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. Pipes and culverts with lateral horizontal dimensions less than 10'-0" are not classified as bridges. Typically these smaller buried structures do not require extensive design and are simply selected from standard design tables. Buried structures with lateral horizontal dimensions greater than or equal to 10'-0" are considered bridges and require a plan be prepared by the Bridge Office. Figure 12.2.1 contains a typical design request from a District.

[12.6.6]

In addition to pipes and box culverts, precast concrete arches and long-span corrugated steel structures are used as buried structures. The loads that are applied to a buried structure vary with the site (trench or embankment condition). A buried structure constructed in a trench condition will carry less vertical load than a structure constructed in an embankment condition. The sidewalls of the trench are assumed to carry a portion of the vertical load. In most cases, buried bridge structure designs should be based on embankment conditions.

The means by which a buried structure carries vertical load varies significantly between different structure types. Box culverts and rigid pipes are assumed to carry the design loads internally. Flexible pipe and corrugated steel structures are assumed to carry loads with soil-structure interaction. Careful backfilling and compaction procedures are required for flexible structures to ensure that the assumed soil-structure capacity is provided and that settlements are not excessive.

Mn/DOT is currently developing guidelines for the design and use of three-sided culverts. Information will be made available in the near future.

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. Mn/DOT 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.

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

Standard designs for precast concrete box culverts are available with openings varying from 6 to 14 feet wide by 4 by 14 feet high. The designs utilize concrete strengths between 5 and 6 ksi and are suitable for fill heights ranging from less than 2'-0" to a maximum of 25'-0".

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. Fill heights extend from 0 to 25 feet. Shop drawing submittals for Mn/DOT 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 between $1^1/_2$ and 2". If conventional rebar is used, the steel area required 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. The maximum thickness of welded wire fabric is 0.5 inch per layer. A maximum of two layers of welded wire fabric can be used for primary reinforcement. If two layers are used, the layers need to be oriented such that the longitudinal and transverse wires always alternate. The minimum amount of longitudinal reinforcement (parallel to the axis of the culvert) is 0.06 in^2/ft.

Minnesota Department of Transportation

Memo

Transportation District 8
2505 Transportation Rd.
P.O. Box 768
Willmar, MN 56201

Office Tel: 320-231-5195
Fax No: 320-231-5168

[*date*]

To: Kevin L. Western
Bridge Design Engineer, M.S. 610

From: Paul Rasmussen
Design/Hydraulics Engineer

Phone: 320-214-3708

Subject: S.P. 6420-20 (T.H. 19)

Please prepare a design for concrete box culvert. Tabulated below and attached is the information required to prepare plans. The letting date for this project is 11/17/00. Please submit completed plans to this office before 07/01/00.

State Project No. **6403-30**		Func. 2 Work Authority: **T80057**	
Location Description (1): **T.H. 19 over JD #33, 11.3 Mi. West of Redwood Falls**			
Reference: **59+00.83**		Station: **342+91.07**	
Section: **1**	Range: **38W**	Township: **112N**	Township Name: **Vesta**
Stream Crossing: **Judicial Ditch #33**		County: **Redwood**	
Structure Type: **Concrete Box Culvert Replacement**			
New Structure Number: **64X07**		Existing Structure Number:	
Number of Barrels: **1**	Opening Width: **10 ft.**	Opening Height: **8 ft.**	Spa btwn Precast Barrels: **NA**
Depth of Cover (2): **6.8 ft.**	Skew Angle: **0°**		End Sections: **Square**
Inlet Elevation (new structure): **1037.34**		Outlet Elevation (new structure): **1036.60**	
Inlet Elevation (inplace structure): **1037.34**		Outlet Elevation (inplace structure): **1036.60**	
Extension Distances from end of inplace: **NA**			
Plans Requested: **Precast**			

(1) T.H. over _____, ___ Mi. ___ of Jct. T.H. _____ & T.H. _____, ___ of ___. (modify as necessary)

(2) Minimum fill over culvert rounded up to the next tenth meter.

Attachments: Sketch of Cross Section
Sketch of Plan View

Cc: David Dahlberg
Bruce Iwen

Figure 12.2.1

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 $7\,/_2^\circ$	0°
$7^1/_2^\circ$ to $22^1/_2^\circ$	15°
$22^1/_2^\circ$ to $37^1/_2^\circ$	30°
$37^1/_2^\circ$ to 45°	45°

A lateral soil pressure of 0.060 ksf should be used for the aprons.

The 45° skew aprons should be designed with a 0.075 ksf pressure on the longer length wall. Mn/DOT also requires on 45° skew aprons over 6'-0" high, additional extra strong ties between the barrel and first end section, and between the first and second end sections. Additional ties are required to resist unequal pressures on opposite sides of the skewed apron.

12.2.2 Cast-In-Place

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

12.3 Design Guidance

The standard designs are also based on a minimum equivalent fluid pressure of 0.030 kcf and on a maximum equivalent fluid pressure of 0.060 kcf.

Box culverts with fill heights less than 2'-0" are constructed with a distribution slab. No structural benefit from the distribution slab is considered during design.

Dead Loads

The F_e factor is used to adjust the vertical earth load carried by the culvert. It is intended to approximate soil-structure interaction effects and installation conditions (trench versus embankment).

Compacted fill on top of buried structures is assumed to have a unit weight of 0.130 kcf.

Concrete self-weight computations should be based on 0.150 kcf.

Water loads should be based on 0.062 kcf.

Live Load

The application of live loads is similar to the Standard Specifications. For fill heights less than 2'-0" a loading similar to that applied to a concrete slab bridge is used. For fill depths 2'-0" or greater the live load wheel pressure spreads with an increase in fill height. The assumed tire contact area for each wheel has a width of 20" and a length of 10". The load is assumed to spread laterally 0.57 feet in both directions 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. Wheel loads are assumed to distribute both longitudinally and transversely. Lane loads are assumed to be very long and consequently only distribute transversely.

The dynamic load allowance (IM) for culverts and other buried structures is reduced based on the depth of fill over the culvert. LRFD designs require that impact (dynamic load allowance) be considered for fill heights of up to 8'-0".

In determining the live load for structural analysis, use the multiple presence factors. One lane loaded for strength and service limit states has a multiple presence factor of 1.20. For two lanes loaded, use 1.00. No multiple presence factor is used for the fatigue limit state. A single HL-93 truck axle is used for the fatigue check.

The approximate strip method is used for the design with the 1'-0" wide design strip oriented parallel to the direction of traffic (longitudinal direction.) The design live loads include the HL-93 truck, lane and tandem loads. For both the strength and service limit states, consider the following three load cases:

1. Maximum vertical load on the roof and maximum outward load on the walls: $DC_{max} + EV_{max} + EH_{min} + (LL+IM)_{max} + WA_{max}$
2. Minimum vertical load on the roof and maximum inward load on the walls: $DC_{min} + EV_{min} + EH_{max}$
3. Maximum vertical load on the roof and maximum inward load on the walls: $DC_{max} + EV_{max} + EH_{max} + (LL + IM)_{max}$

Reactions to vertical loads applied to the culvert (earth, water, live load) are assumed to be carried with uniformly distributed reactions applied to the bottom of the bottom slab. Box culverts supported on stiff or rigid subgrades (rock) require additional investigation.

Include fillets in the analysis by increasing the thickness of the members near each corner. Use the moment at the location where the typical section and fillet meet to determine corner reinforcement requirements. Use the shear at d/2 away from the intersection of the typical section and fillet to determine shear adequacy.

The minimum amount of flexural reinforcement in the cross section is 0.2% of the gross area.

12.4 Arch or 3-Sided Structure Design Data

Designs based on the AASHTO Standard Specifications have been assembled for buried precast concrete arch structures. See Standard Detail 5-397.786.

When fill depths are between table values, use the largest steel area on either side of the design fill height.

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

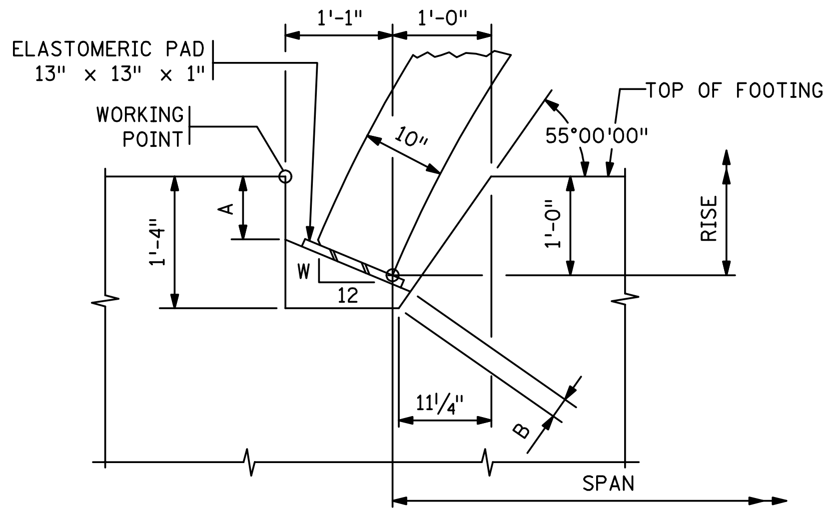
Figures 12.4.1 and 12.4.2 contain standard design information for spans between 24'-0" and 43'-11". For projects with fill heights greater than those listed a custom design is required.

