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36.1 Design Method

36.1.1 Design Requirements

All new box culverts are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*.

36.1.2 Rating Requirements

The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* covers rating of concrete box culverts. Refer to 45.8 for additional guidance on load rating various types of culverts.

36.1.3 Standard Permit Design Check

New structures are also to be checked for strength for the 190 kip Wisconsin Standard Permit Vehicle (Wis-SPV), with a single lane loaded, multiple presence factor equal to 1.0, and a live load factor (γ_{LL}) as shown in Table 45.3-3. See 45.12 for the configuration of the Wis-SPV. The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface (where applicable – future wearing surface loads are only applied to box culverts with no fill). When applicable, this truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the culvert, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM. The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans.

36.2 General

Box culverts are reinforced concrete closed rigid frames which must support vertical earth and truck loads and lateral earth pressure. They may be either single or multi-cell. The most common usage is to carry water under roadways, but they are frequently used for pedestrian or cattle underpasses.

Box culverts used to carry water should consider the following items:

- Hydraulic and other requirements at the site determine the required height and area of the box. Hydraulic design of box culverts is described in Chapter 8.
- Once the required height and area is determined, the selection of a single or multi-cell box is determined entirely from economics. Barrel lengths are computed to the nearest 6 inches. For multi-cell culverts the cell widths are kept equal.
- A minimum vertical opening of 5 feet is desirable for cleaning purposes.

Pedestrian underpasses should consider the following items:

- The minimum opening for pedestrian underpasses is 8 feet high by 10 feet wide. However, when considering maintenance and emergency vehicles or bicyclists the minimum opening should be 10 feet high by 12 feet wide. For additional guidance refer to the Wisconsin Bicycle Facility Design Handbook and the FDM.
- The entire top and 1 foot below the bottom of the top slab should be waterproofed.
- The top of the bottom slab should be sloped with a 1% normal crown to minimize moisture collecting on the travel path. Additionally, 0.5% to 1% longitudinal slope for drainage is recommended.
- Flared wings are recommended at openings. For long underpasses, lighting systems (recessed lights and skylights) should be considered, as well. For additional guidance on user's comfort, safety measures, and lighting refer to the Wisconsin Bicycle Facility Design Handbook.

Cattle underpasses should consider the following items:

- The minimum size for cattle underpasses is 6 feet high by 5 feet wide.
- Consider providing a minimum longitudinal slope of 1%, desirable 3%, to allow for flushing, but not so steep that the stock will slip. Slopes steeper than 5% should be avoided.
- For additional guidance refer to the FDM.

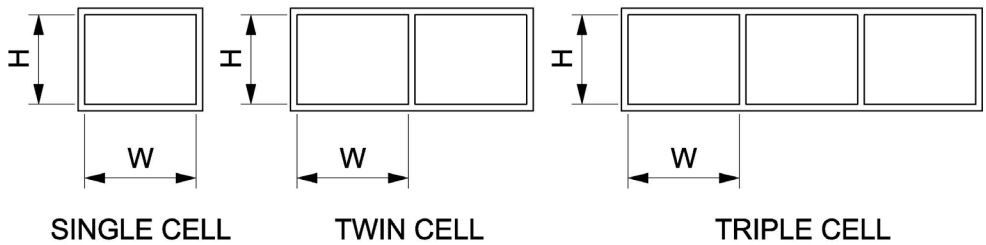


Figure 36.2-1
Typical Cross Sections

36.2.1 Material Properties

The properties of materials used for concrete box culverts are as follows:

f'_c = specified compressive strength of concrete at 28 days, based on cylinder tests

= 3.5 ksi for concrete in box culverts

f_y = 60 ksi, specified minimum yield strength of reinforcement (Grade 60)

E_s = 29,000 ksi, modulus of elasticity of steel reinforcement **LRFD [5.4.3.2]**

E_c = modulus of elasticity of concrete in box **LRFD [C5.4.2.4]**
= $(33{,}000)(K_1)(w_C)^{1.5}(f'_C)^{1/2}$ = 3586 ksi

Where:

K_1 = 1.0

W_C = 0.15 kcf, unit weight of concrete

n = Es / Ec = 8, modular ratio **LRFD [5.6.1]**

36.2.2 Bridge or Culvert

Occasionally, the waterway opening(s) for a highway-stream crossing can be provided for by either culvert(s) or bridge(s). Consider the hydraulics of the highway-stream crossing system in choosing the preferred design from the available alternatives. Estimates of life cycle costs and risks associated with each alternative help indicate which structure to select. Consider construction costs, maintenance costs, and risks of future costs to repair flood damage. Other considerations which may influence structure-type selection are listed in Table 36.2-1.

Bridges	
Advantages	Disadvantages
Less susceptible to clogging with drift, ice and debris	Require more structural maintenance than culverts
Waterway width increases with rising water surface until water begins to submerge structure	Piers and abutments susceptible to scour failure
Natural bottom for waterway	Susceptible to ice and frost forming on deck
Culverts	
Grade rises and widening projects sometimes can be accommodated by extending culvert ends	Silting in multiple barrel culverts may require periodic cleanout
Minimum structural maintenance	No increase in waterway area as stage rises above top of culvert
Usually easier and quicker to build than bridges	May clog with drift, debris or ice

Table 36.2-1
Advantages/Disadvantages of Structure Type

36.2.3 Staged Construction for Box Culverts

The inconvenience to the traveling public has often led to staged construction projects. Box culverts typically work well with staged construction. Any cell joint can be used for a staging joint. When the construction staging line cannot be determined in design to locate a cell joint, a contractor placed construction joint can be done with an extra set of dowel bars and the contractor field cutting the longitudinal bars.

36.3 Limit States Design Method

36.3.1 LRFD Requirements

For box culvert design, the component dimensions and the size and spacing of reinforcement shall be selected to satisfy the following equation for all appropriate limit states, as presented in **LRFD [1.3.2.1]:**

$$Q = \sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r$$

Where:

η_i	=	Load modifier (a function of η_D, η_R, and η_i)
γ_i	=	Load factor
Q_i	=	Force effect: moment, shear, stress range or deformation caused by applied loads
Q	=	Total factored force effect
ϕ	=	Resistance factor
R_n	=	Nominal resistance: resistance of a component to force effects
R_r	=	Factored resistance = ϕR_n

See 17.2.2 for load modifier values.

36.3.2 Limit States

The Strength I Limit State is used to design reinforcement for flexure and checking shear in the slabs and walls, **LRFD [12.5.3]**. The Service I Limit State is used for checking reinforcement for crack control criteria, **LRFD [12.5.2]**, and checking settlement of the box culvert as shown in 36.8.1.

Per **LRFD [C12.5.3, 5.5.3],** buried structures have been shown not to be controlled by fatigue.

WisDOT Policy Item:

Fatigue criteria are not required on any reinforced concrete box culverts, with or without fill on the top slab of the culvert. This policy item is based on the technical paper titled "Fatigue Evaluation for Reinforced Concrete Box Culverts" by H Hany Maximos, Ece Erdogmus, and Maher Tadros, published in the ACI Structural Journal, January/February 2010.

36.3.3 Load Factors

In accordance with **LRFD [Table 3.4.1-1 and Table 3.4.1-2]**, the following Strength I load factors, γ_{st}, and Service I load factors, γ_{s1}, shall be used for box culvert design:

		Strength I Load Factor, γ_{st}		Service I Load Factor, γ_{s1}
Type of Load		Max.	Min.	
Dead Load-Components	DC	1.25	0.90	1.0
Dead Load-Wearing Surface	DW	1.50	0.65	1.0
Vertical Earth Pressure	EV	1.35	0.90	1.0
Horizontal Earth Pressure	EH	1.35	0.50[1]	1.0
Live Load Surcharge	LS	1.75	1.75	1.0
Live Load + IM	LL+IM	1.75	1.75	1.0

[1]Per **LRFD [3.11.7]**, for culverts where earth pressure may reduce effects caused by other loads, a 50% reduction may be used, but not combined with the minimum load factor specified in **LRFD [Table 3.4.1-2]**.

36.3.4 Strength Limit State

Strength I Limit State shall be applied to ensure that strength and stability are provided to resist the significant load combinations that a structure is expected to experience during its design life **LRFD [1.3.2.4]**.

36.3.4.1 Factored Resistance

The resistance factor, ϕ, is used to reduce the computed nominal resistance of a structural element. This factor accounts for the variability of material properties, structural dimensions and workmanship, and uncertainty in prediction of resistance.

The resistance factors, ϕ, for reinforced concrete box culverts for the Strength Limit State per **LRFD [Table 12.5.5-1]** are as shown below:

Structure Type	Flexure	Shear
Cast-In-Place	0.90	0.85
Precast	1.00	0.90
Three-Sided	0.95	0.90

36.3.4.2 Moment Capacity

For rectangular sections, the nominal moment resistance, M_n, per **LRFD [5.6.3.2.3]** (tension reinforcement only) equals:

$$M_n = A_s f_s \left(d_s - \frac{a}{2}\right)$$

The factored resistance, M_r, or moment capacity per **LRFD [5.6.3.2.1]**, shall be taken as:

$$M_r = \phi M_n = \phi A_s f_s \left(d_s - \frac{a}{2}\right)$$

For additional information on concrete moment capacity, including stress and strain assumptions used, refer to 18.3.3.2.1.

The location of the design moment will consider the haunch dimensions in accordance with **LRFD [12.11.5.2]**. No portion of the haunch shall be considered in adding to the effective depth of the section.

36.3.4.3 Shear Capacity

Per **LRFD [12.11.4.1]**, shear in culverts shall be investigated in conformance with **LRFD [5.12.7.3]**. The location of the critical section for shear for culverts with haunches shall be determined in conformance with **LRFD [C5.12.8.6.1]** and shall be taken at a distance d_v from the end of the haunch.

36.3.4.3.1 Depth of Fill Greater than or Equal to 2.0 ft.

The shear resistance of the concrete, V_c, for <u>slabs</u> of box culverts with 2.0 feet or more of fill, for one-way action per **LRFD [5.12.7.3]** shall be determined as:

$$V_c = \left(0.0676\lambda\sqrt{f'_c} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u}\right)bd_e \le 0.126\lambda\sqrt{f'_c}\,bd_e$$

Where:

$$\frac{V_u d_e}{M_u} \le 1$$

Where:

V_c = Shear resistance of the concrete (kip)

A_s = Area of reinforcing steel in the design width (in^2)

d_e	=	Effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)
V_u	=	Factored shear (kip)
M_u	=	Factored moment, occurring simultaneously with V_u (kip-in)
b	=	Design width (in.)
λ	=	Concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

In the absence of shear reinforcing, the nominal shear resistance is equal to the shear resistance of the concrete. The factored resistance, V_r, or shear capacity, per **LRFD [5.7.2.1]** shall be taken as:

$$V_r = \phi V_n = \phi V_c$$

Per **LRFD [5.12.7.3]**, for single-cell box culverts only, Vc for slabs monolithic with walls need not be taken less than:

$$0.0948 \cdot \lambda \sqrt{f'_c}\, b d_e$$

and V_c for slabs simply supported need not be taken less than:

$$0.0791 \cdot \lambda \sqrt{f'_c}\, b d_e$$

The shear resistance of the concrete, V_c, for <u>walls</u> of box culverts with 2.0 feet or more of fill, for one-way action per **LRFD [5.7.3.3]** shall be determined as:

$$V_c = 0.0316 \cdot \beta \lambda \sqrt{f'_c}\, b_v d_v \le 0.25 f'_c\, b_v d_v$$

Where:

V_c	=	Shear resistance of the concrete (kip)
β	=	2.0 (**LRFD [5.7.3.4.1]**)
b_v	=	Effective web width taken as the minimum web width within the depth d_v (in.)
d_v	=	Effective shear depth as determined in **LRFD [5.7.2.8]**. Perpendicular distance between tension and compression resultants. Need not be taken less than the greater of $0.9d_e$ or 0.72h (in.)
λ	=	Concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

36.3.4.3.2 Depth of Fill Less than 2.0 ft

Per **LRFD [5.12.7.3]**, for box culverts with less than 2.0 feet of fill follow **LRFD [5.7]** and **LRFD [5.12.8.6]**.

The shear resistance of the concrete, V_c, for slabs and walls of box culverts with less than 2.0 feet of fill, for one-way action per **LRFD [5.7.3.3]** shall be determined as:

$$V_c = 0.0316 \cdot \beta\lambda\sqrt{f'_c}\, b_v d_v \leq 0.25 f'_c\, b_v d_v$$

With variables defined above in 36.3.4.3.1.

For box culverts where the top slab is an integral part of the wearing surface (depth of fill equal zero) the top slab shall be checked for two-way action, as discussed in 18.3.3.2.2.

36.3.5 Service Limit State

Service I Limit State shall be applied as restrictions on stress, deformation, and crack width under regular service conditions **LRFD [1.3.2.2]**.

36.3.5.1 Factored Resistance

The resistance factor, ϕ, for Service Limit State, is found in **LRFD [1.3.2.1]** and its value is 1.00.

36.3.5.2 Crack Control Criteria

Per **LRFD [12.11.4]**, the provisions of **LRFD [5.6.7]** shall apply to crack width control in box culverts. All reinforced concrete members are subject to cracking under any load condition, which produces tension in the gross section in excess of the cracking strength of the concrete. Provisions are provided for the distribution of tension reinforcement to control flexural cracking.

Crack control criteria does not use a factored resistance, but calculates a maximum spacing for flexure reinforcement based on service load stress in bars, concrete cover and exposure condition.

Crack control criteria shall be applied when the tension in the cross-section exceeds 80% of the modulus of rupture, f_r, specified in **LRFD [5.4.2.6]** for Service I Limit State. The spacing, s, (in inches) of mild steel reinforcement in the layer closest to the tension face shall satisfy:

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad \text{(in.)}$$

Bar spacing, s, need not be less than 5 in. for control of flexural cracking **LRFD [5.6.7]**

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

Where:

γ_e	=	Exposure factor (1.0 for Class 1 exposure condition, 0.75 for Class 2 exposure condition, see **LRFD [5.6.7]** for guidance)
d_c	=	Thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.). For calculation purposes, d_c, need not be taken greater than 2 in. plus the bar radius
f_{ss}	=	Tensile stress in steel reinforcement at the service limit state (ksi) $\leq$ 0.6 f_y
h	=	Overall thickness or depth of the component (in.)

WisDOT Policy Item:

A class 1 exposure factor, γ_e = 1.0, shall be used for all cases for cast-in-place box culverts except for the top steel in the top slab of a box culvert with zero fill, where a class 2 exposure factor, γ_e = 0.75, shall be used.

36.3.6 Minimum Reinforcement Check

Per **LRFD [12.11.5.3]**, the area of reinforcement, A_s, in the box culvert cross-section should be checked for minimum reinforcement requirements per **LRFD [5.6.3.3]**.

The area of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, or moment capacity at least equal to the lesser of:

M_{cr} (or) $1.33M_u$

$M_{cr} = \gamma_3 (\gamma_1 f_r) S = 1.1 f_r (I_g / c)$; $S = I_g / c$

Where:

γ_1	=	1.6 flexural cracking variability factor
γ_3	=	0.67 ratio of minimum yield strength to ultimate tensile strength; for A615 Grade 60 reinforcement
f_r	=	$0.24\lambda\sqrt{f'_c}$ Modulus of rupture (ksi) **LRFD [5.4.2.6]**
I_g	=	Gross moment of inertia (in^4)

c	=	½ *effective slab thickness (in.)
M_u	=	Total factored moment using Strength I Limit State (kip-in)
M_{cr}	=	Cracking strength moment (kip-in)
λ	=	concrete density modification factor ; for normal weight conc. = 1.0, **LRFD [5.4.2.8]**

The factored resistance, M_r or moment capacity, shall be calculated as in 36.3.4.2 and shall satisfy:

$$M_r \geq \min(M_{cr} \text{ or } 1.33\, M_u)$$

36.3.7 Minimum Spacing of Reinforcement

Per **LRFD [5.10.3.1]**, the clear distance between parallel bars in a layer shall not be less than:

- 1.5 times the nominal diameter of the bars
- 1.5 times the maximum size of the course aggregate
- 1.5 inches

36.3.8 Maximum Spacing of Reinforcement

Per **LRFD [5.10.3.2]**, the spacing of reinforcement in walls and slabs shall not exceed:

- 1.5 times the thickness of the member (3.0 times for temperature and shrinkage)
- 18 inches

36.3.9 Edge Beams

Per **LRFD [12.11.2.1]**, for cast-in-place box culverts, and for precast box culverts with top slabs having span to thickness ratios (s/t) > 18 or segment lengths < 4.0 feet, edge beams shall be provided as specified in **LRFD [4.6.2.1.4]** as follows:

- At ends of culvert runs where wheel loads travel within 24.0 inches from the end of the culvert
- At expansion joints of cast-in-place culverts where wheel loads travel over or adjacent to the expansion joint

The edge beam provisions are only applicable for culverts with less than 2.0 ft of fill, **LRFD [C12.11.2.1]**.

36.4 Design Loads

36.4.1 Self-Weight (DC)

Include the structure self-weight based on a unit weight of concrete of 0.150 kcf. When there is no fill on the top slab of the culvert, the top slab thickness includes a ½" wearing surface. The weight of the wearing surface is included in the design, but its thickness is not included in the section properties of the top slab.

36.4.2 Future Wearing Surface (DW)

If the fill depth over the culvert is greater than zero, the weight of the future wearing surface shall be taken as zero. If there is no fill depth over the culvert, the weight of the future wearing surface shall be taken as 20 psf. This load is designated as, DW, dead load of wearing surfaces and utilities, for application of load factors and limit state combinations.

36.4.3 Vertical and Horizontal Earth Pressure (EH and EV)

WisDOT Policy Item:

Box Culverts are assumed to be rigid frames. Use Vertical Earth Pressure load factors for rigid frames, in accordance with **LRFD [Table 3.4.1-2]**.

The weight of soil above the buried structure is taken as 0.120 kcf. A coefficient of lateral earth pressure of 0.5 is used for the lateral pressure from the soil. This coefficient of lateral earth pressure is based on an at-rest condition and an effective friction angle of 30º, **LRFD [3.11.5.2]**. The lateral earth pressure is calculated per **LRFD [3.11.5.1]**:

$$p = k_o \gamma_s z$$

Where:

p	=	Lateral earth pressure (ksf)
k_o	=	Coefficient of at-rest lateral earth pressure
γ_s	=	Unit weight of backfill (kcf)
z	=	Depth below the surface of earth fill or top of roadway pavement (ft)

WisDOT Policy Item:

For modification of earth loads for soil-structure interaction, embankment installations are always assumed for box culvert design, in accordance with **LRFD [12.11.2.2]**.

