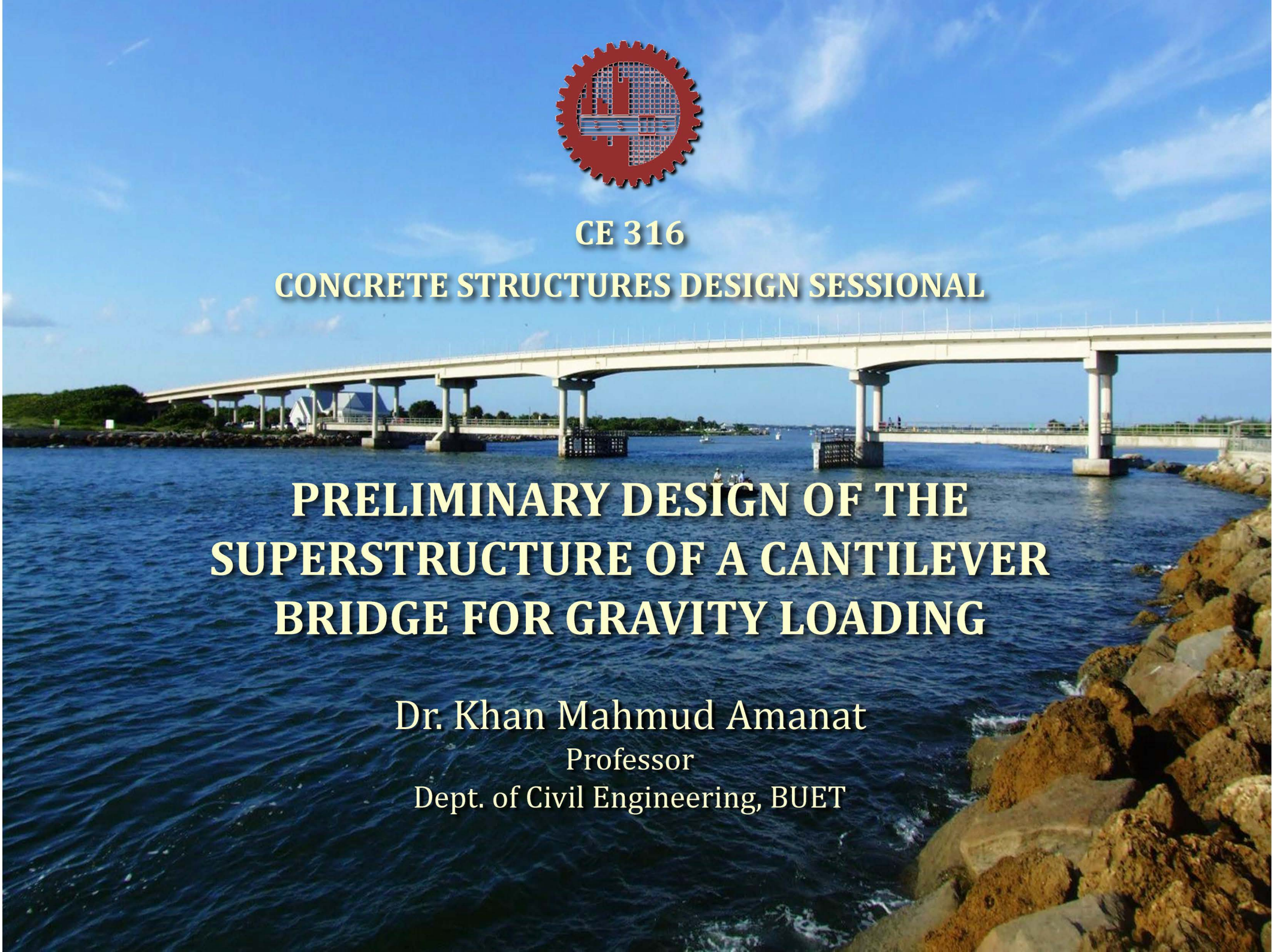


CE 316

CONCRETE STRUCTURES DESIGN SESSIONAL

# PRELIMINARY DESIGN OF THE SUPERSTRUCTURE OF A CANTILEVER BRIDGE FOR GRAVITY LOADING

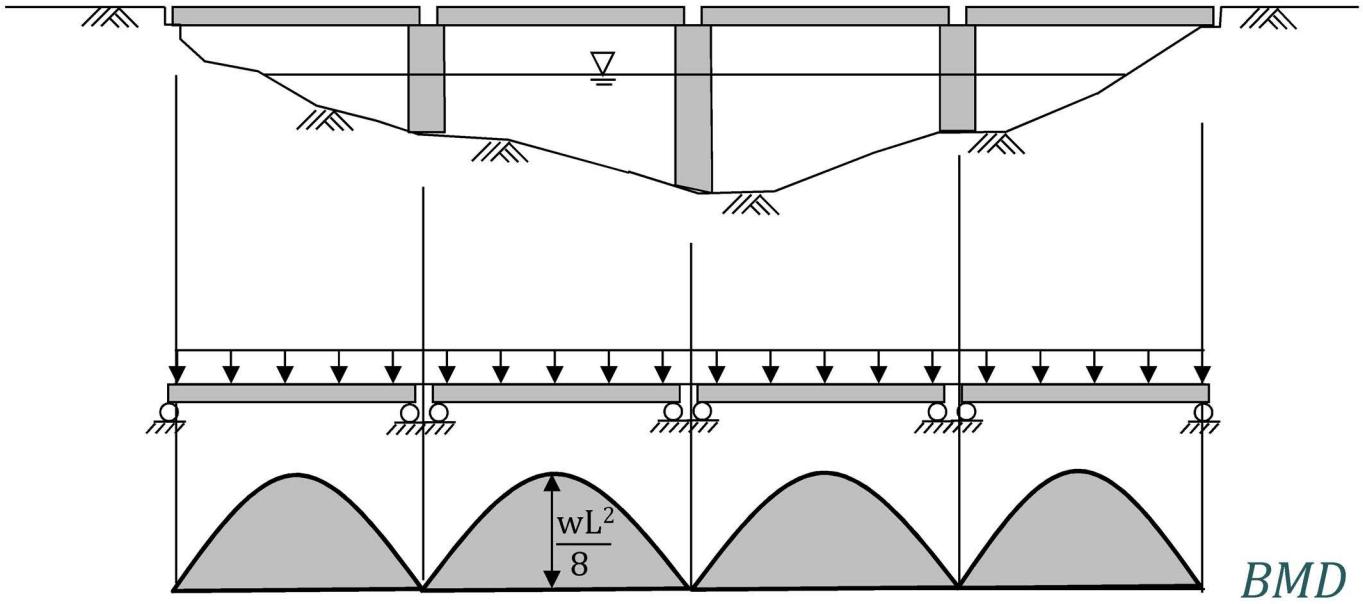
Dr. Khan Mahmud Amanat
Professor
Dept. of Civil Engineering, BUET



**A common RC bridge in rural Bangladesh**

**Multiple Simply Supported Spans**

# Multiple Simply Supported Spans




*BMD*

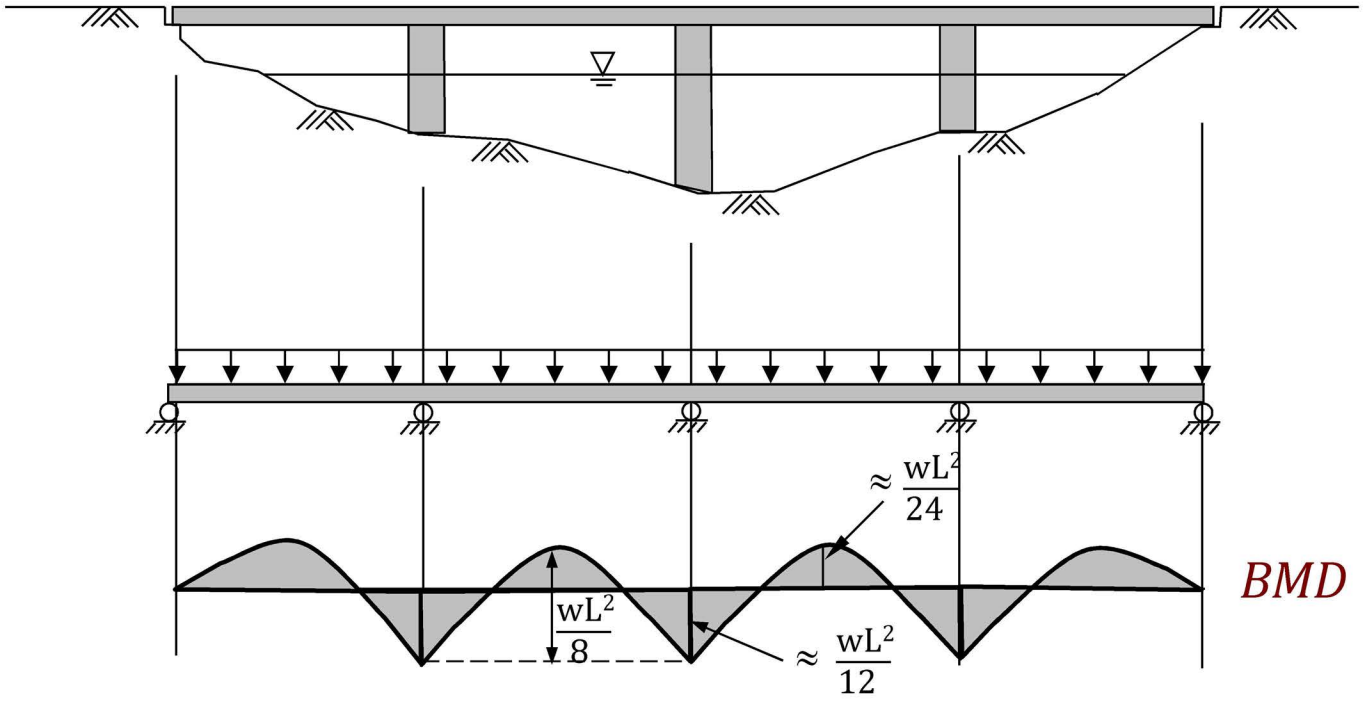
## Advantage

Determinate structure:
No stress due to differential settlement.

## Disadvantage

Large magnitude of bending moment requiring bigger and heavier section: **uneconomic**

