12. BURIED STRUCTURES

Buried structures serve a variety of purposes. They are typically used for conveying water. At other times they are used to provide a grade separated crossing for pedestrian and bicycle traffic. A variety of structure and material types are used. The most prevalent types are pipes and box culverts. Buried structures with horizontal dimensions less than 10'-0" are not classified as bridges. Typically these smaller buried structures do not require extensive design and are selected from standard design tables. Buried structures with horizontal dimensions greater than or equal to 10'-0" are considered bridges and require a plan prepared by the Bridge Office. All box culverts require a Bridge Office prepared plan as well. In addition to pipes and box culverts, precast concrete arches, precast three-sided structures, and long-span corrugated steel structures are used as buried structures.

Buried structures carry vertical loads through a combination of internal capacity and soil arching around the structure; this is termed soil-structure interaction. The means by which a buried structure carries vertical load varies significantly between different structure types due to their relative stiffness. Concrete box culverts and rigid pipes are classified as rigid culverts and are assumed to carry the design loads internally with limited requirements or benefit of the soil. Flexible pipe structures (corrugated steel, thermoplastic, etc.) carry loads through soil-structure interaction. For this reason, material and installation requirements of the pipe and soil are well defined including trench or embankment conditions and backfilling and compaction procedures to ensure that the assumed soil-structure capacity is provided and that settlements are not excessive. AASHTO has developed empirical equations for different pipe types to allow for a simplified procedure that closely matches 3D soil-structure interaction models.

[12.6.6]

For special designs a 3D soil-structure model may be utilized in designing and detailing. This will require additional approvals and procedures to ensure the quality of the analysis and construction sequence. Approval of the State Bridge Design Engineer is required for use.

12.1 Geotechnical Properties

Typically, one or more soil borings will be obtained during the preliminary design process. Foundation recommendations based on field data and the hydraulic requirements will also be assembled during the preliminary design process. MnDOT Spec 2451 describes the excavation, foundation preparation, and backfill requirements for bridges and miscellaneous structures.

Maximum and minimum load factors for different load components should be combined to produce the largest load effects. The presence or absence of water in the culvert should also be considered when assembling load combinations.

12.2 Box Culverts

Where pipe solutions are inappropriate, box culverts are the default buried structure type. Their larger openings are often required to provide adequate hydraulic capacity. Box culverts are also frequently used for pedestrian or cattle underpasses.

The reinforcement used in concrete box culverts can be either conventional bar reinforcement or welded wire fabric. Welded wire fabric has a yield strength slightly larger than conventional bar reinforcement (65 ksi versus 60 ksi).

12.2.1 Precast Concrete Box Culverts

Standard designs for precast concrete box culverts are available with spans varying from 6 to 16 feet and rises varying from 4 to 14 feet. Standard precast concrete box culverts are typically fabricated in 6 foot sections; however larger boxes are fabricated in 4 foot sections to reduce section weight. The designs utilize concrete strengths between 5 and 6 ksi and are suitable for fill heights ranging from less than 2 feet to a maximum of 25 feet. Box culverts outside of the standard size ranges must be custom designed. Figure 12.2.1.1 shows typical precast concrete box culvert dimensions.

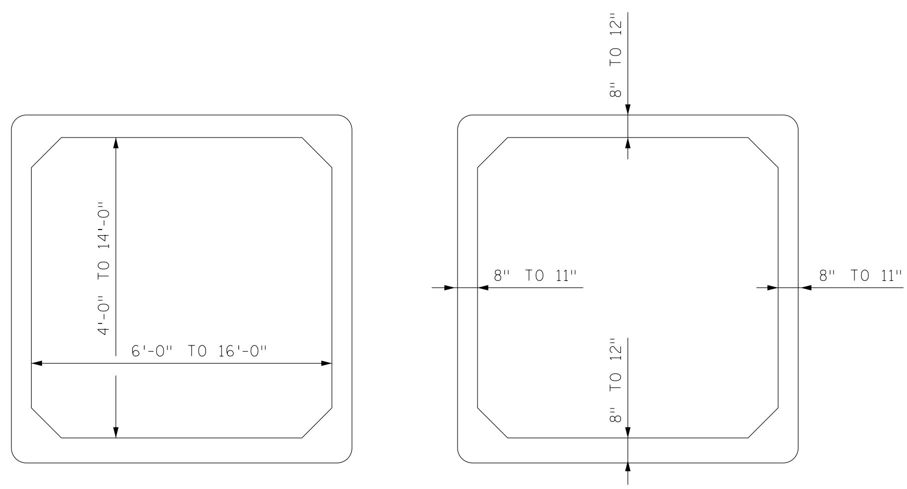


Figure 12.2.1.1
Typical Precast Concrete Box Culvert Dimensions

Each culvert size has three or four classes. Each class has specified wall and slab thicknesses, reinforcement areas, concrete strength, and fill height range to which it applies. Shop drawing submittals for MnDOT approval will not be required when standard culvert sections are used.

The standard design tables are based on welded wire fabric reinforcement with a yield strength of 65 ksi and a concrete clear cover of 2 inches. MnDOT requires that actual clear cover be between 1.5 inches and 2 inches. Design information for welded wire reinforcement can be found at the Wire Reinforcement Institute website:

http://www.wirereinforcementinstitute.org

If conventional rebar is used, the steel area shown on the standard plan sheets needs to be increased 8% to account for the difference in steel yield strength (65 ksi/60 ksi). Also, crack control must be rechecked for the specific bar size and spacing used.

To prevent corrosion at the ends of welded wire fabric, nylon boots are required on the ends of every fourth longitudinal wire at the bottom of the form. Plastic spacers may be utilized in lieu of nylon boots when spaced at a maximum of 48 inches. The maximum allowable size of reinforcement bars is #6 and the maximum allowable size of welded wire is W23. A maximum of two layers of welded wire fabric can be used for primary reinforcement. If two layers are used, the layers may not be nested.

12.2.2 Cast-In-Place Concrete Box Culverts

The first box culverts constructed in Minnesota were made of cast-in-place concrete. The performance of these structures over the years has been very good. Currently, most box culvert installations are precast due to the reduced time required for plan production and construction. Cast-in-place culverts continue to be an allowable option.

12.2.3 Design Guidance for Box Culverts

Material Properties

Concrete Compressive Strength f'_c = 5 ksi or 6 ksi

Steel Yield Strength f_y = 65 ksi (welded wire fabric)

Steel Yield Strength f_y = 60 ksi (rebar)

Reinforced Concrete Unit Weight γ_c = 0.150 kcf

Soil Fill Unit Weight γ_s = 0.120 kcf

Culvert Backfill Angle of Internal Friction ϕ'_f = 30 degrees

Water Unit Weight γ_w = 0.0624 kcf

Geometry

The minimum wall thickness for all box culverts is 8 inches. The minimum slab thickness for culverts with spans of 6 to 8 feet is 8 inches. The minimum top slab thickness is 9 inches, and the minimum bottom slab is 10 inches for all culverts with spans larger than 8 feet. The slab and/or wall thickness is increased when shear requirements dictate or the maximum steel percentages are exceeded. All standard box culverts have haunches that measure 12 inches vertically and horizontally.

Structural Analysis

Various methods can be used to model culverts. Based on past experience, MnDOT prefers a 2-Dimensional (2D) plane frame model be used to analyze culverts. The model is assumed to be externally supported by a pinned support on one bottom corner and roller support on the other bottom corner. The stiffness of the haunch is included in the model. The model is assumed to be in equilibrium so external reactions to loads applied to the structure are assumed to act equal and opposite. This section will assume a 2D plane frame model when referring to modeling, applied loads, and self-weight.

Self Weight (DC)

The self-weight of the top slab must be resisted by the top slab. The benefit of axial compression from the self-weight of the top slab and walls is not included in the analysis. The top slab, wall, and all haunch weights are applied to the bottom slab as an upward reaction from the soil in an equivalent uniform pressure. The bottom slab weight is not applied in the model because its load is assumed to be directly resisted by the soil.

Earth Vertical (EV)

The design fill height is measured from the top surface of the top slab to the top of the roadway or fill. The design fill height is denoted by the abbreviations of H or D_E depending on the equation used. Earth vertical loads refer to soil and pavement loads above the culvert and in adjacent regions slightly outside the span of the culvert based on the soil-structure interaction factor. Culvert walls are assumed to be frictionless, so no vertical component of the earth horizontal resultant force is considered.

[12.11.2.2.1] The soil-structure interaction factor (F_e) is used to adjust the vertical earth load carried by the culvert. It is intended to approximate the arching effects of some of the overburden soil to adjacent regions slightly outside the span of the culvert and account for installation conditions. Culverts placed in trench conditions need to carry less vertical load than those constructed in embankment conditions, because the consolidated material in the adjacent trench walls is typically stiffer than new embankment material. Conservatively assume culverts are installed in embankment conditions.

The factor is:

[12.11.2.2.1-2]

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

where:
H = Depth of backfill (ft)
B_c = Outside width of culvert (2 · sidewall thickness + span) (ft)

Earth Horizontal (EH)

[3.11.5.5] For design and analysis purposes, the equivalent fluid method is used. The maximum for lateral earth pressure on the walls based on at rest pressure is 0.060 kcf.

[3.11.5.2-1] This is computed by taking $k_o \cdot \gamma_S$, where:

$$k_o = 1\text{-}\sin(\phi'_f) = 1 - \sin(30°) = 0.5$$

The resultant earth horizontal force is assumed to act perpendicular to the culvert walls. For maximum force effects, use a strength limit state load factor of 1.35 and a service limit state load factor of 1.0.

[3.11.7] For minimum force effects, the condition of submerged soil pressure acting on the walls is taken as one-half of the earth weight acting on the outside walls, or 0.030 kcf. Use a strength limit state load factor of 0.9 and a service limit state load factor of 1.0.

[3.7.1]

Water (WA)

Designers need to consider two loading conditions: 1) The culvert is full of water, and 2) the culvert is empty.

[3.6.1.2.2]
[3.6.1.2.3]
[3.6.1.3.3]

Design Vehicular Live Load (LL)

The approximate strip method is used for design with the 1 foot wide design strip oriented parallel to the span. The design live loads applied to the top slabs of box culverts include the HL-93 truck and tandem loads for box culverts of any span length. For box culverts with spans of 15 feet or greater lane loads are also applied to the top slabs of box culverts. This practice is consistent with previous versions of the AASHTO Standard Specifications for Highway Bridges.

[3.6.1.2.4]
[12.11.2.1]

Design Lane Loads

The design lane load consists of a load of 0.64 klf uniformly distributed over an area of 1 foot (parallel to the culvert span) by 10 feet perpendicular to the culvert span.

Tire Contact Area

[3.6.1.2.5]

The tire contact area of a wheel consisting of one or two tires is assumed to be a single rectangle, whose width is 20 inches and whose length is 10 inches. The tire pressure is assumed to be uniformly distributed over the rectangular contact area on continuous surfaces.

[3.6.1.1.2]
[C12.11.2.1]

One or Two Lane Loading and Multiple Presence Factor (MPF)

Design box culverts for a single loaded lane with a multiple presence factor of 1.2. MnDOT investigated several live load cases with several box culvert spans at different fill heights and found the live load intensity of 2 lanes with a MPF of 1.0 controlled over a single lane with a multiple presence factor of 1.2 at fill heights of 6.5 feet and greater. However, the maximum live load intensity increase as a percentage of the total load is very small. Based on these findings and the commentary in AASHTO Article C12.11.2.1, multiple loaded lanes are not considered in box culvert design.

[3.6.2.2]

Dynamic Load Allowance (IM)

The dynamic load allowance (IM) for culverts and other buried structures is reduced based on the depth of fill over the culvert. AASHTO LRFD requires that IM be considered for fill heights of up to 8 ft. The equation to calculate the dynamic load allowance is as follows:

$$IM = 33 \cdot (1.0 - 0.125 \cdot D_E) \geq 0\% \text{ (for strength and service limit states)}$$

where:
D_E = the minimum depth of earth cover above the structure (ft)

[3.6.1.3]

Live Load Influence Depth

Include live load for all fill heights.

[12.11.2.1]
[4.6.2.10]

Live Load Distribution With Less Than 2 Feet of Fill

Most box culverts are designed assuming traffic travels parallel to the span. In that scenario, when the depth of fill measured from the top of the roadway or fill to the top of the top slab is less than 2 feet, distribute the design truck or design tandem loads according to AASHTO 4.6.2.10.2 (Case 1: Traffic Travels Parallel to Span). If traffic travels perpendicular to the span, design according to AASHTO 4.6.2.1. Traffic traveling perpendicular to the span is not covered in this manual.

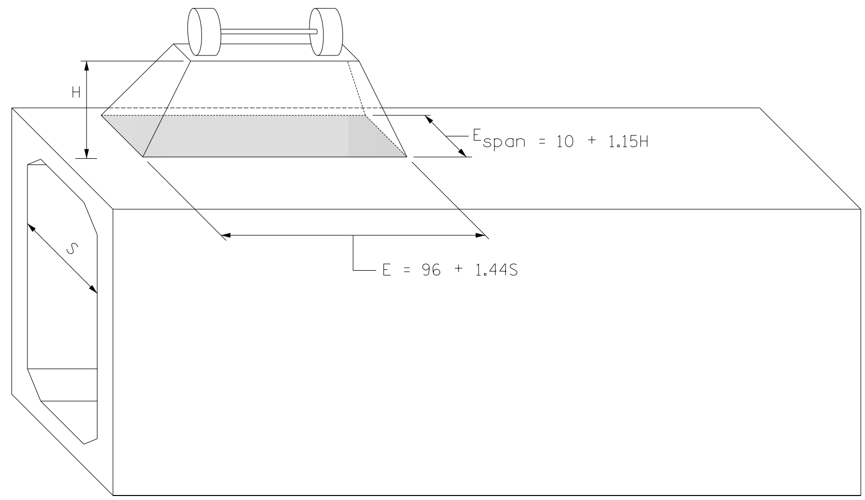


Figure 12.2.3.1
Traffic Traveling Parallel to Span (Less than 2 feet of fill)

The truck axle loads are considered to be uniformly distributed over a rectangular area equal to $E \cdot E_{span}$, as shown in Figure 12.2.3.1, where:

[4.6.2.10.2-1]

$$E = 96 + 1.44 \cdot S$$

[4.6.2.10.2-2]

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

where:

- E = equivalent distribution width perpendicular to span (in)
- S = clear span (ft)
- E_{span} = equivalent distribution length parallel to span (in)
- L_T = length of tire contact area parallel to span (in)
- LLDF = 1.15, factor for distribution of live load through depth of fill
- H = depth of fill from top of culvert to top of pavement (in)

[4.6.2.10.4]

Box culverts with fill heights less than 2 feet require a distribution slab. No structural benefit from the distribution slab is considered during design, other than satisfying AASHTO requirements for shear transfer across joints.

Live Load Distribution With 2 Feet of Fill or Greater

Where the depth of fill exceeds 2 feet, wheel loads may be considered to be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area and increased by either 1.15 times the depth of the fill in select granular backfill, or the depth of the fill in all other cases. Note that the tables in the MnDOT standard plans use 1.15. MnDOT has not adopted the LLDF's as revised in the AASHTO 2013 Interim Revisions, Article 3.6.1.2.6.

The load distribution is shown in Figure 12.2.3.2 for cases where the distributed load from each wheel is separate. Figure 12.2.3.3 shows the areas overlapping. In those cases, the total load will be uniformly distributed over the entire area. In Figure 12.2.3.2, H is measured in inches. In Figure 12.2.3.3, H is measured in feet.

