each rear wheel = P = P_{15} =12000 lb. (for HS-15 loading)

= P = P_{20} =16000 lb. (for HS-20 loading)



E = width of slab over which wheel load is laterally distributed, feet.

$E = 4 + (0.06 \times S) \leq 7$ feet [for truck loading]

$E = 2 \times [4 + (0.06 \times S)] \leq 14$ feet [for lane loading]

Load on a unit width of slab $= \frac{P}{E}$ lb./ft width of slab

Main reinforcement parallel to traffic: Live load moment (LLM)

Span* (simply supported), S	Live loading – HS 20	Live loading – HS 15
$S \le 50$ ft	$900 \times S$ ft-lb/ft	$\frac{3}{4} \times 900 \times S$ ft-lb/ft
$50 \text{ ft} \le S \le 100$ ft	$1000 (1.30 \times S - 20)$ ft-lb/ft	$\frac{3}{4} \times 1000 (1.30 \times S - 20)$ ft-lb/ft

not necessary

* for continuous span, use suitable method.

Impact factor (IF) $= \frac{50}{125 + S} \le 0.3$

Impact moment (IM) = Impact factor (IF) × Live load moment (LLM)

Design moment (M) = Dead load moment (DLM) + Live load moment (LLM) + Impact moment (IM)

= ..... ft-lb/ft width of slab

$f_c^{/}$ = ... psi & f_y ..... psi

$f_c = 0.40 \times f_c^{/}$ & $f_s = f_s = 0.40 \times f_y$

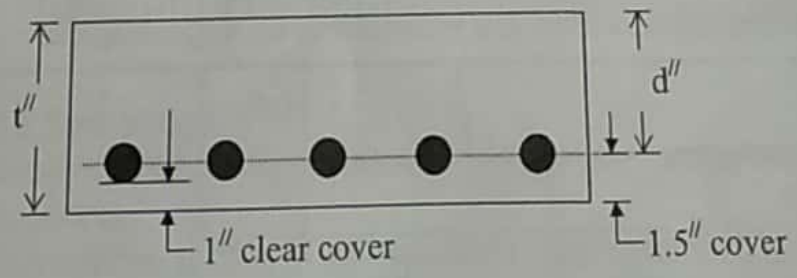
E_c (in psi) $= 57500\sqrt{f_c^{/}}$; where $f_c^{/}$ is in psi & $E_s = 29 \times 10^6$ psi

stress ratio, $r = \frac{f_s}{f_c}$; modular ratio, $n = \frac{E_s}{E_c}$ (should be nearest integer & ≥ 6)

$k = \frac{n}{(n + r)}$; $j = \left(1 - \frac{k}{3}\right)$; b = unit width of slab = 12 inch

$M = \frac{1}{2} f_c kjbd^2$

$d_{(\text{required})} = \sqrt{\frac{M}{\frac{1}{2} f_c kjb}}$



Considering #8 bar, diameter = 1 inch → radius = 0.50 inch

$d_{(\text{provided})} = [t - 0.5^{//} \text{ radius} - 1^{//} \text{ clear cover}] = [t - 1.5^{//}] =$ ..... inch

if, $d_{(provided)} > d_{(required)}$; thickness OK for moment

Dead load shear (DLV) = $\frac{1}{2} w_{DL} S$ lb/ft width of slab

Live load shear (LLV) = $\frac{P}{E}$ lb/ft width of slab

Impact shear (IV) = Impact factor (IF) × Live load shear (LLV)

Design shear (V) = Dead load shear (DLV) + Live load shear (LLV) + Impact shear (IV) = ... lb/ft width of slab

allowable shear (V_c) = $0.95\sqrt{f'_c}\, bd_{(provided)}$

if, $V > V_c$; thickness OK for shear

$+A_{s(main)} = \frac{+M}{f_s(jd)}$ = ... $inch^2$ per foot

maximum spacing for $A_{s(main)}$ = 3t or 18 inch [take minimum value]

$A_{s(distribution)} = \frac{100}{\sqrt{S}} \leq 50\%$ [for main reinforcement perpendicular to traffic]

maximum spacing for $A_{s(distribution)}$ = 5t or 18 inch [take minimum value]

$A_{s(temperature\ \&\ shrinkage)}$ = 1/8 $inch^2$ per foot

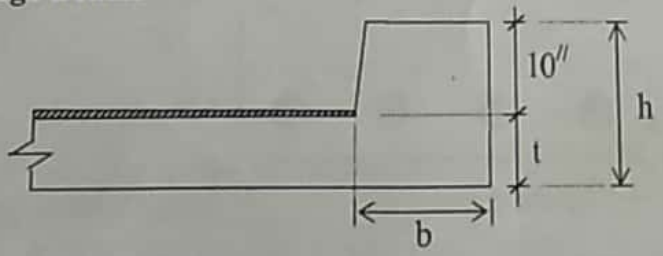
maximum spacing for $A_{s(temperature\ \&\ shrinkage)}$ = 5t or 18 inch

[take minimum value]

Check: $A_{s(main)} \geq A_{s(temperature\ \&\ shrinkage)}$ &

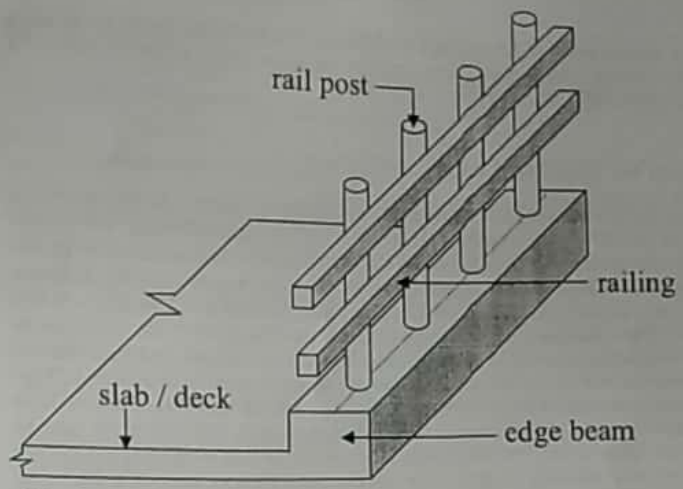
$A_{s(distribution)} \geq A_{s(temperature\ \&\ shrinkage)}$

Design of Edge Beams



b = width of slab acting as edge beam = 24 inch (say)

h = (10 + t) inch, t = slab thickness provided (″) & 10″ being the standard curb height



Design of Edge Beams

Dead load (w_{DL}) $= \frac{b}{12} \times \frac{h}{12} \times 150$ lb./ linear ft & railing weight = 15 lb./ linear ft

Dead load moment (DLM) $= \frac{1}{8}(w_{DL} + 15)S^2$ ft-lb. [S = span of slab (feet)]

Live load moment (LLM) = 0.10 P S ft-lb. [P = 12000 lb. for HS 15 live load]
[P = 16000 lb. for HS 20 live load]

Design moment (M) = Dead load moment (DLM) + Live load moment (LLM)

$M = \frac{1}{2} f_c kjbd^2$; $d_{(required)} = \sqrt{\frac{M}{\frac{1}{2} f_c kjb}}$

$d_{(provided)} = [t - (1.5'' \text{ cover})] = \ldots\ldots$ inch

if, $d_{(provided)} > d_{(required)}$; thickness OK for moment

$A_{s(main)} = \frac{M}{f_s(jd)} = \ldots\ldots$ inch^2; $A_{s(minimum)} = \frac{200}{f_y} bd = \ldots\ldots$ inch^2

$A_{s(main)} \geq A_{s(minimum)}$

Drawings to be Done on Tracing Paper

Design of a Slab Bridge.

(1) Cross-section & longitudinal profile of the bridge with position of supports.

(2) Transverse profile with slab/deck reinforcement details (plan & cross-sections)

(3) Edge beam reinforcement details

Use A3 size tracing paper & suitable scale (scale must be mentioned on drawing sheet).

they equal the width of the approach roadway section, including shoulders. Where a curbed roadway section approaches a bridge, the same section is carried across the bridge [AASHTO, 1992a, Article 2.3.1].

6.6 DESIGN OF SLAB BRIDGES

Slab bridges are characterized by simple or continuous concrete slabs spanning in the direction of traffic. They constitute the simplest type of highway bridges. Design of a reinforced concrete one-way slab to carry highway bridge loading is initiated by assuming a suitable thickness based on the minimum thickness requirements for deflection control, as stipulated in AASHTO 8.9.2 (see Chapter 5, Table 5.3). This gives the first estimate of the dead load due to the self-weight of the slab, to which the weight due to the wearing surface, say 25 to 35 lb/ft^2, is added to obtain the total dead load. The live load moments and shears are calculated according to the methods described in Chapter 4 and are multiplied by the factor $(1 + I)$ to account for the effect of impact. Note that, since the spans of the one-way slab bridges are usually under 35 ft, the value of the impact factor is 0.3. The slab is then designed for the sum of the dead and the live load moments (including impact). Note that the cost difference between a short span designed for HS15 and HS20 loading is generally less than 2 percent. The differences in reinforcing for continuous slab bridges designed for HS15 or HS20 loading, having 20-ft, 30-ft, and 40-ft end spans are only 0.3, 0.6, and 1.5 lb/ft^2 of the deck area, respectively. It is therefore recommended that all short-span concrete bridges be designed for HS20 loading [PCI, 1975]. Since the difference in the cost of Grade 40 and Grade 60 bars is not significant, it is also recommended that Grade 60 reinforcements be used for designing concrete bridges. Example 6.1 illustrates the design of a typical single-span reinforced concrete slab bridge. Both methods of design—the service load method and the load factor design—are illustrated.

EXAMPLE 6.1. Design a two-lane reinforced concrete slab bridge for a clear span of 20 ft to carry HS20 loading. A provision of 30 lb/ft^2 of dead load should be made in the design for the future wearing surface. The following data are given: f'_c = 3000 psi, Grade 40 reinforcement, clear width = 40 ft.

Calculations. The design of a slab bridge essentially consists of two parts: design of the slab and design of the longitudinal edge beams.

Design by the service load method: Design of slab. Assume 6 in. to the center of the bearings from the face of the abutments:

$$L = S = 20 + 2 \times (\tfrac{6}{12}) = 21 \text{ ft}$$

The minimum slab thickness is given by AASHTO 8.9.3 (Table 5.3 in this text) for deflection control:

$$t_{min} = 1.2\left(\frac{S + 10}{30}\right)$$

$$= 1.2\left(\frac{21 + 10}{30}\right)$$

$$= 1.24 \text{ ft}$$

Therefore, try a 15-in.-thick slab.

Self-weight of slab $= 1 \times 1.25 \times 150$	$= 187.5$ lb/ft^2
Wearing surface	$= 30.0$ lb/ft^2
Total load	$= 217.5$ lb/ft^2

Dead-load moment $M_D = wL^2/8 = 0.2175 \times (21)^2/8 = 11{,}990$ lb-ft

To calculate live-load moment, the weight of one rear wheel of the HS20 truck (16 kips) is distributed over a width, E, per AASHTO 3.24.3.2:

$$E = 4 + 0.06S = 4 + 0.06 \times 21 = 5.26 < 7.0 \text{ ft}$$

$$\text{The weight on a unit width of slab} = \frac{\text{weight of one rear wheel}}{E}$$

$$= \frac{16{,}000}{5.26}$$

$$= 3042 \text{ lb}$$

This 3042-lb load is treated as a concentrated load on a span of 21 ft, and the corresponding live-load moment is

$$M_{L,\text{truck}} = \frac{PS}{4} = 3042 \times \frac{21}{4} = 15{,}970 \text{ lb-ft}$$

Compare this with moment due to HS20 lane loading. The lane load (both the uniform and the concentrated) is distributed over a width of $2E$; thus,

$$w = \frac{640}{2 \times 5.26} = 60.8 \text{ psf}$$

The live-load moment due to this load is

$$M_w = \frac{wL^2}{8} = \frac{60.8 \times (21)^2}{8} = 3352 \text{ lb-ft}$$

The concentrated load is

$$P = \frac{18}{2 \times 5.26} = 1.711 \text{ kips}$$

The moment due to this load is given by

$$M_{\text{conc load}} = \frac{PL}{4} = \frac{1.711 \times 21}{4} = 8.98 \text{ kip-ft}$$

The total live-load moment due to lane loading is

$$M_L = 3.352 + 8.98 = 12.332 \text{ kip-ft} < 15.97 \text{ kip-ft (truck loading)}$$

Therefore, the truck loading governs. Use $M_L = 15.97$ kip-ft (the larger value):

$$\text{Impact: } I = \frac{50}{L + 125} = \frac{50}{21 + 125} = 0.342 > 0.3$$

Hence, use $I = 0.3$ (maximum value). Thus,

$$M_I = 0.3 \times 15.97 = 4.79 \text{ kip-ft}$$

$$M_{\text{total}} = 11.99 + 15.97 + 4.79 = 32.75 \text{ kip-ft}$$

For $f'_c = 3000$ psi, $n = 9$ (Table A.14, Appendix A). For the design of rectangular sections, $k = 0.351$ and $j = 0.883$ (Table A.16, Appendix A). Thus, from Eq. 6.4,

$$d = \sqrt{\frac{2M}{f_c kjb}} = \sqrt{\frac{2 \times 32.75 \times 12}{1.2 \times 0.351 \times 0.883 \times 12}} = 13.2 \text{ in.}$$

Assuming #8 for main reinforcing bar, and 1 in. as clear cover, the total required slab thickness is $h = 13.2 + 0.5 + 1 = 14.7$ in. Use $h = 15$ in., required for deflection control. Then, the effective depth is $d = 15 - 0.5 - 1.0 = 13.5$ in. From Eq. 6.5,

$$A_s = \frac{M}{f_s jd} = \frac{32.75 \times 12}{20 \times 0.883 \times 13.5} = 1.65 \text{ in.}^2$$

Therefore, provide #9 @ 7 in. o.c., $A_s = 1.71$ in.^{2} (Appendix A, Table A.19).

Distribution reinforcement (AASHTO 3.24.10.2)

Percentage of reinforcement $= 100/\sqrt{S} = 100/\sqrt{21} = 21.82\% < 50\%$ max.

Therefore, $A_s = 0.2182 \times 1.65 = 0.36$ in.^{2} Provide #6 @ 15 in. o.c., $A_s = 0.36$ in.^{2}, directly above and perpendicular to the main reinforcement.

Temperature reinforcement (AASHTO 8.20.1)

$$A_s = \frac{1}{8} \text{ in.}^2 \text{ per ft}$$

in each direction. Provide #4 @ 18 in. o.c. in each direction below the top face of the slab. All reinforcement details are shown in Fig. E6.1a.

Shear and bond (AASHTO 3.24.4). Slab designed for bending by the previous method is considered satisfactory in shear and bond; hence, no further checking is necessary.

Check for minimum reinforcement (AASHTO 8.17.1.1)

$$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{3000} = 410.8 \text{ psi}$$

$$I_{cr} = \frac{bh^3}{12} = \frac{12 \times 15^3}{12} = 3375 \text{ in.}^4$$

$$M_{cr} = \frac{f_r I_{cr}}{h/2} = \frac{410.8 \times 3375}{7.5 \times 12{,}000} = 15.41 \text{ kip-ft}$$

$$1.2M_{cr} = 1.2 \times 15.41 = 18.5 \text{ kip-ft}$$

$$M_{design} = 32.75 \text{ kip-ft} > 1.2M_{cr} = 18.5 \text{ kip-ft; OK.}$$

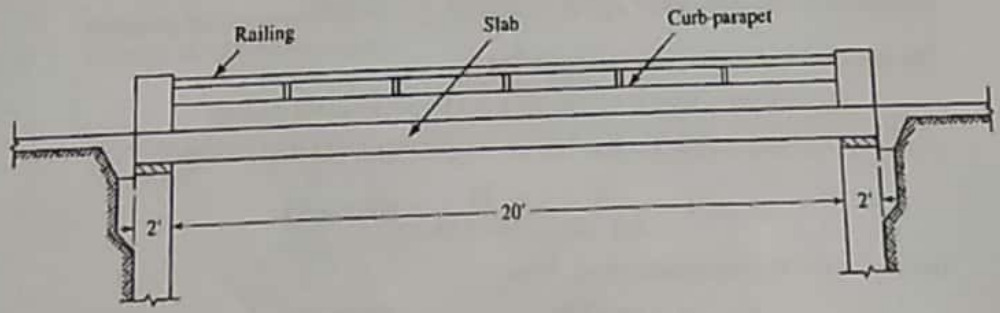


FIGURE E6.1a
A reinforced concrete slab bridge.

Fatigue stress limits (AASHTO 8.16.8.3). Fatigue stress limits are to be checked for the service load conditions, as provided by AASHTO Eq. 8.60.

$$f_f = 21 - 0.33 f_{min} + 8\left(\frac{r}{h}\right)$$

The minimum stress level, f_{min}, is caused by dead loads only. From service load design calculations, $M_D = 11.99$ kip-ft.

$$A_s = 1.71 \text{ in.}^2, \rho = \frac{A_s}{bd} = \frac{1.71}{12 \times 13.5} = 0.0106$$

From Table A.15 (Appendix A), for $n = 9$ and $\rho = 0.0106$ (by interpolation), $k = 0.351$ and $j = 0.883$. Therefore, the minimum stress, f_{min}, is

$$f_{min} = f_s = \frac{M}{A_s jd} = \frac{11.99 \times 12}{1.71 \times 0.883 \times 13.5} = 7.05 \text{ ksi}$$

The maximum stress is caused by the live load plus impact; the corresponding moment, including the dead-load moment, is $M_{total} = 32.75$ kip-ft. The corresponding stress in the main reinforcement is

$$f_s = \frac{M}{A_s jd} = \frac{32.75 \times 12}{1.71 \times 0.883 \times 13.5} = 19.28 \text{ ksi}$$

Thus, the actual stress range is $f_f = 19.28 - 7.05 = 12.23$ ksi. According to AASHTO Eq. 8.60,

$$f_f = 21 - 0.33 \times 7.05 + 8 \times 0.3 = 21.08 \text{ ksi} > 12.23 \text{ ksi}$$

Hence, the fatigue stress limit requirements are satisfied.

Design of longitudinal edge beams. AASHTO 3.24.8 stipulates that edge beams be provided for all slabs having primary reinforcement parallel to traffic. Three alternatives are suggested for providing edge beams:

1. A slab section additionally reinforced
2. A beam integral with and deeper than the slab
3. An integrally reinforced section of slab and curb

Although a safety curb would be provided for the bridge, it will be poured only after the slab has hardened (to facilitate screeding off the slab). As such, the curb is not considered an integral part of the longitudinal edge beam, which eliminates the second and the third alternatives. One would, therefore, choose the first alternative—an additionally reinforced section of the slab—as the longitudinal edge beam. Choose $b = 24$ in., so that the cross section of the longitudinal edge beam is 24 in. × 15 in. Provide a curb 10 in. high (standard depth), which gives the total height of the beam as 25 in. (for computing the self-weight).

Self-weight (including the curb) $= 24 \times 25 \times \frac{150}{144}$	= 625 lb/ft
Weight of railing (assumed)	= 15 lb/ft
Total dead load	= 640 lb/ft

$$M_D = 0.64 \times (21)^2/8 = 35.28 \text{ kip-ft}$$

The live-load moment in a longitudinal edge beam is given by AASHTO 3.24.8.2:

$$M_L = 0.10 P_{20} S = 0.10 \times 16{,}000 \times 21 = 33.6 \text{ kip-ft}$$

$$M_{total} = 35.28 + 33.6 = 68.88 \text{ kip-ft}$$

The required beam depth, with $b = 24$ in., can now be calculated from Eq. 6.4. With $k = 0.351$ and $j = 0.883$ for $n = 9$ (Table A.16, Appendix A),

$$d = \sqrt{\frac{2M}{f_c k j b}} = \sqrt{\frac{2 \times 68.05 \times 12}{1.2 \times 0.351 \times 0.883 \times 24}} = 13.53 \text{ in.}$$

With 1 in. cover and #8 as the main reinforcing bar, $h_{required} = 13.53 + 1 + 0.5 = 15.03$ in., say 15 in., as provided. The required reinforcement can now be calculated from Eq. 6.5:

$$A_s = \frac{M}{f_s j d} = \frac{68.88 \times 12}{20.0 \times 0.883 \times 13.5} = 3.47 \text{ in.}^2$$

Therefore, provide four #9 bars, $A_s = 4.0 \text{ in.}^2 > 3.47 \text{ in.}^2$ (Table A.20, Appendix A).

Check for minimum reinforcement (AASHTO 8.17.1.1). From Eq. 6.30:

$$f_r = 7.5\sqrt{f_c'} = 7.5\sqrt{3000} = 410.8 \text{ psi}$$

$$I_{cr} = \frac{24 \times 15^3}{12} = 6750 \text{ in.}^4$$

Then, from Eq. 6.31:

$$M_{cr} = \frac{f_r I_{cr}}{h/2} = \frac{410.8 \times 6750}{7.5 \times 12{,}000} = 30.81 \text{ kip-ft}$$

$$1.2M_{cr} = 1.2 \times 30.81 = 36.97 \text{ kip-ft}$$

$$M_{design} = 68.88 \text{ kip-ft} > 1.2M_{cr} = 36.97 \text{ kip-ft; OK.}$$

Fatigue stress limits (AASHTO 8.16.8.3). Fatigue stress limits will be checked for the service load conditions. The permissible stress range is given by AASHTO Eq. 8.60:

$$f_f = 21 - 0.33 f_{min} + 8\left(\frac{r}{h}\right)$$

The minimum stress level, f_{min}, is caused by dead loads only. From service load design calculations, $M_D = 35.28$ kip-ft.

$$A_s = 4.00 \text{ in.}^2, \rho = A_s/bd = 4.00/(24 \times 13.5) = 0.0123$$

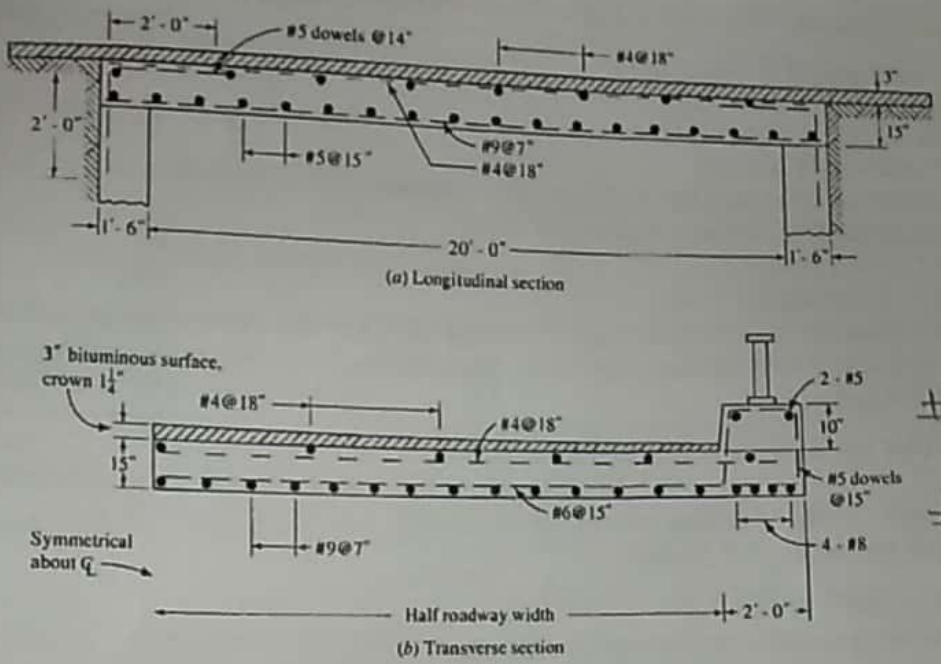
From Table A.15 (Appendix A), for $n = 9$ and $\rho = 0.0123$ (by interpolation), $k = 0.373$, $j = 0.875$. The minimum stress, therefore, is (from Eq. 6.5)

$$f_{min} = f_s = \frac{M}{A_s j d} = \frac{35.28 \times 12}{4.0 \times 0.875 \times 13.5} = 8.96 \text{ ksi}$$

The maximum stress is caused by the live load plus impact; the corresponding moment, including the dead-load moment, is $M_{total} = 68.88$ kip-ft. The corresponding stress in the main reinforcement is

$$f_s = \frac{M}{A_s j d} = \frac{68.88 \times 12}{4.0 \times 0.881 \times 13.5} = 17.49 \text{ ksi}$$

Thus, the actual stress range, $f_f = 17.49 - 8.96 = 8.53$ ksi. According to AASHTO Eq. 8.60, the permissible stress range is $f_f = 21 - 0.33 \times 7.84 + 8 \times 0.3 = 20.81 \text{ ksi} > 8.53$ ksi. Hence, the fatigue stress limit requirements are satisfied.



#9@7
#6@15
#4@18

FIGURE E6.1b
Reinforcement details (slab and the edge beams).

Reinforcement details are shown in Fig. E6.1b. #5 dowels @ 18 in. o.c., along with two #5 hanger bars, are provided for the entire lengths of longitudinal edge beams.

Miscellaneous details. For effective drainage, the roadway surface is crowned by providing varying thicknesses of the wearing surface across the deck cross section. According to AASHTO 8.5.1, devices for expansion and contraction due to temperature changes are to be provided at supports only for spans exceeding 40 ft. Thus, bearing pads are not required to be provided for slab bridges. Instead, the slab rests directly on the abutments—on the bridge seat precoated with a bituminous mixture—and fixed bearings are provided by placing vertical dowels usually @ 12 in. o.c. along the length of the abutments. As shown in Fig. E6.1b, these dowels extend vertically 24 in. into the abutment and bend to extend 24 in. horizontally inside the slab. This detail also acts as a preventive measure against cracking that may occur near the abutment edges.

Design by the load factor method: Design of slab. The following service load moments were calculated earlier:

$$M_D = 11.99 \text{ kip-ft}$$

$$M_L = 15.97 \text{ kip-ft}$$

$$M_I = 4.79 \text{ kip-ft}$$

= Design Example: Balanced Cantilever Bridge Design =

Design of a Balanced Cantilever bridge (Three Span) from the following data

Design Data

Type: Deck Girder / T-Beam Bridge
Material: Reinforced Concrete
Total Span (C/C distance between bearings), L = 204 feet
Number of Longitudinal Girders: 4 (four)
Number of Lanes: 2 (two)
Lane Width: 12 feet (each)
Deign Code / Method: AASHTO, (2012). AASHTO LRFD Bridge Design Specifications, Customary U.S. Units (6th Edition), American Association of State Highway and Transportation Officials, Washington, D.C.

Assume, Web width =

Width of Bridge (total): 2-Lane Bridge with sidewalk on each side.
(=3.50′ sidewalk +12′0″ lane+12′0″ lane+3.50′ sidewalk = 31 feet)
Roadway Width / Width of Bridge (clear): 24 feet (=12′0″ lane+12′0″ lane = 24 feet)
C/C spacing of girders, S_g = 7.50 feet
C/C spacing of girders, Sg = (24 - 9/12 - 9/12),
Clear spacing of girders, S_c = 6 feet
Clear spacing, Sc = 7.5 - 9/12 -9/12 = 6'
H = girder depth (including slab thickness) = 45 inch
b_w = girder web width = 18 inch
t_s = deck slab thickness = 7.50 inch
L_E = End Span / Anchor Span / Side Span / Back Span = 60 feet
L_M = Middle Span / Central Span = 84 feet
L_S = Suspended Span / Supported Span = 52.50 feet
L_C = Cantilever Span / Overhanging Span = 15.75 feet

L= LM + 2LE = 1.4LE + 2
=>L = 3.4 LE
=>204 = 3.4LE
=> LE = 60'

LM = 1.4LE = 1.4*60 = 8

LM= LS + 2LC = LS + 2*0.3LS
=>LM = 1.6 LS
=>84 = 1.6 LS
=> LS = 52.5'

LC = 0.3LS = 0.3*52.5 = 15.75'

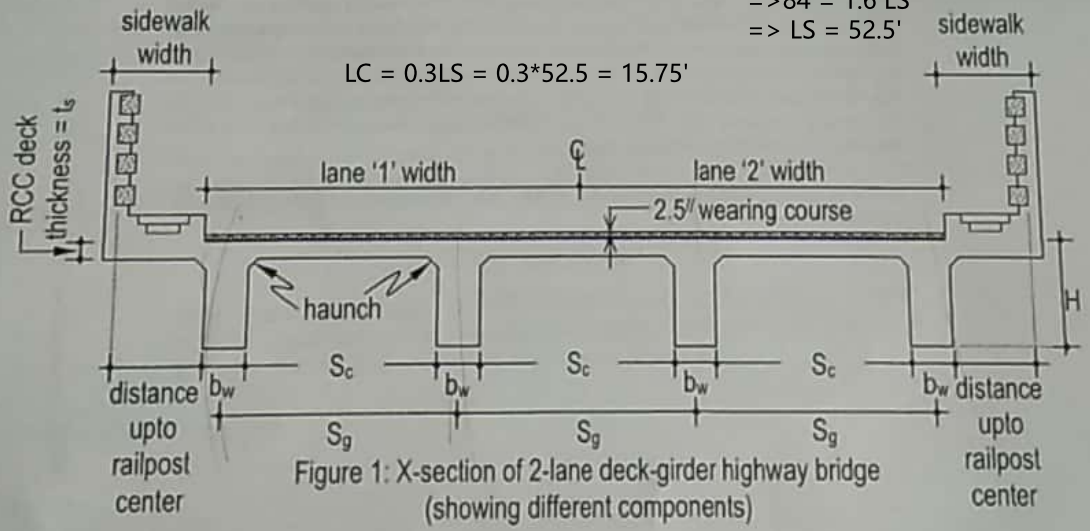


Figure 1: X-section of 2-lane deck-girder highway bridge (showing different components)

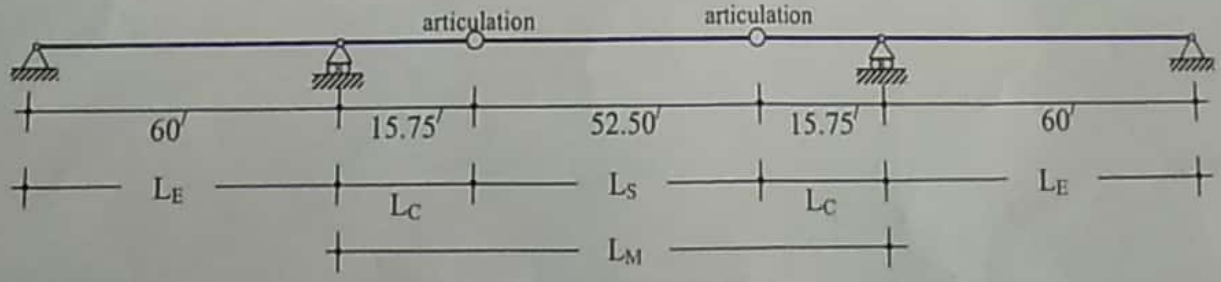


Figure 2: L-section of 2-lane deck-girder highway bridge (balanced cantilever type)

$L = L_E + L_M + L_E = 2L_E + L_M$

$L_M = L_C + L_S + L_C = 2L_C + L_S$

$H \geq 0.70L_S$ (= 0.07×52.50 feet = 0.07×52.50×12 inch = 44.10 inch)

H = 45 inch (upper rounding to 1 inch)

$b_w \geq \frac{H}{4}$ to $\frac{H}{3}$ (must be at least 18 inch wide)

$b_w \geq \frac{45\text{ inch}}{4}$ to $\frac{45\text{ inch}}{3} \geq 11.25$ inch to 15 inch

b_w = 18 inch (minimum width controls) (upper rounding to 1 inch)

$t_s \geq \frac{S_e}{12}$ (must be at least 7 inch thick)

$t_s \geq \frac{7.5\text{ feet}}{12} \geq \frac{7.5\times12\text{ inch}}{12} \geq 7.50$ inch

t_s = 7.50 inch (upper rounding to 0.50 inch)

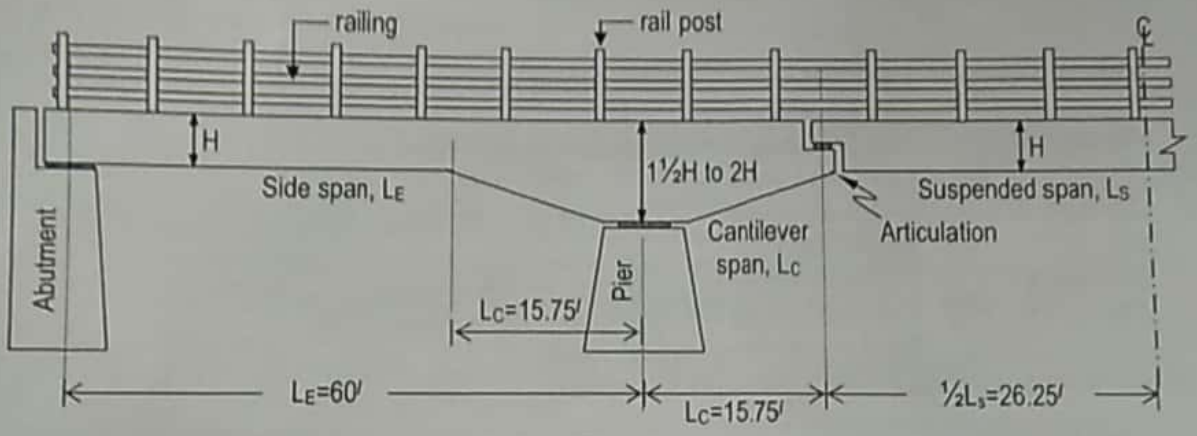


Figure 3: Longitudinal Profile of Three Span balanced cantilever bridge

Future wearing (2.50″ thickness on average) = 30 psf

Deck slab concrete, (f_c') = 3500 psi = 3.50 ksi

Girder concrete, (f_c') = 3500 psi = 3.50 ksi

Steel: Grade 40 (mild-steel, yield point, f_{yield} = 40 ksi, E = 29000 ksi)

Design Code / Design Method: AASHTO 2012 / LRFD

[AASHTO: American Association of State Highway & Transportation Officials

LRFD: Load & Resistance Factor Design]

Live Load: AASHTO HL-93

= Application of Bridge Live Loads =

- Standard Lane Width = 12 feet (Load occupies 10 feet width across Lane)
- Fractional Lanes are not permitted.
- Lane Load may be reduced as follows

Number of Lanes	Lane Load Reduction (%)
1 and 2 Lane Bridge	No reduction
2 Lane Bridge	90%
4 or more Lanes	75%

► Service Limit States: Concrete stresses, deformations, cracking, distribution of reinforcement, deflection, and camber are investigated at service limit states.