## Continuous Spans



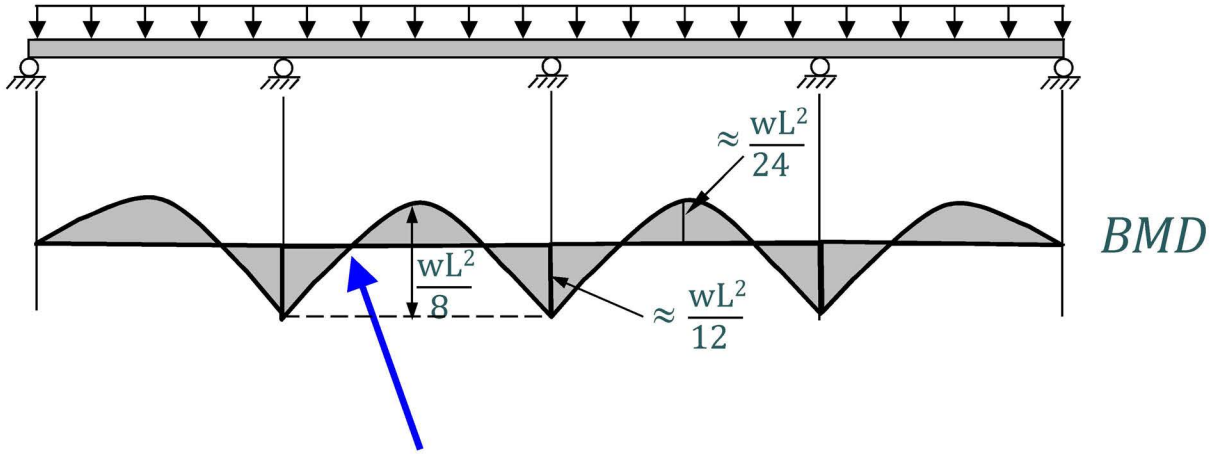

**Advantage**

Magnitude of maximum moment reduced:
Resulting in economic section

**Disadvantage**

Large bending moment due to uneven/differential settlement

# Developing the Idea of Cantilever Form




**Point of contraflexure**

→ **Zero moment**

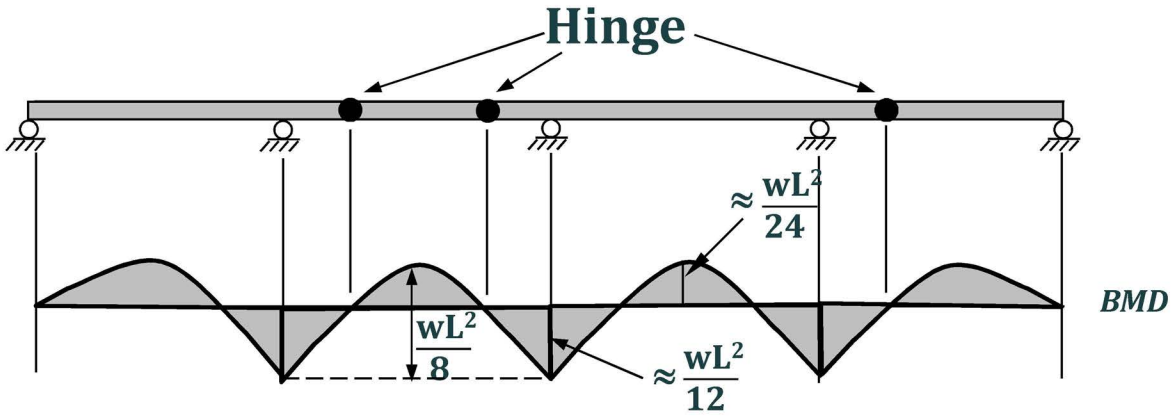
→ **Hinge**

**Advantages of both the simply-supported and continuous span can be retained**
**&**
**Disadvantages can be eliminated by**

**Inserting structural hinges at some of the points of contraflexure……**

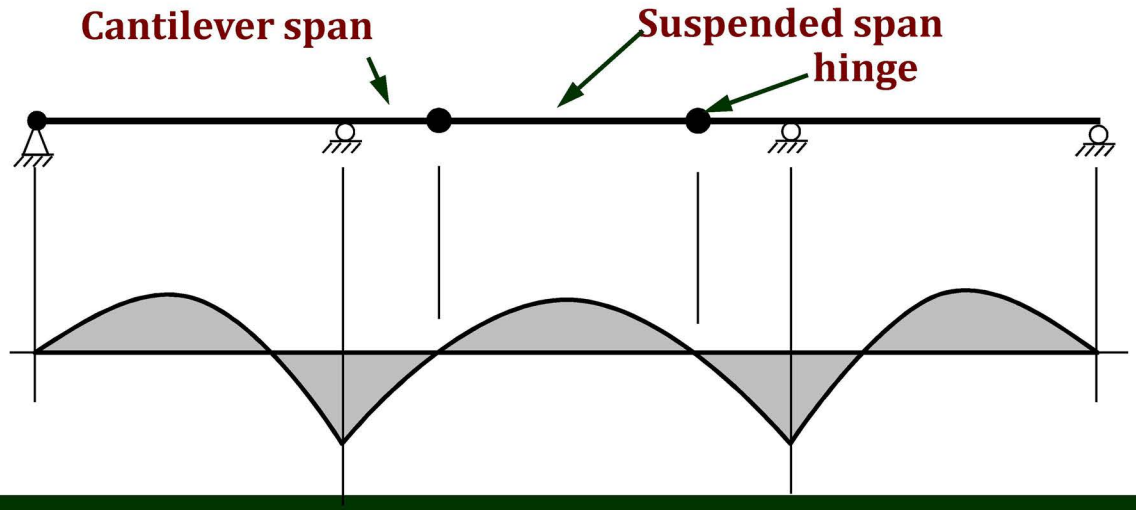
# Developing the Idea of Cantilever Form

Statically indeterminate to 3rd degree. Therefore, three hinges are required to make it determinate.




## THREE SPAN BRIDGE

Statically indeterminate to 2nd degree. Therefore, two hinges are required to make it determinate.




Hinges render the structure determinate:

**Thus the problem of large stress due to settlement is eliminated.**

Bending moment diagram of indeterminate structure is retained:

**Thus the design section becomes economic**

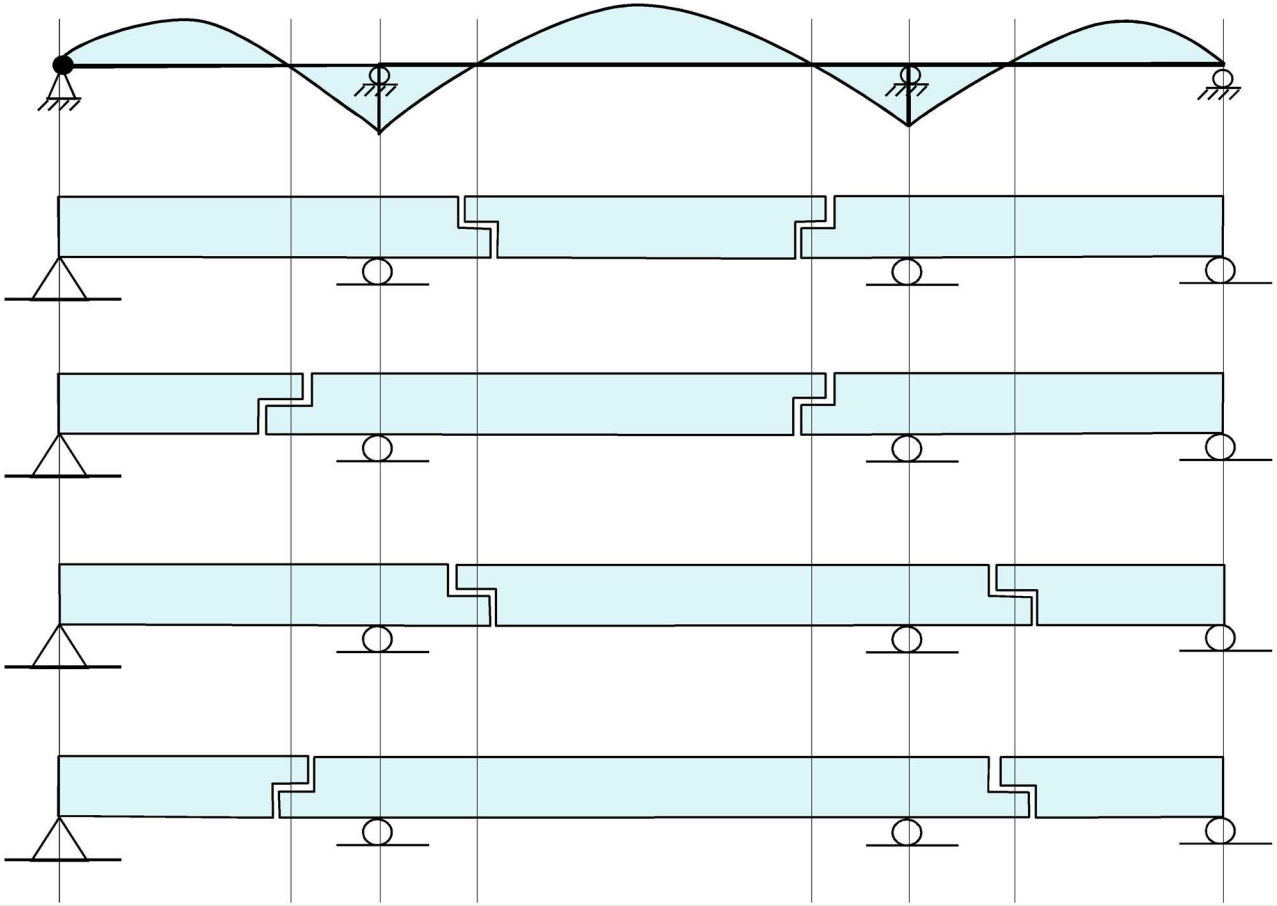
# Possible Cantilever Configurations For Three Span Bridge

No. of points of contraflexure = 4, Degree of static indeterminacy = 2

**Thus the beam can be made determinate by inserting hinge at any two of the four points of contraflexure.**

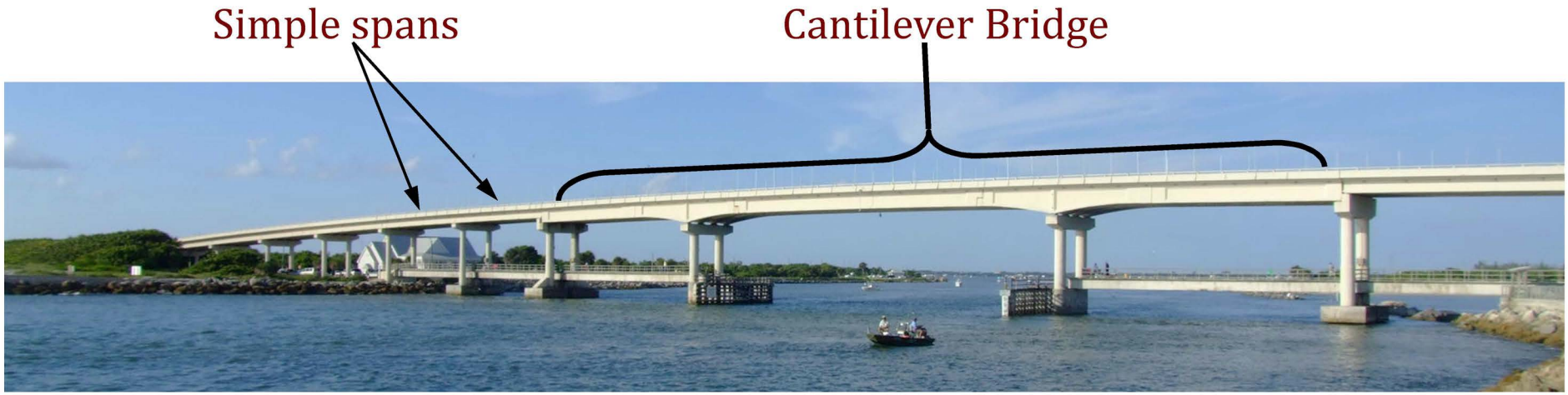
Therefore, there can be $^4C_2 = 6$ different arrangements of hinges.