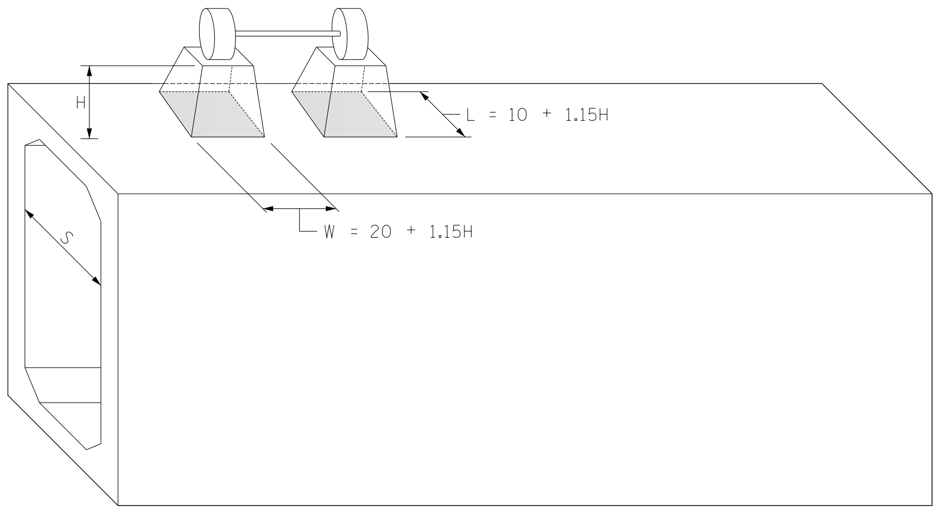


Figure 12.2.3.2
Traffic Traveling Parallel to Span (2 feet of fill or greater)

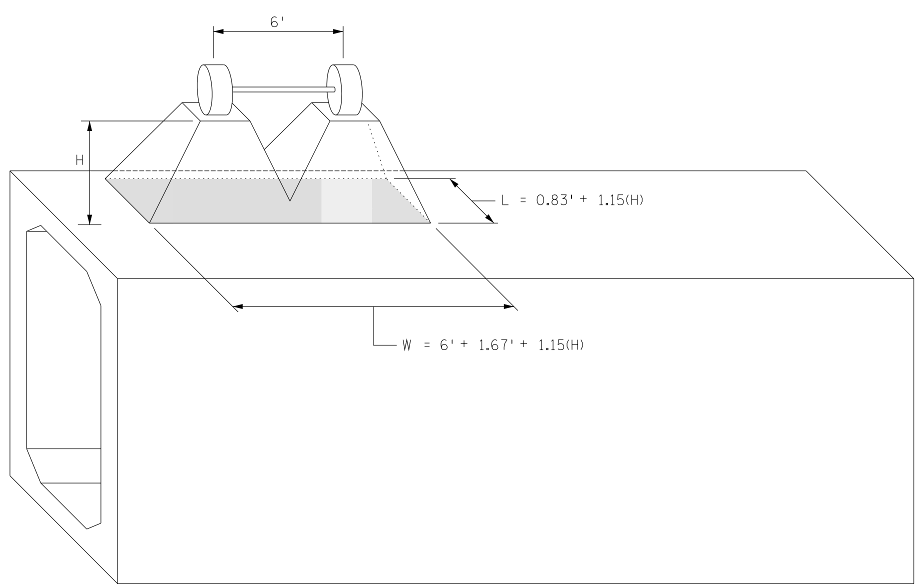


Figure 12.2.3.3
Traffic Traveling Parallel to Span
(2 feet of fill or greater showing load projection overlap)

Live Load Surcharge (Approaching Vehicle Load)

[3.11.6.4] AASHTO requires that a live load surcharge be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall. MnDOT uses a modified form of AASHTO Article 3.11.6.4 to compute the approaching vehicle load. A trapezoidal pressure distribution is assumed with the maximum pressure Δ_{pmax} at the top of the box culvert and the minimum pressure Δ_{pmin} at the bottom of the box culvert. The live load surcharge is only to be applied to one wall of the culvert. For simplification of the analysis, MnDOT applies an equal and opposite reaction to the other wall.

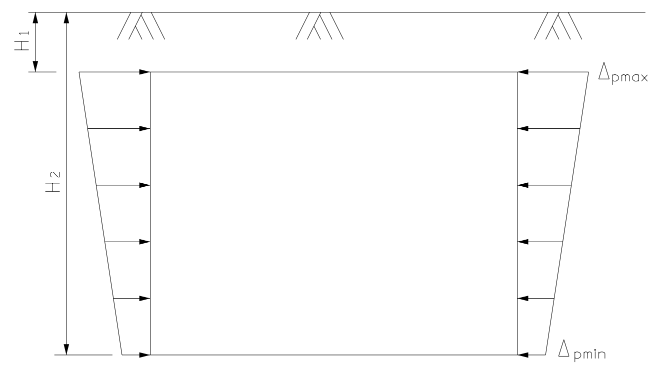


Figure 12.2.3.4
Live Load Surcharge

This methodology more closely approximates a Boussinesq load distribution than assuming a rectangular distribution with an at rest coefficient of lateral earth pressure. Use AASHTO, Equation 3.11.6.4-1 to compute the horizontal earth pressures (Δ_{pmax} and Δ_{pmin}) assuming an active coefficient of lateral earth pressure (k_a = 0.33).

$$\Delta_p = k_a \cdot \gamma_s \cdot h_{eq}$$

where:

Δ_p = horizontal earth pressure due to live load surcharge (ksf)
k_a = coefficient of lateral earth pressure
γ_s = total unit weight of soil
h_{eq} = equivalent height of soil for vehicular load (ft), from AASHTO Table 3.11.6.4-1

For calculating Δ_{pmin}, determine h_{eq} based on the distance from the top of the top slab to the top of the pavement or fill (H_1). For calculating Δ_{pmax} determine h_{eq} based on the distance from the bottom of the bottom slab to the top of the pavement or fill (H_2). Use linear interpolation for intermediate heights.

[3.4.1]
[12.5]

Limit States and Load Combinations

Design for the Strength I and Service I limit states. Evaluation of extreme event and fatigue limit states is unnecessary because culvert design is not governed by these limit states.

Load Combinations

The following load combinations were developed by varying the Strength I and Service I load factors in order to maximize moments and shears for the various box culvert members. At a minimum, consider the following load cases:

Strength Limit States:

Ia. Maximum vertical load and maximum horizontal load:

$1.25DC + (1.30)(1.05)EV + 1.75(LL+IM) + (1.35)(1.05)EH_{max} + 1.75LS$

Ib. Maximum vertical load and minimum horizontal load:

$1.25DC + (1.30)(1.05)EV + 1.75(LL+IM) + 1.00WA + (0.9/1.05)EH_{min}$

Ic. Minimum vertical load and maximum horizontal load:

$0.90DC + (0.90/1.05)EV + (1.35)(1.05)EH_{max} + 1.75LS$

Service Limit States:

Ia. Maximum vertical load and maximum horizontal load:

$1.00DC + 1.00EV + 1.00(LL+IM) + 1.00EH_{max} + 1.00LS$

Ib. Maximum vertical load and minimum horizontal load:

$1.00DC + 1.00EV + 1.0(LL+IM) + 1.00WA + 1.00EH_{min}$

Ic. Minimum vertical load and maximum horizontal load:

$1.00DC + 1.00EV + 1.00EH_{max} + 1.00LS$

[1.3.4]
[12.5.4]

Use a value of 1.0 for all load modifiers (η) for box culvert design, except for earth EV and EH loads, EV & EH where $\eta_R = 1.05$ is used due to the lack of redundancy.

[C12.11.3]

Axial Thrust

Do not consider the benefit of axial thrust in the design of box culverts for the strength limit state. It may be used in the service limit state crack control check.

[12.11.4.2]

Flexure

Flexural reinforcement is designed for positive and negative moment at all design locations (see Figure 12.2.3.5). The flexural resistance factor, ϕ_f, is 1.0 for precast concrete. Reinforcing areas, shown in Figure 12.2.3.6, are selected based on the following:

[12.5.5]

- As1 is based on the negative moment requirements in the side wall and in the outside face of the top slab and bottom slab
- As2 is based on the positive moment in the top slab.
- As3 is based on the positive moment in the bottom slab.
- As4 is based on the positive moment in the side
- As7 is based on the negative moment at Section F1
- As8 is based on the negative moment at Section F9.

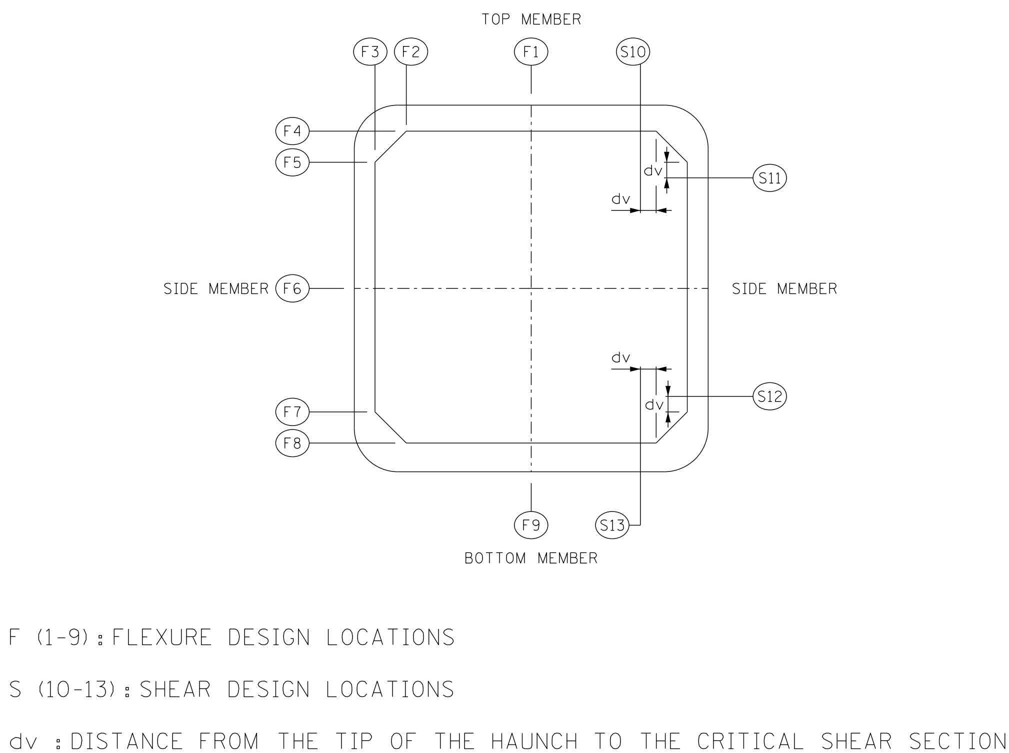


Figure 12.2.3.5
Box Culvert Flexure and Shear Design Locations

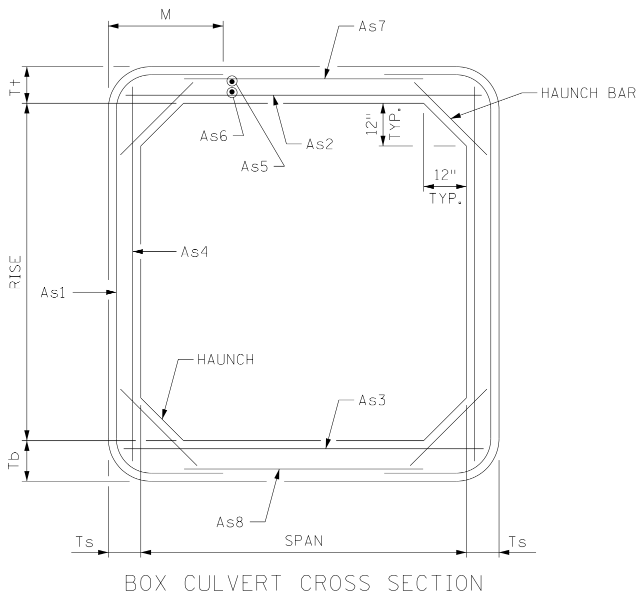


Figure 12.2.3.6
Box Culvert Reinforcement

[5.7.3.4]
[C12.11.3]

Crack Control

Restrict the stress in the reinforcement to 60% of the yield strength. For welded wire fabric, assume a maximum spacing of 4 inches. Check crack control using the Class II exposure condition (γ_e=0.75). Compute the tensile stress in the steel reinforcement at the service limit state using the benefits of axial thrust as shown in AASHTO equation C12.11.3-1. Fabricators have discretion in choosing wire spacing, but the spacing cannot exceed 4 inches.

[5.5.4.2]
[5.7.2.1]
[12.5.5]

Maximum Reinforcement

The standards and typical designs use a resistance factor of 1.0 with a section that is tension-controlled. Special designs may require a reduced resistance factor.

Reinforcement is limited to $0.6\rho_b$. This ensures that the reinforcement is not too congested, allowing for easier and more efficient fabrication.

[5.10.8]
[9.7.3.2]
[12.11.4.3.2]

Minimum Reinforcement

MnDOT requires reinforcement in all slabs and walls in both directions on both faces regardless of fill height. In top and bottom slabs for all fill heights, use 0.002 x b x h as the minimum primary reinforcement denoted as As7 and As8. Distribution reinforcement is not needed, since a distribution slab is required for all boxes with less than 2.0 feet of fill.

A minimum amount of reinforcement is required to be placed in each face in each direction in the top and bottom slabs and walls for all box sections regardless of cover. The MnDOT minimum value for this reinforcement is 0.06 in^2/ft, which is greater than the AASHTO minimum.

[5.13.3.6]
[C5.13.3.6.1]

Shear Critical Section

Because of the 1:1 slope of the haunch, the critical section for shear may be taken at d_v past the tip of the haunch.

[5.8]
[12.5.5]

Shear in Slabs of Box Culverts with Less Than 2 Feet of Fill and Walls of Box Culverts at All Fill Heights

For top slabs of boxes with less than 2 feet of fill and walls of boxes at all fill heights calculate the shear resistance using the greater of that computed using the "Simplified Procedure for Nonprestressed Sections" given in AASHTO LRFD Article 5.8.3.4.1 and the "General Procedure" given in AASHTO Article 5.8.3.4.2.

[5.14.5.3]
[12.5.5]

Shear in Slabs of Box Culverts with 2 Feet of Fill or Greater

For top and bottom slabs of boxes with 2 feet of fill or greater calculate the shear resistance using the shear provisions specific to slabs of box culverts.

For slabs of boxes with thicknesses greater than 12 inches, contact the MnDOT Bridge Standards Unit for shear provisions.

[C12.5.3]

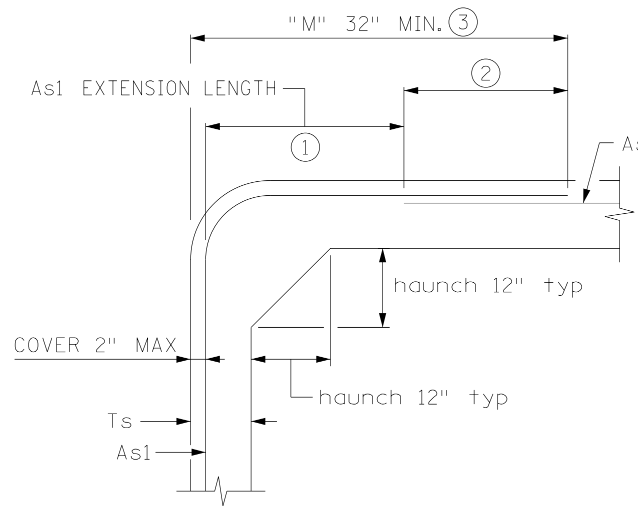
Fatigue

Fatigue is not considered in the design of buried structures.

