Unpublished Appx A, F, G, H
NCHRP Project 12-45
Copy No. 1

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 473

Recommended Specifications for Large-Span Culverts

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Subject Areas
Bridges, Other Structures, and Hydraulics and Hydrology

Research Sponsored by the American Association of State Highway and Transportation Officials
in Cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD — NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY PRESS
WASHINGTON, D.C. — 2002

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NCHRP REPORT 473

Project C12-45 FY'96

ISSN 0077-5614

ISBN 0-309-06757-X

Library of Congress Control Number 2002106859

Price $21.00

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Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

and can be ordered through the Internet at:

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Printed in the United States of America

APPENDIX A

STATE-OF-THE-ART OF LARGE-SPAN CULVERT DESIGN AND CONSTRUCTION PRACTICE

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APPENDIX A – STATE-OF-THE-ART OF LARGE-SPAN CULVERT DESIGN AND CONSTRUCTION PRACTICE

The state-of-the-art of design and installation of large-span culverts has been relatively stable for some time. Design methods for metal culverts are highly empirical, relying heavily on experience; for example, in current AASHTO Specifications, gage and minimum cover are generally controlled by a simple table, rather than by analysis and design calculations. This approach is not consistent with AASHTO's desire to adopt LRFD design approaches. Design of concrete culverts is not well defined, but generally consists of the analysis of a rigid frame loaded with assumed pressure distributions, or finite element analysis followed by design using AASHTO methods for reinforced concrete design. Pressure distribution assumptions, and in the case of FE analysis, ultimate load considerations, are not clearly defined. This chapter reviews key issues in the structural design of metal and concrete large-span culverts.

The order in which the topics are presented is not based on the importance of each subject. A complete list of references compiled from the literature search is attached at the end of this Appendix.

DOCUMENTED FIELD EXPERIENCE

Considerable field experience with large-span culverts is available in the literature. Information can be grouped into two categories: 1) performance monitored during construction using a planned instrumentation program, and 2) performance monitored after a defect has been noted or a failure has occurred. The information found in each category is summarized in this section.

Monitored Performance

Documented installation cases for both corrugated metal and reinforced concrete long-span culverts were identified and are summarized in Tables A-1, A-2, and A-3. Most cases had culvert deflection measurements as a minimum. Some cases included pipe wall strain measurements and soil stress, strain, and deformation measurements. Laboratory test results on soil properties are also available for some cases. Soil tests conducted varied, including standard and/or modified Proctor, relative density, soil classification, CBR tests, one-dimensional (1D) compression tests, and triaxial compression tests. Many of the case studies served as research projects to advance the state of the art and have been evaluated by numerical models, such as the finite element computer program CANDE (Computer Analysis and Design).

Table A-1 – Summary of Instrumented Field Installations for Steel Culverts

Project Name	Reference	Shape	Span x Rise (m x m)	Cover (m)	Corrug. Dimens. (mm)	Plate Thick. (mm)	Measurements: Pipe ε	Pipe Δ	Soil σ	Soil ε	Soil Δ
Deux Rivieres	Bakht, 1981	Round	7.7	2.36-2.6	152 x 51	5.45	X				
Adelaide Creek	Bakht, 1981	Hor. Ellipse	7.24 x 4.08	1.1-1.35	152 x 51	4.67	X				
White Ash Creek	Bakht, 1981	Round	7.62	0.9-1.25	152 x 51	4.67	X				
German Test	Demmin, 1965	Pipe arch	6.2 x 4.0	1.5		4.8	X	X	X		
Kettle Creek	Selig, 1975; Moore et al., 1995b	Hor. Ellipse	11.3 x 8.2	8.7	152 x 51	7.1 & 6.3				X	
Port Dover	Selig, 1975	Arch	15 x 5.4	1.5		6.3				X	
Thunder Bay	Selig and Calabrese, 1975; Calabrese, 1974	Hor. Ellipse	8.2 x 4.9	0.8	152 x 51	5.4	X	X	X	X	
Bucks County	Selig and McVay, 1980; McVay, 1982	Arch	11.5 x 4.8	3.4	152 x 51	5.5	X	X	X	X	
Sacramento County	Selig and Musser, 1985	Hor. Ellipse	7 x 4.3	0.9	152 x 51	3.6	X	X			
Stenner Creek	Bacher and Kirkland, 1986	Arch	10.7 x 6	1.2				X			
McIntyre River Bridge	Bakht, 1985	Hor. Ellipse	8.8 x 5.0	1.5-1.7	152 x 51	5.5	X				
Hamilton County	Bowers and Swaminathan, 1995	Round	4.4	20				X	X		
James Bay	Lefebvre at al., 1976	Arch	15.5 x 7.9	13.4	152 x 51	7.0	X	X	X		X
Leigh Creek	Kay and Flint, 1982	Arch	11.8 x 4.7	2.1	152 x 51	7.0	X	X			X
Blairmore Creek	Playdon and Simmonds, 1988	Hor. Ellipse w/ Cap	8.5 x 4.3		160 x 50	5		X			
Paulding County	Brewer, 1992	Pipe Arch	3.8 x 2.4	0.8				X	X		
Big Creek	Brewer, 1992	Arch	10.5 x 4.0					X	X		
Euroroad 6	Vaslestad, 1990	Hor. Ellipse	10.8 x 7.1	4.2	200 x 55	7.0	X	X	X		
Obed River Bridge	Musser et al., 1990	Arch	12.4 x 6.0	1.4	381 x 140	4.3		X			
Culvert B	Sargand et al., 1993	Box	4.8 x 1.5	2.7	152 x 51	4.3	X	X		X	X
Culvert C	Sargand et al., 1993	Box	4.8 x 1.5	2.5	152 x 51	3.1	X	X		X	X
Culvert D	Sargand et al., 1993	Box	4.6 x 1.5	2.5	381 x 140	3.5	X	X		X	X
Tolpinrud	Vaslestad, 1989	Pipe Arch	7.8 x 6.9	1.1-1.6	200 x 55	6.8			X		
Spring Creek	Mayberrry and Goodman, 1989	Round	3.8	23.4		4.8	X	X	X		X
Deep Creek	Mayberrry and Goodman, 1989	Round	4.6	13.7		3.5	X	X	X		X
Newtown	Selig et al., 1977; Lockhart, 1975	Arch	7.9 x 4.6	7.0	152 x 51	5.4	X	X	X	X	

See notes at end of Table A-3 for description of symbols

1 m = 3.28 ft; 1 mm = 0.0394 in.

Table A-2 – Summary of Instrumented Field Installations for Aluminum Culverts

Project Name	Reference	Shape	Span x Rise (m x m)	Cover (m)	Corrug. Dimens. (mm)	Plate Thick. (mm)	Measurements				
							Pipe		Soil		
							E	Δ	σ	ε	Δ
Greenbrier County	Duncan, 1975	Pipe Arch	10.7 x 6.1					X			
Santa Clara County	Duncan, 1975	Pipe Arch	7.3 x 5.8	2.5		6.4 & 5.1		X			
Walnut Creek	Duncan and Jeyapalan, 1982	Hor. Ellipse	7.6 x 3.9	1.2		3.8					
Promontory, Mesa	Seed and Ou, 1986	Arch	11.7 x 4.8	0.8	229 x 64	4.4		X			
Richmond Field Sta.	Duncan et al., 1985	Box	5.3 x 1.9	0.5		4.4	X	X			
Van Campen Creek	Beal, 1982	Pipe Arch	8.7 x 5.4		229 x 64	4.4	X	X			
Culvert A	Sargand et al., 1993	Box	4.2 x 1.5	2.3	229 x 64	5.1	X	X		X	X
Charlotte	Seed et al., 1989	Box	3.2 x 1.4	0.61	Smooth wall & ribs			X			

See notes at end of Table A-3 for description of symbols

1 m = 3.28 ft; 1 mm = 0.0394 in.

Table A-3 – Summary of Instrumented Field Installations for Reinforced Concrete Culverts

Project Name	Reference	Shape	Span x Rise (m x m)	Cover (m)	Measurements				
					Pipe		Soil		
					ε	Δ	σ	ε	Δ
North Philadelphia	Selig and McGrath, 1994	Arch	11 x 2.7	0.9		X	X		
Byron Bay	Kay and Rigon, 1986	Arch	9.0 x 3.0	4.2	X	X	X	X	X
Montgomery County	Beach, 1988	Arch	5.8 x 2.1	0.3		X			
San Antonio	Oswald and Furlong, 1993	Arch	12.3 x 2.9	7.3	X	X	X		
San Martinez Grande	Bacher et al., 1988	Semi-Circular Arch	6.1			X			

Measurements: ε = strain, Δ = deflection and/or other shape change, σ = stress

1 m = 3.28 ft; 1 mm = 0.0394 in.

The instrumented cases summarized in Tables A-1, A-2, and A-3 include corrugated steel, corrugated aluminum, and reinforced concrete large-span culverts, respectively. Reported culvert shapes include round, horizontal ellipses, low- and high-profile arches, semi-circular arches, and metal boxes. Culvert sizes vary from 3.8 m to 15 m (12.5 ft to 49.2 ft) horizontal span and from 1.5 m to 8.2 m (4.9 ft to 27 ft) vertical rise. Cover depths above the culvert crowns vary from as little as 0.5 m (1.6 ft) to as much as 23 m (75 ft) for a 3.8 m (12.5 ft) diameter round culvert in Montana. Standard plate thicknesses and corrugation dimensions for the metal culverts are reported as well as special corrugations (deep corrugations). Some of the metal culverts include longitudinal stiffeners, transverse rib stiffeners, or reinforced concrete relieving slabs. Some of the literature includes testing under live loads in addition to testing under earth load. A few culvert studies have included long-term performance monitoring with observation periods of up to seven years after installation.

Failure Cases

Documented failure cases are summarized in Table A-4. To present the information concisely, Table A-4 uses indices, which are explained in Table A-5.

A total of thirty-three culvert failures were identified. Thirty-two of the failures identified were either corrugated steel or aluminum plate culverts, and the other was a reinforced concrete pipe-arch. Of the thirty-two metal culvert failures identified, only one culvert had a span less than 3.0 m (10 ft).

Nine failures occurred as a result of poor backfill procedures and/or poor backfill material selection, including excessive compaction pressures and the use of frozen soil and/or frost-susceptible soils as backfill. Three failures occurred as a result of a design error, and six more occurred for each of: 1) excessive construction load with or without shallow cover, and 2) flotation or invert uplift. Loading a culvert to failure for the purpose of research and inadequate cover resulted in two failures each. One failure case for each of the remaining causes in Table A-4 was also reported.

Table A-4 – Summary of Failure Causes

Project Name	Reference	Shape	Span x Rise (m x m)	Cover (m)	Corrug. (mm)	Plate Thick. (mm)	Special Features	Cause (Note 1)	Limit State (Note 1)
Kettle Creek	Moore et al. (1995 a); Moore et al. (1995 b)	Hor. Ellipse	11.3 x 8.2	8.7		7.1 to 4.8	Long. Stiff.	10	1, 6
Elkhart Creek	Byrne et al. (1993)	Arch	13.4 x 7.3	1.0		7	Long. Stiff., Rib Stiffeners	5	4
Wolf Creek Canyon	Kraft and Eagle (1966); Macadam (1966); Scheer and Willet (1969)	Round	5.6	25		9.5	None	2	2
Cooper City – 1985	Seed and Raines (1988)	Pipe Arch	8.7 x 5.4	N/A	229 x 64	3.8	Rib Stiffeners	3	3, 5
Cooper City – 1986	Seed and Raines (1988)	Pipe Arch	8.2 x 5.2		229 x 64	3.2	Rib Stiffeners, Relieving Slab	3	3
Rancho	Seed and Raines (1988)	Box	4.5 x 1.2	0.61	229 x 64	4.4	Rib Stiffeners	3	4
West Williamson	Sehn and Duncan (1994)	Round	1.83	9.0	68 x 13	4.3	None	5	3
Newport	Temporal et al. (1985)	Pipe Arch		0.36	100 x 20	3	None	1	3
Iowa	Lohnes et al. (1995)	Hor. Ellipse	4.54 x 2.92					8	–
		Hor. Ellipse	4.52 x 2.92					8	–
		Hor. Ellipse	9.78 x 5.85					8	–
		Round	4.57					8	–
		Round	5.18					8	–
German Test	Demmin (1965)	Pipe Arch	6.2 x 4.0	1.5		4.8	None	1	3
France	Luong (1988)	Pipe Arch	10.95 x 8.1	N/A	152 x 52	7	None	5	3, 4
Vista	Seed and Ou (1988)	Arch	11.7 x 4.8	0.61	229 x 64	5.1	Rib Stiffeners	3, 5	3
Zuber Creek	HNTB' and Selig (1983)	Semi-Circular Arch	9.1 x 4.6	2.1-2.4		6.3	None	5	3, 4
La Plata County	Eldorado Eng. (1982)	Arch					Compaction Wing	6	6, 5, 4
Hayden Creek	Selig (1988 b)	Arch	10.5 x 3.6				Rib Stiffeners	7	3
Lamy	Selig (1992)	Hor. Ellipse	6.9 x 4.8	N/A		3.6		8	3, 4

1. See Table A-5

1 m = 3.28 ft; 1 mm = 0.0394 in.

Table A-4 – Summary of Failure Causes (cont'd)

Project Name	Reference	Shape	Span x Rise (m x m)	Cover (m)	Corrug. (mm)	Plate Thick. (mm)	Special Features	Cause (Note 1)	Limit State (Note 1)
Minnesota	Selig (1991)	Arch	6.8 x 2.4	3.0		5.5, 3.6	Long. Stiff.	9	3, 4
Cocagne	Selig et al. (1978) Selig et al. (1977)	Hor. Ellipse	R_t = 7.9 m				Long. Stiff.	3, 5	4
Namur		Hor. Ellipse	R_t = 9.1 m					3, 11	4
Muzroll Brook		Hor. Ellipse	R_t = 7.0 m					5	3, 4
Baie James		Hor. Ellipse	8.8					4, 5	4
Roger's Pass		Pear		N/A				5	3
Fort Simpson		Hor. Ellipse	Span = 9.1	6-8				5	3
Rengleng River		Hor. Ellipse	Span = 11.6					5	3
Irvine		Arch	9.8 x 3.4	0.6			Long. Stiff.	4	3, 4
Licking County							Soil Bin	5	4
Red Mountain Creek		Arch	15 x 3			5.5	Soil Bin	–	4
Newfane		Arch	18 x 7.3	2.1		6.3	Soil Bin	10	3, 4
Worthington	Hill and Laumann (1994)	Pipe Arch	3.1 x 1.98	7.2				10	7

1. See Table A-5

1 m = 3.28 ft; 1 mm = 0.0394 in.

Table A-5 – Explanation of Indices Used in Table A-4 for "Cause" and "Limit States"

Number	Failure Cause(s)	Limit State
1	Research – excessive load	Buckling of plates
2	Hydrogen embrittlement	Seam/bolt failure
3	Excessive construction load with/out shallow cover	Excessive deflections (flattening, reversal of curvature)
4	Inadequate cover	Collapse
5	Poor backfill procedures and/or material	Flexure
6	Saturation of backfill	Bolt line cracking
7	Footing settlements	Cracking (concrete)
8	Flotation/invert uplift	
9	Deviation from design	
10	Design error	
11	Other	

Limit states approached before or at failure are summarized in Table A-4. In seventeen cases, the first limit state reached was excessive deformation, characterized as deflection, flattening of structural plates, or reversal of curvature. In six cases of culvert collapse, no specific limit states were reported, although most of the cases involved excessive deformations prior to collapse. In five failures associated with invert uplift and reported by Lohnes et al. (1995), no specific limit state was reported. One case was reported for each of the remaining limit states in Table A-5.

Horizontal elliptical culverts were identified as the most common shape of culvert to fail, with a total of eleven failures. Ten failures were identified involving arch culverts, six involving pipe-arch culverts, and four involving round culverts. One failure each involving a metal box and a pear-shaped culvert was identified.

From the identified failure cases in Table A-4, the following relationships between culvert shape and most likely cause of failure can be made:

- Round culverts failed most often by invert uplift (50% of cases).
- Horizontal elliptical culverts failed most often by invert uplift (36% of cases), followed by poor backfill procedures or poor backfill material (27% of cases), and excessive construction loads (18% of cases).
- Pipe-arches failed in 50% of the cases due to excessive construction loads (neglecting the two cases that involved research work with the purpose of failing the culverts).

- The eight failures of arches were reported to be a result of eight different causes.

Bakht and Agarwal (1988) concluded from a survey conducted in parts of Canada on distress in metal culverts that more pipe-arch culverts are in distress than any other type of culvert. The most common cause of failure was "bolt-hole tears," involving the formation of horizontal cracks through bolt-holes. Possible ways of reducing bolt-hole tears, according to Bakht and Agarwal (1988), are to ensure the following:

- nesting of seams,
- controlling relative radii of curvature,
- preventing over-tightening of bolts, and
- reducing high radial pressures occurring in haunch areas.

TIME EFFECTS ON CULVERT-SOIL INTERACTION

A few monitored performance studies have included measurements of time effects on culvert-soil interaction. These time effects may be attributed to a single or a combined cause, including soil creep and consolidation, and reduced soil support caused by changes in water content, seepage, and freeze-thaw cycles. In the past, dealing specifically with the effects of time on culvert-soil interaction and designing for such effects, have been largely ignored or handled indirectly by means of empirical "lag" factors, such as the one used in the well-known Iowa deflection formula. This approach is reasonable given the complexities involved in quantifying the time effects. Furthermore, the scarcity of data on this subject has limited the development of reliable new models for predicting changes in culvert-soil systems with time.

Cases in which culvert-soil behavior has been monitored over a period of time following construction show that significant changes can occur in earth pressure distribution around large-span culverts and overall structural response with time after construction. In some cases, earth pressure at the springline elevation increased with time, but in other cases, it decreased. Significant seasonal fluctuations in earth pressure were observed resulting from thermal expansion and contraction. Observations show that increases in thrust also can be significant. The use of frozen soils as backfill material can cause structural distress of the culverts and subsequent collapse. Reports on time effects since 1982 include Kay and Flint (1982);

Vaslestad (1990), Selig (1991); Oswald and Furlong (1993), Sargand (1993), and Selig and McGrath, (1996).

LIMIT STATES

Reliability based design requires knowledge of limit states for the structures under consideration. The limit states considered in current codes (AASHTO LRFD Specifications, AASHTO Standard Specifications and Ontario Highway Bridge Design Code, 1991 a and 1991 b) and other proposals are presented here.

Large-Span Concrete Culverts

Limit states for large-span concrete culverts do not differ from those of smaller concrete culverts and are consistent with the limit states for most concrete structures of all types. The LRFD and Standard Specifications have specific requirements for concrete culverts, but not for large-span concrete culverts. The limit states to be considered for concrete culverts include:

- Service limit state: crack width and, for shallow culverts, service live load reinforcement stress
- Strength limit states: flexure, shear, thrust, and radial tension

The OHBDC does not have a section for concrete culverts of any type, but the provisions for general concrete structures include:

- Service limit state: cracking, deformation, and fatigue
- Strength limit states: flexure, shear, and stability

The one significant difference between OHBDC and AASHTO Specifications is the requirement for a check on radial tension strength. This is a requirement that results from the curved reinforcement in concrete pipe. It is also applicable to large-span concrete culverts.

Large-Span Metal Culverts

Limit states for large-span metal culverts are less consistent than those for reinforced concrete culverts and are more controversial as well. The LRFD Specifications require consideration of the following:

- Service limit states: none
- Strength limit states: wall area (compression), seam failure, and for box sections only, flexure

The above limit states are modified somewhat by other code provisions:

- The construction specifications include limits on change in shape of the structure during construction, which is actually a service limit state.
- Section 12.6.3.2 of the LRFD Specifications requires the corner backfill for metal pipe arches be designed to account for corner pressure. This is a limit state for the backfill envelope.
- Special features (longitudinal and transverse stiffeners) are required for large-span culverts. Special features improve the performance of large-span culverts; however, since there are no design models to quantify the improvement, specific limit states are not set forth.

In addition to the above, the design of metal culverts that are not large spans include service limit states for flexibility and buckling capacity. These limit states are not considered for large-span metal culverts because the models used for smaller spans would reject designs that experience has shown provide good performance. Thus, it appears that these limit states are omitted not because they are inappropriate for large-span culverts, but rather because there is no suitable analytical model.

One proposal that has received considerable attention for the design of large-span metal culverts is the SCI method (Duncan 1978). This method incorporates two criteria that address the question of flexural strength of metal culverts:

- Moment capacity when the backfill is at the top of the culvert: This criterion is developed to provide minimum structural stiffness during construction, similar to the flexibility limit for smaller culverts.

- Moment capacity under depths of fill less than one-fourth of the span: This criterion accounts for the fact that, under low fill heights, a large-span culvert may require a minimum flexural strength to resist live loads. No similar criterion for this exists in the current LRFD Specifications.

A design model that includes the benefit of circumferential and transverse stiffeners would allow consideration of flexural affects as recommended by Duncan.

DESIGN PRACTICE

Design practice, as currently specified in bridge design codes, is very different for large-span concrete and metal culverts. In some ways, the design approach parallels the limit states presented in the previous section, in that large-span concrete culverts are treated similarly to smaller concrete culverts and other concrete structures, while large-span metal culverts are treated as special structures and are designed based on experience more than on analytical models.

Design of Concrete Culverts

Large-span concrete culverts are not singled out for special treatment in the current AASHTO Bridge Specifications; thus, as noted above, they are designed in accordance with common practice for reinforced concrete. Analysis of large-span concrete culverts is generally completed in either of two fashions:

- **Finite element models:** Finite element models account for the stiffness of both the soil and structure and can address nonlinear soil and structural behavior.
- **Assumed pressure distributions:** A common model for reinforced concrete is the assumption of a linear reinforced concrete structure with the stiffness based on the modulus of an unreinforced, uncracked section. The model is then loaded with an assumed pressure distribution. This approach is demonstrated by the SIDD design method for reinforced concrete pipe that is incorporated into the Standard Specifications.

The finite element model does not require any assumptions about total load (i.e., arching) or pressure distribution. The assumed pressure distribution approach is commonly used and generally provides adequate accuracy; however, it requires design assumptions for arching and pressure distribution. No guidance is provided in current AASHTO Specifications for large-span concrete culverts.

Design of Metal Culverts

Design of large-span metal culverts is largely experience based, as noted in Section 2.4. The Standard and LRFD Specifications provide no guidance on analysis of the structures except to compute the hoop compressive stress.

The LRFD Specifications provide designs for large-span culverts based on the following steps:

- Wall area and seam strength must be adequate to carry the computed thrust, which is based on the "crown pressure;" however, no guidance is provided on how to determine the crown pressure. Past experience shows that the crown pressure on metal culverts can be substantially less than the geostatic soil stress.
- Large-span metal culverts must have special features consisting of longitudinal or transverse stiffeners. No criteria are provided for determining the required strength or stiffness of these elements. Section 12.8.3.5.2 of the LRFD Specifications does require that circumferential stiffeners be fastened to the structure to ensure integral action with the corrugated plates and be spaced at intervals as necessary to provide the required moment of inertia of the structure; however, since there are no flexural strength or stiffness criteria, there is no way to determine the required moment of inertia.
- The metal plate thickness is subject to minimum values based on depth of fill and top arc radius.
- Geometric constraints are provided in the form of limitations on the maximum central angle of the top arc and the ratio of the top arc radius to the side arc radius.

The OHBDC follows an ultimate limit state design approach for conduit wall failure in compression and failure of seams. The effects of bending moment in the conduit wall are neglected, and only thrust in the conduit wall is considered. Therefore, design of the conduit wall against compression failure is either by wall crushing, elastic buckling, or a combination of the two. The OHBDC Specifications do not distinguish between "small-" and "large-" span culverts. Dead load thrust in the conduit wall is based on finite element studies by Haggag (1989) and is computed as a function of axial and flexural stiffness parameters, culvert shape, cover depth, quality and extent of structural backfill, and foundation stiffness. The conduit wall is designed for the combined effect of dead load and live load thrusts.

The OHBDC places an upper limit on culvert handling and installation stiffness, which has been taken from the Standard Specifications. Minimum depth of cover is specified as a function of culvert shape. Additional limits are placed on radii of curvature of the conduit wall as well as maximum differences in plate thicknesses for mating plates.

Similar to the Standard and LRFD codes, no structural design provisions are provided for special features.

The SCI method (Duncan 1978), previously discussed, is an empirical design method based on coefficients that were derived from the results of extensive finite element analyses. Coefficients for thrust are based on the rise to span ratio, while coefficients for moment are based on the flexibility number, N_f, which is defined as:

$$N_f = \frac{E_s S^3}{E I} \tag{A-1}$$

where:

- E_s = Young's modulus of elasticity of the soil, MPa, psi
- S = span, m, in.
- E = modulus of elasticity of the culvert material, MPa, psi
- I = moment of inertia of the pipe wall, m^4/m, $in.^4/in.$

Bakht (1985) reported the use of a reinforced concrete relieving slab placed above a large horizontal ellipse, designated as the McIntyre River Bridge in Thunder Bay, Ontario, Canada, and compared the structural behavior to that of a similar bridge without a relieving slab, designated as the Adelaide Creek culvert. Measured results indicated a 45% to 50% reduction in maximum live load thrusts for the culvert with a relieving slab, as well as a more uniform thrust distribution around the culvert circumference compared to live load thrusts for the culvert without a relieving slab. For the culvert with the relieving slab, measured live load moments were negligible.

Katona and Akl (1987 a, 1987 b, 1985, and 1984) investigated the use of slotted bolt-holes to reduce axial wall thrusts specifically for culverts under high embankments. The principle is based on reducing the circumferential stiffness of the culvert by allowing a predetermined amount of joint slippage. Conclusions from the study were that slotted bolt-holes can allow significant increases in burial depths, up to a factor of 2 or more, compared to culverts without slotted bolt holes, provided that good quality backfill material is used. For poor quality backfill material or smaller diameter-to-thickness ratios, the benefits of slotted bolt-holes are marginal.

Lefebvre et al. (1976 and 1972) included compressible squeeze blocks inserted between the arch and the concrete footings for a 15.5 m (51 ft) span x 7.9 m (26 ft) rise metal arch culvert under a 13.4 m (44 ft) high embankment. The squeeze blocks were intended to induce positive arching and thus reduce the load on the culvert. Measurements after construction showed a reduction in soil pressure on the culvert crown to 25% of the overburden value, and a substantial reduction in wall thrust.

Additional special features that may be used for construction of large-span metal culverts include the arch-beam-culvert system (ABC), concrete-arch-buried-bridge (CABB), and a system that reinforces the surrounding backfill soil (reinforced earth discussed in Kennedy and Laba, 1989). More information may be found in Abdel-Sayed, (1993).

CONSTRUCTION PRACTICE AND METHODS

Metal Culverts

Maintaining a well-organized construction site for the storage and assembly of corrugated metal plates can greatly speed up the construction process (Abdel-Sayed et al., 1993). Plates of similar sizes should be stacked together and kept apart from other sizes to prevent accidental use of the wrong size plate in erecting the culvert.

Construction operations should commence in dry conditions, thus excavations below the groundwater table should be dewatered or, when the culvert is installed in a stream or river bed, the water should be diverted or separated by cofferdams.

The foundation soil must uniformly support the imposed loads. It should prevent excessive down-drag forces from developing due to higher settlements underneath the structural backfill than under the culvert itself and must be free of local rock ledges and/or soft spots. A bedding layer is used when the natural foundation material does not provide adequate support. The LRFD code recommends the use of a shaped bed for pipe arch, horizontal ellipse, and underpass shapes with spans greater that 3.6 m (12 ft). If sag in the longitudinal profile is to be prevented or minimized due to the higher loads under the center of the embankment, the bedding can be cambered relative to the desired final grade.

The OHBDC code specifies a stone-free bedding of granular material, pre-shaped to accommodate the invert curvature. The upper 200 mm (8 in.) in direct contact with the culvert wall is left uncompacted.

The LRFD Specifications indicate that unless the culvert plates are held in place by cables, struts, or backfill, the longitudinal seams should be tightened when the plates are hung. The plates should be carefully hung to ensure proper nesting of seams. Longitudinal seams should be staggered such that no more than three plates meet at any point. Staggered longitudinal seams are preferred since this arrangement is not prone to "zipper" types of failure (Abdel-Sayed, 1993). The LRFD code further requires that deformations during construction be held within the following limits:

- For horizontal ellipse shapes (with ratio of top to side radii of 3 or less), the span and rise dimensions must be within 2% of those specified.
- For arch shapes (with ratio of top to side radii of 3 or more), the deviation in the rise dimension must be within 1% of the span dimension.
- For all other long-span culverts, the span and rise dimensions must be within 2% of those specified, or less than 125 mm (5 in.), whichever is less.