Guidelines for Scour Protection of Arch or 3-Sided Bridge Footings

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

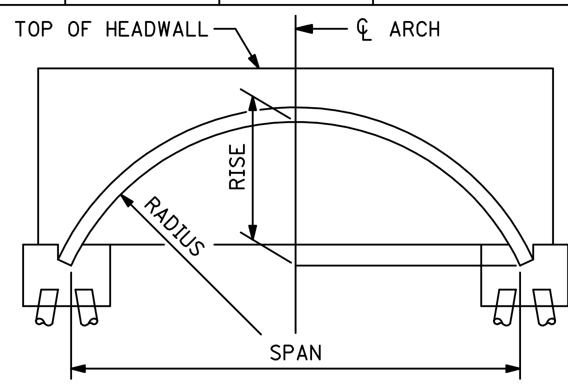
There are several options available for protection of the footings against scour. These options include rock riprap, concrete bottom, piling supported footings, and spread footings keyed into bedrock. The preferred option choice will depend on a number of factors including:

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



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	MAX FILL HEIGHT ③
24'-0"	24'-0"	8'-0"	13'-0"	139	117	16'-0"
25'-4"	25'-3⅝"	10'-0"	13'-0"	188	165	8'-0"
32'-3"	32'-3"	10'-0"	18'-0"	232	202	12'-0"
34'-5"	34'-4⅝"	12'-8"	18'-0"	320	288	6'-0"
40'-3"	40'-2½"	10'-6"	24'-6"	296	258	16'-0"
43'-11"	43'-11⅜"	13'-8"	24'-6"	430	388	8'-0"

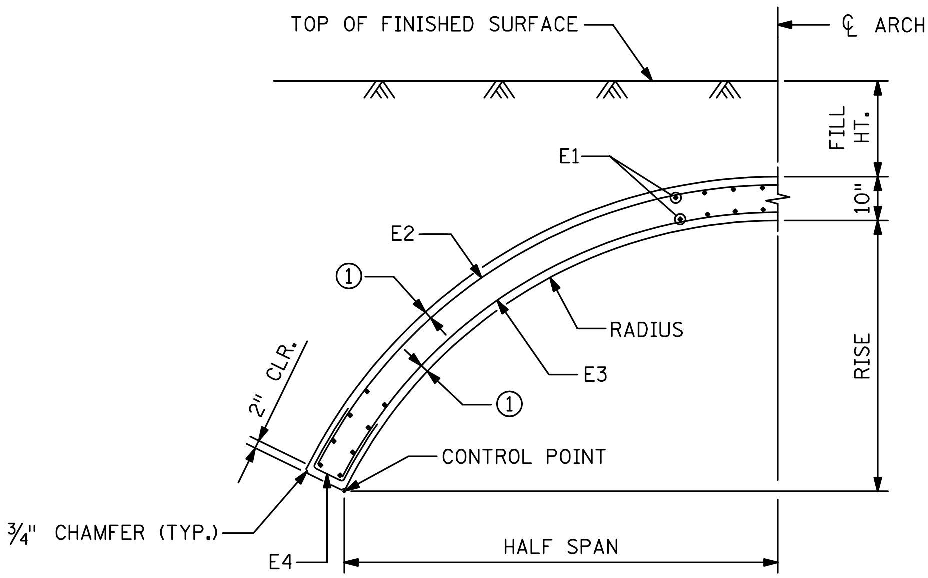
① AREA UNDER ARCH ABOVE A LEVEL LINE AT LOWEST POINT OF ARCH.

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

③ SEE ARCH ELEMENT REINFORCEMENT TABLE ON SHEET XX FOR REINFORCEMENT.

PRECAST CONCRETE ARCH STRUCTURE DESIGN DATA

Figure 12.4.1



HALF END ELEVATION

① 1½" MIN., 2" MAX. CLEAR COVER.

② REQUIRED AREA OF REINF. IF WELDED WIRE FABRIC IS USED.

ARCH ELEMENT REINFORCEMENT		
BAR	SIZE AND MAX. SPG.	As ② in²/ft
E1	NO. 13 @ 1'-0"	0.18
E2	NO. 22 @ 1'-0"	0.56
E3	NO. 22 @ 1'-0"	0.56
E4	NO. 22 @ 1'-0"	0.56

BASIS OF DESIGN:

- AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES
- SOIL W_{SOIL} = 130 lb/ft^2
- CONCRETE f'c = 5.0 ksi
- REINFORCEMENT:
  fy = 60 ksi STEEL FOR REINFORCEMENT BARS
  fy = 65 ksi FOR WELDED WIRE FABRIC REINFORCEMENT
- LATERAL PRESSURE:
  30 P.C.F. MINIMUM
  60 P.C.F. MAXIMUM
- COEFFICIENTS FOR LOAD FACTOR DESIGN:
  DL = 1.3
  LL = 2.167
- HS25 LIVE LOAD

NOTES:

FOR ADDITIONAL DETAILS OF ARCH SECTIONS, SEE BRIDGE STANDARD DETAIL SHEET 5-397.786

MINIMUM FILL HEIGHT TO BE 1'-6" AT EDGE OF PAVEMENT.

PRECAST CONCRETE ARCH STRUCTURE DESIGN DATA

Figure 12.4.2

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. Subcut unstable material below spread footings and replace it with granular backfill or a lean concrete. The maximum depth of subcutting for this purpose is 2'-0". A pile footing should be used if the depth of unstable material below a footing is greater than 2'-0".

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" Class II riprap with 6" granular filter or geotextile filter is required.

Option 1 (Figure 12.4.3, left side)

The riprap may be placed on a slope of 1:2.5 maximum. Cover to the bottom of footing shall be 6'-0" minimum measured perpendicular to the slope. The riprap shall be toed in vertically 2'-0" minimum. The bottom of footing shall be at or below the channel bottom.

Option 2 (Figure 12.4.3, 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'-0" from edge of structure and be toed in vertically a minimum of 2'-0".

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 12.4.4, 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'-0" minimum measured perpendicular to the slope. The bottom of footing shall be 2'-0" minimum below the channel bottom.

Option 2 (Figure 12.4.4, upper right side)
The riprap may be placed horizontally on the channel bottom. Cover to the bottom of footing shall be 6'-0" minimum. The riprap shall extend a minimum of 10'-0" from edge of footing and be toed in vertically a minimum of 2'-0".

Design 2B Scour Protection (Figure 12.4.4, 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' min. beyond the bottom of Area “B”. This toe shall have a minimum thickness of 24". Cover to the bottom of footing shall be 6'-0" minimum measured perpendicular to the slope. The bottom of footing shall also be at or below the bottom of Area “B”.

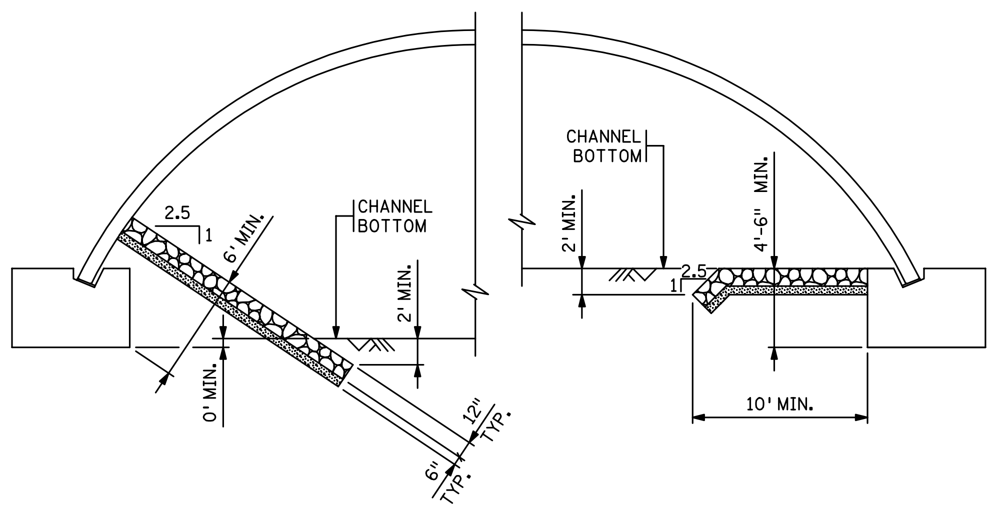
Design 3 Scour Protection
Option 1 (Figure 12.4.5, left side)
Articulated concrete with geotextile backing may be placed on a maximum slope of 1:2.5 with a minimum cover of 4'-6" to bottom of footing measured perpendicular to the articulated concrete. The average velocity for the 100-year flood must be no greater than 7.5 feet per second. For higher velocities, contact the Bridge Office.

Option 2 (Figure 12.4.5, 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 12.4.6, 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 12.4.6, right side)
If footings are on hard bedrock, they shall be keyed in a minimum of 12".

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 inplace 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.



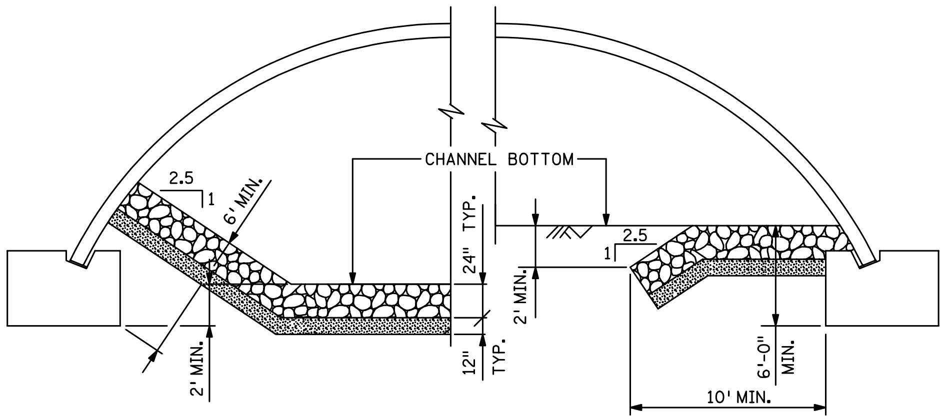
DESIGN 1 SCOUR PROTECTION FOR ARCH OR 3-SIDED BRIDGE

12" CLASS II RIPRAP
6" GRANULAR FILTER OR GEOTEXTILE FILTER

AVERAGE VELOCITY THROUGH STRUCTURE:
$V_{100} \leq 3$ f.p.s.
$V_{500} \leq 5$ f.p.s.