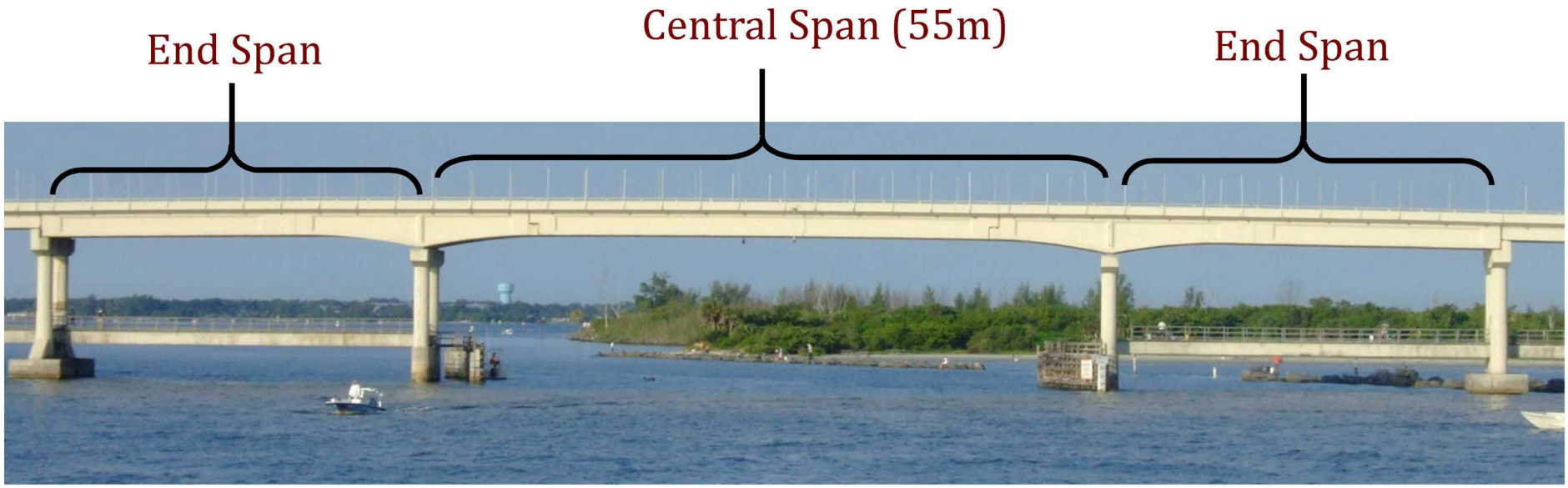
**Four are shown below.....**



# Example:
# Sebastian Inlet Bridge, Florida, USA

**Year of construction: 1965, Total length= 472m, Central span = 55m.**



# Example: Sebastian Inlet Bridge, Florida, USA

Cantilever Span | Suspended Span | Cantilever Span



End Span



Halving Joint / Articulation

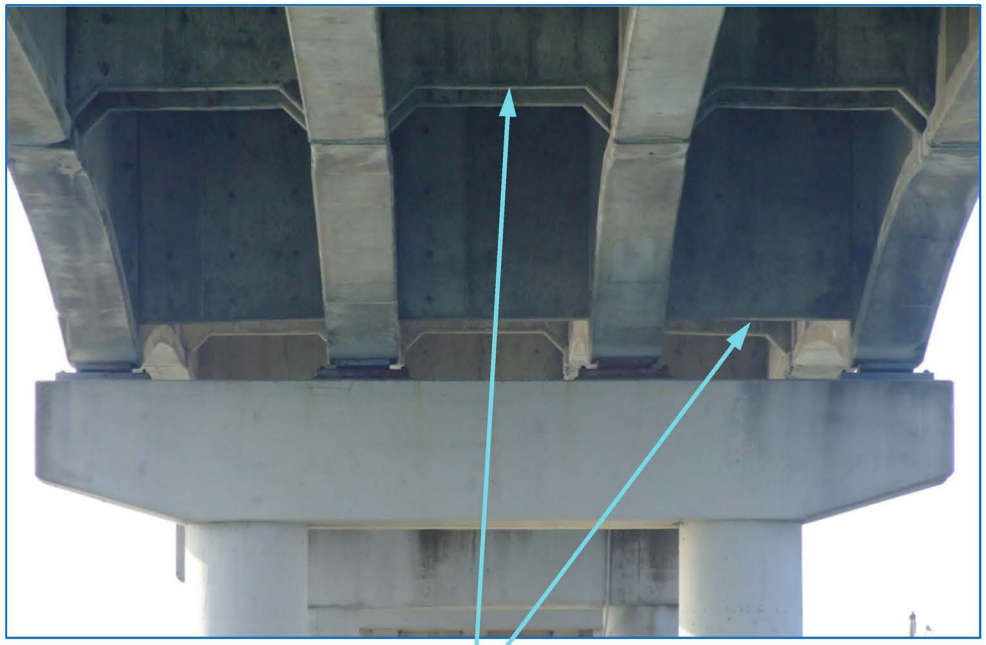


# Example: Sebastian Inlet Bridge, Florida, USA

## Support Details



Bearing



Diaphragms

# Example: Sebastian Inlet Bridge, Florida, USA

## Support Details



# Example: Sebastian Inlet Bridge, Florida, USA

## Diaphragm Details

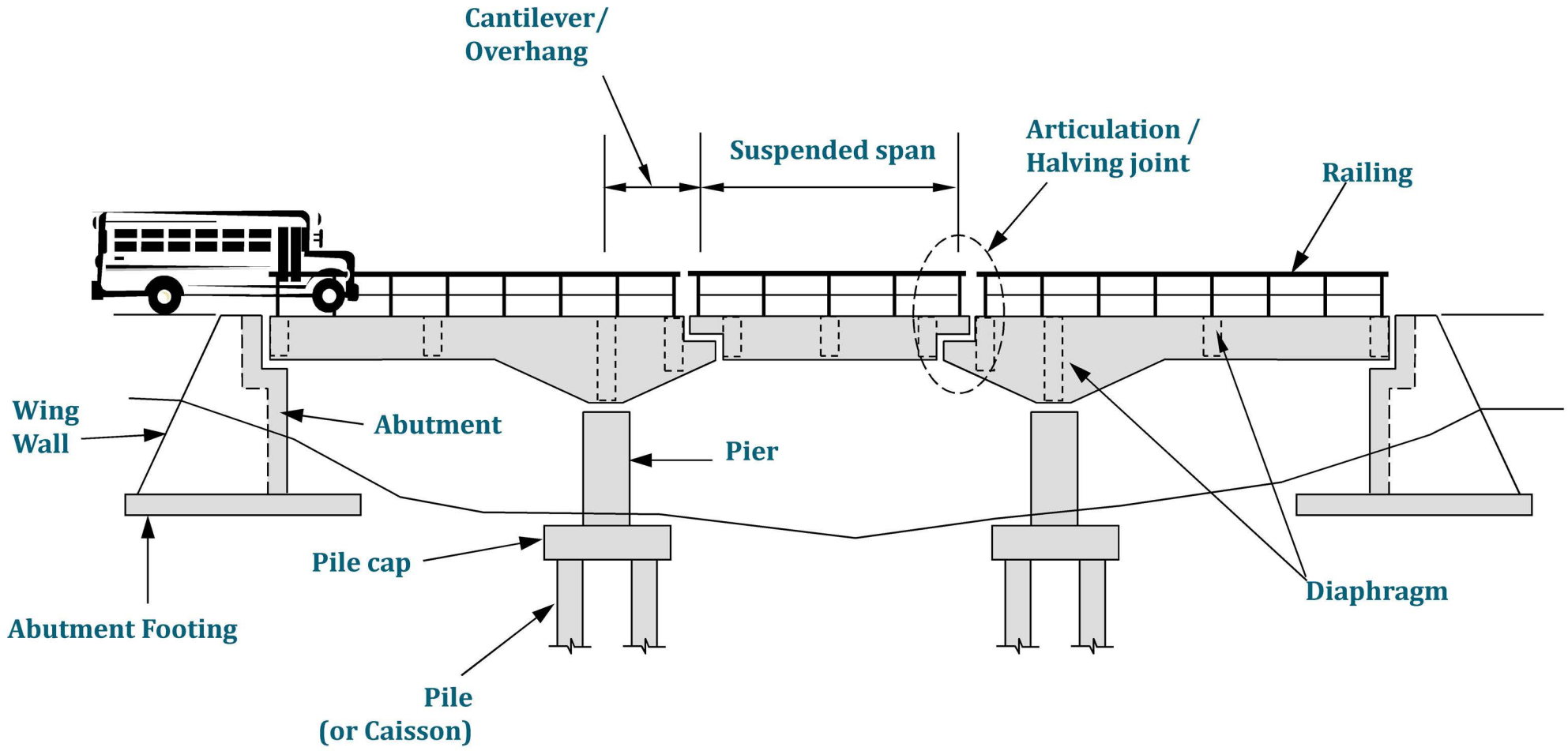


Diaphragms

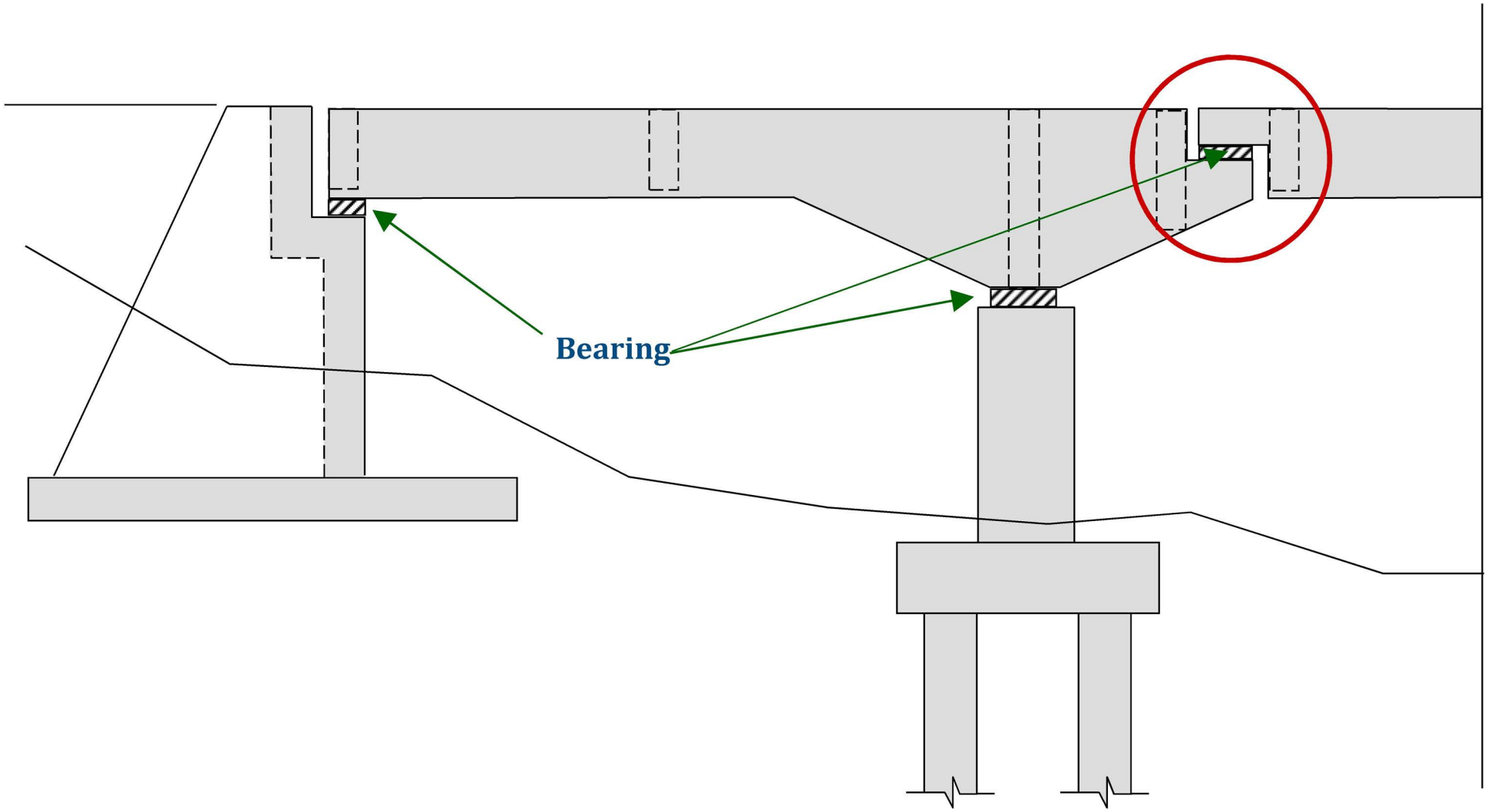


Diaphragm

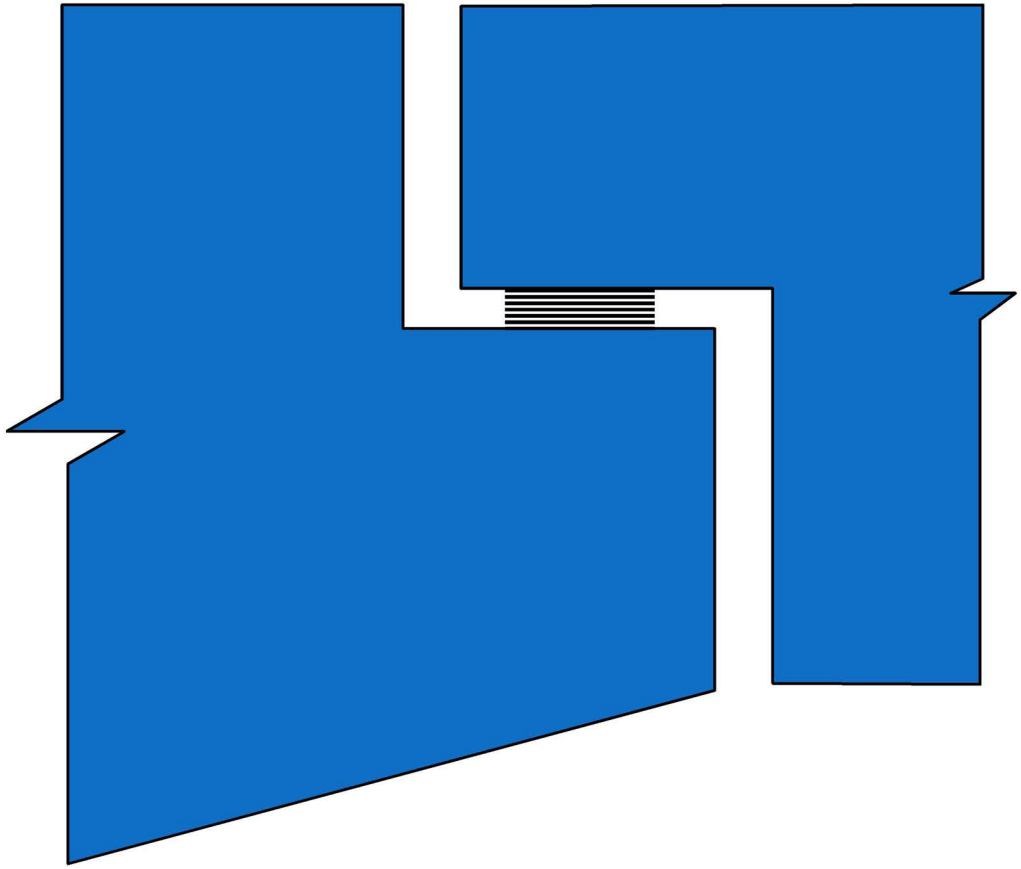
## Components of a Cantilever Bridge

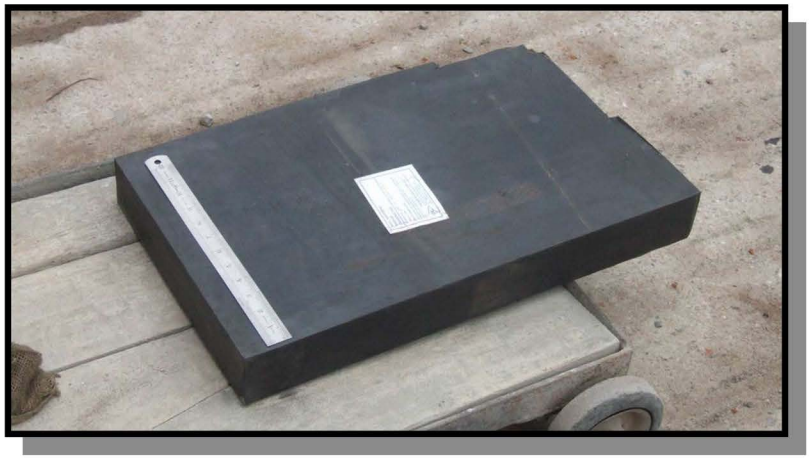


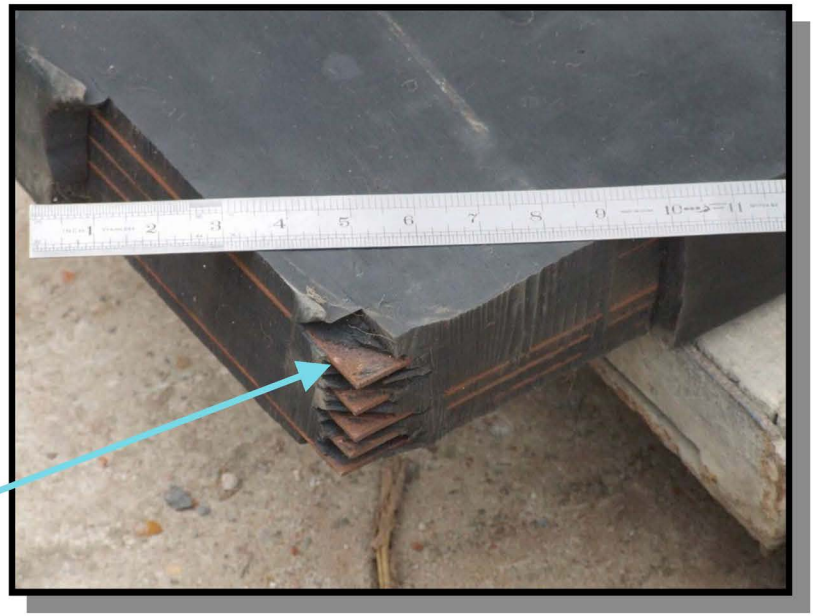
## Components of a Cantilever Bridge



## Neoprane Bearing Pad



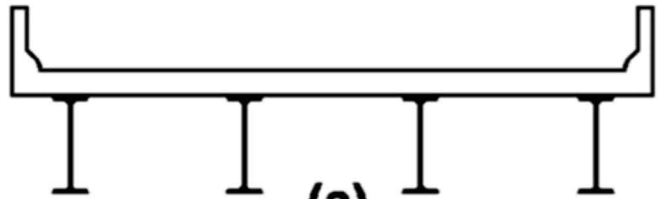
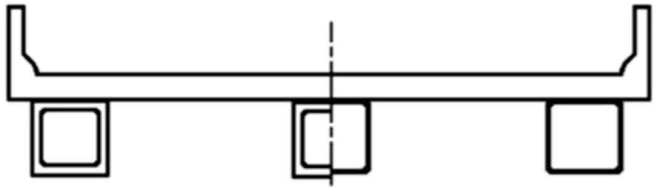
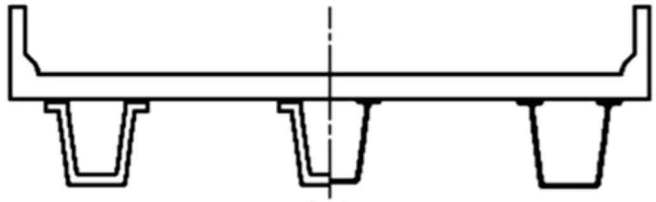




**Steel plate sandwitched between neoprene rubber**

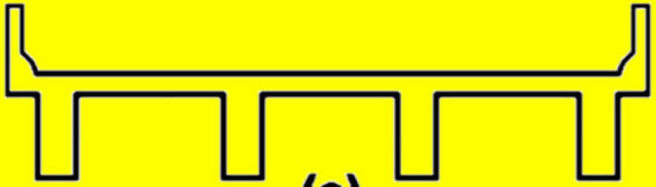
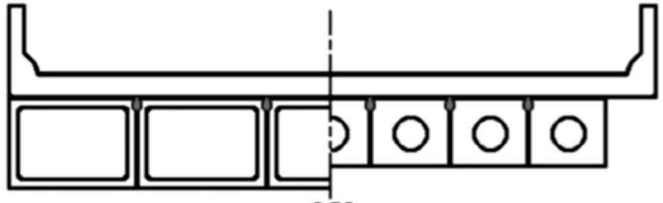
# Bridge Deck Configurations (AASHTO 2012 Chapter 4)

**Table 4.6.2.2.1-1—Common Deck Superstructures Covered in Articles 4.6.2.2.2 and 4.6.2.2.3**

| Supporting Components | Type Of Deck | Typical Cross-Section |
|---|---|---|
| Steel Beam | Cast-in-place concrete slab, precast concrete slab, steel grid, glued/spiked panels, stressed wood |  (a) |
| Closed Steel or Precast Concrete Boxes | Cast-in-place concrete slab |  (b) |
| Open Steel or Precast Concrete Boxes | Cast-in-place concrete slab, precast concrete deck slab |  (c) |