OHBDC limits construction deformations to 5% of the conduit rise for culvert shapes other than round or elliptical. For these latter shapes, the limits are calculated as a function of culvert dimension and shape factor.

Abdel-Sayed et al. (1993) recommend placing the backfill material in the haunch zone in layers not exceeding 150 mm (6 in.) compacted thickness. Controlled low strength material (CLSM, also known as flowable fill) can also be used for uniform support in this zone.

After the haunches are backfilled, material is placed at the sides of the culvert to the level of the culvert shoulder, or to the location of the longitudinal stiffener, if used. This is the sidefill zone. Abdel-Sayed et al. (1993) report on surcharging (top loading) the culvert during sidefilling by placing backfill soil above the culvert crown to control peaking deformations of the crown. They report that this is common practice.

OHBDC specifies that the structural backfill zone should extend laterally to at least half the span on both sides of the culvert and vertically to the minimum cover depth requirement, and it requires continuous inspection and supervision by an engineer for the construction of large-span metal culverts with special features.

The next important stage of backfilling begins when the fill reaches the top of the longitudinal stiffeners or shoulder. Selig et al. (1977) suggests that a light dozer, such as the D2, push backfill material in from the sides and gradually up over the top of the culvert. If stiffening ribs are present, this should be done first at the location of the ribs. The shape of the culvert must be continually monitored during this process. This process should continue until 0.3 to 0.6 m (1 to 2 ft) of loose material has been placed in a uniform layer over the crown. A vibratory roller should then be used to compact the material over the top; however, for the first layer or so, vibration should not be used when the roller is between the vertical faces of the longitudinal stiffeners. Selig et al. (1977) report that very little further change in shape will occur after the fill has reached a level of 0.6 to 1.3 m (2 to 4 ft) above the crown of the culvert. Therefore, the shape must be proper at the time the crown is covered.

Large-Span Concrete Culverts

While none of the existing codes, including AASHTO LRFD Construction Specifications (Modjeski and Masters, Inc., 1995), AASHTO Standard Specifications (AASHTO, 1989), and/or the OHBDC (OHBDC, 1991) explicitly deal with the construction of large-span concrete culverts, they do deal with smaller culvert shapes and box culverts. Construction methods specified by manufacturers for two types of large-span concrete culverts (CON/SPAN, 1995; FitzSimons, 1996 a and 1996 b) have their own specifications, which form the basis for the following discussion.

Specifications vary regarding whether footings may be precast. Precast arch segments are always placed in a keyway formed in the footings. Leveling pads are used to bring the segments to grade. The keyway is then filled with grout.

Allowable backfill materials are generally AASHTO A-1, A-2, and A-3 classifications (GW, GM, SW, and SM according to USCS classification ASTM D2487), although one manufacturer also allows A-4 material up to depths of 3.7 m (12 ft). Backfill is placed and compacted in lifts not exceeding 300 mm (12 in.) in thickness and compacted to a minimum of 90% or 95% of AASHTO T180 density. The backfill around the concrete culvert within about

300 mm (12 in.) of the culvert must be compacted using small walk-behind machines, whereas larger ride-on vibratory rollers may be used farther away.

Manufacturers commonly supply precast reinforced concrete headwalls and wingwalls.

Following an inspection of arch culverts constructed in Minnesota, Hill (1986, 1985 a, and 1983) and Hill and Shirole (1986) reported the following recommendations:

- Granular backfill material should be extended to at least 2.4 m (8 ft) on both sides of the culverts.
- Lateral outward movement of the arch sections at their footing lines, which will continue until passive soil resistance has been mobilized, may result in hairline cracking at midspan; therefore, piling of the concrete footings to restrain this movement should be considered.
- Culverts should be protected against scour at the footings.

CONSTRUCTION EFFECTS

Satisfactory performance of a large-span culvert, especially a flexible culvert, requires proper construction procedures, and deviation from well-established construction procedures may adversely affect culvert performance. Close supervision and monitoring of large-span metal culverts by a manufacturer's representative is common.

Deviation from the original design when installing culverts may have detrimental affects. Mirza and Porter (1981) suggested that large-span metal culverts are more forgiving when design errors are made than when poor construction procedures are followed. However, this does not imply that construction deviations from the original design are acceptable. A low profile structural plate arch in Minnesota, with a maximum horizontal span of 6.8 m (22 ft 3 in.) and a rise of 2.4 m (7 ft 11 in.) was designed for a 20° skew to the embankment, but was installed at a 40° skew. In addition, 3.0 m to 3.7 m (10 ft to 12 ft) high rock berms, not included in the original design, were installed over the culvert ends, creating additional loading. The culvert failed, and the cause was assessed as the increased loads due to the deviations from the original design (Selig, 1991).

LIVE LOADS ON CULVERTS

Current practice for the distribution of live loads through fills and onto culverts is variable. The AASHTO LRFD Specifications have changed the criteria from that of the AASHTO Standard Specifications, while the OHBDC uses a different method of distribution. All of these codes use simple linear distributions with depth. Some agencies use elasticity type solutions, such as those based on Boussinesq theory. All methods evaluate the spread of live load through fill without considering any influence of the culvert itself.

Live Load Distribution through Fill

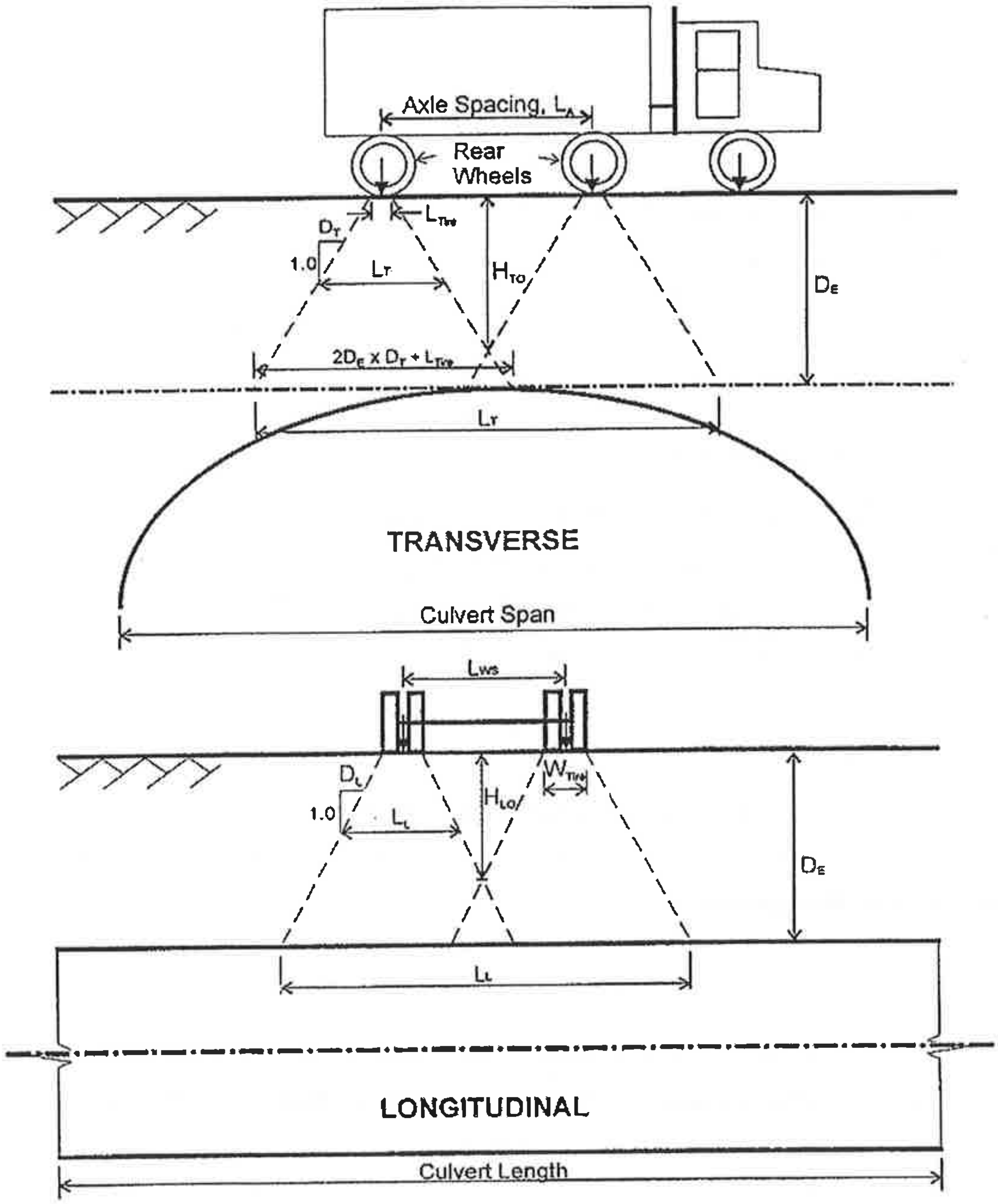
The simplified assumptions used to model the distribution of live load through fill in most current codes (AASHTO, 1994, OHBDC, 1991) are shown schematically in Figure A-1.

AASHTO Standard Specifications

The AASHTO Standard Specifications assume that a live load acts as a point load at the ground surface and, for any depth greater than 0.6 m (2 ft), acts on a square area with sides of length 1.75 times the depth of fill.

AASHTO LRFD Specifications

The 1st Edition of the AASHTO LRFD Specifications assume that a live load acts on an area (footprint) at the ground surface that is variable with the magnitude of the load, while later editions change this to a fixed area. The width of the footprint is 500 mm (20 in.), while the length was varied to produce a contact pressure of 860 kPa (125 psi), but has now been set at 10 in. The philosophy of increasing the size of the footprint applied to impact and factored loads as well as service loads. The live load distribution area at any given depth of fill is the size of the initial footprint plus the depth of fill multiplied by a live load distribution factor (LLDF, twice the value of the factors DT and DL shown in Figure A-1). The factor is 1.15 for select granular soils and 1.0 for other soils.



	OHBDC:	STANDARD:	LRFD:
D_T:	1.0	0.875	0.575 Granular 0.5 Other
D_L:	0.5	0.875	0.575 Granular 0.5 Other
L_{Tire} :	0.25 m	Point Load	Variable
W_{Tire}:	0.60 m	Point Load	0.51 m
L_A :	1.20 m - Tandem	4.3 - 9.1 m - Single 1.22 m - Tandem	4.3 - 9.1 m - Single 1.22 m - Tandem
Axle Load:	160 kN - Tandem	142 kN - Single 107 kN - Tandem	142 kN - Single 111 kN - Tandem

Figure A-1 – Summary of Design Vehicles and Live Load Distribtion

OHBDC

For soil steel structures, the OHBDC requires live load calculations only for tandem axles and assumes that the two axles interact at all depths of fill. The magnitude of the tandem axles is 160 kN (36,000 lb) per axle, and the axles are spaced at 1.2 m (4 ft). The size of the distribution area increases with the depth of fill; however, the LLDF varies in each direction (Bakht, 1981). In the direction of the longitudinal axis of the culvert, the LLDF is 1.0, while in the direction of the span of the culvert, the LLDF is 2.0.

Impact

AASHTO Standard Specifications

For culverts, the AASHTO Standard Specifications require an increase in live loads to consider dynamic effects. The relative increase in live load effect is 30% for zero depth of cover, and 0% at depths of cover of 0.9 m (3 ft) or greater. The magnitude varies stepwise, decreasing in 10% increments at depths of 0.3 m, 0.6 m, and 0.9 m (1 ft, 2 ft, and 3 ft).

AASHTO LRFD Specifications

The AASHTO LRFD Specifications require an increase in live loads to consider dynamic effects by a factor varying linearly from 33% for zero depth of cover to 0% at depths of cover of 2.5 m (8 ft) or greater.

OHBDC

For soil-steel structures, the OHBDC Specifications require an increase in live loads for dynamic load by a factor varying linearly from 40% at zero depth of cover to a minimum of 10% for depths of cover of 1.5 m (4.9 ft) and greater.

Live Load Distribution on Concrete Slabs

The AASHTO Standard and LRFD Specifications both allow a distribution of live load forces on concrete slabs if the depth of fill is less than 0.6 m (2 ft). The distribution is a function of the culvert span.

AASHTO Standard Specifications (positive moment regions):

Distribution width (ft) = 4 + 0.06 (Span, ft)

AASHTO LRFD Specifications (positive moment region):

Distribution width (in.) = 26 + 6.6 (Span, ft)

Both specifications allow wider distribution widths for negative moment regions, but this is normally ignored in culvert design practice because the presence of joints in culverts made up of relatively short precast segments prevents load transfer from the loaded segment to adjacent segments and consideration of the possibility that loaded adjacent lanes would restrict the distribution width to a single lane.

Other Aspects of Live Load Calculation

In addition to the above live load distribution items, the LRFD Specifications have introduced other changes from the Standard Specifications that affect the total design live load:

- The live load factor is reduced from 2.17 (Section 3.22 of the Standard Specifications) to 1.75.
- A multiple presence factor, called "m," for single trucks, is set at 1.2.
- The lane load and the live load vehicle must be considered simultaneously.

Since the product of the load factor of 1.75 and a multiple presence factor of 1.2 (for single lanes) is 2.1, the LRFD Specifications essentially require the same ultimate load as the Standard Specifications with a load factor of 2.17. However, the application of the multiple presence factor to the service limit live load effectively increases the load for that limit state by 20% over the service load computed according to the Standard Specifications.

The magnitude of live load on culverts in accordance with the three codes is demonstrated in Figure A-2, which compares the design live load per unit length of culvert, and demonstrates that the LRFD code is more conservative than the older Standard Specification. This increase in design load for culverts is contrary to some research, which suggested that the

Standard Specifications were too conservative for box culverts at low depths of fill (Frederick et al., 1988).

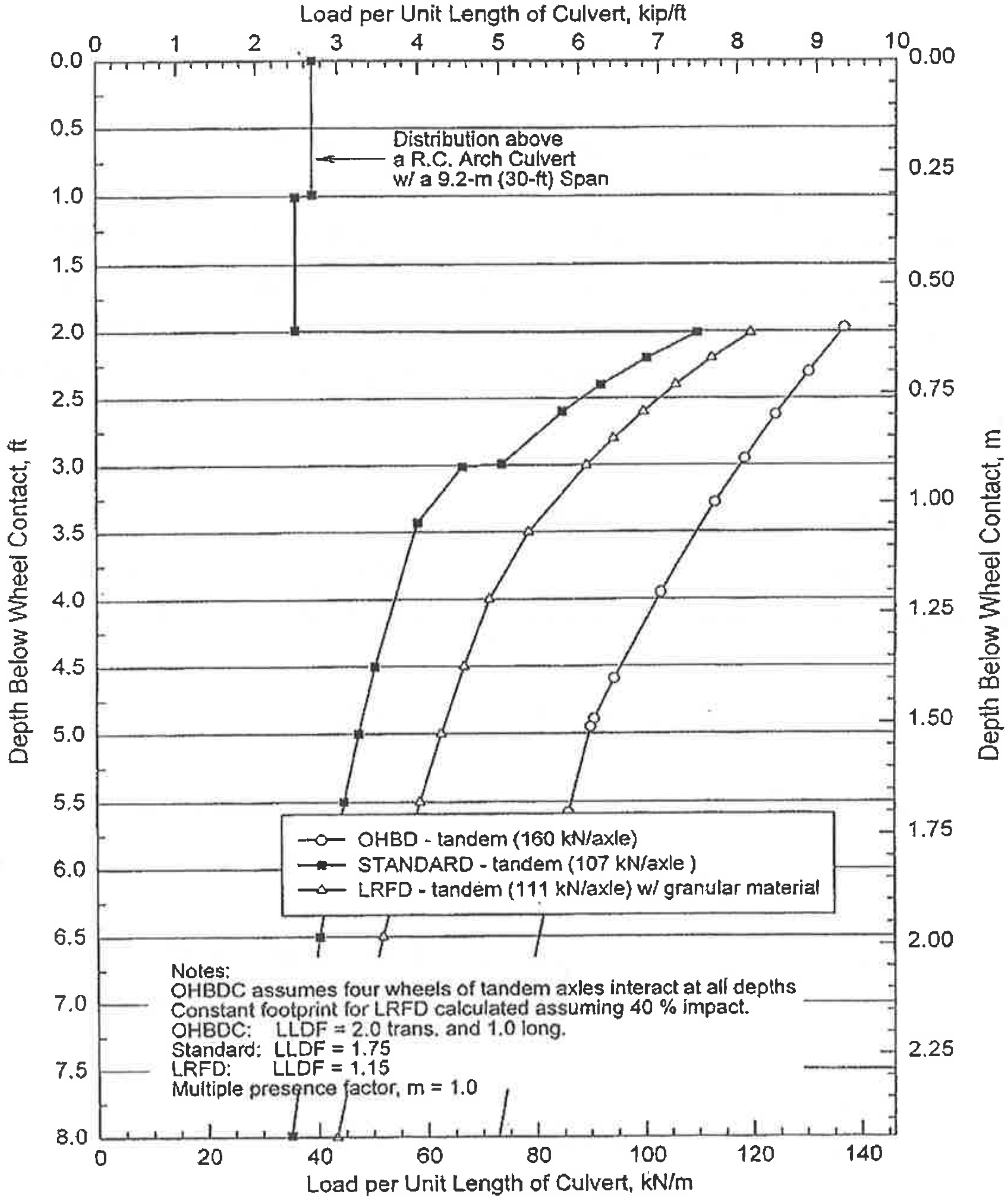


Figure A-2 – Comparison of Design Loads for AASHTO and OHBDC Specifications

BUCKLING OF LARGE-SPAN BURIED CULVERTS

The potential for a flexible metal culvert to buckle has been recognized ever since engineers began working to increase the span of corrugated metal culverts in the late 1950s and early 1960s (Meyerhof and Baikie, 1963).

Workers have taken two different approaches to design for buckling. Empirical design guidelines have been developed, which use backfill specifications to ensure high quality granular support to the structure, normally preventing the buckling mechanism (e.g., AASHTO, 1996). Others have worked to develop theoretical analyses that can be used to predict, and therefore control, the phenomenon.

Previous literature reviews discussing the buckling issue include those of Leonards and Stetkar (1978), Baikie and Meyerhof (1982), and Moore (1989).

Analysis

Elastically Supported Circular Rings

Analysis of flexible culvert buckling has largely been based on the development of theoretical solutions for circular elastic rings with uniform soil support and uniform hoop compression. Most solutions have used the Winkler (elastic spring) model to characterize the soil support. These include: Stevens (1952), Booy (1957), Meyerhof and Baikie (1963), Luscher and Hoeg (1964), Luscher (1966), Chelapati (1966), Sonntag (1966), Hain (1970), Kloppel and Glock (1970), Cheney (1971 a, b), Chelapati and Allgood (1972), and Falter (1980).

Some solutions have been developed using the elastic continuum theory to characterize the soil support. These include: Forrestal and Herrmann (1965), Duns (1966), Duns and Butterfield (1971), Cheney (1976), Moore and Booker (1985a), and Moore (1985).

Some workers recognize that non-uniform thrust develops in most metal culverts. Anderson and Boresi (1926) considered a non-uniformly loaded ring, and Moore and Booker (1985b) considered the case of circular culverts with uniform elastic continuum support under non-uniform hoop compression. Three-dimensional circular ring solutions have also been developed to examine the impact of rib stiffeners on buckling strength, such as Moore (1990).

Finite Element Solutions

Finite element analysis permits the evaluation of structural stability for more complex (and more realistic) ground support and loading conditions. Moore (1987) considered the impact of the free ground surface above shallow buried structures, Moore (1988) considered buried elliptical structures, and Haggag (1989) and Moore et al. (1994) evaluated critical hoop thrust for circular and elliptical structures in non-uniform elastic ground. Moore et al. (1995 b) used solutions obtained from finite element analysis to identify distress in three deeply buried elliptical structures prior to undertaking repairs.

Model Calibrations

Luscher (1966), Leonards and Stetkar (1978), Gumbel (1983), and Moore (1989) have made comparisons between theory and measured pipe behavior, to evaluate the performance of the theoretical buckling models. Moore (1989) also undertook a calibration exercise to determine performance factors for pipes buried in granular soils. Unfortunately, all of these studies were for small-diameter pipes buried within uniform granular soils. No field tests of large-span culverts have been available for model evaluation.

Tests suitable for model evaluation include those of Luong (1964), Allgood and Ciani (1968), Howard (1972), Gumbel (1983), Trott et al. (1983), and Crabb and Carder (1985). Others might also be used if techniques could be developed to infer soil support conditions used in the tests, such as Dorris (1965), Luscher (1966), Albritton (1968), Gaube (1981), and Bulson (1985).

Design for Buckling

Some design methods make no explicit reference to buckling potential, but rely on empirical backfill specifications to prevent buckling instability (e.g., AASTHO, 1992 and 1994). Some design procedures for small diameter culverts examine buckling using an elastic spring solution, but with constant spring stiffness (e.g., AISI, 1983). Unfortunately, the solution is very conservative for long-span structures. Meyerhof (1966) and Katona et al. (1976 a, b) use Winkler spring models to obtain design predictions for culvert buckling, and the Ontario Bridge Design Code (1991) modifies such predictions using a number of empirical correction factors to account for complexities such as shallow burial. Abdel-Sayed et al. (1992, 1994), Bulson (1972), Gumbel and Wilson (1981), and Moore et al. (1988) present design approaches that use elastic continuum models to predict metal culvert buckling. The latter features consideration of shallow burial and non-uniform thrust. Moore et al. (1994) modified this approach to account for non-uniform soil support. Again, there is a scarcity of field test data to evaluate the performance of these different design approaches.

SOIL BEHAVIOR

The term "soil" in this section is intended to include backfill materials, embankment soil, and undisturbed natural soils, all of which influence the culvert-soil interaction. The properties of these soils to be considered in design and construction of the culvert-soil system are:

- total unit weight,
- stiffness,
- strength, and
- compactibility.

Total unit weight is used together with geometry of the soil zones and degree of arching to define the dead load on the culvert. Representative values are readily available in the literature, and AASHTO, for use in design. Also, total unit weight is normally a required field measurement for assessing compaction during earthwork construction when the culvert is installed (Selig, 1982).

Stiffness is represented by stress-strain-volume change relationships of the soil. These relationships are usually derived from laboratory tests on re-compacted or undisturbed soil samples. Simple linear elastic soil models are commonly used, but nonlinear models are preferred. Nonlinear, stress-dependent representations include hyperbolic models (Duncan and Chang, 1970, Selig, 1988 a).

Representative values of compacted soil stiffness properties are available for culvert design using the hyperbolic model (Selig, 1988 a; Boscardin et al., 1990; Haggag, 1989; Musser, 1989; and Duncan et al., 1980). These values can be used for backfill and embankment materials as an approximation if property tests are not performed on the actual soils. Comparable data for undisturbed in situ soils have not been compiled, but some representative values of linear elastic parameters have been proposed (Heger et al., 1985).

Soil strength influences the ability of shallow earth cover to resist live load induced deformation of the culvert, and resist the high soil pressure at the locations of the small corner radius on flexible pipe arches. Soil strength also controls the bearing capacity of culvert arch foundations. Representative values of strength parameters for unsaturated, compacted soils are available for design from the hyperbolic model (Selig, 1988 a; Boscardin et al., 1990; and Musser, 1989).

Compactibility represents the compaction effort required to achieve a desired soil unit weight and stiffness. For the same effort, the percent compaction achieved varies significantly with the soil type. Granular soils are much easier to compact than silty soils, which are easier to compact than clayey soils. Considerably higher compaction effort is required to obtain a specified percent compaction for clay (CL) than for silt (ML) and, in turn, for sand (SW). What is not universally recognized is that even when the same percent compaction is achieved, the resulting stiffness and strength properties are not the same for all soils. This results in a dramatic difference in stiffness among soils when related to compaction effort.

Soil properties used in culvert design are normally assumed to be static, which is to say that the values at the end of construction are assumed to remain constant over time after construction. In this event, the culvert loads, deformations, stresses, and strains would remain constant. In actual fact, the properties do change for a number of reasons:

- moisture content change,
- freezing and thawing,
- repeated loading,
- consolidation, and
- creep.

These effects are much more significant for fine-grained soils (silts and clays) than for coarse-grained soils (sands and gravels). Thus, the soil properties used in design need to be modified to reflect these changes. In present practice, time effects are not usually considered, except for consolidation associated with design of the culvert foundations and embankment foundations. Time-dependent soil property modeling is much more difficult than static property modeling. However, changes with time are recognized, and at least some simplified approximations need to be developed for culvert design.

COMPUTER MODELING

Computer methods have seen increasing use since the 1960s. This discussion commences with a review of structural analysis methods and those that use elastic continuum theory. Most of the section discusses the numerical procedures based on the finite element method. Lastly, the literature demonstrating the use of computer modeling to examine field test and laboratory test measurements as well as culvert design is described.

Structural Analysis

The first analyses of buried structures used specific simplifying assumptions regarding the distribution of earth pressures (e.g., Spangler, 1956). Structural design is often still performed by prescribing earth pressure distributions on the structure and proceeding with a conventional structural analysis (e.g., McGrath et al., 1988); however, the soil pressures are generally determined considering soil structure interaction.

The simple structural models were subsequently improved using the Winkler model (independent elastic springs) to represent the soil surrounding the structure (e.g., Drawsky, 1966 and Szechy, 1966), who summarized various methods used by Soviet workers. Kloppel and Glock (1970) used similar methods, with modifications. They introduced the concept of separate analysis of the top part of the culvert as an arch, with elastic spring support across its span and rotational and translational springs at the supports. Some workers have continued to focus on the structural behavior using elastic springs to model the soil (e.g. Abdel-Sayed and Girges, 1992).

Elastic Continuum Methods

Various workers who used an elastic continuum representation of the soil recognized the inherent weakness of using elastic springs to model soil behavior, i.e., the culvert-soil interaction is focused through discreet points.

Elastic continuum theory has been used to develop a number of different solutions for predicting the response of buried structures (e.g., Mindlin, 1939), for a circular cavity; Burns and

Richard (1964), Hoeg (1968), and Einstein and Schwartz (1979) for a thin circular tube in elastic continuum; and Rude (1982) for a thick tube in elastic continuum. Katona et al (1976 a, b) used the Burns and Richard (1964) solution as the "Level 1" solution in CANDE.

Krizek et al. (1971) conducted a parametric study using the Burns and Richard (1964) solution to illustrate how the structural response is affected by the hoop stiffness and flexural stiffness, and to demonstrate the importance of interface condition.

These solutions differ in the stress paths. Burns and Richard considered geostatic stresses based on an elastic earth pressure coefficient $K = \nu / (1-\nu)$, while Hoeg's solution provides pipe response for any value of earth pressure coefficient, K. Einstein and Schwartz examined pipe response for burial in a prestressed medium (this is appropriate to a tunneling or pipe jacking problem), while the other workers considered the earth stresses to be applied after burial (better for the culvert burial case). Solutions for these two different load cases are actually related (Gumbel, 1980).

More recent continuum theory contributions include those of Gumbel and Wilson (1981) to account for static and buckling response, Moore (1987) who developed a solution for considering surface loads, and Moore (1990) who developed a three-dimensional form of the Burns and Richard (1964) solution for use in solving problems with three-dimensional features, such as rib stiffeners and for non-uniform continua (soil composed of different zones of backfill material).

Finite Element Analyses

The finite element method has the potential to consider a variety of geometric and material issues associated with long-span culvert problems. This has been recognized by various different workers who have developed and/or used such procedures to consider the impact of culvert shape, the different soil zones around the structure, the construction conditions, culvert response to live loads at the ground surface, and the impact of nonlinear soil, structural, and interface behavior.

Elastic Soil Structure Interaction

Elastic finite element analysis appears to have first been used by Nataraja (1973) to examine the stresses and displacements around a flexible culvert structure. Other use includes the work of Abel et al. (1973) and Moore (1988 a) to examine elliptical culverts.

McVay (1982) showed that linear elastic analysis works well to predict the stress distribution in the soil mass, and the thrust in the culvert structure. However, it was demonstrated that linear elasticity did not adequately predict culvert movements and soil strains, and this was attributed to the absence of considerations of shear failure. Leonards et al. (1982) stressed that significant errors result from the use of linear elasticity.

Nonlinear Analysis – Finite Element Codes

A number of workers have developed finite element codes specifically for analysis of culverts:

- CANDE: the code developed by Katona et al. (1976 a, b) for both flexible and rigid culverts,
- NLSSIP: the code developed for flexible culverts, Byrne and Duncan (1979), and
- SPIDA: a modified form of NLSSIP developed for concrete culverts, Heger et al. (1985).