[5.11.2.5.2]

Development Lengths

To ensure reinforcement continuity, proper development length is required. See Figure 12.2.3.7 for extension of As1 into the top or bottom slab. For constructability, make the bent legs on As1 the same length on the top and bottom. This length is typically calculated based on the bottom slab. MnDOT uses AASHTO LRFD equation 5.11.2.5.2-1 to calculate development lengths in box culverts. In some cases, As1 is needed to resist shear. In these cases, As1 should be developed past d_v from the tip of the haunch.



1 As1 EXTENSION LENGTH - DISTANCE TO THE POINT WHERE THE NEGATIVE MOMENT CAN BE RESISTED BY As7 OR As8.

2 GREATER OF DEVELOPMENT LENGTH OF As1 OR THE LAP SPLICE LENGTH OF As7.

3 "M" = 2" (COVER) + 1 + 2
MINIMUM "M" DISTANCE = Ts + HAUNCH (12") + MAX (d_e, d_v) + 6"

Figure 12.2.3.7
Box Culvert Reinforcement Development Length

Unless the specific size of welded wire fabric to be used by the fabricator is known, use the largest size that can provide the area required in one mat. If two mats are required, use a W23 for the development length calculation.

Aprons

Precast apron segments are provided for each size of barrel. There are four different details relating the culvert's skew to the roadway above.

Culvert Skew Range	Apron Skew
$0°$ to $7^1/_2°$	$0°$
$7^1/_2°$ to $22^1/_2°$	$15°$
$22^1/_2°$ to $37^1/_2°$	$30°$
$37^1/_2°$ to $45°$	$45°$*

* Boxes with spans of 16 feet or greater have a maximum apron skew angle of 30°. All other boxes have a maximum apron skew of 45°.

Based on past practice, lateral soil pressure of 0.060 ksf is used for the apron design except for the 45° skew aprons which are designed with a 0.075 ksf pressure on the longer length wall. MnDOT also requires additional extra strong ties between the barrel and first end section, and between the first and second end sections on the high fill side only for 45° skew aprons over 6 feet high. Conventional ties can be used on aprons between multiple boxes and on the low fill side of the apron. Additional ties are required to resist unequal pressures on opposite sides of the skewed apron. See the culvert standards Figure 5-395.110(A) for more information.

Software

Various commercially available off-the-shelf software programs have been developed to analyze and design precast box culverts. These software programs can be used to assist in the design of precast box culverts provided that the parameters, modeling methods, AASHTO LRFD code provisions and MnDOT code modifications specified in this manual are compatible with the software. In some instances, it may be easier to develop custom software or spreadsheets depending on the differences between the available software and the AASHTO and MnDOT practices detailed in this manual. Any piece of software is subject to the Design QC/QA Process outlined in Section 4.1.

District Box Culvert Request

Figure 12.2.3.8 shows a typical box culvert request memo from a District.

Design Example

Refer to Section 12.5 for a 10 ft x 10 ft precast concrete box culvert design example.

Minnesota Department of Transportation

MEMO

Transportation District #7
2151 Bassett Drive
Mankato MN 56001-6888

Office Tel: 507-304-6100
Fax No: 507-304-6119

Date: 06-27-2012

To: Kevin L. Western, Bridge Design Engineer
Mn/DOT Bridge Office • MS 610
3485 Hadley Avenue North • Oakdale, MN 55128-3307

From: Scott Morgan, District Hydraulics Engineer

Subject: S.P. 7201-112

Please prepare a design for concrete box culvert 8045. Tabulated below and attached is the information required to prepare plans. The letting date for this project is 03/22/2013. Please submit completed plans to this office before 09/15/2012. If you determine you are unable to meet the above deadline, please contact me.

State Project No. **7201-112**		Func. 2 Work Authority: **T 71559** (charge ID)	
Location Description [1]: **0.3 MI S OF GREEN ISLE**			
Reference: **14.091**		Station: **741+71.47**	
Section: **13**	Range: **R 27 W**	Township: **T 114 N**	Twp. Name: **GREEN ISLE**
Stream Crossing: **Sibley County Ditch No. 29**		County: **SIBLEY**	
Structure Type: **Box Culvert**			
New Structure Number: **72X04**		Existing Structure Number: **8045**	
Number of Barrels: **1**	Opening Width: **16 ft.**	Opening Height: **8 ft.**	Space btwn. Precast Barrels: **NA**
Depth of Cover [2]: **10.75 ft.**	Skew Angle: **22.5°**	End Sections: **Pre-Cast**	
Inlet Elevation (new structure): 980.13		Outlet Elevation (new structure): **979.63**	
Inlet Elevation (inplace structure): **982.20**		Outlet Elevation (inplace structure): **980.82**	
Extension Distances from End of Inplace: **NA**			
Plans Requested: **Precast Only**			

(1) T.H. 5 over Ditch No. 29, 0.26 Mi. SW of Jct. T.H. 5 & T.H. 25 *(modify as necessary)*
(2) Minimum fill over culvert rounded up to the next $^1/_4$ ft.

Attachments: Sketch of Cross Section
Sketch of Plan View

Cc: Angel Staples, Bridge Office Design Unit Leader
Dustin Thomas, Bridge Office South Region Construction

Figure 12.2.3.8
District Box Culvert Request Memo Example

12.3 Arched & Three-Sided Structures

Arched or three-sided precast concrete structures offer an alternative to single or multiple barrel box culvert structures. These structures can be constructed rapidly, thus minimizing road closure time, and they allow for a natural stream bottom. Potential applications include pedestrian underpasses and stream crossings where the waterway opening requirements are on the low end of a conventional bridge but are at the high end of box culvert capabilities. As with all structure type selections, the designer should consider speed of construction and economics, including cost comparisons to cast-in-place structures or multiple barrel precast concrete box culverts.

12.3.1 Three-Sided Precast Concrete Structures

There are two types of three-sided bridge structures: arch top and flat top. The design of such structures shall be in conformance with the AASHTO LRFD Bridge Design Specifications and the current Three-Sided Structures Technical Memorandum. The design methods vary between suppliers. The technical memorandum contains guidance for design, submittal requirements, material specifications, construction quality assurance, and the MnDOT Bridge Office review and approval process for use of three-sided structures.

In general, precast three-sided structures may be used where:

A. Design span is less than or equal to 42 feet. Larger spans may be considered on a case-by-case basis, but only with prior approval of the Bridge Design Engineer. Span is measured from inside face of sidewalls along the longitudinal axis of the unit;

B. Rise is less than or equal to 13 feet. Rise is measured from top of footing/pedestal wall to bottom of top slab;

C. Fill height is less than or equal to 10 feet but is greater than or equal to 3 feet. Fill heights larger than 10 feet may be considered on a case-by-case basis, but only with prior approval of the Bridge Design Engineer;

D. Skew is less than 30°;

E. No foundation limitations exist such as unusually weak soil;

F. No site access limitations exist for transporting and erecting the three-sided structures;

G. Clogging from debris or sediment precludes the use of multiple barrel structures.

Since these are vendor supplied structures, their final structural design occurs after the award of the construction contract. The time required for final design and the subsequent review/approval periods impact the total contract length.

This technical memorandum can be viewed at the following web site:
http://techmemos.dot.state.mn.us/techmemo.aspx

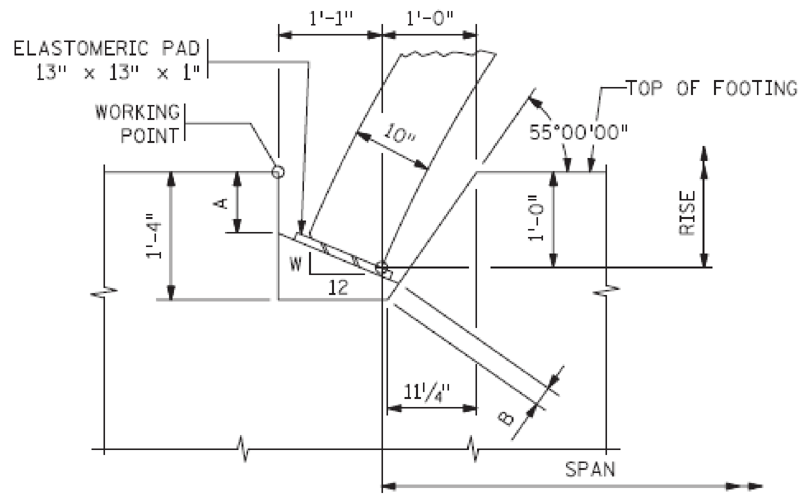
The list of pre-qualified suppliers for three-sided bridge structures is available at the Bridge Office website:
http://www.dot.state.mn.us/products/bridge/3sidedprecastconc.html

12.3.2 Precast Concrete Arch Structures

Sample plan sheets for the design of buried precast concrete arch structures are available from the MnDOT Bridge Standards Unit.

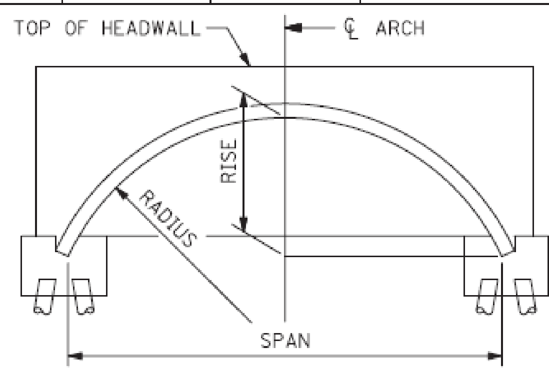
Figure 12.3.2.1 contains standard geometric information for spans between 24'-0" and 43'-11".

The minimum fill height is 1'-6" at the low edge of pavement at the crown of the arch.



SECTION THROUGH FOOTING NOTCH

ACTUAL SPAN	W	A	B	WORKING POINT TO WORKING POINT
24'-0"	5.00	7⅝"	2½"	26'-2"
25'-3⅝"	2.85	10"	2⅞"	27'-5⅝"
32'-3"	5.95	6⅝"	2¼"	34'-5"
34'-4⅝"	3.72	9"	2¾"	36'-6⅝"
40'-2½"	8.36	4⅛"	1⅞"	42'-4½"
43'-11⅜"	5.92	6¾"	2¼"	46'-1⅜"



ELEVATION
(PILE FOOTING SHOWN)

NOMINAL SPAN	ACTUAL SPAN	RISE	RADIUS	AREA SQ. FT. ①	NET AREA SQ. FT. ②
24'-0"	24'-0"	8'-0"	13'-0"	139	117
25'-4"	25'-3⅝"	10'-0"	13'-0"	188	165
32'-3"	32'-3"	10'-0"	18'-0"	232	202
34'-5"	34'-4⅝"	12'-8"	18'-0"	320	288
40'-3"	40'-2½"	10'-6"	24'-6"	296	258
43'-11"	43'-11⅜"	13'-8"	24'-6"	430	388

① AREA UNDER ARCH ABOVE A LEVEL LINE AT LOWEST POINT OF ARCH.

② AREA UNDER ARCH ABOVE A LEVEL LINE AT TOP OF FOOTING.

Figure 12.3.2.1
Precast Concrete Arch Structure Geometric Data

12.3.3 Scour Protection Guidelines

The following guidelines are provided for the design and installation of scour protection for arch or 3-sided bridge footings.

There are several options available for protection of the footings against scour. These options include rock riprap, concrete bottom, piling supported footings, and spread footings keyed into bedrock. The preferred option will depend on a number of factors including:

- Foundation design
- Stream bed material
- Scour potential
- Velocity of flow
- Environmental considerations such as fish migration
- Economics

The foundation design will depend on the type and allowable bearing capacity of the soil, the height of fill, and the proximity of bedrock. Scour should be considered during foundation design. Sub-cut unstable material below spread footings and replace it with granular backfill or a lean concrete. Due to the difficulty of achieving adequate compaction in wet conditions, the maximum depth of sub cutting for this purpose is 2 feet. A pile footing should be used if the depth of unstable material below a footing is greater than 2 feet.

Four standard designs for scour protection for concrete arch structures have been assembled. The appropriate design is selected based on the average velocity through the structure for the 100-year flood. A more recurrent flood event should be used if it results in a faster average velocity through the structure.

Design 1 Scour Protection

The average velocity for the 100 year flood must be no greater than three feet per second, and for the 500-year flood no greater than five feet per second. Use of 12 inch Class II riprap with 6 inch granular filter or geotextile filter is required.

- <u>Option 1 (Figure 12.3.3.1, left side)</u>
  The riprap may be placed on a slope of 1:2.5 maximum. Cover to the bottom of footing shall be 6 feet minimum measured perpendicular to the slope. The riprap shall be toed in vertically 2 feet minimum. The bottom of footing shall be at or below the channel bottom.

- Option 2 (Figure 12.3.3.1, right side)
  The riprap may be placed horizontally on the channel bottom. Cover to the bottom of footing shall be 4'-6" minimum. The riprap shall extend a minimum of 10 feet from edge of structure and be toed in vertically a minimum of 2 feet.

Design 2A Scour Protection

The average velocity for the 100-year flood must be less than 5.5 feet per second, and for the 500-year flood less than 6.5 feet per second. Use of 24" Class IV riprap with 12" granular filter or geotextile filter is required.

- Option 1 (Figure 12.3.3.2, upper left side)
  The riprap may be placed on a slope of 1:2.5 maximum. It shall extend across the entire width of the structure. Cover to the bottom of the footing shall be 6 feet minimum measured perpendicular to the slope. The bottom of footing shall be 2 feet minimum below the channel bottom.

- Option 2 (Figure 12.3.3.2, upper right side)
  The riprap may be placed horizontally on the channel bottom. Cover to the bottom of footing shall be 6 feet minimum. The riprap shall extend a minimum of 10 feet from edge of footing and be toed in vertically a minimum of 2 feet.

Design 2B Scour Protection (Figure 12.3.3.2, lower right side)

The average velocity for the 100-year flood must be no greater than 5.5 feet per second. The average velocity for the 500-year flood must be no greater than 6.5 feet per second. The area for calculating the average velocity of the 100-year flood shall be Area, "A" which is bounded by the channel bottom and the water surface. The area for calculating the 500-year flood shall be Area "A" plus Area "B", where Area "B" is bounded by the channel bottom and the 500-year flood scoured channel bottom. The toe of riprap shall extend 2 feet min. beyond the bottom of Area "B". This toe shall have a minimum thickness of 2 feet. Cover to the bottom of footing shall be 6 feet minimum measured perpendicular to the slope. The bottom of footing shall also be at or below the bottom of Area "B".

Design 3 Scour Protection

- Option 1 (Figure 12.3.3.3, left side)
  Articulated concrete with geotextile backing may be placed on a maximum slope of 1:2.5 with a minimum cover of 4'-6" to bottom of footing measured perpendicular to the articulated concrete. The average velocity for the 100-year flood must be no greater

than 7.5 feet per second. For higher velocities, contact the Bridge Office.

- Option 2 (Figure 12.3.3.3, right side)
  A reinforced concrete floor placed horizontally only with 4'-6" minimum cover to bottom of footing may be used. The same velocity constraints as for Option 1 apply.

Design 4 Scour Protection

- Option 1 (Figure 12.3.3.4, left side)
  If footings are on piling, riprap shall be placed on a slope of 1:2.5 maximum. The bottom of footing shall be at or below the channel bottom.

- Option 2 (Figure 12.3.3.4, right side)
  If footings are on hard bedrock, they shall be keyed in a minimum of 1 foot.