Soil-structure interaction for vertical earth loads is computed based on **LRFD [12.11.2.2]**. For embankment installations, the total unfactored earth load is:

$$W_E = F_e \gamma_s B_c H$$

In which:

$$F_e = 1 + 0.20 \frac{H}{B_c}$$

Where:

W_E	=	Total unfactored earth load (kip/ft width)
F_e	=	Soil-structure interaction factor for embankment installations (F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section)
γ_s	=	Unit weight of backfill (kcf)
B_c	=	Outside width of culvert, as specified in Figure 36.4-1 (ft)
H	=	Depth of fill from top of culvert to surface of earth fill or top of roadway pavement (ft)

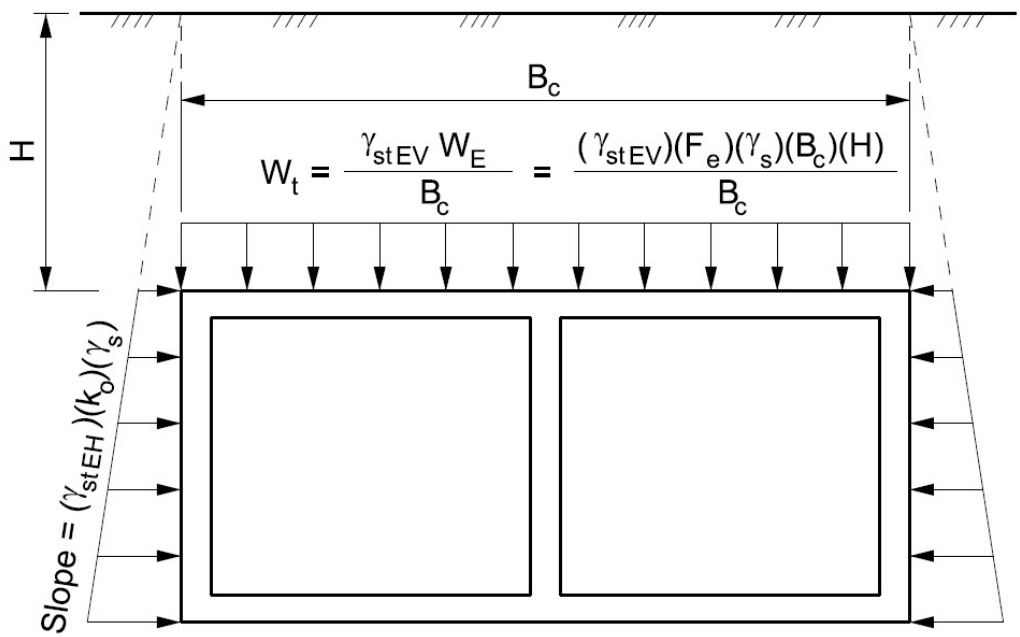


Figure 36.4-1
Factored Vertical and Horizontal Earth Pressures

Where:

W_t	=	Factored earth pressure on top of box culvert (ksf)
γ_{stEV}	=	Vertical earth pressure load factor
γ_{stEH}	=	Horizontal earth pressure load factor
k_o	=	Coefficient of at-rest lateral earth pressure
γ_s	=	Unit weight of backfill (kcf)

Figure 36.4-1 shows the factored vertical and horizontal earth load pressures acting on a box culvert. The soil pressure on the bottom of the box is not shown, but shall be determined for the design of the bottom slab. Note: vertical earth pressures, as well as other loads (e.g. DC and DW), are typically distributed equally over the bottom of the box when determining pressure distributions for the bottom slab. Pressure distributions from a refined analysis is typically not warranted for new culvert designs, but should be considered when evaluating existing culvert sections on culvert extension projects.

36.4.4 Live Load Surcharge (LS)

Per **LRFD [3.11.6.4]**, a live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the distance from top of pavement to bottom of the box culvert.

Per **LRFD [Table 3.11.6.4-1]**, the following equivalent heights of soil for vehicular loading shall be used. The height to be used in the table shall be taken as the distance from the bottom of the culvert to the roadway surface. Use interpolation for heights other than those listed in the table.

Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥ 20.0	2.0

Table 36.4-1
Equivalent Height of Soil for Vehicular Loading

Surcharge loads are computed based on a coefficient of lateral earth pressure times the unit weight of soil times the height of surcharge. A coefficient of lateral earth pressure of 0.5 is used for the lateral pressure from the soil, as discussed in 36.4.3. The uniform distributed load is applied to both exterior walls with the load directed toward the center of the box culvert. The load is designated as, LS, live load surcharge, for application of load factors and limit state combinations. Refer to **LRFD [3.11.6.4]** for additional information regarding live load surcharge.

36.4.5 Water Pressure (WA)

Static water pressure loads are computed when the water height on the outside of the box is greater than zero. The water height is measured from the bottom inside of the box culvert to the water level. The load is designated as, WA, water pressure load, for application of load factors and limit state combinations. Water pressure in culvert barrels is ignored. Refer to **LRFD [3.7.1]** for additional information regarding water pressure.

36.4.6 Live Loads (LL)

Live load consists of the standard AASHTO LRFD trucks and tandem. Per **LRFD [3.6.1.3.3]**, design loads are always axle loads (single wheel loads should not be considered) and the lane load is not used. The depth of fill is measured from top of culvert to surface of earth fill or top roadway pavement.

Where the depth of fill over the box is less than 2 feet, the wheel loads are distributed per **LRFD [4.6.2.10]**. Where the depth of fill is 2 feet or more, the wheel loads shall be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area **LRFD [3.6.1.2.5]**, increased by the live load distribution factor (LLDF) in **LRFD [Table 3.6.1.2.6a-1]**, using the provisions of **LRFD [3.6.1.2.6b-c]**. Where areas from distributed wheel loads overlap at the top of the culvert, the total load is considered as uniformly distributed over the rectangular area (A_{LL}) defined by the outside limits described in **LRFD [3.6.1.2.6b-c]**.

Per **LRFD [3.6.1.2.6a]**, for single-span culverts, the effects of live load may be neglected where the depth of fill is more than 8.0 feet and exceeds the span length. For multiple span culverts, the effects may be neglected where the depth of fill exceeds the distance between inside faces of end walls.

Skew is not considered for design loads.

36.4.6.1 Depth of Fill Less than 2.0 ft.

Where the depth of fill is less than 2.0 ft, follow **LRFD [4.6.2.10]**.

36.4.6.1.1 Case 1 – Traffic Travels Parallel to Span

When the traffic travels primarily parallel to the span, follow **LRFD [4.6.2.10.2]**. Use a single lane and the single lane multiple presence factor of 1.2.

Distribution length perpendicular to the span:

$$E = (96 + 1.44(S))$$

Where:

E = Equivalent distribution width perpendicular to span (in.)

S = Clear span (ft)

The distribution of wheel loads perpendicular to the span for depths of fill less than 2.0 feet is illustrated in Figure 36.4-2.

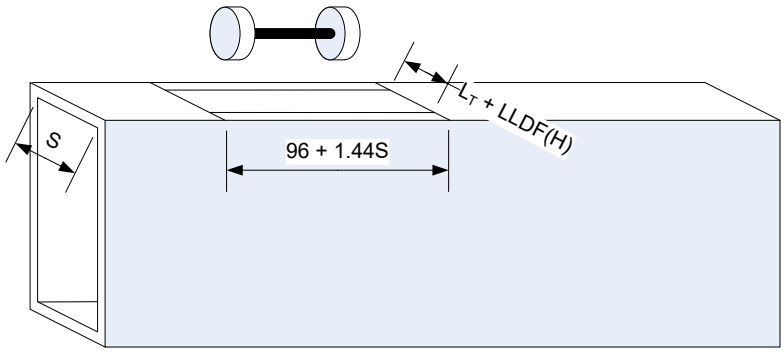


Figure 36.4-2
Distribution of Wheel Loads Perpendicular to Span, Depth of Fill Less than 2.0 feet

Distribution length parallel to the span:

$E_{span} = (L_T + LLDF\ (H))$

Where:

E_{span}	=	Equivalent distribution length parallel to span (in.)
L_T	=	Length of tire contact area parallel to span, as specified in **LRFD [3.6.1.2.5]** (in.)
LLDF	=	Factor for distribution of live load with depth of fill, 1.15, as specified in **LRFD [Table 3.6.1.2.6a-1]**.
H	=	Depth of fill from top of culvert to top of pavement (in.)

The distribution of wheel loads parallel to the span for depths of fill less than 2.0 feet is illustrated in Figure 36.4-3.

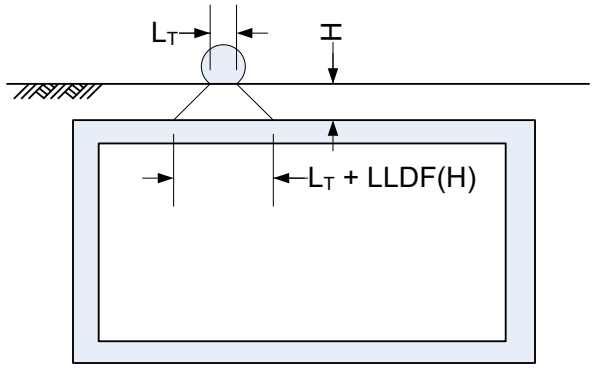


Figure 36.4-3
Distribution of Wheel Loads Parallel to Span, Depth of Fill Less than 2.0 feet

36.4.6.1.2 Case 2 - Traffic Travels Perpendicular to Span

When traffic travels perpendicular to the span, live load shall be distributed to the top slab using the equations specified in **LRFD [4.6.2.1]** for concrete decks with primary strips perpendicular to the direction of traffic per **LRFD[4.6.2.10.3]**. The effect of multiple lanes shall be considered. Use the multiple presence factor, m, as required per **LRFD [3.6.1.1.2]**.

For a cast-in-place box culvert, the width of the primary strip, in inches is:

+M: 26.0 + (6.6)(S)

-M: 48.0 + (3.0)(S)

as stated in **LRFD [Table 4.6.2.1.3-1]**

Where:

S	=	Spacing of supporting components (ft)
+M	=	Positive moment
-M	=	Negative moment

36.4.6.2 Depth of Fill Greater than or Equal to 2.0 ft.

Where the depth of fill is 2.0 ft or greater, follow **LRFD [3.6.1.2.6b-c]**. The effect of multiple lanes shall be considered. Use the multiple presence factor, m, as required per **LRFD [3.6.1.1.2]**.

36.4.6.2.1 Case 1 – Traffic Travels Parallel to Span

When the traffic travels primarily parallel to the span, follow **LRFD [3.6.1.2.6b]**.

For live load distribution <u>transverse</u> to span, the wheel/axle load interaction depth, $H_{int\text{-}t}$, shall be:

$$H_{\text{int}-t} = \frac{S_w - W_t/12 - 0.06D/12}{LLDF} \qquad \text{(ft)}$$

where $H < H_{int\text{-}t}$ (no lateral interaction); then $W_w = W_t/12 + LLDF\cdot(H) + 0.06\cdot(D/12)$

where $H \geq H_{int\text{-}t}$ (lateral interaction); then $W_w = W_t/12 + S_w + LLDF\cdot(H) + 0.06\cdot(D/12)$

For live load distribution <u>parallel</u> to span, the wheel/axle load interaction depth $H_{int\text{-}p}$ shall be:

$$H_{\text{int}-p} = \frac{S_a - \ell_t/12}{LLDF} \qquad \text{(ft)}$$

where $H < H_{int\text{-}p}$ (no longit. interaction); then $\ell_w = \ell_t/12 + LLDF\cdot(H)$

where $H \geq H_{int\text{-}p}$ (longit. interaction); then $\ell_w = \ell_t/12 + S_a + LLDF\cdot(H)$

Where:

D	=	Clear span of the culvert (in)
H	=	Depth of fill from top of culvert to top of pavement (in)
$H_{int\text{-}t}$	=	Wheel interaction depth transverse to span (ft)
$H_{int\text{-}p}$	=	Axle interaction depth parallel to span (ft)
LLDF	=	Live load distribution factor per **LRFD [Table 3.6.1.2.6a-1]**; (1.15)
W_t	=	Width of tire contact area, per **LRFD [3.6.1.2.5]**; (20 in)
ℓ_t	=	Length of tire contact area, per **LRFD [3.6.1.2.5]**; (10 in)
S_w	=	Wheel spacing; (6.0 ft)

S_a	=	Axle spacing (ft)
W_w	=	Live load patch width at depth H (ft)
ℓ_w	=	Live load patch length at depth H (ft)

$$A_{LL} = \ell_w \cdot W_w$$

Where:

A_{LL}	=	Rectangular area at depth H (ft^2)

The live load vertical crown pressure shall be:

$$P_L = \frac{P(1 + IM/100)(m)}{A_{LL}}$$

Where:

IM	=	Dynamic load allowance (%); (see 36.4.8)
m	=	Multiple presence factor per **LRFD [3.6.1.1.2]**
P	=	Live load applied at surface on all interacting wheels (kip)
P_L	=	Live load vertical crown pressure (ksf)

The longitudinal and transverse distribution widths for depths of fill greater than or equal to 2.0 feet are illustrated in Figure 36.4-4.

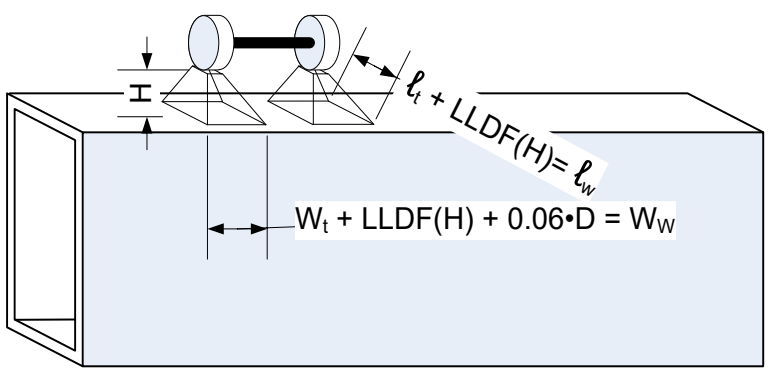


Figure 36.4-4
Distribution of Wheel Loads, Depth of Fill ≥ 2.0 feet (no lateral interaction)

36.4.6.2.2 Case 2 – Traffic Travels Perpendicular to Span

When traffic travels perpendicular to the span, live load shall be distributed to the top slab as described in **LRFD [3.6.1.2.6c]**.

36.4.7 Live Load Soil Pressures

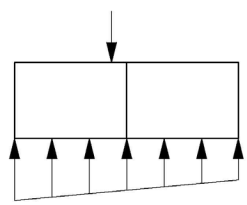


Figure 36.4-5
Vertical Soil Pressure under Culvert

The soil pressure on the bottom of the box is determined by moving the live load across the box. Find the location where the live load causes the maximum effects on the top slab of the box. At that location, determine the soil pressure diagram that will keep the system in equilibrium. Use the effects of this soil pressure in the bottom slab analysis.

36.4.8 Dynamic Load Allowance

Dynamic load allowance decreases as the depth of fill increases. **LRFD [3.6.2.2]** states that the impact on buried components shall be calculated as:

$$IM = 33(1.0 - 0.125(D_E)) \geq 0\%$$

Where:

D_E = Minimum depth of earth cover above the structure (ft)

36.4.9 Location for Maximum Moment

Create influence lines and use notional loading to determine the location for maximum moment. In this analysis, include cases for variable axle spacing and reverse axle order for unsymmetrical loading conditions.

For notional vehicles, only the portion of the loading that contributes to the effect being maximized is included. This is illustrated in Figure 36.4-6.

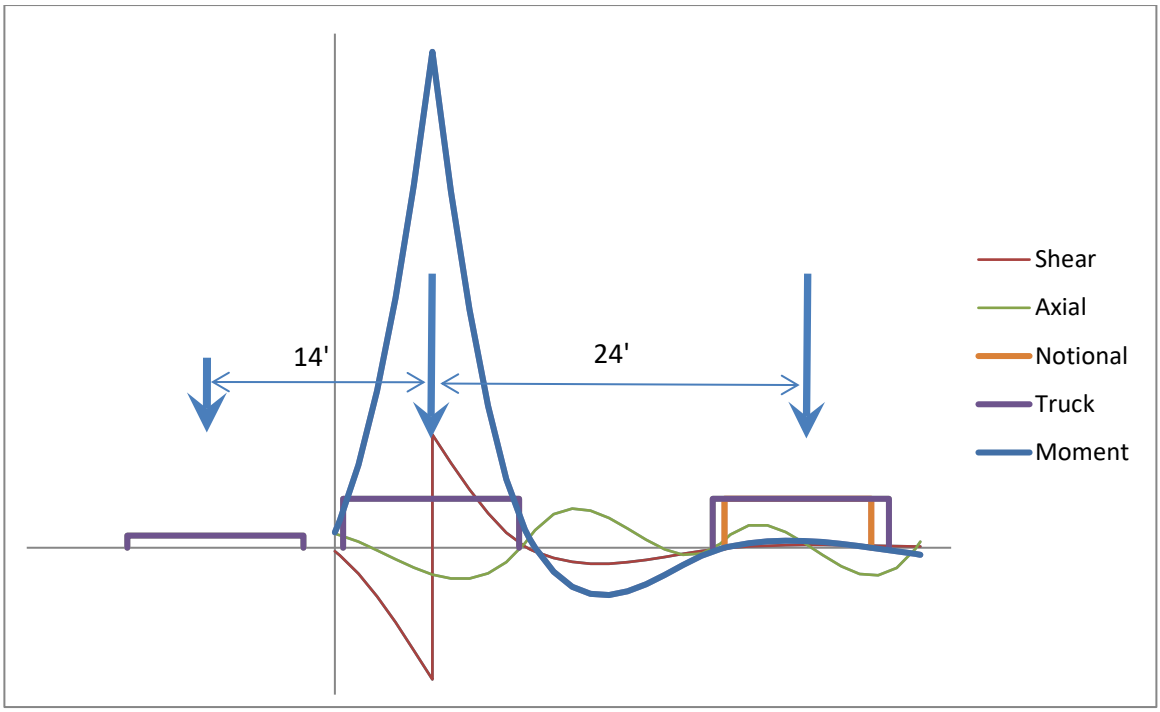


Figure 36.4-6
Application of Notional Loading using Influence Lines

The maximum positive moment results when the middle axial load is centered at the first positive peak while the variable rear axial spacing is 24 feet. Only the portion of the rear axial load in the positive region of the moment influence line is considered. The middle axial load and the portion of the rear axial in the positive region of the moment influence line are loaded on the shear and axial influence lines to compute the corresponding effects. Both positive and negative portions of the shear and axial influence lines are used when computing the corresponding effects. This process is repeated for maximizing the negative moment, shear and axial effects and computing the corresponding effects.

36.5 Design Information

Sidesway of the box is not considered because of the lateral support of the soil.