Service I: Crack control and limiting compression in pre-stressed concrete
Service III: Crack control/tension in pre-stressed concrete
Service IV: Post-tensioned precast column sections

Service I :	1.00 Self-weight + 1.00 Barrier + 1.00 Wearing + 1.00 Live Load + 0.3 Wind Load + 1.0 Wind on Live Load
Service II :	1.00 Self-weight + 1.00 Barrier + 1.00 Wearing + 1.30 Live Load
Service III :	1.00 Self-weight + 1.00 Barrier +1.00 Wearing + 0.80 Live Load
Service IV :	1.00 Self-weight + 1.00 Barrier +1.00 Wearing + 0.70 Wind Load

[*barrier means railing, rail post, sidewalk & medians]

► Ultimate/Strength Limit States: Axial, flexural, shear strength, and stability of concrete components are investigated at strength limit states. Resistance factors are based on AASHTO Article 5.5.4.2 (AASHTO, 2012).

Strength I: Basic load (HL-93)
Strength II: Owner specified load (Permit)
Strength III: Wind on structure
Strength IV: Structure with high DL/LL (>7)
Strength V: Wind on structure and live load

Strength I :	1.25 Self-weight + 1.50 Barrier +1.50 Wearing + 1.75 Live Load
Strength II :	1.25 Self-weight + 1.50 Barrier +1.50 Wearing + 1.35 Live Load
Strength III :	1.25 Self-weight + 1.50 Barrier +1.5 Wearing + 1.40 Wind Load
Strength IV :	1.50 Self-weight + 1.50 Barrier +1.5 Wearing
Strength V :	1.25 Self-weight + 1.50 Barrier +1.5 Wearing + 0.40 Wind + 1.00 Wind on Live Load

[*barrier means railing, rail post, sidewalk & medians]

AASHTO HL-93 Loading Highway Design*: This is a hypothetical Live Load Model proposed by AASHTO for analysis of bridges. Reason for proposing this live load model is to prescribe a set of loads such that it produces extreme load effect approximately same as that produced by the exclusion vehicles. (Article 3.6.1.2.1)

Exclusion vehicles were the vehicles above the legal limit but due to grand fathering provision in the state they were allowed to operate routinely.

It has three (3) basic Live Loads for bridges called HL-93 Loading (where H stands for highway and L stands for Loading, developed in 1993).

1. Design Truck 2. Design Tandem 3. Design Lane

1. Design Truck: It is commonly called as HS-20 44 (where H stands for highway, S for semi-trailer, 20 Ton weight of the tractor (1st two axles) and was proposed in 1994).

2. Design Tandem: It consists of two axles weighing 25 kips (110 kN) each spaced at 4 feet (1.2 meter).

3. Design Lane: It consists of uniformly distributed load of 0.64 kip/ft (9.33 N/mm) and assumed to occupy 10 feet (3 meter) transversely. (64 psf on bridge floors)

HL-93 is the maximum of the two

(i) Design Truck + Design Lane

or (ii) Design Tandem + Design Lane

[*taken directly from: http://www.aboutcivil.com/]

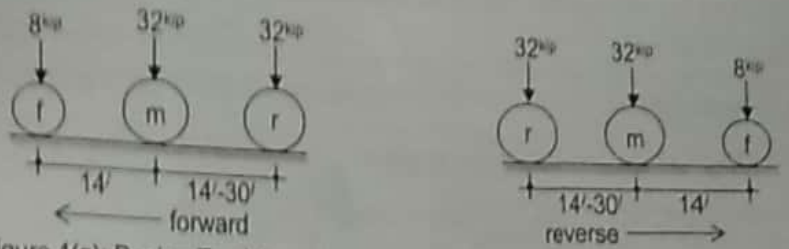


Figure 4(a): Design Truck Load (f = front wheel, m = middle wheel, r = rear wheel)

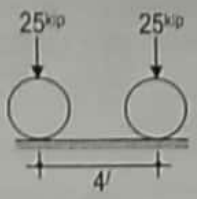


Figure 4(b): Design Tandem Load

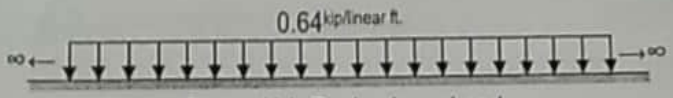


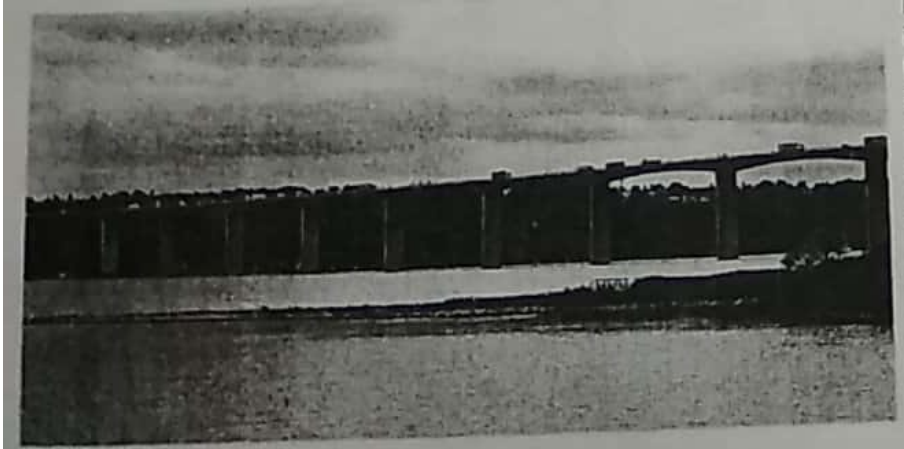
Figure 4(c): Design Lane Load

The dynamic load allowance shall not be applied to pedestrian loads or to the design lane load. The static effects of the design truck or tandem, other than centrifugal and braking forces, shall be increased by the percentage specified in the following table for dynamic load allowance.

Component	Dynamic Load allowance (%)
Deck Joint - all Limit States	45%
all other Components	
Fatigue and Fracture Limit State	15%
all other Limit States	33%

Strength Reduction Factor (φ)

- φ = 0.70 (for bearing)
- φ = 0.75 (for column/compression member)
- φ = 0.85 (for beam/flexure)
- φ = 0.90 (for shear (flexural-shear & shear-friction)
- φ = 0.90 (for axial tension)



Vejle Fjord Highway Bridge, Denmark, (Multiple span balanced cantilever, total length: 1,712 m (5,617 ft), width: 27.6 m (90.6 ft), Number of Lanes: 6 (six), Daily Tracffic: Up to 65,000 road vehicles)

Vejle Fjord Bridge (Danish: Vejlefjordbroen) is a cantilever bridge that spans Vejle Fjord between Mølholm and Nørremarken near the town of Vejle in Denmark. The bridge is 1712 metres long, the longest span is 110 metres, and the maximum clearance to the sea is 40 metres. The bridge has 15 spans. Vejle Fjord Bridge was built to lead traffic past the town of Vejle, where traffic congestion had become a problem in the 1970s. The building started in 1975, and the bridge was opened on July 1, 1980. It cost 350 million kroner. The bridge now has more traffic than any other highway bridge in Denmark.

https://en.wikipedia.org/wiki/Vejle_Fjord_Bridge

Design of Non-Pre-Stressed Cast-in-Situ RCC Deck

slab thickness, t_{slab} = 7.50 inch

slab self-weight, $w_{slab} = \frac{t_{slab}}{12}\text{ feet} \times 150\text{ pcf} = \frac{7.50}{12}\text{ feet} \times 150\text{ pcf} = 93.75$ psf/ft width of slab

wearing surface thickness, $t_{wearing}$ = 2.50 inch (average)

wearing surface self-weight, $w_{wearing} = \frac{t_{wearing}}{12}\text{ feet} \times 140\text{ pcf}$

$$= \frac{2.50}{12}\text{ feet} \times 140\text{ pcf} = 30\text{ psf/ft width of slab}$$

[reinforced concrete unit weight = 150 pcf & asphaltic concrete unit weight = 140 pcf]

span of slab (for dead load moment computation) = center-to-center distance of girder, S_g = 7.50 feet

dead load moment (due to slab self-weight), $\pm M_{DL\,(slab)} = \pm\frac{w_{slab}S_g^2}{10} = \pm\frac{93.75\text{ psf} \times (7.50\text{ feet})^2}{10}$

$= \pm 527.34375$ lb.-ft/ft width of slab $= \pm 0.52734375$ kip-ft/ft width of slab

dead load moment (due to wearing surface self-weight), $\pm M_{DL\,(wearing)} = \pm\frac{w_{wearing}S_g^2}{10}$

$$= \pm\frac{30\text{ psf} \times (7.50\text{ feet})^2}{10} = \pm 168.75\text{ lb.-ft/ft width of slab} = \pm 0.16875\text{ kip-ft/ft width of slab}$$

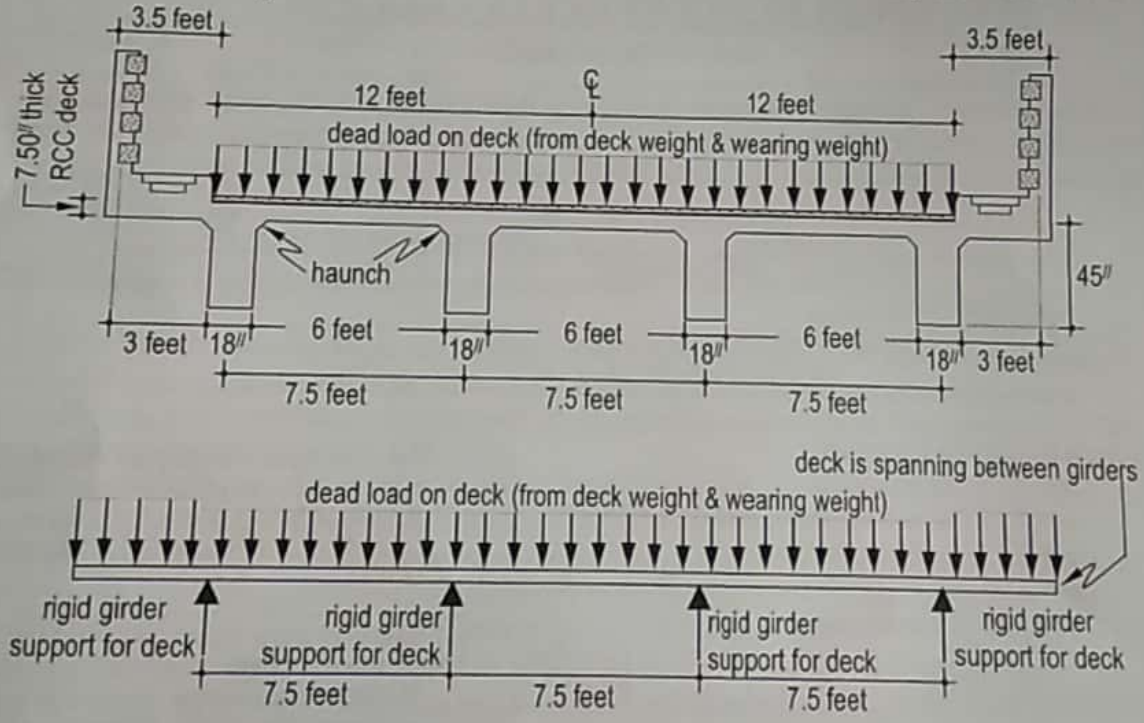


Figure 5: Dead load on deck slab

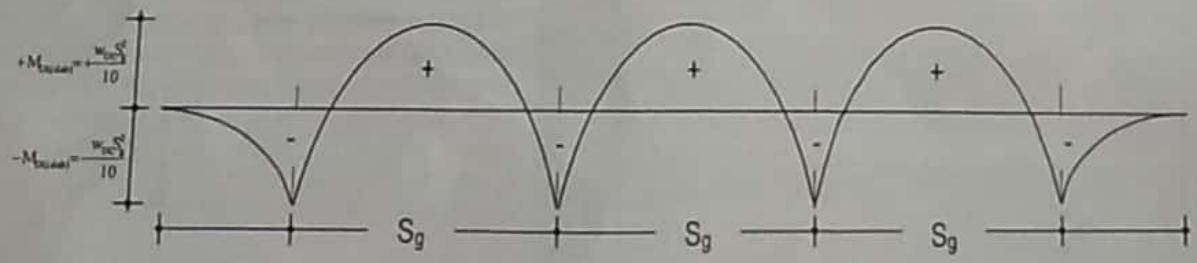


Figure 6: Dead load Moment Diagram for deck slab

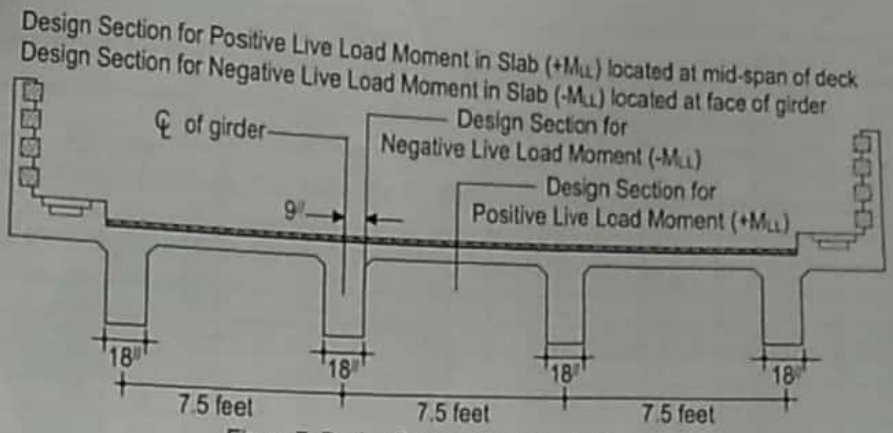


Figure 7: Design Sections for $+M_{LL}$ & $-M_{LL}$

center-to-center distance of girder, $S_g = 7.50$ feet

positive live load moment will occur at mid-span section (i.e. at mid-way of slab panel)

positive live load moment, $+M_{LL} = +5.44$ kip-ft/ft width of slab (from table 1; linear interpolation may be needed if your S_g value not tabulated directly)

negative live load moment, $-M_{LL} = -3.78$ kip-ft/ft width of slab (from table 1; linear interpolation may be needed if your S_g & $\frac{1}{2}b_w$ values not tabulated directly)

section for negative live load moment located at face of girder from centerline of support (i.e. from centerline of girder) $\Rightarrow\Rightarrow \frac{1}{2}b_w = \frac{1}{2} \times 18$ inch $= 9$ inch (from centerline of girder)

S_g	$+M_{LL}$ (kip-ft/ft width of slab)	$-M_{LL}$ (kip-ft/ft width of slab)						
		distance from centerline of girder to design section for negative moment**						
		0"	3"	6"	9"	12"	18"	24"
7'-6"	5.44	6.26	5.43	4.61	3.78	3.15	1.88	1.72

Table 1: Maximum Live Load Moments per Unit Width (kip-ft/ft)

c/c spacing of girder	Positive Moment	Negative Moment						
		distance from centerline of girder to design section for negative moment						
		0"	3"	6"	9"	12"	18"	24"
4'-0"	4.68	2.68	2.07	1.74	1.60	1.50	1.34	1.25
4'-3"	4.66	2.73	2.25	1.95	1.74	1.57	1.33	1.20
4'-6"	4.63	3.00	2.58	2.19	1.90	1.65	1.32	1.18
4'-9"	4.64	3.38	2.90	2.43	2.07	1.74	1.29	1.20
5'-0"	4.65	3.74	3.20	2.66	2.24	1.83	1.26	1.12
5'-3"	4.67	4.06	3.47	2.89	2.41	1.95	1.28	0.98
5'-6"	4.71	4.36	3.73	3.11	2.58	2.07	1.30	0.99
5'-9"	4.77	4.63	3.97	3.31	2.73	2.19	1.32	1.02
6'-0"	4.83	4.88	4.19	3.50	2.88	2.31	1.39	1.07
6'-3"	4.91	5.10	4.39	3.68	3.02	2.42	1.45	1.13
6'-6"	5.00	5.31	4.57	3.84	3.15	2.53	1.50	1.20
6'-9"	5.10	5.50	4.74	3.99	3.27	2.64	1.58	1.28
7'-0"	5.21	5.98	5.17	4.36	3.56	2.84	1.63	1.37
7'-3"	5.32	6.13	5.31	4.49	3.68	2.96	1.65	1.51
7'-6"	5.44	6.26	5.43	4.61	3.78	3.15	1.88	1.72

c/c spacing of girder	Positive Moment	Negative Moment						
		distance from centerline of girder to design section for negative moment						
		0"	3"	6"	9"	12"	18"	24"
7'-9"	5.56	6.38	5.54	4.71	3.88	3.30	2.21	1.94
8'-0"	5.69	6.48	5.65	4.81	3.98	3.43	2.49	2.16
8'-3"	5.83	6.58	5.74	4.90	4.06	3.53	2.74	2.37
8'-6"	5.99	6.66	5.82	4.98	4.14	3.61	2.96	2.58
8'-9"	6.14	6.74	5.90	5.06	4.22	3.67	3.15	2.79
9'-0"	6.29	6.81	5.97	5.13	4.28	3.71	3.31	3.00
9'-3"	6.44	6.87	6.03	5.19	4.40	3.82	3.47	3.20
9'-6"	6.59	7.15	6.31	5.46	4.66	4.04	3.68	3.39
9'-9"	6.74	7.51	6.65	5.80	4.94	4.21	3.89	3.58
10'-0"	6.89	7.85	6.99	6.13	5.26	4.41	4.09	3.77
10'-3"	7.03	8.19	7.32	6.45	5.58	4.71	4.29	3.96
10'-6"	7.17	8.52	7.64	6.77	5.89	5.02	4.48	4.15
10'-9"	7.32	8.83	7.95	7.08	6.20	5.32	4.68	4.34
11'-0"	7.46	9.14	8.26	7.38	6.50	5.62	4.86	4.52
11'-3"	7.60	9.44	8.55	7.67	6.79	5.91	5.04	4.70
11'-6"	7.74	9.72	8.84	7.96	7.07	6.19	5.22	4.87
11'-9"	7.88	10.01	9.12	8.24	7.36	6.47	5.40	5.05
12'-0"	8.01	10.28	9.40	8.51	7.63	6.74	5.56	5.21
12'-3"	8.15	10.55	9.67	8.78	7.90	7.02	5.75	5.38
12'-6"	8.28	10.81	9.93	9.04	8.16	7.28	5.97	5.54
12'-9"	8.41	11.06	10.18	9.30	8.42	7.54	6.18	5.70
13'-0"	8.54	11.31	10.43	9.55	8.67	7.79	6.38	5.86
13'-3"	8.66	11.55	10.67	9.80	8.92	8.04	6.59	6.01
13'-6"	8.78	11.79	10.91	10.03	9.16	8.28	6.79	6.16
13'-9"	8.90	12.02	11.14	10.27	9.40	8.52	6.99	6.30
14'-0"	9.02	12.24	11.37	10.50	9.63	8.76	7.18	6.45
14'-3"	9.14	12.46	11.59	10.72	9.85	8.99	7.38	6.58
14'-6"	9.25	12.67	11.81	10.94	10.08	9.21	7.57	6.72
14'-9"	9.36	12.88	12.02	11.16	10.30	9.44	7.76	6.86
15'-0"	9.47	13.09	12.23	11.37	10.51	9.65	7.94	7.02

Limit State	Moment	Value (kip-ft./ft. width)
Service I	positive	$+M_{service\ I} = 1.00(0.52734375)+1.00(0.16875)+1.00(5.44)$ $= +6.13609375$
Strength I	positive	$+M_{strength\ I} = 1.25(0.52734375)+1.50(0.16875)+1.75(5.44)$ $= +10.43230469$
Service I	negative	$-M_{service\ I} = 1.00(-0.52734375)+1.00(-0.16875)+1.00(-3.78)$ $= -4.47609375$
Strength I	negative	$-M_{strength\ I} = 1.25(-0.52734375)+1.25(-0.16875)+1.75(-3.78)$ $= -7.527304688$

temperature & shrinkage steel, $A_{s\,(temperature\ \&\ shrinkage)} = \dfrac{1.3bh}{2(b+h)f_y}$ inch^2/ft width of slab

where, b = width of slab = width of bridge = 31 feet = 31×12 inch

h = overall thickness or depth of the component (inch) = thickness of slab = t_{slab} = 7.50 inch

$$A_{s\,(temperature\ \&\ shrinkage)} = \frac{1.3\times(31\times12\ inch)\times7.50\ inch}{2\times(31\times12\ inch+7.50\ inch)\times40\ ksi} = 0.119466403\ inch^2/ft\ width\ of\ slab$$

limit: 0.11 $inch^2$/ft width of slab $\le A_{s\,(temperature\ \&\ shrinkage)} \le$ 0.60 $inch^2$/ft width of slab; OK

so, $A_{s\,(temperature\ \&\ shrinkage)} = 0.119466403\ inch^2/ft$ width of slab

This temperature & shrinkage steel indicates the minimum steel needs to be provided in deck slab.

---

design for positive moment (positive main steel, $+A_{s\,(main)}$)

1st trial: assume, a = 1 inch

$$+A_{s\,(main)} = \frac{+M_{strength\ I}}{\varphi f_y\left(d-\frac{a}{2}\right)} = \frac{(+)10.43230\times12\ kip-ft}{0.90\times40\ ksi\times\left(6.1875^{"}-\frac{1^{"}}{2}\right)} = 0.611417\ inch^2/ft\ width$$

$$\text{check, } a = \frac{+A_{s\,(main)}f_y}{0.85f_c'b} = \frac{(+)0.611417\ inch^2/ft\times40\ ksi}{0.85\times3.5\ ksi\times12\ inch} = 0.68506\ inch \neq 1\ inch;\ not\ OK$$

2nd trial: assume, a = 0.665 inch

$$+A_{s\,(main)} = \frac{+M_{strength\ I}}{\varphi f_y\left(d-\frac{a}{2}\right)} = \frac{(+)10.43230\times12\ kip-ft}{0.90\times40\ ksi\times\left(6.1875^{"}-\frac{0.665^{"}}{2}\right)} = 0.593925\ inch^2/ft\ width$$

$$\text{check, } a = \frac{+A_{s\,(main)}f_y}{0.85f_c'b} = \frac{(+)0.593925\ inch^2/ft\times40\ ksi}{0.85\times3.5\ ksi\times12\ inch} = 0.66546\ inch \approx 0.665\ inch;\ OK$$

check, $+A_{s(main)}$ [= 0.593925 $inch^2$/ft width] > $A_{s\,(temperature\ \&\ shrinkage)}$ [=0.119466403 $inch^2$/ft width]; OK

$$\text{spacing (\#5 bar), } s = \frac{12\times A_{s(one\,bar,\,inch^2)}}{A_{s(required,\,inch^2/ft)}} = \frac{12\times0.31\ inch^2}{0.593925\ inch^2/ft} = 6.26336\ inch \approx 6\ inch\ c/c$$ [lower rounding to 0.50 inch]

check maximum spacing of main bar ($+s_{maximum}$)

[i.e., check cracking of concrete at service load ($+M_{service\,I}$)]

$$s_{maximum} \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$

where, γ_e = exposure factor = 1.00 (for Class 1 exposure condition)

= 0.75 (for Class 2 exposure condition)

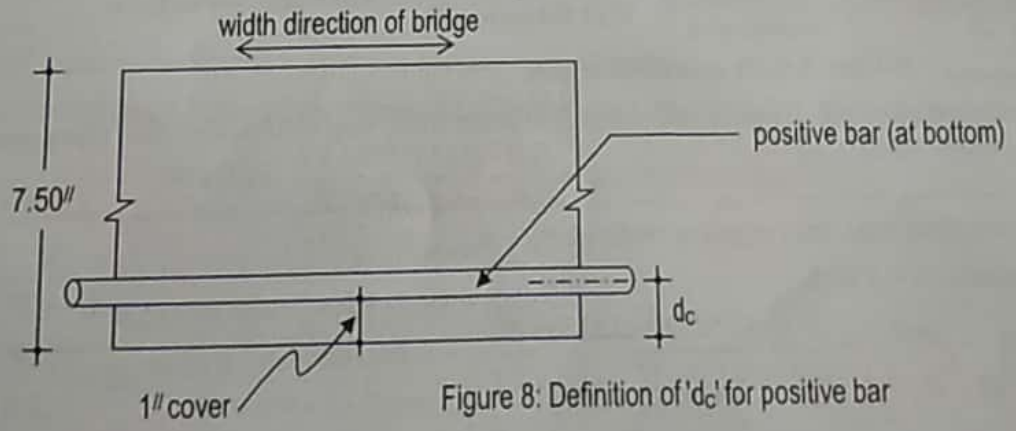


Figure 8: Definition of 'd_c' for positive bar

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (inch) [see figure 8]

if # 5 bar is chosen (bar diameter = 5/8 inch)

d_c (for positive bar) = clear cover + ½ of bar diameter = 1 inch + ½ of (5/8) inch = 1.3125 inch

[limit of d_c : must be ≤ (2 inch + ½ of bar diameter) ≤ 2.3125 inch; OK]

f_{ss} = tensile stress in steel reinforcement at service limit state (ksi)

h = overall thickness or depth of the component (inch) = thickness of slab = t_{slab} = 7.50 inch

$$f_{ss} = f_y \times \frac{+M_{service\ I}}{+M_{strength\ I}} = 40\ \text{ksi} \times \frac{+6.13609375\ \text{kip-ft/ft}}{+10.43230469\ \text{kip-ft/ft}} = 23.52727967\ \text{ksi}$$

$$\beta_{ss} = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{1.3125^{"}}{0.7(7.50^{"} - 1.3125^{"})} = 1.303030303$$

$$s_{maximum} \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \le \left[\frac{700 \times 1.00}{1.303030 \times 23.52727\ \text{ksi}} - 2 \times 1.3125^{"}\right] \le 20.20846\ \text{inch} \le 20\ \text{inch}$$

[lower rounding to 0.50 inch]

actual spacing of steel (s) shall not be more than $s_{maximum}$ calculated above. [$s \le s_{maximum}$]

as in this case actual spacing (s = 6 inch c/c) of positive steel ($+A_{s(main)}$) does not exceed maximum permitted spacing ($s_{maximum}$ = 20 inch c/c) =>> provide 6 inch c/c spacing

Use **# 5 bar @ 6 inch center-to-center** (for positive moment). This $+A_{s(main)}$ will be provided at bottom along transverse direction (width direction of bridge). Clear cover for $+A_{s(main)}$ will be at least = 1.00 inch (measured from bottom of bar to concrete bottom surface) [see figure 8 & 10]

$$\text{distribution steel (\% of } +A_{s(main)}) = \frac{220}{\sqrt{S_{c(feet)}}}\% \le 67\% \text{ (for main steel perpendicular to traffic)}$$