These guidelines are anticipated to cover most cases, however, there may be factors such as high natural channel velocity, dense hardpan channel bottom, historical evidence of no scour on the in place structure or other pertinent data that can be considered when designing scour protection for the concrete arch structures. Exceptions to these guidelines must be approved by the Hydraulics Engineer.

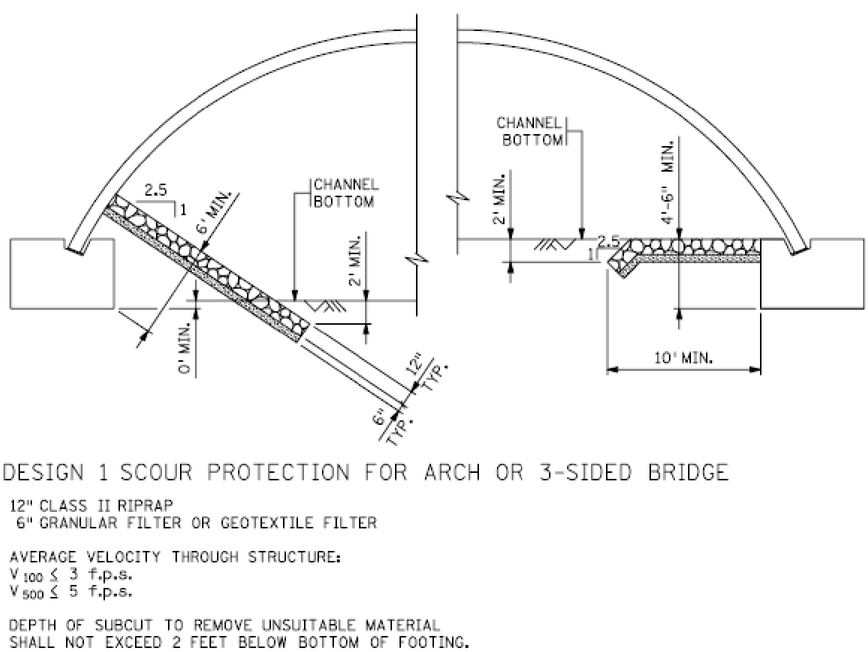
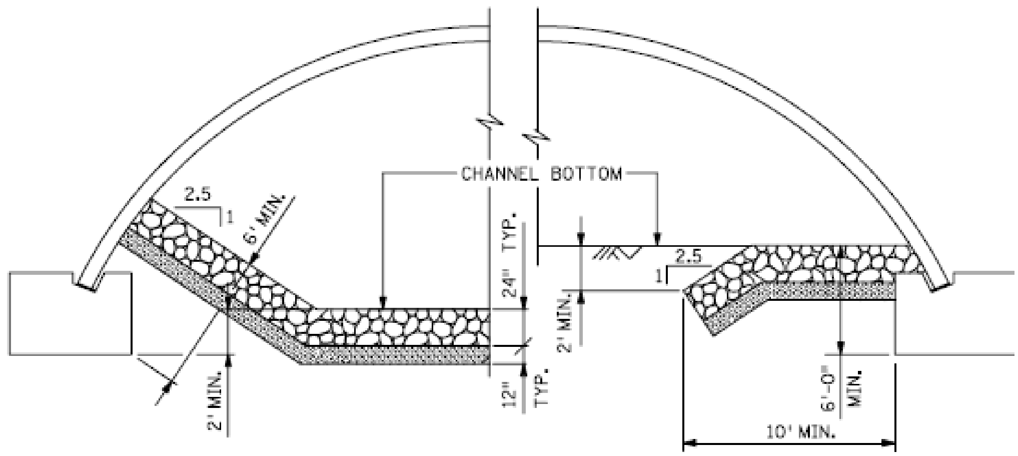


Figure 12.3.3.1
Design 1 Scour Protection for Arch or 3-Sided Bridge

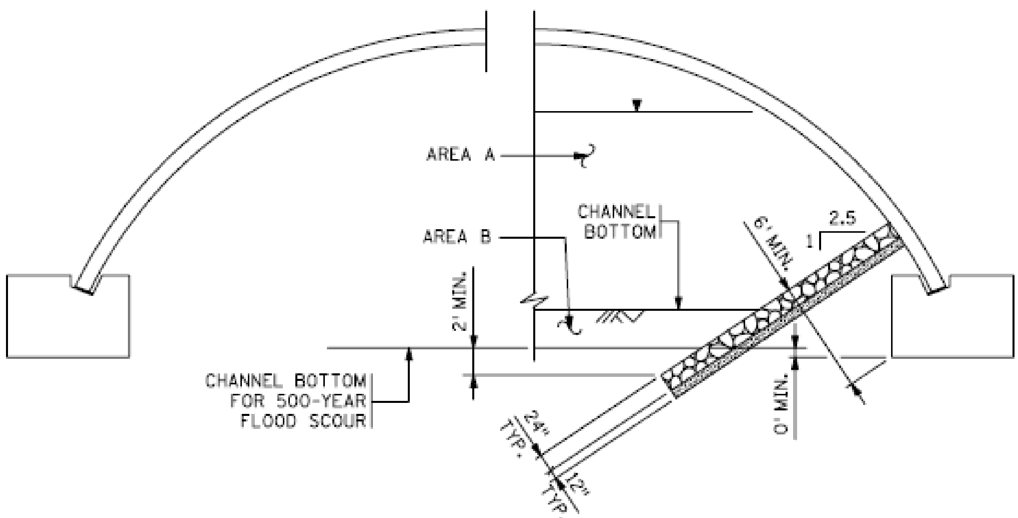


DESIGN 2A SCOUR PROTECTION FOR ARCH OR 3-SIDED BRIDGE

24" CLASS IV RIPRAP
12" GRANULAR FILTER

AVERAGE VELOCITY THROUGH STRUCTURE:
$V_{100} \leq$ 5.5 f.p.s.
$V_{500} \leq$ 6.5 f.p.s.

DEPTH OF SUBCUT TO REMOVE UNSUITABLE MATERIAL
SHALL NOT EXCEED 2 FEET BELOW BOTTOM OF FOOTING.



DESIGN 2B SCOUR PROTECTION FOR ARCH OR 3-SIDED BRIDGE

24" CLASS IV RIPRAP
12" GRANULAR FILTER

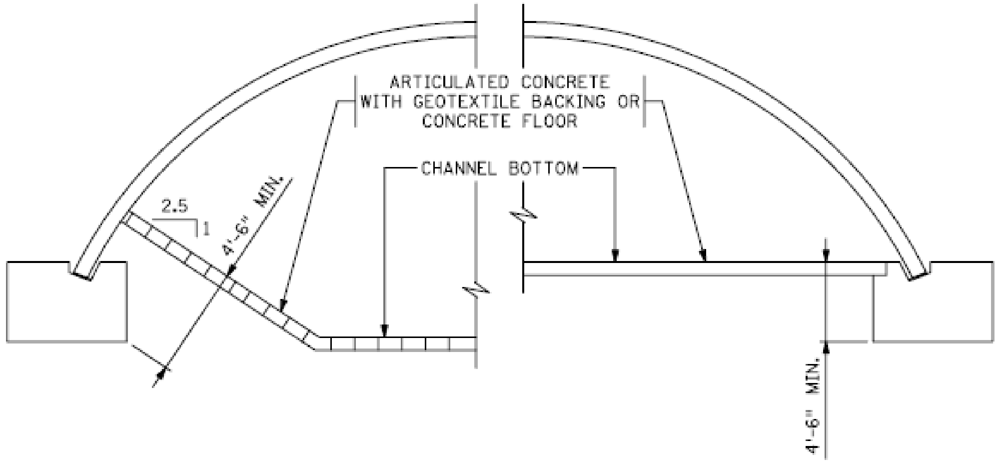
AVERAGE VELOCITY THROUGH STRUCTURE:
$V_{100} \leq$ 5.5 f.p.s.
$V_{500} \leq$ 6.5 f.p.s.

DEPTH OF SUBCUT TO REMOVE UNSUITABLE MATERIAL
SHALL NOT EXCEED 2 FEET BELOW BOTTOM OF FOOTING.

AREA A = AREA BOUNDED BY CHANNEL BOTTOM AND WATER SURFACE.

AREA B = AREA BOUNDED BY CHANNEL BOTTOM AND 500-YEAR FLOOD SCOURED CHANNEL BOTTOM.

Figure 12.3.3.2
Design 2A and 2B Scour Protection for Arch or 3-Sided Bridge

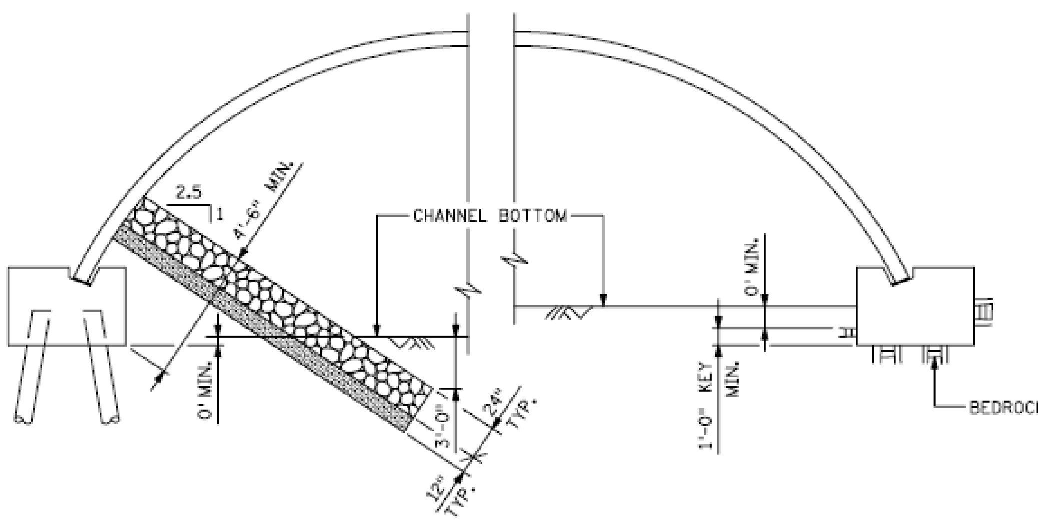


DESIGN 3 SCOUR PROTECTION FOR ARCH OR 3-SIDED BRIDGE

A. CONCRETE FLOOR OR
B. ARTICULATED CONCRETE WITH GEOTEXTILE BACKING

AVERAGE VELOCITY THROUGH STRUCTURE:
$V_{100} \leq 7.5$ f.p.s.

Figure 12.3.3.3
Design 3 Scour Protection for Arch or 3-Sided Bridge



DESIGN 4 SCOUR PROTECTION FOR ARCH OR 3-SIDED BRIDGE

A. FOOTINGS ON PILING OR
B. SPREAD FOOTING KEYED INTO HARD BEDROCK

AVERAGE VELOCITY THROUGH STRUCTURE:
$V_{100} > 7.5$ f.p.s.

NOTE: IT IS THE CONTRACTOR'S RESPONSIBILITY TO PROTECT THE FOOTING FOUNDATION FROM SCOUR UNTIL THE RIPRAP HAS BEEN PLACED.

Figure 12.3.3.4
Design 4 Scour Protection for Arch or 3-Sided Bridge

12.4 Use of Long-Span Corrugated Steel Structures

The design requirements for long span corrugated steel structures are currently being updated. At a minimum, corrugated steel structures shall meet the criteria below. Contact the State Bridge Design Engineer for approval before utilizing on a project.

1. These structures are not permitted for use as vehicle underpasses.
2. All designs utilizing Federal, State, or State Aid funds will be reviewed by MnDOT. The review will verify compliance with AASHTO Specifications, MnDOT Specifications, MnDOT detail sheets, and MnDOT design guidelines.
3. Standard Plan Sheets developed by MnDOT indicate the span, rise, invert elevation, profile grade over, and hydraulic characteristics of the structure.
4. These structures are considered arch bridges that depend structurally on the interaction of the structural plate liner and good quality soil, which is carefully compacted. Balanced placement of backfill and close field supervision of backfilling operations is required.
5. Detailed plans that include structural computations and special provisions shall be certified by a qualified professional engineer registered in the State of Minnesota.
6. MnDOT Projects
   a. When MnDOT proposes the use of these structures, the Bridge Office will determine the shape, invert elevation, roadway and hydraulic data, and complete a design plan with special provisions to be forwarded to the District Engineer.
   b. A copy of the design plan and special provisions shall be forwarded to the Bridge Standards Engineer.
7. State Aid Projects
   a. For State Aid approval, the county or municipal engineer shall submit 3 copies of an engineering report on the structure's hydraulic characteristics, special provisions, a design detail plan, and funding request forms to the MnDOT State Aid Office.
   b. Final funding approval by the State Aid Office to include approval of the design plan and hydraulic characteristics by the Bridge Office.
8. Scour protection for long span corrugated steel structures shall be the same as those for precast concrete arch structures (see Section 12.3.3).
9. The Foundation Engineer will determine the suitability of the foundation material, and provide recommendations regarding required sub-cuts.

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12.5 10'x10' Precast Concrete Box Culvert Design Example

This example illustrates the design of a single barrel precast concrete box culvert. After determining the load components and design load combinations, the design of the flexural reinforcement is presented. The example concludes with a shear check and an axial load capacity check.

Inside dimensions of the box culvert (Span x Rise) are 10'-0" by 10'-0" with 12" haunches (Th). The fill height (H) above the culvert is 6'-0". A typical section of the culvert is shown in Figure 12.5.1. Material and design parameters are given in Table 12.5.1.

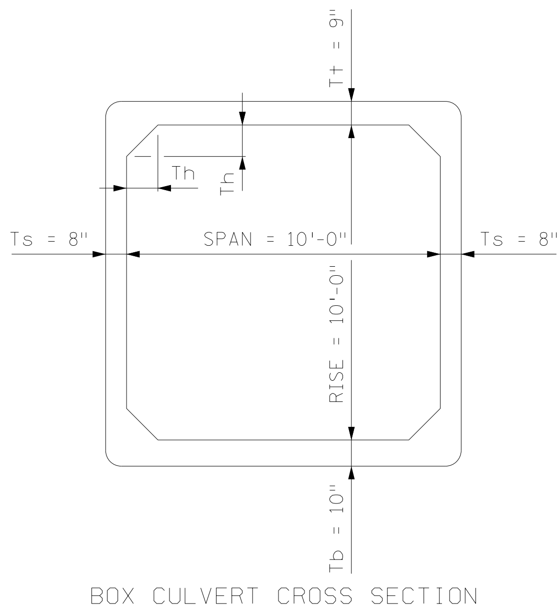


Figure 12.5.1

Table 12.5.1
Material and Design Parameters

Unit Weights	Reinforced Concrete, γ_c	0.150 kcf
	Water, γ_w	0.0624 kcf
	Soil, γ_s	0.120 kcf
Concrete	Compressive Strength, f'_c	5.0 ksi
	Top Slab Thickness, Tt	0.75 ft
	Bottom Slab Thickness, Tb	0.83 ft
	Wall Thickness, Ts	0.67 ft
	Haunch Thickness, Th	12 in
	Reinforcement Clear Cover	2 in
Steel Reinforcement	Modulus of Elasticity, E_s	29,000 ksi
	Yield Strength, f_y	65 ksi
	Maximum Wire Size	W23
	Maximum Wire Spacing	4 in transverse 8 in longitudinal

The approximate strip method is used for the design with the 1'-0" wide design strip oriented parallel to the direction of traffic.