The centerline of the walls and top and bottom slabs are used for computing section properties and dimensions for analysis.

WisDOT Policy Item:

For skews 20 degrees or less, place the reinforcing steel along the skew. For skews over 20 degrees, place the reinforcing steel perpendicular to the centerline of box.

Culverts are analyzed as if the reinforcing steel is perpendicular to the centerline of box for all skew angles.

The minimum thickness of the top and bottom slab is 6½ inches. For pedestrian underpasses and slabs with fills less than 2 feet, the minimum thickness of the top slab should be 1 foot. Minimum wall thickness is based on the inside opening of the box (height) and the height of the apron wall above the floor. Use the following table to select the minimum wall thickness that meets or exceeds the three criteria in the table.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

All slab thicknesses are rounded to the next largest ½ inch.

Top and bottom slab thicknesses are determined by shear and moment requirements. Slab thickness shall be adequate to carry the factored shear without shear reinforcement.

All bar steel is detailed as being 2 inches clear with the following exceptions:

- The bottom steel in the bottom slab is detailed with 3 inches clear
- The top steel in the top slab of a box culvert with no fill is detailed with 2½ inches clear

A haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Only 45º haunches shall be used. Minimum haunch depth and length is 6 inches. Haunch dimensions are increased in 3 inch increments.

The slab thickness required is determined by moment or shear, whichever governs.

The shear in the top and bottom slabs is assumed to occur at a distance "d" from the face of the walls. The value for "d" equals the distance from the centroid of the reinforcing steel to the face of the concrete in compression. When a haunch is used, shear must also be checked at the end of the haunch.

For multi-cell culverts make interior and exterior walls of equal thickness.

Culverts shall be designed for live load and the range of fill between the shoulders of the roadway. The depth of fill is measured from the top of culvert to the surface of earth fill or top of roadway pavement. To accommodate future widening of the roadway, reduced sections may not be used on the ends of the culvert where there is less fill. Exceptions may be made with the approval of the Bureau of Structures where the culvert has high fills and a reduced section could be used for at least two panel pours per end of culvert. Culvert extensions shall be designed for the same range of fills as the original culvert. The extension design shall not have lower capacity than the original culvert. Maximum length of panel pour is 40 feet.

Barrel lengths are based on the roadway sections and wing lengths are based on a minimum 2 1/2:1 slope of fill from the top of box to apron. Consideration shall be given to match the typical roadway cross slope.

Dimensions on drawings are given to the nearest ¼ inch only.

36.6 Detailing of Reinforcing Steel

To calculate the required bar steel area and cutoff points a maximum positive and negative moment envelope is computed. It is assumed that the required bar lengths in the top slab are longer than those in the bottom slab. Therefore, cutoff points are computed for the top slab and are also used in the bottom slab.

36.6.1 Bar Cutoffs

Per **LRFD [5.10.8.1.2a]**, all flexural reinforcement shall be extended beyond the point at which it is no longer required to resist flexure for a distance not less than:

- The effective depth of the member
- 15 times the nominal diameter of the bar
- 1/20 of the clear span

Continuing reinforcement shall extend not less than the development length, ℓ_d (**LRFD [5.10.8.2]**) beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

Per **LRFD [5.10.8.1.2b]**, at least one-third of the positive moment reinforcement in simple span members and one-fourth of the positive moment reinforcement in continuous span members shall extend along the same face of the member beyond the centerline of the support. In beams, such extension shall not be less than 6.0 in.

Per **LRFD [5.10.8.1.2c]**, at least one-third of the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection not less than:

- The effective depth of the member
- 12 times the nominal diameter of the bar
- 0.0625 times the clear span

36.6.2 Corner Steel

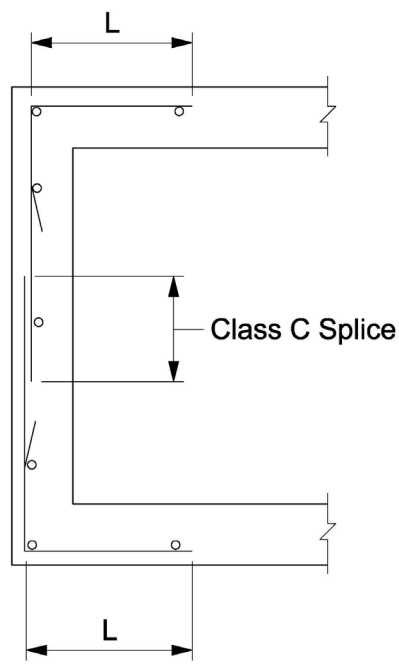


Figure 36.6-1
Layout of Corner Steel

The area of steel required is the maximum computed from using the top and bottom corner moments and the thickness of the slab or wall, whichever controls. Identical bars are used in the top and bottom corners. Identical length bars are used in the left and right corners if the bar lengths are within 2 feet of one another. Top and bottom negative steel is cut in the walls and detailed in two alternating lengths when a savings of over 2 feet in a single bar length can be obtained. Corner steel is always lapped at the center of the wall. If two bar lengths are used, only alternate bars are lapped.

Distance "L" is computed from the maximum negative moment envelope for the top slab and shall include the extension lengths discussed in 36.6.1.

36.6.3 Positive Moment Slab Steel

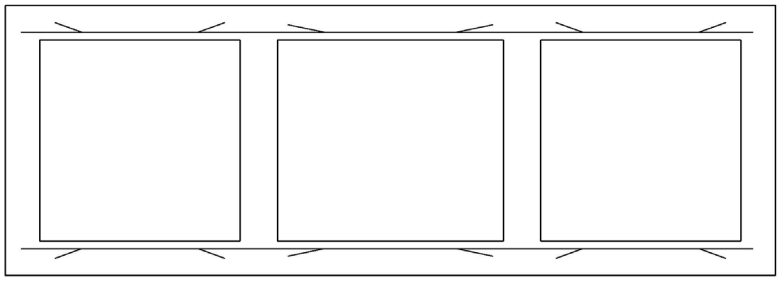


Figure 36.6-2
Layout of Positive Moment Steel

The area of steel required is determined by the maximum positive moments in each span. Top and bottom slab reinforcing steel may be of different size and spacing, but will have identical lengths. Detail two alternating bar lengths in a slab if 2 feet or more of bar steel can be saved in a single bar length.

When two alternating bar lengths are detailed in multi-cell culverts, run every other positive bar across the entire width of box. If this requires a length longer than 40 feet, lap them over an interior wall. For 2 or more cells, if the distance between positive bars of adjacent cells is 1 foot or less, make the bar continuous.

The cutoff points of alternate bars are determined from the maximum positive moment envelope for the top slab and shall include the extension lengths discussed in 36.6.1. These same points are used in the bottom slab. Identical bar lengths are used over multiple cells if bars are within 2 feet of one another.

36.6.4 Negative Moment Slab Steel over Interior Walls

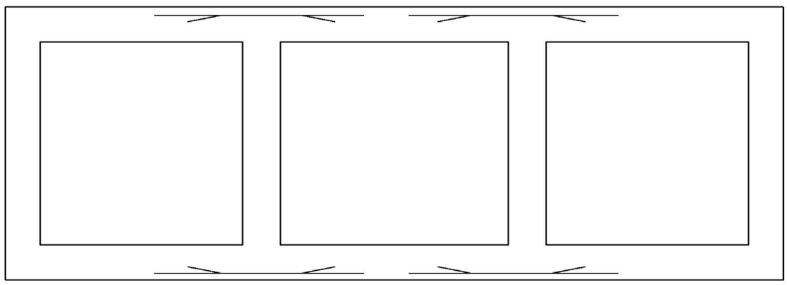


Figure 36.6-3
Layout of Negative Moment Steel

If no haunch is present, the area of steel required is determined by using the moment and effective depth at the face of the interior wall. If the slab is haunched, the negative reinforcement is determined per **LRFD [12.11.5.2],** which states that the negative moment is determined at the intersection of the haunch and uniform depth member. Top and bottom slab

reinforcing steel may be of different size and spacing, but will have identical lengths. Detail two alternating bar lengths in a slab if 2 feet or more of bar steel can be saved in a single bar length.

Cutoff points are determined from the maximum negative moment envelope of the top slab and shall include the extension lengths discussed in 36.6.1. The same bar lengths are then used in the bottom slab. Identical bar lengths are used over multiple interior walls if bars are within 2 feet of one another. The minimum length of any bar is 2 times the development length. For culverts of 3 or more cells, if the clear distance between negative bars of adjacent spans is 1 foot or less, make the bar continuous across the interior spans.

When there is no fill over the top slab, run the negative moment reinforcing steel across the entire width of the culvert. Refer to 36.6.8 for temperature and shrinkage requirements.

36.6.5 Exterior Wall Positive Moment Steel

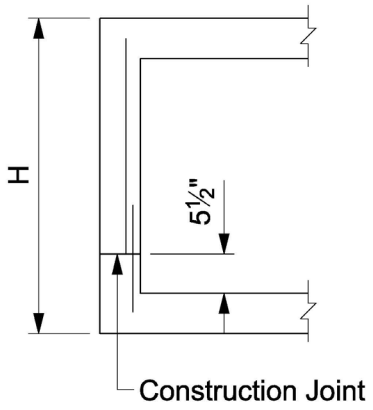


Figure 36.6-4
Layout of Exterior Wall Steel

The area of steel is determined by the maximum positive moment in the wall. A minimum of #4 bars at 18 inches is supplied. The wall bar is extended to 2 inch top clear and the dowel bar is extended to 3 inch bottom clear. A construction joint, 5 ½ inches above the bottom slab, is always used so a dowel bar must be detailed.

36.6.6 Interior Wall Moment Steel

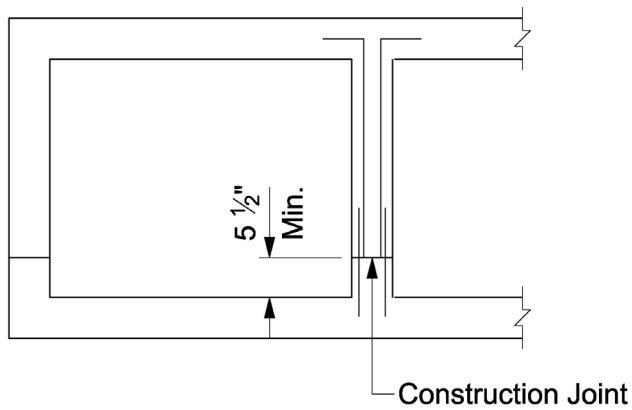


Figure 36.6-5
Layout of Interior Wall Steel

The area of steel is determined from the maximum moment at the top of the wall and the effective wall thickness. A minimum of #4 bars at 18 inches is supplied. Identical steel is provided at both faces of the wall. A 1 foot, 90 degree bend, is provided in the top slab with the horizontal portion being just below the negative moment steel. The dowel bar is extended to 3 inch bottom clear. A construction joint, 5 ½ inches above the bottom slab, is always used so a dowel bar must be detailed. When a haunch is provided, the construction joint is placed a distance above the bottom slab equal to the haunch depth plus 2 inches.

36.6.7 Distribution Reinforcement

Per **LRFD [5.12.2.1]**, transverse distribution reinforcement is not required for culverts where the depth of fill exceeds 2.0 feet.

Per **LRFD [12.11.2.1]**, provide distribution reinforcement for culverts with less than or equal to 2 feet of fill in accordance with **LRFD [9.7.3.2]**, which states that reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows (for primary reinforcement parallel to traffic):

$$\text{Percentage} = \frac{100}{\sqrt{S}} \leq 50\%$$

Where:

S = Effective span length (ft) (for slabs monolithic with walls, this distance is taken as the face-to-face distance per **LRFD [9.7.2.3]**)

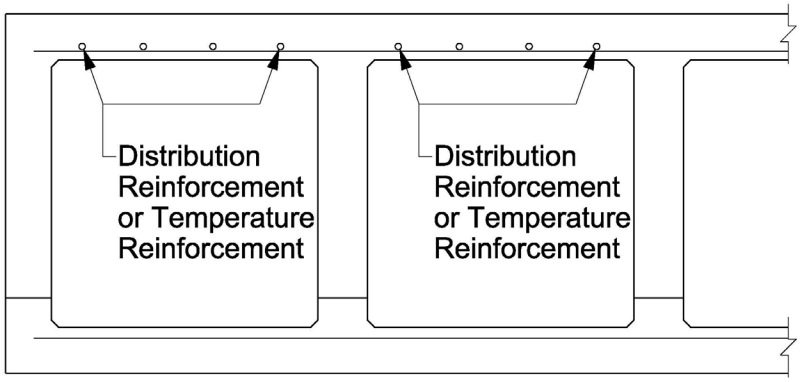


Figure 36.6-6
Layout of Distribution Steel

36.6.8 Shrinkage and Temperature Reinforcement

Shrinkage and temperature reinforcement is required on all inside culvert faces, negative moment regions in top slabs, and on both wingwall faces in each direction that does not already have strength or distribution reinforcement. Shrinkage and temperature reinforcement is not required on the outside (soil) face for culvert barrels. This includes exterior walls, the bottom of the bottom slab, and in some cases the top face of the top slab in the positive moment region. Per **LRFD [12.11.5.3.1]**, provide shrinkage and temperature reinforcement near the inside surfaces of walls and slabs in accordance with **LRFD [5.10.6]**, which states that the area of shrinkage and temperature steel per foot on each face and in each direction shall satisfy:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y}$$

$$0.11 \leq A_s \leq 0.60$$

Where:

A_s	=	Area of reinforcement in each direction and each face (in^2/ft)
b	=	Least width of component section (in.)
h	=	Least thickness of component section (in.)
f_y	=	Specified yield strength of reinforcing bars ≤ 75 (ksi)

Where the least dimension varies along the length of the component, multiple sections should be examined to represent the average condition at each section.

Shrinkage and temperature reinforcement shall use a minimum of #4 bars at 18 inch centers in both directions.

36.7 Box Culvert Aprons

Five types of box culvert aprons are used. They are referred to as Type A, B, C, D and E. The angle that the wings make with the direction of stream flow is the main difference between the five types. The allowable headwater and other hydraulic requirements are what usually determine the type of apron required. Physical characteristics at the site may also dictate a certain type. For hydraulic design of different apron types see Chapter 8.

36.7.1 Type A

Type A, because of its poor hydraulic properties, is generally not used except for cattle or pedestrian underpasses.

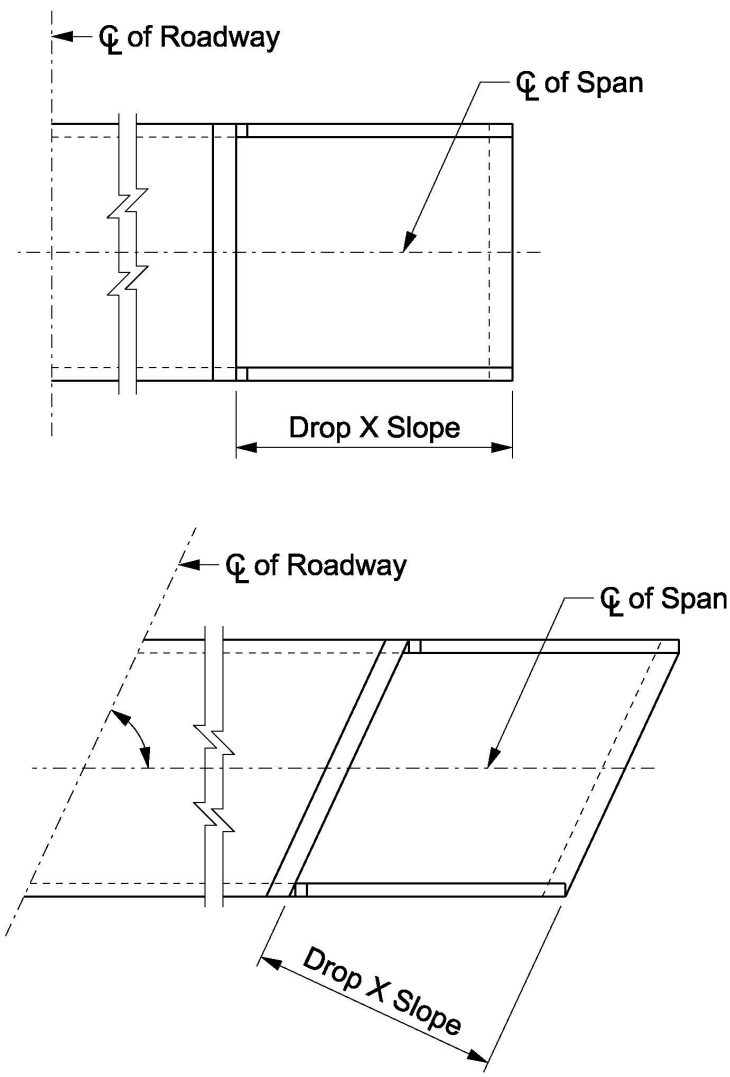
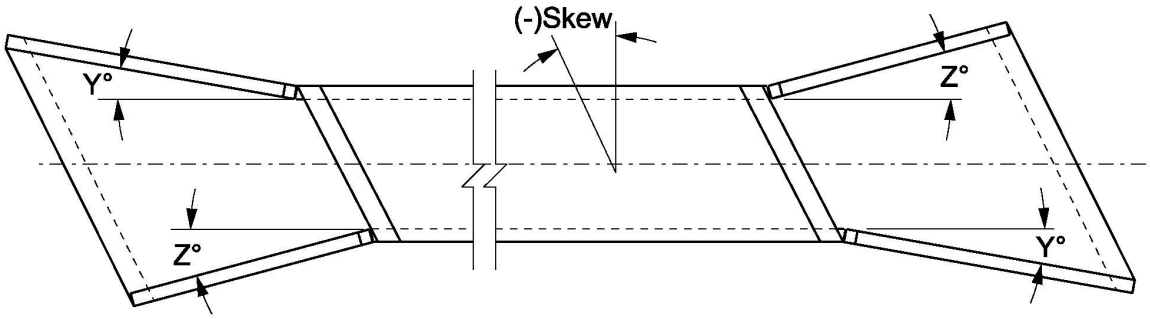


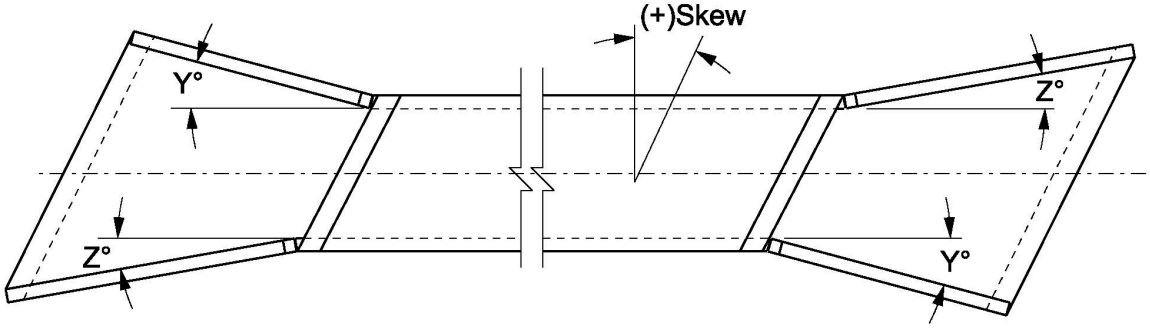
Figure 36.7-1
Plan View of Type A

36.7.2 Type B, C, D

Type B is used for outlets. Type C & D are of equal efficiency but Type C is used most frequently. Type D is used for inlets when the water is entering the culvert at a very abrupt angle. See Figure 36.7-2 for Wing Type B, C and D for guidance on wing angles for culvert skews.