S_c = the effective span length (feet) taken as equal to the effective length specified in Article 9.7.2.3; the distance between sections for negative moment and sections at the ends of one deck

$$\text{span} = S_g - \frac{1}{2}b_w - \frac{1}{2}b_w = 7.50\ \text{feet} - \frac{18\ \text{inch}}{2} - \frac{18\ \text{inch}}{2} = 6\ \text{feet}$$

$$A_{s\,(distribution)} = \frac{220}{\sqrt{6\ \text{feet}}}\% \le 67\%\ \text{of} + A_{s(main)} = 89.81462\% \le 67\%\ \text{of} + A_{s(main)} = 67\%\ \text{of} + A_{s(main)}$$

$$= 67\%\ \text{of}\ 0.59393\ \text{inch}^2/\text{ft width} = 0.39793\ \text{inch}^2/\text{ft width}$$

check, $A_{s(distribution)}$ [=0.39793 inch²/ft width] > $A_{s(temperature\ \&\ shrinkage)}$ [=0.119466403 inch²/ft width]; OK

$$\text{spacing (\#4 bar), } s = \frac{12 \times A_{s(one\,bar,\,inch^2)}}{A_{s(required,\,inch^2/ft)}} = \frac{12 \times 0.20\ \text{inch}^2}{0.39793\ \text{inch}^2/\text{ft}} = 6.03121\ \text{inch} \approx 6\ \text{inch c/c}$$

so, $A_{s(distribution)}$ = **# 4 bar 6 inch center-to-center**

This distribution steel will be provided at top & bottom along longitudinal direction (span direction of bridge / traffic direction)

---

design for negative moment (negative main steel, $-A_{s(main)}$)

1st trial: assume, a = 1 inch

$$-A_{s(main)} = \frac{-M_{strength\ I}}{\varphi f_y\left(d - \frac{a}{2}\right)} = \frac{(-)7.52730 \times 12\ \text{kip-ft}}{0.90 \times 40\ \text{ksi} \times \left(6.1875^{"} - \frac{1^{"}}{2}\right)} = 0.441160\ \text{inch}^2/\text{ft width}$$

$$\text{check, } a = \frac{-A_{s(main)} f_y}{0.85 f_c' b} = \frac{(-)0.441160\ \text{inch}^2/\text{ft} \times 40\ \text{ksi}}{0.85 \times 3.5\ \text{ksi} \times 12\ \text{inch}} = 0.49430\ \text{inch} \ne 1\ \text{inch; not OK}$$

2nd trial: assume, a = 0.472 inch

$$-A_{s(main)} = \frac{-M_{strength\ I}}{\varphi f_y\left(d-\frac{a}{2}\right)} = \frac{(-)7.52730\times 12\ \text{kip}-\text{ft}}{0.90\times 40\ \text{ksi}\times\left(6.1875^{"}-\frac{0.472^{"}}{2}\right)} = 0.421591\ \text{inch}^2/\text{ft width}$$

$$\text{check, } a = \frac{-A_{s(main)} f_y}{0.85 f_c' b} = \frac{(-)0.421591\ \text{inch}^2/\text{ft}\times 40\ \text{ksi}}{0.85\times 3.5\ \text{ksi}\times 12\ \text{inch}} = 0.47237\ \text{inch} \approx 0.472\ \text{inch; OK}$$

check, $-A_{s(main)}$ [= 0.421591 inch²/ft width] > $A_{s(temperature\ \&\ shrinkage)}$ [= 0.20 inch²/ft width]; OK

$$\text{spacing (\#5 bar), } s = \frac{12\times A_{s(onebar,\ inch^2)}}{A_{s(required,\ inch^2/ft)}} = \frac{12\times 0.31\ \text{inch}^2}{0.421591\ \text{inch}^2/\text{ft}} = 8.82371\ \text{inch} \approx 8.50\ \text{inch c/c}$$ [lower rounding to 0.50 inch]

check maximum spacing of main bar ($-s_{maximum}$)

[i.e., check cracking of concrete at service load ($-M_{service\ I}$)]

$$s_{maximum} \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$

where, γ_e = exposure factor = 1.00 (for Class 1 exposure condition)
= 0.75 (for Class 2 exposure condition)

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (inch) [see figure 9]

if # 5 bar is chosen (bar diameter = 5/8 inch)

d_c (for negative bar) = clear cover + ½ of bar diameter = 1 inch + ½ of (5/8) inch = 1.3125 inch

[limit of d_c : must be ≤ (2 inch + ½ of bar diameter) ≤ 2.3125 inch; OK]

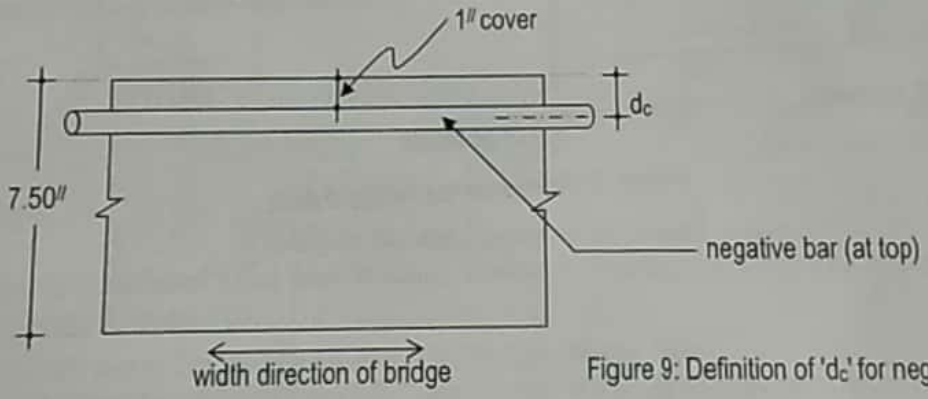


Figure 9: Definition of 'd_c' for negative bar

f_{ss} = tensile stress in steel reinforcement at service limit state (ksi)

h = overall thickness or depth of the component (inch) = thickness of slab = t_{slab} = 7.50 inch

$$f_{ss} = f_y \times \frac{+M_{service\ I}}{+M_{strength\ I}} = 60\ \text{ksi}\times\frac{-4.47609375\ \text{kip-ft/ft}}{-7.527304688\ \text{kip-ft/ft}} = 23.78590444\ \text{ksi}$$

$$\beta_{ss} = 1 + \frac{d_c}{0.7(h-d_c)} = 1 + \frac{1.3125^{"}}{0.7(7.50^{"}-1.3125^{"})} = 1.30303$$

$$s_{maximum} \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \le \left[\frac{700\times 1.00}{130303\times 23.78590\ \text{ksi}} - 2\times 1.3125^{"}\right] \le 19.96019\ \text{inch} \le 19.50\ \text{inch}$$

[lower rounding to 0.50 inch]

actual spacing of steel (s) shall not be more than $s_{maximum}$ calculated above. [$s \le s_{maximum}$]

as in this case actual spacing (s = 8.50 inch c/c) of negative steel ($-A_{s(main)}$) exceeds maximum permitted spacing ($s_{maximum}$ = 19.50 inch c/c) =>> provide 8.50 inch c/c spacing

Use **# 5 bar @ 8.50 inch center-to-center** (for negative moment). This $-A_{s(main)}$ will be provided at top along transverse direction (width direction of bridge). Clear cover for $-A_{s(main)}$ will be at least = 1.00 inch (measured from top of bar to concrete top surface) [see figure 9 & 10]

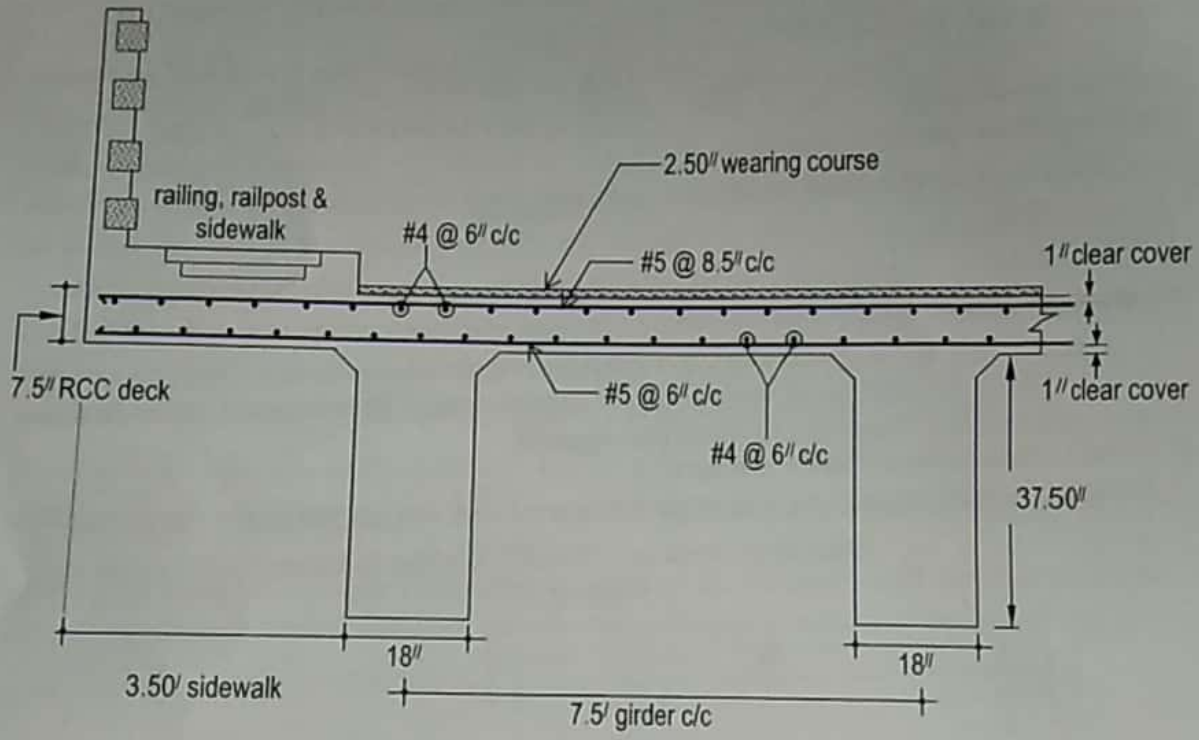


Figure 10: Reinforcement details of deck

Dead Load on Girders (Exterior & Interior)

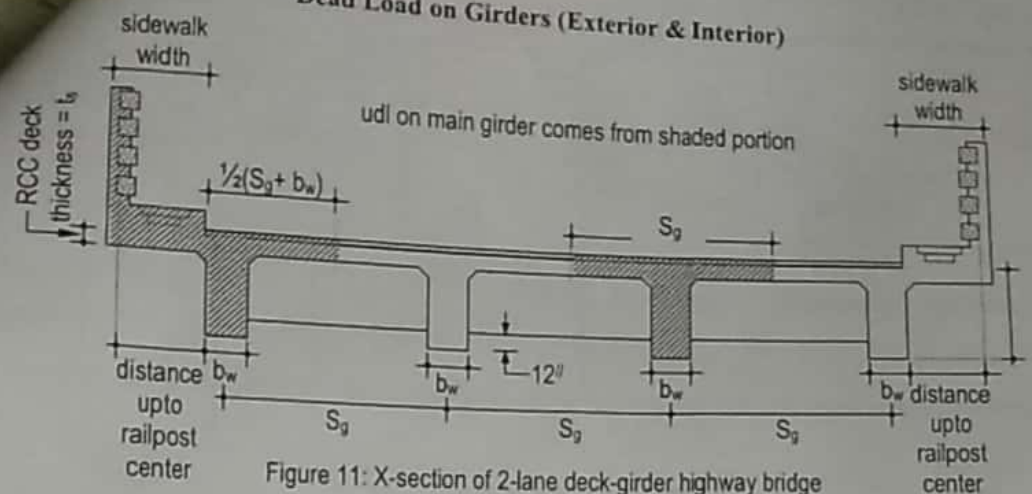


Figure 11: X-section of 2-lane deck-girder highway bridge
udl on exterior girder from a length of ½(S_g+ b_w)
& udl on interior girder from a length of S_g

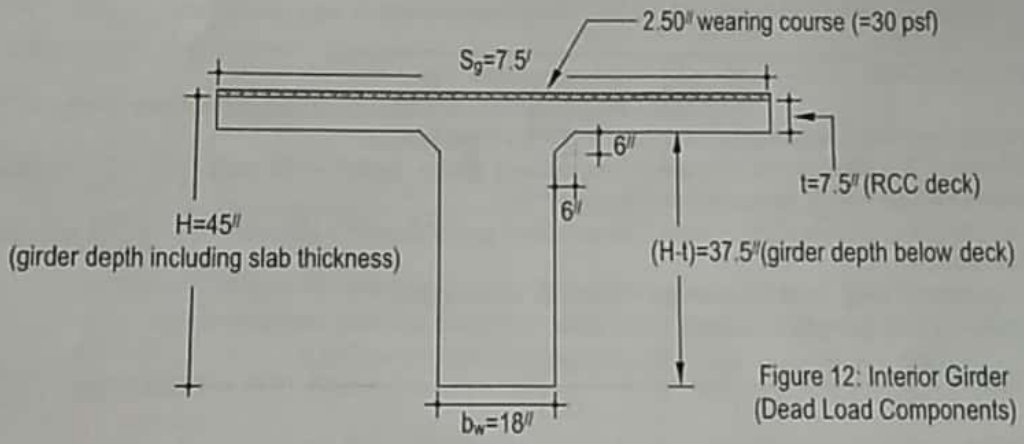


Figure 12: Interior Girder
(Dead Load Components)

= Interior Girder: Analysis for Dead Load =

► Uniformly Distributed Load from Wearing Surface = Wearing Surface (psf) × girder spacing (feet)

$= w_{wearing}(psf) \times S_g(feet)$

= 30 psf × 7.50 feet = 225 lb. / linear foot = **0.225 kip / linear foot**

► Uniformly Distributed Load from RC Deck = Deck Weight (psf) × girder spacing (feet)

$$= w_{slab}(psf) \times S_g(feet) = \frac{t_{slab}}{12}\text{ feet} \times 150\text{ pcf} \times S_g(feet) = \frac{7.50''}{12}\text{ feet} \times 150\text{ pcf} \times 7.50\text{ feet}$$

= 93.75 psf × 7.5 feet = 703.125 lb. / linear foot = **0.703125 kip / linear foot**

► Uniformly Distributed Load from 6 inch × 6 inch triangular haunch

$$= 2\text{ haunch} @ \frac{1}{2} \times \frac{6''}{12}\text{ feet} \times \frac{6''}{12}\text{ feet} \times 150\text{ pcf} = 37.50\text{ lb. / linear foot} = \textbf{0.0375 kip / linear foot}$$

► Uniformly Distributed Load from constant portion of girder (i.e., girder portion below deck slab*)

= girder width (b_w) × girder depth (H−t)* × 150 pcf

$$= \frac{18''}{12}\text{ feet} \times \left(\frac{45''}{12} - \frac{7.5''}{12}\right)\text{feet} \times 150\text{ pcf} = 703.125\text{ lb. / linear foot} = \textbf{0.703125 kip / linear foot}$$

Constant udl on interior girder due to self weight of various components excluding wearing surface (total) = (0.703125 + 0.0375 + 0.703125) kip / linear foot = **1.44375 kip / linear foot**

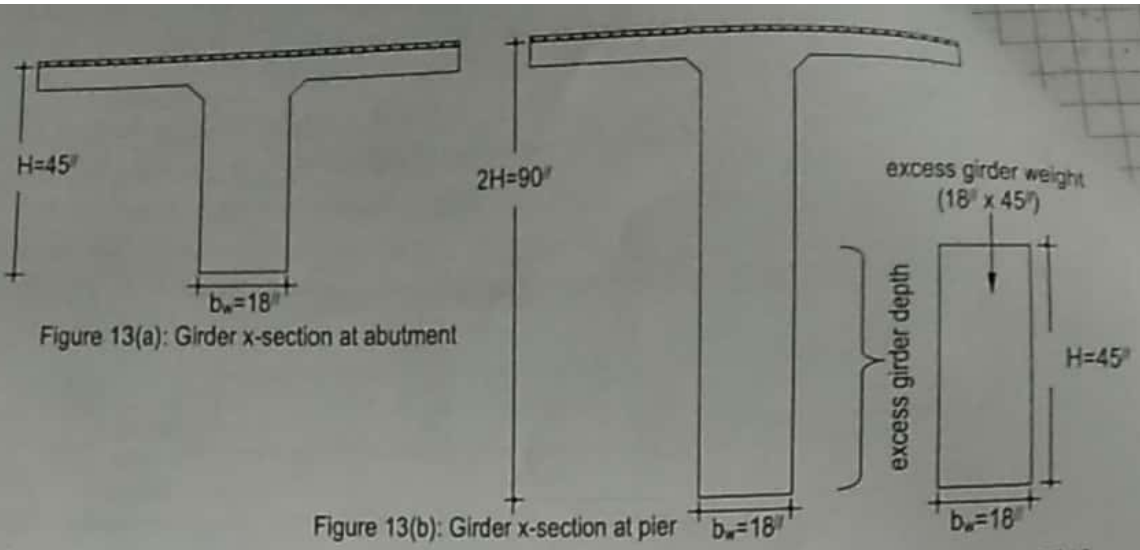


Figure 13(a): Girder x-section at abutment

Figure 13(b): Girder x-section at pier

► Uniformly Varying Load at pier location where girder depth is increased (H to 2H) [figure 13(b)]
= girder width (b_w) × excess girder depth (H) × 150 pcf

$$= \frac{18''}{12}\text{ feet} \times \frac{45''}{12}\text{ feet} \times 150\text{ pcf} = 843.75\text{ lb. / linear foot} = 0.84375\text{ kip / linear foot}$$

Diaphragm / Cross-girder width = 18 inch (at supports – abutments & piers)
= 10 inch (at articulations & other/intermediate locations, if needed**)
**Spacing between Diaphragm / X-girder = 30 feet (maximum)
Diaphragm / X-girder depth = main / longitudinal girder depth* – 12 inch = H – t – 12 inch (at abutment, articulation & intermediate locations**)
Diaphragm / X-girder depth = = main / longitudinal girder depth* – 12 inch = 2H – t – 12 inch (at pier locations)

► Concentrated load from Diaphragm / X-girder at abutment
= X-girder (width × depth × length) × 150 pcf

$$= \frac{18''}{12}\text{ feet} \times \left(\frac{45''}{12} - \frac{7.5''}{12} - \frac{12''}{12}\right)\text{feet} \times \left(7.5 - \frac{18''}{12}\right)\text{feet} \times 150\text{ pcf} = 2868.75\text{ lb.} = 2.86875\text{ kip}$$

► Concentrated load from Diaphragm / X-girder at articulation & intermediate locations
= X-girder (width × depth × length) × 150 pcf

$$= \frac{10''}{12}\text{ feet} \times \left(\frac{45''}{12} - \frac{7.5''}{12} - \frac{12''}{12}\right)\text{feet} \times \left(7.5 - \frac{18''}{12}\right)\text{feet} \times 150\text{ pcf} = 1593.75\text{ lb.} = 1.59375\text{ kip}$$

► Concentrated load from Diaphragm / X-girder at pier locations
= X-girder (width × depth × length) × 150 pcf

$$= \frac{18''}{12}\text{ feet} \times \left(\frac{90''}{12} - \frac{7.5''}{12} - \frac{12''}{12}\right)\text{feet} \times \left(7.5 - \frac{18''}{12}\right)\text{feet} \times 150\text{ pcf} = 7931.25\text{ lb.} = 7.93125\text{ kip}$$

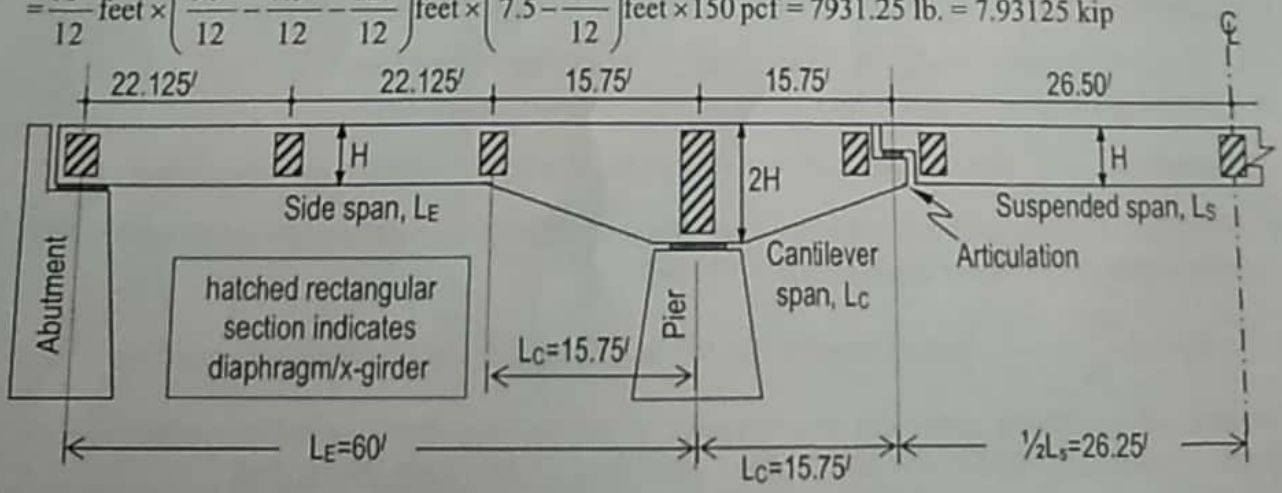


Figure 14: Longitudinal Profile of Three Span balanced cantilever bridge with position of x-girders/diaphragms

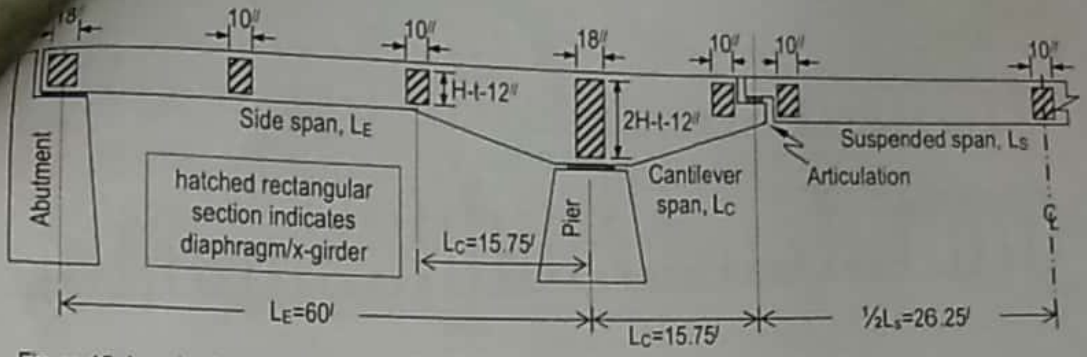


Figure 15: Longitudinal Profile of Three Span balanced cantilever bridge with position of x-girders/diaphragms

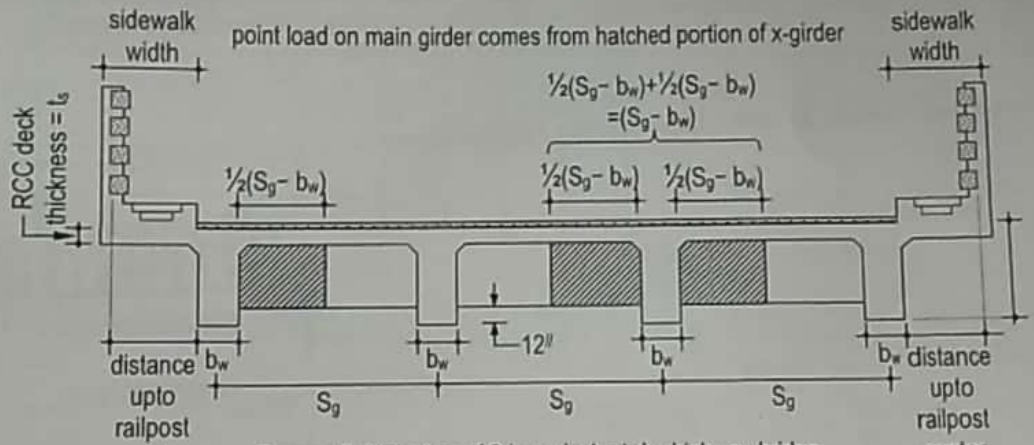


Figure 16: X-section of 2-lane deck-girder highway bridge
x-girder imposed point load on exterior girder from a length of ½(S_g– b_w)
& on interior girder from a length of (S_g– b_w)

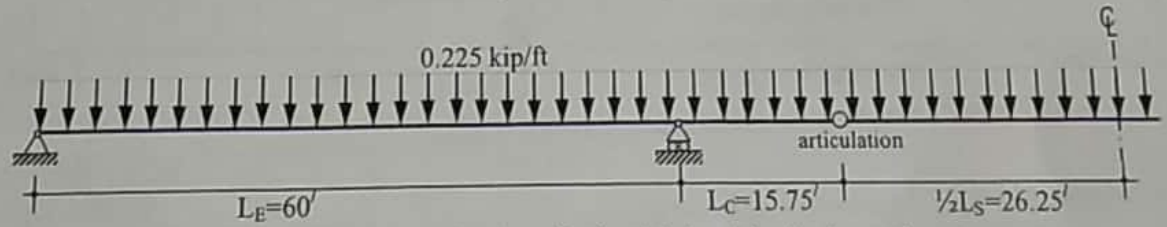


Figure 17: Uniformly distributed load on interior girder due to wearing course

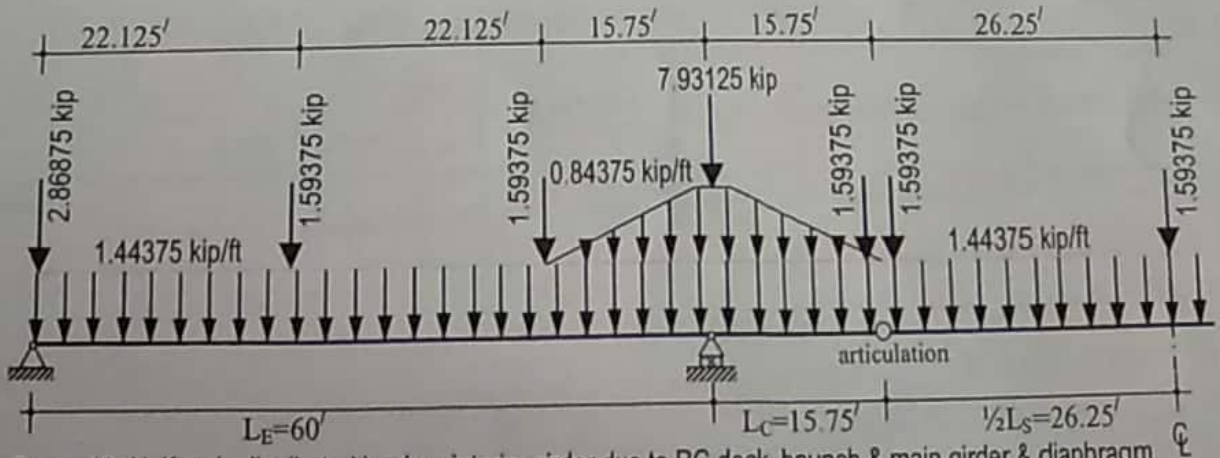


Figure 18: Uniformly distributed load on interior girder due to RC deck, haunch & main girder & diaphragm

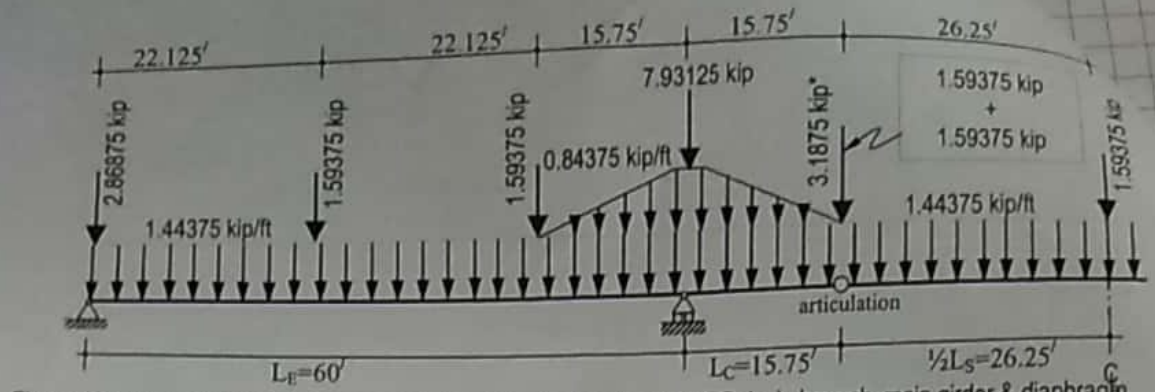


Figure 19: Uniformly distributed load on interior girder due to self-weight RC deck, haunch, main girder & diaphragm (two closely spaced concentrated loads at articulation can be replaced by a single concentrated load (i.e., 1.59375 kip +1.59375 kip = 3.1875 kip)

= **Dead Load Moment & Shear Computation: Interior Girder** =

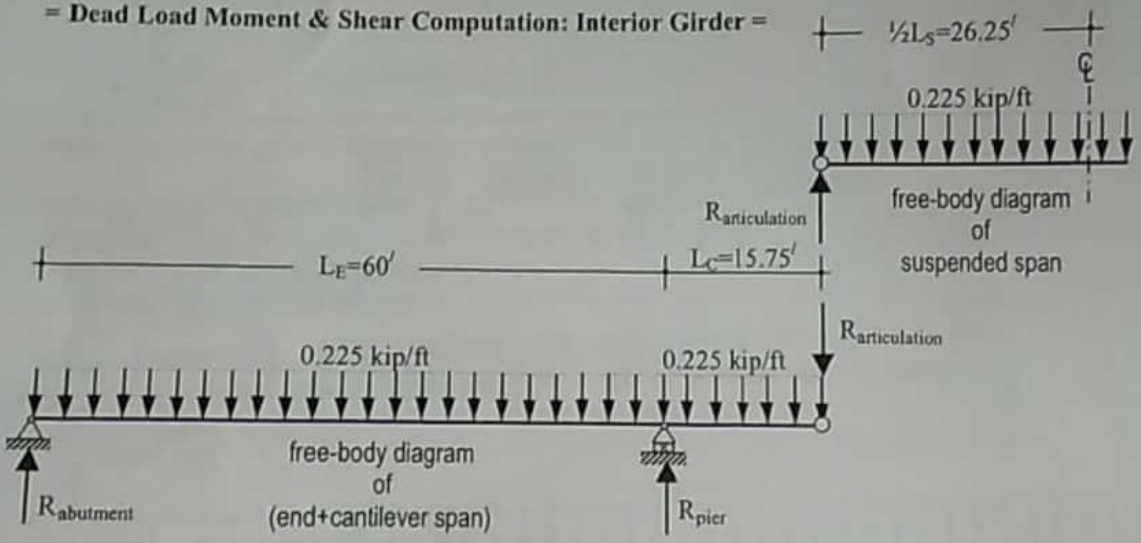


Figure 20: Free-body diagram of interior girder due to wearing course loading

Considering free-body diagram of suspended span and applying $\sum F_y = 0$ (upward force positive)

$$R_{articulation} = w_{wearing} \times \frac{1}{2} L_s = 0.225 \text{ kip/ft} \times 26.25 \text{ ft} = 5.90625 \text{ kip}$$