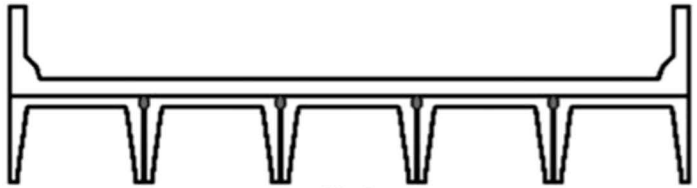
# Bridge Deck Configurations (AASHTO 2012 Chapter 4)

**Table 4.6.2.2.1-1—Common Deck Superstructures Covered in Articles 4.6.2.2.2 and 4.6.2.2.3**

| Supporting Components | Type Of Deck | Typical Cross-Section |
|---|---|---|
| Cast-in-Place Concrete Multicell Box | Monolithic concrete |  (d) |
| Cast-in-Place Concrete Tee Beam | Monolithic concrete |  (e) |
| Precast Solid, Voided or Cellular Concrete Boxes with Shear Keys | Cast-in-place concrete overlay |  (f) |

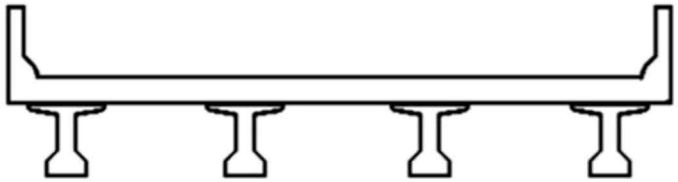
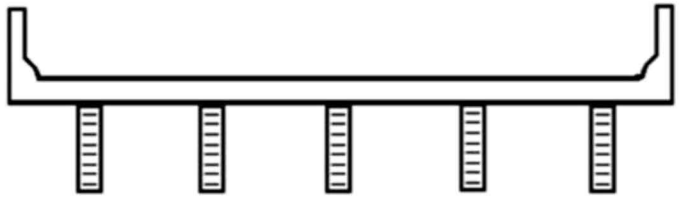
# Bridge Deck Configurations (AASHTO 2012 Chapter 4)

**Table 4.6.2.2.1-1—Common Deck Superstructures Covered in Articles 4.6.2.2.2 and 4.6.2.2.3**

| Supporting Components | Type Of Deck | Typical Cross-Section |
|---|---|---|
| Precast Solid, Voided, or Cellular Concrete Box with Shear Keys and with or without Transverse Post-Tensioning | Integral concrete |  (g) |
| Precast Concrete Channel Sections with Shear Keys | Cast-in-place concrete overlay |  |
| Precast Concrete Double Tee Section with Shear Keys and with or without Transverse Post-Tensioning | Integral concrete |  (i) |

# Bridge Deck Configurations (AASHTO 2012 Chapter 4)

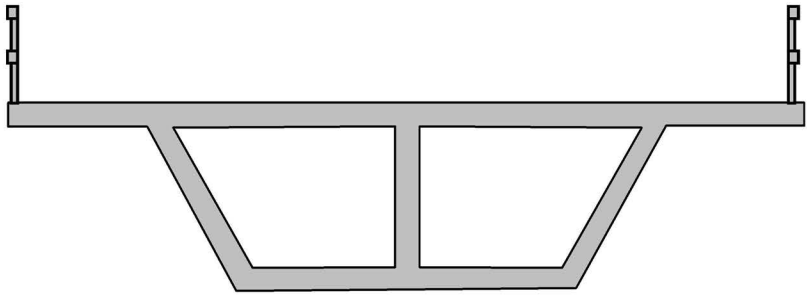
**Table 4.6.2.2.1-1—Common Deck Superstructures Covered in Articles 4.6.2.2.2 and 4.6.2.2.3**

| Supporting Components | Type Of Deck | Typical Cross-Section |
|---|---|---|
| Precast Concrete Tee Section with Shear Keys and with or without Transverse Post-Tensioning | Integral concrete |  (j) |
| Precast Concrete I or Bulb-Tee Sections | Cast-in-place concrete, precast concrete |  (k) |
| Wood Beams | Cast-in-place concrete or plank, glued/spiked panels or stressed wood |  (l) |