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

Figure 12.4.3

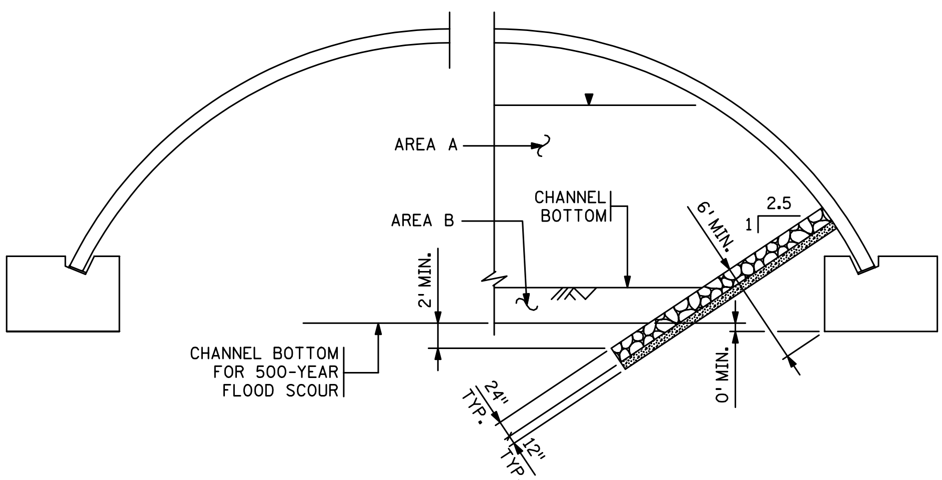


DESIGN 2A SCOUR PROTECTION FOR ARCH OR 3-SIDED BRIDGE

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

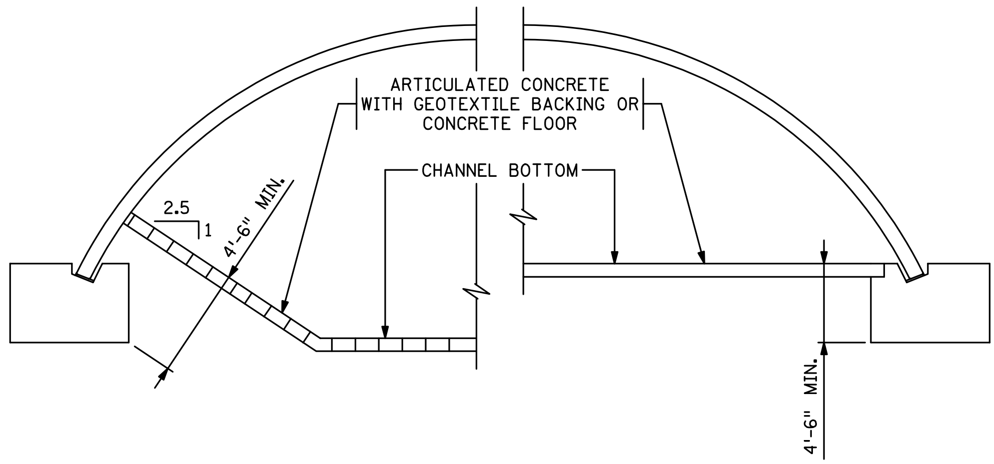
AVERAGE VELOCITY THROUGH STRUCTURE:
$V_{100} \leq 5.5$ f.p.s.
$V_{500} \leq 6.5$ f.p.s.

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 12.4.4

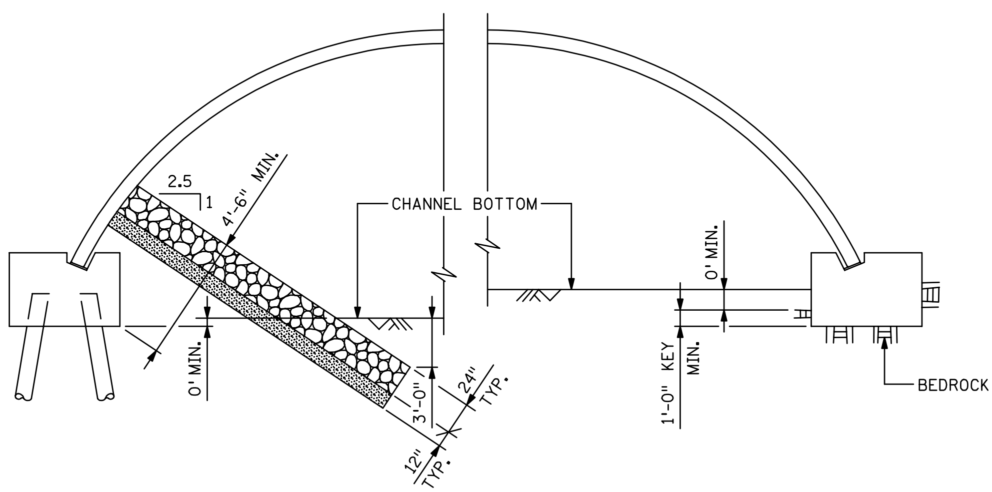


DESIGN 3 SCOUR PROTECTION FOR ARCH OR 3-SIDED BRIDGE

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

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

Figure 12.4.5



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 12.4.6

12.5 Design Criteria for Long-Span Corrugated Steel Structures

Standard designs for long span corrugated steel structures have been assembled. The standard designs are based on the AASHTO Standard Specifications and additional criteria specified by Mn/DOT. The following criteria shall be used for the design of long span corrugated steel structures, identified as horizontal ellipse, low profile arch and high profile arch.

Section 12 of the 17th Edition of the AASHTO Standard Specifications for Highway Bridges should be modified as follows:

1. Delete the values of I. Top Arc of Table 12.7.2A and substitute the following:

. TOP ARC GAGE-MINIMUM THICKNESS IN INCHES (6X2) CORRUGATED STEEL PLATES		MAXIMUM TOP RADIUS IN FEET
10	0.138	15
8	0.168	17
7	0.188	18
5	0.218	20
3	0.249	23
1	0.280	24

   The minimum steel thickness to be used is 10 gage

2. Delete values of II. Minimum cover in feet of Table 12.7.2A and add the following:

GAGE- STEEL THICKNESS (IN)		TOP RADIUS (FT)							
		11	**13**	**15**	**17**	**18**	**20**	**23**	**24**
10	0.138	3.0	3.0	3.0					
8	0.168	3.0	3.0	3.0	3.0				
7	0.188	3.0	3.0	3.0	3.0	3.0			
5	0.218	3.0	3.0	3.0	3.0	3.0	3.0		
3	0.249	3.0	3.0	3.0	3.0	3.0	3.0	4.0	
1	0.280	3.0	3.0	3.0	3.0	3.0	3.0	4.0	4.0

   Minimum cover is defined as the cover from the top of subgrade section at the shoulder P.I to the top of the structure.

3. Delete III Geometric Limits of Table 12.7.2A and substitute the following:
   a. Maximum Plate Radius – 24'-0"
   b. Maximum Central Angle of Top Arc = 80 °
   c. Minimum Ratio, Top Radius to Side Radius = 2.0

d. Maximum Ratio, Top Radius to Side Radius:
   For structures without concrete floors 3.5
   For structures with concrete floors 3.0

NOTE: On high profile arch structures, the upper side radius equals the side radius.

e. Maximum ratio of span to top rise for low profile arches = 3.2
f. Maximum ratio of span to $^1/_2$ the top rise for horizontal ellipses = 3.2

4. Revise the following factors in Section 12 for maximum depths of cover as follows:
   k = .22, soil stiffness coefficient for good side fill material
   E' = Modulus of Passive soil (side fill) resistance: 1400 psi
   Safety factors used: Longitudinal test seam strength = 3.33

Long span structures shall be designed in accordance with the following additional criteria:

1. Maximum spans for long span corrugated steel structures shall not exceed 20'-0" with a minimum of 8'-0" between units.
2. Low profile arch and high profile arch structures must have concrete floors or equivalent, to prevent footing scour, unless footings are placed on bedrock.
3. At each structure site the following soils information and durability investigation are required:
   a. Soils survey, design, and engineering report shall be made in accordance with 2-8.02 of the Mn/DOT Road Design Manual.
   b. Durability investigation and recommendations shall be made in accordance with 5-294.343K of the Mn/DOT Drainage Manual.
   c. Copies of soils survey and durability investigation, etc. shall be furnished to the Hydraulic Unit.

Long span corrugated steel structure designs shall be based on:

1. Concrete footings of low profile arch and high profile arch structures are designed on the basis of 4 tons per square foot allowable soil stress and fill heights of approximately 30'-0" fill heights above the top of the structure. Modification of the footing widths for lesser fill heights or poorer soil conditions is necessary. The minimum footing width is 1'-6".
2. The maximum height of fill above structures is 30'-0" unless specially designed by the Bridge Office.