Considering free-body diagram of (end+cantilever) span and applying $\sum M_{\text{about "abutment"}} = 0$ (clockwise moment positive)

$$\Rightarrow - R_{pier} \times 60 \text{ ft} + 0.225 \text{ kip/ft} \times (60+15.75)\text{ft} \times \frac{(60+15.75)\text{ ft}}{2} + R_{articulation} \times (60+15.75) \text{ ft} = 0$$

$$\Rightarrow - R_{pier} \times 60 \text{ ft} + 0.225 \text{ kip/ft} \times (60+15.75)\text{ft} \times \frac{(60+15.75)\text{ ft}}{2} + 5.90625 \text{ kip} \times (60+15.75) \text{ ft} = 0$$

$$\Rightarrow R_{pier} = +18.21550781 \text{ kip } (\uparrow)$$

$$\sum F_y = 0 (\uparrow +ve);\ R_{pier} + R_{abutment} - 0.225 \text{ kip/ft} \times (60+15.75) \text{ ft} - R_{articulation} = 0$$

$$\Rightarrow 18.21550781 \text{ kip} + R_{abutment} - 0.225 \text{ kip/ft} \times (60+15.75) \text{ ft} - 5.90625 \text{ kip} = 0$$

$$\Rightarrow R_{abutment} = +4.734492188 \text{ kip } (\uparrow)$$

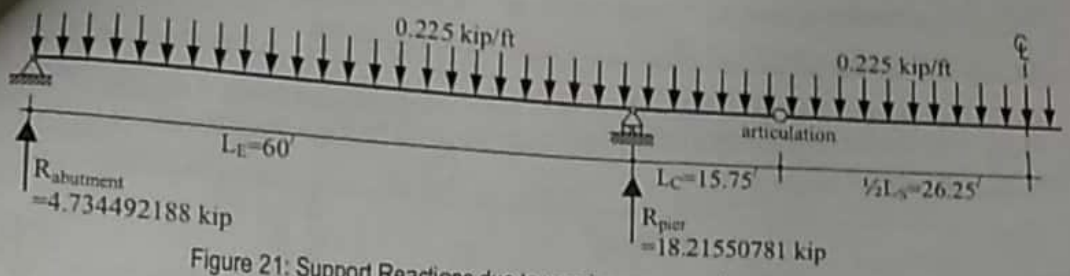


Figure 21: Support Reactions due to wearing course on interior girder

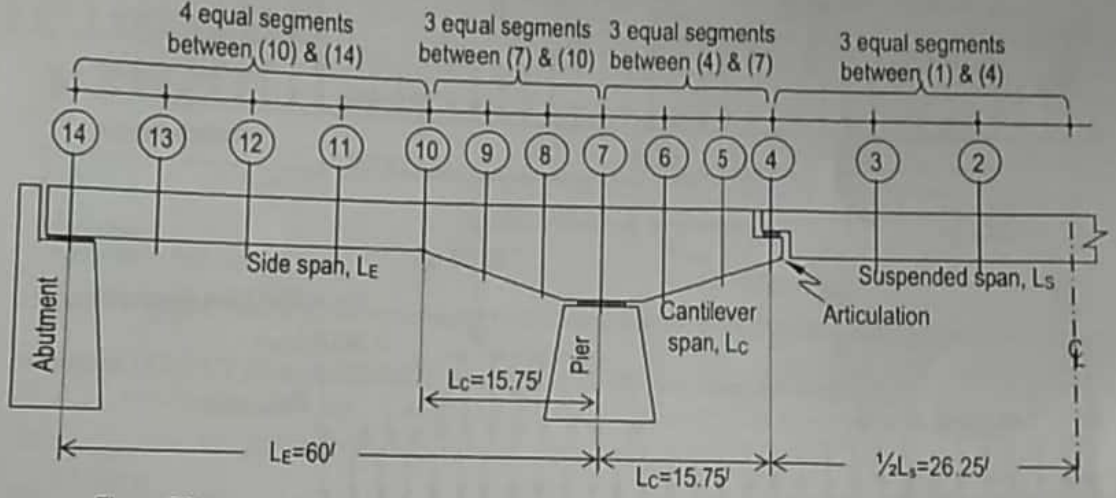


Figure 22(a): Longitudinal Profile of Three Span balanced cantilever bridge (Design Sections)

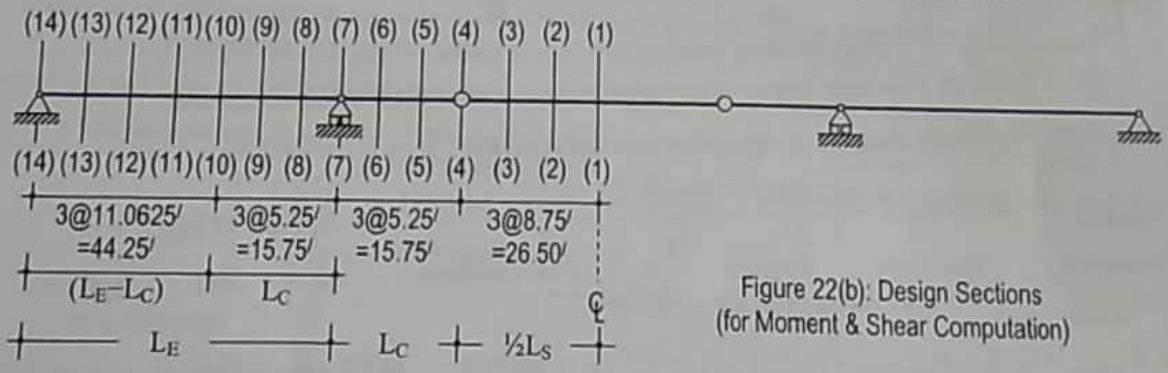


Figure 22(b): Design Sections
(for Moment & Shear Computation)

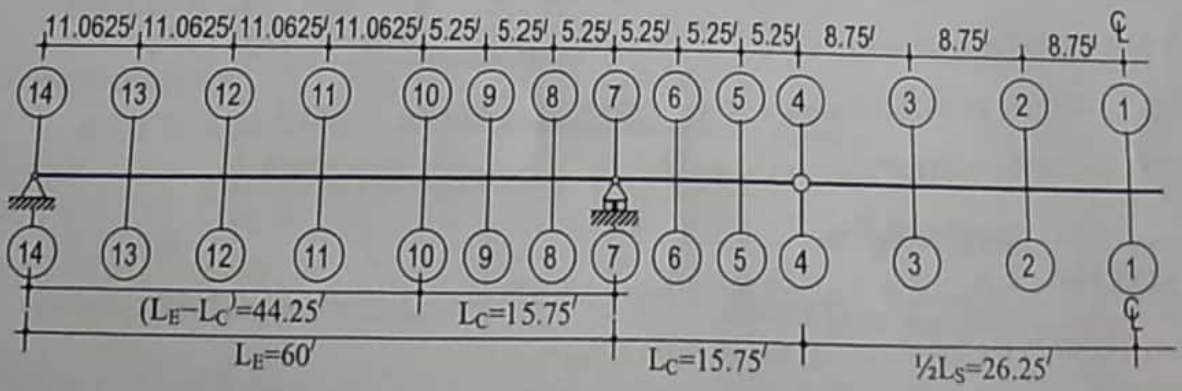


Figure 22(c): Design Sections (for Moment & Shear Computation)

at section (12)-(12) [section (12)-(12) is located at 22.125 feet from left support)

$V_{wearing} = +4.734492188 \text{ kip} - 0.225 \text{ kip/ft} \times 22.125 \text{ ft} = -0.243632812 \text{ kip}$

$M_{wearing} = +4.734492188 \text{ kip} \times 22.125 \text{ ft} - 0.225 \text{ kip/ft} \times 22.125 \text{ ft} \times \frac{22.125 \text{ ft}}{2} = +49.68013185 \text{ kip-ft}$

at section (4)-(4) [section (4)-(4) is located at 75.75 feet = (60+15.75) feet from left support)

$V_{wearing} = +4.734492188 \text{ kip} + 18.2155781 \text{ kip} - 0.225 \text{ kip/ft} \times 75.75 \text{ ft} = +5.90625 \text{ kip} (= R_{articulation})$

$M_{wearing} = 0$ (internal hinge)

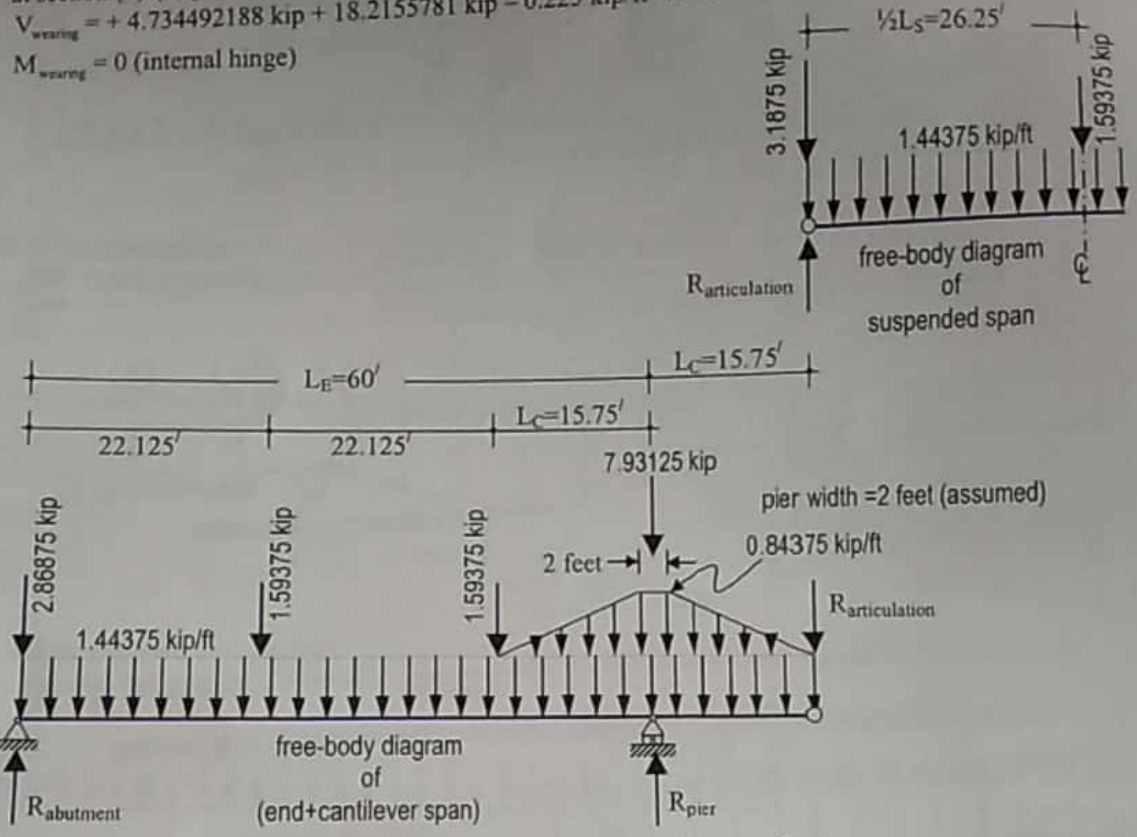


Figure 23: Free-body diagram of interior girder due to self-weight of various components

Considering free-body diagram of suspended span and applying $\sum F_y = 0$ (upward force positive)

$$R_{articulation} = 1.44375 \text{ kip/ft} \times \frac{52.50 \text{ ft}}{2} + \frac{(3.1875 + 1.59375 + 3.1875) \text{ kip}}{2} = 41.8828125 \text{ kip}$$

Considering free-body diagram of (end+cantilever) span and applying $\sum M_{about\ "pier"} = 0$ (clockwise moment positive)

$\Rightarrow R_{abutment} \times 60 \text{ ft} + R_{articulation} \times 15.75 \text{ ft} - 1.59375 \text{ kip} \times 15.75 \text{ ft} - 1.59375 \text{ kip} \times 37.875 \text{ ft} - 2.86875 \text{ kip} \times 60 \text{ ft} + 1.44375 \text{ kip/ft} \times 15.75 \text{ ft} \times \frac{15.75 \text{ ft}}{2} - 1.44375 \text{ kip/ft} \times 60 \text{ ft} \times \frac{60 \text{ ft}}{2} = 0$

$\Rightarrow R_{abutment} \times 60 \text{ ft} + 41.8828125 \text{ kip} \times 15.75 \text{ ft} - 1.59375 \text{ kip} \times 15.75 \text{ ft} - 1.59375 \text{ kip} \times 37.875 \text{ ft} - 2.86875 \text{ kip} \times 60 \text{ ft} + 1.44375 \text{ kip/ft} \times 15.75 \text{ ft} \times \frac{15.75 \text{ ft}}{2} - 1.44375 \text{ kip/ft} \times 60 \text{ ft} \times \frac{60 \text{ ft}}{2} = 0$

$\Rightarrow R_{abutment} = +33.62692383 \text{ kip} (\uparrow)$

$\sum F_y = 0 (\uparrow +ve); + R_{abutment} + R_{pier} - 2.86875 \text{ kip} - 1.59375 \text{ kip} - 1.59375 \text{ kip} - 7.93125 \text{ kip} - R_{articulation} - 1.44375 \text{ kip/ft} \times (60+15.75) \text{ ft} - 14.1328125 \text{ kip}^* = 0$

[*uniformly varying load => area of trapezoid $= \frac{1}{2} \times (a+b) \times h$

$= \frac{1}{2} \times (2 \text{ feet} + 31.5 \text{ feet}) \times 0.84375 \text{ kip/ft} = 14.1328125 \text{ kip}]$

$\Rightarrow +33.62692383 \text{ kip} + R_{pier} - 2.86875 \text{ kip} - 1.59375 \text{ kip} - 1.59375 \text{ kip} - 7.93125 \text{ kip} - 41.8828125 \text{ kip} - 1.44375 \text{ kip/ft} \times (60+15.75) \text{ ft} - 14.1328125 \text{ kip} = 0$

$\Rightarrow R_{pier} = +145.7402637 \text{ kip} (\uparrow)$

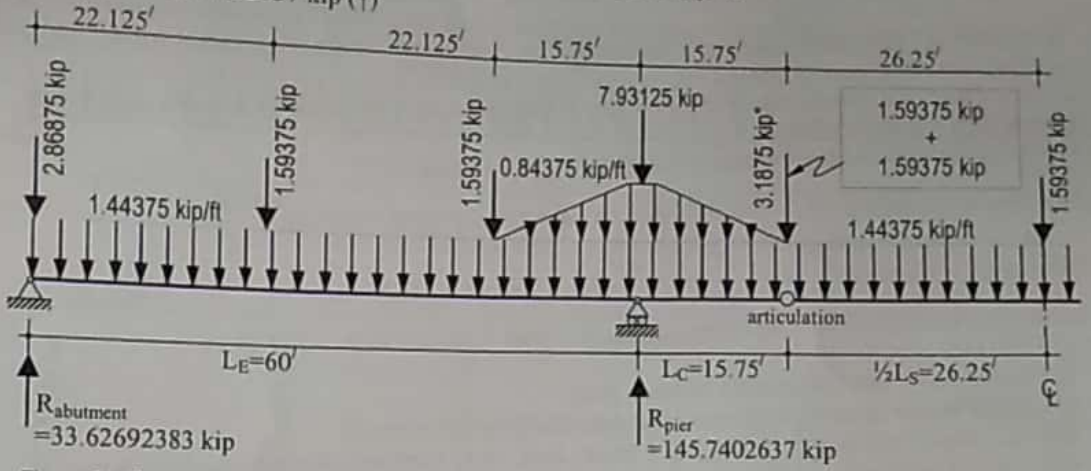


Figure 24: Support reactions due to self-weight of RC deck, haunch & main girder, diaphragm on interior girder

at section (12)-(12) [section (12)-(12) is located at 22.125 feet from left support)

$V_{self-weight} = +33.62692383 \text{ kip} - 2.86875 \text{ kip} - 1.44375 \text{ kip/ft} \times 22.125 \text{ ft} = -1.18479492 \text{ kip}$

$M_{self-weight} = +33.62692383 \text{ kip} \times 22.125 \text{ ft} - 2.86875 \text{ kip} \times 22.125 \text{ ft} - 1.44375 \text{ kip/ft} \times 22.125 \text{ ft} \times \frac{22.125 \text{ ft}}{2} = +327.1555042 \text{ kip-ft}$

at section (4)-(4) [section (4)-(4) is located at 75.75 feet = (60+15.75) feet from left support)

$V_{self-weight} = +33.62692383 \text{ kip} - 2.86875 \text{ kip} - 1.59375 \text{ kip} - 1.59375 \text{ kip} - 7.93125 \text{ kip} + 145.7402637 \text{ kip} - 1.44375 \text{ kip/ft} \times 75.75 \text{ ft} - 14.1328125 \text{ kip}^* = +41.8828125 \text{ kip} (= R_{articulation})$

[*trapezoidal area load = 14.1328125 kip (previously calculated] & $M_{self-weight} = 0$ (internal hinge)

= Exterior Girder: Analysis for Dead Load =

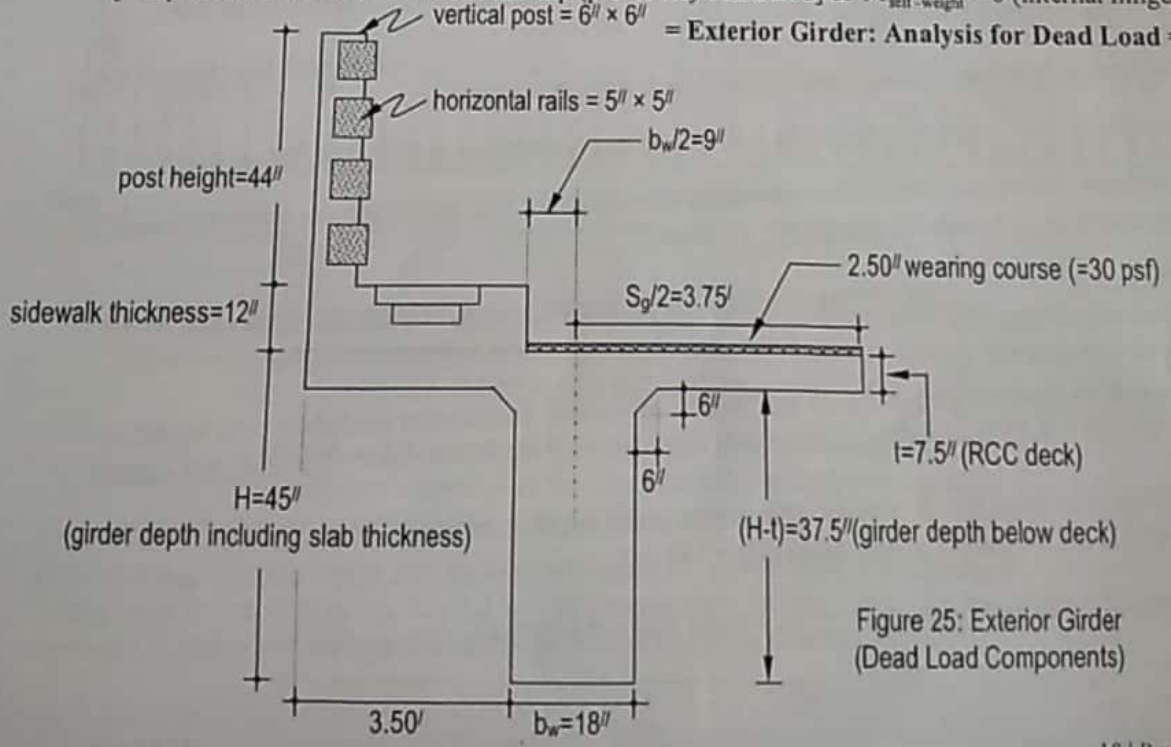


Figure 25: Exterior Girder (Dead Load Components)

► Uniformly Distributed Load from Wearing Surface = Wearing Surface (psf) × [½ of girder spacing + ½ of girder width] (feet) = $w_{wearing}(psf) \times \left[\frac{S_g}{2} + \frac{b_w}{2}\right](feet)$

$= 30\ psf \times \left[\frac{7.50'}{2} + \frac{18''/12}{2}\right] feet = 135$ lb. / linear foot = **0.135 kip / linear foot**

► Uniformly Distributed Load from Railing & Post (horizontal members are called rail & vertical members are called post/railpost)

Railing x-sectional dimension = 5 inch × 5 inch & length = 5 feet* (c/c distance between post)

[*consider a repetitive section]

Post x-sectional dimension = 6 inch × 6 inch & height = 44 inch

Rail weight = 4 railings @ $\frac{5''}{12}$ feet $\times \frac{5''}{12}$ feet $\times 5$ feet $\times 150$ pcf = 520.83333 lb.

Post weight = 1 post @ $\frac{6''}{12}$ feet $\times \frac{6''}{12}$ feet $\times \frac{44''}{12}$ feet $\times 150$ pcf = 137.50 lb.

Rail & Post weight (within a repetitive portion of 5 feet) = 520.83333 lb. + 137.50 lb.
= 658.33333 lb. (total)
= 658.33333 lb. / 5 feet (c/c spacing between post)
= 131.66667 lb. per linear foot along bridge span (uniformly distributed)
= 125 lb. / linear foot (say) = **0.125 kip / linear foot**, since there are overlapping of rail & post x-sections at 4 (four) locations along post height → so, weight will be reduced to some extent than that calculated

► Uniformly Distributed Load from sidewalk (length = 3.50 feet & height/thickness = 19.50 inch = 12 inch above RC deck +7.50 inch deck thickness)

$= 3.50$ feet $\times \frac{19.50''}{12}$ feet $\times 150$ pcf = 853.125 lb./linear ft = **0.853125 kip / linear foot**

► Uniformly Distributed Load from RC Deck = Deck Weight (psf) × [½ of girder spacing + ½ of girder width] (feet)

$= w_{slab}(psf) \times S_g(feet) = \frac{t_{slab}}{12}$ feet $\times 150$ pcf $\times \left[\frac{S_g}{2} + \frac{b_w}{2}\right](feet)$

$= \frac{7.50''}{12}$ feet $\times 150$ pcf $\times \left[\frac{7.50'}{2} + \frac{18''/12}{2}\right]$ feet $= 93.75$ psf $\times 4.50$ feet $= 421.875$ lb. / linear foot

$= 0.421875$ **kip / linear foot**

► Uniformly Distributed Load from 6 inch × 6 inch triangular haunch

$= 2$ haunch @ $\frac{1}{2} \times \frac{6''}{12}$ feet $\times \frac{6''}{12}$ feet $\times 150$ pcf = 37.50 lb. / linear foot = **0.0375 kip / linear foot**

► Uniformly Distributed Load from constant portion of girder (i.e., girder portion below deck slab*)
= girder width $(b_w) \times$ girder depth $(H-t)^* \times 150$ pcf

$= \frac{18''}{12}$ feet $\times \left(\frac{45''}{12} - \frac{7.5''}{12}\right)$ feet $\times 150$ pcf = 703.125 lb. / linear foot = **0.703125 kip / linear foot**

Constant udl on exterior girder due to self weight of various components excluding wearing surface (total) = (0.125 + 0.853125 + 0.421875 + 0.0375 + 0.703125) kip / linear foot = **2.150625 kip / linear foot**

► Uniformly Varying Load at pier location where girder depth is increased (H to 2H)
= girder width $(b_w) \times$ excess girder depth $(H) \times 150$ pcf

$= \frac{18''}{12}$ feet $\times \frac{45''}{12}$ feet $\times 150$ pcf = 843.75 lb. / linear foot = **0.84375 kip / linear foot**

oncentrated load from Diaphragm / X-girder on Exterior Girder will be half of that for Interior Girder (see Figure 16 for explanation)

- Concentrated load from Diaphragm / X-girder at abutment = ½ of 2.86875 kip = 1.434375 kip
- Concentrated load from Diaphragm / X-girder at articulation & intermediate locations =½ of 1.59375 kip = 0.796875 kip
- Concentrated load from Diaphragm / X-girder at pier locations = ½ of 7.93125 kip = 3.965625 kip

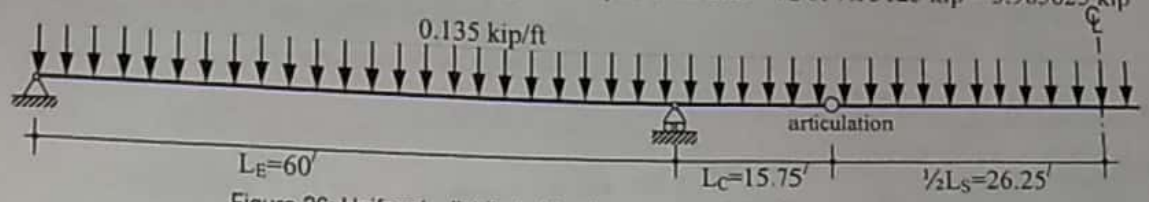


Figure 26: Uniformly distributed load on exterior girder due to wearing course

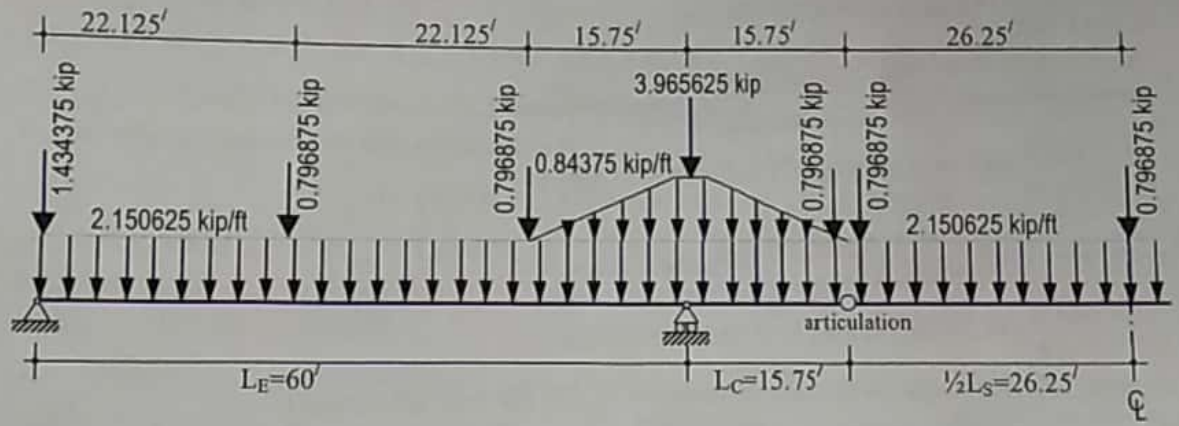


Figure 27: Uniformly distributed load on exterior girder due to RC deck, haunch & main girder & diaphragm

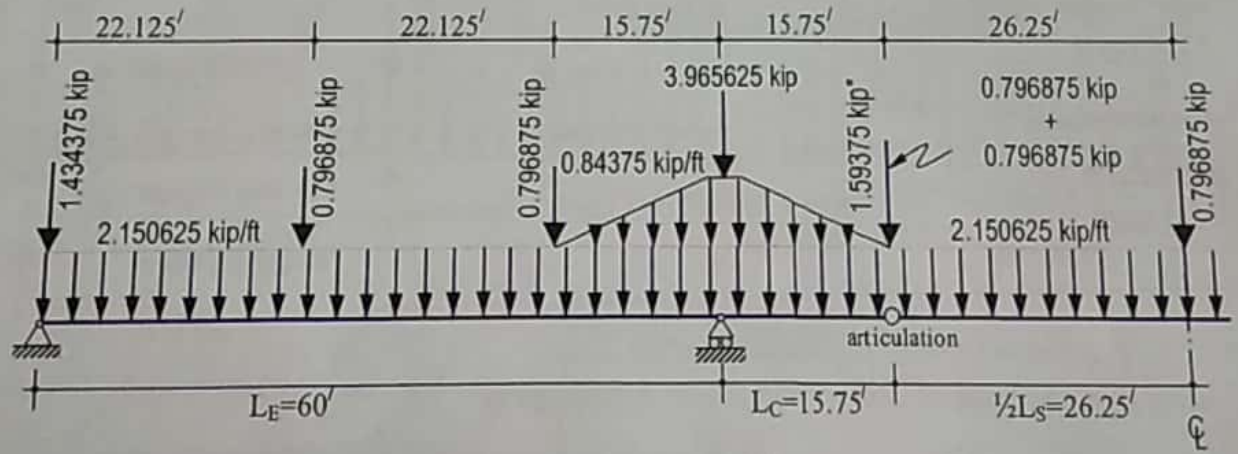


Figure 28: Uniformly distributed load on exterior girder due to RC deck, haunch & main girder & diaphragm (two closely spaced concentrated loads at articulation can be replaced by a single concentrated load (i.e., 0.796875 kip +1. 796875 kip = 1.59375 kip)

= Dead Load Moment & Shear Computation: Exterior Girder =

(similar to the procedure that described above for interior girder)

For wearing course on exterior girder, it is not necessary to compute $R_{articulation}$, $R_{abutment}$ & R_{pier} ass like interior girder (see figure 20). As $M \propto wL^2$ and $V \propto wL$ or more specifically, $M \propto w$ and $V \propto w$ (since, span length L (i.e, L_E, L_C, L_M & L_S) remain same in this case). So, reactions at supports, moments & shears will change in accordance with w (wearing course).