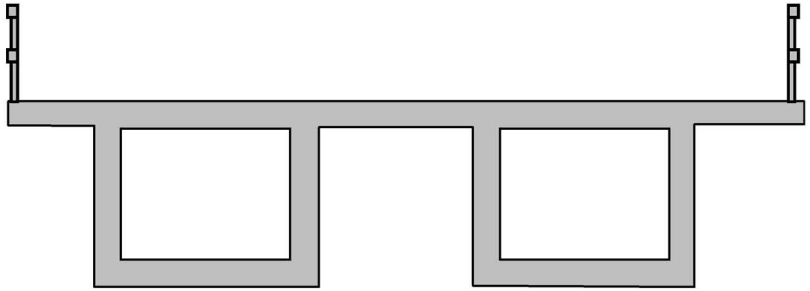
## Bridge Deck Configurations



**Box girder**
(Bangabandhu Bridge, Meghna Bridge, Lalon Shah Bridge, Gabkhan Bridge etc.)



**Multiple cell Box girder**
(Hanif Flyover, Maghbazar Flyover)



**Spread Box girder**
(Bangladesh China 1st Friendship Bridge)

# Bridge Design Items

1. Bridge superstructure:
   - Deck
   - Girders
   - Diaphragm beams
   - Railing
   - Articulation etc.
2. Bridge Pier
3. Foundation
   - Pile cap
   - Pile (or caisson)
4. Abutment and wing walls
5. Approach road
6. River training works

# Loads on Bridge (AASHTO 2012, Sec. 3.3.2)

The following permanent and transient loads and forces are considered to act on a bridge structure:

## Permanent Loads

*CR* = force effects due to creep
*DD* = downdrag force
*DC* = dead load of structural components and nonstructural attachments
*DW* = dead load of wearing surfaces and utilities
*EH* = horizontal earth pressure load
*EL* = miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
*ES* = earth surcharge load
*EV* = vertical pressure from dead load of earth fill
*PS* = secondary forces from post-tensioning
*SH* = force effects due to shrinkage

# Transient Loads

*BL* = blast loading
*BR* = vehicular braking force
*CE* = vehicular centrifugal force
*CT* = vehicular collision force
*CV* = vessel collision force
*EQ* = earthquake load
*FR* = friction load
*IC* = ice load
*IM* = vehicular dynamic load allowance
*LL* = vehicular live load
*LS* = live load surcharge
*PL* = pedestrian live load
*SE* = force effect due to settlement
*TG* = force effect due to temperature gradient
*TU* = force effect due to uniform temperature
*WA* = water load and stream pressure
*WL* = wind on live load
*WS* = wind load on structure

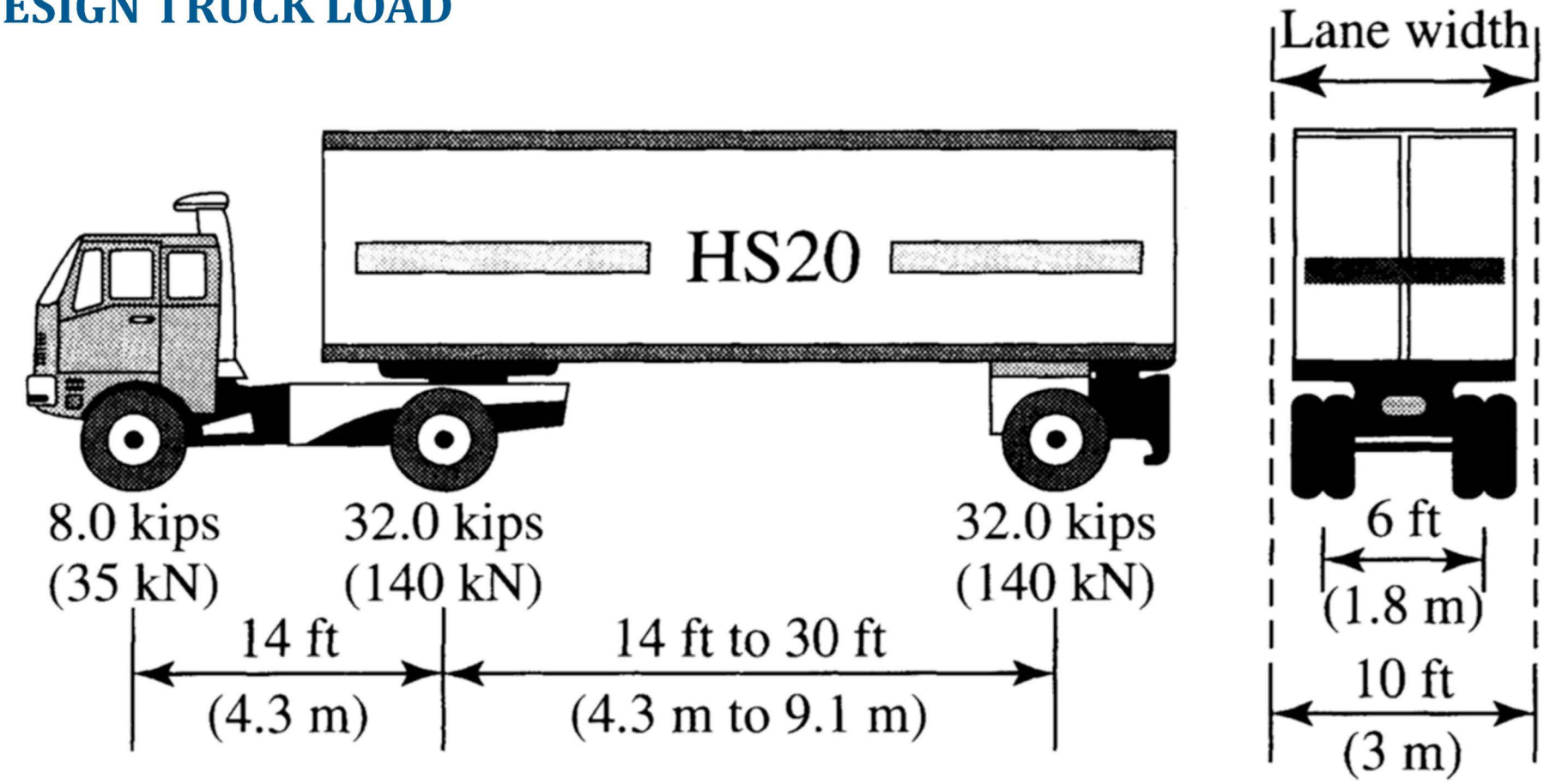
## DESIGN VEHICULAR LIVE LOAD

Vehicular live loading on the roadways of bridges or incidental structures, designated HL-93, shall consist of a combination of the:

- Design truck or design tandem, and
- Design lane load.

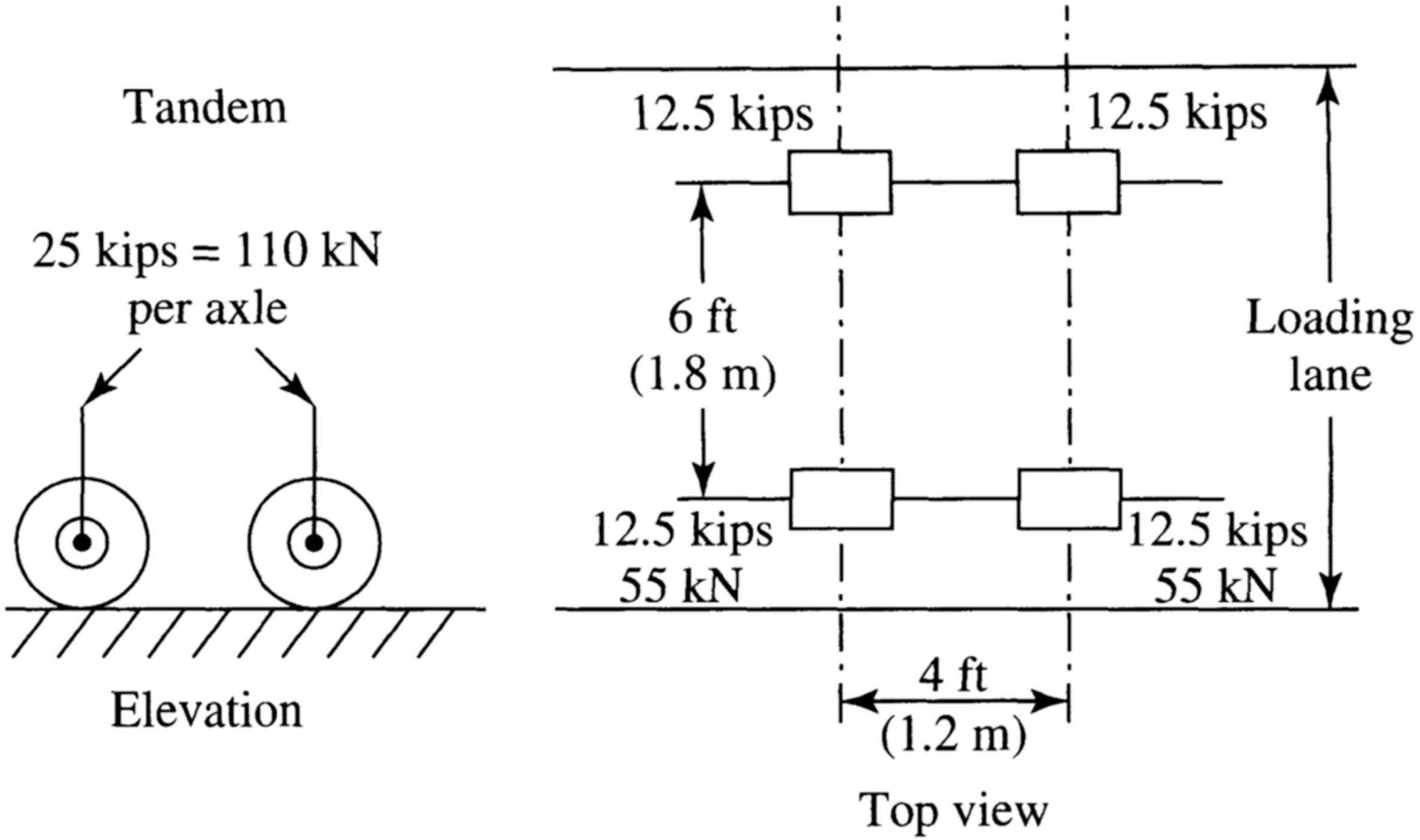
Each design lane under consideration shall be occupied by either the design truck or tandem, coincident with the lane load, where applicable. The loads shall be assumed to occupy 10.0 ft transversely within a design lane.