These codes generally include the impact of construction procedure, culvert and soil geometry, material nonlinearity, and nonlinear soil structure interaction (Leonards et al., 1982).

Other general purpose nonlinear finite element codes also exist (e.g., ABAQUS, Hibbit et al., 1988, AFENA, Carter, 1992, and ADINA, Bathe, 1995) and can be used to undertake nonlinear finite element analysis of culverts, though these are primarily research tools rather than software suitable for use in routine culvert design.

Nonlinear Analysis – Soil Modeling

The finite element method is ideal for representing the various regions of soil used in the vicinity of a long-span culvert, and for considering the nonlinear soil response. McVay

(1982) and Leonards et al. (1982) have presented studies evaluating different soil models in use. Vaslestad (1990) presents a useful literature review.

A number of different nonlinear constitutive laws have been employed:

- Overburden dependent modulus: this approach uses linear elasticity in a series of steps, with modulus adjusted based on the overburden stress (Katona et al., 1976 a, b). These models are only valid while soil is in a state of confinement, so that shear failure does not occur (Vaslestad, 1990). McVay (1982) and Leonards et al. (1982) all concluded that this approximation was still unable to provide reasonable predictions of culvert deformations, since shear failure is not modeled.

- Bilinear model: McVay (1982) introduced a bilinear elastic approximation, where low modulus is used following shear failure. Elastic modulus is chosen as a function of confining stress (Janbu, 1963). McVay (1982) reported that this bilinear approximation provided reasonable deformation predictions for a long-span metal culvert. The zone of yielded soil predicted using this approach is greater than that predicted using other methods (Vaslestad, 1990).

- Hyperbolic elastic model: Duncan and Chang (1970) developed hyperbolic stress-strain relationships for analysis of embankments. This has also been used extensively in computer analysis of structures (e.g., Katona et al., 1976 a, b and Duncan, 1979).

- Extended Hardin model: Katona et al. (1976 a, b) implemented an extended form of the Hardin model in CANDE. This model uses relationships between shear stress and shear strain, as opposed to nonlinear Young's modulus used in the hyperbolic formulation. Chang et al. (1980) examined performance of the extended Hardin model in predictions of behavior of a long-span steel arch culvert. It was concluded that the model overpredicted soil stiffness, and that there were convergence problems for granular soils. Leonards et al. (1982) also concluded that the Hardin model was unconservative at high levels of shear strain.

- Modified hyperbolic model: Selig (1988 a) and his co workers have modified the hyperbolic model to provide more consistent predictions of soil deformations in fundamental stress states associated with triaxial compression, hydrostatic compression, and 1D compression. The modified hyperbolic model has been incorporated in CANDE (Musser, 1989).

- Elasto-plastic soil models: General purpose finite element codes generally include elasto-plastic soil models based on either the Drucker Prager or Mohr Coulomb failure criteria, together with either an associated or non-associated flow rule (e.g., Carter, 1992). Other models being used include cap models (Lade, 1977). Often, they are used together with stress-dependent elastic modulus to improve predictions for granular soils (e.g., Brachman et al., 1996 b).

- Geometrically nonlinear soil behavior: The procedure of Carter et al. (1978) has been used by some workers to account for deformations on materially nonlinear soil response. General purpose finite element codes generally feature a procedure for considering the geometrically nonlinear soil response (e.g., Hibbit et al., 1988).

Nonlinear Analysis – Structural Models

A variety of pipe and culvert models have been used to obtain finite element solutions:

- Elastic beam theory: most workers employ simple beam theory based on the assumption that the structure can be modeled as a thin elastic shell.
- Multilinear beam theory: the nonlinear material response of the structure can be modeled by adjusting the effective area and second moment of area of the structural cross section (Katona et al., 1976 a, b). This model still assumes that the structure is thin, neglecting transverse stress and strain, and shear strains. This is generally implemented using a bilinear model for metals, and material response for reinforced concrete is modeled by treating it as a composite composed of two different nonlinear materials.
- Degenerated shell theory: the behavior of thicker structures can be evaluated using degenerated shell theory (e.g., Bathe and Balourchi, 1980). This can be implemented using either linear elastic or nonlinear material models (e.g., Teng and Rotter, 1989). It can be formulated to consider structures with complex profiles.
- Buckling and geometrical nonlinearity: the effects of thrust on the structural response can be accommodated considering beam column elements (Moore, 1987), or using materially nonlinear structural models (e.g., El Sawy, 1996). Either updated Lagrangian or total Lagrangian solution schemes can be employed to take account of changes in structural geometry. Section 2.9 includes a discussion of finite element analysis of the buckling of flexible culverts.

Nonlinear Analysis – Compaction

A few workers have tackled the difficult problem of modeling compaction. Katona (1976) introduced temporary surface tractions to model soil compression to simulate compaction-induced deformations in long-span metal culverts. Seed and Duncan (1984) developed an elaborate "semi-empirical" stress correction model to accomplish the same thing. McGrath and Selig (1994) introduced additional lateral forces on the structure to simulate compaction effects on glass-reinforced plastic pipe.

Three-Dimensional Analysis

While computer analysis for culverts has been predominantly two-dimensional in nature, some workers have used general finite element codes to undertake full three-dimensional analysis to examine three-dimensional issues of concern, such as finite culvert length, and the impact of three-dimensional surface live loads, such as Wei Cao (1993) and

Girges and Abdel-Sayed (1995), who examined the response of finite lengthened culverts to surface live load, and Selvadurai (1989), who studied pipeline response to non-uniform ground deformations. Unfortunately, the cost of this analysis and the geometrical scale of culvert problems make it difficult to solve with sufficient numerical accuracy, and analysis can feature very crude choices for finite element mesh.

Moore and Brachman (1994) recently developed an alternative approach by using Fourier methods to determine three-dimensional elastic response of culverts to surface live load. This approach has the advantage of using a two-dimensional finite element mesh and is considerably more efficient than conventional three-dimensional procedures; however, it is restricted to linear problems where culvert or pipeline response is not affected by the ends of the structure.

Time-Dependent Response

Specialized culvert analysis codes do not include procedures to predict the time-dependent response of long-span culverts. Such procedures can be used to estimate long-term response by simply adjusting the material parameters. Chua and Lytton (1991) modified CANDE to include modeling of visco-elastic material response, and used the procedure to examine the impact of visco-elastic polyethylene and visco-elastic soil behavior. Other related work has been performed by Moore and Hu (1995). Some general purpose codes include visco-elastic and visco-plastic soil models, and consolidation and critical state soil models that might be used to predict time-dependent response for culverts within fine grained soils (e.g., Carter, 1992). Some workers have examined dynamic culvert response (e.g., Byrne et al., 1996).

Use of Computer Analysis in Design

A number of different workers report on the use of computer analysis in design. Krizek et al. (1971) then Kay and Abel (1976) reported on the use of elastic continuum solutions for buried pipe design; Katona et al. (1976) introduced CANDE as a numerical tool for use in culvert design, and subsequently used CANDE to examine the design of reinforced concrete culverts (Katona, 1976, Katona and Vittes, 1982); pipe with slotted joints (Katona and Akl, 1987 a, b); and buried polyethylene pipe (Katona, 1988); Duncan (1978, 1979) developed his SCI design method for long-span metal culverts using finite element analysis, and later a design method for metal box culverts using the same numerical tools (Duncan et al., 1985); Heger (1982), Packard

(1982), and Heger et al. (1985) used finite element analysis to consider the design of buried concrete pipes; Moore et al. (1988) provide a design procedure to control metal culvert buckling based on finite element analysis; Haggag (1989) used finite element analysis to consider the design of the structural backfill envelope around long span metal culverts.

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APPENDIX F

PROPOSED DESIGN SPECIFICATIONS AND COMMENTARY FOR LARGE-SPAN CULVERTS

APPENDIX F – PROPOSED DESIGN SPECIFICATIONS AND COMMENTARY FOR LARGE-SPAN CULVERTS

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PROPOSED DESIGN SPECIFICATIONS AND COMMENTARY FOR LARGE-SPAN METAL CULVERTS

SPECIFICATIONS

1. Limit States, Load, and Resistance Factors

1.1 Service Limit States

Long-span culverts shall be investigated at Service Load Combination I, as specified in AASHTO LRFD Table 3.4.1-1 as follows:

- Deformations during construction and under service loads

1.2 Strength Limit States

Long-span culverts shall be investigated for construction loads and at Strength Load Combinations I and II, as specified in AASHTO LRFD Table 3.4.1-1.

1.2.1 Flexure Related

Flexural forces shall be evaluated for the following conditions:

- earth load
- construction effects
- live load

For load conditions where live or other transient loads produce flexural stresses less than 15% of the plastic moment capacity of the section, the flexure limit state need not be evaluated.

COMMENTARY

C1.

Long-span metal culverts are defined as structures with spans greater than 4.5 m (15 ft) or any size culvert with longitudinal or transverse stiffeners.

C1.1

Limit states on deformation are set based on flexural strength limit states, but are easily monitored in the field and become an excellent method for establishing field quality control and monitoring structural safety.

C1.2

Large-span culverts are flexible. Ensuring that the structures have adequate strength and stiffness to resist construction loading is a significant design requirement.

C1.2.1

This provision eliminates consideration of flexural stresses for installations with deep cover. Construction loading conditions will still require evaluating flexural stress for all long-span culvert designs.

SPECIFICATIONS

COMMENTARY

1.2.2 Thrust Related

Hoop compression thrust forces must be evaluated for the following conditions:

- yielding
- seam strength
- general buckling

C1.2.2

Yielding and seam strength are the traditional limit states for long-span culverts. General buckling is new for AASHTO.

1.2.3 Combined Flexure and Axial Forces

The combined effects of flexure and axial forces must not exceed the capacity of the section. For load conditions where the live or other transient loads produce flexural stresses less than 15% of the plastic moment capacity of the section, the combined flexure and axial force limit state need not be evaluated.

C1.2.3

This limit state is common in steel design, and has been considered in the past by some designers of long-span culverts. This limit state controls designs under shallow fills where thrust and flexure forces are both significant.

1.2.4 Geotechnical Considerations

The following limit states related to soil capacity shall be considered in design:

- Foundation capacity
- Bearing capacity at small radius locations

C1.2.4

Foundation design should be addressed in accordance with Section 10.

Bearing capacity at small radius locations can be evaluated directly in the comprehensive design method. In simplified design, this limit state is handled through limits on ratio of the radii of adjacent sections or with backfill controls.

1.3 Load and Resistance Factors

C1.3

SPECIFICATIONS

1.3.1 Load Factors

Load factors for earth and live loads shall be taken as (For Section 3 of LRFD):

- Earth γ_E = 1.3 max
  = 0.9 min
- Live γ_L = 1.75

1.3.2 Resistance Factors

Resistance factors for long-span culverts shall be taken as:

- culvert material yield strength ϕ_c = 0.7
- seam strength ϕ_{sm} = 0.7
- bending ϕ_b = 0.9
- buckling capacity ϕ_{bck} = 0.7
- soil stiffness ϕ_s = 0.9

1.4 Flexibility Limits and Construction Stiffness

The flexibility factor of the top and side plates of long span culverts shall not exceed the value:

$$FF \leq 0.17 \text{ mm/N (30 in./kip)} \tag{1.4-1}$$

For long-span culverts, the flexibility factor is computed as:

$$FF = \frac{(2\,R)^2\ (1 - \sin\phi_{loose})^3}{0.07\,E_p\,I_p} \tag{1.4-2}$$

where:

FF = flexibility factor for the stiffened structure, mm/N, in./k

R = radius of top or side plates of culvert, m, in.

COMMENTARY

C1.3.1

Traditionally, load factors for metal culverts have been on the order of 2.0, while load factors for concrete culverts have been on the order of 1.3. The basis for these different values is not clear, and the new values have been set to bring uniformity to culvert design practice. The new load and resistance factors together have been calibrated in part to produce the same overall safety as traditional design methods.

C1.3.2

C1.4

The flexibility factor has not previously been applied to long-span culverts because the role of longitudinal and circumferential stiffeners in controlling construction deformations was not defined. The new provisions assign specific roles to these stiffeners, thus allowing the flexibility factor to be used for this purpose. In addition, a term to account for the increased compactive effort required for soils with low friction angles (moderate fines content) is included. The limiting value of 0.17 mm/N (30.0 in./k) is the value for steel structural plate with 150 mm x 50 mm (6 in. x 2 in.) corrugations that has been in the AASHTO Specifications for some time.

SPECIFICATIONS

ϕ_{loose} = friction angle of structural backfill in loose condition; in lieu of specific data, use 36° for A-1 or A-3 backfill, and 28° for A-2-4 and A-2-5 backfill.

E_p = modulus of elasticity of culvert material, MPa, ksi,

I_p = effective moment of inertia of stiffened top or side plates, m^4/m, $in.^4/in.$

For culverts stiffened with circumferential stiffeners, I_p should be taken as the average I considering the effect of the stiffener, which may be intermittent. See Section 2.8.1 for additional information on circumferential stiffeners.

For culverts with longitudinal stiffeners, I_p may be modified to account for the length of structure mobilized by the stiffener. The design must include the basis for determining the mobilized length. See Section 2.8.2.

Shapes such as high profile arch and pear shapes have large radius side plates. These side plates need not be more than one gage thickness heavier than the top plates, provided that construction controls are implemented to minimize deformation during backfilling.

COMMENTARY

The term $(1 - \sin \phi)^3$ is an empirical estimate of the lateral pressure at rest raised to the third power. This representation of compaction force is empirical, based on McGrath, et al. (1998). The term represents the increased lateral deformations that take place when backfill with low friction angles is compacted. The use of this term will require greater structural stiffness when using lower quality backfill materials.

Forces applied to culverts while placing and compacting backfill at the sides are very localized and result in deformations of the culvert, usually upward peaking of the crown. When a culvert is stiffened with a longitudinal stiffener, the compaction forces are spread out mobilizing a longer length of culvert and the deformation is reduced. If justified through analysis, this stiffening effect may be represented in design by increasing the moment of inertia of the structural plate, I_p.

The provision to relax the flexibility criterion for side plates of high profile sections continues current practice. These sections are known to be more sensitive to backfill forces than typical long span shapes, and construction controls are important.

SPECIFICATIONS

1.5 Minimum Plate Thickness

Structural plate used for large-span culverts shall not be less than that specified in Table 1.4-1. The minimum plate thickness is required around the entire culvert. Where design calls for more than the minimum thickness in the side or top plates, the change in thickness at the junction of any two plates shall not exceed one gage thickness.

Table 1.4-1
Minimum Structural Plate Thickness
for 150 mm x 50 mm (6 in. x 2 in.) Corrugations

Plate Radius m (ft)	Minimum Plate Thickness mm (in.)
< 5.2 (15)	2.82 (0.110)
5.2 to 6.1 (17 to 20)	3.56 (0.140)
6.1 to 7.0 (20 to 23)	4.78 (0.188)
7.0 to 7.6 (23 to 25)	5.54 (0.218)
7.6 to 10 (25 to 32)	6.32 (0.249)

COMMENTARY

C1.5

Some culvert failures have been attributed to stress concentrations at sudden changes in stiffness. This provision controls the change in stiffness around the perimeter of the culvert.

Previous AASHTO Specifications used the minimum thickness specification as a means of setting the minimum gage at the minimum depth of fill. In the current specifications, installations at the minimum depth of fill will likely require a greater thickness than the minimum.

For sizes not considered in Table 1.4-1, a reasonable alternate approach to determining the minimum thickness is to verify the capacity of the culvert to carry construction vehicles at a depth of 1.2 m.

SPECIFICATIONS

2. General Features of Design

2.1 Backfill

2.1.1 Backfill Types

The type, compacted unit weight, and strength properties of the soil envelope adjacent to the buried structure shall be established. As a minimum, the bedding and backfill materials shall meet the requirements of AASHTO M145 for A-1, A-2-4, A-2-5, or A-3 soils. Frost-susceptible soils shall not be used for backfill where ice lens formation is possible. Further restrictions on granular backfill are:

COMMENTARY

C2.

C2.1

C2.1.1

The restriction on materials passing the 0.150 mm (No. 100) sieve and the 0.075 mm (No. 200 sieve) is intended to eliminate soils composed of significant amounts of fine sands and silts. These materials are difficult to work with, sensitive to moisture content, and do not provide support comparable to coarser or more broadly graded materials at the same percentage of

SPECIFICATIONS	COMMENTARY
a) For all structural plate sizes, a maximum of 50% of the particle sizes may pass the 0.150 mm (No. 100) sieve, and a maximum of 20% may pass the 0.075 mm (No. 200) sieve. If the engineer approves use of A-1 or A-3 soil not meeting these criteria, design shall be based on Si type soils; See Table C2.3-1.	maximum density. This includes some A-1-b, A-3, A-2-4, and A-2-5 soils. A-2-6 and A-2-7 soils display similar characteristics and are also eliminated from use as backfill materials. The engineer may permit exceptions to these restrictions in special cases. If so, a suitable plan must be submitted for control of moisture content and compaction procedures. These silty and clayey materials should never be used in a wet site. Increased inspection levels should be considered if such a plan is approved.
b) For long-span sizes of structural plate: • A-1-b may be used only for high profile arches and pear shapes with depth of fill over the top of the culvert up to 4 m (12 ft), and for low profile and elliptical structures with depths of cover up to 6 m (20 ft). • A-2-4 and A-2-5 materials are restricted to structures with depths of fill over the top of the structure of less than 4 m (12 ft), and are not allowed for shapes with large radius side plates (pear, pear arch, and high profile arch).	
Controlled low strength material (CLSM) may be used as backfill if the mix design provides properties equivalent to those of the acceptable soil backfills listed above.	CLSM (also known as flowable fill along with several other names) is a mixture of sand, water, cement, and a fluidizing agent (fly ash or other additives that produce good flow characteristics). Cement contents can be on the order of 30.0 kg/m^3 (50 lb/yd^3). After setting, strengths are high relative to compacted soil and low relative to concrete.

SPECIFICATIONS

If CLSM backfill is not extended to the top of the culvert, then the design shall consider the effects of high local stresses at the elevation where the CLSM terminates.

COMMENTARY

Metal culverts deflect under load. A sudden transition from a backfill with high stiffness, such as CLSM, to one with low stiffness could result in high local bending. Generally, CLSM should not be terminated in areas where the culvert is moving outward significantly, as the sudden change in support may lead to large bending moments.

2.1.2 Soil Properties

For purposes of computing load, unit weight of the soil placed over the culvert should be determined for use in design. In the absence of actual data, the unit weight shall be estimated per Table 3.5.1-1 of these specifications.

C2.1.2

2.2 Minimum Spacing between Multiple Lines of Culverts

When the distance between adjacent culverts is less than 0.25 times the span, backfill used in the space shall be A-1 or A-3 soil, CLSM, or concrete. The space must always allow adequate space for placing and compacting backfill. The clear space between long-span culverts shall not be less than 1.5 m.

If the space between the culverts is filled with concrete or CLSM, then the elevation at which these materials are terminated should be set to avoid the introduction of stress concentrations.

C2.2

This provision parallels the general provision for backfill with CLSM in Section 2.1.1. Also see the Commentary to Section 2.1.1

2.3 Width of Structural Backfill

Structural backfill shall extend outward sufficiently at the sides of the culvert to ensure proper structural support for the culvert.

C2.3

SPECIFICATIONS	COMMENTARY

If the native or embankment material outside of the structural backfill has a stiffness, M_{s-N}, equal to or greater than 90% of the stiffness of the structural backfill, M_{s-SB}, then the width of the structural backfill may be reduced to the minimum that allows proper placement and compaction of the backfill, but not less than the larger of 0.2 times the span or 1 m (3 ft) at the widest part of the culvert.

When the stiffness of the native or embankment material is at or near the stiffness level of the structural backfill, then constructability is the only concern for the width of structural backfill. Guidance for stiffness of backfill materials and undisturbed native materials is provided in Tables C2.3-1 and C2.3-2.

If the native or embankment material outside of the structural backfill has a stiffness, M_{s-N}, equal to or greater than 90% of the stiffness of the structural backfill, M_{s-SB}, the width of structural backfill, W, used in the design of the culvert may be taken as 1.0 times the culvert span, and the backfill modulus used in the design method may be that of the structural backfill.

If vertical wall trenches are used it is likely that a temporary support system will be used. If this system is removed, the backfill and structural support to the culvert must not be disrupted.

Table C2.3-1
Constrained Modulus for Structural Backfill Materials, M_{s-SB}, MPa

Stress Level (kPa)	Soil Type and Compaction Condition									
	Sn100	Sn95	Sn90	Sn85	Si95	Si90	Si85	Cl95	Cl90	Cl85
7	16.2	13.8	8.8	3.2	9.8	4.6	2.5	3.7	1.8	0.9
35	23.8	17.9	10.3	3.6	11.5	5.1	2.7	4.3	2.2	1.2
70	29.0	20.7	11.2	3.9	12.2	5.2	2.8	4.8	2.4	1.4
140	37.9	23.8	12.4	4.5	13.0	5.4	3.0	5.1	2.7	1.6
280	51.7	29.3	14.5	5.7	14.4	6.2	3.5	5.6	3.2	2.0
420	64.1	34.5	17.2	6.9	16.4	7.7	4.8	6.2	3.6	2.4

1. 1.0 MPa = 145 psi; 1.0 kPa = 0.145 psi
2. Soil types are defined in Table 27.5.2.2-3 of the LRFD Construction Specifications, which need to be incorporated into the design specifications. Compaction levels are percent of maximum density per AASHTO T99.
3. Modulus values are secant moduli for stress variations from unstressed to the indicated stress level.

Table C2.3-2
Constrained Modulus for Native Soils, $M_{s\text{-}N}$
Ref. AWWA Manual M45 *Fiberglass Pipe Design*

In Situ Soil Type				$M_{s\text{-}N}$ (MPa)
Granular		Cohesive		
Blows/0.3 m	Description	q_u (MPa)	Description	
> 0-1	very, very loose	> 0-0.012	very, very soft	0.4
1-2	very loose	0.012-0.025	very soft	1.5
2-4		0.025-0.050	soft	5
4-8	loose	0.050-0.100	medium	10
8-15	slightly compact	0.100-0.200	stiff	20
15-30	compact	0.200-0.400	very stiff	35
30-50	dense	0.400-0.600	hard	70
> 50	very dense	> 0.600	very hard	140

1. q_u = unconfined compressive strength of soil
2. 1.0 MPa = 145 psi; 1 m = 3.28 ft

If the stiffness of the native or embankment material outside of the structural backfill zone is less than 90% of the stiffness of the structural backfill, W shall be taken as the actual width of backfill at the midrise of the structure, and the modulus of the structural backfill shall be computed as:

$$M_s = S_c \; M_{s\text{-}SB} \qquad (2.3\text{-}1)$$

where:

W = width of structural backfill at the midrise of the culvert, m, ft; W shall not be less than the minimum values set above

S_c = coefficient based on width and modulus of structural backfill and native or embankment material beyond the structural backfill. See Table C2.3-3.

If the stiffness of the native or embankment material at the sides of the culvert is soft then the stiffness of the structural backfill used in the design must be reduced. The proposed equation is taken from AWWA Manual M45, *Fiberglass Pipe Design*, which is considered conservative. In the absence of actual data on soil stiffness, see Tables C2.3-1 and C2.3-2

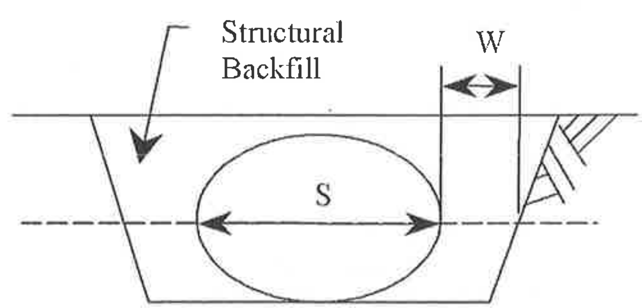


Figure C2.3-1 – Width of Structural Backfill

SPECIFICATIONS

M_s = constrained modulus of soil used in design equations, MPa, ksi

$M_{s\text{-}SB}$ = constrained modulus of structural backfill, MPa, ksi

$M_{s\text{-}N}$ = constrained modulus of native soils at sides of structural backfill, MPa, ksi

COMMENTARY

Table C2.3-3 S_C Values for Modifying M_s to Consider Stiffness of Native Soil Ref. AWWA Manual M45 – *Fiberglass Pipe Design*

M_{s-N}/M_{s-SB} \ W/S	0.25	0.50	0.75	1.00
0.1	0.15	0.30	0.60	1.00
0.2	0.30	0.45	0.70	1.00
0.4	0.50	0.60	0.80	1.00
0.6	0.70	0.80	0.90	1.00
0.8	0.85	0.90	0.95	1.00
1.0	1.00	1.00	1.00	1.00

2.4 Foundation Design

Footings for culverts shall be designed in accordance with Section 10. Analysis shall include design for soil bearing capacity and evaluation of potential for longitudinal or transverse differential settlement.

C2.4

2.5 Soil Bearing at Changes in Plate Radius

The ratio of the radius of adjacent structural plate sections, taken as the ratio of the larger radius to the smaller radius, shall not exceed 5.

C2.5

Large changes in the radius of adjacent structural plates result in high local soil stresses. Large ratios should be avoided, particularly under deep fills.

2.6 Minimum Cover

When subjected to live loads, depth of fill over the top of a long-span culvert must be at least 0.1 times the span, but not less than 0.6 m (2 ft), provided all provisions of these design specifications are met.

C2.6

Issues to be considered for culverts with low burial depths include upward heave of the culvert, cyclic fatigue stresses, and pavement durability. Designers should consider increasing the minimum cover for axle loads greater than 180 kN (40 kips).

2.7 Skewed Ends

C2.7

The treatment of skewed ends will be based on existing provisions.

SPECIFICATIONS

2.8 Special Features

Special features may consist of circumferential stiffeners, longitudinal stiffeners, or other structural features that will ensure proper structural performance under all design load conditions.

2.8.1 Circumferential Stiffeners

Circumferential stiffeners consist of structural members mounted parallel to the culvert span. Any assumption of composite action must be demonstrated by test or calculation. In the absence of composite action, the moment of inertia of the stiffened structure per unit length is the sum of the unit moment of inertia of the structural plate, plus the moment of inertia of the stiffener divided by the stiffener spacing.

Spacing of circumferential stiffeners shall not be greater than 750 mm (30 in.) if designed to resist live loads after construction or 1,500 mm (60 in.) if designed only for shape control during backfilling.

2.8.2 Longitudinal Stiffeners

Longitudinal stiffeners consist of continuous structural elements attached along the length of the culvert, typically at the junction of the top and side plates. Longitudinal stiffeners assist in shape control during backfilling by increasing the effective length of culvert that resists compaction forces. Designers must demonstrate the effective length assumed for a longitudinal stiffener. See Section 1.3. Longitudinal stiffeners do not contribute to flexural strength to resist earth, live, or construction loads.

COMMENTARY

C2.8

C2.8.1

Circumferential stiffeners are normally used to increase the flexural stiffness and strength. While they can also be designed to increase the area to resist thrust, circumferential stiffeners are normally discontinuous and do not provide a complete load path for such application. There is evidence that stiffeners bolted to the structural plate do not provide composite action (Byrne, 1997, and McCavour, et al., 1998). This includes assumptions of the transfer of horizontal shear at the bolted interface and the effective width of the plate acting as a flange for the stiffener.

C2.8.2

Longitudinal stiffeners contribute to the resistance of construction loads by increasing the longitudinal distribution of loads when placing and compacting backfill.

SPECIFICATIONS

2.9 Concrete Relieving Slabs

Concrete relieving slabs may be placed over the top of the culvert to increase the lateral and longitudinal distribution of live loads. Use of relieving slabs to control loads on culverts must include provisions for long-term maintenance of the slab.

The minimum clear distance from the bottom of the slab to the top of the culvert shall be 150 mm (6 in.).

The slab shall extend to at least 300 mm (12 in.) beyond each side of the widest part of the culvert.

The basis for the design of concrete relieving slabs shall be to consider the load case of a concentrated load on a slab on grade distributing the effect of a wheel load over a broader area.

Minimum thickness of relieving slabs shall be 250 mm (10 in.) for loads up to the magnitude of the Design Truck or the Design Tandem. Slab design shall include consideration of durability.

COMMENTARY

C2.9

Since relieving slabs are part of the structural system for a culvert, deterioration of the slab may have serious consequences for the culvert performance. Slab durability is an important issue.

This minimum cover limit is somewhat arbitrary. Current provisions for metal box sections state that "as little as 25 mm to 75 mm (1 in. to 3 in.) clearance is thought to be sufficient." The proposed value is increased to consider uncertainties in construction control of the actual depth of fill.