Skew		Wing Type B		Wing Type C		Wing Type D	
Greater Than	To Include	Angle Y	Angle Z	Angle Y	Angle Z	Angle Y	Angle Z
0°	7.5°	15°	15°	30°	30°	45°	45°
7.5°	15.0°	15°	15°	25°	30°	40°	45°
15.0°	22.5°	10°	15°	25°	30°	35°	45°
22.5°	37.5°	10°	15°	20°	30°	30°	45°
37.5°	45.0°	10°	15°	15°	30°	25°	45°
45.0°	52.5°	5°	15°	15°	30°	20°	45°
52.5°	67.5°	5°	15°	10°	30°	15°	45°
67.5°	75.0°	5°	15°	5°	30°	10°	45°
75.0°	82.5°	0°	15°	5°	30°	5°	45°
82.5°	90.0°	0°	15°	0°	30°	0°	45°



Skew		Wing Type B		Wing Type C		Wing Type D	
Greater Than	To Include	Angle Y	Angle Z	Angle Y	Angle Z	Angle Y	Angle Z
0°	7.5°	15°	15°	30°	30°	45°	45°
7.5°	15.0°	15°	15°	30°	25°	45°	40°
15.0°	22.5°	15°	10°	30°	25°	45°	35°
22.5°	37.5°	15°	10°	30°	20°	45°	30°
37.5°	45.0°	15°	10°	30°	15°	45°	25°
45.0°	52.5°	15°	5°	30°	15°	45°	20°
52.5°	67.5°	15°	5°	30°	10°	45°	15°
67.5°	75.0°	15°	5°	30°	5°	45°	10°
75.0°	82.5°	15°	0°	30°	5°	45°	5°
82.5°	90.0°	15°	0°	30°	0°	45°	0°

Figure 36.7-2
Wing Type B, C, D (Angles vs. Skew)

36.7.3 Type E

Type E is used primarily in urban areas where a sidewalk runs over the culvert and it is necessary to have a parapet and railing along the sidewalk. For Type E the wingwalls run parallel to the roadway just like the abutment wingwalls of most bridges. It is also used where Right of Way (R/W) is a problem and the aprons would extend beyond the R/W for other types. Wingwall lengths for Type E wings are based on a minimum channel side slope of 1.5 to 1.

36.7.4 Wingwall Design

Culvert wingwalls are designed using a 1 foot surcharge height, a unit weight of backfill of 0.120 kcf and a coefficient of lateral earth pressure of 0.5, as discussed in 36.4.3. When the wingwalls are parallel to the direction of traffic and where vehicular loads are within ½ the wall height from the back face of the wall, design using a surcharge height representing vehicular load per **LRFD [Table 3.11.6.4-2]**. Load and Resistance Factor Design is used, and the load factor for lateral earth pressure of γ_{EH} = 1.69 is used, based on past design experience. The lateral earth pressure was conservatively selected to keep wingwall deflection and cracking to acceptable levels. Many wingwalls that were designed for lower horizontal pressures have experienced excessive deflections and cracking at the footing. This may expose the bar steel to the water that flows through the culvert and if the water is of a corrosive nature, corrosion of the bar steel will occur. This phenomena has led to complete failure of some wingwalls throughout the State.

For wing heights of 7 feet or less determine the area of steel required by using the maximum wall height and use the same bar size and spacing along the entire wingwall length. The minimum amount of steel used is #4 bars at 12 inch spacing. Wingwall thickness is made equal to the barrel wall thickness.

For wing heights over 7 feet the wall length is divided into two or more segments to determine the area of steel required. Use the same bar size and spacing throughout each segment, as determined by using the maximum wall height in the segment.

Wingwalls must satisfy Strength I Limit State for flexure and shear, and Service I Limit State for crack control, minimum reinforcement, and reinforcement spacing. Adequate shrinkage and temperature reinforcement shall be provided.

36.8 Box Culvert Camber

Camber of culverts is a design compensation for anticipated settlement of foundation soil beneath the culvert. Responsibility for the recommendation and calculation of camber belongs to the Regional Soils Engineer. Severe settlement problems with accompanying large camber are to be checked with the Geotechnical Section.

Both total and differential settlement need to be considered to determine the amount of box camber required to avoid adverse profile sag and undesirable separation at culvert joints per **LRFD [12.6.2.2]**. If the estimated settlement is excessive, contingency measures will need to be considered, such as preloading with embankment surcharge, undercutting and subgrade stabilization. To evaluate differential settlement, it will be necessary to calculate settlement at more than one point along the length of the box culvert.

36.8.1 Computation of Settlement

Settlement should be evaluated at the Service Limit state in accordance with **LRFD [12.6.2.2]** and **LRFD [10.6.2]**, and consider instantaneous elastic consolidation and secondary components. Elastic settlement is the instantaneous deformation of the soil mass that occurs as the soil is loaded. Consolidation settlement is the gradual compression of the soil skeleton when excess pore pressure is forced out of the voids in the soil. Secondary settlement, or creep, occurs as a result of plastic deformation of the soil skeleton under constant effective stress. Secondary settlement is typically not significant for box culvert design, except where there is an increase in effective stress within organic soil, such as peat. If secondary settlement is a concern, it should be estimated in accordance with **LRFD [10.6.2.4]**.

Total settlement, including elastic, consolidation and secondary components may be taken in accordance with **LRFD [10.6.2.4.1]** as:

$$S_t = S_e + S_c + S_s$$

Where:

S_t = Total settlement (ft)

S_e = Elastic settlement (ft)

S_c = Primary consolidation settlement (ft)

S_s = Secondary settlement (ft)

To compute settlement, the subsurface soil profile should be subdivided into layers based on stratigraphy to a depth of about 3 times the box width. The maximum layer thickness should be 10 feet.

Primary consolidation settlement for normally-consolidated soil is computed using the following equation in accordance with **LRFD [10.6.2.4.3]**:

$$S_c = \left[\frac{H_c}{1+e_o}\right] c_c \log_{10}\left[\frac{\sigma'_f}{\sigma'_o}\right]$$

Where:

S_c	=	Primary consolidation settlement (ft)
H_c	=	Initial height of compressible soil layer (ft)
e_o	=	Void ratio at initial vertical effective stress
C_c	=	Compression index which is a measure of the compressibility of a soil. It is the slope of the straight-line part of the e-log p curve from a conventional consolidation (oedometer) test.
σ'_f	=	Final vertical effective stress at midpoint of soil layer under consideration (ksf)
σ'_o	=	Initial vertical effective stress at midpoint of soil layer under consideration (ksf)

If the soil is over-consolidated, reference is made to **LRFD [10.6.2.4.3]** to estimate consolidation settlement.

Further description for the above equations and consolidation test can be found in most textbooks on soil mechanics.

For preliminary investigations C_c can be determined from the following approximate formula, found in most soil mechanics textbooks:

Non organic soils: $C_c = 0.007\ (LL-10)$

Where:

LL	=	Liquid limit expressed as whole number.

If the in-place moisture content approaches the plastic limit the computed C_c is decreased by 75%. If the in-place moisture content is near the liquid limit use the computed value. If the in-place moisture content is twice the liquid limit the computed C_c is increased by 75%. For intermediate moisture contents the percent change to the computed C_c is determined from a straight line interpolation between the corrections mentioned above.

If settlements computed by using the approximate value of C_c exceed 1.5 feet, a consolidation test is performed. As in-place moisture content approaches twice the liquid limit, settlement is caused by a local shear failure and the consolidation equation is no longer applicable.

The consolidation equation is applied to only compressible silts and clays. Sands are of a lower compressibility and no culvert camber is required until the fill exceeds 25 feet. When the fill exceeds 25 feet for sand, a camber of 0.01 feet per foot of fill is used.

36.8.2 Configuration of Camber

The following guides are to be followed when detailing camber.

- It is unnecessary to provide gradual camber. "Brokenback" camber is closer to the actual settlement which occurs.
- Settlement is almost constant from shoulder point to shoulder point. It then reduces to the ends of the culvert at the edge of the fill.
- The ends of the culvert tend to come up if side slopes are steeper than 2½ to 1. With 2 to 1 side slopes camber is increased 10% to compensate for this rise.

36.8.3 Numerical Example of Settlement Computation

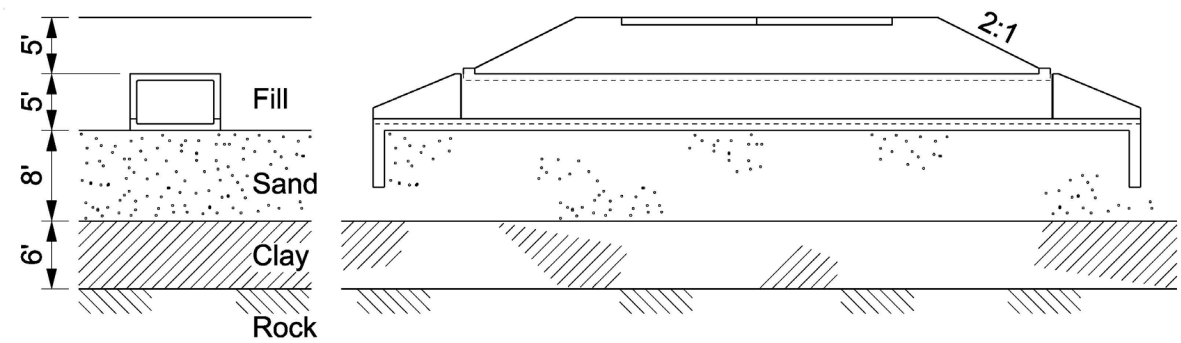


Figure 36.8-1
Soil Strata under Culvert

A box culvert rests on original ground consisting of 8 feet of sand and 6 feet of clay over bedrock. Estimate the settlement of the culvert if 10 feet of fill is placed on the original ground after the culvert is constructed. The in-place moisture content and liquid limit equal 40%. The initial void ratio equals 0.98. The unit weight of the clay is 105 pcf and that of the fill and sand is 110 pcf. There is no water table.

σ'_o = (8 ft)(110 pcf) + (3 ft)(105 pcf) = 1195 psf

$\sigma'_f = \sigma'_o$ + (10 ft)(110 pcf) = 1195 psf + 1100 psf = 2295 psf

C_c = 0.007 (40-10) = 0.21 (approximate value)

$$S_c = \left[\frac{H_c}{1+e_o}\right] c_c \log_{10}\left[\frac{\sigma'_f}{\sigma'_o}\right] = \frac{6\text{ft}}{1+0.98} 0.21 * \log_{10}\left[\frac{2295\text{psf}}{1195\text{psf}}\right] = 0.18\text{ft}$$

36.9 Box Culvert Structural Excavation and Structure Backfill

All excavations for culverts and aprons, unless on bedrock or fill, are to include a 6 inch minimum undercut and backfilled with structural backfill, as described in the specification. This undercut is for construction purposes and provides a solid base for placing reinforcement and pouring the bottom slab. For fill sections, it is assumed that placed fills provide a solid base and structural backfill is not needed. For cut sections, deeper under cuts may be warranted to mitigate differential settlement.

All volume excavated and not occupied by the new structure should be backfilled with structure backfill for the full length of the box culvert, including the apron.

See Standard Detail 9.01 – Structure Backfill Limits and Notes – for typical pay limits and plan notes.

36.10 Box Culvert Headers

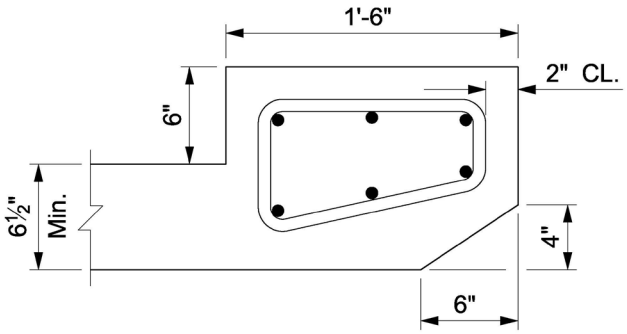
For skews of 20 degrees and less the main reinforcing steel is parallel to the end of the barrel. A header is not required for structural purposes but is used to prevent the fill material from spilling into the apron. A 12 inch wide by 6 inch high (above the top of top slab) header with nominal steel is therefore used for skews of 20 degrees and less on the top slab. No header is used on the bottom slab.

For skews over 20 degrees the main reinforcing is not parallel to the end of the barrel. The positive reinforcing steel terminates in the header and thus the header must support, in addition to its own dead load, an additional load from the dead load of the slab and fill above it. A portion of the live load may also have to be supported by the header.

The calculation of the actual load that a header must support becomes a highly indeterminate problem. For this reason a rational approach is used to determine the amount of reinforcement required in the headers. The design moment capacity of the header must be equal to or greater than 1.25 times the header dead load moment (based on simple span) plus 1.75 times a live load moment from a 16 kip load assuming 0.5 fixity at ends.

To prevent a traffic hazard, culvert headers are designed not to protrude above the ground line. For this reason the height of the header above the top of the top slab is typically selected to be 6 inches. The width of the header is standardized at 18 inches.

The header in the following figure gives the design moment capacities listed using d = 8.5 inches.



Bar Size	Moment Capacity ft.-kip
#6	46.1
#7	60.7
#8	76.5
#9	92.1
#10	109.1

Figure 36.10-1
Header Details (Skews > 20°)

The following size bars are recommended for the listed header lengths where "Header Length" equals the distance between C/L of walls in one cell measured along the skew.

Header Length	Bar Size [1]
To 11’	#7
Over 11’ to 14’	#8
Over 14’ to 17’	#9
Over 17’ to 20’	#10

Table 36.10-1
Header Reinforcement

[1] Use the bar size listed in each header and place 3 bars on the top and 3 bars on the bottom. Use a header on both the top and bottom slab. See the Standard *Box Culvert Details* in Chapter 36.

Where headers greater than 6 inches in height are used to retain roadway fill, the top slab shall be designed to handle the bending moment transmitted from the header. Additional reinforcement may be required.

Where barriers are placed on top of the culvert header, the barrier, header, and top slab shall be designed for vehicular impact forces.

36.11 Plan Detailing Issues

36.11.1 Weep Holes

Investigate the need for weep holes for culverts in cohesive soils. These holes are to relieve the hydrostatic pressure on the sides of the culverts. Where used, place the weep holes 1 foot above normal water elevation but a minimum of 1 foot above the lower sidewall construction joint. Do not place weep holes closer than 1 foot from the bottom of the top slab.

36.11.2 Cutoff Walls

Where dewatering the cutoff wall in sandy terrain is a problem, the concrete may be poured in the water. Place a note on the plans allowing concrete for the cutoff wall to be placed in the water.

36.11.3 Nameplate

Designate a location on the wingwall for placement of the nameplate. Locate nameplate on the first right wing traveling in the Cardinal direction (North/East).

36.11.4 Plans Policy

If a cast-in-place reinforced concrete box culvert is used, full plans must be provided and sealed by a professional engineer to the Bureau of Structures for approval. The plans must be in accordance with the *Bridge Manual* and Standards.

36.11.5 Rubberized Membrane Waterproofing

When required by the Standard Details, place the bid item "Rubberized Membrane Waterproofing" on the final plans. The quantity is given square yards.

36.12 Precast Four-Sided Box Culverts

In general, structural contractors prefer cast-in-place culverts while grading contractors prefer precast culverts. Precast culverts have been more expensive than cast-in-place culverts in the past, but allow for reduced construction time. Box culverts that are less than 20 square feet are considered roadway culverts. All other culverts require a B or C number along with the appropriate plans. All culverts requiring a number should be processed through the Bureau of Structures.

When a precast culvert is selected as the best structure type for a particular project during the design study phase, preliminary plans and complete detailed final plans are required to be sent to the Bureau of Structures for approval. The design and fabrication must be in accordance with ASTM Specification C1577, *AASHTO LRFD Specifications*, and the Bridge Manual.

Sometimes a complete set of plans is created for a cast-in-place culvert and a precast culvert is stated to be an acceptable alternate. If the contractor selects the precast alternate, the contractor is to submit shop drawings, sealed by a professional engineer, to the Bureau of Structures for approval. The design and fabrication must be in accordance with ASTM Specification C1577, *AASHTO LRFD Specifications*, and the Bridge Manual.

36.13 Other Buried Structures

The following section provides general guidance on cross-drain alternatives to concrete box culverts.

36.13.1 General

Typical alternatives to four-sided (box) concrete structures include three-sided (bottomless) concrete structures and metal buried structures. These structures are available in a variety of shapes, sizes, and material types. In general, three-sided structures may be cost prohibitive when deep foundations are required.

Concrete buried structures are rigid structures that can be constructed using cast-in-place or precast concrete. These structures obtain strength through reinforced concrete sections that have proven to be durable and long-lasting. Refer to 36.13.2 for additional information on three-sided concrete structures.

Metal buried structures are typically constructed with factory assembled corrugated sections or field assembled structural plates. Commonly used shapes include pipes and pipe-arches consisting of steel or aluminum alloy. These flexible structures obtain strength through soil-structure interactions that allow for the use of thin-walled sections. Some advantages of metal buried structures include; increased speed of installation, potential initial cost savings, and the variety of available shapes. Some disadvantages include their susceptibility to damage and/or degradation and performance being dependent on the quality of installation. Refer to 36.13.3 and FDM 13-1 for additional information on metal buried structures.

Buried structures assigned a structure number shall be coordinated with the Bureau of Structures and follow the policies and procedures as stated in the Bridge manual and FDM 13-1. Refer to 2.5 for information on assigning structure numbers.

Refer to AASHTO LRFD Section 12 – Buried Structures and Tunnel Liners for additional information.

36.13.2 Three-Sided Concrete Structures

Three-sided box culvert structures are divided into two categories: cast-in-place three sided structures and precast three-sided structures. These structures shall follow the criteria outlined below.