## DESIGN TRUCK LOAD



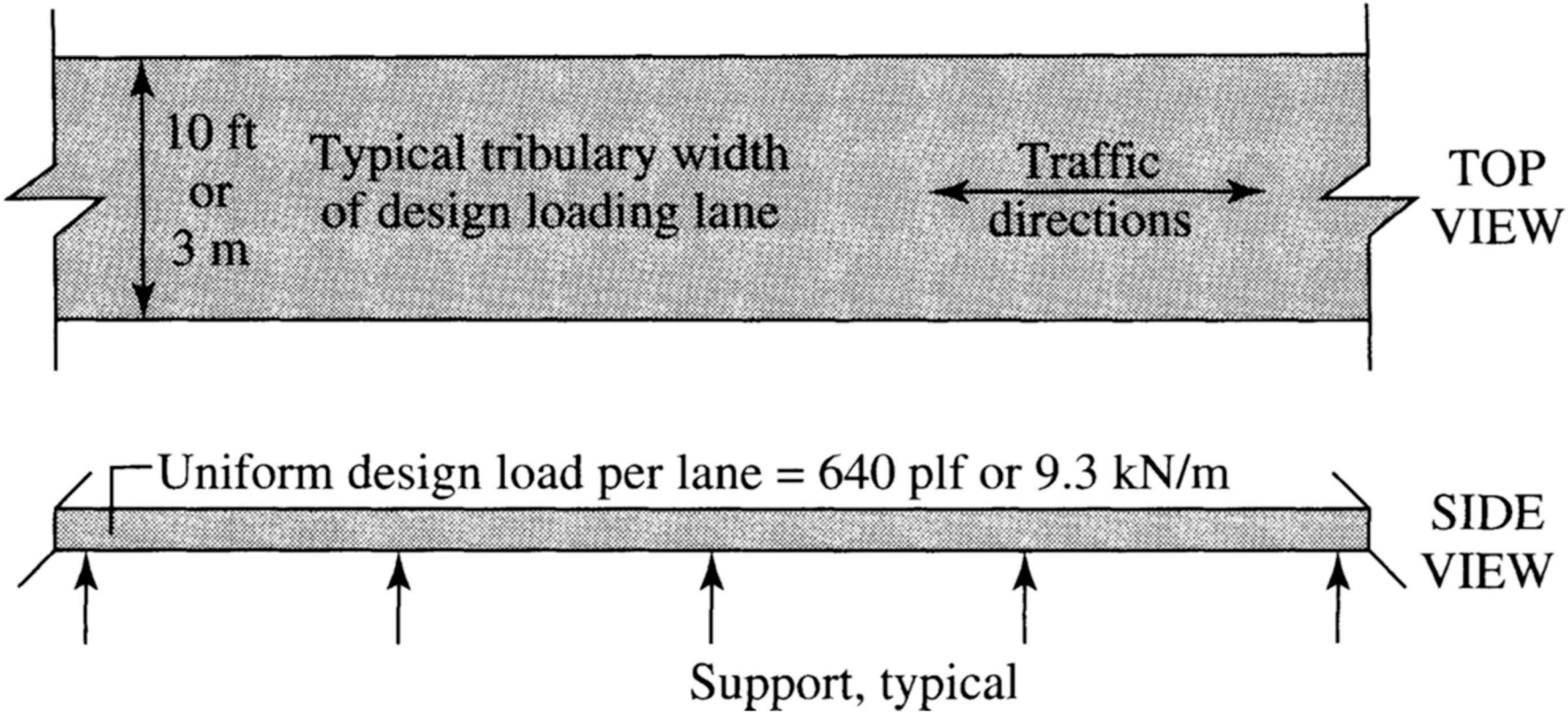

**Truck load is subjected to dynamic allowance (impact)**

## DESIGN TANDEM LOAD




**TANDEM load is subjected to dynamic allowance (impact)**

## DESIGN LANE LOAD




**LANE load is NOT subjected to dynamic allowance (impact)**

# COMBINATION OF LOADS: AASHTO 2012, Sec. 3.4.1

**Design load is obtained by combining different basic loads with appropriate factors for various combination groups**

The total factored force effect shall be taken as:

$$Q = \Sigma \eta_i \gamma_i Q_i \qquad (3.4.1\text{-}1)$$

where:

$\eta_i$ = load modifier specified in Article 1.3.2
$Q_i$ = force effects from loads specified herein
$\gamma_i$ = load factors specified in Tables 3.4.1-1 and 3.4.1-2

# COMBINATION OF LOADS: AASHTO 2012, Sec. 3.4.1

## LIMIT STATES:

**Strength I**—Basic load combination relating to the normal vehicular use of the bridge without wind.

**Strength II**—Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.

**Strength III**—Load combination relating to the bridge exposed to wind velocity exceeding 55 mph.

**Strength IV**—Load combination relating to very high dead load to live load force effect ratios.

**Strength V**—Load combination relating to normal vehicular use of the bridge with wind of 55 mph velocity.

# COMBINATION OF LOADS: AASHTO 2012, Sec. 3.4.1

## LIMIT STATES:

**Extreme Event I**—Load combination including earthquake. The load factor for live load $\gamma_{EQ}$, shall be determined on a project-specific basis.

**Extreme Event II**—Load combination relating to ice load, collision by vessels and vehicles, check floods, and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load, *CT*. The cases of check floods shall not be combined with *BL*, *CV*, *CT*, or *IC*.

**Fatigue I**—Fatigue and fracture load combination related to infinite load-induced fatigue life.

**Fatigue II**—Fatigue and fracture load combination related to finite load-induced fatigue life.

# COMBINATION OF LOADS: AASHTO 2012, Sec. 3.4.1

## LIMIT STATES:

**Service I**—Load combination relating to the normal operational use of the bridge with a 55 mph wind and all loads taken at their nominal values. Also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders. This load combination should also be used for the investigation of slope stability.

**Service II**—Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load.

**Service III**—Load combination for longitudinal analysis relating to tension in pre-stressed concrete superstructures with the objective of crack control and to principal tension in the webs of segmental concrete girders.

**Service IV**—Load combination relating only to tension in pre-stressed concrete columns with the objective of crack control.

# COMBINATION OF LOADS: AASHTO 2012, Sec. 3.4.1

**Table 3.4.1-1—Load Combinations and Load Factors**

| Load Combination Limit State | *DC DD DW EH EV ES EL PS CR SH* | *LL IM CE BR PL LS* | *WA* | *WS* | *WL* | *FR* | *TU* | *TG* | *SE* | Use One of These at a Time | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | *EQ* | *BL* | *IC* | *CT* | *CV* |
| Extreme Event I | $\gamma_p$ | γEQ | 1.00 | — | — | 1.00 | — | — | — | 1.00 | — | — | — | — |
| Extreme Event II | $\gamma_p$ | 0.50 | 1.00 | — | — | 1.00 | — | — | — | — | 1.00 | 1.00 | 1.00 | 1.00 |
| Service I | 1.00 | 1.00 | 1.00 | 0.30 | 1.0 | 1.00 | 1.00/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | — | — | — | — | — |
| Service II | 1.00 | 1.30 | 1.00 | — | — | 1.00 | 1.00/1.20 | — | — | — | — | — | — | — |
| Service III | 1.00 | 0.80 | 1.00 | — | — | 1.00 | 1.00/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | — | — | — | — | — |
| Service IV | 1.00 | — | 1.00 | 0.70 | — | 1.00 | 1.00/1.20 | — | 1.0 | — | — | — | — | — |
| Fatigue I—*LL, IM & CE* only | — | 1.50 | — | — | — | — | — | — | — | — | — | — | — | — |
| Fatigue II—*LL, IM & CE* only | — | 0.75 | — | — | — | — | — | — | — | — | — | — | — | — |

# COMBINATION OF LOADS: AASHTO 2012, Sec. 3.4.1

**Table 3.4.1-1—Load Combinations and Load Factors**

| Load Combination Limit State | DC DD DW EH EV ES EL PS CR SH | LL IM CE BR PL LS | WA | WS | WL | FR | TU | TG | SE | Use One of These at a Time: EQ | BL | IC | CT | CV |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Strength I (unless noted) | $\gamma_p$ | 1.75 | 1.00 | — | — | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | — | — | — | — | — |
| Strength II | $\gamma_p$ | 1.35 | 1.00 | — | — | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | — | — | — | — | — |
| Strength III | $\gamma_p$ | — | 1.00 | 1.40 | — | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | — | — | — | — | — |
| Strength IV | $\gamma_p$ | — | 1.00 | — | — | 1.00 | 0.50/1.20 | — | — | — | — | — | — | — |
| Strength V | $\gamma_p$ | 1.35 | 1.00 | 0.40 | 1.0 | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | — | — | — | — | — |

## Strength-I Combination:

$$\gamma_p(DC) + \gamma_p(DW) + 1.75(LL)(1+IM/100)_{\text{Truck/Tandem}} + 1.75(LL)_{\text{Lane}} + 1.75(PL)$$

# COMBINATION OF LOADS: AASHTO 2012, Sec. 3.4.1

**Table 3.4.1-2—Load Factors for Permanent Loads, $\gamma_p$**

| Type of Load, Foundation Type, and Method Used to Calculate Downdrag | | Load Factor | |
|---|---|---|---|
| | | Maximum | Minimum |
| *DC*: Component and Attachments | | 1.25 | 0.90 |
| *DC:* Strength IV only | | 1.50 | 0.90 |
| *DD*: Downdrag | Piles, α Tomlinson Method | 1.4 | 0.25 |
| | Piles, λ Method | 1.05 | 0.30 |
| | Drilled shafts, O'Neill and Reese (1999) Method | 1.25 | 0.35 |
| *DW*: Wearing Surfaces and Utilities | | 1.50 | 0.65 |
| *EH*: Horizontal Earth Pressure | | | |
| • Active | | 1.50 | 0.90 |
| • At-Rest | | 1.35 | 0.90 |
| • *AEP* for anchored walls | | 1.35 | N/A |
| *EL*: Locked-in Construction Stresses | | 1.00 | 1.00 |

## Strength-I Combination:

$$\gamma_p(DC) + \gamma_p(DW) + 1.75(LL)(1+IM/100)_{\text{Truck/Tandem}} + 1.75(LL)_{\text{Lane}} + 1.75(PL)$$

$$= 1.25(DC) + 1.5(DW) + 1.75(LL)(1+IM/100)_{\text{Truck/Tandem}} + 1.75(LL)_{\text{Lane}} + 1.75(PL)$$

## Dynamic Effect of Live Load (for Truck or Tandem)
## IMPACT ALLOWANCE

- The term impact as ordinarily used in structural design refers to the dynamic effect of a suddenly applied load.
- In the building of a structure, the materials are added slowly; people entering a building are also considered a gradual loading. Dead loads are static loads; i.e., they have no effect other than weight.
- Live loads may be either static or they may have a dynamic effect. Any live load that can have a dynamic effect should be increased by an impact factor. While a dynamic analysis of a structure could be made, such a procedure is unnecessary in ordinary design. Thus, empirical formulas and impact factors are usually used.
- For highway bridge design, impact is always to be considered. AASHTO prescribes empirically that the static effect of live load be multiplied by a factor

$$(1 + IM/100)$$

to take into account the dynamic effect of live load.

## Dynamic Effect of Live Load (for Truck or Tandem)
## IMPACT ALLOWANCE

**Table 3.6.2.1-1—Dynamic Load Allowance, *IM***

| Component | IM |
|---|---|
| Deck Joints—All Limit States | 75% |
| All Other Components: | |
| • Fatigue and Fracture Limit State | 15% |
| • All Other Limit States | 33% |

**The dynamic load allowance shall not be applied to pedestrian loads or to the design lane load.**

## COMBINATION OF LOADS

# For the present case

***DC*** = Self weight of structural components

***DW***= Weight of wearing course

***LL*** = Lane load with vehicle or tandem

***IM*** = Impact effect of vehicle or tandem load

***PL*** = Pedestrian load

$$\gamma_p(DC) + \gamma_p(DW) + 1.75(LL)(1+IM/100)_{\text{Truck/Tandem}} + 1.75(LL)_{\text{Lane}} + 1.75(PL)$$

$$= 1.25(DC) + 1.5(DW) + 1.75(LL)(1+IM/100)_{\text{Truck/Tandem}} + 1.75(LL)_{\text{Lane}} + 1.75(PL)$$