It is not necessary to design relieving slabs as spanning the entire structure. Procedures for distribution of live loads on rigid pavements (Westergaard, 1926) should be conservative. Procedures presented in the Concrete Pipe Handbook (ACPA, 1988) should be appropriate to consider the load effect on a culvert under a rigid pavement. ACI 360 and AASHTO requirements for design of concrete pavements should be considered for slab design.

Current AASHTO provisions for metal box sections allow relieving slabs as thin as 190 mm (7.5 in.). A cast-in-place slab requires 150 mm (3 in.) of clear cover on the bottom reinforcement. In a 190 mm (7.5 in.) slab, this leaves a depth to reinforcement, d, of about 100 mm (4 in.). A thicker slab is warranted.

SPECIFICATIONS

2.10 Hydraulic Protection

2.11 Keyhole Slotted Joints

Keyhole slotted joints may be used to allow long-span culverts to decrease in circumference under deep earth fills. The design for culverts with keyhole slotted joints cannot be completed with the simplified methods proposed herein and should be performed using a comprehensive analysis.

2.12 Camber

Where settlement of the culvert is expected to be such that the required grade under high fills will not be maintained after construction, the culvert may be cambered to prevent excessive sag. The amount of camber shall be determined based on consideration of the flow line, gradient, fill height, the compressive characteristics of the foundation material, and the depth to incompressible strata. The use of camber under a high fill is shown in Figure 26.6. This is not typically done for long-span culverts.

COMMENTARY

C2.10

No change proposed from current AASHTO provisions.

C2.11

The use of keyhole slotted joints allows the circumference of the culvert to shorten under deep earth fills, which can greatly reduce thrust forces (Katona and Akl, 1987).

C2.12

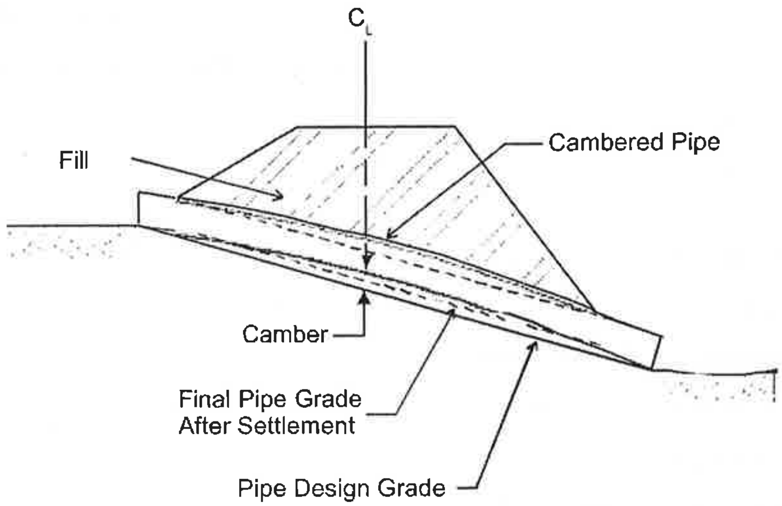


Figure 2.12-1 – Pipe Camber for Settlement Control under High Fills

3. Design

3.1 Load

3.1.1 Earth Load

Compute the earth load on the structure as the weight of the soil directly over the culvert modified by a vertical arching factor:

$$W_{SP} = \gamma_s \; S \; (H + K_{VAF} \; R_u) \qquad (3.1.1\text{-}1)$$

$$W_E = VAF \; W_{SP} \qquad (3.1.1\text{-}2)$$

$$VAF = F_{W/S} + F_{S/R} + F_{H/S} \qquad (3.1.1\text{-}3)$$

The three components of the VAF are:

$$K_{W/S} = (1.9 - 1.15\frac{W}{S}) \geq 1.2 \qquad (3.3.1\text{-}4)$$

$$F_{W/S} = 1.2 + 0.5 * \log\left(\frac{M_{s-Side}}{M_{s-N}}\right)(K_{ws} - 1.2) \qquad (3.3.1\text{-}5)$$

$$F_{S/R} = 1 - \frac{S}{R} \geq 0 \qquad (3.3.1\text{-}6)$$

C3.

C3.1

C3.1.1

For consistency with other culvert specifications (concrete pipe), the earth load is stated in terms of the soil prism load and the vertical arching factor.

Selection of the soil unit weight should consider the effect of groundwater as well as soil weight.

The term $F_{W/S}$ accounts for arching of load from soft in situ or embankment soils onto the structural backfill and thus, onto the culvert. The maximum value of $F_{W/S}$ is limited to 1.67 by the requirement for a minimum width of structural backfill of 0.2 S. In computing the log function, the ratio $M_{s\text{-}Side}/M_{s\text{-}N}$ need not be taken larger than 100.

The term $F_{S/R}$ accounts for increased arching in culverts where the rise is greater than the span.

SPECIFICATIONS

$$F_{H/S} = 2.5\left((H/S)_{lim} - \frac{H}{S} \right) \geq 0 \qquad (3.3.1\text{-}7)$$

$$(H/S)_{lim} = 0.8 - 0.5\frac{S}{R} \geq 0.3 \qquad (3.3.1\text{-}8)$$

where:

W_{sp} = weight of soil directly over culvert, kN/m, k/ft

γ_s = unit weight of soil, kN/m^3, k/ft^3

S = outside span of culvert, m, ft

R = culvert rise, m, ft

H = depth of fill over top of culvert, m, ft

R_u = vertical rise of culvert from the point of maximum span to the top of the culvert, m, ft

W_E = total earth load on culvert, kN/m, k/ft

K_{VAF} = $0.172+0.019*S/R_u$, factor to account for culvert shape

VAF = vertical arching factor to account for soil-structure interaction

W = width of structural backfill at the midrise of the culvert, m, ft

$F_{W/S}$, $F_{S/R}$, $F_{H/S}$, $(H/S)_{lim}$, $K_{W/S}$
= factors contributing to the design value for VAF

$M_{s\text{-}Side}$ = constrained modulus of structural backfill at the springline of the culvert, MPa, psi

$M_{s\text{-}N}$ = constrained modulus of native soil at the springline of the culvert, MPa, psi

COMMENTARY

The term $F_{H/S}$ accounts for the increased arching that occurs for culverts under shallow fill.

SPECIFICATIONS

3.1.2 Live Load

Compute the live load on a unit length of the structure due to a wheel as:

$$W_{LL} = \frac{0.7\, m\, I\, P\, R_t}{L_L\ L_W} \qquad (3.1.2\text{-}1)$$

$L_L = L_o + LLDF\,(H)$

$L_W = W_t + LLDF\,(H)$

where:

W_{LL} = live load on structure, kN/m, k/ft

m = multiple presence factor. For culverts, the single loaded lane value of 1.2 always controls; see AASHTO LRFD Section 3.6.1.1.2.

P = axle load, or for tandem axles, the load on both axles, kN, k

I = 1 + IM/100, IM is the dynamic load allowance for culverts determined in accordance with AASHTO LRFD, Section 3.6.2.2

R_T = top radius, m, ft

L_L = distribution width parallel to span, m, ft

L_W = distribution width perpendicular to span, m, ft

L_o = length of tire footprint, m ft

W_t = width of wheels on an axle, m, ft

LLDF = factor for distributing live load with depth of fill per Section 3.6.1.2.6

H = depth of fill over top of culvert, m, ft

COMMENTARY

C3.1.2

The expression for live load, W_{LL}, was developed based on a parametric study of long-span culverts (NCHRP Project 12-45, Final Report), which shows that under shallow fills, live loads produce large thrusts directly under the wheel.

Live load, W_{LL}, is calculated based on axle loads, either single or tandem. Thrust due to live load is high at the crown, but spreads quickly, and is low at the springlines. Thus, design for thrust alone under deep fills need not consider the contribution of live load.

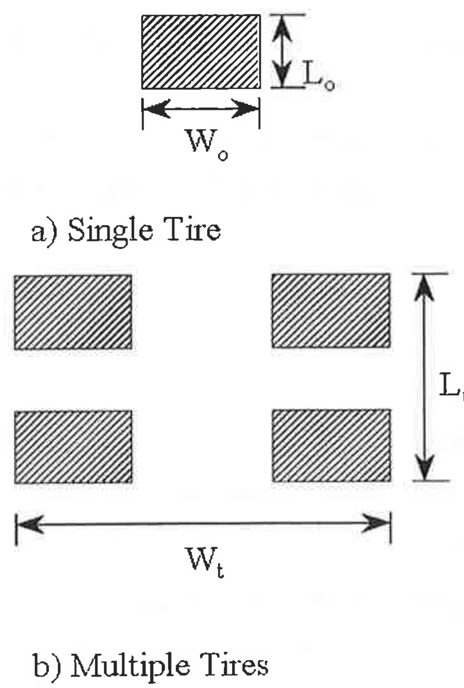


Figure C3.1.2-1 – Tire and Wheel Dimensions

SPECIFICATIONS

The lane load may be treated as a uniformly distributed load:

$$W_{Lane} = \text{Lane load} \frac{Lane_w}{Lane_w + LLDF(H)} \quad (3.1.2\text{-}2)$$

where:

W_{Lane} = magnitude of lane load per unit length of culvert, kN/m, k/ft

Lane load= lane load, kN/m, k/ft, of lane length

$Lane_w$ = width of lane load (3 m, 10 ft)

H = depth of fill over crown, m, ft

COMMENTARY

The lane load is to be considered for all live load conditions, except construction vehicles. Since the live load is governed by a single lane condition, the lane load is allowed to spread laterally with increasing depth of fill.

The calculation of W_{Lane} assumes the typical case of a culvert running perpendicular to a roadway.

3.1.3 Other Loads

3.1.3.1 Seismic

Seismic loads need not be applied to the culvert barrel, unless warranted by special conditions of a specific site. Headwalls and wingwalls, and culverts that change direction, should be evaluated for seismic loading.

C3.1.3

C3.1.3.1

Culverts generally perform well in seismic events. Culverts that cross, or are unusually close to a fault may require special design. The Multidisciplinary Center for Earthquake Engineering Research at SUNY Buffalo has recently published the monograph *Response of Buried Pipelines Subject to Earthquake Effects,* which discusses many issues pertinent to seismic effects on culverts.

3.2 Thrust

Factored thrust must be calculated at the springline:

$$T_f = \frac{\gamma_E W_E + \gamma_L W_{LL} + \gamma_L W_{Lane}}{2} \quad (3.2\text{-}1)$$

C3.2

Thrust is maximum at the springline. This thrust is used to evaluate section capacity under compression alone. At the crown and shoulder, where moments and thrusts interact, the thrust is reduced.

SPECIFICATIONS

COMMENTARY

The maximum factored thrust at the crown or shoulder is used to evaluate resistance to combined thrust and moment as prescribed in Section 3.4:

$$T_{f\,cr} = \frac{0.5\,\gamma_E\,W_E + \gamma_L\,W_{LL} + 0.5\,\gamma_L\,W_{Lane}}{2} \qquad (3.2\text{-}3)$$

$$T_{f\,sh} = \frac{0.67(\,\gamma_E\,W_E + \gamma_L\,W_{LL} + 0.5\,\gamma_L\,W_{Lane})}{2} \qquad (3.2\text{-}4)$$

where:

T_f = factored thrust at the springline, kN/m, k/ft

$T_{f\text{-}cr}$ = factored thrust at the crown, kN/m, k/ft

$T_{f\text{-}sh}$ = factored thrust at the shoulder, kN/m, k/ft

γ_E = load factor for earth loads

γ_L = load factor for live loads

The factored thrust must be less than the axial resistance, R_T, seam strength, R_s, and general buckling capacity, R_b:

$$T_f \leq \text{Minimum}[\,R_T, R_S, R_b\,] \qquad (3.2\text{-}5)$$

R_T, R_S, and R_b shall be calculated in accordance with Sections 3.2.1, 3.2.2, and 3.2.3, respectively.

3.2.1 Axial Resistance

C3.2.1

The factored axial resistance of the culvert wall in pure thrust per unit length of culvert is:

$$R_T = \phi_c\ F_y\ A_p \qquad (3.2.1\text{-}1)$$

where:

R_T = factored axial resistance to thrust, kN/m, k/ft

SPECIFICATIONS

COMMENTARY

φ_c = resistance factor as specified in Section 1.3.2

F_y = yield strength of culvert material, kPa, ksi

A_p = cross-sectional area of structural plate per unit length, m^2/m, $in.^2/ft$

The area of the structural plate shall neglect the area of any stiffeners unless calculations demonstrate that the stiffeners are attached to the culvert in a manner that contributes to axial load resistance.

3.2.2 Seam Strength

C3.2.2

The factored resistance of the longitudinal seams, R_s, shall be greater than the applied factored thrust, T_f:

$$R_s = \phi_{sm}\ SS \quad (3.2.2\text{-}1)$$

where:

R_s = factored resistance of longitudinal seams, kN/m, k/ft

φ_{sm} = resistance factor for seam strength

SS = nominal resistance of seams; See AASHTO LRFD Table A12-8

3.2.3 General Buckling Capacity

The nominal resistance to general buckling capacity of the culvert can be computed as:

$$R_b = 1.2\,\phi_b\,C_n\,(E_p I_p)^{1/3}\left(\left[\phi_s M_s K_b\right]\right)^{2/3} R_h \quad (3.2.3\text{-}1)$$

For designs meeting all other requirements of these specifications and the AASHTO LRFD Bridge Construction Specifications:

$$R_h = \frac{11.4}{11 + S/H} \quad (3.2.3\text{-}2)$$

C3.2.3

The equations for general buckling resistance presented here are a conservative simplification of the continuum buckling theory proposed by Moore, et al. (1994). Detailed analyses using the full theory may be applied in lieu of the method presented here.

SPECIFICATIONS

where:

R_b = nominal axial force in culvert wall to cause general buckling, kN/m, k/in.

ϕ_b = resistance factor for general buckling

C_n = scalar calibration factor to account for some nonlinear effects = 0.55

E_p = modulus of elasticity of pipe wall material, kPa, k/in.^{2}

I_p = average moment of inertia of stiffened culvert wall per unit length, m^4/m, in.^{4}/in.

φ_s = resistance factor for stiffness of compacted soil

M_s = constrained modulus of embedment; see Section 2.3, kN/m^2, k/in.^{2}

K_b = $\dfrac{(1-2\nu)}{(1-\nu)^2}$

ν = Poisson's ratio of soil

R_h = correction factor for backfill soil geometry

S = culvert span, m, ft

H = depth of fill over top of culvert, m, ft

COMMENTARY

K_b converts the constrained modulus to the plane strain modulus. Poisson's ratio for backfill soils is often not available. A common assumption is that ν = 0.3, giving K_b = 0.82.

The complete theory proposed by Moore, et al. (1994) provides more detailed methods of determining values of R_h for a number of support conditions. See also final report for NCHRP Project 12-45, *Recommended Specifications for Large-Span Culverts*.

SPECIFICATIONS

Buckling capacity should be evaluated in both stiffened and unstiffened elements of the culvert. The constrained modulus should be selected based on the depth of the element being evaluated. Thrust due to live load need not be considered when evaluating elements of the culvert below the springline.

3.3 Flexure

Design for flexural effects in the culvert is based on application of minimum stiffness criteria, control of deformations during backfilling, and an evaluation of moments due to live loads and earth loads over the structure.

The total factored moment on the structure is taken as the maximum of:

$$M_U = -\gamma_E\, M_{side\text{-}min} - \gamma_E\, M_E + \gamma_L\, M_{LL} \qquad (3.3\text{-}1)$$

$$M_U = \gamma_E\, M_{side\text{-}max} + \gamma_E\, M_E + \gamma_L\, M_{LL} + \gamma_L M_{Lane} \qquad (3.3\text{-}2)$$

where:

M_U = total factored moment in the culvert and may be positive or negative, kN-m/m, in.-k/ft

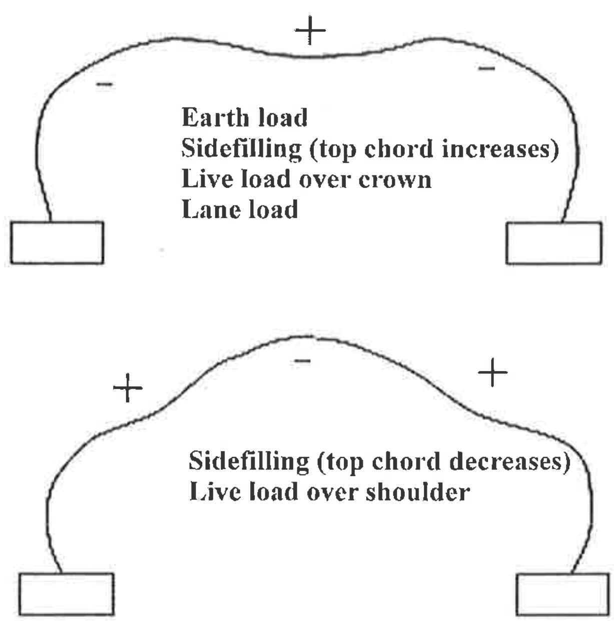
M_{LL}, M_E, M_{Lane} = moment due to backfilling at the sides of the culvert, live load, earth load, and the lane load, respectively; see following sections.

$M_{side\text{-}min}$ = moment due to sidefilling that results in the minimum allowable top chord dimension, kN-m/m, in.-k/ft

COMMENTARY

C3.3

In the simplified design approach, loads are assumed to deform the culvert into one of two shapes. For each load condition, the positive and negative moment is assumed to be of equal magnitude. This is demonstrated below:



+,- = assumed sign of bending moment

Figure 3.3-1– Load Cases and Deformed Shapes for Simplified Design

SPECIFICATIONS

$M_{side\text{-}max}$ = moment due to the sidefilling that results in the maximum allowable top chord dimension, kN-m/m, in.-k/ft

The total moment in the structure shall be investigated for all cases where the live load moment exceeds 15% of the total plastic moment capacity of the culvert section. Evaluation shall include live loads during and after construction.

Total moment is evaluated against the nominal plastic moment capacity of the stiffened section:

$$M_U \leq \phi_f \, M_P \qquad (3.3\text{-}3)$$

where:

ϕ_f = resistance factor for flexural capacity of section

M_P = developable plastic moment capacity of stiffened culvert in the absence of thrust, kN-m/m, ft-k/ft

COMMENTARY

In computing factored moments in equations 3.3-1 and 3.3-2, M_E, M_{LL}, and M_{Lane} are considered positive in sign. $M_{sidemax}$ and $M_{sidemin}$ are taken with a sign consistent with the assumed maximum and minimum top chord length. Positive moment produces tension on the inside surface of the culverts. See Section 3.3.1

Long-span culverts under deep earth fills are designed only for thrust forces, as described in Section 3.2. The 15% criterion provides a convenient way to eliminate moment as a design condition.

SPECIFICATIONS

3.3.1 Minimum Distortion While Placing Sidefill: M_{side}

Moment due to placing backfill to the top of the culvert, M_{side}, shall be limited by controlling the deformation. In lieu of a more detailed analysis, curved sections shall be assumed to deform into sections of constant radius, and moment may be computed based on the change of curvature. Checks should be conducted on all large radius elements of the culvert.

Shortening of the chord is considered to cause negative moment, and lengthening of the chord causes positive moment. The sidefilling moment should be checked for the upper and lower bounds that are allowed in the field when sidefilling is completed to the top of the culvert.

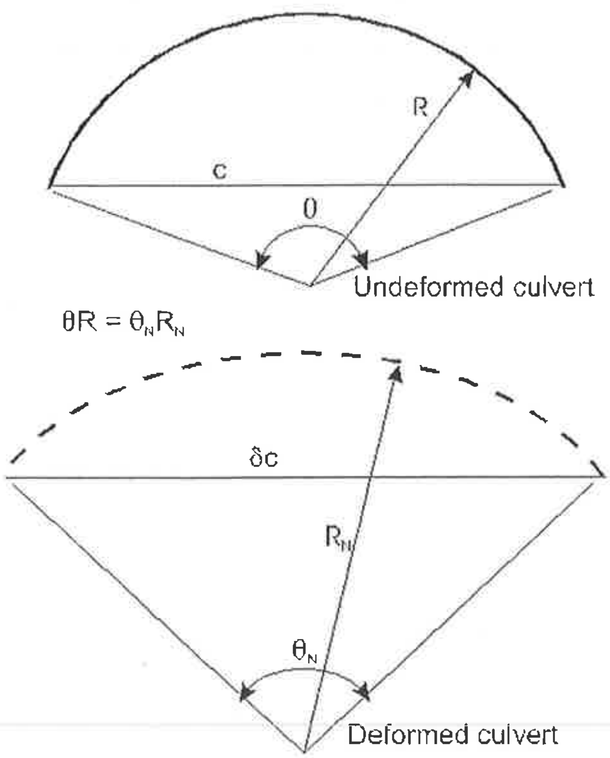


Figure 3.3.1-1 – Terminology for Computing M_{side}

COMMENTARY

C3.3.1

Reliably predicting applied forces and resulting moments due to placing sidefill at the side of a culvert is almost impossible. Thus, the approach taken here is to limit deformation to values that can be imposed in the field and then use moment curvature relationships to establish the associated moment.

Note that adding stiffeners will reduce the allowable deformation of culverts, but will also reduce the expected deformation during construction.

SPECIFICATIONS

Bending moment may be estimated based on the change in curvature. The radius of curvature of the deformed section is related to the radius of curvature of the undeformed section through the relationship:

$$\delta\, c = 2\ R_N\ \sin\left(\frac{\theta\ R}{2\ R_N}\right) \qquad (3.3.1\text{-}1)$$

where:

- δ = ratio of the deformed chord length to the original chord length
- c = chord length of undeformed culvert, m, ft
- R_N = radius of curvature of the deformed culvert element, m, ft
- θ = included angle of undeformed culvert element, radians
- R = radius of curvature of the undeformed culvert element

The construction related bending moment, M_{side}, of the deformed culvert can then be computed as:

$$M_{side} = E_p\ I_p\left(\frac{1}{R} - \frac{1}{R_N}\right) \qquad (3.3.1\text{-}2)$$

The absolute value of the term $1/R - 1/R_N$ shall not be taken less than 0.005 m^{-1}, (0.0015 ft^{-1}).

$$M_{side} \leq M_y \qquad (3.3.1\text{-}3)$$

3.3.2 Moment Due to Live Load

The moment due to live load shall be computed as:

$$M_{LL} = 2\ W_{LL}\ R_t\ K_{LL} \qquad (3.3.2\text{-}1)$$

COMMENTARY

For example, the top arc of a low profile arch has a radius of 6.3 m (20 ft-7 in.) with an included angle of 80° (1.39 radians) and a chord length 8.08 m (318 in.), then if the top chord lengthens by 2%, the new radius is determined by a trial and error solution of the equation:

$$1.02\,(8.08) = R_N\ \sin\left(\frac{1.39\,(247)}{2\,R_N}\right)$$

giving R_N = 7.11 m (280 in.), and a construction moment of 8.38 kN-m/m (22.600 in.-k/ft).

Some culverts tend to peak, and a typical design assumption will be no outward movement of the sides during sidefilling.

The minimum value specified for change in curvature ($1/R - 1/R_N$) accounts for local distortions, even when the overall chord length remains unchanged.

Moment due to sidefill is restricted to the yield moment to control construction deformations.

C3.3.2

For this simplified approach, the positive and negative moment due to live load are considered to be of the same magnitude. M_{LL} should be taken positive when

SPECIFICATIONS

$$K_{LL} = 0.02\left(1.05 - \frac{S_B}{S_B + 800}\right) \geq 0.001 \qquad (3.3.2\text{-}2)$$

$$S_B = \frac{\phi_s\ M_s\ S^3}{E_p\ I_p} \qquad (3.3.2\text{-}3)$$

where:

M_{LL} = factored live load moment, kN-m/m, in.-k/ft

W_{LL} = live load determined in accordance with Section 3.1.2, kN/m, k/ft

R_t = radius of top plates, m, in.

K_{LL} = moment calibration coefficient

S_B = the bending stiffness factor for the culvert soil system.

ϕ_s = resistance factor for stiffness of compacted backfill

M_S = constrained modulus of compacted backfill, or, if appropriate, the composite value representing the stiffness of the native soil and the backfill, as computed in Section 2.3. The vertical stress used to select M_s should be based on the depth of the top of the culvert for live load and at the depth of the widest point of the culvert for earth loads, MPa, ksi.

S = culvert span, m, in.

Lane load is treated as a uniform load on the culvert. Moment due to the lane load is computed as:

$$M_{Lane} = W_{Lane}\ S\ K_E \qquad (3.3.2\text{-}4)$$

where:

M_{Lane} = moment due to lane load, kN m/m, in. k/ft

COMMENTARY

computing the combined moment in Section 3.3.

The live load moment equation is based on procedures first proposed by Duncan, which rely on elastic soil-structure interaction theory that shows moments are a function of the relative stiffness of the soil to that of the culvert flexural properties. The coefficients presented here have been recalibrated based on a parametric study conducted as a part of NCHRP Project 12-45.

SPECIFICATIONS | **COMMENTARY**

W_{Lane} = Lane load, kN/m, k/ft

S = culvert span, m, ft

K_E = moment coefficient for uniform loads; see Section 3.3.3.

3.3.3 Moment Due to Earth Load

The factored moment due to earth load over the culvert shall be computed as:

$$M_E = \gamma_s S^2 H K_E \quad (3.3.3\text{-}1)$$

$$K_E = 0.05\left(1 - \frac{S_B}{S_B + 400}\right) \geq 0.0025 \quad (3.3.3\text{-}2)$$

where:

M_E = factored moment due to earth load, kN-m/m, in.-k/ft

γ_S = unit weight of soil, kN/m^3, k/ft^3

K_E = moment coefficient for uniform loads

s = culvert span, m, ft

H = depth of fill over top of culvert, m, ft

C3.3.3

The earth load moment equation is based on the method proposed by Duncan and recalibrated as part of NCHRP Project 12-45. Moment is always computed with a positive sign. Positive or negative moment due to earth load is considered by the signs in the load combination equations in Section 3.3

3.4 Combined Thrust and Moment

For designs where the live load moments exceed 15% of the plastic moment capacity, the combined moment and thrust shall be evaluated as:

$$\frac{T_{f-ta}}{R_T} + \frac{8}{9}\left(\frac{M_u}{M_n}\right) \leq 1.00 \quad \text{for} \quad \frac{T_{f-ta}}{R_T} \geq 0.2 \quad (3.4\text{-}1)$$

and

$$\frac{T_{f-ta}}{2\,R_T} + \frac{M_u}{M_n} \leq 1.00 \quad \text{for} \quad \frac{T_{f-ta}}{R_T} < 0.2 \quad (3.4\text{-}2)$$

C3.4

The expression for combined moment and thrust capacity is the same as AASHTO LRFD Equations 6.8.2.3.1-1 and 6.8.2.3.1-3, and AISC (1994). The equation produces the envelope in the attached figure. The interaction evaluation is only applicable for shallow culverts where failure by flexure is a concern and the presence of significant axial thrust can affect section capacity.

SPECIFICATIONS

where $T_{f\text{-}ta}$ and M_u are the applied factored thrust in the top arc and moment and R_T and M_n are the factored compressive strength and bending moment resistances in the absence of other forces. $T_{f\text{-}ta}$ is the maximum of $T_{f\text{-}cr}$ and $T_{f\text{-}sh}$ determined in Section 3.2.

COMMENTARY

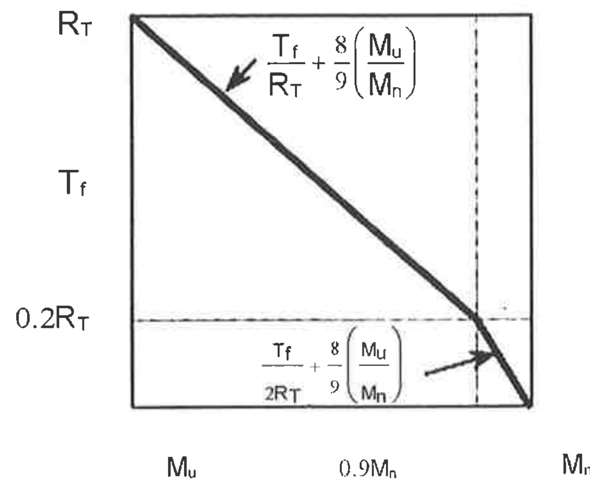


Figure C3.4-1 – Combined Thrust and Moment Interaction

The expression is slightly less conservative than an assumption of a simple linear interaction, but is more conservative than the power law interactions proposed by some.