A 2-Dimensional (2D) plane frame model is used to analyze the box culvert. Beam elements in the 2D model are assumed to be centered in the concrete members. The model is assumed to be externally supported by a pinned support on one end and a roller support on the other end. In addition, the model is always assumed to be in equilibrium so external reactions to loads applied to the structure were assumed to act equal and opposite. A "w" dimension of 1 ft is added to the calculations to convert the units to klf for consistency with national conventions.

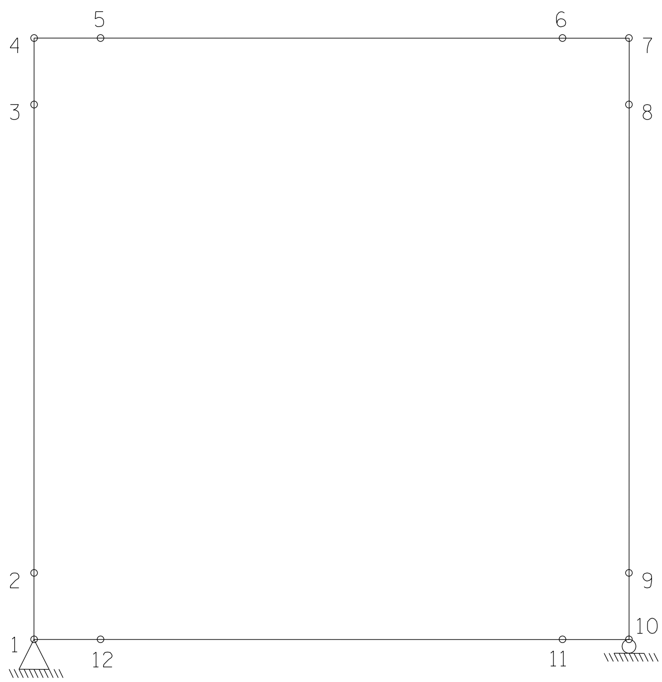


Figure 12.5.2
2D Plane Frame Model

A. Dead Load

The self-weight of the culvert top slab is:

$$DC_{top} = Tt \cdot w \cdot \gamma_c = 0.75 \cdot 1 \cdot 0.150 = 0.113 \text{ klf}$$

The total self-weight of the culvert top slab is:

$$DC_{top} = Tt \cdot w \cdot \gamma_c \cdot (Span + Ts) = 0.75 \cdot 1 \cdot 0.150 \cdot (10+0.67) = 1.20 \text{ kips}$$

The self-weight of one culvert side wall is:

$$DC_{side} = Ts \cdot w \cdot \gamma_c \cdot \left(Rise + \frac{Tt}{2} + \frac{Tb}{2}\right) = 0.67 \cdot 1 \cdot 0.150 \cdot \left(10 + \frac{0.75}{2} + \frac{0.83}{2}\right)$$
$$= 1.08 \text{ kips}$$

The self-weight of one haunch is:

$$DC_{haunch} = 0.5 \cdot Th \cdot w \cdot Th \cdot \gamma_c = 0.5 \cdot 1 \cdot 1 \cdot 1 \cdot 0.150 = 0.075 \text{ kips}$$

The top slab weight, wall weights, and all four haunch weights are applied to the bottom slab as an upward reaction from the soil assuming an equivalent uniform pressure. The bottom slab weight is not applied in the model because its load is assumed to be directly resisted by the soil.

$$DC_{bottom} = (DC_{top} + 4DC_{haunch} + 2DC_{side}) \cdot \left(\frac{1}{Span + Ts}\right)$$

$$= (1.20 + 4 \cdot 0.075 + 2 \cdot 1.08) \cdot \left(\frac{1}{10 + 0.67}\right) = 0.343 \text{ klf}$$

B. Earth Pressure Loads [12.11.2]

The weight of fill on top of the culvert produces vertical earth pressure (EV). The fill height is measured from the top surface of the top slab to the top of the pavement or fill. Per Table 12.5.1, the unit weight of the fill is 0.120 kcf.

The interaction factor for embankment conditions is dependent on the height of fill (H) and the outside width of the culvert (B_c):

[12.11.2.2.1-2]

$$F_e = 1 + 0.20 \cdot \left(\frac{H}{B_c}\right) = 1 + 0.20 \cdot \left(\frac{6}{2 \cdot 0.67 + 10}\right) = 1.11$$

The design vertical earth pressure at the top of the culvert is:

[12.11.2.2.1-1]

$$EV = F_e \cdot \gamma_s \cdot H \cdot w = 1.11 \cdot 0.120 \cdot 6 \cdot 1 = 0.799 \text{ klf}$$

[3.11.5]
[3.11.7]

The lateral earth pressure (EH) on the culvert is found using the equivalent fluid method. For at-rest conditions, a maximum equivalent fluid unit weight of 0.060 kcf and a minimum equivalent fluid unit weight of 0.030 kcf are used.

At the top of the culvert, the lateral earth pressure is:

$$EH_{max} = \gamma_{max} \cdot H \cdot w = 0.060 \cdot 6 \cdot 1 = 0.360 \text{ klf}$$

$$EH_{min} = \gamma_{min} \cdot H \cdot w = 0.030 \cdot 6 \cdot 1 = 0.180 \text{ klf}$$

At the bottom of the culvert, the lateral earth pressure is:

$$EH_{max} = \gamma_{max} \cdot (H + Tt + Rise + Tb) \cdot w$$

$$= 0.060 \cdot (6 + 0.75 + 10 + 0.83) \cdot 1 = 1.05 \text{ klf}$$

$$EH_{min} = \gamma_{min} \cdot (H + Tt + Rise + Tb) \cdot w$$

$$= 0.030 \cdot (6 + 0.75 + 10 + 0.83) \cdot 1 = 0.527 \text{ klf}$$

Figure 12.5.3 illustrates the vertical and lateral earth pressures applied to the box culvert.

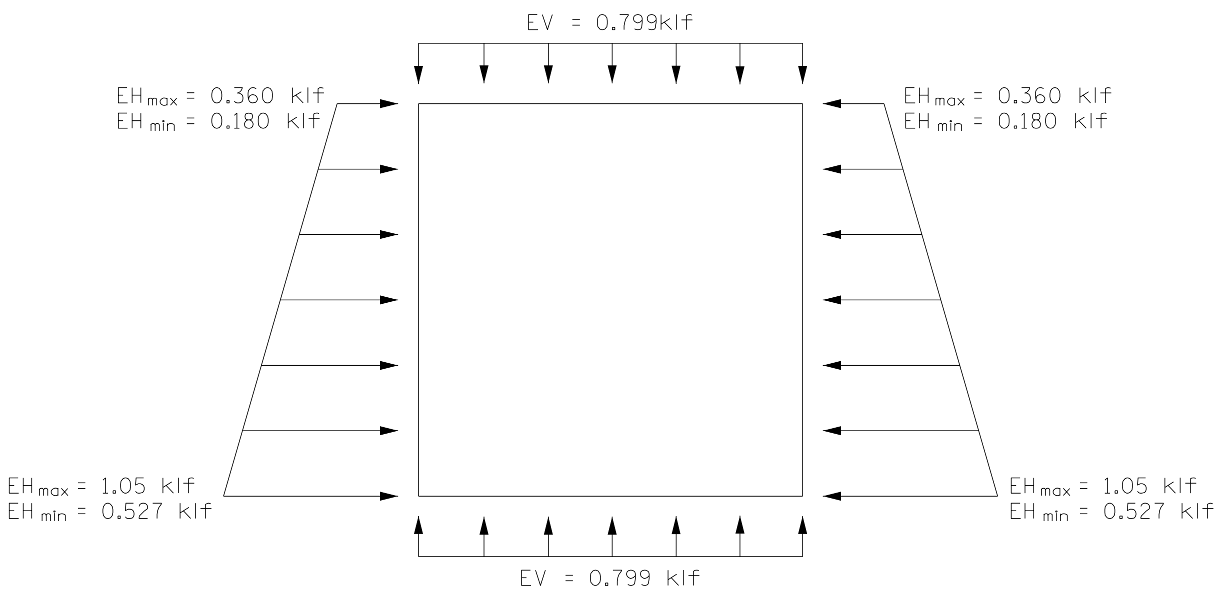


Figure 12.5.3
Earth Loads

C. Live Load Surcharge [3.11.6.4]

Use an active coefficient of lateral earth pressure k_a equal to 0.33.

The height for the live load surcharge calculation at the top of the culvert is the distance from the top surface of the top slab to the top of the pavement or fill.

The height is:

$$H_{\text{top of culvert}} = H_1 = 6 \text{ ft}$$

The equivalent fill height, h_{eq} is dependent on the depth of fill and can be found using AASHTO Table 3.11.6.4-1.

By interpolation, the equivalent height for a fill depth of 6 ft is:

$$h_{eq1} = 4 - \left(\frac{6 - 5}{10 - 5}\right)(4 - 3) = 3.80 \text{ ft}$$

The corresponding lateral live load surcharge on the top of the culvert is given as:

$$LS_{top} = k_a \cdot \gamma_s \cdot h_{eq} \cdot w = 0.33 \cdot 0.120 \cdot 3.80 \cdot 1 = 0.150 \text{ klf}$$

The height for the live load surcharge calculation at the bottom of the culvert is the distance from the bottom surface of the bottom slab to the top of the pavement or fill.

$$H_2 = H + Tt + Rise + Tb = 6 + 0.75 + 10 + 0.83 = 17.58 \text{ ft}$$

Again using interpolation and AASHTO Table 3.11.6.4.1, the equivalent height is:

$$h_{eq2} = 3 - \left(\frac{17.58 - 10}{20 - 10}\right)(3 - 2) = 2.24 \text{ ft}$$

The lateral live load surcharge located at the bottom of the culvert is given as:

$$LS_{bottom} = k_a \cdot \gamma_s \cdot h_{eq} \cdot w = 0.33 \cdot 0.120 \cdot 2.24 \cdot 1 = 0.089 \text{ klf}$$

D. Water Load [3.7.1]

Designers need to consider load cases where the culvert is full of water as well as cases where the culvert is empty. A simple hydrostatic distribution is used for the water load:

At the inside of the culvert, the lateral water pressure is:

$$WA_{top} = 0.00 \text{ klf}$$

$$WA_{bottom} = \gamma_w \cdot Rise \cdot w = 0.0624 \cdot 10 \cdot 1 = 0.624 \text{ klf}$$

Using a 2D frame model there is an opposite upward reaction from the soil caused by the water inside the culvert:

$$WA_{bottom\ reaction} = \frac{WA_{bottom} \cdot Span}{(Span + Ts)} = \frac{0.624 \cdot 10}{(10 + 0.67)} = 0.585 \text{ klf}$$

The water load is illustrated in Figure 12.5.4.

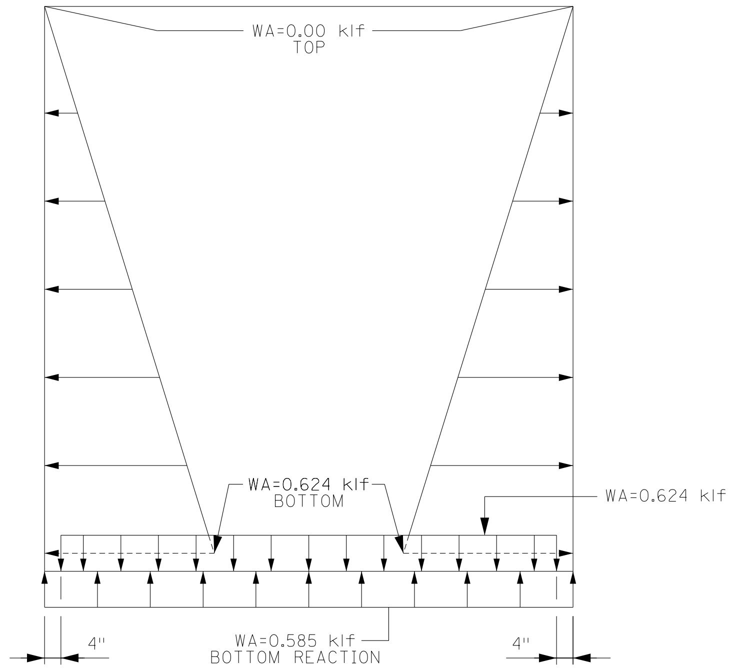


Figure 12.5.4
Water Load

E. Live Load
[3.6.1.3.3]

The design live loads include the HL-93 truck and tandem loads. Since the span of the box culvert is less than 15 ft, no lane load is applied.

[3.6.2.2]

Dynamic Load Allowance

The dynamic load allowance (IM) for culverts and other buried structures is reduced based on the depth of fill over the culvert. For strength and service limit states:

[3.6.2.2-1]

$$IM = 33 \cdot (1.0 - 0.125 \cdot D_E) = 33 \cdot (1.0 - 0.125 \cdot 6.0) = 8.3\%$$

The dynamic load allowance may not be taken less than zero.

[3.6.1.2.6]

Live Load Distribution

Live loads are assumed to distribute laterally with depth. The specifications permit designers to increase the footprint of the load with increasing depth of fill. The load is assumed to spread laterally 1.15 times H horizontally in each direction for every foot of fill above the culvert. The intensity of live loads at any depth is assumed to be uniform over the entire footprint.

[3.6.1.2.5]

The assumed tire contact area for each wheel has a width of 20 inches and a length of 10 inches.

Using the distances between wheel lines and axles, the live load intensities at the top of the box culvert can be found. For truck and tandem loadings, the influence area or footprint of the live load is found first. Then the sum of the weights of the wheels is used to determine the intensity of the live load.

[3.6.1.1.2] To determine the live load, use multiple presence factors (MPF). A single loaded lane with a MPF of 1.20 is used for strength and service limit states.