Additional requirements on long span corrugated steel structures includes:

1. These structures are not be permitted to be used as vehicle underpasses.
2. All designs utilizing Federal, State, or State Aid funds will be reviewed by Mn/DOT. The review will verify compliance with AASHTO Specifications, Mn/DOT Specifications, Mn/DOT detail sheets, and Mn/DOT design guidelines.
3. Standard Plan Sheets developed by Mn/DOT 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. Detail plans that include structural computations and special provisions, shall be certified by a qualified professional engineer registered in the State of Minnesota.
6. Mn/DOT Projects
    a. When the Department of Transportation proposes the use of these structures, the Bridges and Structures 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 of a long span corrugated steel structure, 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 Mn/DOT State Aid Office.
    b. Final funding approval by the State Aid Office to include approval of the design detail plan and hydraulic characteristics by the Office of Bridges and Structures.
8. Scour protection for long span corrugated steel structures shall be the same as those for precast concrete arch structures (see Section 12.4).

The geometry of standard low profile arches with compaction wings is provided in the following table.

MAX SPAN	BOTTOM SPAN	TOP RISE	TOTAL RISE	TOP RADIUS (R_T)	SIDE RADIUS (R_S)	ANGLE BELOW HORIZ. Δ	STEP V (3)	W APPROX.	SLOPE COLLAR LENGTH	APPROX. AREA (SQ. FT.)
13'-6"	13'-3"	4'-9"	5'-9"	8'-6"	3'-7"	15° 37'	2'-4"	5'-6"	9'-11"	63
15'-9"	15'-2"	5'-2"	6'-2"	10'-3"	3'-7"	15° 37'	2'-7"	6'-7"	10'-10"	78
17'-2"	16'-11"	5'-5"	6'-5"	11'-5"	3'-7"	15° 37'	2'-10"	7'-4"	11'-4"	88
17'-2"	16'-11"	6'-0"	7'-0"	10'-10"	4'-6"	12° 29'	2'-8"	7'-0"	12'-8"	96

NOTES:

1. The Foundation Engineer will determine the suitability of the foundation material under the structure, and will provide recommendations relative to subcutting as required.
2. The side excavation limits may be increased as recommended by the Foundation Engineer.
3. See Bridge Detail Standard Plan Sheet 5-397.744 (Side Elevation) for step location.
4. Footing width at collar to be 4'-9", see 5-397.744 for footing sizes.

12.6 10'x10' Precast Concrete Box Culvert Design Example

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

The inside dimensions of the box culvert are 10'-0" by 10'-0". The fill height above the culvert is 6'-0". A typical section of the culvert is shown in Figure 12.8.1.

The following material and design parameters are used for this example:

Unit Weights	Reinforced Concrete	0.150 k/ft^3
	Water	0.062 k/ft^3
	Soil (computed)	0.130 k/ft^3
Concrete	Compressive Strength, f'_c	5.0 ksi
	Top Slab Thickness	9 in
	Bottom Slab Thickness	10 in
	Wall Thickness	8 in
	Concrete Cover	1.5 in
	Crack Control, Z	130 kip/in
Steel Reinforcement	Modulus of Elasticity, E_s	29,000 ksi
	Yield Strength, f_y	65 ksi
	Maximum Wire Size	W20
	Minimum Wire Spacing	2 in
	Maximum Wire Spacing	4 in (transv. wires) 8 in (longit. Wires)

A. Earth Pressure Loads [3.11.5 –3.11.7, 12.11.2]

The weight of fill on top of the culvert produces vertical earth pressure (EV). To account for the variability of the fill on top of box culverts a unit weight of 0.130 kcf is used.

Vertical earth pressures are modified to account for soil structure interaction. The interaction or the amount of earth load carried by the culvert is dependent on the construction site and the methods used during construction. Culverts placed in trench conditions need to carry less vertical load than those constructed in embankment conditions (the consolidated material in the adjacent trench walls is typically stiffer than new embankment material.). The design example assumes the box culvert will be constructed with embankment conditions.

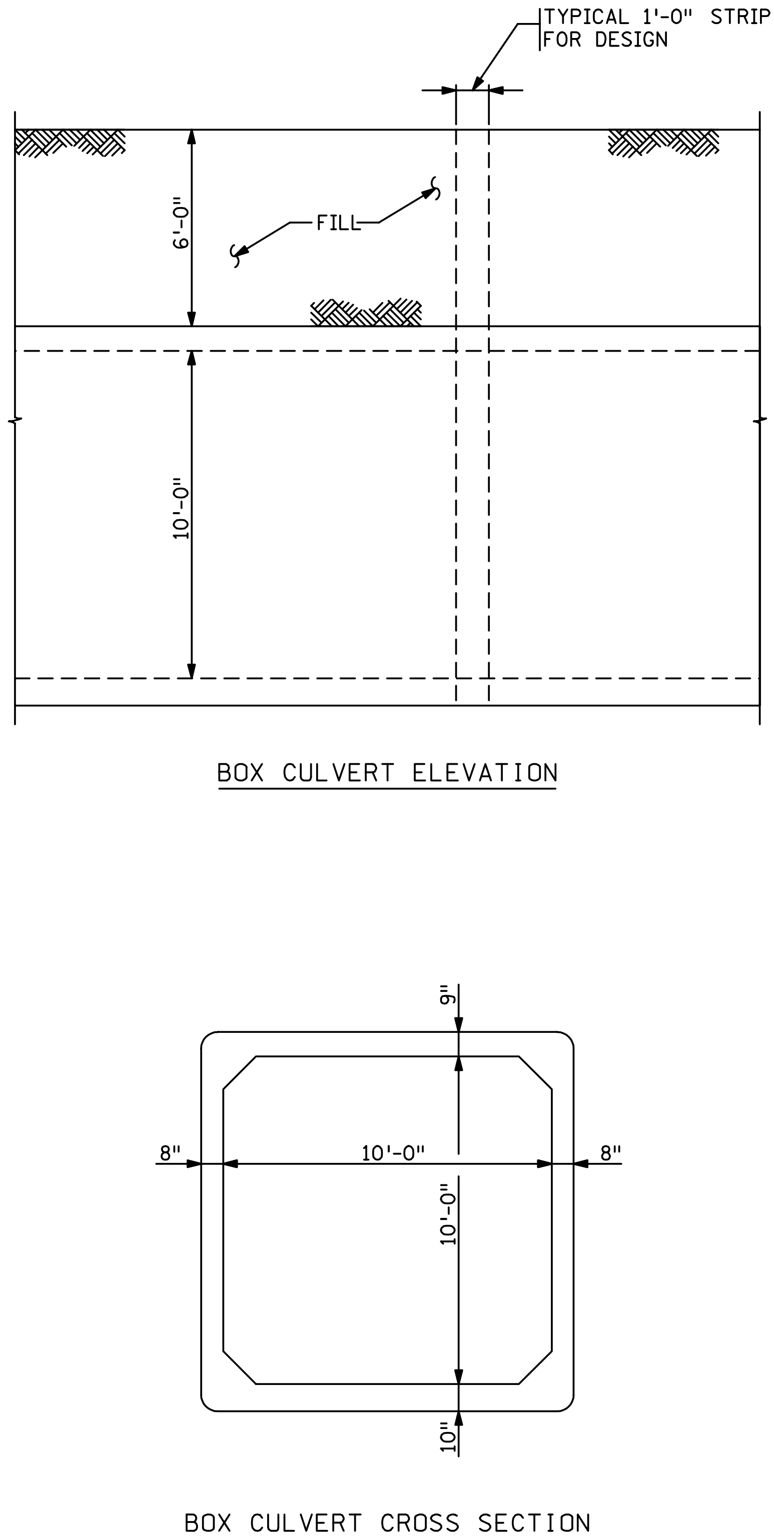


Figure 12.6.1 - Design Example Geometry

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

[12.11.2.2.1-2]

$$F_e = 1 + 0.20 \cdot \left[\frac{H}{B_c}\right] = 1 + 0.20 \cdot \left[\frac{6}{0.67 + 10.0 + 0.67}\right] = 1.11 \leq 1.15$$

[12.11.2.2.1-1]

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

$$EV = W_E = F_e \cdot \gamma \cdot D_E = 1.11 \cdot 0.130 \cdot 6.0 = 0.866 \text{ ksf}$$

The lateral earth pressure (EH) on the culvert is found using the equivalent fluid method. An at-rest, maximum equivalent-fluid unit weight of 0.060 kcf is used.

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

$$EH = \gamma_{eq} \cdot Z = 0.060 \cdot 6.0 = 0.36 \text{ ksf}$$

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

$$EH = 0.060 \cdot \left[6.0 + \frac{9}{12} + 10.0 + \frac{10}{12}\right] = 1.05 \text{ ksf}$$

Figure 12.8.2 illustrates the vertical and lateral earth pressures applied to the box culvert.