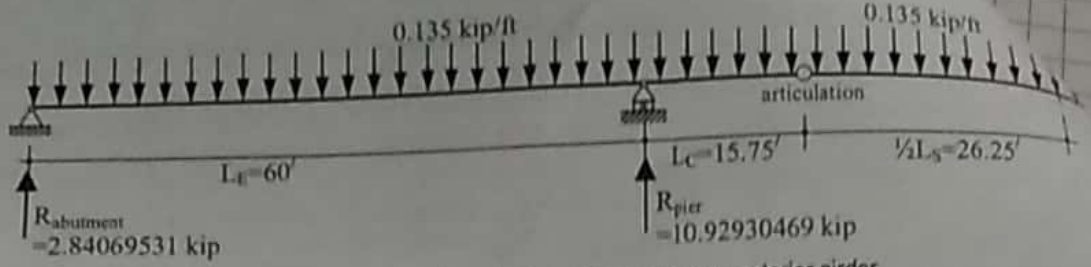


Figure 29: Support Reactions due to wearing course on exterior girder

$$w_{wearing\ (for\ interior\ girder)} = 0.225\ kip/ft.\ \&\ w_{wearing\ (for\ exterior\ girder)} = 0.135\ kip/ft.$$

$$R_{articulation\ (for\ exterior\ girder)} = \frac{w_{wearing\ (for\ exterior\ girder)}}{w_{wearing\ (for\ interior\ girder)}} \times R_{articulation\ (for\ interior\ girder)}$$

$$= \frac{0.135\ kip/ft}{0.225\ kip/ft} \times 5.90625\ kip = 3.543750\ kip$$

$$R_{abutment\ (for\ exterior\ girder)} = \frac{w_{wearing\ (for\ exterior\ girder)}}{w_{wearing\ (for\ interior\ girder)}} \times R_{abutment\ (for\ interior\ girder)}$$

$$= \frac{0.135\ kip/ft}{0.225\ kip/ft} \times 4.734492188\ kip\ (\uparrow) = 2.84069531\ kip\ (\uparrow)$$

$$R_{pier\ (for\ exterior\ girder)} = \frac{w_{wearing\ (for\ exterior\ girder)}}{w_{wearing\ (for\ interior\ girder)}} \times R_{pier\ (for\ interior\ girder)}$$

$$= \frac{0.135\ kip/ft}{0.225\ kip/ft} \times 18.21550781\ kip\ (\uparrow) = 10.92930469\ kip\ (\uparrow)$$

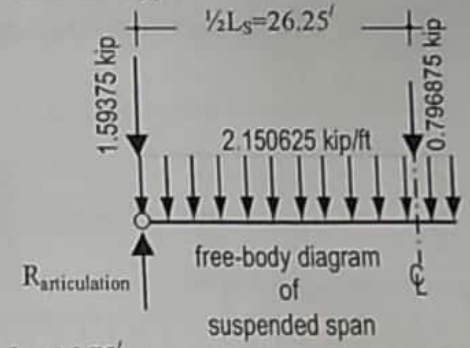
at section (12)-(12) [section (12)-(12) is located at 22.125 feet from left support)

$$V_{wearing\ (for\ exterior\ girder)} = \frac{w_{wearing\ (for\ exterior\ girder)}}{w_{wearing\ (for\ interior\ girder)}} \times V_{wearing\ (for\ interior\ girder)}$$

$$= \frac{0.135\ kip/ft}{0.225\ kip/ft} \times (-0.243632812\ kip) = -0.146179687\ kip$$

$$V_{wearing\ (for\ exterior\ girder)} = \frac{w_{wearing\ (for\ exterior\ girder)}}{w_{wearing\ (for\ interior\ girder)}} \times V_{wearing\ (for\ interior\ girder)}$$

$$= \frac{0.135\ kip/ft}{0.225\ kip/ft} \times (+49.68013185\ kip) = +29.80807911\ kip\text{-}ft$$



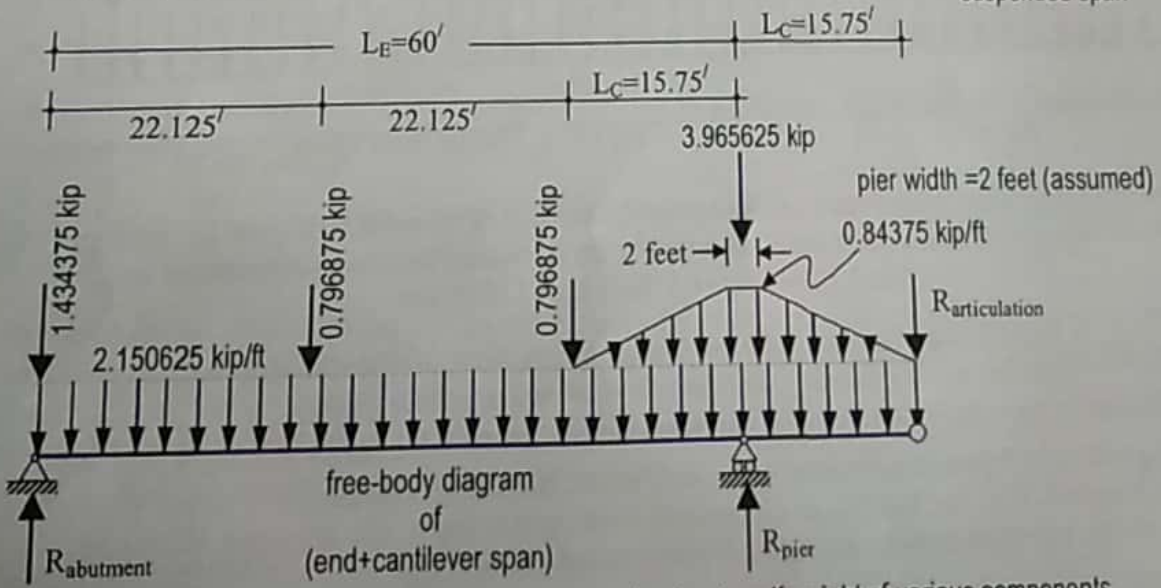


Figure 30: Free-body diagram of interior girder due to self-weight of various components

Considering free-body diagram of suspended span and applying $\sum F_y = 0$ (upward force positive)

$$R_{articulation} = 2.150625 \text{ kip/ft} \times \frac{52.50 \text{ ft}}{2} + \frac{(1.59375 + 0.796875 + 1.59375) \text{ kip}}{2} = 58.44609375 \text{ kip}$$

Considering free-body diagram of (end+cantilever) span and applying $\sum M_{about\ "pier"} = 0$ (clockwise moment positive)

$\Rightarrow R_{abutment} \times 60 \text{ ft} + R_{articulation} \times 15.75 \text{ ft} - 0.796875 \text{ kip} \times 15.75 \text{ ft} - 0.796875 \text{ kip} \times 37.875 \text{ ft} - 1.434375 \text{ kip} \times 60 \text{ ft} + 2.150625 \text{ kip/ft} \times 15.75 \text{ ft} \times \frac{15.75 \text{ ft}}{2} - 2.150625 \text{ kip/ft} \times 60 \text{ ft} \times \frac{60 \text{ ft}}{2} = 0$

$\Rightarrow R_{abutment} \times 60 \text{ ft} + 58.44609375 \text{ kip} \times 15.75 \text{ ft} - 0.796875 \text{ kip} \times 15.75 \text{ ft} - 0.796875 \text{ kip} \times 37.875 \text{ ft} - 1.434375 \text{ kip} \times 60 \text{ ft} + 2.150625 \text{ kip/ft} \times 15.75 \text{ ft} \times \frac{15.75 \text{ ft}}{2} - 2.150625 \text{ kip/ft} \times 60 \text{ ft} \times \frac{60 \text{ ft}}{2} = 0$

$\Rightarrow R_{abutment} = +46.8774873 \text{ kip } (\uparrow)$

$\sum F_y = 0 (\uparrow +ve); + R_{abutment} + R_{pier} - 1.434375 \text{ kip} - 0.796875 \text{ kip} - 0.796875 \text{ kip} - 3.965625 \text{ kip} - R_{articulation} - 2.150625 \text{ kip/ft} \times (60+15.75) \text{ ft} - 14.1328125 \text{ kip}^* = 0$

[*uniformly varying load => area of trapezoid =

$$= \frac{1}{2} \times (a+b) \times h = \frac{1}{2} \times (2 \text{ feet} + 31.5 \text{ feet}) \times 0.84375 \text{ kip/ft} = 14.1328125 \text{ kip}]$$

$\Rightarrow + 46.8774873 \text{ kip} + R_{pier} - 1.434375 \text{ kip} - 0.796875 \text{ kip} - 0.796875 \text{ kip} - 3.965625 \text{ kip} - 58.44609375 \text{ kip} - 2.150625 \text{ kip/ft} \times (60+15.75) \text{ ft} - 14.1328125 \text{ kip} = 0$

$\Rightarrow R_{pier} = +195.6050127 \text{ kip } (\uparrow)$

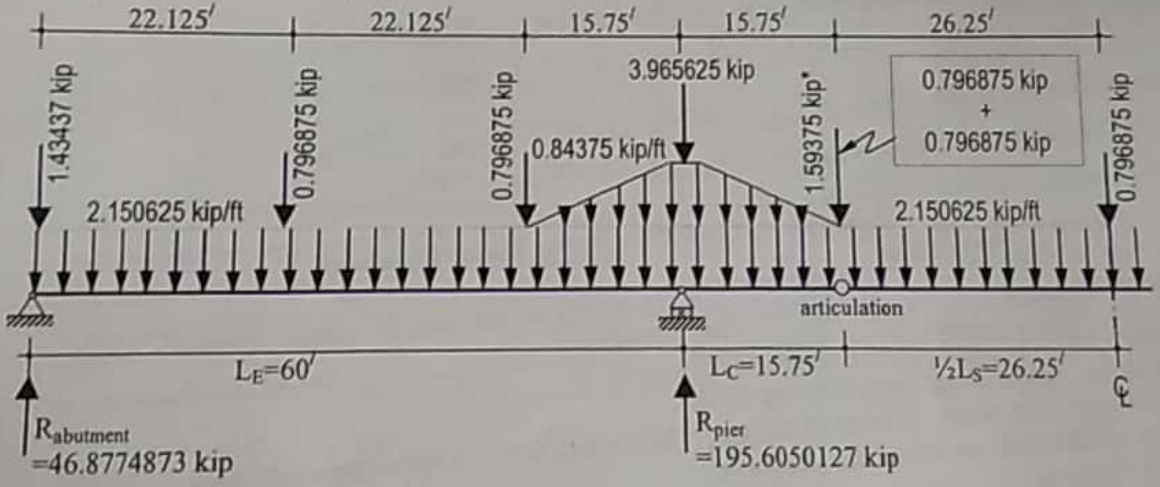


Figure 31: Support reactions due to self-weight of RC deck, haunch & main girder, diaphragm on exterior girder

at section (12)-(12) [section (12)-(12) is located at 22.125 feet from left support)

$V_{self-weight} = + 46.8774873 \text{ kip} - 1.434375 \text{ kip} - 2.150625 \text{ kip/ft} \times 22.125 \text{ ft} = -2.139465825 \text{ kip}$

$M_{self-weight} = + 46.8774873 \text{ kip} \times 22.125 \text{ ft} - 1.434375 \text{ kip} \times 22.125 \text{ ft} - 2.150625 \text{ kip/ft} \times 22.125 \text{ ft} \times \frac{22.125 \text{ ft}}{2} = + 479.0465891 \text{ kip-ft}$

at section (4)-(4) [section (4)-(4) is located at 75.75 feet = (60+15.75) feet from left support)

$V_{self-weight} = + 46.8774875 \text{ kip} - 1.434375 \text{ kip} - 0.796875 \text{ kip} - 0.796875 \text{ kip} - 3.965625 \text{ kip} + 195.6050127 \text{ kip} - 2.150625 \text{ kip/ft} \times 75.75 \text{ ft} - 14.1328125 \text{ kip}^* = +58.44609375 \text{ kip } (= R_{articulation})$

[*trapezoidal area load = 14.1328125 kip (previously calculated)]

$M_{self-weight} = 0$ (internal hinge)

Live Load on Girders (Exterior & Interior)

modular ratio between girder & deck material, $n = \frac{E_{\text{girder}}}{E_{\text{deck}}} = ?$

cross-sectional area of basic beam, A = ? inch^2; moment of inertia of the basic beam, I = ? inch^4

distance between the centers of gravity of the girder and deck, e_g = ? inch

longitudinal stiffness parameter, $K_g = n(I + Ae_g^2) = ?$ inch^4

assume, $\left(\frac{K_g}{12\,Lt_s^3}\right)^{0.1} = 1.05$

= approximate equations for distribution factors (see Figure) =

distribution **f**actor for **m**oment (α) for **i**nterior girders when multiple/two or more design lane is loaded ($_{i,m}$) [subscript 'i' for interior girder & 'm' for moment]

$$\alpha_{i,m} = \left[0.075 + \left(\frac{S_g}{9.5}\right)^{0.6}\left(\frac{S_g}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}\right] = ____?____ \text{ lanes / girder}$$

distribution **f**actor for **s**hear (α) for **i**nterior girders when multiple/two or more design lane is loaded ($_{i,v}$) [subscript 'i' for interior girder & 'v' for shear]

$$\alpha_{i,v} = 0.20 + \left(\frac{S_g}{12}\right) - \left(\frac{S_g}{35}\right)^2 = ____?____ \text{ lanes / girder}$$

Live Load Moment at any section $M_{LL} = (\alpha_{i,m}) \times [(1+0.33)\{M_{truck} \text{ or } M_{tandem}\} + M_{lane}] = ?$

Live Load Shear at any section $V_{LL} = (\alpha_{i,v}) \times [(1+0.33)\{V_{truck} \text{ or } V_{tandem}\} + V_{lane}] = ?$

Design Moment at any section, $M_{design} = 1.25M_{self\text{-}weight} + 1.50M_{wearing} + 1.75M_{LL}$

Design Shear at any section, $V_{design} = 1.25V_{self\text{-}weight} + 1.50V_{wearing} + 1.75V_{LL}$

'L' for use in Live Load Distribution Factor Equations

Force Effect	L (feet)
Positive Moment	The length of the span for which moment is being calculated
Negative Moment –near interior supports of continuous spans from point of contra-flexure (inflection point) to point of contra-flexure (inflection point) under uniform load on all spans	The average length of the adjacent two spans
Negative Moment –other than near interior supports of continuous spans	The length of the span for which moment is being calculated
Shear	The length of the span for which shear is being calculated
Exterior Reaction (abutment)	The length of the exterior span
Interior Reaction of Continuous Spans (pier)	The average length of the adjacent two spans

---

Prepare similar tow tables for exterior girder

$$\alpha_{e,m} = \left[0.77 + \frac{(b_w/2)}{9.1}\right]\alpha_{i,m} \text{ and } \alpha_{e,v} = \left[0.60 + \frac{(b_w/2)}{10}\right]\alpha_{i,v}, \text{ where } b_w \text{ in feet unit}$$

$\alpha_{e,m}$ = distribution factor for moment (α) for exterior girders when multiple/two or more design lane is loaded ($_{e,m}$) [subscript 'e' for exterior girder & 'm' for moment]

Where, $\alpha_{e,v}$ = distribution factor for shear (α) for exterior girders when multiple/two or more design lane is loaded ($_{e,v}$) [subscript 'e' for exterior girder & 'v' for shear]

Distribution Factor for Moment (interior girder, $\alpha_{i,m}$ & exterior girder, $\alpha_{e,m}$)

S_g = 7.50 feet, t_s = 7.50 inch, b_w = 18 inch = 1.50 feet & assume $\left(\frac{K_g}{12\,Lt_s^3}\right)^{0.1} = 1.05$

Section/Location	L (feet)	$\alpha_{i,m}$ (lanes/girder)	$\alpha_{e,m}$ (lanes/girder)
1-1	L_1=52.50′	0.692408048	0.590220794
2-2	L_1=52.50′	0.692408048	0.590220794
3-3	L_1=52.50′	0.692408048	0.590220794
4-4	-	-	-
5-5	½×(L_c+L_1)=34.125′	0.747960563	0.637574735
6-6	½×(L_c+L_1)=34.125′	0.747960563	0.637574735
7-7	½×(L_c+L_1)=34.125′	0.747960563	0.637574735
8-8	½×(L_c+L_E)=37.875′	0.734073149	0.625736859
9-9	½×(L_c+L_E)=37.875′	0.734073149	0.625736859
10-10	½×(L_c+L_E)=37.875′	0.734073149	0.625736859
11-11	L_E=60′	0.676137605	0.576351583
12-12	L_E=60′	0.676137605	0.576351583
13-13	L_E=60′	0.676137605	0.576351583
14-14	-	-	-

Distribution Factor for Shear (interior girder, $\alpha_{i,v}$ & exterior girder, $\alpha_{e,v}$)

S_g = 7.50 feet, b_w = 18 inch = 1.50 feet

Section/Location	$\alpha_{i,v}$ (lanes/girder)	$\alpha_{e,v}$ (lanes/girder)
1-1 to 14-14	0.779081632	0.525880102

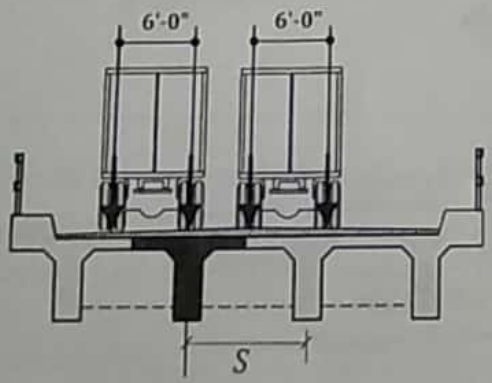


Figure 32(a): Interior Girder: Live Load Multiplier/Distribution Factor

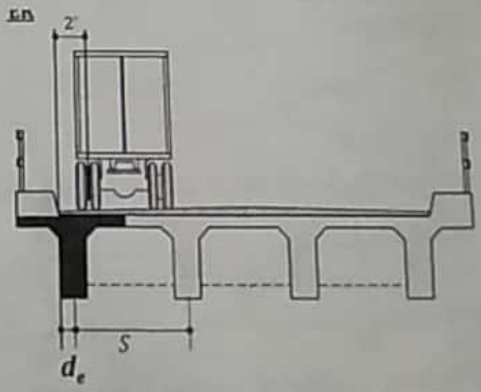


Figure 32(b): Exterior Girder: Live Load Multiplier/Distribution Factor

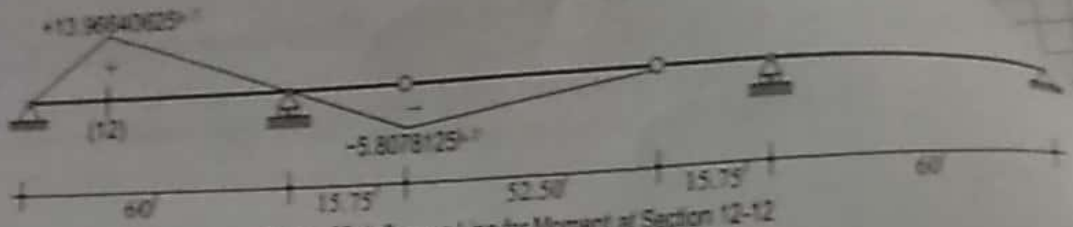


Figure 33: Influence Line for Moment at Section 12-12

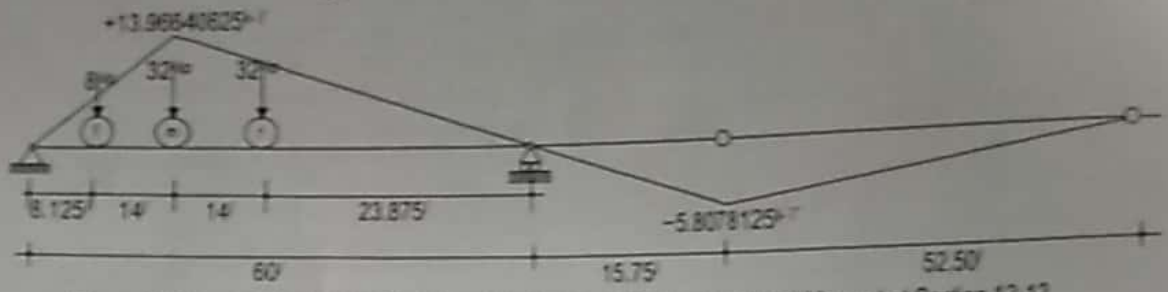


Figure 34: Placement of HL-93 Truck Load for Maximum Positive Live Load Moment at Section 12-12

$$+M\,(\text{Truck}) = \frac{+13.96640625^{k/'} \times 32^{k} + \dfrac{+13.96640625^{k/'}}{37.875'}(32^{k} \times 23.875') + \dfrac{+13.96640625^{k/'}}{22.125'}(8^{k} \times 8.125')}{} = 769.68125\ \text{kip-ft}$$

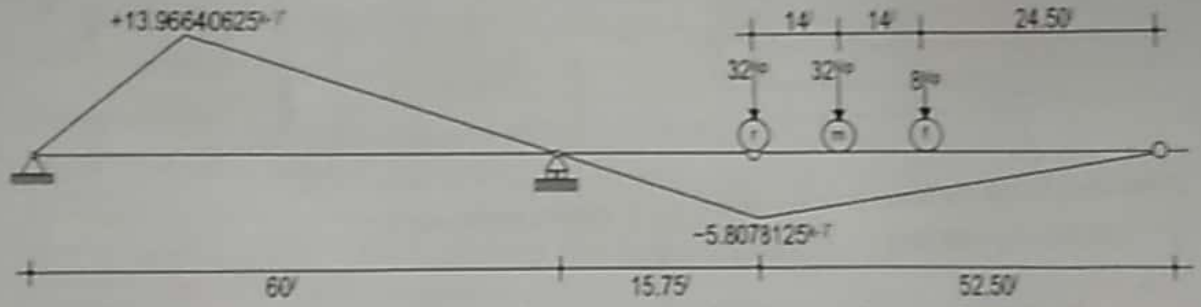


Figure 35: Placement of HL-93 Truck Load for Maximum Negative Live Load Moment at Section 12-12

$$-M\,(\text{Truck}) = \frac{-5.8078125^{k/'}}{52.50'}(32^{k} \times 52.50' + 32^{k} \times 38.50' + 8^{k} \times 24.50') = -343.8225\ \text{kip-ft}$$

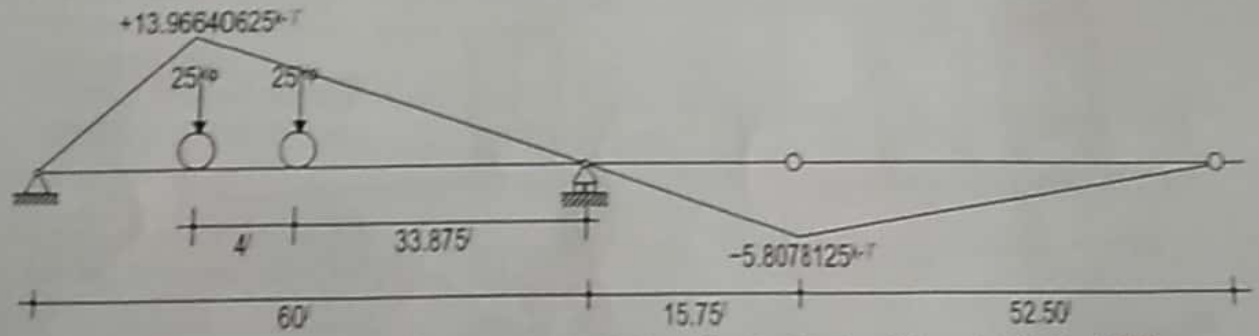


Figure 36: Placement of HL-93 Tandem Load for Maximum Positive Live Load Moment at Section 12-12

$$+M\,(\text{Tandem}) = \frac{+13.96640625^{k/'}}{37.875'}(25^{k} \times 37.875' + 25^{k} \times 33.875') = +661.4453125\ \text{kip-ft}$$

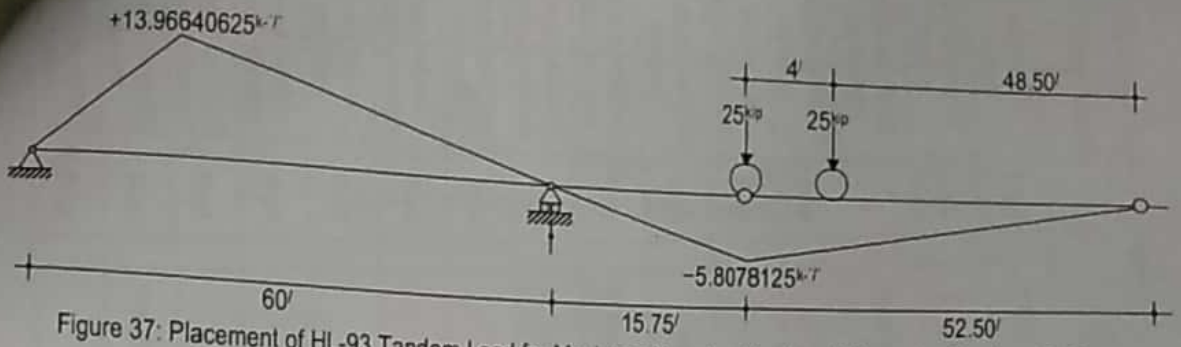


Figure 37: Placement of HL-93 Tandem Load for Maximum Negative Live Load Moment at Section 12-12

$$-M\,(\text{Tandem}) = \frac{-5.8078125^{k\text{-}ft}}{52.50^{ft}}\left(25^{k} \times 52.50^{ft} + 25^{k} \times 48.50^{ft}\right) = -279.328125\ \text{kip-ft}$$

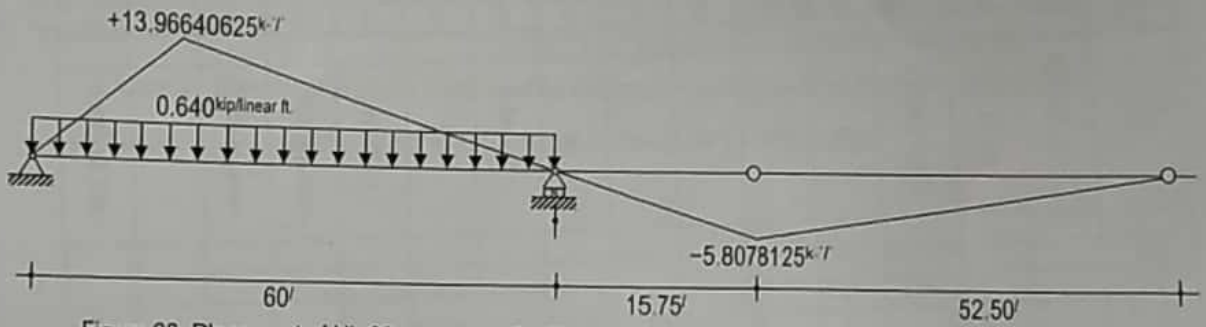


Figure 38: Placement of HL-93 Lane Load for Maximum Positive Live Load Moment at Section 12-12

$$+M\,(\text{Lane}) = \frac{1}{2} \times \left(+13.96640625^{k\text{-}ft}\right) \times \left(0.640^{k/ft} \times 60^{ft}\right) = +268.155\ \text{kip-ft}$$

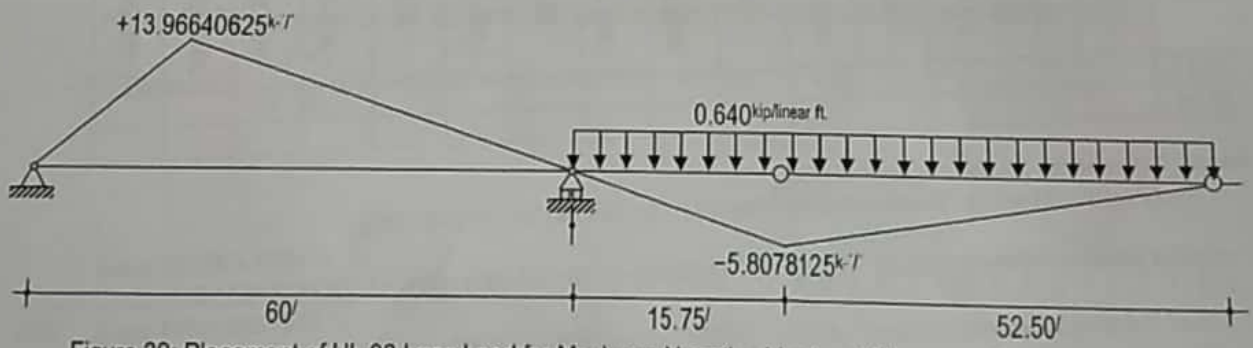


Figure 39: Placement of HL-93 Lane Load for Maximum Negative Live Load Moment at Section 12-12

$$-M\,(\text{Lane}) = \frac{1}{2} \times \left(-5.8078125^{k\text{-}ft}\right) \times \left[0.640^{k/ft} \times \left(15.75^{ft} + 52.50^{ft}\right)\right] = -126.842625\ \text{kip-ft}$$

Table 1: Combination of **Moment**: **Interior** Girder

Location/Section		1-1	2-2	3-3	4-4	5-5	6-6	7-7	8-8	9-9	10-10	11-11	12-12	13-13	14-14
Self-weight moment	kip-ft		+456.09375		0		−530.392929						+327.1555042		
Wearing course moment	kip-ft		+68.90625		0		−74.41875						+49.68013185		
Distribution factor for moment	g_{I-m}		0.692408048		0		0.747960563						0.676137605		
Truck load moment (positive)	kip-ft		+616		0		0						+769.68125		
Tandem load moment (positive)	kip-ft		+550		0		0						+661.4453125		
Lane load moment (positive)	kip-ft		+196		0		0						+268.155		
Combined positive moment (truck)	kip-ft		**1903.7056377**		0		0						**2012.0118092**		
Combined positive moment (tandem)	kip-ft		**1797.3413754**		0		0						**1841.6798021**		
Truck load moment (negative)	kip-ft		0		0		−621.60						−343.8225		
Tandem load moment (negative)	kip-ft		0		0		−505						−279.328125		
Lane load moment (negative)	kip-ft		0		0		−211.68						−126.842625		
Combined negative moment (truck)	kip-ft		**0**		0		**−807.8413703**						**−207.6977934**		
Combined negative moment (tandem)	kip-ft		**0**		0		**−604.854971**						**−106.2023328**		
Design positive moment	kip-ft		**1903.7056377**		0		**0**						**2012.0118092**		
Design negative moment	kip-ft		**0**		0		**−807.8413703**						**−207.6977934**		

Sample Calculation: Moment at section (12)-(12):

Live load Positive Moment (Truck) = 0.676137605×[1.33(+769.68125)+(+268.155)]

= +873.4555607 kip-ft

Combined Positive Moment (Truck) = 1.25(+327.1555042) +1.50(+49.68013185) +1.75(+873.4555607)

= **+2012.0118092** kip-ft

Live load Positive Moment (Tandem) = 0.676137605×[1.33×(+661.4453125) +(+268.155)]

= +776.1229852 kip-ft

Combined Positive Moment (Tandem)

= 1.25(+327.1555042)+1.50(+49.68013185) +1.75(+776.1229852) = **+1841.6798021** kip-ft

Live load Negative Moment (Truck) = 0.676137605×[1.33×(−343.8225)+(−126.842625)]

= −394.9499265 kip-ft

Combined Negative Moment (Truck) = 1.25(+327.1555042)+1.50(+49.68013185) +1.75(−394.9499265)

= **−207.6977934** kip-ft

Live load Negative Moment (Tandem) = 0.676137605×[1.33×(−279.328125)+(−126.842625)]

= −336.9525204 kip-ft

Combined Negative Moment (Tandem)

= 1.25(+327.1555042)+1.50(+49.68013185) +1.75(−336.9525204) = **−106.2023328** kip-ft