A single HL-93 truck axle configuration produces a live load intensity of:

$$w_{LL+IM} = \frac{2 \cdot P_w \cdot MPF \cdot (1 + IM)}{W \cdot L} = \frac{2 \cdot 16 \cdot 1.20 \cdot (1 + 0.083)}{14.57 \cdot 7.73} = 0.369 \text{ klf}$$

where:

$W = Axle_{spacing} + W_{tire} + 1.15 \cdot H = 6 + 1.67 + 1.15 \cdot 6 = 14.57$ ft
$L = L_{tire} + 1.15 \cdot H = 0.83 + 1.15 \cdot 6 = 7.73$ ft

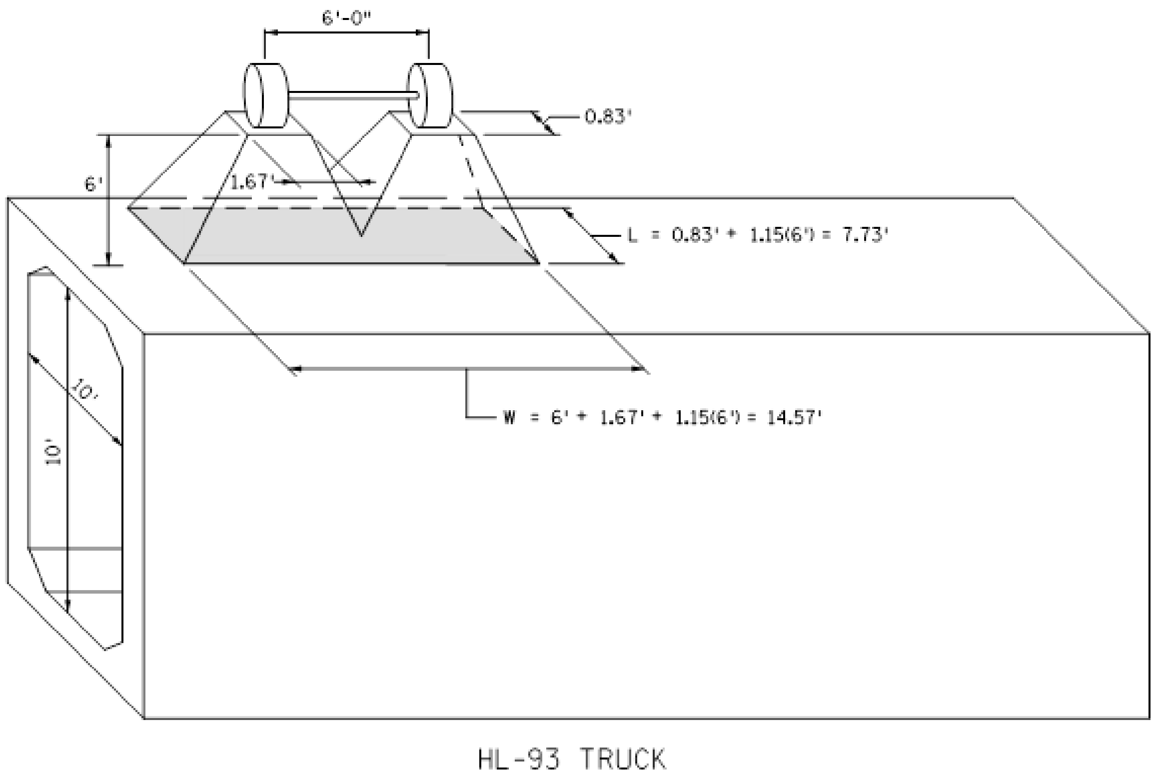
A tandem truck axle configuration produces a live load intensity of:

$$W_{LL+IM} = \frac{4 \cdot P_w \cdot MPF \cdot (1+IM)}{W \cdot L} = \frac{4 \cdot 12.5 \cdot 1.20 \cdot (1+0.083)}{14.57 \cdot 11.73} = 0.380 \text{ klf}$$

where:

W = as previously defined
$L = \text{Axle Spacing} + L_{tire} + 1.15 \cdot H = 4 + 0.83 + 1.15 \cdot 6 = 11.73$ ft

The live load intensities of the single and tandem axle configurations are compared. Since the tandem axle configuration produces a live load intensity slightly larger than that of the single axle configuration, the tandem axle configuration is used for design in both the strength and service limit states. Figure 12.5.5 illustrates the different live loads.



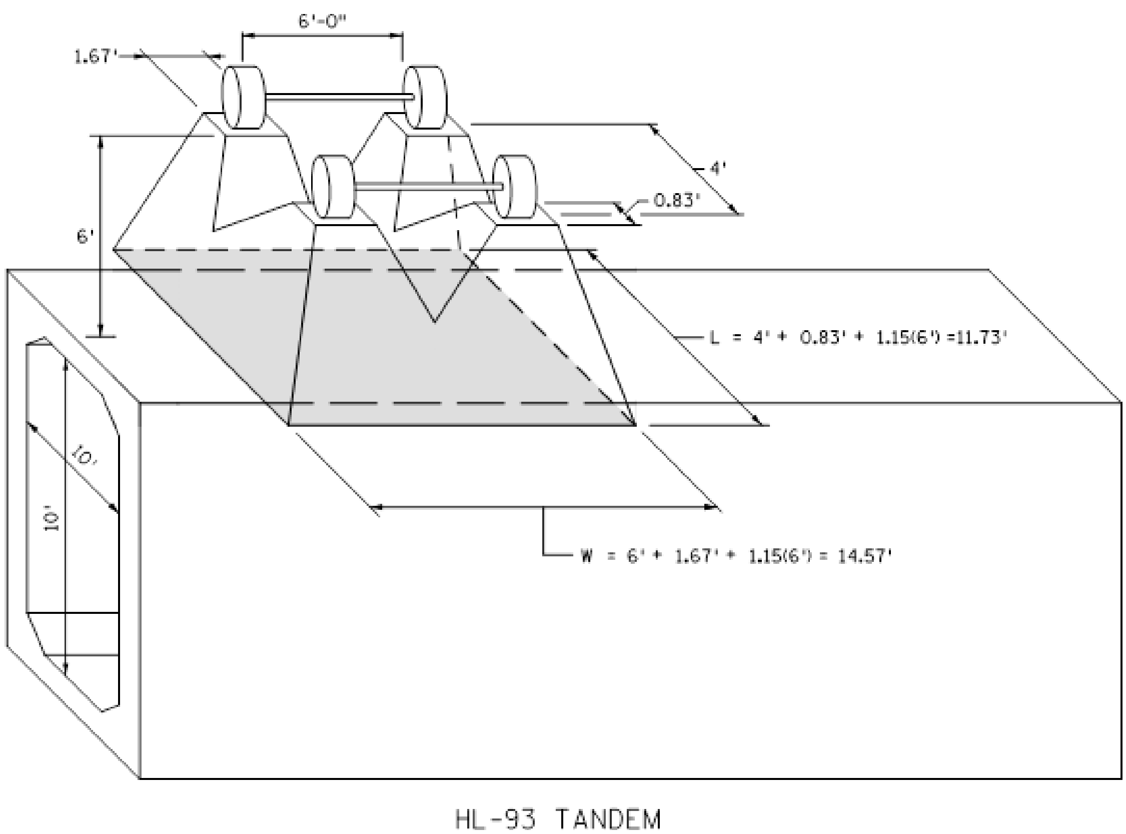


Figure 12.5.5
HL-93 Truck and Tandem Live Load Distribution

F. Select Applicable Load Combinations, Load Factors, and Load Modifiers

Strength Limit State:

Ia. Maximum vertical load and maximum horizontal load:

$1.25DC + (1.30)(1.05)EV + 1.75(LL+IM) + (1.35)(1.05)EH_{max} + 1.75LS$

Ib. Maximum vertical load and minimum horizontal load:

$1.25DC + (1.30)(1.05)EV + 1.75(LL+IM) + 1.00WA + (0.9/1.05)EH_{min}$

Ic. Minimum vertical load and maximum horizontal load:

$0.90DC + (0.90/1.05)EV + (1.35)(1.05)EH_{max} + 1.75LS$

Service Limit State:

Ia. Maximum vertical load and maximum horizontal load:

$1.00DC + 1.00EV + 1.00(LL+IM) + 1.00EH_{max} + 1.00LS$

Ib. Maximum vertical load and minimum horizontal load:

$1.00DC + 1.00EV + 1.0(LL+IM) + 1.00WA + 1.00EH_{min}$

Ic. Minimum vertical load and maximum horizontal load:

$1.00DC + 1.00EV + 1.00EH_{max} + 1.00LS$

G. Summary of Analysis Results

A structural analysis is performed using a standard commercial matrix-analysis program. The bottom slab of the box culvert is assumed rigid compared to the subgrade. Reactions to vertical loads applied to the culvert (earth, water, live load) are assumed to be carried by uniform, triangular or trapezoidal distributed reactions applied to the bottom slab. Box culverts supported on stiff or rigid subgrades (rock) would require further investigation. The haunches are included in the analysis by increasing the thickness of members near each corner.

[C5.13.3.6.1]

The internal forces at several locations of the box are presented in Tables 12.5.2 through 12.5.6. The sign convention for moment in the tables is: positive moment causes tension on the inside face of the culvert and negative moment causes tension on the outside face. The sign convention for thrust is: positive represents compression. The moments and thrust presented at top, bottom, or end locations are at the location where the typical section and haunch meet (Figure 12.5.6). The shear forces presented in Tables 12.5.4 and 12.5.5 are at the critical shear location, which is taken as the effective depth for shear (d_v) beyond the haunch to typical section intersection. The shear forces presented are the “governing” shear forces which are the shear with corresponding moments that give the lowest capacity/design (c/d) ratios.

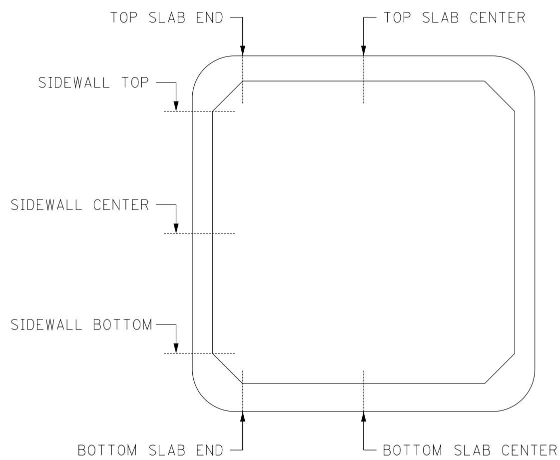


Figure 12.5.6
Structural Analysis Locations

Table 12.5.2
Structural Analysis Results: Moments (unfactored, kip-in)

	DC	EV	EH_{max}	EH_{min}	LS	WA	LL+IM (Pos)	LL+IM (Neg)
Sidewall Top	-4.31	-44.97	-4.84	-2.42	0.36	4.13	-	-21.57
Sidewall Center	-11.06	-39.44	63.02	31.51	10.7	-26.77	-	-18.82
Sidewall Bottom	-17.73	-33.97	-4.80	-2.40	-1.93	2.32	-	-16.21
Top Slab Center	17.19	89.03	-50.54	-25.27	-9.30	19.93	42.49	-
Top Slab End	6.39	12.60	-50.54	-25.27	-9.30	19.93	7.93	-2.35
Bottom Slab Center	38.43	103.87	-70.62	-35.31	-11.09	28.7	49.57	-
Bottom Slab End	5.50	27.43	-70.62	-35.31	-11.09	32.45	14.47	-

Table 12.5.3
Moment Load Combinations (kip-in)

	Strength			Service		
	Ia	Ib	Ic	Ia	Ib	Ic
Sidewall Top	-110.74	-102.45	-48.66	-75.33	-69.13	-53.76
Sidewall Center	40.39	-100.36	64.29	23.22	-64.58	23.22
Sidewall Bottom	-107.08	-96.63	-55.26	-74.64	-67.99	-58.43
Top Slab Center	129.47	215.66	3.87	88.88	143.39	46.39
Top Slab End	-66.85	37.33	-71.37	-43.20	21.58	-40.85
Bottom Slab Center	157.05	275.01	4.09	110.16	185.27	60.58
Bottom Slab End	-75.20	71.83	-91.06	-48.78	44.54	-48.78

Table 12.5.4
Structural Analysis Results: Shear (unfactored, kips)

	DC	EV	EH_{max}	EH_{min}	LS	WA	LL+IM (Pos)	LL+IM (Neg)
Sidewall Top*	0.14	-0.11	-2.26	-1.13	-0.42	0.9	0.01	-0.09
Sidewall Center	0.14	-0.11	-0.28	-0.14	0.02	0.23	0.01	-0.09
Sidewall Bottom*	0.14	-0.11	2.74	1.37	0.43	-1.31	0.01	-0.09
Top Slab Center	0.00	0.00	0.00	0.00	0.00	0.00	0.41	-0.4
Top Slab End*	-0.39	-2.75	0.00	0.00	0.00	0.00	-	-1.35
Bottom Slab Center	0.00	0.00	0.00	0.00	0.00	0.00	0.26	-0.27
Bottom Slab End*	1.16	2.69	0.00	0.00	0.00	-0.13	1.29	0.29

*Shear given at d_v away from haunch

Table 12.5.5
Governing Shear Load Combinations (kips)

	Strength			Service		
	Ia	Ib	Ic	Ia	Ib	Ic
Sidewall Top*	-4.10	-0.22	-3.93	-2.76	-0.30	-2.66
Sidewall Center	-0.31	0.15	-0.32	-0.22	0.13	-0.22
Sidewall Bottom*	4.48	-0.28	4.65	3.09	-0.01	3.19
Top Slab Center	0.72	0.72	0.00	0.41	0.41	0.00
Top Slab End*	-6.61	-6.61	-2.71	-4.50	-4.50	-3.14
Bottom Slab Center	-0.47	-0.47	0.00	-0.27	-0.27	0.00
Bottom Slab End*	7.36	7.23	3.35	5.13	5.0	3.84

*Shear given at d_v away from haunch

Table 12.5.6
Axial Thrust Load Combinations (kips)

	Strength			Service		
	Ia	Ib	Ic	Ia	Ib	Ic
Sidewall Top	11.43	11.43	4.96	7.87	7.87	5.73
Sidewall Center	11.44	11.44	4.96	7.88	7.88	5.73
Sidewall Bottom	11.44	11.44	4.96	7.88	7.88	5.73
Top Slab Center	5.88	0.47	5.77	3.95	0.65	3.89
Top Slab End	5.85	0.45	5.77	3.93	0.64	3.89
Bottom Slab Center	8.26	-0.11	8.29	5.69	0.38	5.70
Bottom Slab End	8.26	-0.11	8.29	5.69	0.38	5.70

The values in Tables 12.5.2 through 12.5.6 include dynamic load allowance and multiple presence factors.

H. Investigate Strength Limit State for Flexure [5.7.2.2] [5.7.3.2] [12.5.5]

Determine the required area of flexural reinforcement to satisfy the Strength I load combinations.

The resistance factor, ϕ, for flexure is 1.0 for precast box culverts.

$$M_u = \phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$$

The depth of the compression block is:

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$

Substituting for "a" in the first equation:

$$M_u = \phi \cdot A_s \cdot f_y \cdot \left[d - \frac{A_s \cdot f_y}{1.7 \cdot f'_c \cdot b}\right]$$

Inserting values for f_y, b, and ϕ:

$$M_u = 1.0 \cdot A_s \cdot 65 \cdot \left[d - \frac{A_s \cdot 65}{1.7 \cdot 5 \cdot 12}\right] \cdot \left[\frac{1}{12}\right]$$

Manipulate to get a quadratic equation:

$$3.45 \cdot A_s^2 - 5.42 \cdot A_s \cdot d + M_u = 0$$

$$A_s = \frac{5.42 \cdot d - \sqrt{29.34 \cdot d^2 - 13.81 \cdot M_u}}{6.91}$$

Sidewall:

Size the reinforcement assuming "d" dimensions based on an average 1 inch diameter wire, (d_w = 1.00 in) and a clear cover of 2 in.

$$d = \text{thickness} - \text{cover} - \frac{d_w}{2} = 8 - 2 - \frac{1}{2} = 5.50 \text{ in}$$

Referring to Table 12.5.2, the peak moment for tension on the outside face is 110.74 k-in (top, Strength Ia). Insert d and M_u values to compute A_s. The required area of steel is 0.321 in^2/ft. For conservatism round up to 0.33 in^2/ft.

The peak moment for tension on the inside face is 64.29 k-in (center, Strength Ic). The required area of steel is 0.19 in^2/ft.

Top Slab:

For the top slab “d” is:

$$d = 9 - 2 - \frac{1}{2} = 6.50 \text{ in}$$

The peak moment for tension on the outside face is 71.37 k-in (Strength Ic). The required area of steel is 0.18 in^2/ft.

The peak moment for tension on the inside face is 215.66 k-in (Strength Ib). The required area of steel is 0.54 in^2/ft.

Bottom Slab:

$$d = 10 - 2 - \frac{1}{2} = 7.50 \text{ in}$$

The peak moment for tension on the outside face is 91.06 k-in. The required area of steel is 0.19 in^2/ft.

The peak moment for tension on the inside face is 275.01 k-in. The required area of steel is 0.60 in^2/ft.

I. Check Crack Control
[5.7.3.4]

To ensure that the primary reinforcement is well distributed, crack control equations are checked. The equations are dependent on the tensile stress in steel reinforcement at the service limit state, the concrete cover, and the geometric relationship between the crack width at the tension face versus the crack width at the reinforcement level (β_s). The exposure factor, γ_e, is 0.75, since culverts are substructures exposed to water (Class 2).