3.5 Shape Control

Project plans and construction guidelines shall include detailed requirements for limiting culvert deformations during construction. Included shall be at least limits on change in top chord deformations set in Section 3.3.1, overall changes in span, and rise and culvert racking. In addition, construction precautions presented in the AASHTO LRFD Bridge Construction Specifications, Section 26, shall be adhered to.

C3.5

The number of possible shapes and sizes of long-span culverts precludes advanced guidelines for all of the shape parameters that should be controlled during construction. The overall limitations on changes in top chord length in Section 3.3.1 provide significant insight on overall changes that can be anticipated and tolerated.

4. Foundation Design

4.1 Footings

Footings must be designed in accordance with Section 10 for bearing capacity and Sections 5 or 6 for structural capacity.

C4.

C4.1

SPECIFICATIONS

4.2 Settlement

Potential for settlement across the span and along the alignment of the culvert must be investigated. The culvert and footing must be designed to accommodate anticipated settlements.

COMMENTARY

C4.2

5. References

ACPA, *Concrete Pipe Design Handbook*, Irving, TX, American Concrete Pipe Association, 1988.

AISC, *Manual of Steel Construction, Load and Resistance Factor Design,* 2nd Edition, Chicago, IL, American Institute of Steel Construction, 1994.

AWWA, *AWWA Manual M45, Fiberglass Pipe Design Manual*, Denver, CO, American Water Works Association, 1995.

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McGrath, T.J., Moore, I.D., Selig, E.T., and Webb, M.C., "Recommended Specifications for Large-Span Culverts," *NCHRP Project 12-45, Final Report*, Washington, DC, National Cooperative Highway Research Program, 2001.

Moore, I.D., Haggag, A., and Selig, E.T., "Buckling Strength of Flexible Cylinders with Nonuniform Elastic Support," *International Journal of Solids and Structures*, Vol. 31, No. 22, 1994, pp. 3041-3058.

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PROPOSED DESIGN SPECIFICATIONS AND COMMENTARY FOR LARGE-SPAN REINFORCED CONCRETE CULVERTS

SPECIFICATIONS

1. Limit states and Resistance Factors

1.1 Service Limit States

At service conditions, concrete culverts shall be designed to control crack width by limiting reinforcement stress. Maximum crack width is 0.01 in.

1.2 Strength Limit States

Under strength load combinations, long-span concrete culverts shall be investigated for the following:

Culvert

flexure

shear

radial tension

Soil

Foundation capacity

COMMENTARY

C1.

For the purposes of this document, long-span concrete culverts are considered any culvert with an open bottom. Thus, rectangular, three-sided culverts and arch culverts are included. Although some "short-span" culverts will meet this definition, the provisions of the specifications are considered appropriate for these structures.

C1.1

Equations to control cracking are all stated in terms of the allowable reinforcement stress. The design equations for crack control are developed from a large number of tests on reinforced concrete pipe, curved beams, slabs, and box sections, and are calibrated to a maximum crack width of 0.01 in.

C1.2

These are the traditional design limits for reinforced concrete culverts. The radial tension limit applies only to tension reinforcement on the inside of curved elements.

Under extreme loads, culvert elements may be designed as compression members (columns). The provisions of Section 5, including limits for spacing of ties on compression reinforcement, should be applied for such design.

Foundations are designed under other sections of the Specifications.

SPECIFICATIONS

1.3 Load and Resistance Factors

1.3.1 Load Factors

Type of Load	Maximum	Minimum
Self Weight	1.25	0.9
Vertical Earth	1.30	0.9
Lateral Earth	1.35	0.9
Live	1.75	0.9

1.3.2 Resistance Factors

Resistance factors for reinforced concrete culverts are required for:

Mode	Resistance Factor
Flexure	0.95
Shear	0.90
Radial Tension	0.90

2. General Design Features

2.1 Backfill

2.1.1 Backfill Types

Backfill materials shall be granular materials as specified in the contract documents and *AASHTO LRFD Bridge Deign Specifications*, and shall be free of organic material, rock fragments larger than 75 mm in the greatest dimension, and frozen lumps, and shall have a moisture content within the limits required for compaction.

As a minimum, backfill materials shall meet the requirements of AASHTO M145 for A-1, A-2, or A-3 soils. Frost-susceptible soils shall not be used for backfill where ice lens formation is possible. Further restrictions on granular backfill are:

COMMENTARY

C1.3

C1.3.1

Load factors are taken from the current LRFD Specification for reinforced concrete culverts, Section 3.4.1. Lateral earth pressures are based on the at rest condition.

C1.3.2

Resistance factors for reinforced concrete remain unchanged from existing practice. In Table 12.5.5.1, change the heading "Reinforced Concrete Precast Concrete Three-Sided Structures" to "Reinforced Concrete Precast Concrete Arch and Three-Sided Structures." Adding curved arch structures to this section requires adding a resistance factor for radial tension.

C2.

C2.1

C2.1.1

Backfill materials have traditionally been considered a construction issue; however, as the backfill plays a role in the structural performance of the culvert, selection of material must be made as part of the design process.

Allowable backfill materials are consistent with current practice. Lower stiffness soils are allowed in backfill for concrete culverts relative to flexible culverts.

SPECIFICATIONS	COMMENTARY
a) A maximum of 50% of the particle sizes may pass the 0.150 mm (No. 100) sieve, and a maximum of 20% may pass the 0.075 mm (No. 200) sieve.	Long-span concrete culvert installations can be designed with all of the specified soils as backfill material; however, they are not all equivalent soil types, and the use of soils with higher fines content will require higher compaction levels and additional reinforcement in the culvert. Selection of backfill type is a design decision that affects the structural performance.
b) A-2-6 and A-2-7 soils shall not be used as backfill for long-span culverts with 3.5 m (12 ft) or more cover.	Under low depth of fill, some of the backfill types may not be suitable for use under roadways. This decision should be based on the pavement design criteria.
The gradation of backfill materials shall be selected to prevent particle migration between adjacent materials. Gradations of in situ, backfill, and embankment materials shall be evaluated for compliance with this requirement. Alternatively, a suitable geotextile may be used to maintain separation of incompatible materials.	
Controlled low strength material (CLSM), also known as flowable fill, may be used as structural backfill. If not specified in the contract documents, a mix design and complete construction details must be submitted. Minimum construction details include methods for control of flotation forces and waiting time between placing CLSM and backfilling over the structure.	CLSM is a mixture of sand, water, cement, and a fluidizing agent (fly ash or additives that produce high air volumes). Cement contents can be on the order of 30 kg/m^3 (50 lb/yd^3). After setting, strengths are quite high when compared to soil and quite low when compared to concrete.

SPECIFICATIONS

COMMENTARY

2.1.2 Soil properties

For purposes of computing load, unit weight of the soil placed over the culvert should be determined for the actual materials used. In the absence of actual data, backfill unit weight may be estimated as per Table 3.5.1-1 of these specifications.

C2.1.2

2.2 Minimum Spacing Between Multiple Lines of Culverts

The space between multiple lines of culverts shall allow adequate space for the compaction of backfill to meet the design specifications. If the space is less than 150 mm (6 in.), then it shall be filled with lean concrete or CLSM. This fill should extend to the top of straight-legged portion of culverts with vertical sides and to the top of the sidefill zone of curved culverts.

C2.2

2.3 Width of Structural Backfill

Structural backfill shall extend outward at the sides of the culvert sufficiently to ensure proper structural support for the culvert and to allow proper compaction of the backfill material, but not less than 1 m (3 ft).

If the stiffness of the in situ material is less than 50% of the stiffness of the structural backfill, then the minimum width of the structural backfill shall be increased to at least 0.25 times the span.

C2.3

Long-span concrete culverts require less width of structural backfill than metal culverts because of the lower soil stresses developed at the sides of the culverts.

The stiffness of the structural backfill and the in situ material can be estimated using the constrained modulus. Values of the constrained modulus for several types of compacted backfill, $M_{s\text{-}SB}$ and undisturbed native materials, $M_{s\text{-}N}$ are provided in Tables C2.3-1 and C2.3-2.

SPECIFICATIONS COMMENTARY

Table C2.3-1
Constrained Modulus, $M_{s\text{-}SB}$, Values for Backfill Materials, MPa

Stress Level (kPa)	Soil Type and Compaction Condition									
	Sn100	Sn95	Sn90	Sn85	Si95	Si90	Si85	Cl95	Cl90	Cl85
7	16.2	13.8	8.8	3.2	9.8	4.6	2.5	3.7	1.8	0.9
35	23.8	17.9	10.3	3.6	11.5	5.1	2.7	4.3	2.2	1.2
70	29.0	20.7	11.2	3.9	12.2	5.2	2.8	4.8	2.4	1.4
140	37.9	23.8	12.4	4.5	13.0	5.4	3.0	5.1	2.7	1.6
280	51.7	29.3	14.5	5.7	14.4	6.2	3.5	5.6	3.2	2.0
420	64.1	34.5	17.2	6.9	16.4	7.7	4.8	6.2	3.6	2.4

1.0 MPa = 145 psi; 1.0 kPa = 0.145 psi
Soil types are defined in Table 27.5.2.2-3 of the LRFD Construction Specifications, which need to be incorporated into the design specifications. Compaction levels are percent of maximum density per AASHTO T99.
Modulus values are secant moduli for stress variations from unstressed to the indicated stress level.

Table C2.3-2
Constrained Modulus, $M_{s\text{-}N}$, Values for Native Soils
Ref. AWWA Manual M45 *Fiberglass Pipe Design*

In Situ Soil Type				$M_{s\text{-}N}$ (MPa)
Granular		Cohesive		
Blows/0.3 m	Description	q_u (MPa)	Description	
> 0-1	very, very loose	> 0-0.012	very, very soft	0.4
1-2	very loose	0.012-0.025	very soft	1.5
2-4		0.025-0.050	soft	5
4-8	loose	0.050-0.100	medium	10
8-15	slightly compact	0.100-0.200	stiff	20
15-30	compact	0.200-0.400	very stiff	35
30-50	dense	0.400-0.600	hard	70
> 50	very dense	> 0.600	very hard	140

1.0 MPa = 145 psi; 1 m = 3.28 ft
Results of standard penetration test, AASHTO T206, ASTM D1586
Unconfined compressive strength of undisturbed soil

2.4 Foundation Design

Footings for culverts shall be designed in accordance with Section 10. Analysis shall include design for soil bearing capacity, evaluation of potential for longitudinal or differential settlement, and hydraulic considerations, such as scour.

C2.4

SPECIFICATIONS | **COMMENTARY**

2.5 Minimum Cover

C2.5

There is no minimum cover requirement for culverts with flat tops, but flat top sections with less than 0.3 m (1 ft) of cover shall be provided with shear keys between adjacent precast segments unless the design assumes no load transfer across joints. For sections with curved tops, the depth of fill over the top shall not be less than 0.3 m (1 ft).

2.6 Skewed Alignments

C2.6

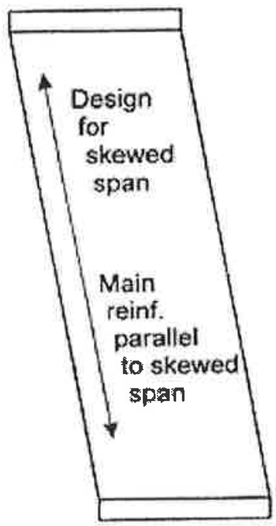
The special structural configuration of skewed alignments must be addressed in design. In particular:

- For culvert elements with skews greater than 15°, the effect of the skew shall be considered in the analysis.
- Ends of skewed culverts must be designed as suggested in Figure 2.6-1, or other steps taken to address the indicated features.
- The unbalanced lateral loads at the ends of a culvert on a skewed alignment must be considered in the design.

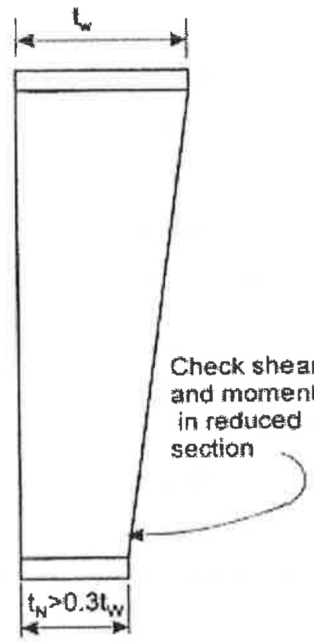
Wheel loads on skewed culverts may be distributed using the provisions for culverts with main reinforcement parallel to traffic; however, loads applied to the culvert shall consider traffic directions both parallel and transverse to main reinforcement.

SPECIFICATIONS

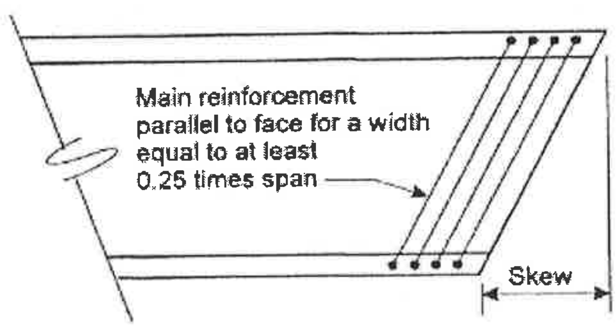
COMMENTARY



a. Precast culvert manufactured with skew



b. Precast culvert with tapered end section



c. Cast-in-place Culvert with Skewed End

Figure 2.6-1 – Design Considerations for Culverts on Skewed Alignments

SPECIFICATIONS	COMMENTARY

3. Design

Structural design of long-span concrete culverts may be by comprehensive methods, such as finite element analyses with proven structural and soil models, or by frame analyses using the simplified pressure distributions described here.

C3.

Finite element analysis with programs such as CANDE has been effectively used to analyze and design many long-span concrete culverts. Other computer programs can also been effectively used in design.

3.1 Load

3.1.1 Handling

Segments of precast concrete long-span culverts must be designed to resist all forces that will be imposed on the sections during stripping, handling, and shipping.

C3.1

C3.1.1

3.1.2 Earth Pressure Distribution for Simplified Analysis

The vertical earth pressure on long-span concrete culverts is assumed to vary linearly from a pressure of 1.0 times the geostatic soil pressure at the culvert top to 1.2 times the geostatic pressure at the edge of the culvert, as shown in Figure 3.1.1-1.

C3.1.2

For culverts with curved tops, as shown in Figure 3.1.1-1, the pressure distribution on the top slab of the culvert is nonlinear, varying as a function of h, the distance from the crown of the culvert to the elevation of interest. Thus, computing the total load on the culvert requires integrating over the span:

SPECIFICATIONS

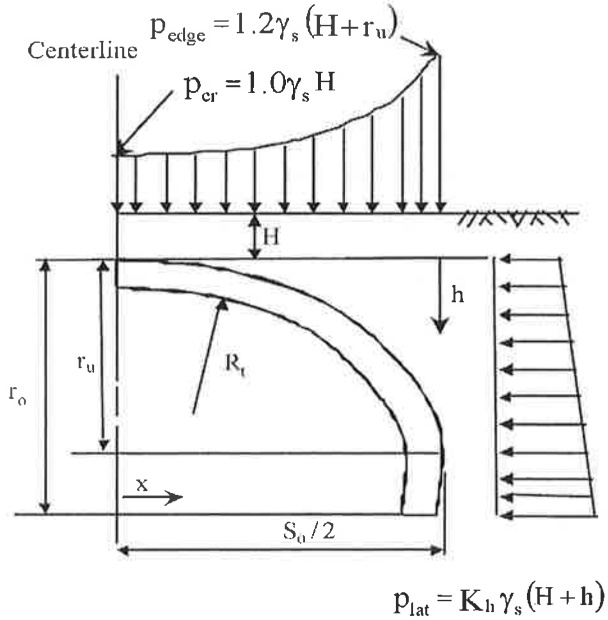


Figure 3.1.1-1 – Pressure Distribution on Long-Span Concrete Culverts

The lateral pressure distribution on culverts is assumed to vary linearly from top to bottom of the culvert as shown in Figure 3.1.1-1. The lateral pressure coefficient, K_h, is a function of the backfill type, as shown in Table 3.1.1-1

COMMENTARY

$$W_E = 2\int_0^{S_o/2} \gamma_s \left(1 + \frac{x}{S_o/2}\,0.2\right)(H + h(x))\,dx \qquad \text{(C3.1.1-1)}$$

where:

W_E	=	earth load on culvert, kN/m, k/ft
γ_s	=	soil unit weight, kN/m^3, k/ft^3
S_o	=	outside span of culvert, m, ft
x	=	distance from culvert centerline, m, ft
H	=	depth of fill over crown of culvert, m, ft
h(x)	=	distance from crown of culvert to top of culvert at coordinate x, m, ft

or computing the pressure at several locations and assuming the pressure varies in a piecewise linear fashion.

SPECIFICATIONS

Table 3.1.1-1
Lateral Pressure Coefficients for Long-Span Concrete Culverts

Soil Type & Compaction Level (Note 1)		K_h Curved Top	K_h Flat Top
Sn	95	$0.40+0.05H \leq 0.6$ (H in m) $0.40+0.016H \leq 0.6$ (H in ft)	0.40
Sn	90	$0.40+0.025H \leq 0.6$ (H in m) $0.40+0.008H \leq 0.6$ (H in ft)	0.40
Si	95		
Sn	85	0.40	0.37
Si	90		
Cl	95		
Other Soils		0.30	0.30

1. Soil types are defined in Table 27.5.2.2-3 of the LRFD Construction Specifications, which need to be incorporated into the design specifications. Compaction levels are percentage of maximum density per AASHTO T99.

COMMENTARY

The proposed values for lateral pressure coefficients were developed under NCHRP Project 12-45 with consideration of the results of the SIDD research program for concrete pipe. Long-span concrete structures with curved tops are sufficiently flexible to develop a modest passive soil pressure at the sides of the culvert, thus the values for K_h are slightly higher than for flat top structures.

3.1.3 Water Loads

Pressure on the structure caused by groundwater must be considered in the analysis if it increases the design force under consideration.

C3.1.3

Groundwater effects on buried structures can be significant for some structural shapes.

3.1.4 Live Load

Culverts shall be designed for the effects of all wheels that may be applied to the culvert strip being analyzed. Live load at the surface shall be considered as single wheels or as groups of wheels, whichever produces the greatest pressure.

C3.1.4

Definitions of L_o and W_o are presented in Figure C3.1.3-1 for single wheels and groups of wheels.

SPECIFICATIONS

Live load effects shall be computed as follows:

$$W_{LL} = \frac{m\,I\,P}{L_L\,L_W} \quad (3.1.4\text{-}1)$$

$$L_L = L_o + LLDF\,(H) + D_w \quad (3.1.4\text{-}2)$$

$$L_W = W_o + LLDF\,(H) + D_w \quad (3.1.4\text{-}3)$$

where:

W_{LL}= live load on structure, kN/m^2, lb/ft^2

m = multiple presence factor, per Section 3.6.1.1.2

I = 1 + IM/100, IM is the dynamic load allowance for culverts, Section 3.6.2.2

P = magnitude of live load, kN, lb

L_L = distribution length of live load effect parallel to culvert crown, m, ft

L_w = distribution length of live load effect parallel to culvert span, m, ft

L_o = outside length of the tire or group of tires under consideration, m, ft

W_o = outside width of the tire or group of tires under consideration, m, ft

LLDF = coefficient for distribution of live load with depth of fill from Section 3.6.1.2.6

COMMENTARY

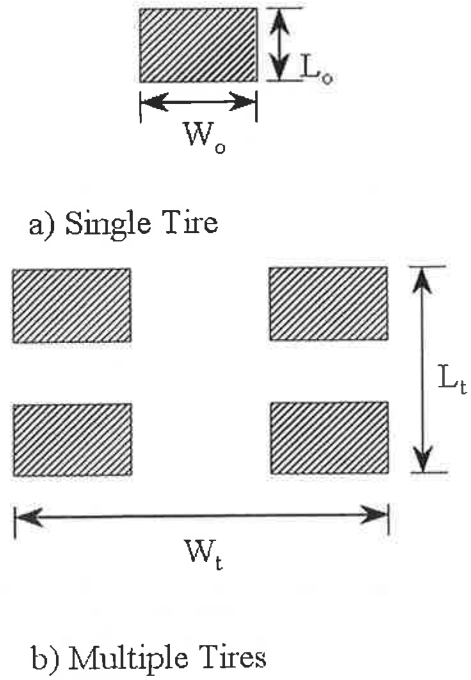


Figure C3.1.2-1 – Terminology for Tire and Wheel Group Dimensions

SPECIFICATIONS

D_W = additional live load distribution width based on stiffness of concrete slab, 1 m in SI units and 40 in. in Customary U.S. Units

For precast concrete culverts with less than 0.3 m (1 ft) of cover, L_L shall not exceed the width of a single precast segment unless:

1. adjacent segments are connected with joints capable of transferring shear without vertical slip, or
2. a distribution slab is provided

3.1.5 Footing Movement

The design shall consider anticipated vertical and lateral footing movement. Unless footings are structurally attached to each other, the footings shall be designed to withstand increase or decrease of distance between footings of at least 0.001 times the mean span, and more if warranted by site conditions.

3.1.6 Seismic Loads

Seismicity of culvert sites should be evaluated and considered in design when appropriate.

COMMENTARY

The live load distribution includes the distribution factor, D_W, which accounts for distribution of live load within the top slab of the culvert after the distribution of load through the fill is accounted for. While strip widths have commonly been used for slabs and culverts with less than 2 ft of fill, the application to culverts with greater depths of fill is new. This change is based on research for long-span culverts, NCHRP Project 12-45, which indicates that live load moments are much smaller than predicted with previously developed procedures.

C3.1.5

C3.1.6

Research has shown that seismic events generally do not cause significant damage to culverts except at:

- changes in direction,
- active fault crossings, and
- locations where stiffness changes abruptly.

SPECIFICATIONS

3.2 Analysis

Culverts shall be analyzed for forces produced by all applicable load conditions. Analysis shall be on the basis of assumed pressure distributions or by culvert-soil interaction analysis.

Analysis shall be based on pinned connections at the footing unless specific reinforcement is designed to develop a moment resistance.

3.3 Design

Design of the structural section shall be in accordance with Sections 12.10.4.2.4 through 12.4.7, except as specified herein.

3.3.1 Ultimate Flexure

Members may be designed solely as flexural members using Equation 12.10.4.2.4a-1. If the required tensile reinforcement exceeds A_{smax} in Equation 12.10.4.2.4c-2, then the design should be as a compression member, in accordance with Chapter 5.

3.3.2 Radial Tension in Curved Elements

Tension reinforcement on the inside of curved sections shall be evaluated for radial tension using the provisions of Article 12.10.4.2.4c. If reinforcement required to meet the flexural requirements of Section 12.4.2.4a exceeds the value A_{smax} as calculated by Equation 12.10.4.2.4c-1, then the tensile reinforcement shall be anchored per the provisions of Equation 12.10.4.2.6-1.

COMMENTARY

C3.2

Long-span concrete culverts may be designed by finite element analysis or by frame analysis with the pressure distribution specified in Section 3.1. In frame analysis, it is common to model the concrete culvert as linear elastic with the area and moment of inertia of a plain concrete section. If desired, analysis may be completed with cracked section properties, but it is difficult to assess which portions of the arch, if any, will crack.

C3.3

C3.3.1

C3.3.2

APPENDIX G

PROPOSED CONSTRUCTION SPECIFICATIONS AND COMMENTARY FOR LARGE-SPAN CULVERTS

APPENDIX G

PROPOSED CONSTRUCTION SPECIFICATIONS AND COMMENTARY FOR LARGE-SPAN CULVERTS

TABLE OF CONTENTS

PROPOSED AASHTO LRFD BRIDGE CONSTRUCTION SPECIFICATIONS AND COMMENTARY FOR LARGE-SPAN CORRUGATED METAL CULVERTS

SPECIFICATIONS

1. General

1.1 Description

This work shall consist of furnishing, fabricating, and installing corrugated metal culverts in conformance with these specifications, and the details given in the contract documents. Included are pipe, metal box culverts, and long-span structures, with either closed shapes (full 360° circumference) or arches on footings. As used in this specification, long-span structures are constructed of corrugated structural plate, assembled to form horizontal elliptic, inverted-pear, and multiple radius arch shapes, as well as special shape culverts as defined in the *AASHTO LRFD Bridge Design Specifications*, Section 12.

1.2 Importance

Satisfactory performance of flexible culverts requires proper construction procedures. The embedment material placed around them provides a significant amount of support. Together, the culvert and embedment form an integral soil-structure system. Therefore, suitable quality backfill materials, properly placed and compacted, are essential.

COMMENTARY

C1.

C1.1

The term "pipe" refers to the smaller sizes of culverts; long-span sizes are not considered pipe. Pipe may be formed from a single corrugated structural plate with helical or longitudinal seams or from multiple corrugated structural plates bolted together. Long-span and corrugated metal box culverts are always made from multiple plates, have features such as stiffening ribs or longitudinal stiffeners, and have special construction requirements. The contract documents should identify structures that are considered long-spans and are thus subject to the special requirements set forth herein. A corrugated, structural plate arch is supported on reinforced concrete footings, with or without a paved invert slab. Pipe arches are constructed from corrugated plate to form a culvert having an arch-shaped crown and a relatively flat invert.

The metal culvert description is further covered in Section 12, "Buried Structures and Tunnel Liners" of the *AASHTO LRFD Bridge Design Specifications*.

C1.2

SPECIFICATIONS

COMMENTARY

Control of construction procedures during all stages of erection and backfilling are important. Each culvert installation may be unique because of different external and structural factors, and so they may require modification of construction methods.

1.3 Terminology

C1.3

Terminology used in this Specification is illustrated in Figures 26.1 and 26.2. Definitions of important terms are given below:

Bedding is the material on which the structure is seated. It may be in situ soil, if such soil meets all necessary requirements, or imported backfill material. The bedding may be specified as a different material than the structural backfill.

Culvert bottom is the lowest point on the outside of the culvert.

Culvert crown is the highest point on the inside of the culvert.

Culvert *invert* is the lowest point on the inside of the culvert.

Culvert top is the highest point on the outside of the culvert.

Embankment is the soil placed and compacted in layers at the sides of, and above, the embedment zone.

Embedment zone is the zone of structural backfill around the culvert. It consists of: bedding, haunch material, sidefill, and initial topfill.

Footings are the structural foundations bearing on the foundation soil and supporting arch culverts.

SPECIFICATIONS	COMMENTARY

Foundation soil is the soil supporting the bedding (if any), the culvert, and the structural backfill. It must provide a firm stable surface and may be undisturbed, existing (in situ) soil, replaced and compacted in situ soil, or an imported material.

Haunch is the portion of the culvert between the bottom and the springline.

Haunch zone is the region of the backfill between the bedding or foundation soil and the culvert surface from the bottom to near the springline. It is a region where hand placement and compaction methods are normally required for the backfill. Backfill in the haunch zone is the same material as the structural backfill.

In Situ soil is the native undisturbed soil existing at the site of the culvert installation.

Shoulder is the portion of the culvert between the top and the springline.

Sidefill is the embedment zone between the haunch and the shoulders of the culvert supporting the sides of the culvert.

Springline is the line along the side of the culvert where the tangent to the culvert wall is vertical. It occurs at the widest point in the culvert.

Structural backfill is the material placed and compacted around the culvert to help support the culvert.

Topfill is the embedment zone over the top of the culvert beginning at the shoulders and extending upward to the limit of the structural backfill zone. The topfill is generally the same material as the structural backfill. For long-span culverts, it must be the same as the structural backfill.