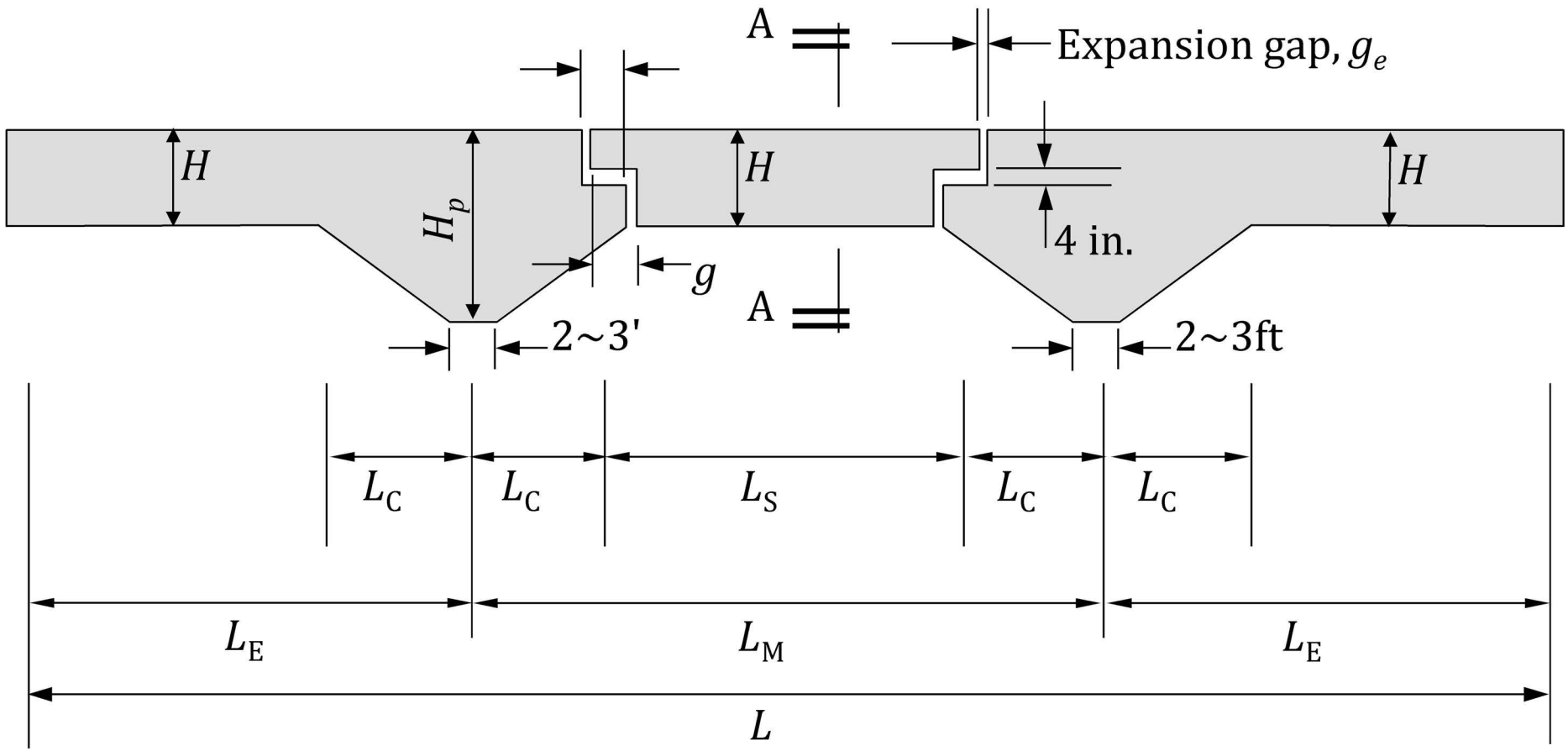
## APPLICATION OF BRIDGE LIVE LOADS

1. Standard lane width: 12 ft, Load occupies 10 ft width across lane.

2. Fractional lanes not permitted.

3. For total bridge load: lane loads may be reduced as follows:

| | | |
|---|---|---|
| 1 or 2 lane bridge: | No reduction | Abutment load, Pier load etc. |
| 3 lanes: | 90 percent | |
| 4 or more lanes: | 75 percent | |

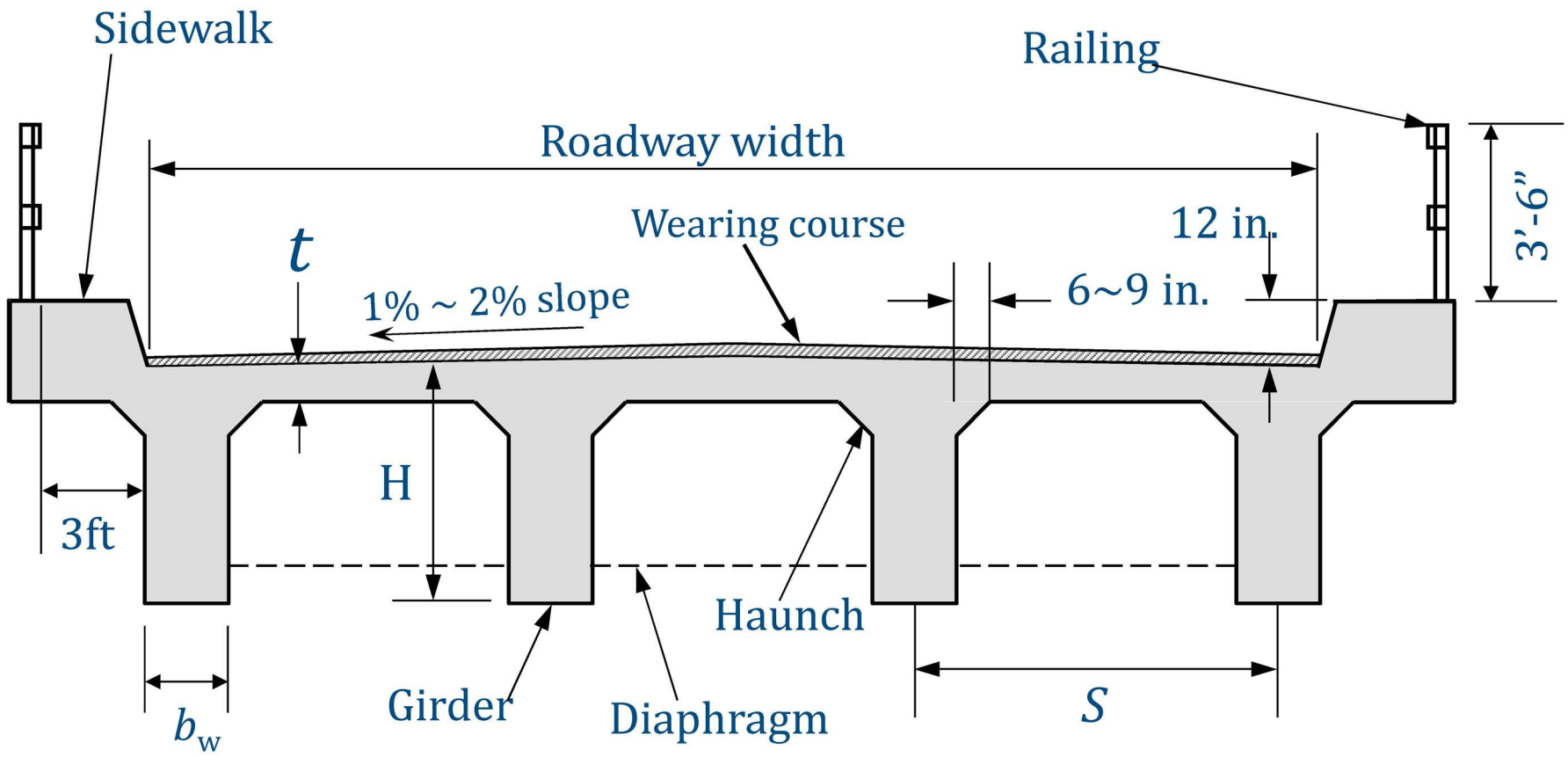



**Total span:** $L$
**End span:** $L_E$
**Middle span:** $L_M$
**Suspended span:** $L_S$
**Cantilever span:** $L_C$

## Assumed Relations

$L_M = 1.4\, L_E$
$L_C = 0.3\, L_S$

$H \geq 0.07 L_S$ (TABLE 2.5.2.6.3-1, AASHTO 2012)




Assume

$b_W = H/4 \sim H/3 \geq 18"$

$t \cong S/12 \geq 7"$

Typical section A-A

# DESIGN DATA FOR STUDENTS

## PER STUDENT DATA

| Student Sl | Total Span, *L ft* | $f_c'$ (ksi) | $f_y$ (ksi) | Student Sl | Total Span, *L ft* | $f_c'$ (ksi) | $f_y$ (ksi) | Student Sl | Total Span, *L ft* | $f_c'$ (ksi) | $f_y$ (ksi) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 250 | 4 | 72 | 24 | 319 | 5 | 72 | 47 | 388 | 4 | 60 |
| 2 | 253 | 4 | 72 | 25 | 322 | 5 | 72 | 48 | 391 | 4 | 60 |
| 3 | 256 | 4 | 60 | 26 | 325 | 5 | 72 | 49 | 394 | 4 | 60 |
| 4 | 259 | 4 | 60 | 27 | 328 | 5 | 72 | 50 | 397 | 4 | 60 |
| 5 | 262 | 4 | 60 | 28 | 331 | 5 | 72 | 51 | 400 | 4 | 60 |
| 6 | 265 | 4 | 60 | 29 | 334 | 5 | 72 | 52 | 403 | 5 | 72 |
| 7 | 268 | 4 | 60 | 30 | 337 | 5 | 72 | 53 | 406 | 5 | 72 |
| 8 | 271 | 4 | 60 | 31 | 340 | 5 | 72 | 54 | 409 | 5 | 72 |
| 9 | 274 | 4 | 60 | 32 | 343 | 5 | 72 | 55 | 412 | 5 | 72 |
| 10 | 277 | 4 | 60 | 33 | 346 | 5 | 72 | 56 | 415 | 5 | 72 |
| 11 | 280 | 4 | 60 | 34 | 349 | 5 | 72 | 57 | 418 | 5 | 72 |
| 12 | 283 | 4 | 60 | 35 | 352 | 4 | 60 | 58 | 421 | 5 | 72 |
| 13 | 286 | 4 | 60 | 36 | 355 | 4 | 60 | 59 | 424 | 5 | 72 |
| 14 | 289 | 4 | 60 | 37 | 358 | 4 | 60 | 60 | 427 | 5 | 72 |
| 15 | 292 | 4 | 60 | 38 | 361 | 4 | 60 | 61 | 430 | 5 | 72 |
| 16 | 295 | 4 | 60 | 39 | 364 | 4 | 60 | 62 | 433 | 5 | 72 |
| 17 | 298 | 4 | 60 | 40 | 367 | 4 | 60 | 63 | 436 | 5 | 72 |
| 18 | 301 | 5 | 72 | 41 | 370 | 4 | 60 | 64 | 439 | 5 | 72 |
| 19 | 304 | 5 | 72 | 42 | 373 | 4 | 60 | 65 | 442 | 5 | 72 |
| 20 | 307 | 5 | 72 | 43 | 376 | 4 | 60 | 66 | 445 | 5 | 72 |
| 21 | 310 | 5 | 72 | 44 | 379 | 4 | 60 | 67 | 448 | 5 | 72 |
| 22 | 313 | 5 | 72 | 45 | 382 | 4 | 60 | 68 | 451 | 5 | 72 |
| 23 | 316 | 5 | 72 | 46 | 385 | 4 | 60 | 69 | 454 | 5 | 72 |