[C5.7.3.4]

The wire spacing, s, must satisfy:

[5.7.3.4-1]

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

Solve the equation above for the reinforcement stress at service, $f_{ss:}$

$$f_{ss} \le \frac{700 \cdot \gamma_e}{\beta_s \cdot (s + 2 \cdot d_c)} \le 0.6 f_y$$

The strain ratio, β_s, is defined as:

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

Top Slab:

For the top slab inside face, the governing service limit state moment is 143.39 k-in. The axial thrust is 0.65 kips and is accounted for in the crack control check per AASHTO C.12.11.3-1. Spacing of the wires is assumed to be 4 inches and the area of flexural reinforcement is 0.54 in^2/ft.

$$d_c = \text{Cover} + \frac{1}{2} d_w = 2 + \frac{1}{2} \cdot 1 = 2.50\text{in}$$

Then solve for β_s:

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{2.50}{0.7 \cdot (9 - 2.50)} = 1.55$$

The allowable reinforcement stress, f_{ss} can then be calculated as:

$$f_{ss} = \frac{700 \cdot \gamma_e}{\beta_s \cdot (s + 2d_c)} = \frac{700 \cdot 0.75}{1.55 \cdot (4 + 2 \cdot 2.50)} = 37.63 \text{ ksi}$$

$$0.6f_y = 0.6 \cdot 65 = 39.00 \text{ ksi} > 37.63 \text{ ksi}$$ <u>Use 37.63 ksi</u>

[C12.11.3] Find the actual stress provided in the steel:

$$e = \frac{M_s}{N_s} + d - \frac{h}{2} = \frac{143.39}{0.65} + 6.5 - \frac{9}{2} = 222.60$$

$$j = 0.74 + 0.1 \cdot \left(\frac{e}{d}\right) = 0.74 + 0.1 \cdot \left(\frac{222.60}{6.5}\right) = 4.16$$

For "j" use the smaller of 4.16 or 0.9, then j=0.9

$$i = \frac{1}{1 - \frac{j \cdot d}{e}} = \frac{1}{1 - \frac{0.9 \cdot 6.5}{222.60}} = 1.027$$

[C12.11.3-1]

$$f_s = \frac{M_s + N_s \cdot (d - \frac{h}{2})}{A_s \cdot j \cdot i \cdot d} = \frac{143.39 + 0.65 \cdot (6.5 - \frac{9}{2})}{0.54 \cdot 0.9 \cdot 1.027 \cdot 6.5}$$

$$= 44.60 \text{ ksi} > 37.63$$ <u>No Good</u>

Increase the area of steel provided, so that f_s is less than f_{ss}. The new area of steel is given as:

$$A_{scrack} = \frac{f_s}{f_{ss}} \cdot A_s = \frac{44.60}{37.63} \cdot 0.54 = 0.64 \text{ in}^2/\text{ft}$$

For the top slab outside face crack control did not govern. See Table 12.5.6 for results.

Bottom Slab:

The area of steel for the bottom slab inside face is evaluated with a service moment of 185.27 k-in, an axial thrust of 0.38 kips, and d_c equal to 2.50 inches. The required area of steel to satisfy crack control for the bottom slab inside face is 0.70 in^2/ft.

Sidewall:

The area of steel for the sidewall inside face is evaluated with a service moment of 23.22 k-in, an axial thrust of 5.73 kips, and d_c equal to 2.50 inches. The required area of steel to satisfy crack control for the sidewall inside face is 0.03 in^2/ft.

J. Check Fatigue [C12.5.3]

Fatigue check calculations are not required for the design of box culverts.

K. Check Minimum Reinforcement [12.11.4.3.2]

For precast culverts, the minimum amount of flexural reinforcement in the cross section is a percentage of the gross area:

Minimum sidewall flexural reinforcement:

$$A_s = 0.002 \cdot b \cdot Tt = 0.002 \cdot 8 \cdot 12 = 0.20 \text{ in}^2/\text{ft}$$

Minimum top slab flexural reinforcement:

$$A_s = 0.002 \cdot b \cdot Ts = 0.002 \cdot 9 \cdot 12 = 0.22 \text{ in}^2/\text{ft}$$

Minimum bottom slab reinforcement:

$$A_s = 0.002 \cdot b \cdot Tb = 0.002 \cdot 10 \cdot 12 = 0.24 \text{ in}^2/\text{ft}$$

For precast concrete box culverts, the MnDOT minimum reinforcement requirement is 0.06 in^2/ft, regardless of the size of the box culvert.

L. Check Maximum Reinforcement Limit [5.5.4.2] [5.7.2.1]

The strain in the reinforcement is checked to ensure that the section is tension controlled. For a resistance factor of 1.0 to be used for flexure, the reinforcement strain must be at least 0.005.

This is satisfied if:

$$\frac{c}{d} < 0.375$$

where:

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b}$$

$f'_c = 5$ ksi

$$0.65 \le \beta_1 = 0.85 - 0.05 \cdot \left(f'_c - 4.0\right) = 0.80 \le 0.85$$

Sidewall:

Outside face $\quad c = 0.513$ in $\quad \frac{c}{d} = \frac{0.513}{5.5} = 0.09$ OK

Top Slab:

Inside face $\quad c = 1.02$ in $\quad \frac{c}{d} = \frac{1.02}{6.5} = 0.16$ OK

Bottom Slab:

Inside face $\quad c = 1.11$ in $\quad \frac{c}{d} = \frac{1.11}{7.5} = 0.15$ OK

Minnesota Concrete Pipe Association (MCPA) members also prefer to have a maximum reinforcement ratio of $0.6\rho_b$ to limit congestion during fabrication. The balanced reinforcement ratio is given by:

$$\rho_b = \frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \left[\frac{87}{87 + f_y}\right] = \frac{0.85 \cdot 0.80 \cdot 5}{65} \cdot \left[\frac{87}{87 + 65}\right] = 0.0299$$

$$\rho \le 0.60 \cdot \rho_b = 0.60 \cdot 0.0299 = 0.018$$

For the top slab b=12 in., d= 6.5 in., and $A_s = 0.64$ in^2, the member reinforcement ratio is given as:

$$\rho = \frac{A_s}{A_c} = \frac{A_s}{b \cdot d} = \frac{0.64}{12 \cdot 6.5} = 0.0082 < 0.018$$ OK

Sidewall:

For the sidewall with b=12 in., d= 5.5 in., and $A_s = 0.20$ in^2 the reinforcement ratio is 0.0030 < 0.018. OK

Bottom Slab:

For the bottom slab with b=12 in., d= 7.5 in., and $A_s = 0.70$ in^2 the reinforcement ratio is 0.0077 < 0.018. OK

Table 12.5.7
Flexural Design Calculation Summary

		Sidewall		Top Slab		Bottom Slab	
		Inside	**Outside**	**Inside**	**Outside***	**Inside**	**Outside***
Strength	Moment (k-in)	64.29	110.74	215.66	-	275.01	-
	Assumed d (in)	5.5	5.5	6.5	-	7.5	-
	Req'd steel area (in^2/ft)	0.19	**0.33**	0.54	-	0.60	-
Service	Moment (k-in)	23.22	75.33	143.39	-	185.27	-
	Axial Thrust (kip)	5.73	7.87	0.65	-	0.38	-
	Assumed d (in)	5.5	5.5	6.5	-	7.5	-
	Assumed d_c (in)	2.5	2.5	2.5	-	2.5	-
	f_{ss} (ksi)	35.37	35.37	37.64	-	39.00	-
	f_s (ksi)	9.13	29.49	37.64	-	39.00	-
	Req'd steel area for crack control (in^2/ft)	0.03	0.28	**0.64**	-	**0.70**	-
Min Check	0.002 A_g (in^2/ft)	**0.20**	0.20	0.22	**0.22**	0.24	**0.24**

*The minimum reinforcement always governs (given MnDOT's reinforcement lap criteria).

M. Summary of Required Flexural Reinforcement

The final amount of reinforcement is:

Sidewall:
Outside face As1 = 0.33 in^2/ft
Inside face As4 = 0.20 in^2/ft

Top Slab:
Outside face As7 = 0.22 in^2/ft
Inside face As2 = 0.64 in^2/ft

Bottom Slab:
Outside face As8 = 0.24 in^2/ft
Inside face As3 = 0.70 in^2/ft

N. Check Shear
[5.14.5.3]
[12.5.5]

Sidewall

The critical section for shear is taken at d_v from the tip of the haunch. The maximum design shear at this location is:

V_u = 4.48 kips with associated M_u=80.18 k-in

[5.8.3.3]

The nominal shear resistance without the presence of shear reinforcement is given as:

$$V_r = \phi \cdot V_n$$

where:

$$V_n = \text{Lesser of } 0.25 \cdot f'_c \cdot b_v \cdot d_v \text{ or } V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

The parameter, b_v, is the assumed member width and d_v is the effective shear depth. d_v is calculated as:

[5.8.2.9]

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.33 \cdot 65}{0.85 \cdot 5 \cdot 12} = 0.42 \text{ in}$$

$$d_v = \max(0.72h,\ 0.9d,\ d - a/2) = \max(0.72 \cdot 8,\ 0.9 \cdot 5.5,\ 5.5 - 0.42/2)$$
$$= \max(5.76,\ 4.95,\ 5.29) = 5.76 \text{ in} \qquad \underline{\text{Use } d_v = 5.76 \text{ in}}$$

MnDOT takes the shear resistance for box culverts to be the greater of that computed using LRFD Article 5.8.3.4.1 and 5.8.3.4.2. Using the "General Procedure", the crack spacing parameter, s_{xe}, is taken as:

[5.8.3.4.2]

$$s_{xe} = s_x \frac{1.38}{a_g + 0.63} = 5.76 \cdot \frac{1.38}{0.75 + 0.63} = 5.76 \text{ in and } 12 \text{ in} \le s_{xe} \le 80 \text{ in}$$

where:

$s_x = d_v = 5.76$ in
a_g = maximum aggregate size (in) = 0.75 in

Use s_{xe}=12 in

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u|\right)}{E_s \cdot A_S} = \frac{\left(\frac{|80.18|}{5.76} + 0.5 \cdot 0 + |4.48|\right)}{29000 \cdot 0.33} = 0.0019$$

where the magnitude of the moment, M_u, is not to be less than:

$$M_u \ge V_u \cdot d_v = 4.48 \cdot 5.76 = 25.8 \text{ k-in}$$

Because there is no shear reinforcement the value of β is taken as:

$$\beta = \frac{4.8}{1+750\varepsilon_s} \cdot \frac{51}{39 + s_{xe}} = \frac{4.8}{1+750 \cdot 0.0019} \cdot \frac{51}{39+12} = 1.98$$

AASHTO LRFD 5.8.3.4.1 allows a value of 2.0 to be used since the depth of the member is less than 16 in. and it is not subjected to axial tension. Therefore, use β=2.00.

[5.8.3.3] The factored shear resistance is then:

$$\phi V_c = \phi \cdot 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v = 0.9 \cdot 0.0316 \cdot 2.0 \cdot \sqrt{5} \cdot 12 \cdot 5.76$$

$$= 8.79 \text{ kips}$$

$$\phi V_n = 8.79 \text{ kips} \le \phi \cdot 0.25 \cdot f'_c \cdot b_v \cdot d_v$$

$$= 0.9 \cdot 0.25 \cdot 5 \cdot 12 \cdot 5.76 = 77.8 \text{ kips}$$ Use 8.79 kips

$$\phi V_n = 8.79 \text{ kips} > V_u = 4.48 \text{ kips}$$ OK

Top Slab

The maximum design shear at a distance d_v from the tip of the haunch is:

V_u = 6.61 kips with associated M_u = 6.34 k-in

[5.14.5.3] The shear resistance is:

$$\phi V_c = \phi \cdot \left[0.0676 \cdot \sqrt{f'_c} + 4.6 \cdot \frac{A_s}{b \cdot d_e} \cdot \frac{V_u \cdot d_e}{M_u}\right] \cdot b \cdot d_e$$

where the quantity

$$\frac{V_u \cdot d_e}{M_u} \le 1.0 \quad \frac{6.61 \cdot 6.5}{6.34} = 6.78 > 1.0$$ Use 1.0

then

$$\phi V_c = 0.9 \cdot \left[0.0676 \cdot \sqrt{5} + 4.6 \cdot \frac{0.22}{12 \cdot 6.5} \cdot 1\right] \cdot 12 \cdot 6.5 = 11.52 \text{ kips}$$

The shear capacity for the top slab cast monolithically with the sidewalls is not to be taken less than:

$$\phi V_c = \phi \cdot 0.0948 \cdot \sqrt{f'_c} \cdot b \cdot d_e = 0.9 \cdot \left(0.0948 \cdot \sqrt{5} \cdot 12 \cdot 6.5\right)$$

$$= 14.88 \text{ kips} > 11.52 \text{ kips}$$

$$\phi V_c = 14.88 \text{ kips} > V_u = 6.61 \text{ kips}$$ OK

Bottom Slab

The maximum design shear at a distance d_v from the tip of the haunch is:

V_u = 7.36 kips with associated M_u=33.28 k-in

[5.14.5.3] The shear capacity is:

$$\phi V_c = \phi \cdot \left[0.0676 \cdot \sqrt{f'_c} + 4.6 \cdot \frac{A_s}{b \cdot d_e} \cdot \frac{V_u \cdot d_e}{M_u}\right] \cdot b \cdot d_e$$

where the quantity

$$\frac{V_u \cdot d_e}{M_u} \leq 1.0 \quad \frac{7.36 \cdot 7.5}{33.28} = 1.66 \geq 1.0 \qquad \underline{\text{Use } 1.0}$$

then

$$\phi V_c = 0.9 \cdot \left(0.0676 \cdot \sqrt{5} + 4.6 \cdot \frac{0.24}{12 \cdot 7.5} \cdot 1.0\right) \cdot 12 \cdot 7.5 = 13.24 \text{ kips}$$

The shear capacity for the bottom slab cast monolithically with the sidewalls is not to be taken less than:

$$\phi V_c = \phi \cdot 0.0948 \cdot \sqrt{f'_c} \cdot b \cdot d_e = 0.9 \cdot \left(0.0948 \cdot \sqrt{5} \cdot 12 \cdot 7.5\right)$$

$$= 17.17 \text{ kips} > 13.24 \text{ kips}$$

$$\phi V_c = 17.17 \text{ kips} > V_u = 7.36 \text{ kips} \qquad \underline{\text{OK}}$$

O. Check Thrust

[5.7.4] The axial capacity of the culvert should be checked to ensure it satisfies the provisions of LRFD Article 5.7.4. The sidewall member will be checked since it has the largest thrust value and least amount of thickness. The design axial load is then:

P_u = 11.44 kips (top, Strength Ia and Ib)

[5.5.4.2.1] Without stirrups in the section, the resistance factor for compression is 0.70.

[5.7.4.5]

$$\phi P_n = \phi \cdot 0.10 \cdot f'_c \cdot A_g = 0.70 \cdot 0.10 \cdot 5 \cdot 8 \cdot 12 = 33.6 \text{ kips} > 11.44 \text{ kips} \qquad \underline{\text{OK}}$$

[5.5.4.2.1] The axial capacity is adequate. MnDOT does not allow the consideration of the benefit from the applied axial force in computation of bending resistance of the sidewalls.