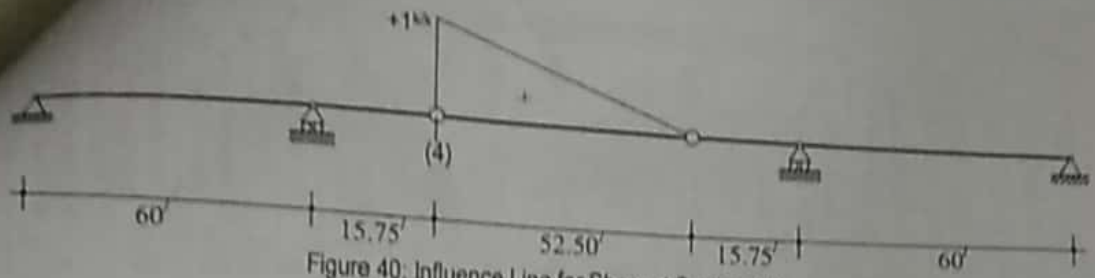


Figure 40: Influence Line for Shear at Section 4-4

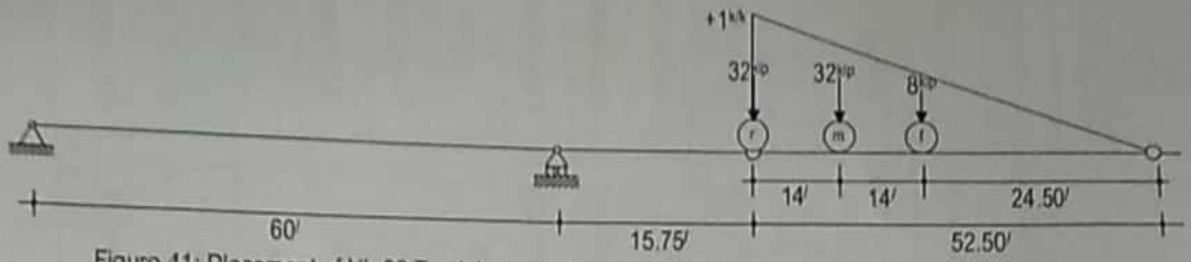


Figure 41: Placement of HL-93 Truck Load for Maximum Positive Live Load Shear at Section 12-12

$$+V\,(\text{Truck}) = \frac{+1^{k/k}}{52.50'}(32^{k} \times 52.50' + 32^{k} \times 38.50' + 8^{k} \times 24.50') = +59.20\ \text{kip}$$

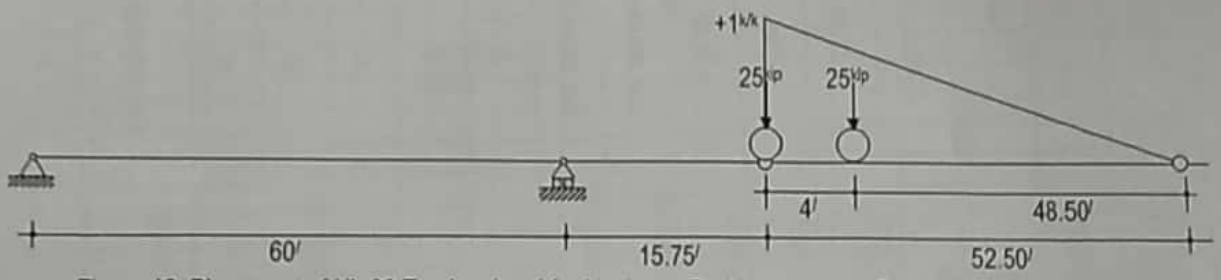


Figure 42: Placement of HL-93 Tandem Load for Maximum Positive Live Load Shear at Section 12-12

$$+V\,(\text{Tandem}) = \frac{+1^{k/k}}{52.50'}(25^{k} \times 52.50' + 25^{k} \times 48.50') = +48.0952381\ \text{kip}$$

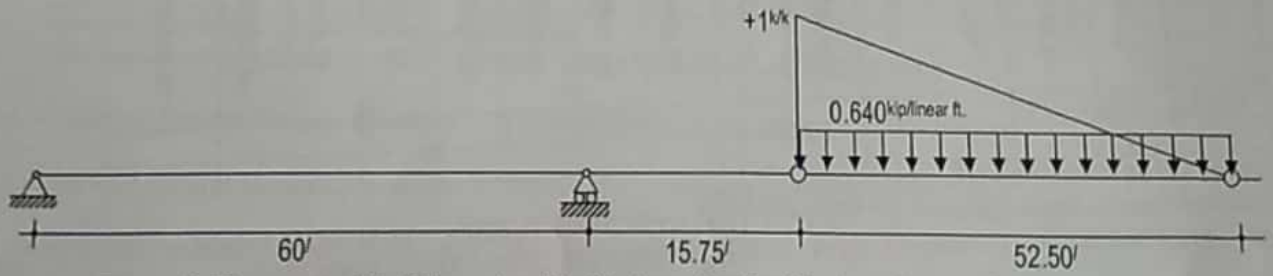


Figure 43: Placement of HL-93 Lane Load for Maximum Positive Live Load Shear at Section 12-12

$$+V\,(\text{Lane}) = \frac{1}{2} \times (+1^{k/k}) \times (0.640^{k/'} \times 52.50') = +16.80\ \text{kip}$$

There is no negative zone in influence line for shear at (4)-(4)

$-V\,(\text{Truck}) = 0$ kip

$-V\,(\text{Tandem}) = 0$ kip

$-V\,(\text{Lane}) = 0$ kip

Table 2: Combination of **Shear**: **Interior** Girder

Location/Section	1-1	2-2	3-3	4-4	5-5	6-6	7-7(R)	7-7(L)	8-8	9-9	10-10	11-11	12-12	13-13	14-14
Self-weight shear (kip)	−1.18479492	+13.4296875		+41.8828125		+60.19552436							−1.18479492		
Wearing course shear (kip)	−0.243632812	+19.6875		+5.90625		+3.54375							−0.243632812		
Distribution factor for shear (g_{I-E})	0.779081632	0.779081632		0.779081632		0.779081632							0.779081632		
Truck load shear (positive) (kip)	+34.25	+35.20		+59.20		+67.20							+34.25		
Tandem load shear (positive) (kip)	+29.89583333	+31.42857143		+48.0952381		+50							+29.89583333		
Lane load shear (positive) (kip)	+7.65075	+7.46666667		+16.80		+23.52							+7.65075		
Combined positive shear (truck) (kip)	**70.6904881**	**120.3269593**		**191.4659905**		**234.4816303**							**70.6904881**		
Combined positive shear (tandem) (kip)	**62.7950233**	**113.4881808**		**171.3295870**		**203.2926553**							**62.7950233**		
Truck load shear (negative) (kip)	−16.13333333	−12.80		0		0							−16.13333333		
Tandem load shear (negative) (kip)	−16.77083333	−14.76190476		0		0							−16.77083333		
Lane load shear (negative) (kip)	−8.34375	−1.86666667		0		0							−8.34375		
Combined negative shear (truck) (kip)	−42.4770270	**0**		**0**		**0**							−42.4770270		
Combined negative shear (tandem) (kip)	−43.6330137	**0**		**0**		**0**							−43.6330137		
Design shear (absolute maximum of column 8, 9, 13 & 14) (kip)	**70.6904881**	**120.3269593**		**191.4659905**		**234.4816303**							**70.6904881**		

Sample Calculation: Shear at section (4)-(4):

Live load Positive Shear (Truck) = 0.779081632×[1.33×(+59.20)+(+ 16.80)] = +74.430342795 kip

Combined Positive Shear (Truck) = 1.25(+41.8828125)+1.50(+5.90625) + 1.75(+74.430342795)

= **+191.4659905** kip

Live load Positive Shear (Tandem) = 0.779081632×[1.33×(+48.0952381)+(+16.80)] = +62.923826483 kip

Combined Positive Shear (Tandem) = 1.25(+41.8828125)+1.50(+5.90625)+1.75(+62.923826483)

= **+171.3295870** kip

Live load Negative Shear (Truck) = 0.779081632×[1.33×(0)+(0)] = 0 kip

Combined Negative Shear (Truck) = 1.25(+41.8828125)+1.50(+5.90625)+1.75(0)

= +61.212890625 kip = **0 kip***

Live load Negative Shear (Tandem) = 0.779081632×[1.33×(0) + (0)] = 0 kip

Combined Negative Shear (Tandem) = 1.25(+41.8828125)+1.50(+5.90625)+1.75(0)

= +61.212890625 kip = **0 kip*** [*no negative shear found]

Table 3: Combination of **Moment: Exterior** Girder

Location/Section	Self-weight moment	Wearing course moment	Distribution factor for moment	Truck load moment (positive)	Tandem load moment (positive)	Lane load moment (positive)	Combined positive moment (truck)	Combined positive moment (tandem)	Truck load moment (negative)	Tandem load moment (negative)	Lane load moment (negative)	Combined negative moment (truck)	Combined negative moment (tandem)	Design positive moment	Design negative moment
1-1															
2-2	+665.6015625	+41.34375	0.590220794	+616	+550	+196	**1942.68647166**	**1852.01970439**	0	0	0	**0**	**0**	**1942.6864716**	**0**
3-3															
4-4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5-5															
6-6	−743.2738665	−44.65125	0.637574735	0	0	0	**0**	**0**	−621.60	−505	−211.68	**−2154.67894261**	**−1981.64976679**	**0**	**−2154.678942**
7-7															
8-8															
9-9															
10-10															
11-11															
12-12	+479.0465891	+29.80807911	0.576351583	+769.68125	+661.4453125	+268.155	**1946.48089126**	**1801.28689352**	−343.8225	−279.328125	−126.842625	**0**	**0**	**1946.4808912**	**0**
13-13															
14-14															

Sample Calculation: Moment at section (12)-(12):

Live load Positive Moment (Truck) = 0.576351583×[1.33(+769.68125) +(+268.155)]

= +744.5488778 kip-ft

Combined Positive Moment (Truck) = 1.25(+479.0465891) +1.50(+29.80807911) +1.75(+744.5488778)

= **+1946.48089126** kip-ft

Live load Positive Moment (Tandem) = 0.576351583×[1.33×(+661.4453125) +(+268.155)]

= +661.5808791 kip-ft

Combined Positive Moment (Tandem)

= 1.25(+479.0465891)+1.50(+29.80807911) +1.75(+661.5808791) = **+1801.28689352** kip-ft

Live load Negative Moment (Truck) = 0.576351583×[1.33×(−343.8225)+(−126.842625)]

= −336.66226176 kip-ft

Combined Negative Moment (Truck) = 1.25(+479.0465891)+1.50(+29.80807911) +1.75(−336.66226176)

= **+54.36139696** kip-ft = **0** kip-ft*

Live load Negative Moment (Tandem) = 0.576351583×[1.33×(−279.328125)+(−126.842625)]

= −287.22425305 kip-ft

Combined Negative Moment (Tandem)

= 1.25(+479.0465891)+1.50(+29.80807911) +1.75(−287.22425305) = **+140.8779122** kip-ft = = **0** kip-ft*

[*no negative moment found]

Table 4: Combination of **Shear: Exterior** Girder

Location/Section	1-1	2-2	3-3	4-4	5-5	6-6	7-7(R)	7-7(L)	8-8	9-9	10-10	11-11	12-12	13-13	14-14
Self-weight shear (kip)	−2.139465825	+19.21640625		+58.44609375		+84.18099311							−2.139465825		
Wearing course shear (kip)	−0.146179687	+11.8125		+3.54375		+2.12625							−0.146179687		
Distribution factor for shear $\alpha_{L,V}$	0.525880102	0.525880102		0.525880102		0.525880102							0.525880102		
Truck load shear (positive) (kip)	+34.25	+35.20		+59.20		+67.20							+34.25		
Tandem load shear (positive) (kip)	+29.89583333	+31.42857143		+48.0952381		+50							+29.89583333		
Lane load shear (positive) (kip)	+7.65075	+7.46666667		+16.80		+23.52							+7.65075		
Combined positive shear (truck) (kip)	**46.06882663**	**91.69506281**		**166.29408468**		**212.31269638**							**46.06882663**		
Combined positive shear (tandem) (kip)	**40.73938785**	**87.07888728**		**152.70201228**		**191.26013826**							**40.73938785**		
Truck load shear (negative) (kip)	−16.13333333	−12.80		0		0							−16.13333333		
Tandem load shear (negative) (kip)	−16.77083333	−14.76190476		0		0							−16.77083333		
Lane load shear (negative) (kip)	−8.34375	−1.86666667		0		0							−8.34375		
Combined negative shear (truck) (kip)	**−30.319246108**	**24.354362811**		**78.373242188**		**108.415616388**							**−30.319246108**		
Combined negative shear (tandem) (kip)	**−31.099537143**	**21.953018974**		**78.373242188**		**108.415616388**							**−31.099537143**		
Design shear (absolute maximum of column 8, 9, 10 & 11) (kip)	**46.06882663**	**91.69506281**		**166.29408468**		**212.31269638**							**46.06882663**		

Sample Calculation: Shear at section (4)-(4):

Live load Positive Shear (Truck) = 0.525880102×[1.33×(+59.20)+(+ 16.80)] = +50.24048142 kip

Combined Positive Shear (Truck) = 1.25(+58.44609375)+1.50(+3.54375) + 1.75(+50.24048142)

= **+166.2940847** kip

Live load Positive Shear (Tandem) = 0.525880102×[1.33×(+48.0952381)+(+16.80)] = +42.47358291 kip

Combined Positive Shear (Tandem) = 1.25(+58.44609375)+1.50(+3.54375)+1.75(+42.47358291)

= **+152.7020123** kip

Live load Negative Shear (Truck) = 0.525880102×[1.33×(0)+(0)] = 0 kip

Combined Negative Shear (Truck) = 1.25(+58.44609375)+1.50(+3.54375)+1.75(0)

= +78.37324219 kip = **0 kip***

Live load Negative Shear (Tandem) = 0.525880102×[1.33×(0) + (0)] = 0 kip

Combined Negative Shear (Tandem) = 1.25(+58.44609375)+1.50(+3.54375)+1.75(0)

= +78.37324219 kip = **0 kip*** [*no negative shear found]

Design on Interior Girder

Design for Moment [φ=0.90 for moment]

$\alpha = 0.72 - \frac{0.04}{1000}(f_c' - 4000)$; for 4000 psi $\le f_c' \le$ 8000 psi

$\beta = 0.425 - \frac{0.025}{1000}(f_c' - 4000)$, for 4000 psi $\le f_c' \le$ 8000 psi

for $f_c' <$ 4000 psi (say 3500 psi, 3000 psi, 2500 psi, ... & so on); use α & β value for $f_c' =$ 4000 psi [i.e., $\alpha = 0.72$ & $\beta = 0.425$]

for $f_c' >$ 8000 psi (say 8500 psi, 9000 psi, 9500 psi, ... & so on); use α & β value for $f_c' =$ 8000 psi [i.e., $\alpha = 0.56$ & $\beta = 0.325$]

Concrete Stress Block Parameters

	f_c'				
	≤ 4000 psi	5000 psi	6000 psi	7000 psi	≥ 8000 psi
α	0.72	0.68	0.64	0.60	0.56
β	0.425	0.40	0.375	0.35	0.325
$\beta_1 = 2\beta$	0.85	0.80	0.75	0.70	0.65
β/α	0.59	0.59	0.59	0.59	0.59
$\gamma = \alpha/\beta_1$	0.85	0.85	0.85	0.85	0.85

Properties of Reinforcing Bar (ASTM Standard size reinforcing bars (ASTM 6615, 6616, 6617))

Bar No.	Bar diameter		Cross-sectional area	
	(inch)	(mm)	($inch^2$)	(cm^2)
#3	0.375 inch	10 mm	0.11	0.71
#4	0.500 inch	12 mm	0.20	1.29
#5	0.625 inch	16 mm	0.31	2.00
#6	0.750 inch	19 mm	0.44	2.84
#7	0.875 inch	22 mm	0.60	3.87
#8	1.000 inch	25 mm	0.79	5.10
#9	1.128 inch	28 mm	1.00	6.45
#10	1.270 inch	32 mm	1.27	8.19
#11	1.410 inch	36 mm	1.56	10.06
#14	1.693 inch	43 mm	2.25	14.52
#18	2.257 inch	57 mm	4.00	25.81

[The nominal dimensions of a deformed bar are equivalent to those of a plain round bar having the same weight per feet as the deformed bar]

[Bar numbers are based on the number of eights of an inch included in the nominal diameter of the bars]

■ For positive moment ($+M_u$), the girder section may behave as T-beam or rectangular beam. You must check it before renforcement calculation. [if $a > t_{slab}$; use T-beam / L-beam formula & if $a \le t_{slab}$; use rectangular beam formula (use $b = b_{effective}$ in the formula)]

let, $a_{initial} = t$ (deck slab thickness) inch

$$A_{s(initial)} = \frac{M_u}{\varphi f_y\left(d - \frac{a_{initial}(=t)}{2}\right)} = \text{inch}^2$$

$$a_{revised} = \frac{A_{s(initial)} f_y}{0.85 f_c' b_{effective}} = \text{inch}$$

If $a_{revised} > t$; T-beam design
If $a_{revised} \le t$; rectangular beam design [in singly reinforced (under-reinforced) rectangular beam formula, 'b' will be replaced by '$b_{effective}$' and other parameter will remain same.]
- For negative moment ($-M_u$), the girder section must be a rectangular beam (use $b = b_w$ in the formula)

Design for Shear (Stirrup Design) [φ=0.90 for shear]

If $V_u > 0.25\varphi\sqrt{f_c'}\,b_w d$, girder dimension ($b_w$ & d) is not OK & need to be increased.

If $V_u \le 0.25\varphi\sqrt{f_c'}\,b_w d$; girder dimension ($b_w$ & d) is OK & need not to be increased.

Design shear of concrete, $\varphi V_c = \varphi(0.0316)\beta\sqrt{f_c'}\,b_w d$ (f_c' in ksi, b_w & d in inch, V_c in kip & β=2.0)

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear (For non-prestressed concrete sections not subjected to axial tension and containing at least the minimum amount of transverse reinforcement, or having an overall depth of less than 16.0 inch, β =2.0)

s = tirrup spacing (always rounded to 0.50 inch (lower rounding)) & $s \le s_{maximum}$

$$s_{maximum} = \text{minimum of } [\frac{A_v f_y}{0.0316\sqrt{f_c'}\,b_w}, 0.80d \text{ \& } 24 \text{ inch}] \text{ if } V_u < 0.125\varphi f_c' b_w d$$

$$s_{maximum} = \text{minimum of } [\frac{A_v f_y}{0.0316\sqrt{f_c'}\,b_w}, 0.40d \text{ \& } 12 \text{ inch}] \text{ if } V_u > 0.125\varphi f_c' b_w d$$

Stirrup is required when $V_u > 0.50\varphi V_c$

$$s_{required} = \frac{\varphi A_v f_y d}{V_u - \varphi V_c} = ? \text{ inch}$$

$s_{required} < s_{maximum}$

Stirrup is not required when $V_u < 0.50\varphi V_c$ (but provide a minimum amount ($A_{v(minimum)}$) / maximum spacing ($s_{(maximum)}$))

Material properties, $f_c' = 3.50$ ksi $= 3500$ psi & $f_y = 40$ ksi $= 40000$ psi

girder web width, $b_{web} = b_w = 45''$

girder flange width, $b_{effective} = S_g$ (girder c/c distance) $= 7.50$ feet $= 90''$

girder depth, H (at section 2-2) = 45 inch

girder depth, H (at section 6-6) $= y + \frac{\Delta y}{\Delta x} \times (x) = H + \frac{2H - H}{L_c} \times (L_c/3 + L_c/3)$

$$= 45'' + \frac{90'' - 45''}{15.75'} \times (5.25' + 5.25') = 75''$$

[distance between section 4-4 & 6-6 = $(L_c/3 + L_c/3) = 5.25' + 5.25'$]

girder depth, H (at section 12-12) = 45 inch

►Design of **Interior Girder** for **Moment** at **Section (2)-(2)**

Design Positive Moment, $+M_u = 1903.7056377$ kip-ft & Design Negative Moment, $-M_u = 0$ kip-ft

---

for positive moment, check for T-beam or rectangular beam

$H = 45''$; assume triple layer of tension bars, $d = H - 2.50'' - 2.50'' = 45'' - 2.50'' - 2.50'' = 40''$

let, $a_{initial} = t = 7.50$ inch

$$A_{s(initial)} = \frac{1903.7056377 \times 12000 \text{ lb.-inch}}{0.9 \times 40000 \text{ psi} \times \left(40 \text{ inch} - \frac{7.50 \text{ inch}}{2}\right)} = 17.5053392 \text{ inch}^2$$

$$a_{revised} = \frac{A_{s(initial)} f_y}{0.85 f_c' b_{effective}} = \frac{17.5053392 \text{ inch}^2 \times 40000 \text{ psi}}{0.85 \times 3500 \text{ psi} \times 90 \text{ inch}} = 2.615176724 \text{ inch}$$

$a_{(initial)}$ (=2.615176724 inch) < t (= 7.50 inch); rectangular (singly-reinforced) beam design formula will be applicable.

1st trial: assume, a (initial) = 2 inch

$$A_s = \frac{M_u}{\varphi f_y\left(d - \frac{a}{2}\right)} = \frac{1903.7056377 \times 12000 \text{ lb.-inch}}{0.9 \times 40000 \text{ psi} \times \left(40 \text{ inch} - \frac{2 \text{ inch}}{2}\right)} = 16.27098836 \text{ inch}^2$$

$$a \text{ (final)} = \frac{A_s f_y}{0.85 f_c' b_{effective}} = \frac{16.27098836 \text{ inch}^2 \times 40000 \text{ psi}}{0.85 \times 3500 \text{ psi} \times 90 \text{ inch}} = 2.430773237 \text{ inch}$$; not match with a (initial) = 2 inch

[value of 'a' indicates that it's value increasing (a=2 inch → a=2.4307 inch); next trial should be given with 'a' > 2.43 inch]

2nd trial: assume, a (initial) = 2.444 inch

$$A_s = \frac{M_u}{\varphi f_y\left(d - \frac{a}{2}\right)} = \frac{1903.7056377 \times 12000 \text{ lb.-inch}}{0.9 \times 40000 \text{ psi} \times \left(40 \text{ inch} - \frac{2.444 \text{ inch}}{2}\right)} = 16.36413807 \text{ inch}^2$$

$$a \text{ (final)} = \frac{A_s f_y}{0.85 f_c' b_{effective}} = \frac{16.36413807 \text{ inch}^2 \times 40000 \text{ psi}}{0.85 \times 3500 \text{ psi} \times 90 \text{ inch}} = 2.44468916 \text{ inch}$$; matches with a (initial) = 2.444 inch (OK)

$A_{s(main)} = 16.36413807 \text{ inch}^2 = 14$ (# 9) + 2 (# 10) = 14 (1.00 inch^2) + 2 (1.27 inch^2) = 16.54 inch^2

$c = a/\beta_1 = 2.44468916 \text{ inch}/0.85 = 2.85973322 \text{ inch}$ & d = 40 inch

check, $c < \frac{3}{8}d$, => c (=2.85973322 inch) < $\frac{3}{8}$d (=$\frac{3}{8}$ ×40 inch) [tension controlled]

check for minimum steel, $A_{s(minimum)} = \frac{200}{f_y} b_w d = \frac{200}{40000 \text{ psi}} \times 18'' \times 40'' = 3.60 \text{ inch}^2 < A_{s(main)}$; OK

Design of **Interior Girder** for **Shear** at **Section (2)-(2)**

Design Shear, V_u = 120.3269593 kip; d = 40 inch

V_u (=120.3269593 kip) < 0.25φ $f_c' b_w d$ (=0.25×0.90×3.50 ksi×18 inch×40 inch = 567 kip); concrete dimension is OK & need not to be revised.

$$\varphi V_c = \varphi(0.0316)\beta\sqrt{f_c'} b_w d = 0.90 \times 0.0316 \times 2.0 \times \sqrt{3.50 \text{ ksi}} \times 18 \text{ inch} \times 40 \text{ inch} = 76.61716998 \text{ kip}$$

V_u (=120.3269593 kip) > ½φV_c (=38.30858499 kip); Stirrup required

#3 bar & 4-legged stirrup, $A_v = 4A_s = 4 \times 0.11 \text{ inch}^2 = 0.44 \text{ inch}^2$

$$s_{required} = \frac{\varphi A_v f_y d}{V_u - \varphi V_c} = \frac{0.90 \times 0.44 \text{ inch}^2 \times 40 \text{ ksi} \times 40 \text{ inch}}{120.3269593 \text{ kip} - 76.6171699 \text{ kip}} = 14.49560865 \text{ inch}$$

check for maximum spacing

V_u (=120.3269593 kip) < 0.125φ $f_c' b_w d$ (=0.125×0.90×3.50 ksi×18 inch×40 inch = 283.50 kip)

$$s_{maximum} = \text{minimum of } [\frac{A_v f_y}{0.0316\sqrt{f_c'} b_w}, 0.80d \text{ \& } 24 \text{ inch}]$$

$$s_{maximum} = \text{minimum of } [\frac{0.44 \text{ inch}^2 \times 40 \text{ ksi}}{0.0316\sqrt{3.50 \text{ ksi}} \times 18 \text{ inch}}, 0.80 \times 40 \text{ inch \& } 24 \text{ inch}]$$

$s_{maximum}$ = minimum of [16.53937362 inch, 32 inch & 24 inch] = 16.53937362 inch

$s_{required}$ (=14.49560865 inch) < $s_{maximum}$ (=16.53937362 inch); OK

$s_{provided}$ at section (2)-(2)= #3-4L stirrup @ 14 inch (lower rounding to 0.50 inch)

Skin Steel, Link/Ties for Skin Steel & **Haunch Steel** for **Interior Girder** at **Section (2)-(2)**

$d_l = d = 40$ inch, $A_s = 16.54 \text{ inch}^2$

$A_{sk(minimum)} \le A_{sk} \le A_{sk(maximum)}$

$0.012(d_i - 30) \le A_{sk} \le \frac{A_s}{4}$ [unit of A_{sk} : inch^2 per foot height per face of girder]

$A_{sk\,(minimum)} = 0.012(d_i - 30) = 0.012\,(40'' - 30'') = 0.12$ inch^2 per foot height per face of girder

$A_{sk\,(maximum)} = \frac{A_s}{4} = \frac{16.54\text{ inch}^2}{4} = 4.135$ inch^2 per foot height per face of girder

maximum spacing permitted ($s_{maximum}$) [along the height of girder[for skin reinforcement (A_{sk}) = minimum of [$\frac{d_i}{6}$ & 12"]= minimum of [$\frac{40''}{6} = 6.6667''$ & 12"] = $6.6667'' = 6.50''$

(lower rounding to 0.50")

Skin reinforcement should be provided to a height of $\frac{d_i}{2} = \frac{40''}{2} = 20$ inch (minimum) from nearest flexural tensile reinforcement.

let, we provide # 7 bar as skin steel @ 5" c/c (≤6.5") (along girder height per girder face)

$A_{sk\,(provided)} = \frac{12 \times A_{s\,(\#7\,bar)}}{s_{provided}} = \frac{12 \times 0.60\text{ inch}^2}{5\text{ inch}} = 1.44$ inch^2 per foot height per face of girder (falls within the range of $A_{sk\,(minimum)}$ & $A_{sk\,(maximum)}$ as calculated above; OK)

Haunch steel: #3 or #4 bar @ 6" to 9" c/c along the length of the girder. (provide #3 @ 6" c/c)

Link/Tie for skin reinforcement: use #3 or #4 bar. Vertical & Longitudinal spacing should not exceed 24 inch. (provide #3 @ 15" c/c) [see figure 44]

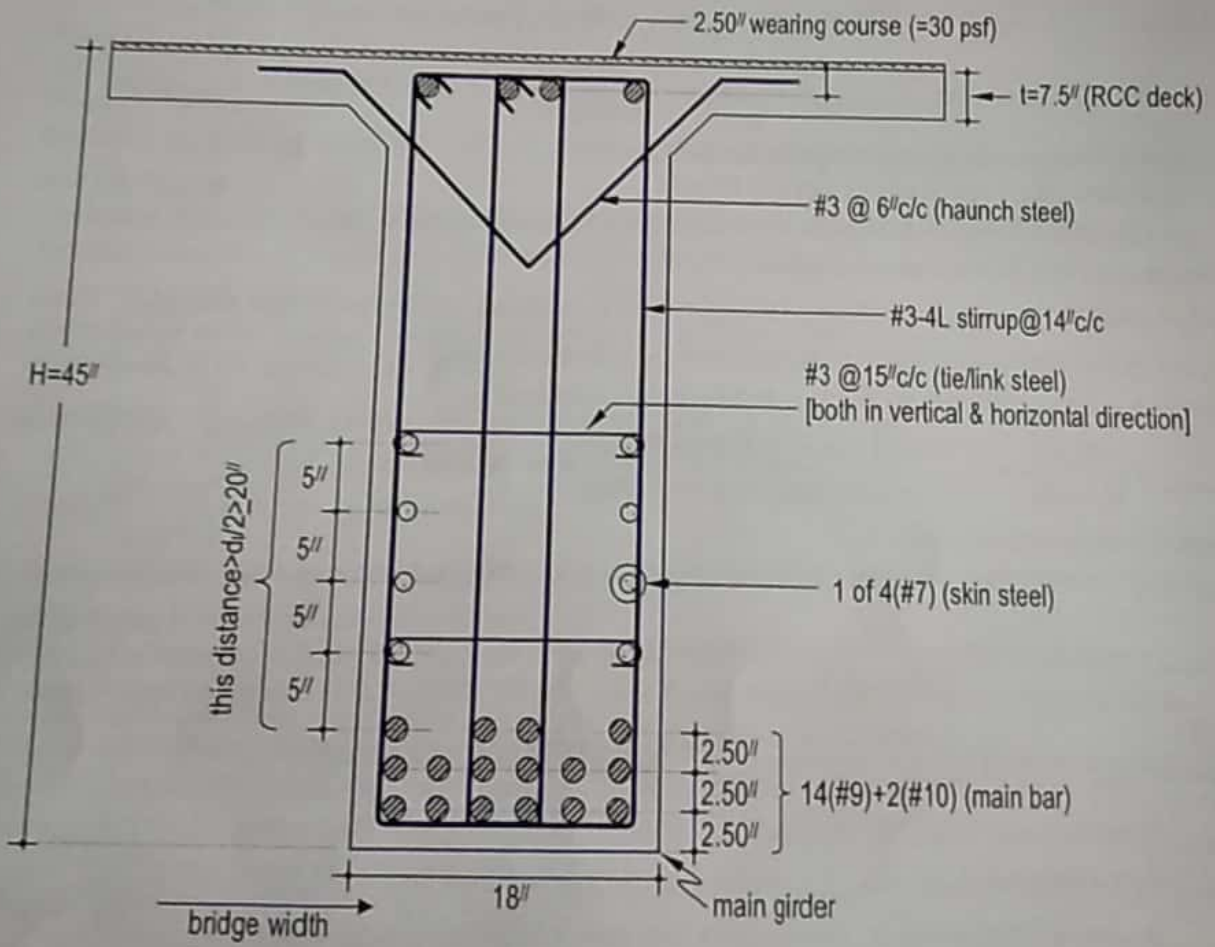


Figure 44: Interior Girder at Section (2)-(2) (reinforcement details)

Design of **Interior Girder** for **Moment** at **Section (6)-(6)**

Design Positive Moment, $+M_u = 0$ kip-ft & Design Negative Moment, $-M_u = -807.8413703$ kip-ft

for negative moment, always rectangular beam

$H = 75''$; assume single layer of tension bars, $d = H - 2.50'' = 75'' - 2.50'' = 72.50''$

1st trial: assume, a (initial) = 3 inch

$$A_s = \frac{M_u}{\varphi f_y\left(d - \frac{a}{2}\right)} = \frac{807.8413703 \times 12000 \text{ lb.-inch}}{0.9 \times 40000 \text{ psi} \times \left(72.50 \text{ inch} - \frac{3 \text{ inch}}{2}\right)} = 3.79268249 \text{ inch}^2$$

$$a \text{ (final)} = \frac{A_s f_y}{0.85 f_c' b_{web}} = \frac{3.79268249 \text{ inch}^2 \times 40000 \text{ psi}}{0.85 \times 3500 \text{ psi} \times 18 \text{ inch}} = 2.833002793 \text{ inch; not match with a (initial) = 3 inch}$$

[value of 'a' indicates that it's value decreasing (a=3 inch → a=2.833inch); next trial should be given with 'a' < 2.833 inch]

2nd trial: assume, a (initial) = 2.829 inch

$$A_s = \frac{M_u}{\varphi f_y\left(d - \frac{a}{2}\right)} = \frac{807.8413703 \times 12000 \text{ lb.-inch}}{0.9 \times 40000 \text{ psi} \times \left(72.50 \text{ inch} - \frac{2.829 \text{ inch}}{2}\right)} = 3.788120739 \text{ inch}^2$$

$$a \text{ (final)} = \frac{A_s f_y}{0.85 f_c' b_{web}} = \frac{3.788120739 \text{ inch}^2 \times 40000 \text{ psi}}{0.85 \times 3500 \text{ psi} \times 18 \text{ inch}} = 2.829595323 \text{ inch; matches with a (initial) = 2.829 inch (OK)}$$

$c = a/\beta_1 = 2.829595323 \text{ inch}/0.85 = 3.328935674 \text{ inch}$ & $d = 72.50$ inch

check, $c < \frac{3}{8} d$, => c (=3.328935674 inch) < $\frac{3}{8} d$ (=$\frac{3}{8} \times 72.50$ inch) [tension controlled]

check for minimum steel, $A_{s(minimum)} = \frac{200}{f_y} b_w d = \frac{200}{40000 \text{ psi}} \times 18'' \times 72.50'' = 6.525 \text{ inch}^2 < A_{s(main)}$; not OK

$A_{s(main)} = 6.525 \text{ inch}^2 = 2$ (# 10) + 4 (# 9) = 2 (1.27 inch^2) + 4 (1.00 inch^2) = 6.54 inch^2

Design of **Interior Girder** for **Shear** at **Section (6)-(6)**

Design Shear, $V_u = 234.4816303$ kip; $d = 72.50$ inch

V_u (=234.4816303 kip) < $0.25\varphi f_c' b_w d$ (=0.25×0.90×3.50 ksi×18 inch×72.50 inch = 1027.6875 kip); concrete dimension is OK & need not to be revised.