# DESIGN DATA FOR STUDENTS

## COMMON DATA

Wearing course, $w_{wc}$ = 30 psf
Width of side walk = 3'-6“

## DESIGN CODE

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 6TH ED. 2012

## Lane width

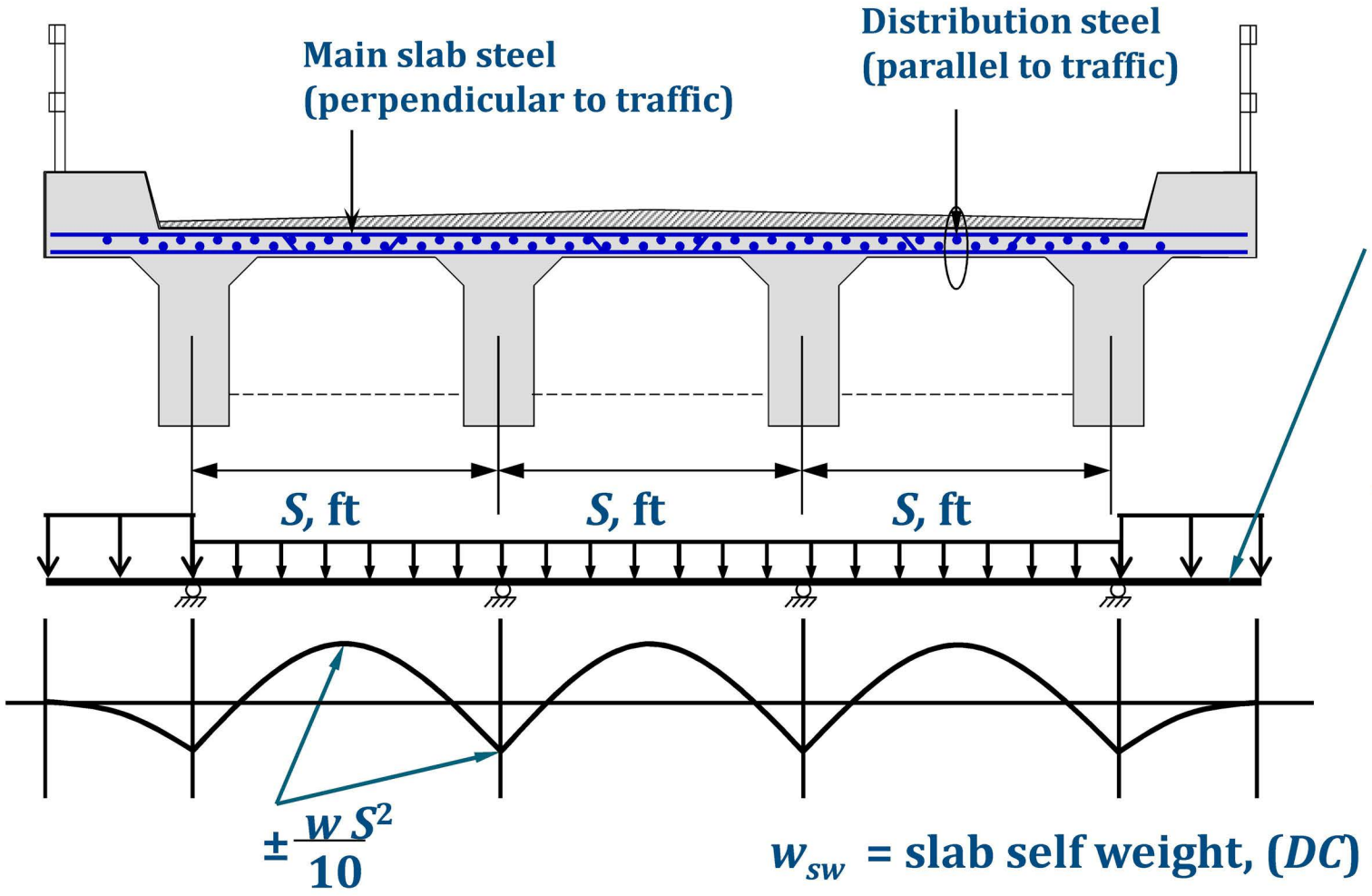
| Sec-A | Sec-B | Sec-C |
|---|---|---|
| 14’ | 13’ | 12’ |

Number of lanes = 2
Concrete clear cover = Beam 1.5”, Slab: 1.0”

## Girder depth at pier

$H_p$ = 2.0$H$ for $L$ < 350’,
= 1.5$H$ for $L \geq$ 350’

# DECK SLAB DESIGN: DEAD LOAD




Deck slab is assumed as a continuous beam spanning over bridge girders. The bridge girders are assumed as rigid continuous support

Assume a unit width of slab strip.

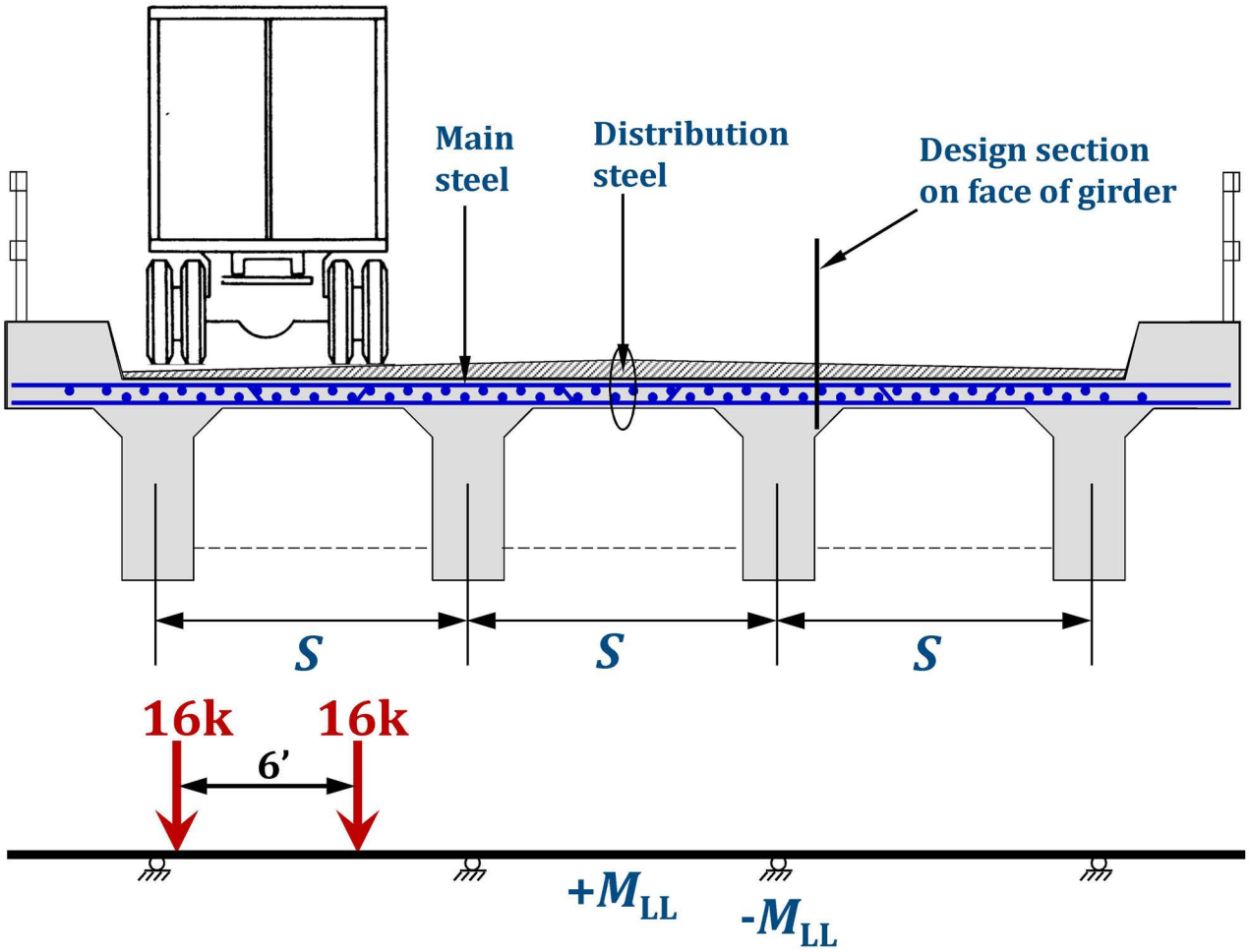
$w_{sw}$ = slab self weight, $(DC) = t/12 \times 150^{\#/'}$ psf

$w_{wc}$ = wearing course, $(DW)$ psf

$M_{DC} = w_{sw} \times S^2/10$ lb-ft

$M_{DW} = w_{wc} \times S^2/10$ lb-ft

# DECK SLAB DESIGN: VEHICLE LOAD




**Detailed analysis can be performed based on influence line to determine the maximum effect.**

**Alternatively, Table A4-1 in Appendix A4 of AASHTO 2012 can be used.**

# DECK SLAB DESIGN: VEHICLE LOAD

**Table A4-1 in Appendix A4 of AASHTO 2012, page 4-98**
**Important Assumptions…**

- Multiple presence factors and the dynamic load allowance are included in the tabulated values.
- The moments are applicable for decks supported on at least three girders and having a width of not less than 14.0 ft between the centerlines of the exterior girders.
- For each combination of girder spacing and number of girders, the following two cases of overhang width were considered:
  - Minimum total overhang width of 21.0 in. measured from the center of the exterior girder, and
  - Maximum total overhang width equal to the smaller of 0.625 times the girder spacing and 6.0 ft. A railing system width of 21.0 in. was used to determine the clear overhang width. For other widths of railing systems, the difference in the moments in the interior regions of the deck is expected to be within the acceptable limits for practical design. The moments do not apply to the deck overhangs and the adjacent regions of the deck that need to be designed taking into account the provisions of Article A13.4.1.

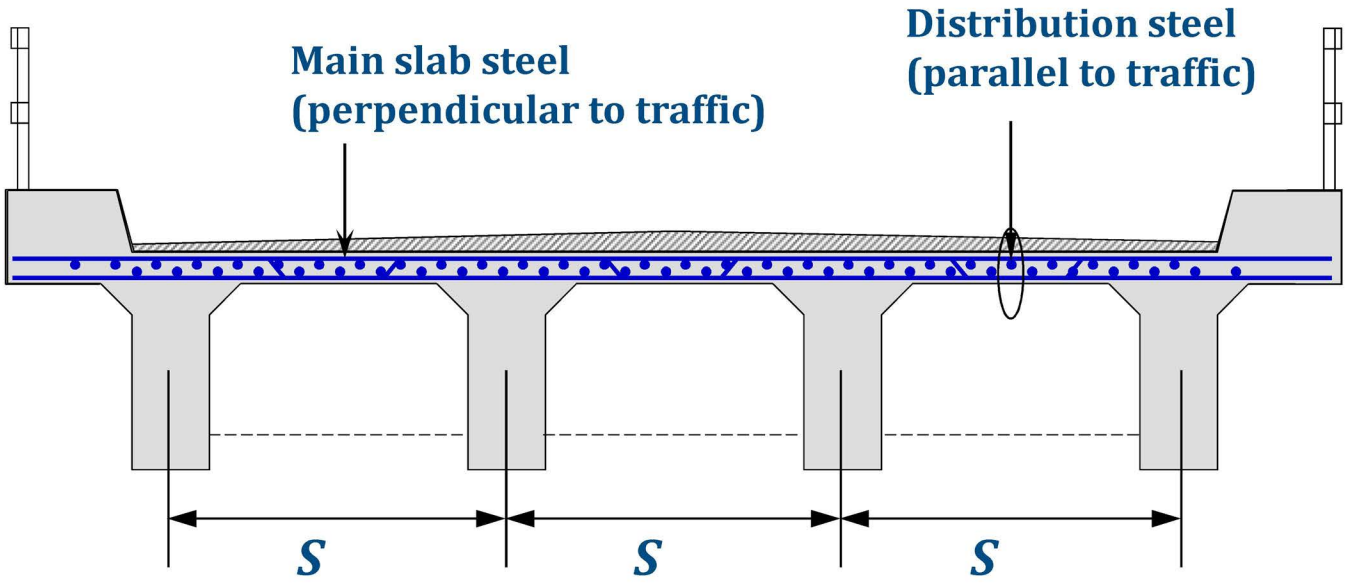
# DECK SLAB DESIGN: VEHICLE LOAD



**Table A4-1—Maximum Live Load Moments per Unit Width, kip-ft/ft**

| S | Positive Moment | Negative Moment | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | Distance from CL of Girder to Design Section for Negative Moment | | | | | | |
| | | 0.0 in. | 3 in. | 6 in. | 9 in. | 12 in. | 18 in. | 24 in. |
| 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 |
| 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 |

# DECK SLAB DESIGN: VEHICLE LOAD




**General Load Combination**

$$1.25(DC) + 1.5(DW) + 1.75(LL)(1+IM/100)_{\text{Truck/Tandem}} + 1.75(LL)_{\text{Lane}} + 1.75(PL)$$

Design slab moment, $M = 1.25\,M_{DC} + 1.5\,M_{DW} + 1.75\,M_{LL}$ $\quad [\rightarrow M_{\text{STRENGTH}}]$

Where $M_{LL}$ is the live load slab moment from Table A4-1 which includes the impact effect.

$M_{\text{SERVICE}} = M_{DC} + M_{DW} + M_{LL}$ [ required for crack control calculations]