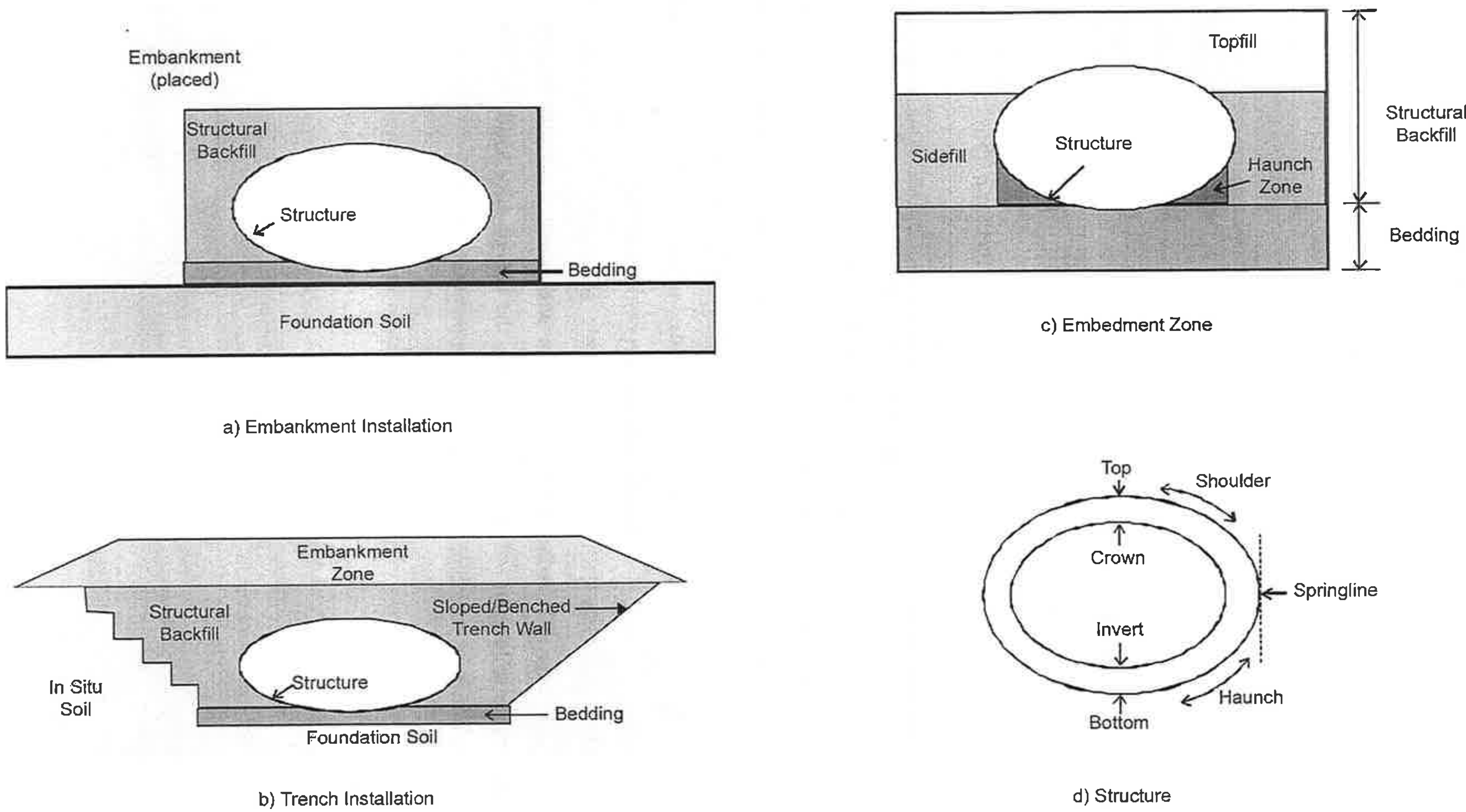


Figure 26.1 – Terminology for Culvert Installation

SPECIFICATIONS

2. Working Drawings

(No changes from existing specifications)

3. Materials

(No changes from existing specifications for culvert materials, Sections 3.1 to 3.7.)

3.8 Bedding and Structural Backfill Materials

3.8.1 General

Bedding shall be granular material with a maximum particle size less than one-half the corrugation depth. Structural backfill materials shall be granular materials as specified in the contract documents and these specifications; shall be free of organic material, rock fragments larger than 75 mm in the greatest dimension, and frozen lumps; and shall have a moisture content within the limits required for compaction.

As a minimum, bedding and backfill materials shall meet the requirements of AASHTO M145 for A-1, A-2-4, A-2-5, or A-3 soils. Frost-susceptible soils shall not be used for backfill where ice lens formation is possible.

COMMENTARY

C2.

C3.

C3.8

C3.8.1

Granular backfill has 35% or less material by weight finer than the 0.075 mm (No. 200) sieve as defined in AASHTO M145.

Construction of culverts during the winter months may pose potential problems when frozen soils are included in the backfill zone or when frost-susceptible soils are used as backfill material. Frozen soil will not compact effectively, and may result in points of concentrated loads when frozen and regions of inadequate support begin thawing.

Frost-susceptible soils should not be used in the embankment zone within the frost penetration depth. This will exclude the use of silty sand or silty gravel where freezing temperatures occur.

SPECIFICATIONS

Further restrictions on granular backfill are:

a) For all structural plate sizes, a maximum of 50% of the particle sizes may pass the 0.150 mm (No. 100) sieve, and a maximum of 20% may pass the 0.075 mm (No. 200) sieve.

b) For long-span sizes of structural plate:

- A-1-b may be used only for high-profile arch and pear shapes up to 4 m (12 ft) cover height, and for low-profile arch and elliptical structures only up to 6 m (20 ft) cover height.
- A-2-4 and A-2-5 materials are restricted to 4 m (12 ft) cover height, and are not allowed for shapes with large radius side plates (pear, pear arch, and high profile arch).

COMMENTARY

The restriction on materials passing the 0.150 mm (No. 100) sieve and the 0.075 mm (No. 200) sieve is intended to eliminate soils composed of significant amounts of fine sands and silts. These materials are difficult to work with, sensitive to moisture content, and do not provide support comparable to coarser or more broadly graded materials at the same percentage of maximum density. This includes some A-1-b, A-3, A-2-4, and A-2-5 soils. A-2-6 and A-2-7 soils display similar characteristics and are also eliminated from use as backfill materials. The engineer may permit exceptions to these restrictions in special cases. If so, a suitable plan must be submitted for control of moisture content and compaction procedures. These silty and clayey materials should never be used in a wet site. Increased inspection levels should be considered if such a plan is approved.

SPECIFICATIONS

The gradation of bedding and backfill materials shall be selected to prevent particle migration between adjacent materials. Gradations of in situ, bedding, backfill, and embankment materials shall be evaluated for compliance with this requirement. Alternatively, a suitable geotextile may be used to maintain separation of incompatible materials.

COMMENTARY

Control of migration is based on the relative gradations of adjacent materials. Acceptable criteria include:

$D_{15}/d_{85} < 5$, where D_{15} is the sieve opening size passing 15% by weight of the coarser material, and d_{85} is the sieve opening size passing 85% by weight of the finer material.

$D_{50}/d_{50} < 25$, where D_{50} is the sieve opening size passing 50% by weight of the coarser material, and d_{50} is the sieve opening size passing 50% by weight of the finer material. This criterion need not apply if the coarser material is well graded as defined in ASTM D2487.

SPECIFICATIONS

3.8.2 Box Culverts

(No changes proposed from existing specifications for Section 3.8.2)

3.8.3 Controlled Low Strength Material

Controlled low strength material (CLSM), also known as flowable fill, may be used as bedding and/or structural backfill. If not specified in the contract documents, a mix design and complete construction details must be submitted. Minimum construction details include methods for control of flotation forces and waiting time between placing CLSM and backfilling over the structure.

4. Assembly

Culverts shall be assembled in accordance with the manufacturer's instructions and as specified in the contract documents. Copies of the manufacturer's assembly instructions shall be furnished as specified in Article 26.2.

All culverts shall be unloaded and handled with reasonable care. Pipe or plates shall not be rolled or dragged over gravel or rock, and shall be prevented from striking rock or other hard objects during placement in the trench or on the bedding.

Culverts shall be placed on the bedding, generally starting at the downstream end. Culverts with circumferential seams shall be installed with their inside circumferential sheet laps pointing downstream.

COMMENTARY

C3.8.2

C3.8.3

FHWA Report FHWA-RD-98-191 *Pipe Interaction with the Backfill Envelope* (FHWA, 1998) indicates that CLSM can be an effective backfill material for culverts.

C4.

SPECIFICATIONS

Bituminous-coated pipe, polymer-coated pipe, and paved invert pipe shall be handled and installed with special care to avoid damage to coatings. Paved invert pipe shall be installed with the invert pavement placed and centered on the bottom.

4.1 Bolted Seams

Bolted longitudinal seams shall be well fitted with the lapping plates parallel to each other. Longitudinal seams should be staggered such that no more than three plates meet at any point. The applied bolt torque for M20 high-strength steel bolts (ASTM A449) for the assembly of steel structural plate shall be a minimum of 135,000 N-mm and a maximum of 407,000 N-mm. Aluminum structural plate shall be assembled using M20 aluminum bolts (ASTM F468) or standard-strength steel bolts (ASTM A307), which shall be torqued to a minimum of 135,000 N-mm and a maximum of 203,000 N-mm.

When staggered bolting patterns are used, bolts shall be placed in the valleys of the row closer to the visible edge.

After all seams are nested properly, the bolts may be tightened as plates are hung, or they may be tightened once all plates are in place and all bolts are inserted. If bolts are left loose, the structure must be supported to prevent damage to plates and connections.

COMMENTARY

C4.1

There is no structural requirement for residual torque; the important factor is the seam fit-up.

When seam sealant tape or a shop-applied asphalt coating is used, bolts should be retightened no more than once, and generally within 24 hrs after initial tightening.

Abdel-Sayed (1993) and Mikhailovsky, et al. (1992) have shown that this pattern minimizes prying action between plates and reduces tearing around boltholes.

Retightening of bolts after the first sequence of bolt tightening (whether plates are hung and tightened one at a time or all the plates are hung and then tightened) is preferred, since bolts may be loosened by the first bolting sequence. Furthermore, when the nuts are located on the inside of the culvert, another sequence of bolt tightening can be performed after backfilling to ensure that no bolts have been loosened by the backfilling operations.

SPECIFICATIONS

Unless held in shape by cables, struts, or backfill, longitudinal seams should be tightened when the plates are hung. Bolt tightening procedure should be as required to ensure proper lapping of plates and proper shape and dimensions after completion of assembly and bolt tightening with adequately torqued bolts.

Erection and bolt tightening procedures should minimize change in structural shape from the design shape.

If not otherwise specified in the contract documents, the allowable variation in structure dimensions after assembly are:

- For arch shapes having a ratio of top to side radii of three or more, the rise shall not deviate from the specified dimensions by more than 1% of the span.
- For all other culverts, the span and rise shall not deviate from the specified dimensions by more than 2%, nor more than 125 mm (5 in.), whichever is less.

4.2 Temporary Support

When required, temporary bracing shall be installed and shall remain in place as long as necessary to protect workers and to maintain structure shape during erection.

For structures that require temporary bracing or cabling to maintain the structure shape, the supports shall not be removed until the structure backfill is placed to an elevation that provides the necessary support. In no case shall internal braces be left in place after backfilling reaches the top quadrant of the pipe or the top radius arc portion of a long-span structure.

COMMENTARY

The deviation from design shape after assembly and bolt tightening are added to the deviations during backfilling to the top. The total deformation will be evaluated against the specified deflection limits. The allowable deflection limits are as specified in the contract plans or these specifications.

C4.2

SPECIFICATIONS

4.3 Circumferential Stiffeners

When required, circumferential stiffeners shall be attached to the structural plate prior to backfilling, using the specified bolt spacing, but not more than 300 mm. Legible identifying letters or numbers shall be placed on each stiffener to designate its proper position in the finished structure.

4.4 Longitudinal Stiffeners

Unless otherwise directed, longitudinal stiffeners are installed after the level of backfill reaches the lowest elevation of the stiffener.

4.5 Other Features

Substructures and headwalls are designed and constructed in accordance with the applicable requirements of the *AASHTO LRFD Bridge Design Specifications*, and other Articles of these specifications.

The base of metal arches shall rest in a keyway formed into continuous concrete footings, or shall rest on a metal anchorage, usually an angle or channel shape, secured to or embedded in the concrete footing.

When specified, the metal anchorage may be a hot-rolled or cold-formed galvanized steel angle or channel, or an extruded aluminum angle or channel. These shapes shall be not less than 5 mm in thickness and shall be securely anchored to the footing at a maximum spacing of 600 mm. When the metal bearing member is not completely embedded in a keyway in the footing, one vertical leg shall be punched to allow the end of the corrugated plates to be bolted to this leg of the bearing member.

COMMENTARY

C4.3

Circumferential stiffeners are an important element in providing stiffness to resist deformation during backfilling. Circumferential stiffeners also assist in resisting live load forces.

C4.4

Longitudinal stiffeners help distribute compaction forces along the length of the culvert and minimize deformation during backfilling.

C4.5

SPECIFICATIONS

Where an invert slab is provided that is not integral with the arch footing, the invert slab shall be continuously reinforced.

5. Joints

(No changes from current specifications)

6. Site Preparation and Excavation

Construction operations should commence in dry conditions. Sites requiring excavation below the groundwater table shall be dewatered to at least 0.3 m below the deepest portion of the excavation or, when the culvert is installed in a stream or river bed, the water shall be diverted or separated by cofferdams. Obtain advanced approval of the engineer if construction must continue in water. Under these conditions, free-draining gravels shall be used as foundation and bedding.

Excavation shall be to the width, depth, and grade shown in the contract documents. Increase the width of excavation from that shown, if necessary, to provide adequate space at the sides of the culvert for construction equipment to work, including compaction equipment used during backfilling.

If in situ materials are inadequate to provide support to the pipe, increase the width of excavation to provide 0.3 m or one half the span, whichever is larger, on each side of the culvert.

The bottom of the excavation shall be undisturbed in situ material. If the excavation bottom is loose, soft, or disturbed, recompact as directed by the engineer. Avoid creating pockets of loose and/or wet material.

COMMENTARY

C5.

C6.

Space should be provided at the site for storage of the corrugated metal pipe unless it is installed as delivered.

In the case of corrugated metal plate structures, maintaining a well-organized construction site for the storage and assembly of the plates can greatly speed up the construction process (Abdel-Sayed, et al., 1993). Plates of similar sizes should be stacked together and kept apart from other sizes to prevent accidental use of the wrong size plate in erecting the culvert. Access roads to and from the site should be maintained to speed up delivery of materials to the site. Furthermore, constructing a working platform next to the culvert for the lifting equipment to operate on may reduce construction time.

AASHTO LRFD Bridge Design Specifications provide guidance on evaluating the suitability of in situ soils for use in the structural backfill zone.

SPECIFICATIONS

For installations where the top of the culvert extends above or within the rise of the existing ground, and the existing ground will be covered with an embankment, remove vegetation, organic or frozen material, and other soft materials that do not meet the stiffness requirements of the structural backfill for a distance at least 0.5 times the span on each side of the culvert springline. Replace with embankment material.

Trench walls shall be sloped or braced to ensure stability throughout construction. Trench walls should be undisturbed at the time of backfilling, at least up to the top of the structure. Sloped walls may be benched back to facilitate compaction of structural backfill against them. If used, trench bracing shall be removed as backfill progresses upward. In other cases, bracing can be pulled (if thin bracing is used to avoid leaving gaps between the structural backfill and the trench wall) when the trench is stable, or left in place if non-decomposable material is used.

COMMENTARY

Experience has shown that leaving soft material in the sidefill zone can increase the load on the culvert.

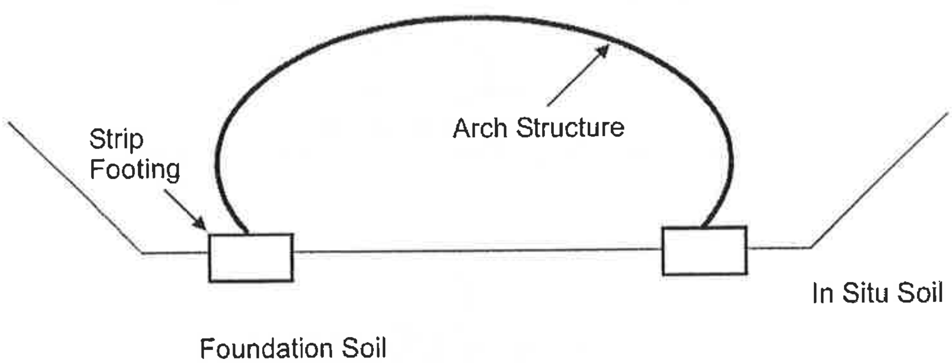


Figure 26.2 – Structural Foundations for Arch Structures

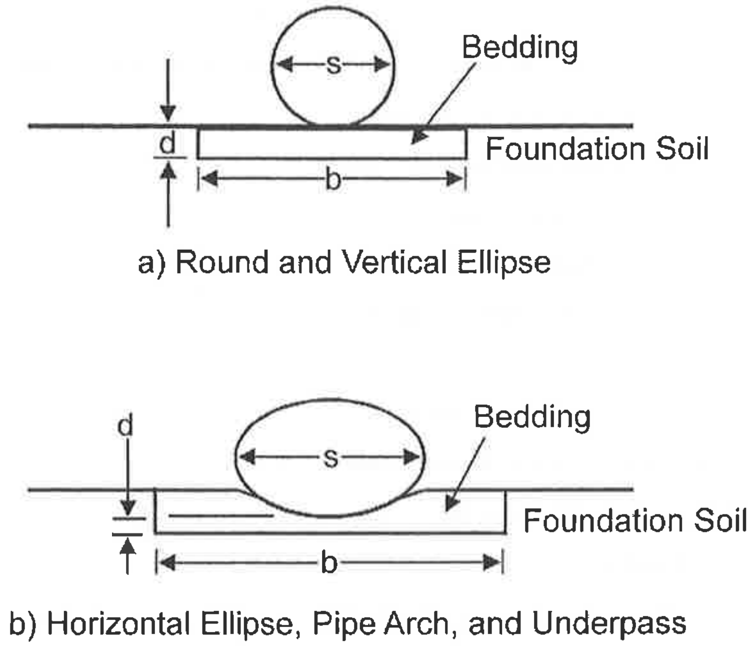


Figure 26.3 – Foundation Treatment with Placed Bed

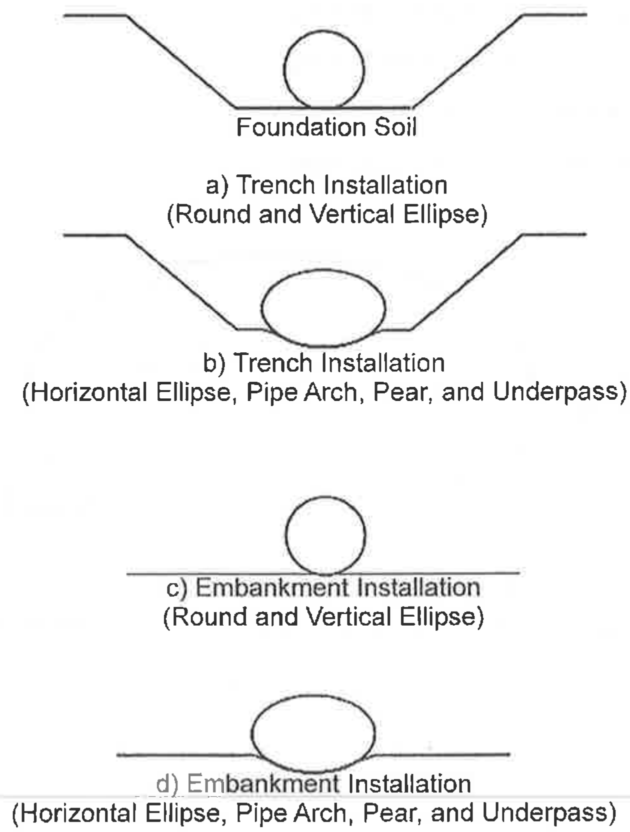


Figure 26.4 – Foundation Treatment without Placed Bed

SPECIFICATIONS

7. Foundation and Bedding Preparation

Arch structures are supported on footings that bear on foundation soil (Figure 26.2). Other shape structures bear directly on bedding or foundation soil Figures 26.3 and 26.4). Proper preparation of footings, foundation soil, and bedding material, where required, shall precede the installation of all culverts. This work shall include necessary leveling of the in situ trench bottom or the top of the foundation material, as well as placement and compaction of required bedding material to a uniform grade so that the entire length of the culvert will be properly supported.

The foundation soil under the culvert or footings, as well as under the structural backfill, shall be investigated for its adequacy to support the imposed loads. The foundation soil shall be investigated for the full width of the trench, or, for wide trench or embankment installations, a width of 0.3 m or one-half the span of the culvert, whichever is larger, on each side of the culvert springline. The remedies for soft or inadequate foundation soils noted below shall apply to the same widths as investigated. Report all conditions not anticipated in the contract documents to the engineer.

7.1 Footings

Cast-in-place concrete footings shall be placed on undisturbed earth, unless otherwise directed by the contract documents. Precast concrete footings shall be placed either on undisturbed earth or on granular backfill compacted to 100% of maximum density, per AASHTO T99.

COMMENTARY

C7.

Shaped bedding cannot be reliably constructed for shapes like round and vertical ellipses; for large radius bottom plates, shaping is possible and is preferred because backfilling the lower haunch area is difficult.

If the foundation is firm under the culvert but soft at the sides, compression of the soft material can cause increased load on the culvert due to down drag. Thus the foundation quality must be evaluated for a width greater than the culvert.

C7.1

SPECIFICATIONS | **COMMENTARY**

Construction of footings shall comply with the appropriate sections of these specifications. Excavate the foundation soil as required for placement of the footings. Minimize disturbance at the base and sides of the excavation. Any disturbed soil on which the footings are to be placed shall be compacted to provide the bearing required. Any excavated zones at the sides of the footings shall be backfilled with the same material and placed and compacted to the same requirements as the structural backfill.

7.2 Foundation Soil

C7.2

If the foundation soil is suitable for directly supporting the structure, the foundation shall be prepared as shown in Figure 26.4. If the foundation soil is suitable but in a loose condition, it shall be compacted as specified, but to not less than to 90% of the maximum dry density per AASHTO T99 for granular soils, or 95% of maximum dry density for fine-grained soils, before shaping the foundation soil and placing the bedding, or placing the structure.

Boulders, rock, or soft spots in the foundation soil beneath closed-shape culverts shall be excavated to a suitable depth and backfilled with bedding material installed in accordance with 7.4 to provide bearing as shown in Figure 26.3. Unless specified otherwise by the engineer, the bedding width, b, and excavation depth, d, shall be as follows:

- For rock and boulders, use b = culvert span D, and use minimum d = 50 mm (2 in.) + 25 mm (1 in.) for each 300 mm (12 in.) of span D, but not less than twice corrugation depth and not more than 300 mm (12 in.).

SPECIFICATIONS

- For soft spots, use b = 2 D or trench width, whichever is smaller, and use minimum d = 50 mm (2 in.) + 25 mm (1 in.) for each 300 mm (12 in.) of span D, but not less than twice the corrugation depth or a depth sufficient to reduce the stress on the soft soil to its allowable bearing value.

When the natural foundation soil is judged inadequate by the engineer to support the culvert or structural backfill, the soil shall be excavated to the depth, d, and width, b, prescribed in the contract documents. The excavation shall be backfilled with bedding material compacted as specified, but to not less than 90% of the maximum dry density per AASHTO T99 for granular soils, or 95% of the maximum dry density for fine-grained soils.

Where relatively large-radius plates adjoin small-radius corners or sides for sections, such as pipe-arches, ellipses, or inverted pears, the foundation soil and structural backfill shall be designed to support the radial pressures exerted by the smaller radius portions of the culvert. The principal soil support shall be provided in the zone extending radially outward from the smaller radius plates such as illustrated for pipe arches in Figure 26.5. When such a corrective measure is necessary for pipe arches, providing less support under the bottom allows the culvert to maintain its shape as minor settlements occur. This is not typically done for large-span culverts.

COMMENTARY

Radial pressures at small radius corners may be two to five times the pressures on top of the culvert, depending on the culvert shape.

SPECIFICATIONS

COMMENTARY

7.3 Camber

Where settlement of the culvert is expected to be such that the required grade under high fills will not be maintained after construction, the culvert may be cambered to prevent excessive sag. The amount of camber shall be determined based on consideration of the flow line, gradient, fill height, the compressive characteristics of the foundation material, and the depth to incompressible strata. The use of camber under a high fill is shown in Figure 26.6. This is not typically done for long-span culverts.

C7.3

7.4 Bedding

Unless indicated otherwise in the contract documents, when placed bedding material is required as specified in 7.2, it shall be placed in appropriate loose layer thickness and compacted as follows:

Compaction shall be to a minimum density equal to 90% of the maximum dry density per AASHTO T-99.

For closed structures of round or vertical ellipse shapes installed as shown in Figure 26.3 a, the portion of the bedding layer under the central one-third of the culvert span should be left uncompacted for a depth of 150 mm.

For closed structures of horizontal ellipse, pipe arch, or pear shapes installed as shown in Figure 26.3 b, all of the bedding shall be compacted, and then the surface shaped to fit the bottom plates with as little disturbance as possible. The shaped area shall be centered beneath the culvert and shall have a minimum width of one-half the span for pipe-arch, pear, and underpass shapes, and one-third the span for horizontal elliptic shapes.

C7.4

SPECIFICATIONS	COMMENTARY

Pre-shaping may consist of a simple "V" graded into the soil, if approved by the engineer, but is generally limited to structures of less than 1,200 mm (48 in.) span.

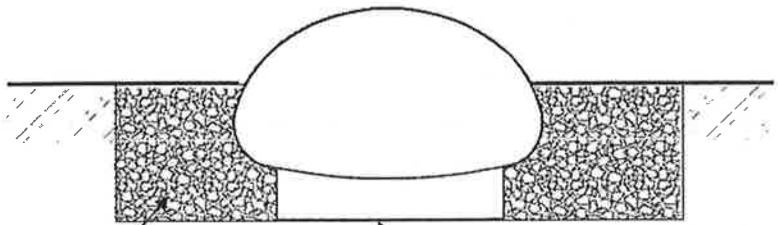


Figure 26.5 – Foundation Treatment for Support of Corner or Side Plate Pressures

8. Backfilling Procedures

Prior to construction of long-span structures, the manufacturer shall advise the contractor(s) and engineer of the more critical functions to be performed during backfilling and to present the intended steps for control of loads, shape, and movements.

Equipment and construction procedures used to backfill culverts shall be selected such that requirements for backfill density and control of structure shape will be met. Structure shape shall be checked regularly during backfilling to verify acceptability of the construction methods used.

C8.

The size and flexibility of structural plate structures, especially long-span, require special control of backfilling and continuous monitoring of structure shape. The connections at the arch footings restrict shape change. Structures with large radius side plates may peak excessively, and structures with large radius top plates may experience curvature flattening in their upper quadrants during backfilling. Using lighter compaction equipment, more easily compacted structural backfill, or top loading such as by placing a small load of structural backfill on the top, will aid proper installation.

SPECIFICATIONS

The magnitudes of the allowable shape changes are provided in the contract documents. For long-span structures, the manufacturer shall provide a qualified shape-control inspector to aid the engineer during the placement of all structural backfill to the minimum cover level over the structure. The shape-control inspector shall advise the engineer on the acceptability of all backfill material and methods and the proper monitoring of the shape.

Backfilling begins with the culvert in place on the bedding or foundation soil as shown in Figures 26.2, 26.3, and 26.4. Arches are on footings, while the closed shapes are directly on the bedding or foundation soil.

There are three basic stages of backfilling: 1) haunch, 2) sidefill, and 3) topfill. Only the second and third apply to arches. For each of these stages of construction, procedures shall be established that will achieve the specified degree of compaction without damaging or excessively distorting the structure. This will improve efficiency of the installation effort and help ensure proper performance without having to rely on time-consuming testing, particularly in the haunch area, which is difficult to access.

COMMENTARY

Specifications typically limit construction deformations to 2 to 5% of the conduit rise.

A bulldozer of approximately 90 kN (20,000 lb) or less may to be used for placing and grading backfill immediately alongside and above the culvert until the minimum depth of cover is reached. Also, the use of a lightweight vibratory plate or roller is suggested for compacting the structural backfill zone.

Once a backfilling procedure is established, the primary inspection effort should be to ensure that the established procedure is followed. Only occasional checks of soil density may then be required, as long as the material and procedures are unchanged.

Unless project specifications provide other limits, granular backfill should be compacted to a minimum of 90% of the maximum dry density in accordance with AASHTO T99.

SPECIFICATIONS

8.1 Backfilling under the Haunch

For closed culvert shapes, material shall be carefully placed in the haunches using mechanical or manual tampers, or other means to fill all voids and meet the specified compaction levels. The installation of haunch fill shall be carried out on both sides simultaneously to avoid rolling the pipe. Also, the compaction force shall be limited and controlled so that the pipe is not lifted out of grade.

COMMENTARY

C8.1

It is important that the selected tamping procedures will meet the design assumptions. In general, a minimum compaction level exceeding 85% of T99 is needed to prevent collapsing the soil structure upon saturation.

When good procedures for compacting material in the haunches are not followed, culvert distress may result, including excessive bending and crimping of plates. The effect of haunching on buried pipe performance was investigated by McGrath, et al., and published in FHWA Report No. FHWA-RD-98-191 *Pipe Interaction with the Backfill Envelope* (FHWA 1998). These studies showed that large void spaces result underneath pipes without good haunching effort.

Loose layers should generally not exceed 150 mm (6 in.) in thickness to permit the backfill material to be worked into the haunch zone. Shovel slicing was shown to be effective in providing haunch support. Different sized tampers were shown to be effective for different backfill soils. A large faced tamper (75 x 150 mm) was effective for silty sand, while a small-faced tamper (25 x 75 mm) was effective for crushed stone backfill.