36.13.2.1 Cast-In-Place Three-Sided Structures

To be developed

36.13.2.2 Precast Three-Sided Structures

Three-sided precast concrete structures offer a cost effective, convenient solution for a variety of bridge needs. The selection of whether a structure over a waterway should be a culvert, a three-sided precast concrete structure or a bridge is heavily influenced by the hydraulic

opening. As the hydraulic opening becomes larger, the selection process for structure type progresses from culvert to three-sided precast concrete structure to bridge. Cost, future maintenance, profile grade, staging, skew, soil conditions and alignment are also important variables which should be considered. Culverts generally have low future maintenance; however, culverts should not be considered for waterways with a history or potential of debris to avoid channel cleanout maintenance. In these cases a three-sided precast concrete structure may be more appropriate. Three-sided precast concrete structures have the advantage of larger single and multiple openings, ease of construction, and low future maintenance costs.

A precast-concrete box culvert may be recommended by the Hydraulics Team. The side slope at the end or outcrop of a box culvert should be protected with guardrail or be located beyond the clear zone.

The hydraulic recommendations will include the Q_{100} elevation, the assumed flowline elevation, the required span, and the required waterway opening for all structure selections. The designer will determine the rise of the structure for all structure sections.

A cost comparison is required to justify a three-sided precast concrete structure compared to other bridge/culvert alternatives.

To facilitate the initiation of this type of project, the BOS is available to assist the Owners and Consultants in working out problems which may arise during plan development.

Some of the advantages of precast three-sided structures are listed below:

- Speed of Installation: Speed of installation is more dependent on excavation than product handling and placement. Precast concrete products arrive at the jobsite ready to install. Raw materials such as reinforcing steel and concrete do not need to be ordered, and no time is required on site to set up forms, place concrete, and wait for the concrete to cure. Precast concrete can be easily installed on-demand and immediately backfilled.

- Environmentally Friendly: Precast concrete is ready to be installed right off the delivery truck, which means less storage space needed for scaffolding and rebar. There is less noise pollution from ready-mix trucks continually pulling up on site and less waste as a result of using precast (i.e. no leftover steel, no pieces of scaffolding and no waste concrete piles). The natural bottom on a three-sided structure is advantageous to meet fish passage and DNR requirements.

- Quality Control: Because precast concrete products are produced in a quality-controlled environment with proper curing conditions, these products exhibit higher quality and uniformity over cast-in-place structures.

- Reduced Weather Dependency: Weather does not delay production of precast concrete as it can with cast-in-place concrete. Additionally, weather conditions at the jobsite do not significantly affect the schedule because the "window" of time required for installation is small compared to other construction methods, such as cast-in-place concrete.

- Maintenance: Single span precast three-sided structures are less susceptible to clogging from debris and sediment than multiple barrel culvers with equivalent hydraulic openings.

36.13.2.2.1 Precast Three-Sided Structure Span Lengths

WisDOT BOS allows and provides standard details for the following precast three-sided structure span lengths:

14'-0, 20'-0, 24'-0, 28'-0, 36'-0, 42'-0

Dimensions, rises, and additional guidance for each span length are provided in the standard details.

36.13.2.2.2 Segment Configuration and Skew

It is not necessary for the designer to determine the exact number and length of segments. The final structure length and segment configuration will be determined by the fabricator and may deviate from that implied by the plans.

A zero degree skew is preferable but skews may be accommodated in a variety of ways. Skew should be rounded to the nearer most-practical 5 deg., although the nearer 1 deg. is permissible where necessary. The range of skew is dependent on the design span and the fabrication limitations. Some systems are capable of fabricating a skewed segment up to a maximum of 45 degrees. Other systems accommodate skew by fabricating a special trapezoidal segment. If adequate right-of-way is available, skewed projects may be built with all right angle segments provided the angle of the wingwalls are adjusted accordingly. The designer shall consider the layout of the traffic lanes on staged construction projects when determining whether a particular three-sided precast concrete structure system is suitable.

Square segments are more economical if the structure is skewed. Laying out the structure with square segments will result in the greatest right-of-way requirement and thus allow ample space for potential redesign by the contractor, if necessary, to another segment configuration.

For a structure with a skew less than or equal to 15 deg., structure segments may be laid out square or skewed. Skewed segments are preferred for short structures (approximately less than 80 feet in length). Square segments are preferred for longer structures. However, skewed segments have a greater structural span. A structure with a skew of greater than 15 deg. requires additional analysis per the AASHTO LRFD Bridge Design Specifications. Skewed segments and the analysis both contribute to higher structure cost.

For a structure with a skew greater than 15 deg, structure segments should be laid out square. The preferred layout scheme for an arch-topped structure with a skew of greater than 15 deg should assume square segments with a sloping top of headwall to yield the shortest possible wingwalls. Where an arch-topped structure is laid out with skewed ends (headwalls parallel to the roadway), the skew will be developed within the end segments by varying the lengths of the legs as measured along the centerline of the structure. The maximum attainable skew is controlled by the difference between the full-segment leg length as recommended by the arch-topped-structure fabricator and a minimum leg length of 2 feet.

36.13.2.2.2.1 Minimum Fill Height

Minimum fill over a precast three-sided structure shall provide sufficient fill depth to allow adequate embedment for any required beam guard plus 6”. Refer to Standard 36.10 for further information.

Barriers mounted directly to the precast units are not allowed, as this connection has not been crash tested.

36.13.2.2.2.2 Rise

The maximum rises of individual segments are shown on the standard details. This limit is based on the fabrication forms and transportation. The maximum rise of the segment may also be limited by the combination of the skew involved because this affects transportation on the truck. Certain rise and skew combinations may still be possible but special permits may be required for transportation. The overall rise of the three-sided structure should not be a limitation when satisfying the opening requirements of the structure because the footing is permitted to extend above the ground to meet the bottom of the three-sided segment.

36.13.2.2.2.3 Deflections

Per **LRFD [2.5.2.6.2]**, the deflection limits for precast reinforced concrete three-sided structures shall be considered mandatory.

36.13.2.3 Plans Policy

If a precast or cast-in-place three-sided culvert is used, full design calculations and plans must be provided and sealed by a professional engineer to the Bureau of Structures for approval. The plans must be in accordance with the *Bridge Manual* and Standards.

The designer should use the span and rise for the structure selection shown on the plans as a reference for the information required on the title sheet. The structure type to be shown on the Title, Layout and General Plan sheets should be Precast Reinforced Concrete Three-Sided Structure.

The assumed elevations of the top of the footing and the base of the structure leg should be shown. For preliminary structure layout purposes, a 2-foot footing thickness should be assumed with the base of the structure leg seated 2 inches below the top-of-footing elevation. With the bottom of the footing placed at the minimum standard depth of 4 feet below the flow line elevation, the base of the structure leg should therefore be shown as 2’-2” below the flow line. An exception to the 4-foot depth will occur where the anticipated footing thickness is known to exceed 2 feet, where the footing must extend to rock, or where poor soil conditions and scour concerns dictate that the footing should be deeper.

The structure length and skew angle, and the skew, length and height of wingwalls should be shown. For a skewed structure, the wingwall geometrics should be determined for each wing. The sideslope used to determine the wing length should be shown on the plans.

If the height of the structure legs exceeds 10 feet, pedestals should be shown in the structure elevation view.

The following plan requirements shall be followed:

1. Preliminary plans are required for all projects utilizing a three-sided precast concrete structure.

2. Preliminary and Final plans for three-sided precast concrete structures shall identify the size (span x rise), length, and skew angle of the bridge.

3. Final plans shall include all geometric dimensions and a detailed design for the three-sided precast structure, all cast-in-place foundation units and cast-in-place or precast wingwalls and headwalls.

4. Final plans shall include the pay item Three-Sided Precast Concrete Structure and applicable pay items for the remainder of the substructure elements.

5. Final plans shall be submitted along with all pertinent special provisions to the BOS for review and approval.

In addition to foundation type, the wingwall type shall be provided on the preliminary and final plans. Similar to precast boxes, a wingwall design shall be provided which is supported independently from the three-sided structure. The restrictions on the use of cast-in-place or precast wings and headwalls shall be based on site conditions and the preferences of the Owner. These restrictions shall be noted on the preliminary and final plans.

36.13.2.4 Foundation Requirements

Precast and cast-in-place three-sided structures that are utilized in pedestrian or cattle underpasses can be supported on continuous spread or pile supported footings. Precast and cast-in-place three-sided structures that are utilized in waterway applications shall be supported on piling to prevent scour.

The footing should be kept level if possible. If the stream grade prohibits a level footing, the wingwall footings should be laid out to be constructed on the same plane as the structure footings. Continuity shall be established between the structural unit footing and the wingwall footing.

The allowable soil bearing pressure should be shown on the plans. Weak soil conditions could require pile foundations. If the footing is on piling, the nominal driving resistance should be shown. Where a pile footing is required, the type and size of pile and the required pile spacing, and which piles are to be battered, should be shown on the plans.

The geotechnical engineer should provide planning and design recommendations to determine the most cost effective and feasible foundation treatment to be used on the preliminary plans.

36.13.2.5 Precast Versus Cast-in-Place Wingwalls and Headwalls

The specifications for three-sided precast concrete structures permits the contractor to substitute cast-in-place for precast wingwalls and headwalls, and vice versa when cast-in-place is specified unless prohibited on the plans. Three-sided structures should be provided with adequate foundation support to satisfy the design assumptions permitting their relatively thin concrete section. These foundations are designed and provided in the plans. Spread footing foundations are most commonly used since they prove cost effective when rock or scour resistant soils are present with adequate bearing and sliding resistance. The use of precast spread footings shall be controlled by the planner and shall only be allowed when soil conditions permit and shall not be allowed to bear directly on rock or when rock is within 2 feet of the bottom of the proposed footing. When lower strength soils are present, or scour depths become large, a pile supported footing shall be used. The lateral loading design of the foundation is important because deflection of the pile or footing should not exceed the manufacturers' recommendations to preclude cracks developing.

36.13.3 Metal Buried Structures

The following section provides guidance on metal buried structures. This guidance should be used in addition to the guidance provided by FDM 13-1.

Use of metal buried structures shall be evaluated on a project-by-project basis to ensure hydraulic, geotechnical, and structural criteria are satisfied. This should include a comparison of alternatives considering, but not limited to; hydraulic sizing, scour potential, costs, project schedule, and structure durability. The evaluation should then be followed by a material selection investigation for structure type justifications.

Use of metal buried structures for long spans, generally defined as spans greater than 7 ft, has been limited. The Department has experienced some corrosion issues with metal structures, which includes metal pipe failures and severe section loss. These issues are likely due to the following sources: low pH environment, low resistivity environment, active anaerobic sulfate reducing bacteria, and exposure to chlorides. While research has shown corrosion and/or abrasion concerns can be addressed to better ensure structures can satisfy their intended service life [1], reinforced concrete structures are still recommended over metal structures, especially for higher volume roadways. To ensure that a metal buried structure is suitable for a given site, the following criteria shall be followed:

Site Investigation: The geotechnical investigation shall investigate corrosion potential and abrasion classification. Document site-specific pH, resistivity, sulfate, and chloride levels of the soil and water. This information shall be used when selecting an appropriate structure material type, size, and foundation support.

Design Life: The minimum service life shall be 75 years.

Usage: Limited to lower-volume roadways (ADT < 1500), unless approved otherwise by Bureau of Structures. Not allowed on Interstate Highways or Divided US Highways.

Cover: The minimum depth of cover shall be 2 ft measured from top of pavement to top of structure. For pipe and pipe arches, refer to FDM 13-1 for maximum depth of cover. For metal box culverts, the maximum depth of cover shall be 5 ft.

Backfill: Place structural backfill equally on both sides of the structure in 8-inch maximum loose lifts. Compact all backfill to 95% of maximum dry density as determined by AASHTO T-99. Backfill shall be free draining and meet the gradation and electrochemical requirements as provided in the most current special provision bid item “Wall Concrete Panel Mechanically Stabilized Earth”.

Membrane: Provide an impervious isolation membrane that extends 10-feet beyond each side of the structure with a minimum thickness of 30 mils (ASTM 5199), regardless of the service life analysis. Membrane shall be sloped to suitable drainage with watertight seams.

Wingwalls: If wingwalls are used, a design shall be provided and supported independently from the metal structure. Metal wingwalls or headers are prohibited, unless approved otherwise by Bureau of Structures.

Guidelines for selecting material type shall be based on engineering judgement and industry practices. Refer to FDM 13-1 for additional requirements on material selection.

36.13.3.1 Metal Pipes and Pipe-Arches

FDM 13-1 provides design guidance and design fill height tables for pipe and pipe-arch shapes. This includes corrugated and structural plates sections for steel and aluminum alloy structures. These fill height tables provide a list of available sizes, minimum metal thicknesses, and depth of cover requirements. Note: the provided minimum metal thicknesses do not consider corrosive and/or abrasive conditions. Structure selection shall be evaluated on a project-by-project basis.

36.13.3.2 Other Shapes

The box culvert shape has been used on locally funded projects and may be an alternative for sites with low clearance that require a wide waterway opening. These semi-rigid structures gain strength through soil-structure interactions and flexural resistance through structural steel plates and reinforcing ribs. While the metal box culvert shape does have its benefits, corrosion concerns and the inability to inspect soil-side flexural members should be considered when selecting a structure type.

36.14 References

1. Wisconsin Highway Research Program (WHRP), *Performance and Policy Related to Aluminum Culverts in Wisconsin*, WisDOT, May 2019. Report No. 0092-17-05

36.15 Design Example

E36-1 Twin Cell Box Culvert LRFD

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E36-1 Twin Cell Box Culvert LRFD

This example shows design calculations for a twin cell box culvert. The *AASHTO LRFD Bridge Design Specifications* are followed as stated in the text of this chapter. ***(Example is current through LRFD Seventh Edition - 2016 Interim)***

E36-1.1 Design Criteria

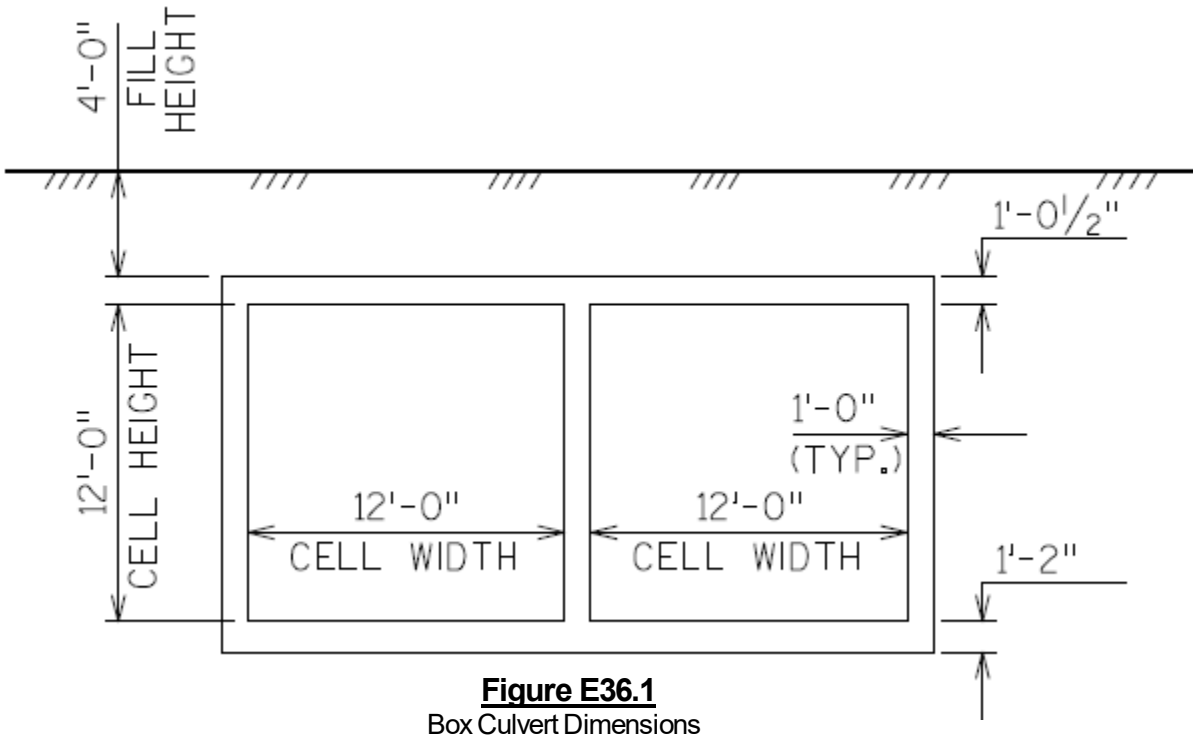


Figure E36.1
Box Culvert Dimensions

$NC = 2$	number of cells
$Ht = 12.0$	cell height, ft
$W_1 = 12.0$	cell 1 width, ft
$W_2 = 12.0$	cell 2 width, ft
$L = 134.0$	culvert length, ft
$t_{ts} = 12.5$	top slab thickness, in
$t_{bs} = 14.0$	bottom slab thickness, in
$t_{win} = 12.0$	interior wall thickness, in
$t_{wex} = 12.0$	exterior wall thickness, in
$H_{apron} := Ht + \frac{t_{ts}}{12}$	apron wall height above floor, ft

$H_{apron} = 13.04$ ft.

$f'_c := 3.5$	culvert concrete strength, ksi
$f_y := 60$	reinforcement yield strength, ksi
$E_s := 29000$	modulus of elasticity of steel, ksi
$skew = 0.0$	skew angle, degrees
$H_s = 4.00$	depth of backfill above top edge of top slab, ft
$w_c := 0.150$	weight of concrete, kcf
$cover_{bot} := 3$	concrete cover (bottom of bottom slab), in
$cover := 2$	concrete cover (all other applications), in
$LS_{ht} := 2.2$	live load surcharge height, ft (See Sect. 36.4.4)

Resistance factors, reinforced concrete cast-in-place box structures, **LRFD [Table 12.5.5-1]**

$\phi_f := 0.9$	resistance factor for flexure
$\phi_v := 0.85$	resistance factor for shear

Calculate the span lengths for each cell (measured between centerlines of walls)

$$S_1 := W_1 + \frac{1}{12}\left(\frac{t_{win}}{2} + \frac{t_{wex}}{2}\right) \qquad \boxed{S_1 = 13.00} \text{ ft}$$

$$S_2 = W_2 + \frac{1}{12}\left(\frac{t_{wex}}{2} + \frac{t_{win}}{2}\right) \qquad \boxed{S_2 = 13.00} \text{ ft}$$

Verify that the box culvert dimensions fall within WisDOT's minimum dimension criteria. Per Sect. 36.2, the minimum size for pedestrian underpasses is 8 feet high by 10 feet wide. The minimum size for cattle underpass is 6 feet high by 5 feet wide. A minimum height of 5 feet is desirable for cleanout purposes.