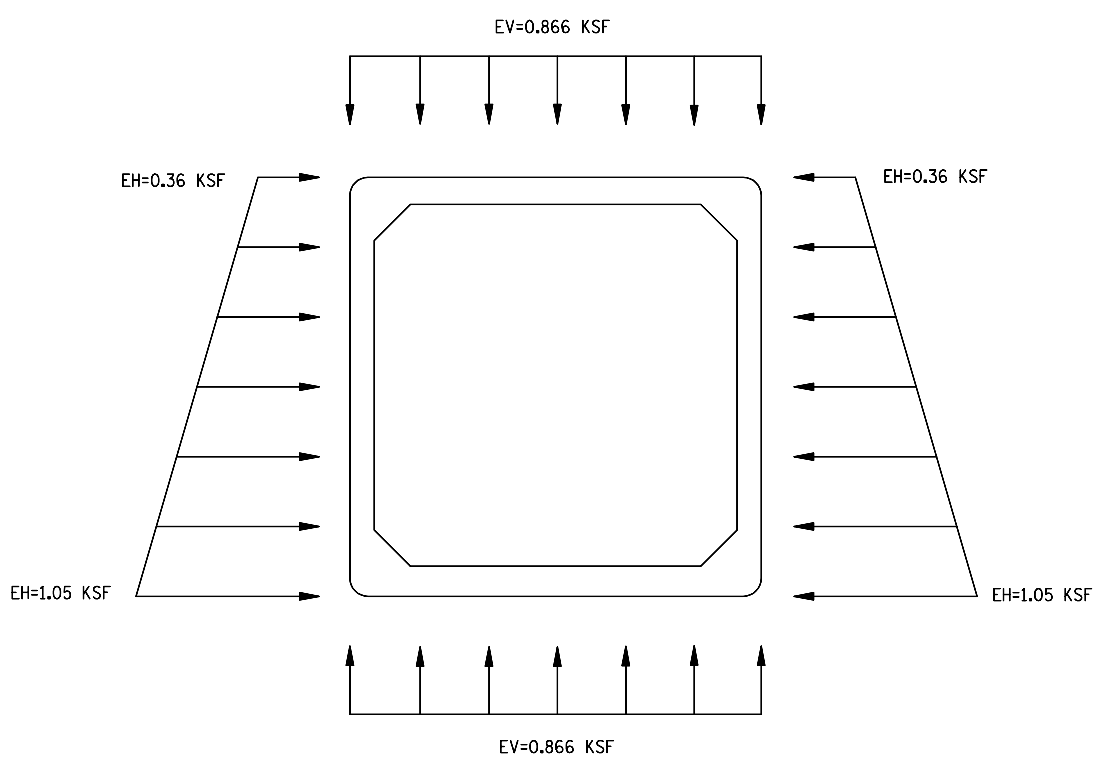


Figure 12.6.2 – Earth Loads and Selfweight Reaction

B. 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 top of the culvert, the lateral water pressure is:

WA_{top} = 0 .00 ksf

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

$WA_{bot} = \gamma \cdot z = 0.062 \cdot 10 = 0.62$ ksf

The vertical pressure of 0.62 ksf applied to the bottom of the culvert from water is assumed to pass directly through the bottom slab into the subgrade. The water load is illustrated in Figure 12.8.3.

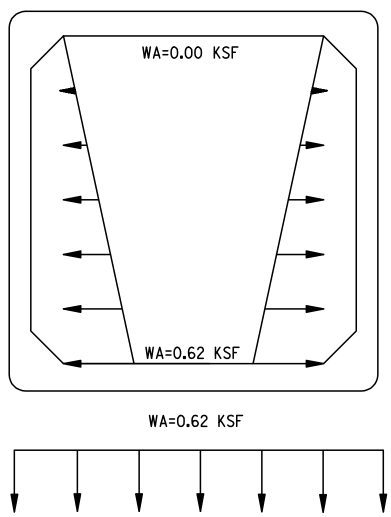


Figure 12.6.3 – Water Loads

C. Live Load [3.6.1.3.3]

The approximate strip method is used for the design with the 1'-0" wide design strip oriented parallel to the direction of traffic (longitudinal direction.) The design live loads include the HL-93 truck, lane and tandem loads.

[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. The deeper the fill, the greater the reduction. For strength and service limit states:

$$IM = 33 \cdot [1.0 - 0.125 \cdot D_E] = 33 \cdot [1.0 - 0.125 \cdot 6.0] = 8.3\%$$

For the fatigue check:

$$IM = 15 \cdot [1.0 - 0.125 \cdot D_E] = 15 \cdot [1.0 - 0.125 \cdot 6.0] = 3.8\%$$

[3.6.1.2.5]
[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 0.57 feet 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" and a length of 10".

Using the distances between wheel lines, axles, and design lanes, 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. After which, the sum of the weights of the wheels is used to determine the intensity of the live load. The lane loading is treated slightly differently from the wheel loads. Wheel loads are assumed to distribute both longitudinally and transversely. Lane loads are assumed to be very long and consequently only distribute transversely.

To determine the live load that should be carried into the structural analysis, use multiple presence factors. One lane loaded for strength and service limit states uses a multiple presence factor of 1.20. For two lanes loaded use 1.00. No multiple presence factor should be used for the fatigue limit state.

Begin by determining the intensities of the different live load components.

A single HL93 truck axle produces a live load pressure of:

$$w_{LL} = \frac{2 \cdot P_W \cdot MPF}{(\text{influence area})} = \frac{2 \cdot 16 \cdot 1.2}{14.58 \cdot 7.75} = 0.339 \text{ ksf}$$

Two HL93 truck axles adjacent to each other (4' apart) produce:

$$w_{LL} = \frac{4 \cdot P_W \cdot MPF}{(\text{influence area})} = \frac{4 \cdot 16 \cdot 1.0}{24.58 \cdot 7.75} = 0.336 \text{ ksf}$$

A single lane load produces:

$$w_{LL} = \frac{W_{lane} \cdot \text{width} \cdot MPF}{(\text{influence width})} = \frac{0.064 \cdot 10 \cdot 1.20}{16.9} = 0.045 \text{ ksf}$$

A single tandem vehicle produces:

$$w_{LL} = \frac{4 \cdot P_w \cdot MPF}{(\text{influence area})} = \frac{4 \cdot 12.5 \cdot 1.20}{14.58 \cdot 11.75} = 0.350 \text{ ksf}$$

Two tandem vehicles adjacent to each other (4' apart) produce:

$$w_{LL} = \frac{8 \cdot P_w \cdot MPF}{(\text{influence area})} = \frac{8 \cdot 12.5 \cdot 1.0}{24.58 \cdot 11.75} = 0.346 \text{ ksf}$$

Figure 12.6.4 illustrates the different live loads.

The tandem vehicle produces a live load intensity slightly larger than that of the HL-93 truck axle and also larger than the double tandem load. It also has a larger influence area than the truck axle. Use lane loading and the single tandem to design for the Strength and Service limit states. A single HL-93 truck axle is used for the fatigue check.

D. Select Applicable Load Combinations and Load Factors [3.4.1]

The load factors for the different load components vary with the limit state being considered. For this example the following load factors are used:

	Strength		**Service**		**Fatigue**	
	Min.	**Max.**	**Min.**	**Max.**	**Min.**	**Max.**
DC	0.9	1.25	1.0	1.0	----	----
EV	0.9	1.3	1.0	1.0	----	----
EH	0.65	1.35	0.5	1.0	----	----
LL	1.75	1.75	1.0	1.0	0.75	0.75
WA	1.0	1.0	1.0	1.0	----	----

For both the strength and service limit states, three load cases will be considered. The load cases correspond to:

1. Maximum vertical load on the roof and maximum outward load on the walls: $DC_{max} + EV_{max} + EH_{min} + (LL+IM)_{max} + WA_{max}$
2. Minimum vertical load on the roof and maximum inward load on the walls: $DC_{min} + EV_{min} + EH_{max}$
3. Maximum vertical load on the roof and maximum inward load on the walls: $DC_{max} + EV_{max} + EH_{max} + (LL+IM)_{max}$