$\varphi V_c = \varphi(0.0316)\beta\sqrt{f_c'} b_w d = 0.90 \times 0.0316 \times 2.0 \times \sqrt{3.50 \text{ ksi}} \times 18 \text{ inch} \times 72.50 \text{ inch} = 138.8686206$ kip

V_u (=234.4816303 kip) > ½φV_c (=38.30858499 kip); Stirrup required

#3 bar & 4-legged stirrup, $A_v = 4A_s = 4 \times 0.11 \text{ inch}^2 = 0.44 \text{ inch}^2$

$$s_{required} = \frac{\varphi A_v f_y d}{V_u - \varphi V_c} = \frac{0.90 \times 0.44 \text{ inch}^2 \times 40 \text{ ksi} \times 40 \text{ inch}}{234.4816303 \text{ kip} - 138.8686206 \text{ kip}} = 12.0109178 \text{ inch}$$

check for maximum spacing

V_u (=234.4816303 kip) < $0.125\varphi f_c' b_w d$ (=0.125×0.90×3.50 ksi×18 inch×72.50 inch = 513.84375 kip)

$$s_{maximum} = \text{minimum of } \left[\frac{A_v f_y}{0.0316\sqrt{f_c'} b_w}, 0.80d \text{ & } 24 \text{ inch}\right]$$

$$s_{maximum} = \text{minimum of } \left[\frac{0.44 \text{ inch}^2 \times 40 \text{ ksi}}{0.0316\sqrt{3.50 \text{ ksi}} \times 18 \text{ inch}}, 0.80 \times 72.50 \text{ inch} \text{ & } 24 \text{ inch}\right]$$

$s_{maximum}$ = minimum of [16.53937362 inch, 58 inch & 24 inch] = 16.53937362 inch

$s_{required}$ (=12.0109178 inch) < $s_{maximum}$ (=16.53937362 inch); OK

$s_{provided}$ at section (6)-(6)= #3-4L stirrup @ 12 inch (lower rounding to 0.50 inch)

Skin Steel, Link/Ties for Skin Steel & **Haunch Steel** for **Interior Girder** at Section (6)-(6)

$d_i = d = 72.50$ inch, $A_s = 6.54$ inch^2

$A_{sk\,(minimum)} \le A_{sk} \le A_{sk\,(maximum)}$

$0.012(d_i - 30) \le A_{sk} \le \frac{A_s}{4}$; [unit of A_{sk} : inch^2 per foot height per face of girder]

$A_{sk\,(minimum)} = 0.012(d_i - 30) = 0.012\,(72.50'' - 30'') = 0.51$ inch^2 per foot height per face of girder

$A_{sk\,(maximum)} = \frac{A_s}{4} = \frac{6.54\text{ inch}^2}{4} = 1.635$ inch^2 per foot height per face of girder

maximum spacing permitted ($s_{maximum}$) [along the height of girder[for skin reinforcement (A_{sk}) = minimum of [$\frac{d_i}{6}$ & $12''$]= minimum of [$\frac{72.50''}{6} = 12.0833''$ & $12''$] = $12''$ (lower rounding to $0.50''$)

Skin reinforcement should be provided to a height of $\frac{d_i}{2} = \frac{72.50''}{2} = 36.25$ inch (minimum) from nearest flexural tensile reinforcement.

let, we provide # 7 bar as skin steel @ $7.50''$ c/c ($\le 12''$) (along girder height per girder face)

$A_{sk\,(provided)} = \frac{12 \times A_{s\,(\#7\,bar)}}{s_{provided}} = \frac{12 \times 0.60\text{ inch}^2}{7.50\text{ inch}} = 0.96$ inch^2 per foot height per face of girder (falls within the range of $A_{sk\,(minimum)}$ & $A_{sk\,(maximum)}$ as calculated above; OK)

Haunch steel: #3 or #4 bar @ $6''$ to $9''$ c/c along the length of the girder. (provide #3 @ $6''$ c/c)

Link/Tie for skin reinforcement: use #3 or #4 bar. Vertical & Longitudinal spacing should not exceed 24 inch. (provide #3 @ $15''$ c/c) [see figure 45]

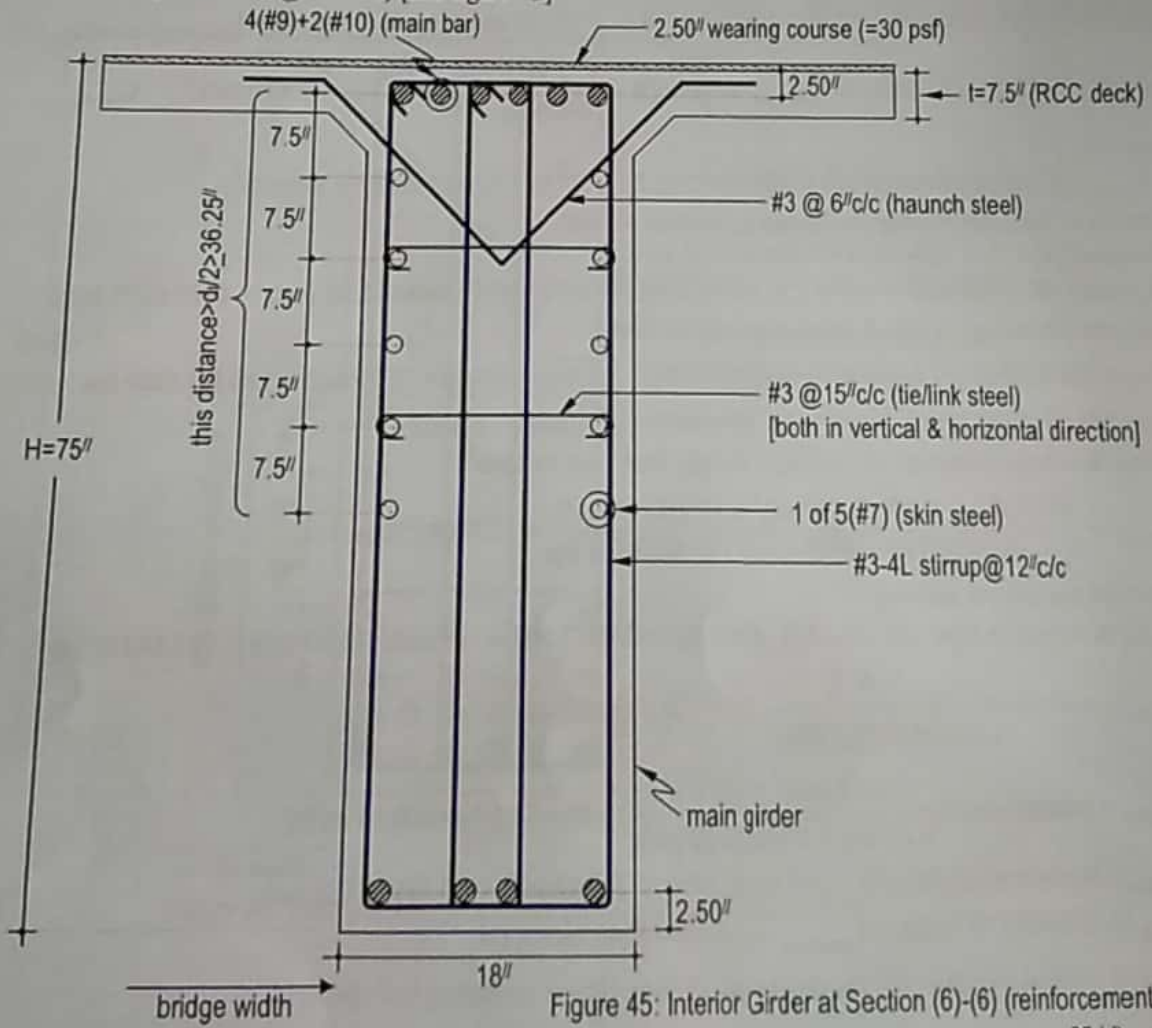


Figure 45: Interior Girder at Section (6)-(6) (reinforcement details)

Design of Interior Girder for Moment at Section (12)-(12)

Design Positive Moment, $+M_u$ = 2012.011809 kip-ft &

Design Negative Moment, $-M_u$ = −207.6977934 kip-ft

for positive moment, check for T-beam or rectangular beam

$H = 45''$; assume triple layer of tension bars, $d = H - 2.50'' - 2.50'' = 45'' - 2.50'' - 2.50'' = 40''$

let, $a_{initial} = t = 7.50$ inch

$$A_{s(initial)} = \frac{2012.011809 \times 12000 \text{ lb.-inch}}{0.9 \times 40000 \text{ psi} \times \left(40 \text{ inch} - \frac{7.50 \text{ inch}}{2}\right)} = 18.50125801 \text{ inch}^2$$

$$a_{revised} = \frac{A_{s(initial)} f_y}{0.85 f_c' b_{effective}} = \frac{18.50125801 \text{ inch}^2 \times 40000 \text{ psi}}{0.85 \times 3500 \text{ psi} \times 90 \text{ inch}} = 2.763960114 \text{ inch}$$

$a_{revised}$ (=2.763960114 inch) < t (= 7.50 inch); rectangular (singly-reinforced) beam design formula will be applicable.

1st trial: assume, a (initial) = 2.50 inch

$$A_s = \frac{M_u}{\varphi f_y \left(d - \frac{a}{2}\right)} = \frac{2012.011809 \times 12000 \text{ lb.-inch}}{0.9 \times 40000 \text{ psi} \times \left(40 \text{ inch} - \frac{2.50 \text{ inch}}{2}\right)} = 17.30762846 \text{ inch}^2$$

$$a \text{ (final)} = \frac{A_s f_y}{0.85 f_c' b_{effective}} = \frac{17.30762846 \text{ inch}^2 \times 40000 \text{ psi}}{0.85 \times 3500 \text{ psi} \times 90 \text{ inch}} = 2.585640107 \text{ inch; not match with a (initial)} = 2.50 \text{ inch}$$

[value of 'a' indicates that it's value increasing (a=2.50 inch → a=2.5856 inch); next trial should be given with 'a' > 2.5856 inch]

2nd trial: assume, a (initial) = 2.588 inch

$$A_s = \frac{M_u}{\varphi f_y \left(d - \frac{a}{2}\right)} = \frac{2012.011809 \times 12000 \text{ lb.-inch}}{0.9 \times 40000 \text{ psi} \times \left(40 \text{ inch} - \frac{2.588 \text{ inch}}{2}\right)} = 17.32730334 \text{ inch}^2$$

$$a \text{ (final)} = \frac{A_s f_y}{0.85 f_c' b_{effective}} = \frac{17.32730334 \text{ inch}^2 \times 40000 \text{ psi}}{0.85 \times 3500 \text{ psi} \times 90 \text{ inch}} = 2.588579397 \text{ inch; matches with a (initial)} = 2.588 \text{ inch (OK)}$$

$A_{s(main)} = 17.32730334 \text{ inch}^2 = 8$ (# 9) + 8 (# 10) = 8 (1.00 inch^2) + 8 (1.27 inch^2) = 18.16 inch^2

$c = a/\beta_1 = 2.588579397 \text{ inch}/0.85 = 3.045387526 \text{ inch}$ & $d = 40$ inch

check, $c < \frac{3}{8} d$, => c (=3.045387526 inch) < $\frac{3}{8} d$ ($= \frac{3}{8} \times 40$ inch) [tension controlled]

check for minimum steel, $A_{s(minimum)} = \frac{200}{f_y} b_w d = \frac{200}{40000 \text{ psi}} \times 18'' \times 40'' = 3.60 \text{ inch}^2 < A_{s(main)}$; OK

for negative moment, always rectangular beam

$H = 45''$; assume single layer of tension bars, $d = H - 2.50'' = 45'' - 2.50'' = 42.50''$

1st trial: assume, a (initial) = 1.25 inch

$$A_s = \frac{M_u}{\varphi f_y \left(d - \frac{a}{2}\right)} = \frac{207.6977934 \times 12000 \text{ lb.-inch}}{0.9 \times 40000 \text{ psi} \times \left(42.50 \text{ inch} - \frac{1.25 \text{ inch}}{2}\right)} = 1.653315768 \text{ inch}^2$$

$$a \text{ (final)} = \frac{A_s f_y}{0.85 f_c' b_{web}} = \frac{1.653315768 \text{ inch}^2 \times 40000 \text{ psi}}{0.85 \times 3500 \text{ psi} \times 18 \text{ inch}} = 1.234969762 \text{ inch; not match with a (initial)} = 1.25 \text{ inch}$$

[value of 'a' indicates that it's value decreasing (a=1.25 inch → a=1.2349 inch); next trial should be given with 'a' < 1.2349inch]

2nd trial: assume, a (initial) = 1.234 inch

$$A_s = \frac{M_u}{\varphi f_y\left(d - \frac{a}{2}\right)} = \frac{207.6977934 \times 12000\ \text{lb.-inch}}{0.9 \times 40000\ \text{psi} \times \left(42.50\ \text{inch} - \frac{1.234\ \text{inch}}{2}\right)} = 1.652999971\ \text{inch}^2$$

$$a\ (\text{final}) = \frac{A_s f_y}{0.85 f_c' b_{web}} = \frac{1.652999971\ \text{inch}^2 \times 40000\ \text{psi}}{0.85 \times 3500\ \text{psi} \times 18\ \text{inch}} = 1.234733872\ \text{inch; matches with a (initial)} =$$

1.234 inch (OK)

$c = a/\beta_1 = 1.234733872\ \text{inch}/0.85 = 1.452628085\ \text{inch}$ & $d = 42.50\ \text{inch}$

check, $c < \frac{3}{8}d$, => c (=1.452628085 inch) < $\frac{3}{8}$ d (=$\frac{3}{8}$ ×42.50 inch) [tension controlled]

check for minimum steel, $A_{s(minimum)} = \frac{200}{f_y} b_w d = \frac{200}{40000\ \text{psi}} \times 18'' \times 42.50'' = 3.8250\ \text{inch}^2 < A_{s(main)}$;

not OK

$A_{s(main)} = 3.8250\ \text{inch}^2 = 4\ (\#\ 9) = 4\ (1.00\ \text{inch}^2) = 4.00\ \text{inch}^2$

Design of **Interior Girder** for **Shear** at **Section (12)-(12)**

Design Shear, $V_u = 70.6904881$ kip; d = 40 inch

V_u (=70.6904881 kip) < $0.25\varphi f_c' b_w d$ (=0.25×0.90×3.50 ksi×18 inch×40 inch = 567 kip); concrete dimension is OK & need not to be revised.

$\varphi V_c = \varphi(0.0316)\beta\sqrt{f_c'} b_w d = 0.90 \times 0.0316 \times 2.0 \times \sqrt{3.50\ \text{ksi}} \times 18\ \text{inch} \times 40\ \text{inch} = 76.61716998$ kip

½φV_c (=38.30858499 kip) < V_u (=70.6904881 kip) < φV_c (=76.61716998 kip), provide a minimum amount [$A_{v(minimum)}$ / maximum spacing ($s_{(maximum)}$)]

#3 bar & 4-legged stirrup, $A_v = 4A_s = 4 \times 0.11\ \text{inch}^2 = 0.44\ \text{inch}^2$

V_u (=70.6904881 kip) < $0.125\varphi f_c' b_w d$ (=0.125×0.90×3.50 ksi×18 inch×40 inch = 283.50 kip)

$$s_{maximum} = \text{minimum of } \left[\frac{A_v f_y}{0.0316\sqrt{f_c'} b_w}, 0.80d \text{ \& } 24\ \text{inch}\right]$$

$$s_{maximum} = \text{minimum of } \left[\frac{0.44\ \text{inch}^2 \times 40\ \text{ksi}}{0.0316\sqrt{3.50\ \text{ksi}} \times 18\ \text{inch}}, 0.80 \times 40\ \text{inch} \text{ \& } 24\ \text{inch}\right]$$

$s_{maximum}$ = minimum of [16.53937362 inch, 32 inch & 24 inch] = 16.53937362 inch

$s_{provided}$ at section (12)-(12)= #3-4L stirrup @ 16.50 inch (lower rounding to 0.50 inch)

Skin Steel, Link/Ties for Skin Steel & **Haunch Steel** for **Interior Girder** at **Section (12)-(12)**

For negative moment (−M_u), $d_l = d = 42.50$ inch, (−)$A_s = 4\ \text{inch}^2$

$A_{sk\ (minimum)} \le A_{sk} \le A_{sk\ (maximum)}$

$0.012(d_l - 30) \le A_{sk} \le \frac{A_s}{4}$; [unit of A_{sk}: inch^2 per foot height per face of girder]

$A_{sk\ (minimum)} = 0.012(d_l - 30) = 0.012\ (42.50'' - 30'') = 0.15\ \text{inch}^2$ per foot height per face of girder

$$A_{sk\ (maximum)} = \frac{A_s}{4} = \frac{4\ \text{inch}^2}{4} = 1.00\ \text{inch}^2 \text{ per foot height per face of girder}$$

maximum spacing permitted ($s_{maximum}$) [along the height of girder[for skin reinforcement (A_{sk}) = minimum of [d_l/6 & 12″]= minimum of [42.50″/6 = 7.0833″ & 12″] = 7.0833″ = 7″ (lower rounding to 0.50″)

Skin reinforcement should be provided to a height of $d_l/2$ = 42.50″/2 = 21.25 inch (minimum) from nearest flexural tensile reinforcement.

let, we provide # 5 bar as skin steel @ 6″ c/c (≤7″) (along girder height per girder face)

$$A_{sk\ (provided)} = \frac{12 \times A_{s\ (\#5\ bar)}}{s_{provided}} = \frac{12 \times 0.31\ \text{inch}^2}{6\ \text{inch}} = 0.62\ \text{inch}^2 \text{ per foot height per face of girder (falls within}$$

the range of $A_{sk\ (minimum)}$ & $A_{sk\ (maximum)}$ as calculated for negative moment (+M_u) above; OK)

Haunch steel: #3 or #4 bar @ 6″ to 9″ c/c along the length of the girder. (provide #3 @ 6″ c/c)
Link/Tie for skin reinforcement: use #3 or #4 bar. Vertical & Longitudinal spacing should not exceed 24 inch. (provide #3 @ 16″ c/c)
For positive moment ($+M_u$), $d_i = d = 40$ inch, $(+)A_s = 18.16$ inch^2

$A_{sk\,(minimum)} \le A_{sk} \le A_{sk\,(maximum)}$

$0.012(d_i - 30) \le A_{sk} \le \frac{A_s}{4}$; [unit of A_{sk}: inch^2 per foot height per face of girder]

$A_{sk\,(minimum)} = 0.012(d_i - 30) = 0.012\,(40'' - 30'') = 0.12$ inch^2 per foot height per face of girder

$A_{sk\,(maximum)} = \frac{A_s}{4} = \frac{18.16\text{ inch}^2}{4} = 4.54$ inch^2 per foot height per face of girder

maximum spacing permitted ($s_{maximum}$) [along the height of girder[for skin reinforcement (A_{sk}) = minimum of [$d_i/6$ & 12″]= minimum of [40″/6= 6.6667″ & 12″] = 6.6667″ = 6.50″ (lower rounding to 0.50″)

Skin reinforcement should be provided to a height of $d_i/2 = 40''/2 = 20$ inch (minimum) from nearest flexural tensile reinforcement.

let, we provide # 5 bar as skin steel @ 5″ c/c (≤6.50″) (along girder height per girder face)

$A_{sk\,(provided)} = \frac{12 \times A_{s\,(\#5\,bar)}}{s_{provided}} = \frac{12 \times 0.31\text{ inch}^2}{5\text{ inch}} = 0.744$ inch^2 per foot height per face of girder (falls within the range of $A_{sk\,(minimum)}$ & $A_{sk\,(maximum)}$ as calculated for positive moment ($+M_u$) above; OK)

Haunch steel: #3 or #4 bar @ 6″ to 9″ c/c along the length of the girder. (provide #3 @ 6″ c/c)
Link/Tie for skin reinforcement: use #3 or #4 bar. Vertical & Longitudinal spacing should not exceed 24 inch. (provide #3 @ 16″ c/c) [see figure 46]

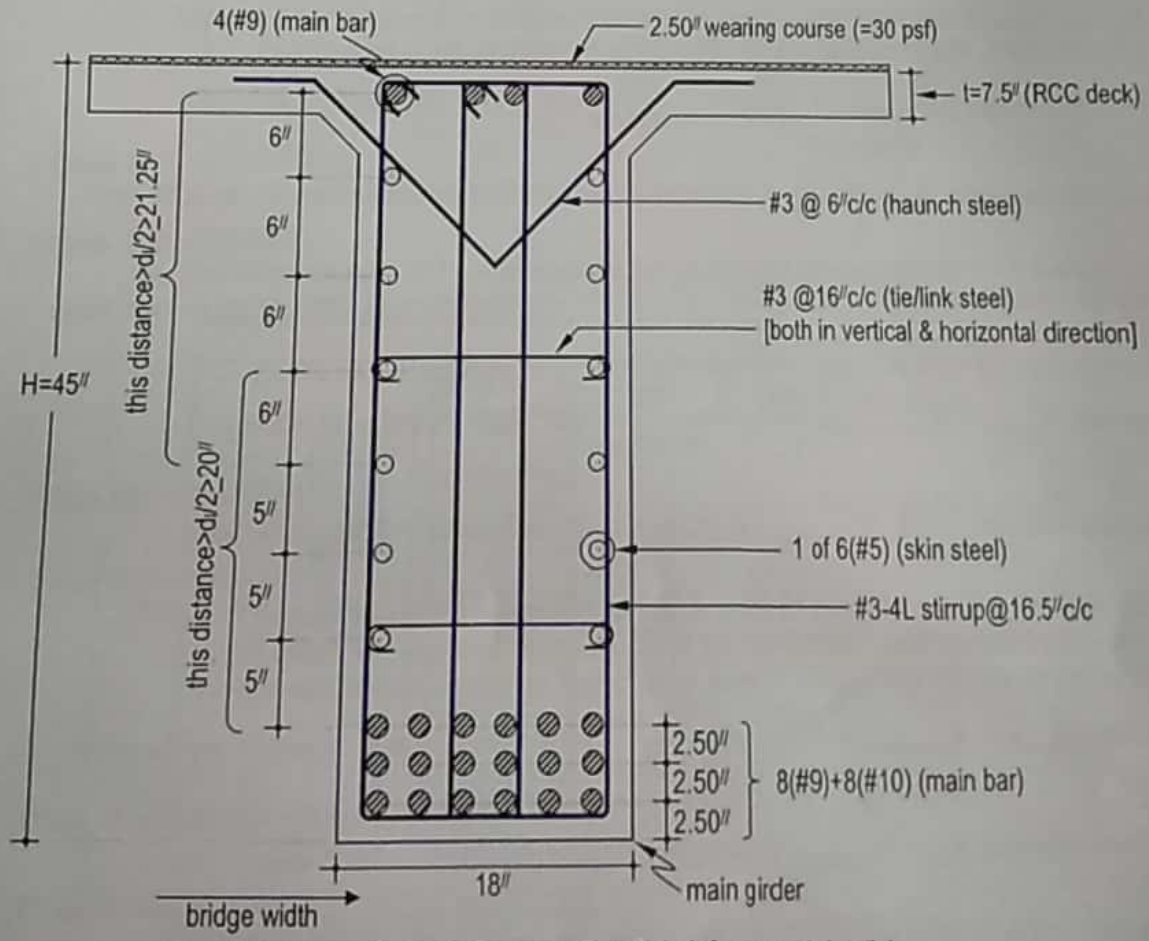


Figure 46: Interior Girder at Section (12)-(12) (reinforcement details)

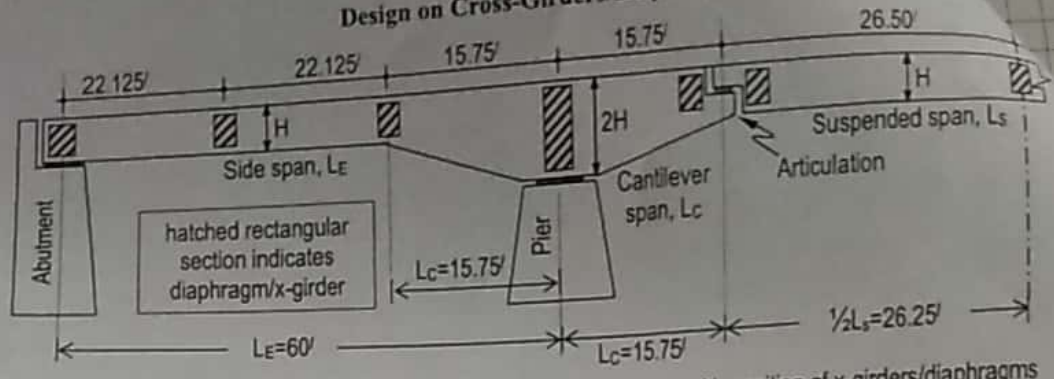


Figure 47: Longitudinal Profile of Three Span balanced cantilever bridge with position of x-girders/diaphragms

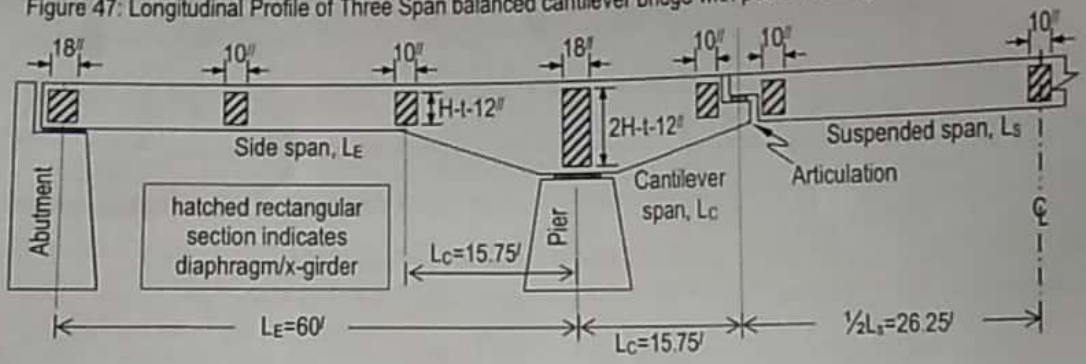


Figure 48: Longitudinal Profile of Three Span balanced cantilever bridge with position of x-girders/diaphragms

►Cross-Girder/Diaphragm (dimension: $18'' \times 25.50''$) at Abutment locations (see figure 50):

Effective depth, d = 25.50 inch – 2.50 inch = 23 inch (assuming single layer of bar)

Main bar, $A_{s(main)} = A_{s(minimum)} = \frac{200}{f_y} bd = \frac{200}{40000\,psi} \times 18'' \times 23'' = 2.07\ inch^2 = 2(\#6)+2(\#7)$

$[= 2(0.44)+2(0.6) = 2.08\ inch^2]$

Skin reinforcement, $A_{sk} = A_{s(minimum)} = 0.002bh = 0.002 \times 18'' \times 25.50'' = 0.918\ inch^2 = 3(\#5)$

$[= 3(0.31) = 0.93\ inch^2]$

Stirrup steel = # 4 @ (maximum) 18 inch c/c & Link/tie steel = # 3 @ (maximum) 24 inch c/c

►Cross-Girder/Diaphragm (dimension: $10'' \times 25.50''$) at Articulations & Other locations (see figure 51):

Effective depth, d = 25.50 inch – 2.50 inch = 23 inch (assuming single layer of bar)

Main bar, $A_{s(main)} = A_{s(minimum)} = \frac{200}{f_y} bd = \frac{200}{40000\,psi} \times 10'' \times 23'' = 1.275\ inch^2 = 3(\#6)$

$[= 3(0.44) = 1.32\ inch^2]$

Skin reinforcement, $A_{sk} = A_{s(minimum)} = 0.002bh = 0.002 \times 10'' \times 25.50'' = 0.51\ inch^2 = 3(\#4)$

$= [3(0.20) = 0.60\ inch^2]$

Stirrup steel = # 4 @ (maximum) 18 inch c/c & Link/tie steel = # 3 @ (maximum) 24 inch c/c

►Cross-Girder/Diaphragm (dimension: $18'' \times 70.50''$) at Pier locations (see figure 52):

Effective depth, d = 70.50 inch – 3.75 inch = 66.75 inch (assuming double layer of bar)

Main bar, $A_{s(main)} = A_{s(minimum)} = \frac{200}{f_y} bd = \frac{200}{40000\,psi} \times 18'' \times 66.75'' = 6.0075\ inch^2 = 4(\#9)+4(\#7)$