Does the culvert meet the minimum dimension criteria? $\boxed{check = \text{"OK"}}$

Verify that the slab and wall thicknesses fall within WisDOT's minimum dimension criteria. Per Sect. 36.5, the minimum thickness of the top and bottom slab is 6.5 inches. Per Sect. 36.5 [Table 36.5-1], the minimum wall thickness varies with respect to cell height and apron wall height.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

Do the slab and wall thicknesses meet the minimum dimension criteria? check = "OK"

Since this example has more than 2.0 feet of fill, edge beams are not req'd, **LRFD [C12.11.2.1]**

E36-1.2 Modulus of Elasticity of Concrete Material

Per Sect. 36.2.1, use f'_c = 3.5 ksi for culverts. Calculate value of E_C per **LRFD [C5.4.2.4]**:

$K_1 := 1$ $\quad E_{c_calc} := 33000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$ $\quad E_{c_calc} = 3586.616$ ksi

$E_c := 3600$ ksi modulus of elasticity of concrete, per Sect. 9.2

E36-1.3 Loads

$\gamma_s := 0.120$ unit weight of soil, kcf

Per Sect. 36.5, a haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Minimum haunch depth and length is 6 inches. Haunch depth is increased in 3 inch increments. For the first iteration, assume there are no haunches.

$h_{hau} := 0.0$ haunch height, in

$l_{hau} := 0.0$ haunch length, in

$wt_{hau} = 0.0$ weight of one haunch, kip

E36-1.3.1 Dead Loads

Dead Load (DC):

top slab dead load:

$$w_{dlts} := w_c \cdot \frac{t_{ts}}{12} \cdot 1$$

$\boxed{w_{dlts} = 0.156}$ klf

bottom slab dead load:

$$w_{dlbs} := w_c \cdot \frac{t_{bs}}{12} \cdot 1$$

$\boxed{w_{dlbs} = 0.175}$ klf

Wearing Surface (DW):

Per Sect. 36.4.2, the weight of the future wearing surface is zero if there is any fill depth over the culvert. If there is no fill depth over the culvert, the weight of the future wearing surface shall be taken as 0.020 ksf.

$w_{ws} = 0.000$ weight of future wearing surface, ksf

Vertical Earth Load (EV):

Calculate the modification of earth loads for soil-structure interaction per **LRFD [12.11.2.2]**. Per the policy item in Sect. 36.4.3, embankment installations are always assumed.

Installation_Type = "Embankment"

$\gamma_s = 0.120$ unit weight of soil, kcf

$B_c = 27.00$ outside width of culvert, ft
(measured between outside faces of exterior walls)

$H_s = 4.00$ depth of backfill above top edge of top slab, ft

Calculate the soil-structure interaction factor for embankment installations:

$$F_e := 1 + 0.20 \cdot \frac{H_s}{B_c}$$

$\boxed{F_e = 1.03}$

F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section:

$\boxed{F_e = 1.03}$

Calculate the total unfactored earth load:

$W_E := F_e \cdot \gamma_s \cdot B_c \cdot H_s$ $\boxed{W_E = 13.34}$ klf

Distrubute the total unfactored earth load to be evenly distributed across the top of the culvert:

$w_{sv} := \frac{W_E}{B_c}$ $\boxed{w_{sv} = 0.494}$

Horizontal Earth Load (EH):

Soil horizontal earth load (magnitude at bottom and top of wall): **LRFD [3.11.5.1]**

$k_o := 0.5$ coefficient of at rest lateral earth pressure per Sect. 36.4.3

$\gamma_s = 0.120$ unit weight of soil, kcf

$w_{sh_bot} := k_o \cdot \gamma_s \cdot \left(Ht + \frac{t_{ts}}{12} + \frac{t_{bs}}{12} + H_s\right) \cdot 1$ $\boxed{w_{sh_bot} = 1.09}$ klf

$w_{sh_top} := k_o \cdot \gamma_s \cdot (H_s) \cdot 1$ $\boxed{w_{sh_top} = 0.24}$ klf

Live Load Surcharge (LS):

Soil live load surcharge: **LRFD [3.11.6.4]**

$k_o = 0.5$ coefficient of lateral earth pressure

$\gamma_s = 0.120$ unit weight of soil, kcf

$LS_{ht} = 2.2$ live load surcharge height per Sect. 36.4.4, ft

$w_{sll} := k_o \cdot \gamma_s \cdot LS_{ht} \cdot 1$ $\boxed{w_{sll} = 0.13}$ klf

E36-1.3.2 Live Loads

For Strength 1 and Service 1:

HL-93 loading = design truck (no lane) **LRFD [3.6.1.3.3]**

design tandem (no lane)

For the Wisconsin Standard Permit Vehicle (Wis-SPV) Check:

The Wis-SPV vehicle is to be checked during the design phase to make sure it can carry a minimum vehicle load of 190 kips. See Section 36.1.3 of the Bridge Manual for requirements pertaining to the Wis-SPV vehicle check.

E36-1.4 Live Load Distribution

Live loads are distributed over an equivalent area, with distribution components both parallel and perpendicular to the span, as calculated below. Per **LRFD [3.6.1.3.3]**, the live loads to be placed on these widths are axle loads (i.e., two lines of wheels) without the lane load. The equivalent distribution width applies for both live load moment and shear.

E36-1.5 Equivalent Strip Widths for Box Culverts

The calculations for depths of fill less than 2.0 ft, per **LRFD [4.6.2.10]** are not required for this example. The calcuations are shown for illustration purposes only.

The calculations below follow **LRFD [4.6.2.10.2]** - Case 1: Traffic Travels Parallel to Span. If traffic travels perpendicular to the span, follow **LRFD [4.6.2.10.3]** - Case 2: Traffic Travels Perpendicular to Span, which states to follow **LRFD [4.6.2.1]**.

Per **LRFD [4.6.2.10.2]**, when traffic travels primarily parallel to the span, culverts shall be analyzed for a single loaded lane with a single lane multiple presence factor (mpf).

Therefore, $mpf = 1.2$

Perpendicular to the span:

It is conservative to use the largest distribution factor from each span of the structure across the entire length of the culvert. Therefore, use the smallest span to calculate the smallest strip width. That strip width will provide the largest distribution factor.

$S := \min(W_1, W_2)$ clear span, ft $\boxed{S = 12.00}$ ft

The equivalent distribution width perpendicular to the span is:

$E_{perp} := \frac{1}{12} \cdot (96 + 1.44 \cdot S)$ $\boxed{E_{perp} = 9.44}$ ft

Parallel to the span:

$H_s = 4.00$ depth of backfill above top edge of top slab, ft

$L_T := 10$ length of tire contact area, in **LRFD [3.6.1.2.5]**

$LLDF = 1.15$ live load distribution factor. From **LRFD [4.6.2.10.2]**, LLDF = 1.15 as specified in **LRFD [Table 3.6.1.2.6a-1]** for select granular backfill

The equivalent distribution width parallel to the span is:

$E_{parallel} := \frac{1}{12} \cdot (L_T + LLDF \cdot H_s \cdot 12)$ $\boxed{E_{parallel} = 5.43}$ ft

The equivalent distribution widths parallel and perpendicular to the span create an area that the axial load shall be distributed over. The equivalent area is:

$E_{area} := E_{perp} \cdot E_{parallel}$ $\boxed{E_{area} = 51.29}$ ft^2

For depths of fill 2.0 ft. or greater calculate the size of the rectangular area that the wheels are considered to be uniformly distributed over, per Sect. 36.4.6.2.

$L_T = 10.0$ length of tire contact area, in **LRFD [3.6.1.2.5]**

$W_T := 20$ width of tire contact area, in **LRFD [3.6.1.2.5]**

The length and width of the equivalent area for 1 wheel are: **LRFD [3.6.1.2.6b]**

$L_{eq_i} := L_T + LLDF \cdot H_s \cdot 12$ $\boxed{L_{eq_i} = 65.20}$ in

$W_{eq_i} := W_T + LLDF \cdot H_s \cdot 12 + 0.06 \cdot \max(W_1, W_2)\, 12$ $\boxed{W_{eq_i} = 83.84}$ in

Where such areas from several wheels overlap, the total load shall be uniformly distributed over the area, **LRFD [3.6.1.2.6a]**.

Check if the areas overlap = "Yes, the areas overlap" therefore, use the following length and width values for the equivalent area for 1 wheel:

	Front and Rear Wheels:		Center Wheel:	
Length	$L_{eq13} = 65.2$	in	$L_{eq2} = 65.2$	in
Width	$W_{eq13} = 77.9$	in	$W_{eq2} = 77.9$	in
Area	$A_{eq13} = 5080.4$	in^2	$A_{eq2} = 5080.4$	in^2

Per **LRFD [3.6.1.2.2]**, the weights of the design truck wheels are below. (Note that one axle load is equal to two wheel loads.)

$W_{wheel1i} := 4000$ front wheel weight, lbs

$W_{wheel23i} := 16000$ center and rear wheel weights, lbs

The effect of single and multiple lanes shall be considered. For this problem, a single lane with the single lane multiple presence factor (mpf) governs. Applying the single lane multiple presence factor:

$W_{wheel1} := mpf \cdot W_{wheel1i}$ $W_{wheel1} = 4800.00$ lbs $mpf = 1.20$

$W_{wheel23} := mpf \cdot W_{wheel23i}$ $W_{wheel23} = 19200.00$ lbs

For single-span culverts, the effects of the live load may be neglected where the depth of fill is more than 8.0 feet and exceeds the span length. For multiple span culverts, the effects of the live load may be neglected where the depth of fill exceeds the distance between faces of endwalls, **LRFD [3.6.1.2.6a]**.
Note: The wheel pressure values shown here are for the 14'-0" variable axle spacing of the design truck, which controls over the design tandem for this example. In general, all variable axle spacings of the design truck and the design tandem must be investigated to account for the maximum response. Dividing the wheel loads (incl. mpf) by the equivalent area gives:

$LL1 = 0.94$ live load pressure (front wheel), psi

$LL2 = 3.78$ live load pressure (center wheel), psi

$LL3 = 3.78$ live load pressure (rear wheel), psi

E36-1.6 Limit States and Combinations

The limit states, load factors and load combinations shall be applied as required and detailed in Chapter 36 of this manual and as indicated below.

E36-1.6.1 Load Factors

From **LRFD [Table 3.4.1-1]** and **LRFD [Table 3.4.1-2]**:

Per the policy item in Sect. 36.4.3: Assume box culverts are closed, rigid frames for Strength 1 (EV-factor).

	Strength 1	Service 1
DC	$\gamma st_{DCmax} := 1.25$	$\gamma s1_{DC} := 1.0$
	$\gamma st_{DCmin} := 0.9$	
DW	$\gamma st_{DWmax} := 1.5$	$\gamma s1_{DW} := 1.0$
	$\gamma st_{DWmin} := 0.65$	
EV	$\gamma st_{EVmax} := 1.35$	$\gamma s1_{EV} := 1.0$
	$\gamma st_{EVmin} := 0.9$	
EH	$\gamma st_{EHmax} := 1.35$	$\gamma s1_{EH} := 1.0$
	$\gamma st_{EHmin} := 0.5$ **LRFD [3.11.7]**	
LS	$\gamma st_{LSmax} := 1.75$	$\gamma s1_{LS} := 1.0$
	$\gamma st_{LSmin} := 0$	
LL	$\gamma st_{LL} := 1.75$	$\gamma s1_{LL} := 1.0$

Dynamic Load Allowance (IM) is applied to the truck and tandem. From **LRFD [3.6.2.2]**, IM of buried components varies with depth of cover above the structure and is calculated as:

$IM := 33 \cdot (1.0 - 0.125 \cdot H_s)$ (where H_s is in feet) $IM = 16.50$

If IM is less than 0, use IM = 0 $\boxed{IM = 16.50}$

E36-1.6.2 Dead Load Moments and Shears

The unfactored dead load moments and shears for each component are listed below (values are per 1-foot width and are in kip-ft and kip, respectively):

Exterior Wall Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-1.52	-1.44	-5.14	-1.01	0.00
0.1	-1.42	-1.54	-0.12	-0.14	0.00
0.2	-1.31	-1.63	3.53	0.55	0.00
0.3	-1.21	-1.73	5.92	1.04	0.00
0.4	-1.10	-1.82	7.14	1.34	0.00
0.5	-1.00	-1.91	7.30	1.46	0.00
0.6	-0.89	-2.01	6.51	1.38	0.00
0.7	-0.79	-2.10	4.87	1.12	0.00
0.8	-0.68	-2.19	2.49	0.66	0.00
0.9	-0.58	-2.29	-0.54	0.01	0.00
1.0	-0.48	-2.38	-4.11	-0.82	0.00

Interior Wall Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.00	0.00	0.00	0.00	0.00
0.1	0.00	0.00	0.00	0.00	0.00
0.2	0.00	0.00	0.00	0.00	0.00
0.3	0.00	0.00	0.00	0.00	0.00
0.4	0.00	0.00	0.00	0.00	0.00
0.5	0.00	0.00	0.00	0.00	0.00
0.6	0.00	0.00	0.00	0.00	0.00
0.7	0.00	0.00	0.00	0.00	0.00
0.8	0.00	0.00	0.00	0.00	0.00
0.9	0.00	0.00	0.00	0.00	0.00
1.0	0.00	0.00	0.00	0.00	0.00

Top Slab Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-0.04	-1.14	-5.47	-1.18	0.00
0.1	0.73	1.45	-4.67	-1.00	0.00
0.2	1.27	3.32	-3.87	-0.83	0.00
0.3	1.60	4.48	-3.07	-0.66	0.00
0.4	1.69	4.93	-2.27	-0.49	0.00
0.5	1.56	4.67	-1.47	-0.32	0.00
0.6	1.21	3.69	-0.67	-0.15	0.00
0.7	0.63	2.01	0.13	0.03	0.00
0.8	-0.18	-0.39	0.93	0.20	0.00
0.9	-1.21	-3.50	1.72	0.37	0.00
1.0	-2.46	-7.32	2.52	0.54	0.00

Bottom Slab Unfactored Dead Load Moments (kip-ft)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	-0.60	-0.17	-7.63	-1.42	0.00
0.1	1.36	2.26	-6.51	-1.21	0.00
0.2	2.76	3.98	-5.39	-1.00	0.00
0.3	3.61	4.99	-4.27	-0.79	0.00
0.4	3.91	5.29	-3.15	-0.59	0.00
0.5	3.65	4.87	-2.03	-0.38	0.00
0.6	2.85	3.75	-0.90	-0.17	0.00
0.7	1.49	1.91	0.22	0.04	0.00
0.8	-0.42	-0.64	1.34	0.25	0.00
0.9	-2.88	-3.90	2.46	0.46	0.00
1.0	-5.89	-7.88	3.58	0.67	0.00

Exterior Wall Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.09	-0.08	4.78	0.73	0.00
0.1	0.09	-0.08	3.60	0.59	0.00
0.2	0.09	-0.08	2.50	0.45	0.00
0.3	0.09	-0.08	1.49	0.30	0.00
0.4	0.09	-0.08	0.56	0.16	0.00
0.5	0.09	-0.08	-0.27	0.01	0.00
0.6	0.09	-0.08	-1.03	-0.13	0.00
0.7	0.09	-0.08	-1.69	-0.27	0.00
0.8	0.09	-0.08	-2.27	-0.42	0.00
0.9	0.09	-0.08	-2.76	-0.56	0.00
1.0	0.09	-0.08	-3.17	-0.71	0.00

Interior Wall Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.00	0.00	0.00	0.00	0.00
0.1	0.00	0.00	0.00	0.00	0.00
0.2	0.00	0.00	0.00	0.00	0.00
0.3	0.00	0.00	0.00	0.00	0.00
0.4	0.00	0.00	0.00	0.00	0.00
0.5	0.00	0.00	0.00	0.00	0.00
0.6	0.00	0.00	0.00	0.00	0.00
0.7	0.00	0.00	0.00	0.00	0.00
0.8	0.00	0.00	0.00	0.00	0.00
0.9	0.00	0.00	0.00	0.00	0.00
1.0	0.00	0.00	0.00	0.00	0.00

Top Slab Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	0.74	2.45	0.67	0.13	0.00
0.1	0.55	1.86	0.67	0.13	0.00
0.2	0.36	1.26	0.67	0.13	0.00
0.3	0.17	0.67	0.67	0.13	0.00
0.4	-0.01	0.08	0.67	0.13	0.00
0.5	-0.20	-0.52	0.67	0.13	0.00
0.6	-0.39	-1.11	0.67	0.13	0.00
0.7	-0.58	-1.70	0.67	0.13	0.00
0.8	-0.76	-2.30	0.67	0.13	0.00
0.9	-0.95	-2.89	0.67	0.13	0.00
1.0	-1.14	-3.48	0.67	0.13	0.00

Bottom Slab Unfactored Dead Load Shears (kip)					
Tenth Point (Along Span)	DC	EV	EH	LS	DW
0.0	1.86	2.32	0.94	0.16	0.00
0.1	1.40	1.73	0.94	0.16	0.00
0.2	0.94	1.14	0.94	0.16	0.00
0.3	0.48	0.54	0.94	0.16	0.00
0.4	0.02	-0.05	0.94	0.16	0.00
0.5	-0.44	-0.64	0.94	0.16	0.00
0.6	-0.90	-1.24	0.94	0.16	0.00
0.7	-1.36	-1.83	0.94	0.16	0.00
0.8	-1.82	-2.42	0.94	0.16	0.00
0.9	-2.28	-3.01	0.94	0.16	0.00
1.0	-2.74	-3.61	0.94	0.16	0.00

The DC values are the component dead loads and include the self weight of the culvert and haunch (if applicable).

The DW values are the dead loads from the future wearing surface (DW values occur only if there is no fill on the culvert).

The EV values are the vertical earth loads from the fill on top of the box culvert.

The EH values are the horizontal earth loads from the fill on the sides of the box culvert.

The LS values are the live load surcharge loads (assuming $LS_{ht} = 2.2$ feet of surcharge)

E36-1.6.3 Live Load Moments and Shears

The unfactored live load load moments and shears (per lane including impact) are listed below (values are in kip-ft and kips, respectively). A separate analysis run will be required if results without impact are desired.