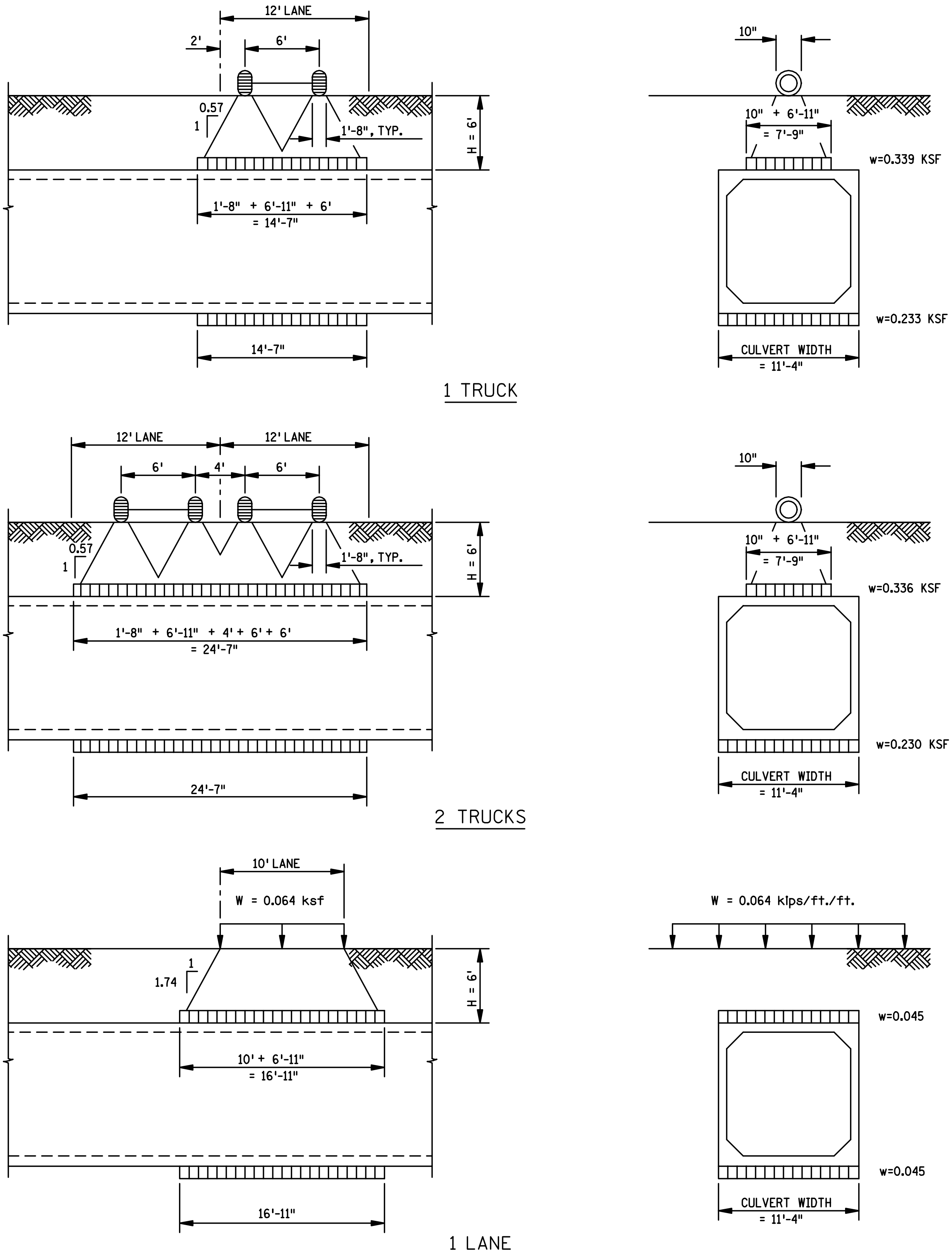


Figure 12.6.4 - Live Load Distribution

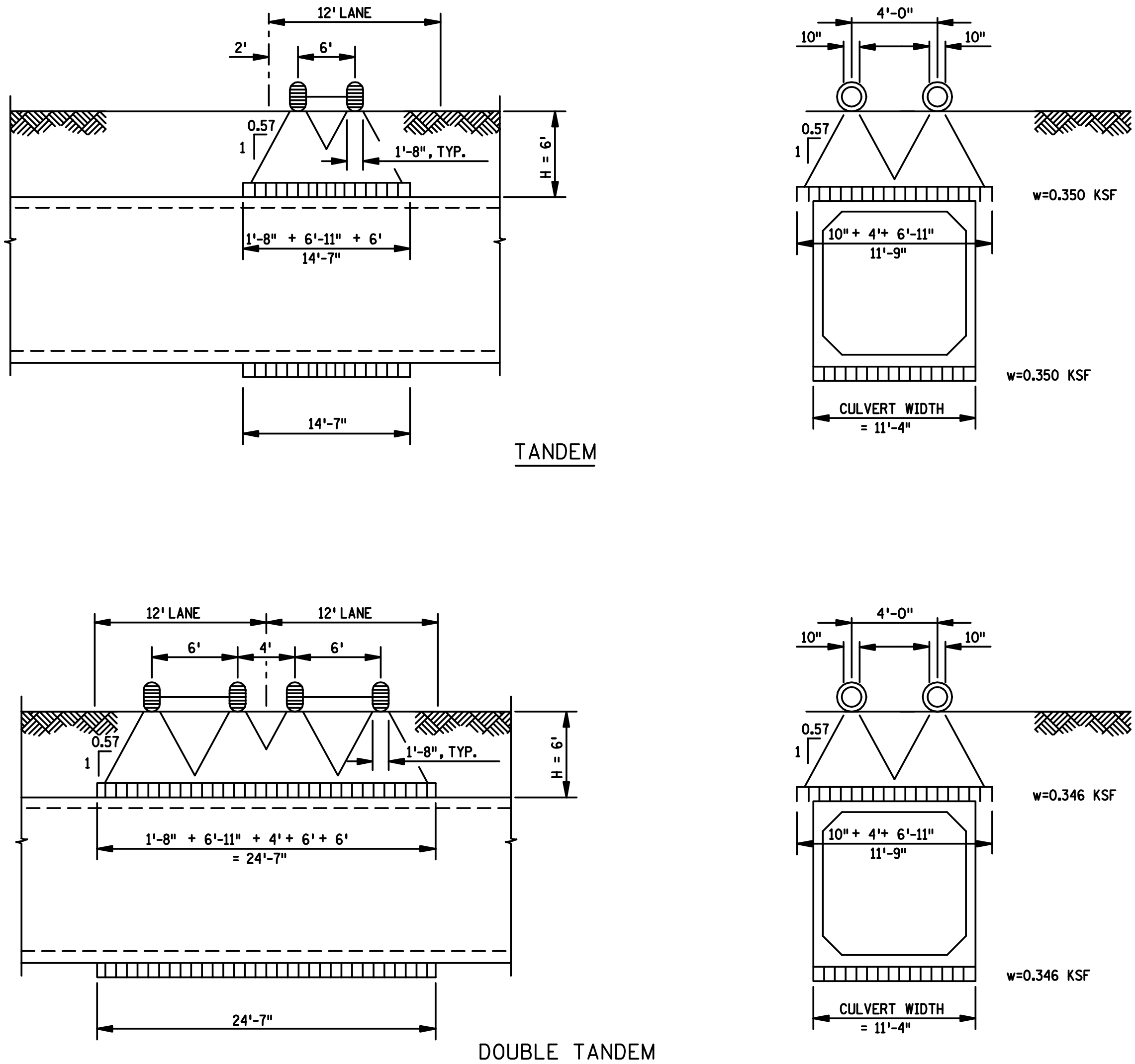


Figure 12.6.4 - Live Load Distribution (cont.)

The Strength I load combinations are:

1. $U = 1.0 \cdot [1.25 \cdot DC + 1.3 \cdot EV + 0.65 \cdot EH + 1.75 \cdot (LL + IM) + 1.0 \cdot WA]$
2. $U = 1.0 \cdot [0.9 \cdot DC + 0.9 \cdot EV + 1.35 \cdot EH]$
3. $U = 1.0 \cdot [1.25 \cdot DC + 1.3 \cdot EV + 1.35 \cdot EH + 1.75 \cdot (LL + IM)]$

The Service I load combinations are:

1. $U = 1.0 \cdot [DC + EV + 0.5 \cdot EH + (LL + IM) + WA]$
2. $U = 1.0 \cdot [DC + EV + EH]$
3. $U = 1.0 \cdot [DC + EV + EH + (LL + IM)]$

One load case is considered for the Fatigue limit state:

1. $U = 0.75 \cdot (LL + IM)$ (range)

E. Summary of Analysis Results

A structural analysis was performed using a standard commercial matrix-analysis program. The bottom slab of the box culvert was assumed rigid compared to the subgrade. Reactions to vertical loads applied to the culvert (earth, water, live load) were assumed to be carried with uniformly distributed reactions applied to the bottom slab. Box culverts supported on stiff or rigid subgrades (rock) would require additional investigation. The fillets were included in the analysis by increasing the thickness of members near each corner. The internal forces at several locations of the box are presented in Tables 12.8.1 and 12.8.2. The forces presented at top, bottom, or end locations are at the location where the typical section and fillet meet. The first table lists the forces associated with each load component. The second table contains the forces at the same locations for the various load combinations.

Table 12.6.1a Structural Analysis Results (unfactored, kip-in)

	Moment					
	DC	**EV**	**EH**	**WA**	**Tandem LL+IM**	**Fatigue (range)**
Sidewall Bottom	-20.0	-36.9	-4.74	-0.06	-16.17	-8.47
Sidewall Center	-12.2	-42.9	63.1	-27.7	-18.78	-11.84
Sidewall Top	-4.36	-48.8	-4.57	4.54	-21.39	-15.22
Top Slab Center	-18.3	-96.9	50.8	-20.8	-42.43	-29.19
Top Slab End	-7.52	-13.8	50.8	-20.8	-6.03	-1.85
Bottom Slab Center	43.4	113	-69.9	32.4	49.49	29.18
Bottom Slab End	5.73	29.8	-69.9	32.4	13.07	8.68

Table 12.6.1b Structural Analysis Results (unfactored, kip)

	Shear					Thrust				
	DC	**EV**	**EH**	**WA**	**Tandem LL+IM**	**DC**	**EV**	**EH**	**WA**	**Tandem LL+IM**
Sidewall Bottom	0.16	-0.12	3.14	-1.35	-0.05	1.74	4.62	0.00	0.00	2.03
Sidewall Center	0.16	-0.12	-0.16	0.21	-0.05	1.34	4.62	0.00	0.00	2.03
Sidewall Top	0.16	-0.12	-2.50	0.96	-0.05	0.94	4.62	0.00	0.00	2.03
Top Slab Center	0.00	0.00	0.00	0.00	0.00	-0.16	0.12	3.08	-0.99	0.05
Top Slab End	-0.45	-3.46	0.00	0.00	-1.52	-0.16	0.12	3.08	-0.99	0.05
Bottom Slab Center	0.00	0.00	0.00	0.00	0.00	0.16	-0.12	4.54	-2.13	-0.05
Bottom Slab End	-1.57	-3.46	0.00	0.00	-1.52	0.16	-0.12	4.54	-2.13	-0.05

Table 12.6.1.2a Strength and Fatigue Load Combinations (kips, kip-in)