P. Detailing/ Reinforcing Lengths [5.11.2] [5.11.6]

The concrete cover must be between 1½ inches minimum and 2 inches maximum. Also, the As1 reinforcing needs to be extended in the top and bottom slabs until the As7 or As8 reinforcing is adequate to resist the negative moment. In addition, the As7 and As8 reinforcement needs to be properly lapped to the As1 reinforcement to ensure reinforcement continuity. In this example As1 is not needed for shear resistance, so it does not need to be lapped past d_v from the tip of the haunch. For conservatism and simplicity of the design and construction, calculate development lengths and lap lengths on the bottom slab and then apply the longer computed length to both the top and bottom slabs. See Figure 12.2.3.7 for more detail. A summary of these calculations follows.

For As1, the reinforcing on the outside of the sidewalls, the area of steel required is 0.33 in^2/ft. The development length, assuming the maximum, worst case wire spacing of 4 inches, is given as:

[5.11.2.5.2-1]

$$l_d = 8.50 \cdot \frac{A_w \cdot f_y}{s_w \cdot \sqrt{f'_c}} = 8.50 \cdot \frac{\frac{0.33}{3} \cdot 65}{4 \cdot \sqrt{5}} = 6.79 \text{ in}$$

Since the minimum development length for smooth wire fabric is the embedment of two cross wires with the closer cross wire not less than 2 inches from the critical section, the minimum development length assuming 4 inch spacing is:

$l_{dmin} = 4 + 4 + 2 = 10 \text{ in} > 6.79 \text{ in}$ <u>Use 10 in</u>

[5.11.6.2]

For As8, the area of steel required is 0.24 in^2/ft. The required lap length is given as the greater of $1.5 \cdot l_d$ or 6 inches.

Then the minimum As8 lap length is

$1.5 \cdot l_d = 1.5 \cdot 10 = 15 \text{ in} > 6 \text{ in}$ <u>Use 15 in</u>

From the structural analysis software results, the distance to the point where the negative moment can be resisted by As8 is 0 inches. The lap length of 15 inches is used, since it is greater than the development length of As1 (10 in). The calculated M length is given as:

$$M = 0 + 15 = 15 \text{ in}$$

However, the minimum M length for the bottom slab based on MnDOT criteria is below. Note that 6 inches is added for consistency with past practice.

$$M_{min} = Ts + haunch + \max(d_e, d_v) + 6" = 8+12+7.5+6 = 33.5 \text{ in. say } 2'\text{-}10"$$

The length of the As1 reinforcement is:

$$120 + 9 + 10 - 1.5 - 1.5 + 34 + 34 - 1.5 - 1.5 = 201 \text{ in. or } 16'\text{-}9"$$

The length of the As7 and As8 bars are then:

$$120 + 8 + 8 - 34 - 34 + 15 + 15 = 98 \text{ in. or } 8'\text{-}2"$$

The lengths of the As2, As3 and As4 bars are the span or rise plus 6 inches to ensure the bar is properly embedded into the member.

A summary of the reinforcing lengths is below.

Table 12.5.8
Reinforcement Length Summary

Reinforcement	Length
"M" Dimension	2'-10"
As1	16'-9"
As2	10'-6"
As3	10'-6"
As4	10'-6"
As7	8'-2"
As8	8'-2"

Q. Summary

Figure 12.5.6 illustrates the required reinforcing for the inside face and outside face of the sidewalls, top slab, and bottom slab. Longitudinal steel area is 0.06 in^2/ft.

Note that if reinforcing bars are used rather than welded wire fabric, the required reinforcement must be increased by a factor of 65/60 = 1.08 to account for the difference in yield strength. Also, crack control must be rechecked.

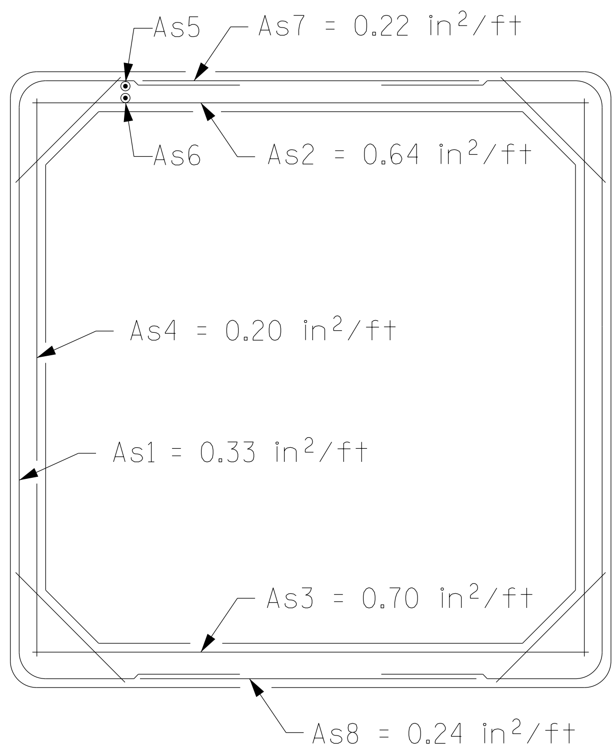


Figure 12.5.7
Box Culvert Reinforcement

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12.6 16'x12' Precast Concrete Box Culvert Live Load Distribution Example

This example illustrates the computation of live load to a precast box culvert with a 16 foot span under 1 foot of fill. The culvert has a top slab thickness of 12 inches, bottom slab thickness of 11 inches and sidewall thicknesses of 8 inches. For an example of all other loading calculations, analysis, design, or detailing, see Article 12.5 of this manual.

A. Live load [3.6.2.2] [3.6.2.2-1]

Dynamic Load Allowance

$$IM = 33 \cdot \left[1.0 - 0.125 \cdot D_E\right] = 33 \cdot \left[1.0 - 0.125 \cdot 1.0\right] = 28.9\%$$

[3.6.1.2.5] [3.6.1.2.6] [4.6.2.10]

Live Load Distribution

Since the depth of fill is less than 2 feet, live loads are distributed using an equivalent strip width.

A single loaded lane with the single lane multiple presence factor is analyzed. Assuming traffic travels primarilly parallel to the span, the axle loads are distributed to the top slab accordingly.

Perpendicular to the span:

$$E = 96 + 1.44 \cdot S = 96 + 1.44 \cdot 16 = 119.04 \text{ in}$$

Parallel to the span:

$$E_{Span} = L_T + LLDF \cdot H = 10 + 1.15 \cdot 12 = 23.8 \text{ in}$$

where:

E = Equivalent distribution width perpendicular to span (in)
E_{span} = Equivalent distribution length parallel to span (in)
L_T = Length of tire contact area parallel to span (in)
$LLDF$ = 1.15, factor for distribution of live load through depth of fill
H = Depth of fill from top of culver to top of pavement (in)
S = Clear span (ft)

AASHTO Article 4.6.2.10.4 states that the load distribution width shall not exceed the length between the adjacent joints without a means of shear transfer across the joint. Since this culvert has less than 2 feet of fill, MnDOT requires a distribution slab. A distribution slab is considered to be a means of shear transfer across the box culvert joints, so in this example the load distribution width is not limited to the section length and the full width of 9.92’ can be used.

[3.6.1.1] A single HL-93 truck axle configuration produces a live load intensity of:

$$w_{LL+IM} = \frac{2 \cdot P_w \cdot MPF \cdot (1+IM)}{\text{Influence Area}} = \frac{2 \cdot 16 \cdot 1.2 \cdot (1+0.289)}{9.92 \cdot 1.98} = 2.52 \text{ klf}$$

A single tandem vehicle produces a live load intensity of:

$$w_{LL+IM} = \frac{4 \cdot P_w \cdot MPF \cdot (1+IM)}{\text{Influence Area}} = \frac{4 \cdot 12.5 \cdot 1.2 \cdot (1+0.289)}{2 \cdot 9.92 \cdot 1.98} = 1.97 \text{ klf}$$

where:

MPF = 1.2 Multiple Presence Factor for one lane
P_w = Wheel load for design vehicle (kips)

[3.6.1.1.2]
[3.6.1.2.2]
[3.6.1.2.3]
[3.6.1.2.4]

The design lane load is a 0.64 klf load uniformly distributed in the longitudinal direction and assumed to be distributed uniformly over ten feet in the transverse direction. The lane load is not subjected to a dynamic load allowance.

$$w_{Lane} = \frac{0.64}{10 \cdot 1.15} = 0.056 \text{ klf}$$

The following figures illustrate the different live loads and how they are applied to the box culvert with less than 2 feet of fill.

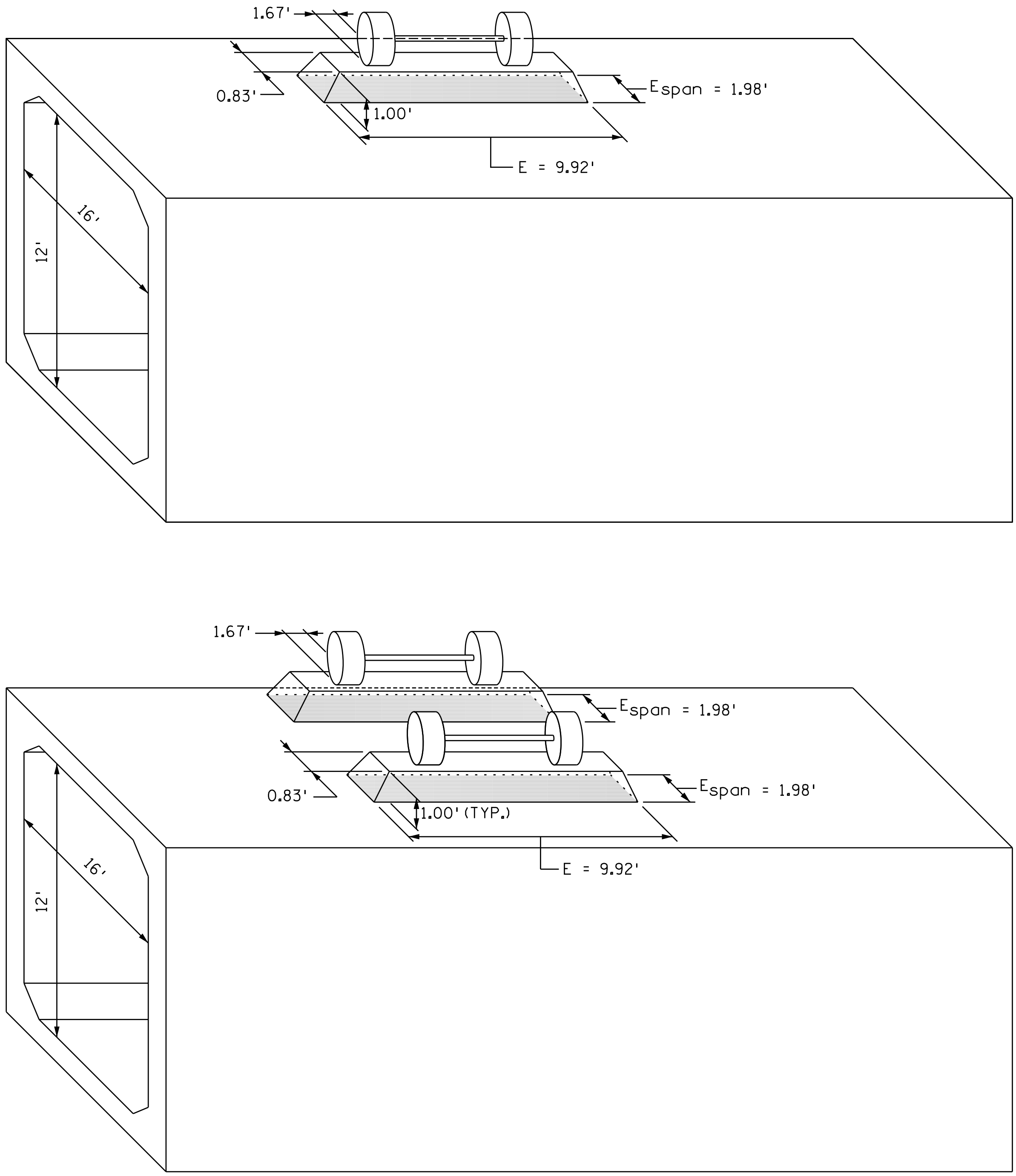

Figure 12.6.1
Live Load Distribution, Single HL-93 Truck and Tandem Axle Configurations

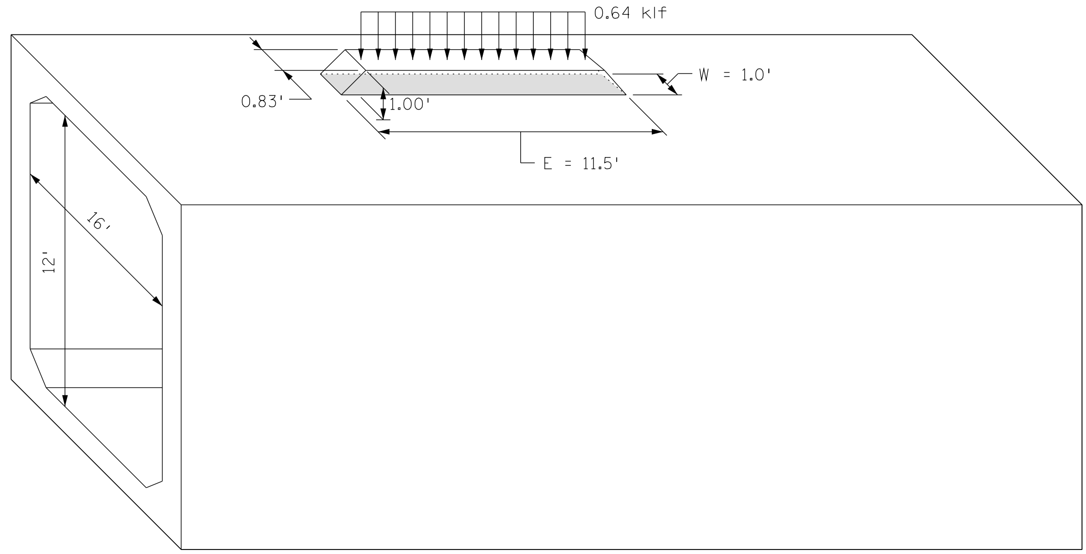

Figure 12.6.2
Live Load Distribution, HL-93 Lane Load

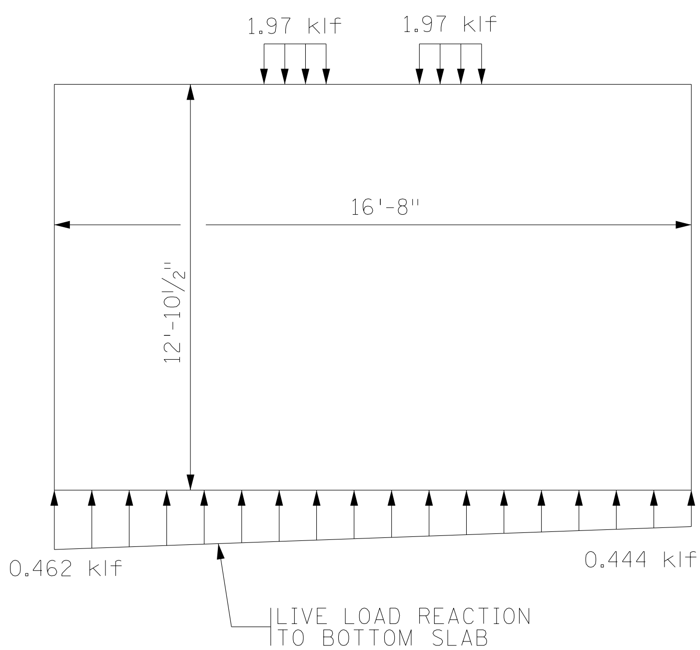

Figure 12.6.3
Live Load Distribution, Single HL-93 Tandem Applied to Top and Bottom Slabs