Exterior Wall Unfactored Live Load Moments (kip-ft)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.73	-1.74	0.74	-1.77
0.1	0.67	-1.70	0.69	-1.92
0.2	0.61	-1.67	0.65	-2.07
0.3	0.55	-1.65	0.62	-2.21
0.4	0.48	-1.68	0.60	-2.36
0.5	0.42	-1.82	0.58	-2.51
0.6	0.37	-1.97	0.56	-2.69
0.7	0.41	-2.12	0.56	-2.86
0.8	0.47	-2.28	0.61	-3.04
0.9	0.55	-2.44	0.68	-3.21
1.0	0.65	-2.61	0.77	-3.39

Interior Wall Unfactored Live Load Moments (kip-ft)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.99	-0.99	0.88	-0.88
0.1	0.93	-0.93	0.99	-0.99
0.2	0.92	-0.92	1.12	-1.12
0.3	0.90	-0.90	1.25	-1.25
0.4	0.90	-0.90	1.38	-1.38
0.5	1.08	-1.08	1.54	-1.53
0.6	1.27	-1.27	1.74	-1.74
0.7	1.47	-1.47	1.99	-1.99
0.8	1.69	-1.69	2.24	-2.24
0.9	1.92	-1.92	2.50	-2.50
1.0	2.17	-2.17	2.75	-2.75

Top Slab Unfactored Live Load Moments (kip-ft)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.81	-1.76	0.65	-2.16
0.1	2.24	-0.34	1.83	-0.20
0.2	3.81	-0.27	4.23	-0.32
0.3	5.06	-0.49	5.92	-0.66
0.4	5.71	-0.75	6.78	-1.04
0.5	5.76	-1.04	6.90	-1.43
0.6	5.22	-1.34	6.21	-1.82
0.7	4.13	-1.64	4.74	-2.22
0.8	2.56	-1.96	2.54	-2.62
0.9	0.86	-3.59	0.76	-3.02
1.0	0.07	-5.89	0.06	-4.81

Bottom Slab Unfactored Live Load Moments (kip-ft)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.46	-0.67	0.40	-0.35
0.1	1.72	-0.29	2.52	-0.32
0.2	3.30	-0.76	4.46	-0.78
0.3	4.25	-1.06	5.63	-1.09
0.4	4.60	-1.24	6.06	-1.30
0.5	4.39	-1.34	5.82	-1.45
0.6	3.68	-1.39	4.96	-1.62
0.7	2.56	-1.46	3.55	-1.86
0.8	1.18	-1.57	1.62	-2.23
0.9	0.00	-2.40	0.00	-2.79
1.0	0.00	-4.90	0.00	-3.75

Exterior Wall Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.11	-0.19	0.09	-0.16
0.1	0.11	-0.19	0.09	-0.16
0.2	0.11	-0.19	0.09	-0.16
0.3	0.11	-0.19	0.09	-0.16
0.4	0.11	-0.19	0.09	-0.16
0.5	0.11	-0.19	0.09	-0.16
0.6	0.11	-0.19	0.09	-0.16
0.7	0.11	-0.19	0.09	-0.16
0.8	0.11	-0.19	0.09	-0.16
0.9	0.11	-0.19	0.09	-0.16
1.0	0.11	-0.19	0.09	-0.16

Interior Wall Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	0.23	-0.23	0.21	-0.21
0.1	0.23	-0.23	0.21	-0.21
0.2	0.23	-0.23	0.21	-0.21
0.3	0.23	-0.23	0.21	-0.21
0.4	0.23	-0.23	0.21	-0.21
0.5	0.23	-0.23	0.21	-0.21
0.6	0.23	-0.23	0.21	-0.21
0.7	0.23	-0.23	0.21	-0.21
0.8	0.23	-0.23	0.21	-0.21
0.9	0.23	-0.23	0.21	-0.21
1.0	0.23	-0.23	0.21	-0.21

Top Slab Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	2.71	-0.26	3.24	-0.33
0.1	2.33	-0.33	2.67	-0.33
0.2	1.95	-0.47	2.11	-0.33
0.3	1.56	-0.69	1.59	-0.39
0.4	1.19	-1.00	1.14	-0.67
0.5	0.85	-1.37	0.78	-1.03
0.6	0.54	-1.74	0.49	-1.46
0.7	0.30	-2.10	0.27	-1.97
0.8	0.14	-2.44	0.12	-2.54
0.9	0.04	-2.76	0.04	-3.11
1.0	0.00	-3.05	0.00	-3.66

Bottom Slab Unfactored Live Load Shears (kip)				
Tenth Point (Along Span)	Truck		Tandem	
	Max	Min	Max	Min
0.0	2.19	-0.68	2.69	-0.68
0.1	1.61	-0.48	1.97	-0.48
0.2	1.06	-0.32	1.29	-0.32
0.3	0.54	-0.19	0.66	-0.21
0.4	0.06	-0.11	0.07	-0.14
0.5	0.01	-0.45	0.00	-0.46
0.6	0.02	-0.90	0.02	-0.96
0.7	0.02	-1.33	0.02	-1.40
0.8	0.01	-1.74	0.01	-1.80
0.9	0.00	-2.12	0.00	-2.15
1.0	0.00	-2.48	0.00	-2.46

E36-1.6.4 Factored Moments

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored moments for each limit state are calculated by applying the appropriate load factors to loads on a 1-foot strip width of the box culvert. The minimum or maximum load factors may be used on each component to maximize the load effects. The results are as follows:

Strength 1 Moments

$$M_{str1} = \eta \cdot \left(\gamma st_{DC} \cdot M_{DC} + \gamma st_{DW} \cdot M_{DW} + \gamma st_{EV} \cdot M_{EV} + \gamma st_{EH} \cdot M_{EH} + \gamma st_{LS} \cdot M_{LS} + \gamma st_{LL} \cdot M_{LL}\right)$$

Corner Bars	$Mstr1_{CB} = 16.73$	kip-ft	(negative moment)
Positive Moment Top Slab Bars	$Mstr1_{PTS} = 19.59$	kip-ft	(positive moment)
Positive Moment Bottom Slab Bars	$Mstr1_{PBS} = 21.05$	kip-ft	(positive moment)
Negative Moment Top Slab Bars	$Mstr1_{NTS} = 22.00$	kip-ft	(negative moment)
Negative Moment Bottom Slab Bars	$Mstr1_{NBS} = 24.77$	kip-ft	(negative moment)
Exterior Wall Bars	$Mstr1_{XW} = 10.81$	kip-ft	(positive moment)
Interior Wall Bars	$Mstr1_{IW} = 4.82$	kip-ft	(positive moment)

Service 1 Moments

$$M_{s1} = \eta \cdot \left(\gamma s1_{DC} \cdot M_{DC} + \gamma s1_{DW} \cdot M_{DW} + \gamma s1_{EV} \cdot M_{EV} + \gamma s1_{EH} \cdot M_{EH} + \gamma s1_{LS} \cdot M_{LS} + \gamma s1_{LL} \cdot M_{LL}\right)$$

Corner Bars	$Ms1_{CB} = 11.18$	kip-ft	(negative moment)
Positive Moment Top Slab Bars	$Ms1_{PTS} = 11.66$	kip-ft	(positive moment)
Positive Moment Bottom Slab Bars	$Ms1_{PBS} = 12.32$	kip-ft	(positive moment)
Negative Moment Top Slab Bars	$Ms1_{NTS} = 13.15$	kip-ft	(negative moment)
Negative Moment Bottom Slab Bars	$Ms1_{NBS} = 15.08$	kip-ft	(negative moment)
Exterior Wall Bars	$Ms1_{XW} = 6.43$	kip-ft	(positive moment)
Interior Wall Bars	$Ms1_{IW} = 2.75$	kip-ft	(positive moment)

E36-1.7 Design Reinforcement Bars

Design of the corner bars is illustrated below. Calculations for bars in other locations are similar.

Design Criteria:

For corner bars, use the controlling thickness between the slab and wall. The height of the concrete design section is:

$h := \min(t_{ts}, t_{bs}, t_{wex})$ $\boxed{h = 12.00}$ in

Use a 1'-0" design width:

$b := 12.0$ width of the concrete design section, in

$cover = 2.0$ concrete cover, in Note: The calculations here use 2" cover for the top slab and walls. Use 3" cover for the bottom of the bottom slab (not shown here).

$Mstr1_{CB} = 16.73$ design strength moment, kip-ft

$Ms1_{CB} = 11.18$ design service moment, kip-ft

$f_s := f_y$ reinforcement yield strength, ksi $f_y = 60.00$ ksi

$Bar_{No} := 5$ assume #5 bars (for d_s calculation)

$Bar_D(Bar_{No}) = 0.63$ bar diameter, in

Calculate the estimated distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement. **LRFD [5.7.3.2.2]**

$$d_{s_i} := h - cover - \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_{s_i} = 9.69}$ in

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ per **LRFD [Table 12.5.5-1]**.

Calculate the coefficient of resistance:

$$R_n := \frac{Mstr1_{CB} \cdot 12}{\phi_f \cdot b \cdot d_{s_i}^2}$$ $\boxed{R_n = 0.20}$ ksi

Calculate the reinforcement ratio:

$$\rho := 0.85 \cdot \frac{f'_c}{f_y} \cdot \left(1 - \sqrt{1.0 - \frac{2 \cdot R_n}{0.85 \cdot f'_c}}\right)$$ $\boxed{\rho = 0.0034}$

Calculate the required area of steel:

$A_{s_req'd} := \rho \cdot b \cdot d_{s_i}$ $\boxed{A_{s_req'd} = 0.40}$ in^2

Given the required area of steel of $A_{s_req'd} = 0.40$, try #5 bars at 7.5" spacing:

$Bar_{No} := 5$ bar size

$spacing := 7.0$ bar spacing, in

The area of one reinforcing bar is:

$A_{s_1bar} := Bar_A(Bar_{No})$ $\boxed{A_{s_1bar} = 0.31}$ in^2

Calculate the area of steel in a 1'-0" width

$$A_s := \frac{A_{s_1bar}}{\frac{spacing}{12}}$$ $\boxed{A_s = 0.53}$ in^2

Check that the area of steel provided is larger than the required area of steel

Is $A_s = 0.53\ in^2 \geq A_{s_req'd} = 0.40\ in^2$ $\boxed{check = "OK"}$

Recalculate d_c and d_s based on the actual bar size used.

$$d_c := cover + \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_c = 2.31}$ in

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2}$$ $\boxed{d_s = 9.69}$ in

Per **LRFD [5.7.2.2]**, The factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi. For concrete strengths exceeding 4.0 ksi, β_1 shall be reduced at a rate of 0.05 for each 1.0 ksi of strength in excess of 4.0 ksi, except that β_1 shall not be taken to be less than 0.65. The factor α_1 shall be taken as 0.85 for concrete strength not exceeding 10.0 ksi.

$\boxed{\beta_1 = 0.85}$ $\boxed{\alpha_1 = 0.85}$

Per **LRFD [5.7.2.1]**, if $\frac{c}{d_s} \leq 0.6$ (for f_y = 60 ksi) then reinforcement has yielded and the assumption is correct.

"c" is defined as the distance between the neutral axis and the compression face (inches).

$$c := \frac{A_s \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b}$$ $\boxed{c = 1.05}$ in

Check that the reinforcement will yield:

Is $\frac{c}{d_s} = 0.11 \leq 0.6$? $\boxed{check = "OK"}$ therefore, the reinforcement will yield

Calculate the nominal moment capacity of the rectangular section in accordance with **LRFD [5.7.3.2.3]**:

$a := \beta_1 \cdot c$ $\boxed{a = 0.89}$ in

$M_n := \left[A_s \cdot f_s \cdot \left(d_s - \frac{a}{2}\right)\frac{1}{12}\right]$ $\boxed{M_n = 24.6}$ kip-ft

For reinforced concrete cast-in-place box structures, $\phi_f = 0.90$ **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 22.1}$ kip-ft

The required capacity:

Corner Moment $\boxed{Mstr1_{CB} = 16.7}$ kip-ft

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]**:

$b = 12.0$ in width of the concrete design section, in

$h = 12.0$ in height of the concrete design section, in

$f_r = 0.24 \cdot \lambda \sqrt{f'_c}$ = modulus of rupture (ksi) **LRFD [5.4.2.6]**

$f_r := 0.24 \cdot \sqrt{f'_c}$ λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]** $f_r = 0.45$ ksi

$I_g := \frac{1}{12} \cdot b \cdot h^3$ gross moment of inertia, in^4 $I_g = 1728.00$ in^4

$\frac{h}{2} = 6.0$ distance from the neutral axis to the extreme element

$S_c := \frac{I_g}{\frac{h}{2}}$ section modulus, in^3 $S_c = 288.00$ in^3

The corresponding cracking moment is:

$M_{cr} = \gamma_3(\gamma_1 \cdot f_r)S_c$ therefore, $M_{cr} = 1.1(f_r)S_c$

Where:

$\gamma_1 := 1.6$ flexural cracking variability factor

$\gamma_3 := 0.67$ ratio of yield strength to ultimate tensile strength of the reinforcement <u>for A615, Grade 60 reinforcement</u>

$M_{cr} := 1.1 f_r \cdot S_c \cdot \frac{1}{12}$ $\boxed{M_{cr} = 11.9}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CB} = 22.2}$ kip-ft

Is $M_r = 22.1$ kip-ft greater than the lesser of M_{cr} and $1.33*M_{str}$? $\boxed{check = "OK"}$

Per **LRFD [5.7.3.4]**, the spacing(s) of reinforcement in the layer closest to the tension face shall satisfy:

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \qquad \text{in which:} \qquad \beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

$\gamma_e := 1.0$ for Class 1 exposure condition

$h = 12.0$ height of the concrete design section, in

Calculate the ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face:

$$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} \qquad \boxed{\beta_s = 1.34}$$

Calculate the reinforcement ratio:

$$\rho := \frac{A_s}{b \cdot d_s} \qquad \boxed{\rho = 0.0046}$$

Calculate the modular ratio:

$$N := \frac{E_s}{E_c} \qquad \boxed{N = 8.06}$$

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi). The moment arm used in the equation below to calculate f_{ss} is: (j) ($h-d_c$)

$$k := \sqrt{(\rho \cdot N)^2 + (2 \cdot \rho \cdot N)} - \rho \cdot N \qquad \boxed{k = 0.2370}$$

$$j := 1 - \frac{k}{3} \qquad \boxed{j = 0.9210}$$

$Ms1_{CB} = 11.18$ service moment, kip-ft

$$f_{ss} := \frac{Ms1_{CB} \cdot 12}{A_s \cdot (j) \cdot (h - d_c)} \le 0.6 f_y \qquad \boxed{f_{ss} = 28.29} \text{ ksi} \quad \le 0.6 f_y \text{ O.K.}$$

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$$s_{max1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \qquad s_{max1} = 13.83 \text{ in}$$

$$s_{max2} := \min(1.5 \cdot h, 18) \qquad s_{max2} = 18.00 \text{ in}$$

$$s_{max} := \min(s_{max1}, s_{max2}) \qquad \boxed{s_{max} = 13.83} \text{ in}$$

Check that the provided spacing is less than the maximum allowable spacing

Is spacing = 7.00 in $\leq$ s_{max} = 13.83 in $\boxed{\text{check} = \text{"OK"}}$

Calculate the minimum spacing requirements per **LRFD [5.10.3.1]**. The clear distance between parallel bars in a layer shall not be less than:

$$S_{min1} := 1.5 \cdot Bar_D(Bar_{No}) \qquad S_{min1} = 0.94 \text{ in}$$

$$S_{min2} := 1.5 \cdot 1.5 \quad \text{(maximum aggregate size = 1.5 inches)} \qquad S_{min2} = 2.25 \text{ in}$$

$$S_{min3} := 1.5 \text{ in}$$

Is spacing = 7.00 in $\geq$ all minimum spacing requirements? $\boxed{\text{check} = \text{"OK"}}$

E36-1.8 Shrinkage and Temperature Reinforcement Check

Check shrinkage and temperature reinforcement criteria for the reinforcement selected in preceding sections.

The area of reinforcement (A_s) per foot, for shrinkage and temperature effects, on each face and in each direction shall satisfy: **LRFD [5.10.8]**

$$A_s \geq \frac{1.30 \cdot b \cdot (h)}{2 \cdot (b + h) \cdot f_y} \quad \text{and} \quad 0.11 \leq A_s \leq 0.60$$

Where:

A_s = area of reinforcement in each direction and each face $\left(\frac{in^2}{ft}\right)$

b = least width of component section (in.)

h = least thickness of component section (in.)

f_y = specified yield strength of reinforcing bars (ksi) $\leq$ 75 ksi

Check the minimum required temperature and shrinkage reinforcement, #4 bars at 15", in the thickest section. For the given cross section, the values for the corner bar design are:

$$A_{s_4_at_15} := \frac{Bar_A(4)}{1.25} \qquad \boxed{A_{s_4_at_15} = 0.16} \ \frac{in^2}{ft}$$

$b_{TS} := \max(t_{ts}, t_{bs}, t_{wex})$ $\boxed{b_{TS} = 14.0}$ in

$h_{TS} := 12(W_1 + W_2) + 2 \cdot t_{wex} + t_{win}$ $\boxed{h_{TS} = 324.0}$ in

$f_y = 60.00$ ksi

For each face, the required area of steel is:

$$A_{s_TS} := \frac{1.30 \cdot (b_{TS}) \cdot h_{TS}}{2 \cdot (b_{TS} + h_{TS}) \cdot f_y} \qquad A_{s_TS} = 0.15 \ \frac{in^2}{ft}$$

is $A_{s_4_at_15} = 0.16$ in2 $\geq$ $A_{s_TS} = 0.15$ in^2? $\boxed{\text{check} = \text{"OK"}}$

is $0.11 < A_{s_4_at_15} < 0.60$? $\boxed{\text{check} = \text{"OK"}}$

Per **LRFD [5.10.8]**, the shrinkage and temperature reinforcement shall not be spaced farther apart than:

- 3.0 times the component thickness, or 18.0 in.
- 12.0 in for walls and footings greater than 18.0 in. thick
- 12.0 in for other components greater than 36.0 in. thick

$s_{max3} = 18.00$ in

Per **LRFD [5.10.3.2]**, the maximum center to center spacing of adjacent bars shall not exceed 1.5 times the thickness of the member or 18.0 in.

$s_{max4} = 18.00$ in

is the 15" spacing $\leq$ both maximum spacing requirements? $\boxed{\text{check} = \text{"OK"}}$

Note: The design of the bottom slab shrinkage and temperature bars is illistrated above. Shrinkage and temperature bars may be reduced or not required at other locations. See Section 36.6.8 and Standard 36.03 for additional information.