	Case 1			Case 2			Case 3			Fatigue
	Moment	Shear	Thrust	Moment	Shear	Thrust	Moment	Shear	Thrust	Moment
Sidewall Bottom	-108	0.45	12.1	-57.6	4.28	5.72	-111	4.18	12.1	-6.35
Sidewall Center	-94.5	0.05	11.6	35.6	-0.18	5.36	-22.6	-0.28	11.6	-8.88
Sidewall Top	-109	-0.73	11.1	-54.1	-3.34	5.00	-117	-3.44	11.1	-11.4
Top Slab Center	-220	0.00	1.07	-35.1	0.00	4.13	-163	0.00	4.23	-21.9
Top Slab End	-26.9	-8.04	1.07	49.4	-3.52	4.13	29.5	-8.04	4.23	-1.39
Bottom Slab Center	285	0.00	0.76	46.3	0.00	6.17	204	0.00	6.07	21.9
Bottom Slab End	58.6	-9.44	0.76	-62.4	-4.53	6.17	-22.8	-9.44	6.07	6.51

Table 12.6.1.2b Service Load Combinations (kips, kip-in)

	Case 1			Case 2			Case 3		
	Moment	Shear	Thrust	Moment	Shear	Thrust	Moment	Shear	Thrust
Sidewall Bottom	-77.5	0.02	8.62	-61.7	3.18	6.36	-79.8	3.12	8.62
Sidewall Center	-72.3	0.11	8.22	8.03	-0.12	5.96	-13.0	-0.17	7.85
Sidewall Top	-74.9	-0.31	7.82	-57.8	-2.46	5.56	-81.7	-2.52	7.82
Top Slab Center	-158	0.00	0.57	-64.4	0.00	3.04	-112	0.00	3.11
Top Slab End	-23.5	-5.61	0.57	29.5	-3.91	3.04	22.8	-5.61	3.11
Bottom Slab Center	209	0.00	0.12	86.4	0.00	4.58	142	0.00	4.52
Bottom Slab End	47.7	-6.73	0.12	-34.4	-5.03	4.58	-19.7	-6.73	4.52

The values in Tables 12.8.1 and 12.8.2 include dynamic load allowance and multiple presence factors.

F. Investigate Strength Limit State for Flexure [5.7.2.2] [5.7.3.2]

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

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

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:

$$M_u = \phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot \left[d - \frac{A_s \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b} \right]$$

Inserting values for f_y, b, and ϕ:

$$M_u = 1.0 \cdot A_s \cdot 65 \cdot \left[d - \frac{A_s \cdot 65}{2 \cdot 0.85 \cdot 5.0 \cdot 12.0} \right]$$

Which becomes:

$$M_u = 65 \cdot A_s \cdot [d - 0.637 \cdot A_s]$$

And eventually:

$$41.41 \cdot A_s^2 - 65 \cdot d \cdot A_s + M_u = 0$$

Solving the quadratic equation for A_s:

$$A_s = \frac{(65 \cdot d) - \sqrt{4225 \cdot d^2 - 165.6 \cdot M_u}}{82.82}$$

Sidewall:

Size the reinforcement assuming "d" dimensions based on a W12 wire (d_w = 0.391 in) and a clear cover of $1^1/_2$". The trial "d" of the sidewall is:

$$d = (\text{thickness}) - (\text{clearcover}) - \left[\frac{d_w}{2} \right] = 8 - 1.5 - \frac{0.391}{2} = 6.30 \text{ in}$$

The peak moment for tension on the outside face is 117 k-in (top, Strength I: Case 3). The required area of steel is 0.29 in^2/ft.

The peak moment for tension on the inside face is 35.6 k-in (center, Strength I: Case 2). The required area of steel is 0.09 in^2/ft.

Top Slab:

For the top slab "d" is:

$$d = 9 - 1.5 - \frac{0.391}{2} = 7.30 \text{ in}$$

The peak moment for tension on the outside face is 49.4 k-in (end, Strength I: Case 2). The required area of steel is 0.11 in^2/ft.

The peak moment for tension on the inside face is 220 k-in (center, Strength I: Case 1). The required area of steel is 0.48 in^2/ft.

Bottom Slab:

$$d = 10 - 1.5 - \frac{0.391}{2} = 8.30 \text{ in}$$

The peak moment for tension on the outside face is 62.4 k-in (end, Strength I: Case 2). The required area of steel is 0.12 in^2/ft.

The peak moment for tension on the inside face is 285 k-in (center, Strength I: Case 1). The required area of steel is 0.55 in^2/ft.

G. 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 yield strength of the reinforcement, the concrete cover, and the area of concrete assumed to participate with a reinforcing bar or wire. The "Z" parameter is varied according to the use of the concrete structure. Mn/DOT uses a Z value of 130 for box culvert design.

[5.7.3.4-1]

$$f_s \le f_{sa} = \frac{Z}{(d_c \cdot A)^{0.33}} \le 0.6 \cdot f_y$$

The service limit state stress in the rebar is found using cracked section analysis methods.

Begin by finding the modular ratio between reinforcing steel and concrete.

[5.4.2.4 & 5.7.1]

$$n = \frac{E_s}{E_c} = \frac{29000}{33000(.145)^{1.5}\sqrt{5.0}} = 7.1 \qquad \text{Use } n=7$$

Sidewall:

Try W12 wire at a 4" spacing, A_s = 0.36 in^2/ft, d = 6.30 in,
d_c = 1.70 in
Outside face tension; Max. service moment M = 81.7 k-in

Area of concrete assumed to participate with the reinforcement:

$$A = \frac{2 \cdot d_c \cdot b}{N} = \frac{2 \cdot 1.70 \cdot 12}{12/4} = 13.60 \text{ in}^2$$

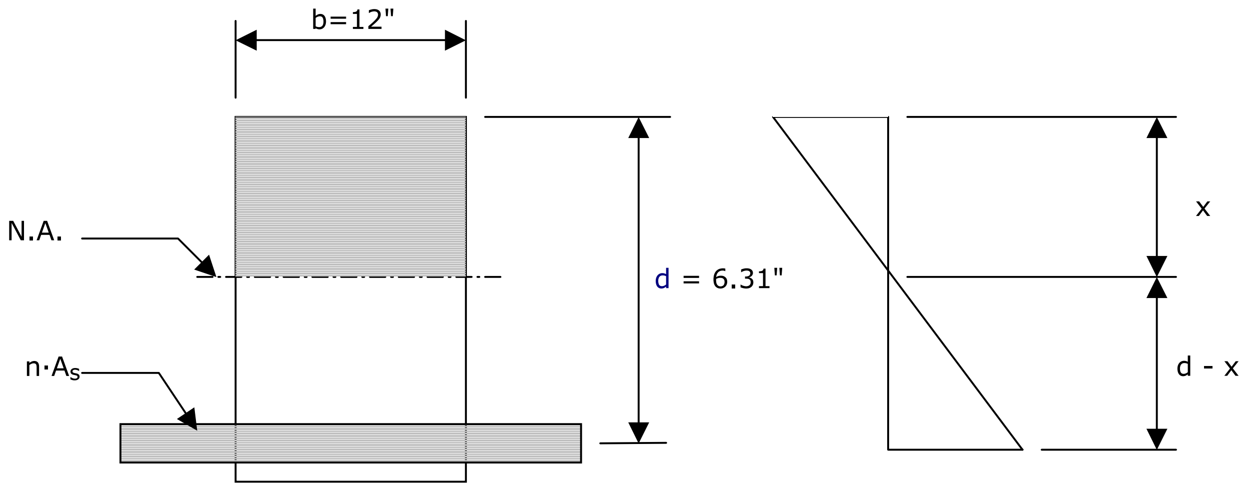
The maximum permitted stress in the reinforcement is:

$$f_{sa} = \frac{Z}{(d_c \cdot A)^{0.33}} = \frac{130}{(1.70 \cdot 13.60)^{0.33}} = 46.1 \ge 0.6 \cdot 65 = 39.0 \text{ ksi}$$

Begin by computing the transformed area of the reinforcement.

$$n \cdot A_s = 7 \cdot 0.36 = 2.52 \text{ in}^2$$

Find the location of the neutral axis:



$$\frac{1}{2} \cdot b \cdot x^2 = n \cdot A_s \cdot (d - x)$$

$$\frac{1}{2} \cdot 12 \cdot x^2 = 2.52 \cdot (6.30 - x) \quad \text{solving, } x = 1.43 \text{ in}$$

Next, find the internal lever arm:

$$j \cdot d = d - \frac{x}{3} = 6.30 - \frac{1.43}{3} = 5.82 \text{ in}$$

The stress in the reinforcement is:

$$f_s = \frac{M}{A_s \cdot j \cdot d} = \frac{81.7}{0.36 \cdot 5.82} = 39.0 = f_{sa} = 39.0 \text{ ksi}$$ <u>OK</u>

Similar calculations were made for the sidewall inside face and both faces of the top slab and the bottom slab. See Table 12.6.3 for calculated values at other locations.

H. Check Fatigue
[5.5.3]
[Table 3.4.1-1]
[3.6.2.1-3.6.2.2]

Mn/DOT practice is to perform fatigue check calculations for the design of box culverts. Typically with fill heights over 2'-0", fatigue will not govern the design.

Calculate the stress in the reinforcement due to the fatigue. Assume a cracked cross section and neglect the benefit of axial compression.