Haunching is best accomplished by placing part of the first layer of backfill, working it into the haunches, and then placing the remainder of the lift. Thick layers block material from being worked into the haunches.

SPECIFICATIONS | **COMMENTARY**

Water jetting has been found to be an effective procedure for compacting backfill and developing uniform support with clean coarse material and good drainage.

If the culvert is to be backfilled with CLSM, follow all requirements of the specifications or the submitted detailed work plan.

8.2 Placing and Compacting Sidefill

Equipment used to compact backfill within 1 m of each side of the culvert or from edge of footing for arches shall be approved by the engineer prior to use. The structures are flexible, thus sidefill material must be carefully placed and compacted to avoid excessive and unsymmetrical deformations. The shape must be continually monitored to ensure satisfactory results.

Structural backfill material in the sidefill zone shall be placed in horizontal, uniform layers not normally exceeding a 150 mm (6 in.) loose lift thickness for pipe or a 200 mm (8 in.) loose lift thickness for corrugated metal plate structures, or as specified. The layers shall be compacted with appropriate equipment to the specified density. The maximum density shall not normally be less than 95% of T99 for A-1 and A-3 soils, and 98% for A-2 soils. However, the 300 mm to 600 mm width of soil immediately adjacent to the large radius side plates of long-span high-profile arches and inverted-pear shapes can have a reduced compaction requirement.

The maximum difference in the backfill surface elevations between the two sides of the culvert at any time shall not exceed 150 (6 in.) for pipe, or 600 mm (2 ft) for corrugated metal plate culverts.

C8.2

For equal performance, the compaction requirements should be a function of soil type for the same application. Performance will vary widely among the acceptable soils when compacted to the same density specification. Also, design soil stiffness is very sensitive to the level of compaction (McGrath, et al., 1998, in publication as an FHWA report).

Experience with compaction indicates that 150 mm (6 in.) thick loose layers using two coverages with a given compactor will give better uniformity and higher average level of compaction than one 300 mm (12 in.) thick loose layer with four coverages of the same compactor. Alternatively, a 300 mm loose layer will require larger compactors to produce the same average compaction as achieved by a smaller compactor with a 150 mm (6 in.) thick layer. A larger compactor must be evaluated for possible induced structural distortions.

SPECIFICATIONS / **COMMENTARY**

The sidefill material shall be constructed to the minimum lines and grades shown in the contract documents, keeping it at or below the level of adjacent soil or embankment. Placement and compaction of the sidefill layers adjacent to the haunch zone shall be carried out concurrently with backfilling under the haunch.

Backfill material shall be placed, spread, and compacted working parallel to the culvert to avoid creating areas of unequal support.

Unequal support may result when compacting perpendicular to the culvert long-axis.

See Section 4, "Assembly," for notes about temporary support.

See Section 9, "Compaction Control," for notes about deformation limits and monitoring structure shape during backfilling.

If longitudinal stiffeners are specified, they shall be installed, and backfill shall be placed and compacted next to them in accordance with the specifications of the manufacturer.

Care shall be taken when backfilling next to cut end plates because of their flexibility. The plates may have to be braced or otherwise supported.

Cut end plates are more flexible than the full structure barrel.

8.3 Placing and Compacting Topfill

Topfill shall be placed and compacted under the direction of the shape control inspector. All culverts shall be protected by sufficient earth cover before permitting heavy construction equipment to pass over them.

When the sidefill elevation reaches the shoulders, placement of structural backfill over the top begins. For pipe, the procedures as approved by the engineer shall be used.

C8.3

Backfill should be pushed in from the sides of the culvert before spreading at the top is done, i.e., the cover should always be brought in from the sides towards the top, rather than pushed from the top back to the sides, to avoid local buckling from the concentrated load of the construction equipment.

Compaction of the first layer of backfill over the culvert may be left uncompacted or loosely compacted over the central third of the culvert. This minimizes the risk of distorting the culvert.

SPECIFICATIONS	COMMENTARY
For corrugated metal plate culverts, backfill material should be pushed in from the sides and gradually moved up over the top of the culvert using a light dozer. If stiffening ribs are present, this placement should be done first at the location of the ribs. The shape of the culvert must be continually monitored during this process.	
If excessive upward movement of the crown (peaking) occurs while placing topfill, construction procedures and equipment shall be changed to obtain satisfactory results. This process should continue until 0.3 to 0.6 m (1 to 2 ft) of loose material has been placed in a uniform layer over the top (topfill).	
A vibratory roller should be used to compact the first topfill layer; however, for the first several passes, vibration should not be used when the roller is above the structure. The correct procedure is for the roller to begin a pass perpendicular to the axis of the culvert, starting some distance away from the culvert, crossing over the top, and then moving away from the culvert on the opposite side in one continuous pass. The soil layer should be a uniform thickness, thus it will not be flat, but rather will be domed over the top.	Judgment is required as to when vibration is used to compact the first layer of topfill. The shape control inspector shall provide guidance on this.

SPECIFICATIONS

COMMENTARY

The backfill beyond the structure must be well compacted at all elevations. After 0.6 m (2 ft) of cover has been established over the top and compacted without vibration, successive layers can be placed and compacted in 150 mm to 300 mm (6 in. to 12 in.) loose thicknesses. The fill can be gradually leveled as the successive layers are added. The vibratory roller should continue to make passes perpendicular to the long axis of the culvert, but with vibration applied over the entire culvert. This procedure will be continued until all of the structural backfill is in place.

Distortion during backfilling must be controlled so that when the initial topfill is in place, the culvert shape and dimensions are within the design tolerance, allowing only for further changes expected during the remaining backfilling over the top.

The culverts covered by this section shall be evaluated at all critical stages in their installation and for the final intended purpose. Construction loads may require additional cover beyond that required for the final condition to which the design loads apply. In the absence of more specific information, the cover depths in Table 26.6-1 may be considered for the smaller structures indicated. The minimum covers indicated should be increased when site conditions so indicate. The engineer or the manufacturer shall provide guidance for structure spans or axle loads not listed.

See design specifications for guidance on minimum cover depths.

See Section 4, "Assembly," for notes about temporary support.

See Section 9, "Compaction Control," for notes about deformation limits and monitoring structure shape during backfilling.

SPECIFICATIONS

COMMENTARY

9. Compaction Control

Field compaction shall be evaluated based on compacted density and moisture content obtained from acceptable methods, such as the cone replacement (AASHTO T191, ASTM D1556) and the nuclear gage (AASHTO T238 and T239, ASTM D2922). A reference density test shall be performed on a representative sample to obtain a value of maximum dry density (MDD) and optimum moisture content (OMC). This test shall be repeated for each new soil type encountered and for composition variations within the same soil type. Thus, samples should be taken periodically during construction to provide an appropriate series of reference tests.

The number and location of field tests shall be determined by the engineer and selected to ensure that the quality of the soil and the compaction obtained is as specified. Furthermore, the engineer shall stipulate acceptance criteria for the compacted soil.

C9.

Compaction variability must be considered in determining the compaction acceptance criteria, i.e., for a given specified compaction level, the engineer must distinguish between whether it is intended to serve as an average, so that 50% of the samples can have a lower value, or a minimum, so that 100% of the samples must exceed the specification, or whether some other allowable percentage below the specification is intended.

It is common to allow some density tests to fall below the specified density level. Retest procedures should be specified to ensure that the overall design objectives are met.

10. Construction of End Treatment

Substructures and headwalls shall be designed and constructed in accordance with the applicable requirements of the *AASHTO LRFD Bridge Design Specifications* and these specifications.

Where single or multiple structures are installed at a skew to the embankment, proper support for the pipe shall be provided. Support may be achieved with a rigid, reinforced concrete headwall or by warping the embankment fill to provide the necessary balanced side support. Figure 26.7 provides guidelines for warping the embankment.

C10.

SPECIFICATIONS

COMMENTARY

Care must be taken to prevent cut end plates from being bent or distorted from the forces of flowing water for partially completed structures. Slope collars or similar end treatment should be installed to prevent this.

11. Measurement

(No changes from existing specifications)

C11.

12. Payment

(No changes from existing specifications)

C12.

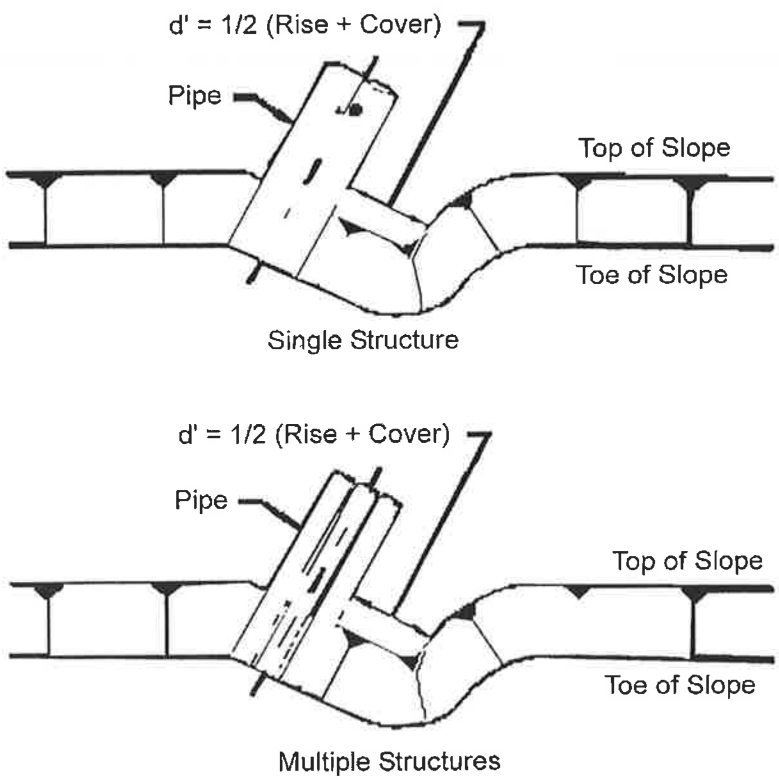


Figure 26.6 – End Treatment of Skewed Flexible Culvert

13. References

Abdel-Sayed, G., Bakht, B., and Jaeger, L.G., *Soil-Steel Bridges: Design and Construction,* New York, NY, McGraw-Hill, 1993, 359 pp.

FHWA, "Pipe Interaction with the Backfill Envelope," *FHWA Report FHWA-RD-98-191,* Washington, DC, Federal Highway Administration, 1998.

Mikhailovsky, L., Kennedy, D.J.L., and Lee, R.W.S., "Flexural Behaviour of Bolted Joints of Corrugated Steel Plates," *Canadian Journal of Civil Engineering,* Vol. 19, 1992, pp. 896-905.

PROPOSED AASHTO LRFD BRIDGE CONSTRUCTION SPECIFICATIONS AND COMMENTARY FOR LARGE-SPAN REINFORCED CONCRETE CULVERTS

SPECIFICATIONS

1. General

1.1 Description

This work shall consist of fabricating, furnishing, and installing buried precast concrete culverts conforming to these specifications and the contract documents. Included are reinforced and non-reinforced pipe, box sections, and long-span structures with flat or curved tops and either curved or vertical sidewalls. Precast reinforced concrete pipe shall be circular, arch, or elliptical, as specified. As used in this specification, long-span structures consist of open bottom precast reinforced concrete segments supported on footings.

Cast-in-place concrete culverts and cast-in-place footings for long-span culverts shall be constructed in accordance with applicable portions of these specifications, and backfilled in accordance with this Section.

COMMENTARY

C1.

C1.1

The term "pipe" refers to closed shapes of concrete culverts that require attention to bedding and backfill under the culvert. Long-span culverts are open shapes set on footings, and are not considered pipe. The sizes of concrete pipe and long-span culverts may overlap.

A long-span culvert may or may not have a paved invert slab. The reinforced concrete culvert description is further covered in Section 12, "Buried Structures and Tunnel Liners," of the *AASHTO LRFD Bridge Design Specifications*.

Sections of these Construction Specifications applicable to cast-in-place culverts and footings for long-span culverts include at least Sections 1, 8, and 9.

Note: The first sentence of Section 8.1.1 of the construction specifications should have the words "cast-in-place" inserted before the word "culvert." Commentary for Section 8.1.1 should be added, stating: "Installation of precast concrete culverts, and backfilling for all concrete culverts, is covered in Section 27 of these specifications." These additions are required to keep the specifications clear on which sections govern which activities.

SPECIFICATIONS

1.2 Importance

Satisfactory performance of rigid culverts requires proper construction procedures. The embedment material placed around a culvert may provide a significant amount of support that is relied upon in the culvert structural design. Together, the culvert and embedment form an integral soil-structure system. Therefore, suitable quality backfill materials, properly placed and compacted, are essential.

Control of construction procedures during all stages of installation and backfilling are important. Each culvert installation may be unique because of different external and structural factors and so may require modification of construction methods.

1.3 Terminology

Terminology used in this specification is illustrated in Figure 27.1 and 27.2. Definitions of important terms are given below:

Bedding is the material on which the structure is seated. It may be in situ soil, if such soil meets all necessary requirements, or imported backfill material. The bedding may be specified as a different material than the structural backfill.

Culvert bottom is the lowest point on the outside of the culvert.

Culvert crown is the highest point on the inside of the culvert.

Culvert *invert* is the lowest point on the inside of the culvert.

Culvert top is the highest point on the outside of the culvert.

COMMENTARY

C1.2

Important aspects of construction procedures for culverts include the selection of the backfill material, as well as procedures for placing and compacting that backfill to the specified density. In general, as the quality of backfill (generally related to particle size and the portion of the backfill material passing the No. 200 sieve) decreases, higher relative compaction levels (e.g., percentage of maximum density per AASHTO T99 or T180) are required to provide equivalent culvert performance.

C1.3

SPECIFICATIONS

COMMENTARY

Embankment is the soil placed and compacted in layers at the sides of, and above, the embedment zone.

Embedment zone is the zone of structural backfill around the culvert. It consists of: bedding, haunch material, sidefill, and initial topfill.

Footings are the structural foundations bearing on the foundation soil and supporting arch culverts.

Foundation soil is the soil supporting the bedding (if any), the culvert, and the structural backfill. It must provide a firm stable surface and may be undisturbed, existing (in situ) soil, replaced and compacted in situ soil, or an imported material.

Haunch is the portion of the culvert between the bottom and the springline.

Haunch zone is the region of the backfill between the bedding or foundation soil and the culvert surface from the bottom to near the springline. It is a region where hand placement and compaction methods are normally required for the backfill. Backfill in the haunch zone is the same material as the structural backfill.

In Situ soil is the native undisturbed soil existing at the site of the culvert installation.

Shoulder is the portion of the culvert between the top and the springline.

Sidefill is the embedment zone between the haunch and the shoulders of the culvert supporting the sides of the culvert.

Springline is the line along the side of the culvert where the tangent to the culvert wall is vertical. It occurs at the widest point in the culvert.

SPECIFICATIONS

COMMENTARY

Structural backfill is the material placed and compacted around the culvert to help support the culvert.

Topfill is the embedment zone over the top of the culvert beginning at the shoulders and extending upward to the limit of the structural backfill zone. The topfill is generally the same material as the structural backfill. For long-span culverts, it must be the same as the structural backfill.

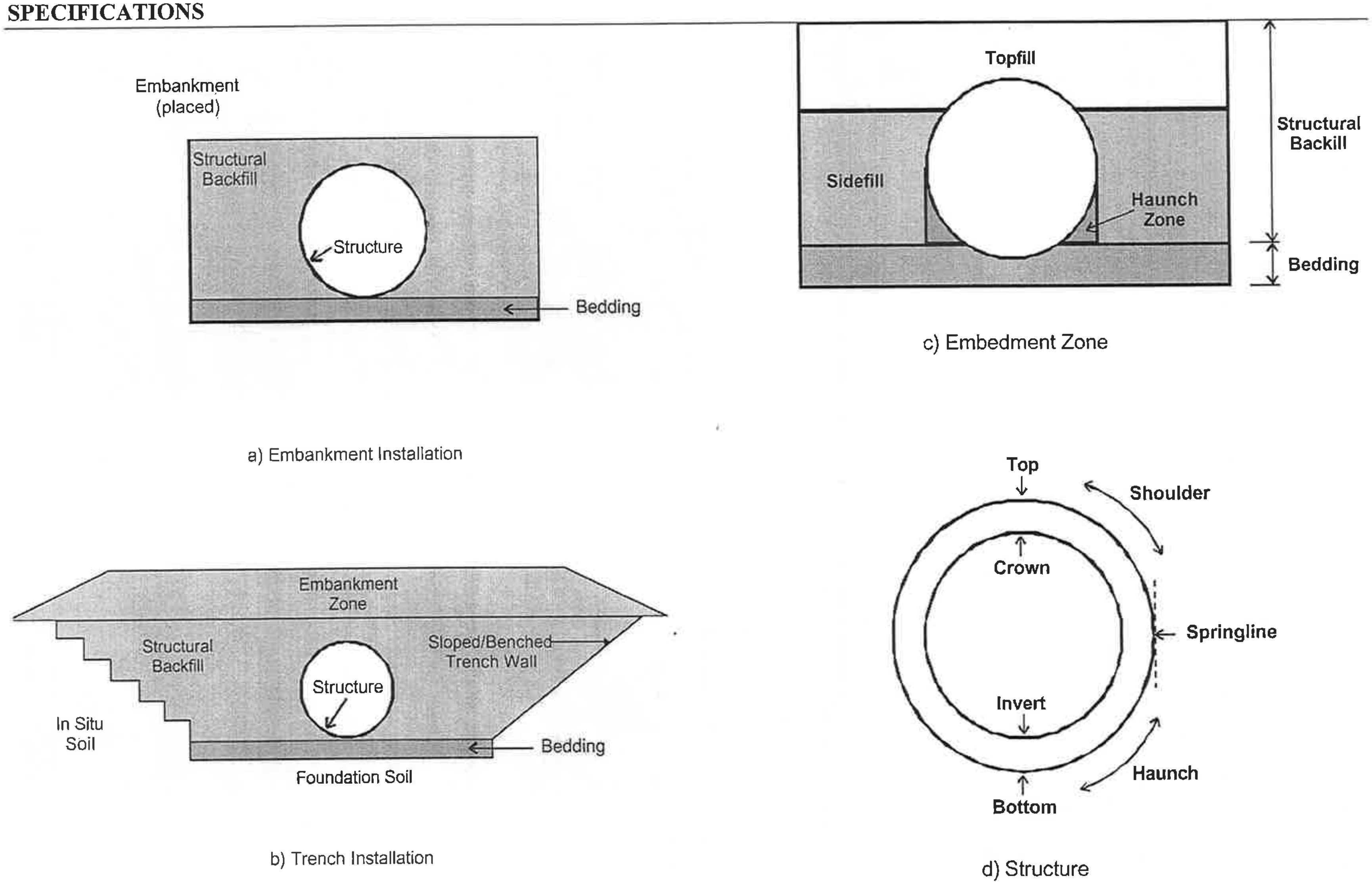


Figure 27.1 – Terminology for Culvert Installation

SPECIFICATIONS

2. Working Drawings

When complete details are not provided in the contract documents, the contractor shall submit working drawings of the proposed structure or installation system. Fabrication or installation of the structure shall not begin until the engineer has approved the drawings. The working drawings shall show complete details and substantiating calculations of the structure, the materials, equipment, and installation methods proposed.

Working drawings and manufacturer's recommendations for installation, assembly, and backfilling procedures shall be submitted in advance of the start of the work to allow for their review, revision, and approval without delay to the work. Approval by the engineer shall not relieve the contractor of any contractual responsibility.

3. Materials

3.1 Reinforced Concrete Culverts

Add to existing *section*

Large-span concrete culverts shall be built to dimensions and tolerances required in the contract documents. Prior to casting precast segments, manufacturers shall submit complete details on concrete, reinforcement, quality control programs for production, and any additional details required in the contract documents.

COMMENTARY

C2.

Revised to add specific requirement for submission of manufacturer's guidelines.

C3.

C3.1

Concrete pipes are all governed by AASHTO material specifications that provide details for quality assurance and quality control. Such standards are not available for concrete large-span culverts, thus manufacturers must make specific submittals.

SPECIFICATIONS

COMMENTARY

3.2 Joints

Joints for reinforced concrete culverts shall comply with the details shown in the contract documents and on the approved working drawings. Each joint shall be sealed to prevent infiltration of soil fines or water as required by the contract documents. The engineer may require field tests whenever there is a question regarding compliance with the contract documents.

(The remainder of this section is the current Section 27.3.2, "Joint Sealants." No changes from existing specification.)

C3.2

3.3 Bedding and Backfill Materials

C3.3

3.3.1 Precast Reinforced Concrete Circular, Arch, and Elliptical Pipe

Bedding, haunch, lower side, and overfill material shall conform to Figures 27.5.2.2-1, 2, 3, and 4, which define soil areas and critical dimensions, and Tables 27.5.2.2-1 and 2, which list generic soil types and minimum compaction requirements, as well as minimum bedding thickness for the four Standard Installation Types. The AASHTO Soil Classifications and the USCS Soil Classifications equivalents to the generic soil types in the Standard Installations are presented in Table 27.5.2.2-3.

C3.3.1

Figures 27.5.2.2-1, 2, 3, and 4, and Tables 27.5.2.2-1, and 2 in the current specifications provide details for concrete pipe installation. They are attached to the back of this specification. No changes are proposed, except to renumber as appropriate.

3.3.2 Precast Reinforced Concrete Box Sections

Bedding for box culverts shall be provided as shown in Figure 27.5.2.3-1 unless the contract documents allow the use of in situ material.

C3.3.2

Figure 27.5.2.3-1 in the current specifications provides details for concrete box culvert installation. It is attached to the back of this specification. No changes are proposed, except to renumber as appropriate.

SPECIFICATIONS

Bedding shall be sand or selected sandy soil, all of which passes a 9.5 mm sieve and not more than 10% of which passes a 0.075 mm sieve. Backfill shall meet the requirements of AASHTO A-1, A-2, A-3, or A-4 to the top of the culvert. Backfill above the top of the culvert shall be select material and shall be free of organic material, rock fragments larger than 75 mm in the greatest dimension, frozen lumps, and shall have a moisture content within the units required for compaction.

3.3.3 Long-Span Structures

Backfill materials shall be granular materials as specified in the contract documents and *AASHTO LRFD Bridge Deign Specifications*, and shall be free of organic material, rock fragments larger than 75 mm in the greatest dimension, and frozen lumps, and shall have a moisture content within the limits required for compaction.

Frost-susceptible soils shall not be used in the embedment zone, where frost penetration may occur.

As a minimum, bedding and backfill materials shall meet the requirements of AASHTO M145 for A-1, A-2, or A-3 soils. Frost-susceptible soils shall not be used for backfill where ice lens formation is possible. Further restrictions on granular backfill are:

COMMENTARY

C3.3.3

Granular backfill has 35% or less material by weight finer than the 0.075 mm (No. 200) sieve as defined in AASHTO M145.

This excludes the use of silty sand or silty gravel where freezing temperatures occur. Frost may penetrate both from the ground surface and around the perimeter of the culvert. Frozen soil will not compact effectively, and may result in points of concentrated loads when frozen, as well as regions of inadequate support.

SPECIFICATIONS

A maximum of 50% of the particle sizes may pass the 0.150 mm (No. 100) sieve and a maximum of 20% may pass the 0.075 mm (No. 200) sieve.

A-2-6 and A-2-7 soils shall not be used as backfill for long-span culverts with 3.7 m (12 ft) or more cover.

The gradation of backfill materials shall be selected to prevent particle migration between adjacent materials. Gradations of in situ, backfill, and embankment materials shall be evaluated for compliance with this requirement. Alternatively, a suitable geotextile may be used to maintain separation of incompatible materials.

COMMENTARY

The restriction on materials passing the 0.150 mm (No. 100) sieve and the 0.075 mm (No. 200) sieve is intended to eliminate soils composed of significant amounts of fine sands and silts. These materials are difficult to work with, sensitive to moisture content, and do not provide support comparable to coarser or more broadly graded materials at the same percentage of maximum density. This includes some A-1-b, A-3, A-2-4, and A-2-5 soils. The engineer may permit exceptions to these restrictions in special cases. If so, a suitable plan must be submitted for control of moisture content and compaction procedures. These silty and clayey materials should never be used in a wet site. Increased inspection levels should be considered if such a plan is approved.

Control of migration is based on the relative gradations of adjacent materials. Acceptable criteria include:

$D_{15}/d_{85} < 5$, where D_{15} is the sieve opening size passing 15% by weight of the coarser material, and d_{85} is the sieve opening size passing 85% by weight of the finer material.

$D_{50}/d_{50} < 25$, where D_{50} is the sieve opening size passing 50% by weight of the coarser material, and d_{50} is the sieve opening size passing 50% by weight of the finer material. This latter criterion need not apply if the coarser material is well graded as defined in ASTM D2487.

SPECIFICATIONS

3.3.4 Controlled Low Strength Material

Controlled low strength material (CLSM), also known as flowable fill, may be used as structural backfill. If not specified in the contract documents, a mix design and complete construction details must be submitted. Minimum construction details include methods for control of flotation forces and waiting time between placing CLSM and backfilling over the structure.

COMMENTARY

C3.3.4

FHWA Report FHWA-RD-98-191, *Pipe Interaction with the Backfill Envelope,* indicates that CLSM can be an effective backfill material for culverts.

CLSM batched with cement and fly ash often reaches an acceptable density and stiffness as soon as the excess water is used up. It is often acceptable to backfill over the CLSM at this point.

4. Assembly

4.1 General

Precast concrete units or elements shall be assembled in accordance with the manufacturer's instructions and as specified in the contract documents. Concrete pipe are assembled after completion of the foundation and bedding described below. Copies of the manufacturer's assembly instructions shall be furnished as specified in Section 27.2.

All units or elements shall be unloaded and handled with reasonable care and shall not be rolled or dragged over gravel or rock. Care shall be taken to prevent the units from striking rock or other hard objects during placement.

C4.

C4.1

SPECIFICATIONS

4.2 Placing Pipe and Box Sections

Unless otherwise authorized by the engineer, the laying of pipe sections on the prepared bedding and foundation soil shall be started at the outlet and with the spigot or tongue end pointing downstream and shall proceed toward the inlet end with the abutting sections properly matched, true to the established lines and grades. Where pipe with raised bells are installed, bell holes shall be excavated in the bedding to such dimensions that the bedding will uniformly support the entire length of the barrel of the pipe when installed. Proper facilities shall be provided for hoisting and lowering the sections of pipe into the trench without disturbing the prepared foundation and the sides of the trench. The ends of the sections shall be carefully cleaned before the section is jointed. The section shall be fitted and matched so that when laid on the bed it shall form a smooth, uniform conduit.

When pipe with reinforcing schemes that vary around the circumference are used, the pipe shall be laid in the trench in such position that the markings for "Top" or "Bottom" shall not be more than 5° from the vertical plane through the longitudinal axis of the pipe.

Multiple installations of reinforced concrete pipe shall be laid with the centerlines of individual barrels parallel at the spacing shown on the plans. Pipe and box sections used in parallel installations require positive lateral bearing between the sides of adjacent pipe or box sections. Compacted granular backfill or grouting between the units are considered means of providing positive bearing.

COMMENTARY

C4.2

All pipe with nonuniform reinforcement schemes must be delivered to the job site marked with the proper orientation for installation. It is preferable that the marking be located on the outside for ease of installation, <u>and</u> on the inside for ease of inspection after construction.

SPECIFICATIONS

4.3 Placing Long-Span Culvert Sections

The base of precast long-span arch segments shall rest in a keyway formed into concrete footings. Segments shall rest on leveling pads at each end of the segment to ensure proper alignment. The keyway shall be filled with grout to ensure proper lateral and vertical support. Grout mix requirements and strength shall be as specified by the manufacturer.

Where an invert slab is provided that is not integral with the footing, the invert slab shall be continuously reinforced.

4.4 Other Features

Substructures, headwalls, and wingwalls are designed and constructed in accordance with the applicable requirements of the *AASHTO LRFD Bridge Design Specifications* and other Sections of these specifications.

5. Site Preparation and Excavation

Construction operations should commence in dry conditions. Sites requiring excavation below the groundwater table shall be dewatered to at least 0.3 m below the deepest portion of the excavation or, when the culvert is installed in a stream or river bed, the water shall be diverted or separated by cofferdams. Obtain advanced approval of the engineer if construction must continue in water. Under these conditions, free-draining gravels shall be used as foundation and bedding.

COMMENTARY

C4.3

Footings may be precast or cast-in-place. See LRFD Bridge Design Specifications.