The results for the other bar locations are shown in the table below:

Results						
Location	ΦMn	$A_{S\ Req'd}$	$A_{S\ Actual}$	Bar Size	S_{max}	S_{actual}
Corner	22.1	0.48	0.53	5	13.8	7.0
Pos. Mom. Top Slab	21.8	0.49	0.50	5	13.0	7.5
Pos. Mom. Bot. Slab	28.9	0.54	0.57	5	18.0	6.5
Neg. Mom. Top Slab	23.3	0.50	0.53	5	12.1	7.0
Neg. Mom. Bot. Slab	28.4	0.54	0.62	5	13.4	6.0
Exterior Wall	16.9	0.34	0.40	4	18.0	6.0
Interior Wall	6.9	0.15	0.16	4	18.0	15.0

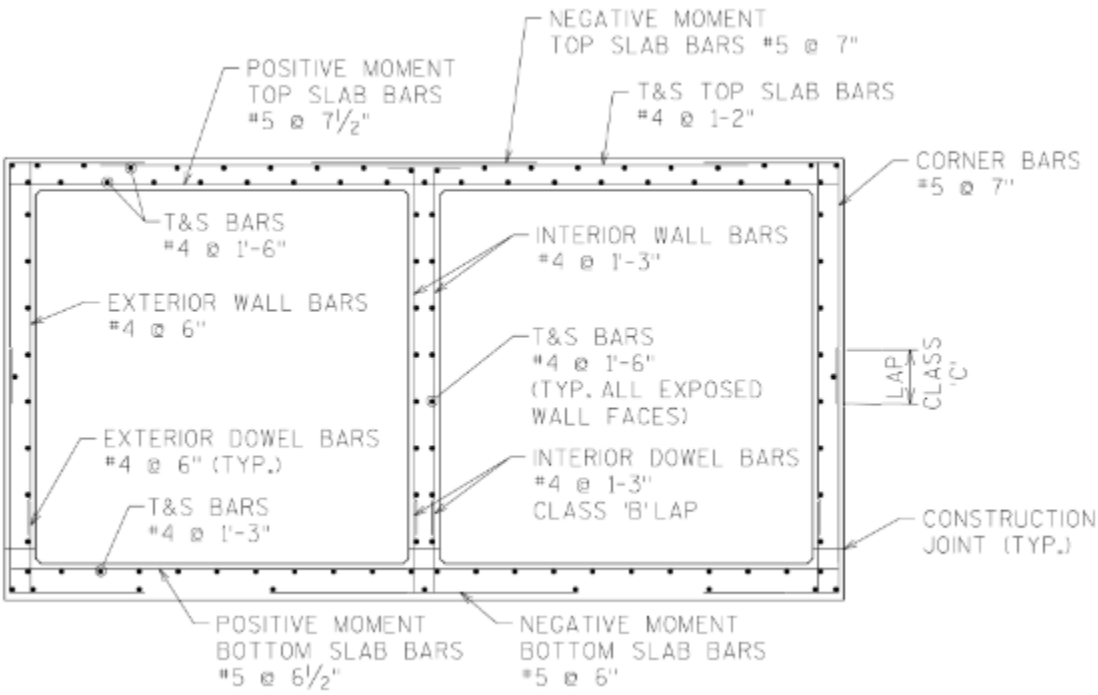
E36-1.9 Distribution Reinforcement

Per **LRFD [9.7.3.2]**, reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows:

Distribution steel is not required when the depth of fill over the slab exceeds 2 feet, **LRFD [5.14.4.1]**.

E36-1.10 Reinforcement Details

The reinforcement bar size and spacing required from the strength and serviceability calcuations above are shown below:



E36-1.11 Cutoff Locations

Determine the cutoff locations for the corner bars. Per Sect. 36.6.1, the distance "L" is computed from the maximum negative moment envelope for the top slab.

The cutoff lengths are in feet, measured from the inside face of the exterior wall.

Initial Cutoff Locations:

The initial cutoff locations are determined from the inflection points of the moment diagrams.

Corner Bars	$CutOff1_{CBH_i} = 2.64$	$CutOff2_{CBH_i} = 1.15$	Horizontal
		$CutOff2_{CBV_i} = 2.07$	Vertical
Positive Moment Top Slab Bars	$CutOff1_{PTS_i} = 1.26$	$CutOff2_{PTS_i} = 1.86$	
Positive Moment Bottom Slab Bars	$CutOff1_{PBS_i} = 1.27$	$CutOff2_{PBS_i} = 1.97$	
Negative Moment Top Slab Bars	$CutOff1_{NTS_i} = 8.63$	$CutOff2_{NTS_i} = 10.32$	
Negative Moment Bottom Slab Bars	$CutOff1_{NBS_i} = 8.97$	$CutOff2_{NBS_i} = 10.56$	

For the second cutoff location for each component, the following checks shall be completed:

Check the section for minimum reinforcement in accordance with **LRFD [5.7.3.3.2]**:

The required capacity at the second cutoff location (for the vertical leg of the corner bar):

$Mstr1_{CBV2} = 7.89$ strength moment at the second cutoff location, kip-ft

The usable capacity of the remaining bars is calculated as follows:

$$A_{s2} := \frac{A_s}{2} \qquad \boxed{A_{s2} = 0.27} \text{ in}^2$$

$$c2 := \frac{A_{s2} \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b} \qquad \boxed{\beta_1 = 0.85} \; \boxed{\alpha_1 = 0.85} \qquad \boxed{c2 = 0.53} \text{ in}$$

$$a2 := \beta_1 \cdot c2 \qquad \boxed{a2 = 0.45} \text{ in}$$

$$M_{n2} := \left[A_{s2} \cdot f_s \cdot \left(d_s - \frac{a2}{2}\right) \frac{1}{12}\right] \qquad \boxed{M_{n2} = 12.6} \text{ kip-ft}$$

$$M_{r2} := \phi_f \cdot M_{n2} \qquad \boxed{M_{r2} = 11.3} \text{ kip-ft}$$

Is $M_{r2} = 11.3$ kip-ft greater than the lesser of M_{cr} and $1.33*M_{str}$? $\boxed{check = "OK"}$

$\boxed{M_{cr} = 11.9}$ kip-ft

$\boxed{1.33 \cdot Mstr1_{CBV2} = 10.5}$ kip-ft

Calculate f_{ss}, the tensile stress in steel reinforcement at the Service I Limit State (ksi).

$Ms1_{CBV2} = 3.43$ service moment at the second cutoff location, kip-ft

$$f_{ss2} := \frac{Ms1_{CBV2} \cdot 12}{A_{s2} \cdot (j) \cdot (h - d_c)}$$ $\boxed{f_{ss2} = 17.35}$ ksi

Calculate the maximum spacing requirements per **LRFD [5.10.3.2]**:

$$s_{max2_1} := \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss2}} - 2 \cdot d_c$$ $s_{max2_1} = 25.47$ in

$s_{max2_2} := s_{max2}$ $s_{max2_2} = 18.00$ in

$s_{max} := \min(s_{max2_1}, s_{max2_2})$ $\boxed{s_{max} = 18.00}$ in

Check that the provided spacing (for half of the bars) is less than the maximum allowable spacing

$spacing2 := 2 \cdot spacing$ $spacing2 = 14.00$ in

Is $spacing2 = 14.00$ in $\leq s_{max} = 18.00$ in $\boxed{check = "OK"}$

Extension Lengths:

The extension lengths for the corner bars are shown below. Calculations for other bars are similar.

Extension lengths for general reinforcement per **LRFD [5.11.1.2.1]**:

$$MaxDepth := \max\left(t_{ts} - cover, t_{wex} - cover, t_{bs} - cover_{bot}\right) \qquad MaxDepth = 11.00 \quad in$$

Effective member depth $$\frac{MaxDepth - \frac{1}{2} Bar_D\left(Bar_{No_CB}\right)}{12} = 0.89 \quad ft$$

15 x bar diameter $$\frac{15 \cdot Bar_D\left(Bar_{No_CB}\right)}{12} = 0.78 \quad ft$$

1/20 times clear span $$\frac{\max\left(W_1, W_2\right)}{20} = 0.60 \quad ft$$

The maximum of the values listed above:

$$ExtendLength_gen_{CB} = 0.89 \quad ft$$

Extension lengths for negative moment reinforcement per **LRFD [5.11.1.2.3]**:

Effective member depth $$\frac{MaxDepth - \frac{1}{2} Bar_D\left(Bar_{No_CB}\right)}{12} = 0.89 \quad ft$$

12 x bar diameter $$\frac{12 \cdot Bar_D\left(Bar_{No_CB}\right)}{12} = 0.63 \quad ft$$

0.0625 times clear span $$0.0625 \max\left(W_1, W_2\right) = 0.75 \quad ft$$

The maximum of the values listed above:

$$ExtendLength_neg_{CB} = 0.89 \quad ft$$

The development length:

$$DevLength_{CB} = 1.00 \quad ft$$

The extension lengths for general reinforcment for the other bars are:

Corner Bars	$ExtendLength_gen_{CB} = 0.89$	ft
Positive Moment Top Slab Bars	$ExtendLength_gen_{PTS} = 0.85$	ft
Positive Moment Bottom Slab Bars	$ExtendLength_gen_{PBS} = 0.97$	ft
Negative Moment Top Slab Bars	$ExtendLength_gen_{NTS} = 0.85$	ft
Negative Moment Bottom Slab Bars	$ExtendLength_gen_{NBS} = 0.97$	ft

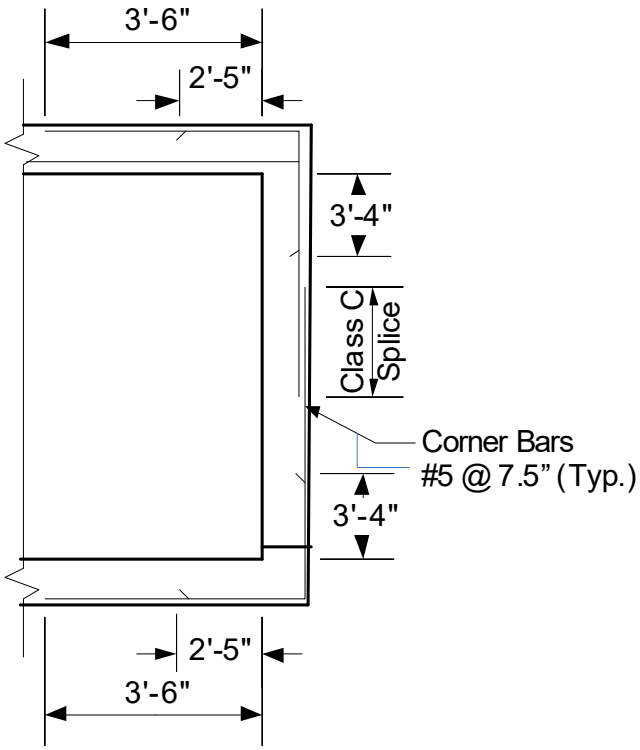
The extension lengths for negative moment reinforcment for the other bars are:

Corner Bars	$ExtendLength_neg_{CB} = 0.89$	ft
Positive Moment Top Slab Bars	$ExtendLength_neg_{PTS} = 0.85$	ft
Positive Moment Bottom Slab Bars	$ExtendLength_neg_{PBS} = 0.97$	ft
Negative Moment Top Slab Bars	$ExtendLength_neg_{NTS} = 0.85$	ft
Negative Moment Bottom Slab Bars	$ExtendLength_neg_{NBS} = 0.97$	ft

The final cutoff locations (measured from the inside face of the exterior wall) are:

Corner Bars	$CutOff1_{CBH}$ = 3.53	$CutOff2_{CBH}$ = 2.04	Horizontal
		$CutOff2_{CBV}$ = 2.96	Vertical
Positive Moment Top Slab Bars	$CutOff1_{PTS}$ = "Run Bar Entire Width of Box"		
		$CutOff2_{PTS}$ = 1.02	
Positive Moment Bottom Slab Bars	$CutOff1_{PBS}$ = "Run Bar Entire Width of Box"		
		$CutOff2_{PBS}$ = 1.00	
Negative Moment Top Slab Bars	$CutOff1_{NTS}$ = 7.78	$CutOff2_{NTS}$ = 9.47	
Negative Moment Bottom Slab Bars	$CutOff1_{NBS}$ = 7.99	$CutOff2_{NBS}$ = 9.59	

The cutoff locations for the corner bars are shown below. Other bars are similar.



E36-1.12 Shear Analysis

Analyze walls and slabs for shear

E36-1.12.1 Factored Shears

WisDOT's policy is to set all of the load modifiers, η, equal to 1.0. The factored shears for each limit state are calculated by applying the appropriate load factors to loads on a 1-foot strip width of the box culvert. The minimum or maximum load factors may be used on each component to maximize the load effects. The results are as follows:

Strength 1 Shears

$$V_{str1} = \eta \cdot \left(\gamma st_{DC} \cdot V_{DC} + \gamma st_{DW} \cdot V_{DW} + \gamma st_{EV} \cdot V_{EV} + \gamma st_{EH} \cdot V_{EH} + \gamma st_{LS} \cdot V_{LS} + \gamma st_{LL} \cdot V_{LL}\right)$$

Exterior Wall	$Vstr1_{XW} = 7.98$	kip
Interior Wall	$Vstr1_{IW} = 0.40$	kip
Top Slab	$Vstr1_{TS} = 12.20$	kip
Bottom Slab	$Vstr1_{BS} = 12.16$	kip

Service 1 Shears

$$V_{s1} = \eta \cdot \left(\gamma s1_{DC} \cdot V_{DC} + \gamma s1_{DW} \cdot V_{DW} + \gamma s1_{EV} \cdot V_{EV} + \gamma s1_{EH} \cdot V_{EH} + \gamma s1_{LS} \cdot V_{LS} + \gamma s1_{LL} \cdot V_{LL}\right)$$

Exterior Wall	$Vs1_{XW} = 5.64$	kip
Interior Wall	$Vs1_{IW} = 0.23$	kip
Top Slab	$Vs1_{TS} = 7.62$	kip
Bottom Slab	$Vs1_{BS} = 7.96$	kip

E36-1.12.2 Concrete Shear Resistance

Check that the nominal shear resistance, V_n, of the concrete in the top slab is adequate for shear without shear reinforcement per **LRFD [5.14.5.3]**.

$$V_n = V_c = \left(0.0676 \cdot \lambda \sqrt{f'_c} + 4.6 \cdot \frac{A_s}{b \cdot d_s} \cdot \frac{V_u \cdot d_s}{M_u}\right) \cdot b \cdot d_s \le 0.126 \cdot \lambda \sqrt{f'_c} \cdot b \cdot d_s$$

$f'_c = 3.5$	culvert concrete strength, ksi	
$A_{s_TS} = 0.15$	area of reinforcing steel in the design width, in²/ft width	
$h := t_{ts}$	height of concrete design section, in	$h = 12.50$ in
$\lambda = 1.0$	normal wgt. conc. **LRFD [5.4.2.8]**	

Calculate d_s, the distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2}$ $\boxed{d_s = 10.19}$ in

$V_u := Vstr1_{TS}$ $\boxed{V_u = 12.2}$ kips

$M_u = 264.01$ factored moment occurring simultaneously with V_u, kip-in

$b := 12$ design width, in

For reinforced concrete cast-in-place box structures, $\phi_v = 0.85$, **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

$\frac{V_u \cdot d_s}{M_u}$ shall not be taken to be greater than 1.0 $\frac{V_u \cdot d_s}{M_u} = 0.47$ < 1.0 OK

$V_{r1s} := \phi_v \cdot \left[\left(0.0676 \cdot \lambda \sqrt{f'_c} + 4.6 \cdot \frac{A_{s_TS}}{b \cdot d_s} \cdot \frac{V_u \cdot d_s}{M_u}\right) \cdot b \cdot d_s\right]$ $\boxed{V_{r1s} = 14.1}$ kips

but $\leq$ $V_{r2s} := \phi_v \cdot \left(0.126 \cdot \lambda \sqrt{f'_c} \cdot b \cdot d_s\right)$ $\boxed{V_{r2s} = 24.5}$ kips

$V_{rs} := \min(V_{r1s}, V_{r2s})$ $\boxed{V_{rs} = 14.1}$ kips

Check that the provided shear capacity is adequate:

Is $V_u = 12.2$ kip $\leq$ $V_{rs} = 14.1$ kip ? $\boxed{check = "OK"}$

Note: For single-cell box culverts only, V_c for slabs monolithic with walls need not be taken to be less than: **LRFD[5.14.5.3]** $0.0948 \cdot \lambda \sqrt{f'_c} \quad b \cdot d_s$

V_c for slabs simply supported need not be taken to be less than: $0.0791 \cdot \lambda \sqrt{f'_c} \quad b \cdot d_s$

$\lambda = 1.0$ (normal wgt. conc.) **LRFD [5.4.2.8]**

LRFD [5.8] and **LRFD [5.13.3.6]** apply to slabs of box culverts with less than 2.0 ft of fill.

Check that the nominal shear resistance, V_n, of the concrete in the walls is adequate for shear without shear reinforcement per **LRFD [5.8.3.3]**. Calculations shown are for the exterior wall.

$V_n = V_c = 0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v \leq 0.25 \cdot f'_c \cdot b_v \cdot d_v$

$\beta := 2$ **LRFD [5.8.3.4.1]**

$f'_c = 3.5$ culvert concrete strength, ksi

$b_v := 12$ effective width, in

$h := t_{wex}$ height of concrete design section, in $h = 12.00$ in

$\lambda = 1.0$ normal wgt. conc. **LRFD [5.4.2.8]**

Distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement:

$$d_s := h - cover - \frac{Bar_D(Bar_{No})}{2}$$ $d_s = 9.69$ in

The effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; **LRFD [5.8.2.9]**

$$d_{v_i} = d_s - \frac{a}{2}$$

from earlier calculations:

$\beta_1 = 0.85$

$f_s = 60$ ksi

$A_{s_XW} = 0.40$ in^2

The distance between the neutral axis and the compression face:

$$c := \frac{A_{s_XW} \cdot f_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b_v}$$ $\beta_1 = 0.85$ $\alpha_1 = 0.85$ $c = 0.79$ in

$$a := \beta_1 \cdot c$$ $a = 0.67$ in

The effective shear depth:

$$d_{v_i} := \left(d_s - \frac{a}{2}\right)$$ $d_{v_i} = 9.35$

d_v need not be taken to be less than the greater of 0.9 d_s or 0.72h (in.)

$$d_v := \max(d_{v_i}, \max(0.9d_s, 0.72t_{wex}))$$ $0.9 \cdot d_s = 8.72$

$d_v = 9.35$ in $0.72 \cdot t_{wex} = 8.64$

For reinforced concrete cast-in-place box structures, $\phi_v = 0.85$, **LRFD [Table 12.5.5-1]**. Therefore the usable capacity is:

λ = 1.0 (normal wgt. conc.) **LRFD [5.4.2.8]**

$$V_{r1w} := \phi_v \cdot (0.0316 \cdot \beta \cdot \lambda \sqrt{f'_c} \cdot b_v \cdot d_v)$$ $V_{r1w} = 11$ kips

but ≤ $$V_{r2w} := \phi_v \cdot (0.25 \cdot f'_c \cdot b_v \cdot d_v)$$ $V_{r2w} = 83$ kips

$$V_{rw} := \min(V_{r1w}, V_{r2w})$$ $V_{rw} = 11$ kips

$$V_u := Vstr1_{XW}$$ $V_u = 8.0$ kips

Check that the provided shear capacity is adequate:

Is $V_u = 8.0$ kip ≤ $V_{rw} = 11.3$ kip ? check = "OK"

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