Sidewall:

Outside face M=11.4 k-in, A_s=0.36 in^2/ft
j·d= 5.82 in, f_s=5.44 ksi

Top Slab:

Inside face M=21.9, A_s=0.60 in^2/ft
j·d = 6.61 in, f_s = 5.52 ksi

Bottom Slab:

Inside face M=21.9 k-in, A_s= 0.72 in^2/ft
j·d = 7.51 in, f_s = 4.05 ksi

[5.5.3.2]

The general equation for allowable fatigue stress in reinforcement is:

$$f_f = 21 - 0.33 \cdot f_{min} + 8 \cdot \frac{r}{h}$$

Use the default value for the ratio of base radius to height of rolled on transverse deformations for the reinforcement:

$$\frac{r}{h} = 0.3$$

Use the Service I: Case 2 moment to compute the minimum stress in the sidewall reinforcement. Use the Service I: Case 1 moment without live load to compute the minimum stress in the slab reinforcement.

Sidewall:

$$f_{min} = \frac{M}{A_s \cdot j \cdot d} = \frac{57.8}{0.36 \cdot 5.82} = 27.6 \text{ ksi}$$

$$f_f = 21 - 0.33 \cdot 27.6 + 8 \cdot 0.3 = 15.1 > 5.44 \text{ ksi}$$ <u>OK</u>

Top slab:

$$f_{min} = \frac{89.8}{0.60 \cdot 6.61} = 22.6 \text{ ksi}$$

$$f_f = 21 - 0.33 \cdot 22.6 + 8 \cdot 0.3 = 15.9 > 5.47 \text{ ksi}$$ <u>OK</u>

Bottom Slab:

$$f_{min} = \frac{121.4}{0.72 \cdot 7.51} = 22.5 \text{ ksi}$$

$$f_f = 21 - 0.33 \cdot 22.5 + 8 \cdot 0.3 = 16.0 > 4.03 \text{ ksi}$$ <u>OK</u>

I. 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 reinforcement:

$$A_s = 0.002 \cdot [\text{thickness}] \cdot [\text{width}] = 0.002 \cdot 8 \cdot 12 = 0.19 \ \text{in}^2/\text{ft}$$

Minimum top slab reinforcement:

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

Minimum bottom slab reinforcement:

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

For cast-in-place culverts the minimum reinforcement check (LRFD Article 5.7.3.3.2) is satisfied if 133% of the reinforcement necessary for strength is provided.

J. Check Maximum Reinforcement Limit [5.7.3.3.1]

The maximum reinforcement check is satisfied if:

$$\frac{c}{d} \le 0.42$$

For $f'_c = 5.0 \text{ ksi}, \beta_1 = 0.80$

Sidewall:

Outside face $c = \dfrac{a}{\beta_1} = 0.57 \text{ in}$ $\dfrac{c}{d} = \dfrac{0.57}{6.30} = 0.09$ <u>OK</u>

Top Slab:

Inside face $c = 0.96 \text{ in}$ $\dfrac{c}{d} = \dfrac{0.96}{7.25} = 0.13$ <u>OK</u>

Bottom Slab:

Inside face $c = 1.15 \text{ in}$ $\dfrac{c}{d} = \dfrac{1.15}{8.26} = 0.14$ <u>OK</u>

Table 12.6.3 Flexural Design Calculation Summary

		Sidewall		Top Slab		Bottom Slab	
		Inside	**Outside**	**Inside**	**Outside**	**Inside**	**Outside**
Strength	Moment (k-in)	35.6	112.6	210.9	49.4	274.7	55.9
	Assumed d (in)	6.30	6.30	7.30	7.30	8.30	8.30
	Req'd steel area (in^2)	0.09	0.29	0.48	0.11	0.55	0.12
Service	Moment (k-in)	8.0	79.2	153.1	22.8	203.4	46.2
	Assumed Wire Size, Spacing (in)	W4 @ 4"	W12 @ 4"	W20 @ 4"	W4 @ 4"	W18 @ 3"	W4 @ 3"
	A_S (in^2/ft)	0.12	0.36	0.60	0.12	0.72	0.12
	d (in)	6.39	6.30	7.25	7.39	8.26	8.39
	A (in^2)	12.88	13.60	14.00	12.88	11.04	9.66
	f_{sa} (ksi)	39.0	39.0	39.0	39.0	39.0	39.0
	jd (in)	6.10	5.82	6.61	7.07	7.51	8.00
	f_s (ksi)	10.9	39.0	38.6	26.9	37.4	36.1
Fatigue	Moment range (k-in)	-	11.4	21.9	-	21.9	-
	Min. Moment	-	57.8	89.8	-	121.4	-
	f_{min} (ksi)	-	27.6	22.6	-	22.5	-
	f_f (ksi)	-	15.1	15.9	-	16.0	-
	f_s (ksi)	-	5.44	5.52	-	4.05	-
Min/ Max Check	0.002 A_g (in^2/ft)	0.19	0.19	0.22	0.22	0.24	0.24
	$^c/_d$	-	0.09	0.13	-	0.14	-

K. Summary of Required Flexural Reinforcement

The area of reinforcement necessary to satisfy strength, crack control, fatigue and minimum reinforcement levels is:

Sidewall:
Outside face $A_s = 0.36\ in^2/ft$
Inside face $A_s = 0.19\ in^2/ft$

Top Slab:
Outside face $A_s = 0.22\ in^2/ft$
Inside face $A_s = 0.60\ in^2/ft$

Bottom Slab:
Outside face $A_s = 0.24\ in^2/ft$
Inside face $A_s = 0.72\ in^2/ft$

L. Check Shear
[5.8.1]
[5.14.5.3]

The concrete shear strength for box culvert slabs is given by:

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

Except under deep fills the shear capacity rarely governs. To simplify the calculations drop the second term in the brackets and use:

$$V_c = \left(0.0676 \cdot \sqrt{f'_c}\right) \cdot b \cdot d_e$$

Sidewall

The maximum design shear is:

$V_u = 4.28$ kips (bottom, Strength I: Case 2)

The capacity is:

$\varphi \cdot V_c = 0.90 \cdot \left(0.0676 \cdot \sqrt{5.0}\right) \cdot 12 \cdot 6.30 = 10.28 > 4.28$ kips <u>OK</u>

Top Slab

The maximum design shear is:

$V_u = 8.04$ kips (end, Strength I: Case 1)

The capacity is:

$\varphi \cdot V_c = 0.90 \cdot \left(0.0676 \cdot \sqrt{5.0}\right) \cdot 12 \cdot 7.25 = 11.84 > 8.04$ kips <u>OK</u>

Bottom Slab

The maximum design shear is:

$V_u = 9.44$ kips (end, Strength I: Case 1)

The capacity is:

$\varphi \cdot V_c = 0.90 \cdot \left(0.0676 \cdot \sqrt{5.0}\right) \cdot 12 \cdot 8.26 = 13.48 > 9.44$ kips <u>OK</u>

M. Check Thrust

The axial capacity of the culvert should be checked to ensure it satisfies the provisions of LRFD Article 5.7.4. For this design example, the sidewall member will be checked. It has the largest thrust value and the least thickness.

The design axial load is:

P_U = 12.1 kips (bottom Strength I, Case 3)

If the factored axial load is less than 10% of the nominal compressive capacity of the section, flexural design can be performed ignoring axial load effects. Without stirrups in the section, the resistance factor for compression is 0.70.

[Eq. 5.7.4.5-3]

$$0.10 \cdot \varphi \cdot f'_c \cdot A_g = 0.10 \cdot 0.70 \cdot 5.0 \cdot (8.0 \cdot 12.0) = 33.6 \;>>\; 12.1 \text{ kips}$$

By inspection, the section has adequate axial capacity. Note that the bending capacity of the sidewalls would benefit from the applied axial force. Since the benefit is small, the interaction is neglected.

N. Summary

Figure 12.6.5 illustrates the required reinforcing for the inside face and outside face of the sidewalls, top slab, and bottom slab.

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.

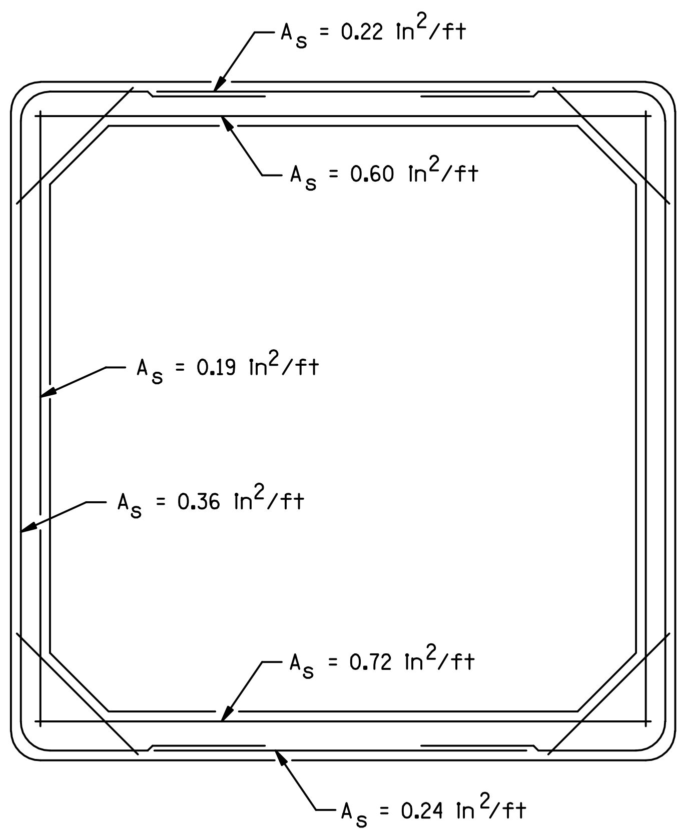


Figure 12.6.5 - Box Culvert Reinforcing