$[= 4(1.00)+4(0.60) = 6.40\ inch^2]$

Skin reinforcement, $A_{sk} = A_{s(minimum)} = 0.002bh = 0.002 \times 18'' \times 70.50'' = 2.538\ inch^2 = 6(\#6)$

$= [6(0.44) = 2.64\ inch^2]$

Stirrup steel = # 4 @ (maximum) 18 inch c/c & Link/tie steel = # 3 @ (maximum) 24 inch c/c

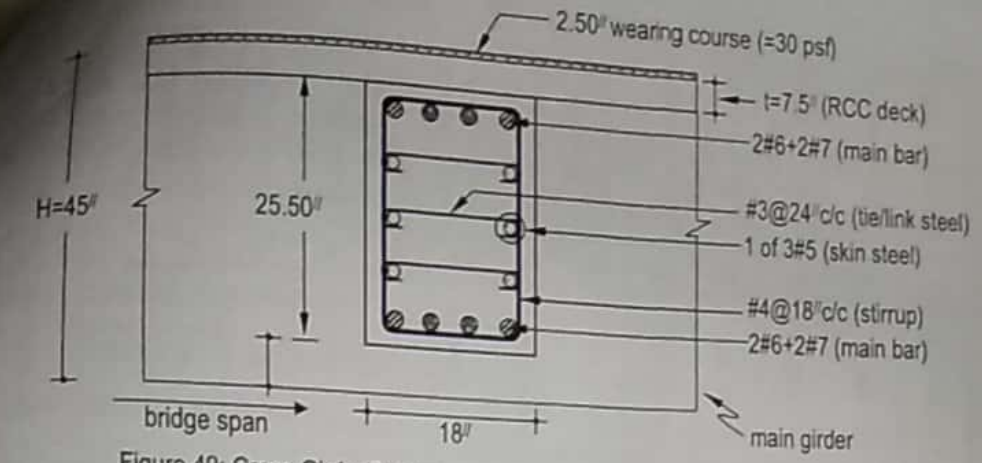


Figure 49: Cross-Girder/Diaphragm at Abutment locations (reinforcement details)

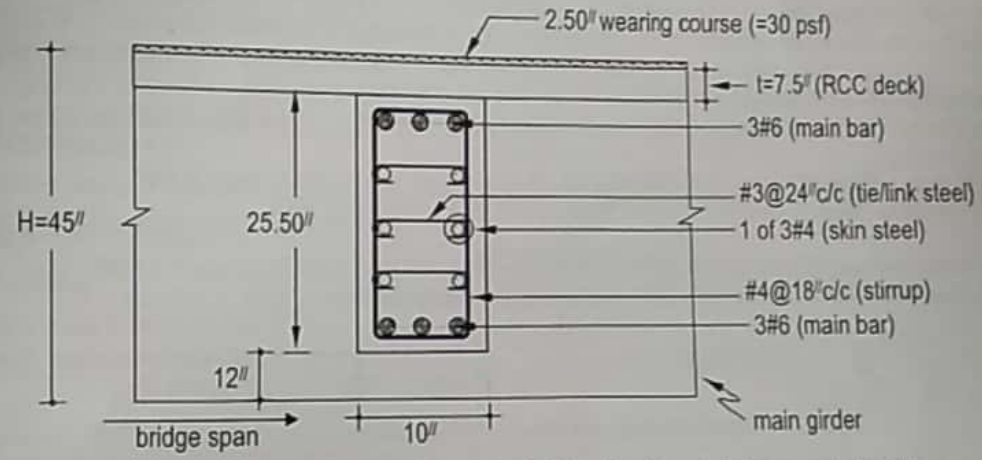


Figure 50: Cross-Girder/Diaphragm at Articulation & Other locations (reinforcement details)

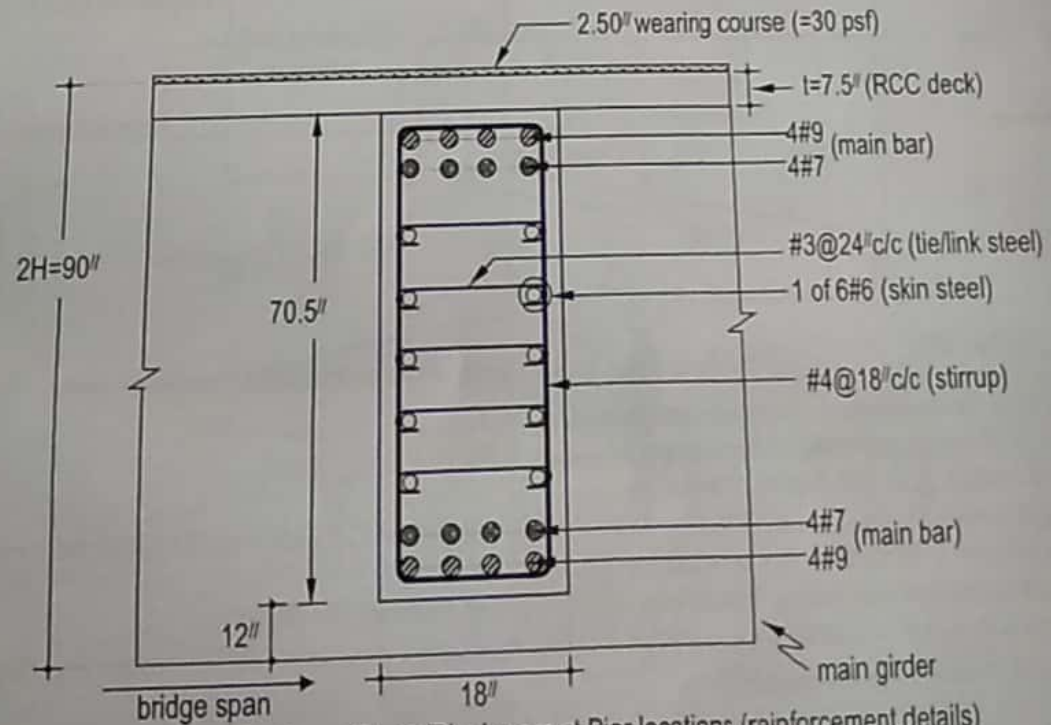


Figure 51: Cross-Girder/Diaphragm at Pier locations (reinforcement details)

Design on Articulation/Halving Joint

Table 5: Combination of Shear: Interior Girder

Location/Section	Self-weight shear	Wearing course shear	Distribution factor for shear	Truck load shear (positive)	Tandem load shear (positive)	Lane load shear (positive)	Combined positive shear (truck)	Combined positive shear (tandem)	Truck load shear (negative)	Tandem load shear (negative)	Lane load shear (negative)	Combined negative shear (truck)	Combined negative shear (tandem)	Design shear (absolute maximum of column 8, 9, 13 & 14
	kip	kip	$\alpha_{L,v}$	kip	kip	kip	kip	kip	kip	kip	kip	kip	kip	kip
4-4	+41.8828125	+5.90625	0.779081632	+59.20	+48.0952381	+16.80	**191.4659905**	**171.3295870**	0	0	0	**0**	**0**	**191.4659905**

Sample Calculation: Shear at section (4)-(4):

Live load Positive Shear (Truck) = 0.779081632×[1.33×(+59.20)+(+ 16.80)] = **+74.430342795 kip**

Combined Positive Shear (Truck) = 1.25(+41.8828125)+1.50(+5.90625) + 1.75(+74.430342795)
= **+191.4659905 kip**

Live load Positive Shear (Tandem) = 0.779081632×[1.33×(+48.0952381)+(+16.80)] = +62.923826483 kip

Combined Positive Shear (Tandem) = 1.25(+41.8828125)+1.50(+5.90625)+1.75(+62.923826483)
= **+171.3295870** kip

Live load Negative Shear (Truck) = 0.779081632×[1.33×(0)+(0)] = +1.036178571 kip = **0 kip***

Combined Negative Shear (Truck) = 1.25(+41.8828125)+1.50(+5.90625)+1.75(0)
= +61.212890625 kip = **0 kip***

Live load Negative Shear (Tandem) = 0.779081632×[1.33×(0) + (0)] = +1.036178571 kip = **0 kip***

Combined Negative Shear (Tandem) = 1.25(+41.8828125)+1.50(+5.90625)+1.75(0)
= +61.212890625 kip = **0 kip***
[*no negative shear found]

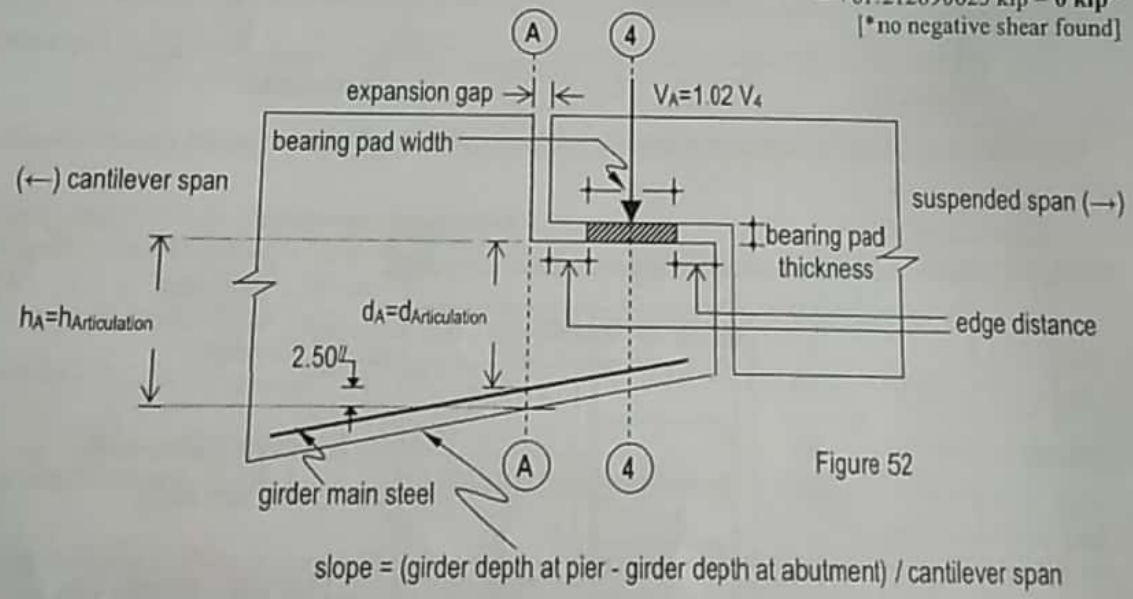


Figure 52

Expansion Gap Determination: temparature change, $\Delta T = T_{maximum} - T_{minimum} = 40°C$

co-efficient of thermal expansion of concrete, $\alpha_{concrete} = 0.00001/°C$

length of the bridge, L = 204 feet = 204×12 inch

expansion gap needed = $(\alpha_{concrete})(\Delta T)(L)$ = 0.00001/°C×40°C×204×12 inch = 0.9792 inch (in two articulation joint) = 0.4896 inch per articulation joint

expansion gap provided (including 1 inch gap in the event of extreme condition*)
= 0.4896 inch + 1 inch* = 1.4896 inch = 1.50 inch per articulation joint (rounded to higher ½ inch)

$V_{4\text{-}4\,(design)}$ = 191.4659905 kip (see table 5)

$V_{4\text{-}4\ design}$ is increased by 2% to account for excess weight due to girder widening

$V_{Articulation}$ = $V_{4\text{-}4\ design}$ +2% of $V_{4\text{-}4\ design}$ = $1.02V_{4\text{-}4\ design}$ = 1.02×191.4659905 kip = 195.2953103 kip

edge distance (≥7 inch) = 7 inch (say)

bearing pad width (≥8 inch) = 8 inch (say)

girder width at articulation = bearing pad length + 2×edge distance = bearing pad length + 14 inch

bearing pad length = girder width at articulation – 14 inch = b_A – 14 inch

design bearing strength of concrete, $\sigma_{bearing} = \varphi\,(0.85 f_c') = 0.70\times0.85\times3.50$ ksi = 2.0825 ksi

bearing force, $F_{bearing}$ (=$V_{Articulation}$)= 195.2953103 kip

bearing area, $A_{bearing}$ = bearing pad width× bearing pad length = $8''$×(girder width at articulation – $14''$)

Bearing Criterion: bearing force = bearing area × bearing strength

$F_{bearing} = \sigma_{bearing} \times A_{bearing}$

=> 195.2953103 kip = 2.0825 ksi × $8''$ × (girder width at articulation – $14''$)

=> girder width at articulation, $b_{Articulation} = \dfrac{195.2953103\,\text{kip}}{2.0825\,\text{ksi}\times 8\,\text{inch}} + 14\,\text{inch} = 25.72240758''$

Shear-Friction Criterion: $V_{Articulation} \le \varphi V_n = \varphi\, 0.2 f_c' b_{Articulation} d_{Articulation}$

and $V_{Articulation} \le \varphi V_n = \varphi\, 0.8 b_{Articulation} d_{Articulation}$

assume, bearing pad thickness = 4 inch

length of articulation = expansion gap + edge distance + bearing pad width + edge distance

$= 1.50'' + 7'' + 8'' + 1.50'' = 23.50$ inch = 1.958333333 feet

total depth at articulation, $h_{Articulation}$ = 20.50 inch + (1.958333333 feet× 2.857142857 inch / ft)

= 26.0952381 inch

effective depth at articulation, $d_{Articulation} = h_{Articulation} - 2.50''^{*} = 26.0952381'' - 2.50'' = 23.5952381''$

[*considering single layer of main bar]

$$b_{Articulation} = \frac{V_{Articulation}}{\varphi 0.20 f_c' d_{Articulation}} = \frac{195.2953103^{kip}}{0.90\times0.20\times3.50^{ksi}\times23.5952381''} = 13.13792871''$$

$$\&\ b_{Articulation} = \frac{V_{Articulation}}{\varphi 0.80 d_{Articulation}} = \frac{195.2953103^{kip}}{0.90\times0.80\times23.5952381''} = 11.49568762''$$

$b_{articulation}$ = maximum of [$b_{Articulation\ (from\ bearing)}$ & $b_{Articulation\ (from\ shear\ friction)}$]

$b_{Articulation}$ = maximum of [25.72240758″, 6.445389778″ & 5.639716056″] = 25.72240758″ = 26″ (say)

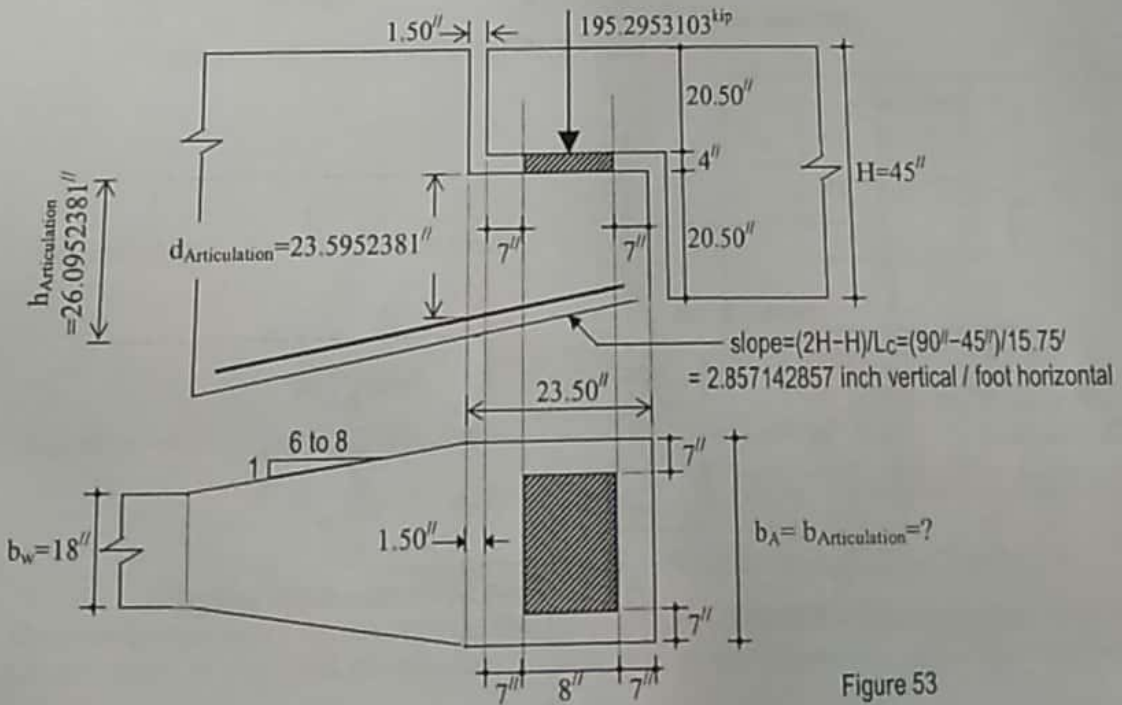


Figure 53

Horizontal Tractive Force (F_A) due to traffic/vehicle movement:

$F_A = 0.5V_{4,LL} \geq 0.2V_A = 0.50 \times 74.430342795$ kip ($\geq 0.20 \times 195.2953103$ kip)
$= 37.2151714$ kip (≥ 39.05906206 kip) $= 39.05906206$ kip

Flexural Steel (A_{sl}) due to cantilever/negative moment, $M_{Articulation}$

a_v = shear span (location of V_A from section A-A) = distance between section 4-4 & section A-A = expansion gap + edge distance + ½ of bearing pad width = 1.50 inch + 7 inch + ½ of 8 inch = 12.50 inch

$$M_A = V_A \times a_v + F_A \times \frac{d_A}{2} = 195.2953103 \text{ kip} \times 12.50 \text{ inch} + 39.05906206 \text{ kip} \times (23.5922381 \text{ inch} / 2)$$
$$= 2901.995313 \text{ kip-inch}$$

assume, a = 1.837 inch

$$A_{sl} = \frac{M_{Articulation}}{\varphi f_y \left(d_{Articulation} - \frac{a}{2} \right)} = \frac{2901.995313 \text{ kip} - \text{inch}}{0.90 \times 40 \text{ ksi} \times \left(23.5952381" - \frac{1.837"}{2} \right)} = 3.554787314 \text{ inch}^2$$

check, $$a = \frac{A_{sl} f_y}{0.85 f'_c b_{Articulation}} = \frac{3.554787314 \text{ inch}^2 \times 40 \text{ ksi}}{0.85 \times 3.50 \text{ ksi} \times 26 \text{ inch}} = 1.838286911 \text{ inch} \approx 1.837 \text{ inch; OK}$$

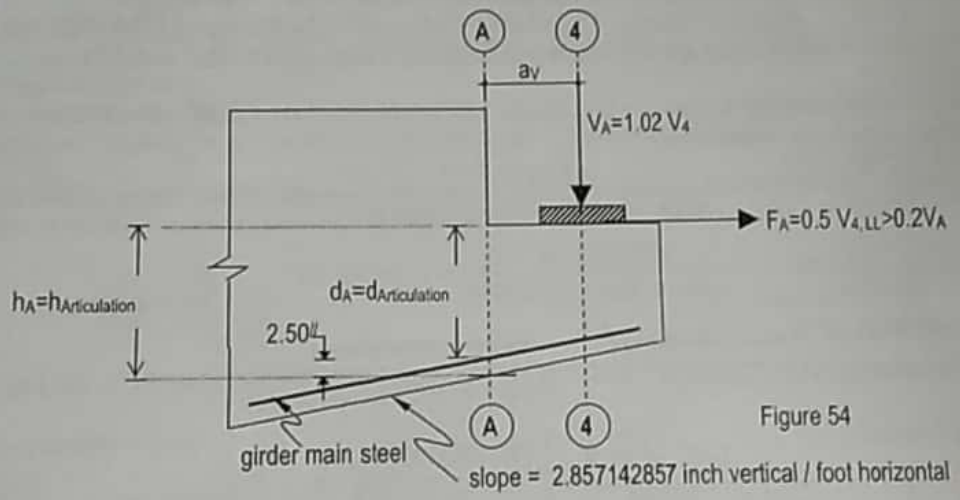


Figure 54

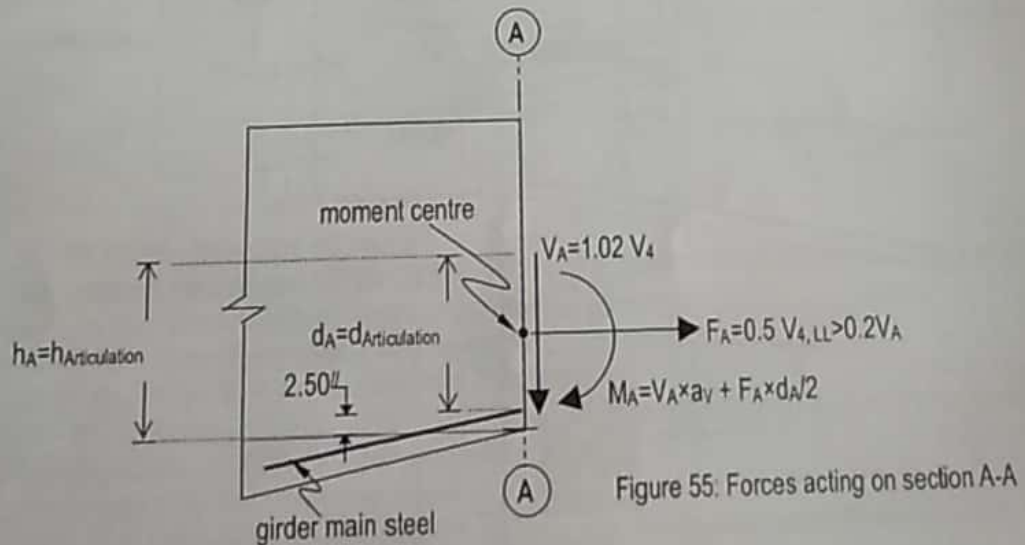


Figure 55: Forces acting on section A-A

strength reduction factor (φ) = 0.90 (for bending/flexure)
= 0.90 (for shear (flexural-shear & shear-friction))
= 0.70 (for bearing)
= 0.85 (for direct tension)

Hanger Steel (A_{s3}) due to Direct Tension of V_A:

$$A_{s3} = \frac{V_A}{\varphi f_y} = \frac{195.2953103\ \text{kip}}{0.85 \times 40\ \text{ksi}} = 5.743979715\ \text{inch}^2 = 6\ \#\ 9\ \text{bar}\ (=6\times 1.00\ \text{inch}^2 = 6\ \text{inch}^2)$$

Tension Steel (A_{s2}) due to Direct Tension of F_A:

$$A_{s2} = \frac{F_A}{\varphi f_y} = \frac{39.05906206\ \text{kip}}{0.85 \times 40\ \text{ksi}} = 1.148795943\ \text{inch}^2$$

Conventional Flexural-Shear Steel (Stirrup Steel) due to V_A:

$s_{maximum}$ = minimum of $(12'' \ \& \ \frac{d_A}{2})$ = minimum of $(12'' \ \& \ \frac{23.5952381''}{2})$

= minimum of $(12'' \ \& \ 11.79761905'') = 11.79761905'' = 11.50$ inch (lower rounding to 0.50 inch)

$\varphi V_c = \varphi(0.0316\beta\sqrt{f_c'}\, b_A d_A) = 0.90\times 0.0316\times 2\times\sqrt{3.50\ \text{ksi}}\times 26''\times 23.5952381'' = 65.28167996$ kip

using 4-legged #4 stirrup, $A_v = 4A_s = 4\times 0.20\ \text{inch}^2 = 0.80\ \text{inch}^2$

$$s_{required} = \frac{\varphi A_v f_y d_A}{V_u - \varphi V_c} = \frac{0.90\times 0.80\ \text{inch}^2\times 40\ \text{ksi}\times 23.5952381\ \text{inch}}{195.2953103\ \text{kip} - 65.28167996\ \text{kip}} = 5.226704735\ \text{inch} = 5\ \text{inch}$$

(lower rounding to 0.50 inch)

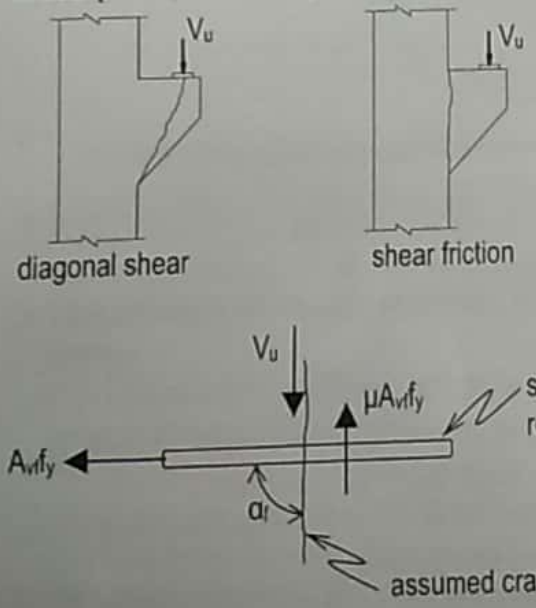
check, $s_{required}\ (=5'') \le s_{maximum}\ (=11.50'')$ => OK

provide #4 - 4L stirrup @ 5 inch c/c (stirrup will be placed d_A distance from section A-A)

Shear-Friction Steel due to V_A $\left(\text{such steel needed when } \frac{a_v}{d_a} < 1\right)$:

$$\frac{a_v}{d_a} = \frac{12.50\ \text{inch}}{23.5952381\ \text{inch}} = 0.529767911 < 1$$

(The smaller the ratio of a/d is, the larger the tendency for pure shear to occur through essentially vertical planes; a = shear span & d = effective depth)



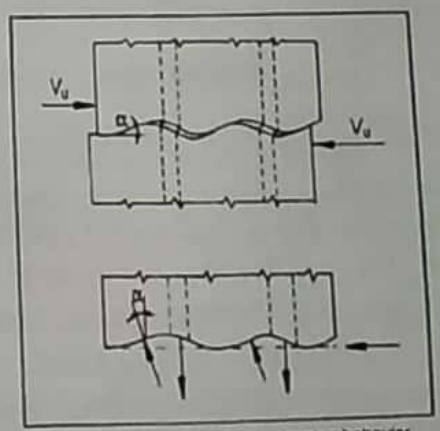


Fig. 4. Shear friction mechnism showing behavior at crack dilatation.

Figure 56: Shear-Friction Theory

The shear-friction theory assumes that the shear strength of a concrete-to-concrete interface subjected simultaneously to shear and compression forces is ensured by friction only. A simple "saw-tooth

model" is usually adopted to exemplify the basic principles of this theory, Figure 56. The influence of both shear reinforcement crossing the interface and normal stresses to the shear plane are considered.

Table 6: Value of Co-efficient of friction (μ) as a function of the failure surface

Characteristics of failure surface	Co-efficient of friction (μ)
Concrete cast monolithically	1.4λ
Concrete placed against hardened, roughened concrete	1.0λ
Concrete anchored to structural steel (Unpainted structural steel heated studs)	0.7λ
Concrete placed against unroughened (smooth)hardened concrete	0.6λ

$\lambda = 1$ (for normal-weight concrete), $\lambda = 0.85$ for sand-lightweight concrete and $\lambda = 0.75$ for all lightweight concrete. Linear interpolation permitted when partial sand replacement is used.

$\varphi = 0.90$ (for shear); $c = 145$ psi (shear stress in concrete due to saw-tooth interlocking (Figure 52)), μ (friction co-efficient between concrete & steel) = 1.40 (Table 3)

A_{cv} = area of shear - crack interface = $b_{Articulation} \times d_{Articulation}$ = 26 inch × 23.5952381 inch = 613.4761906 inch^2

A_{vf} = shear friction steel crossing shear plane (inch^2)

$V_{Articulation}$ = 195.2953103 kip = 195.2953103×1000 lb. & f_y = 40 ksi = 40000 psi

$V_{Articulation} = \varphi[cA_{cv} + \mu A_{vf} f_y]$

=> 195.2953103×1000 lb. = 0.90×[145 psi×613.4761906 inch^2 + 1.40× A_{vf} ×40000 psi]

=> A_{vf} = 2.286441818 inch^2

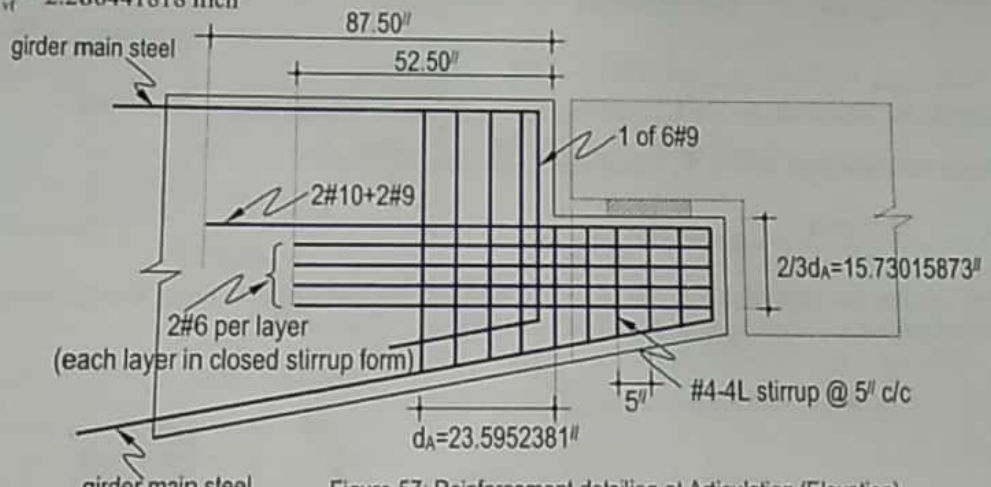


Figure 57: Reinforcement detailing at Articulation (Elevation)

$A_{vf} + A_{s2}$ will be distributed within $\frac{2}{3} d_A$ = 15.73015873 inch from articulation top (from bearing pad level)

We decide that 1/5th of $A_{vf} + A_{s2}$ will be merged with A_{s1} (placed at top) and the remaining 4/5th of $A_{vf} + A_{s2}$ will be placed in 4 layers (placed below A_{s1})

$A_{s2} + A_{vf}$ = 1.148795943 inch^2 + 2.286441818 inch^2 = 3.435237761 inch^2

A_{s1} + 1/5th of ($A_{s2} + A_{vf}$) = 3.554787314 inch^2 + 1/5×(3.435237761 inch^2) = 4.241834866 inch^2

= 2#10 + 2#9 = (2×1.27+2×1.00) inch^2 = 4.54 inch^2 (in a single layer - placed at top)

4/5th of ($A_{s2} + A_{vf}$) = 4/5×(3.435237761 inch^2) = 2.756190209 inch^2 = 4 layers @ 2 # 6 bar per layer

= 4 layers @ 2×0.44 inch^2 per layer = 3.52 inch^2 (in closed stirrup form)

development length in tension, l_d (for top bars) = 70×bar diameter = 70d_b = 70×(10/8) inch = 87.50 inch (from section A-A)

development length in tension, l_d (for bars below top layers) = 70×bar diameter = 70d_b = 70×(6/8) inch = 52.50 inch (from section A-A)