C4.4

C5.

SPECIFICATIONS

Excavation shall be to the width, depth, and grade shown in the contract documents. Increase the width of excavation from that shown, if necessary, to provide adequate space at the sides of the culvert for construction equipment to work, including compaction equipment used during backfilling.

For pipe, in situ materials in the trench wall must meet the requirements for the lower sidefill as listed in Table 27.5.2.2-2. For long-span culverts, if in situ materials are inadequate to provide design lateral support to the pipe, increase the width of excavation to provide a minimum width of backfill of one-half the span on each side of the culvert.

The bottom of the excavation shall be undisturbed in situ material. If the excavation bottom is loose, soft, or disturbed, recompact as directed by the engineer. Avoid creating pockets of loose and/or wet material.

For installations where the top of the culvert extends above or within the rise of the existing ground, and the existing ground will be covered with an embankment, remove vegetation, organic or frozen material, and other soft materials that do not meet the stiffness requirements of the structural backfill for a distance at least 0.5 times the span on each side of the culvert springline. Replace with embankment material.

COMMENTARY

AASHTO LRFD Bridge Design Specifications provide guidance on evaluating the suitability of in situ soils for use in the structural backfill zone.

Experience has shown that leaving soft material in the sidefill zone can increase the total load on the pipe.

SPECIFICATIONS

Trench walls shall be sloped or braced to all applicable safety requirements. Trench walls should be undisturbed at the time of backfilling, at least up to the top of the structure. If within the lower sidefill zone, trench bracing shall be removed as backfill progresses upward or in some other fashion that does not disturb compacted fill or leave voids. Trench sheeting may be left in place only with advanced approval by the engineer.

COMMENTARY

Trench wall bracing should not be left in place if it interferes with existing or possible future structures or roadways. If left in place, the engineer must judge if future deterioration will cause a loss of support to the culvert.

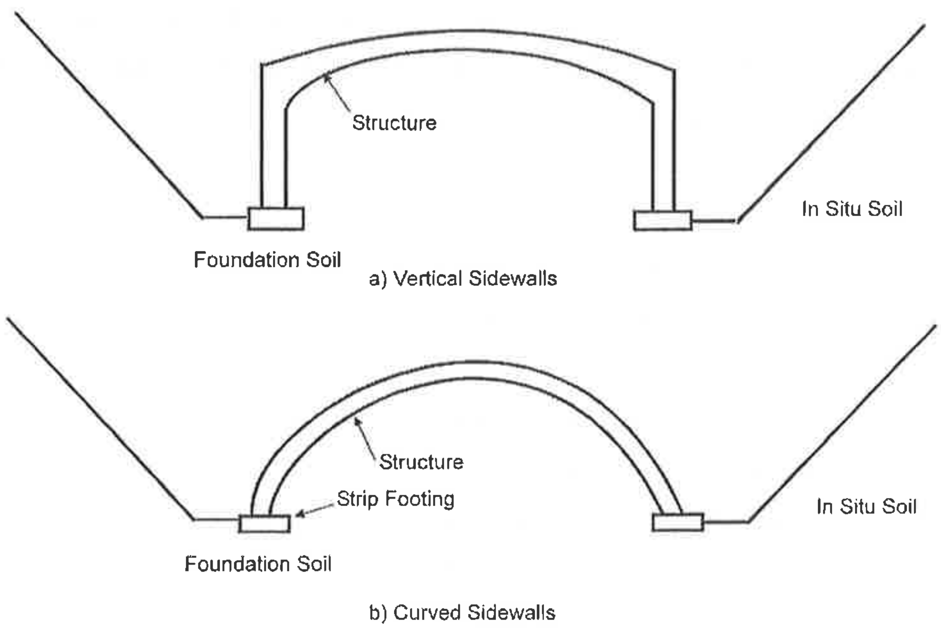


Figure 27.2 – Structural Foundations for Large-Span Culverts

SPECIFICATIONS | **COMMENTARY**

6. Foundation and Bedding Preparation

C6.

6.1 General

C6.1

Pipe bear directly on bedding or foundation soil (Figures 27.5.5.2-1 to 4). Long-span culverts are supported on footings that bear on foundation soil (Figure 27.2). Proper preparation of footings, foundation soil, and bedding material, where required, shall precede the installation of all culverts. This work shall include necessary leveling of the in situ trench bottom or the top of the foundation material as well as placement and compaction of required bedding material to a uniform grade so that the entire length of culvert will be properly supported.

The foundation soil under the culvert or footings and under the structural backfill shall be investigated for its adequacy to support the imposed loads. The foundation soil shall be investigated for the full width of the trench, or, for wide trench or embankment installations, a width of 0.3 m or one-half the span of the culvert, whichever is larger, on each side of the culvert springline. The remedies for soft or inadequate foundation soils noted below shall apply to the same widths as investigated.

If the foundation is firm under the culvert but soft at the sides, compression of the soft material can cause increased load on the culvert due to down drag. Thus, the foundation quality must be evaluated for a width greater than the culvert. Guidance for evaluating firmness of the in situ soil relative to the bedding is contained in Tables C2.3-1 and C2.3-2 of the design specifications.

SPECIFICATIONS

6.2 Long-Span Culvert Foundations (Footings)

Construction of cast-in-place footings shall comply with the appropriate sections of these specifications. Precast footings shall be produced in accordance with the contract documents and the manufacturer's guidelines. Excavate the foundation soil as required for placement of the footings. Minimize disturbance at the base and sides of excavation. Any disturbed soil on which the footings are to be placed shall be compacted to provide the bearing required. Excavated zones at the sides of the footings shall be backfilled with the material specified for structural backfill, placed and compacted to the same requirements as the structural backfill.

6.3 Pipe Foundations

Boulders, rock, or soft spots in the foundation soil beneath pipe culverts shall be excavated to a suitable depth and backfilled with bedding material conforming to Section 6.4.

When the natural foundation soil is inadequate to support the culvert or structural backfill, such material shall be excavated and replaced by a layer of bedding material. Where an unstable and/or unsuitable material (e.g., peat or muck) is encountered at or below invert elevation during excavation, the necessary subsurface exploration and analysis shall be made and corrective treatments shall be as directed by the engineer.

COMMENTARY

C6.2

Special care may be necessary with rock or other unyielding foundations to cushion pipe from shock when blasting can be anticipated in the area.

C6.3

SPECIFICATIONS

6.4 Bedding

Bedding shall be provided for precast reinforced concrete circular, arch, and elliptical pipe for the type of installation specified. These shall be in conformance with Figures 27.5.2.2-1 to 4, which define soil areas and critical dimensions, and Tables 27.5.2.2-1 and 2, which list generic soil types, minimum compaction requirements, and minimum bedding thicknesses for the four Standard Installation Types. Box sections shall be bedded in conformance with Figure 27.5.2.3-1. The engineer may allow the use of natural soil for bedding if suitable.

7. Backfilling Procedures

Prior to construction of long-span structures, the manufacturer shall advise the contractor(s) and engineer of proper backfilling procedures.

Equipment and construction procedures used to backfill culverts shall be appropriate to meet the requirements for backfill density without damaging the structure.

COMMENTARY

C6.4

C7.

Backfilling begins with the culvert in place on the bedding or foundation soil as shown in Figures 27.2 and 27.5.2.2-1 to 4. Arches are on footings, while the pipes are directly on the bedding or foundation soil. There are three basic stages of backfilling: 1) haunch, 2) sidefill, and 3) topfill. Only the second and third apply to long-span concrete culverts.

Once a backfilling procedure is established, the primary inspection effort should be to ensure the procedure is followed. Only occasional checks of soil density may be required as long as the materials and procedures are unchanged.

SPECIFICATIONS

7.1 Backfilling under the Haunch

For pipe, material shall be carefully placed in the haunch zones to the limits shown on Figures 27.5.2.2-1 to 4, using mechanical or manual tampers or other means to fill all voids and meet the specified compaction levels. Installation of haunch fill shall be carried out on both sides simultaneously to avoid rolling the pipe. Also, the compaction force shall be limited and controlled so that the pipe is not lifted off grade.

For the haunch areas of Type 1, 2, and 3 Standard Installations, soils requiring 90% or greater of the maximum dry density per AASHTO T99 shall be placed in layers with a maximum thickness of 100 mm (4 in.) and compacted to obtain the required density.

Water jetting to achieve specified compaction is permitted only with advanced permission of the engineer.

COMMENTARY

C7.1

It is important that the selected tamping procedures achieve the design assumptions. In general, a minimum compaction level exceeding 85% of the maximum dry density per AASHTO T99 is needed to prevent a collapsing soil structure upon saturation.

Investigation of various means of achieving compaction in the haunch zone and the effect of haunch support on buried pipe performance is reported in FHWA Report FHWA-RD-98-191, *Pipe Interaction with the Backfill Envelope* (FHWA 1998). These studies showed that large void spaces result underneath pipes without good haunching effort.

Loose layers should generally not exceed 150 mm (6 in.) in thickness to permit the backfill material to be worked into the haunch zone. Shovel slicing was shown to be effective in providing haunch support. Different sized tampers were shown to be effective for different backfill soils. A large faced tamper (75 x 150 mm) was effective for silty sand, while a small-faced tamper (25 x 75 mm) was effective for crushed stone backfill.

Haunching is best accomplished by placing part of a layer of backfill, working it into the haunches and then placing the remainder of the lift. Thick layers block material from being worked into the haunch zone.

Water jetting can be an effective procedure for compacting backfill and developing uniform support with clean coarse material and good drainage; however, problems have been encountered in achieving consistent results, and verification is difficult.

SPECIFICATIONS

If the culvert is to be backfilled with CLSM, follow all requirements of the specifications and the submitted detailed work plan.

7.2 Placing and Compacting Sidefill

For pipe culverts, sidefill shall be placed to the limits shown on Figures 27.5.2.2-1 to 4. Equipment used to compact backfill within 1 m on each side of the culvert or from the edge of the footing for long-span culverts shall be approved by the engineer prior to use.

Structural backfill material in the sidefill zone shall be placed in horizontal, uniform layers not normally exceeding a 200 mm (8 in.) loose lift thickness for both pipe and long-span structures, or as specified. The layers shall be compacted with appropriate equipment to the specified density. For pipe culverts, backfill density shall be as specified in Tables 27.5.2.2-1 and 27.5.2.2-1. For large-span culverts, the maximum density shall not normally be less than 95% of the maximum dry density per AASHTO T99 for A-1 and A-3 soils, and 98% of the maximum dry density for A-2 soils.

The maximum difference in the backfill surface elevations between the two sides of the culvert at any time shall not exceed one-quarter of the span or diameter, but not more than 600 mm (2 ft). The sidefill material shall be constructed to the minimum lines and grades shown in the contract documents, keeping it at or below the level of adjacent soil or embankment. Placement and compaction of the sidefill layers adjacent to the haunch zone shall be carried out concurrently with backfilling under the haunch.

COMMENTARY

C7.2

Soils with increasing amounts of fines require a compaction to a higher percentage of maximum density to provide equal performance to soils with fewer fines. Design soil stiffness is very sensitive to the level of compaction (See FHWA Report FHWA-RD-98-191, *Pipe Interaction with the Backfill Envelope*).

Experience with compaction indicates that 150 mm (6 in.) thick loose layers using two coverages with a given compactor will give better uniformity and higher average level of compaction than one 300 mm (12 in.) thick loose layer with four coverages of the same compactor. Alternatively, a 300 mm loose layer will require larger compactors to produce the same average compaction as achieved by a smaller compactor with a 150 mm (6 in.) thick layer. Larger compactors should be evaluated for possible induced structural distortions.

SPECIFICATIONS	COMMENTARY

7.3 Placing and Compacting Topfill

When the sidefill elevation reaches the shoulders, placement of structural backfill over the top begins. For long-span culverts, backfill material should be pushed in from the sides and gradually moved up over the top of the culvert using light equipment. This process should continue until 0.3 to 0.6 m (1 to 2 ft) of loose material has been placed in a uniform layer over the top (topfill). A vibratory roller should be used to compact this first layer.

C7.3

As a general rule, construction equipment should not exceed the design load on a culvert. The AASHTO Design Truck is described in the *AASHTO LRFD Bridge Design Specifications*. Culverts can generally support live loads larger than the design truck during construction for short periods and with a limited number of loadings; however, equipment is so varied that specific analysis must be made for each project.

After 0.6 m (2 ft) of cover has been placed and compacted over the top of the culvert, subsequent layers should be placed per project specifications.

The first layer of backfill over the culvert may be left uncompacted or loosely compacted over the central third of the culvert, unless compaction is required for roadway function. This loose layer will reduce the load on the culvert.

The backfill beyond the structure must be well compacted at all elevations.

Unless otherwise provided in the contract documents, concrete culverts shall be covered to the design depth of fill, but not less than 600 mm (2 ft) before permitting heavy construction equipment to pass over. If not provided in the contract documents, specific analysis is required to verify the adequacy of culverts to carry construction loads in excess of the AASHTO design truck.

See design specifications for additional guidance on minimum cover depths.

SPECIFICATIONS

8. Compaction Control

Field compaction shall be evaluated based on compacted density and moisture content obtained from acceptable methods, such as the cone replacement (AASHTO T191, ASTM D1556) or the nuclear gage (AASHTO T238 and T239, ASTM D2922). A reference density test shall be performed on a representative sample to obtain a value of maximum dry density (MDD) and optimum moisture content (OMC). This test shall be repeated for each new soil type encountered and for composition variations within the same soil type. Thus, samples should be taken periodically during construction to provide an appropriate series of reference tests.

The number and location of field tests shall be determined by the engineer and selected to ensure that the quality of the soil and the compaction obtained is as specified. Furthermore, the engineer shall stipulate acceptance criteria for the compacted soil.

9. Construction of End Treatment

Substructures, headwalls, and wingwalls shall be designed and constructed in accordance with the applicable requirements of the *AASHTO LRFD Bridge Design Specifications* and these specifications.

Where single or multiple structures are installed at a skew to the embankment, proper support for the pipe shall be provided. Support may be achieved with a rigid, reinforced concrete headwall or by warping the embankment fill to provide the necessary balanced side support.

COMMENTARY

C8.

Compaction variability must be considered in determining the compaction acceptance criteria, i.e., for a given specified compaction level, the engineer must distinguish whether it is intended to serve as an average, so that 50 % of the samples can have a lower value, or a minimum, so that 100 % of the samples must exceed the specification, or whether some other allowable percentage below the specification is intended.

It is common to allow some density tests to fall below the specified density level. Retest procedures should be specified to ensure that the overall design objectives are met.

Some compaction tests should always be conducted in the sidefill region to verify that the pipe will have the specified side support.

C9.

SPECIFICATIONS

10. Inspections

In addition to the compaction tests described in Section 8, culverts shall be inspected after the completion of construction. Items inspected shall include alignment and grade, joints, and cracking, and other items as listed in the contract documents.

Cracks in an installed precast concrete culvert that exceed 0.01-in. width will be appraised by the engineer considering the structural integrity, environmental conditions, and the design service life of the culvert. Cracks determined to be detrimental shall be sealed by a method approved by the engineer.

11. Measurement

(No changes from existing specifications)

12. Payment

(No changes from existing specifications)

COMMENTARY

C10.

Personnel familiar with performance issues of concrete pipe should inspect all man-entry-size pipe. Pipe sizes smaller than man-entry may be inspected by video camera or, if requested in writing, the engineer may waive the need for inspection.

Generally, in non-corrosive environments, cracks 0.10 in. or less in width are considered acceptable. In corrosive environments, those cracks that are 0.01 in. or less in width are considered acceptable without repair.

C11.

C12.

APPENDIX H

EXAMPLE CALCULATIONS WITH SIMPLIFIED DESIGN PROCEDURES

APPENDIX H

EXAMPLE CALCULATIONS WITH SIMPLIFIED DESIGN PROCEDURES

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Examples of Simplified Design Procedures for Metal Culverts

Example of Simplified Design Procedures for Concrete Culverts

Design 9.7 m Span by 3.7 m Rise Low Profile Arch, H = 1.0 m

MathCad Units and Range Variables

$kN := 224.8 \cdot lbf$ $\quad MPa := 145.1379 \cdot \frac{lbf}{in^2}$ $\quad i := 1, 2 .. 4$

Mathcad Terminology:
:= defines a term
= presents result of a calcuation

$kPa := \frac{MPa}{1000}$ $\quad GPa := 1000 \cdot MPa$

Installation Conditions

Depth of burial	$H := 1.0 \cdot m$
Width of Structural Backfill	$W := 4.85 \cdot m$
Live load	
Design Tandem	$P := 222 \cdot kN$
Multiple presence factor	$mp := 1.2$
Tire length:	$L_o := 250 \cdot mm$
Axle plus Wheel Width	$W_T := 2300 \cdot mm$
Lane load	$Lane := 9.3 \cdot \frac{kN}{m}$
Lane load width	$Lane_W := 3 \cdot m$

Culvert Geometry

Span	$S := 9.7 \cdot m$
Rise	$R := 3.7 \cdot m$
Upper Rise	$R_u := 3.2 \cdot m$
Top Radius	$R_t := 6.3 \cdot m$
Top Angle	$\theta_{top} := 80 \cdot deg$
Span/ Rise Ratio	$\frac{S}{R} = 2.62$

$$Topchord := \sin\left(\frac{\theta_{top}}{2}\right) \cdot R_t \cdot 2$$

$$Topchord = 8.10\,m$$

Impact $\quad I_{mp} := if\left(H < 2.44 \cdot m, 1.33 - 0.33 \cdot \frac{H}{2.44 \cdot m}, 1\right)$ $\quad I_{mp} = 1.19$

Live load distribution with depth of fill $\quad LLDF := 1.15$

Culvert Material Properties

$F_y := 227.6 \cdot MPa$ $\quad E_p := 200 \cdot GPa$

Soil Properties:

Structural Backfill (Sn95): Ms selected from table in Specifications based on vertical pressure:

Density $\quad \gamma_s := 18.84 \cdot kN \cdot m^{-3}$

Friction angle (loose) $\quad \phi := 36 \cdot deg$

Poisson's ratio $\quad \nu := 0.3$

$p_{crown} := \gamma_s \cdot H$ $\quad p_{crown} = 19\,kPa$ $\quad M_{sCrown} := 15.5 \cdot MPa$

$p_{side} := \gamma_s \cdot \left(H + \frac{R}{2}\right)$ $\quad p_{side} = 54\,kPa$ $\quad M_{sSide} := 19.4 \cdot MPa$

$M_{sBottom} := 21 \cdot MPa$

Native soil: Soft Clay (See Table C2.3-1)

$M_{sN} := 5 \cdot MPa$

Design factors

Load factors	Earth	Max	$\gamma_{EMax} := 1.3$	Resistance factors	Thrust	$\phi_c := 0.7$
		Min	$\gamma_{EMin} := 0.9$		Bending	$\phi_b := 0.9$
	Live		$\gamma_{LL} := 1.75$		Soil	$\phi_s := 0.9$
					Buckling	$\phi_{bck} := 0.7$

Trial Section Properties

Structural Plate = 150 mm by 50 mm by 6.324 mm,
with circumferential stiffeners (2nd plate of same gage)

Basic plate: $I_u := 2395 \cdot \frac{mm^4}{mm}$ $\quad A := 7.74 \cdot \frac{mm^2}{mm}$

Stiffened plate: $I_p := 4790 \cdot \frac{mm^4}{mm}$ $\quad M_p := 54.92 \cdot \frac{kN \cdot m}{m}$ $\quad M_y := 38.21 \cdot \frac{kN \cdot m}{m}$

MINIMUM STIFFNESS

$$FF_{max} := 0.17 \cdot \frac{mm}{N}$$

$$FF := \frac{(2 \cdot R_t)^2 \cdot (1 - \sin(\phi))^3}{0.07 \cdot E_p \cdot I_p} \qquad FF = 0.17 \frac{mm}{N}$$

$$MinimumStiffness := if(FF < FF_{max}, "OK", "Stiffeners Required")$$

$$MinimumStiffness = "OK"$$

THRUST CAPACITY

Compute Vertical Arching Factor and Earth Load

1. $F_{W/S}$

$$K_{VAF_i} := \begin{array}{|c|}\hline 1.9 - 1.15\cdot\frac{W}{S} \\ \hline 1.2 \\ \hline\end{array}$$

$$K_{VAF} := \max(K_{VAF})$$

$$K_{VAF} = 1.32$$

$$SoilRatio := if\left(\frac{M_{sSide}}{M_{sN}} < 100, \frac{M_{sSide}}{M_{sN}}, 100\right)$$

$$F_{WS} := 1.2 + 0.5\cdot\log(SoilRatio)(K_{VAF} - 1.2)$$

$$F_{WS} = 1.24$$

2. $F_{S/R}$

$$F_{sr_i} := \begin{array}{|c|}\hline 1 - \frac{S}{R} \\ \hline 0 \\ \hline\end{array}$$

$$F_{SR} := \max(F_{sr})$$

$$F_{SR} = 0.00$$

3. $F_{H/S}$

$$hs_{lim_i} := \begin{array}{|c|}\hline 0.8 - 0.5\cdot\frac{S}{R} \\ \hline 0.3 \\ \hline\end{array}$$

$$HS_{lim} := \max(hs_{lim})$$

$$HS_{lim} = 0.30$$

$$F_{hs_i} := \begin{array}{|c|}\hline 2.5\cdot\left(HS_{lim} - \frac{H}{S}\right) \\ \hline 0 \\ \hline\end{array}$$

$$F_{HS} := \max(F_{hs})$$

$$F_{HS} = 0.49$$

4. VAF

$$VAF := F_{WS} + F_{SR} + F_{HS}$$

$$VAF = 1.73$$

5. Earth Load

$$K_{sp} := \text{if}\left(0.172 + 0.019 \cdot \frac{S}{R_u} < 0.5, 0.172 + 0.019 \cdot \frac{S}{R_u}, 0.5\right) \quad K_{sp} = 0.23$$

$$W_{SP} := \gamma_s \cdot S \cdot \left(H + K_{sp} \cdot R_u\right) \qquad W_{SP} = 317 \frac{kN}{m}$$

$$W_E := VAF \cdot W_{SP}$$

$$W_E = 548 \frac{kN}{m}$$

6. Lane Load

$$W_{Lane} := Lane \cdot \left(\frac{Lane_W}{Lane_W + LLDF \cdot H}\right) \qquad W_{Lane} = 6.7 \frac{kN}{m}$$

7. Live Load

$$L_L := L_o + LLDF \cdot H \qquad L_L = 1.40\,m$$

$$L_W := W_T + LLDF \cdot H \qquad L_W = 3.45\,m$$

$$W_{LL} := \frac{0.7 \cdot mp \cdot I_{mp} \cdot P \cdot R_t}{L_L \cdot L_W} \qquad W_{LL} = 291 \frac{kN}{m}$$

Total Factored Thrust

$$T_f := \frac{\gamma_{EMax} \cdot W_E + \gamma_{LL} \cdot W_{LL} + \gamma_{LL} \cdot W_{Lane}}{2} \qquad T_f = 616 \frac{kN}{m}$$

Check Capacity for Hoop Thrust

Factored Axial Resistance $\quad R_T := \phi_c \cdot F_y \cdot A \qquad R_T = 1234 \frac{kN}{m}$

$$Status_{Thrust} := \text{if}\left(R_T > T_f, \text{"OK"}, \text{"Redesign"}\right)$$

$$Status_{Thrust} = \text{"OK"}$$

Check Buckling Capacity

Stiffened top arc with live load thrust

$$R_h := \frac{11.4}{\left(11 + \frac{S}{H}\right)} \qquad R_h = 0.55$$

$$C_n := 0.55 \qquad K_b := \frac{(1 - 2\cdot\nu)}{(1-\nu)^2} \qquad K_b = 0.82$$

$$R_b := \left[1.2\cdot\phi_{bck}\cdot C_n\cdot\left(E_p\cdot I_p\right)^{\frac{1}{3}}\cdot\left(\left(\phi_s\cdot M_{sCrown}\cdot K_b\right)\right)^{\frac{2}{3}}\right]\cdot R_h \qquad R_b = 1270\,\frac{kN}{m}$$

$$T_f = 616\,\frac{kN}{m}$$

$$Status_{Bucklingtop} := if\left(R_b > T_f, \text{"OK"}, \text{"Redesign"}\right)$$

$$Status_{Bucklingtop} = \text{"OK"}$$

Unstiffened bottom arc without live load thrust

$$T_E := \frac{\gamma_{EMax}\cdot W_E}{2}$$

$$H_{bot} := H + 0.75\cdot R$$

$$R_h := \frac{11.4}{\left(11 + \frac{S}{H_{bot}}\right)} \qquad R_h = 0.84$$

$$C_n := 0.55$$

$$R_{bE} := \left[1.2\cdot\phi_{bck}\cdot C_n\cdot\left(E_p\cdot I_u\right)^{\frac{1}{3}}\cdot\left(\left(\phi_s\cdot M_{sBottom}\cdot K_b\right)\right)^{\frac{2}{3}}\right]\cdot R_h \qquad R_{bE} = 1883\,\frac{kN}{m}$$

$$T_E = 356\,\frac{kN}{m}$$

$$Status_{Bucklingbottom} := if\left(R_b > T_E, \text{"OK"}, \text{"Redesign"}\right)$$

$$Status_{Bucklingbottom} = \text{"OK"}$$

FLEXURAL CAPACITY

Bending stiffness factor $S_B := \dfrac{\phi_s \cdot M_{sSide} \cdot S^3}{E_p \cdot I_p}$ $S_B = 16634$

Earth Load Moment

$$K_{e_i} := \begin{array}{|c|}\hline 0.05 \cdot \left(1 - \dfrac{S_B}{S_B + 400}\right) \\ \hline 0.0025 \\ \hline \end{array}$$

$K_E := \max(K_e)$

$K_E = 0.0025$

$M_E := \gamma_s \cdot S^2 \cdot H \cdot K_E$ $M_E = 4.43 \dfrac{kN \cdot m}{m}$

Live Load Moment

$$K_{ll_i} := \begin{array}{|c|}\hline 0.02 \cdot \left(1.05 - \dfrac{S_B}{S_B + 800}\right) \\ \hline 0.001 \\ \hline \end{array}$$

$K_{LL} := \max(K_{ll})$

$K_{LL} = 0.0019$

$M_{LL} := 2 \cdot W_{LL} \cdot R_t \cdot K_{LL}$ $M_{LL} = 7.02 \dfrac{kN \cdot m}{m}$

Lane Load Moment - Compute with same formula as earth load moment

$$K_{Lane_i} := \begin{array}{|c|}\hline 0.05 \cdot \left(1 - \dfrac{S_B}{S_B + 400}\right) \\ \hline 0.0025 \\ \hline \end{array}$$

$K_{Lane} := \max(K_{Lane})$

$K_{Lane} = 0.0025$

$M_{Lane} := W_{Lane} \cdot S \cdot K_{Lane}$ $M_{Lane} = 0.16 \dfrac{kN \cdot m}{m}$

Check if total live load moment is greater than 15% of plastic moment capacity

$$\text{FlexureCheck} := \text{if}\left[\gamma_{LL} \cdot (M_{LL} + M_{Lane}) > 0.15 \cdot (\phi_b \cdot M_p), \text{"Required"}, \text{"Not Required"}\right]$$

FlexureCheck = "Required"

Construction Moment

Maximum allowed extension of top chord

$Topchord_{max} := 1.0$ times original top chord

Select R_{ntrial} to give $\delta_{chord} = 0.0$

$R_{ntrial} := 6.3 \cdot m$

$CurvMin := 0.005 \cdot m^{-1}$

$$\delta_{chord} := 2 \cdot R_{ntrial} \cdot \sin\left(\frac{\theta_{top} \cdot R_t}{2 \cdot R_{ntrial}}\right) - Topchord_{max} \cdot Topchord \qquad \delta_{chord} = 0.0000\,m$$

$$Curv := if\left(\left|\frac{1}{R_t} - \frac{1}{R_{ntrial}}\right| < CurvMin, CurvMin, \frac{1}{R_t} - \frac{1}{R_{ntrial}}\right) \qquad Curv = 0.005\,m^{-1}$$

$$M_{sidemax} := E_p \cdot I_p \cdot Curv \qquad M_{sidemax} = 4.79\,kN \cdot \frac{m}{m}$$

Minimum allowed top chord

$Topchord_{min} := 0.98$ times original top chord

Select R_{ntrial} to give $\delta_{chord} = 0.0$

$R_{ntrial} := 5.66 \cdot m$

$$\delta_{chord} := 2 \cdot R_{ntrial} \cdot \sin\left(\frac{\theta_{top} \cdot R_t}{2 \cdot R_{ntrial}}\right) - Topchord_{min} \cdot Topchord \qquad \delta_{chord} = 0.00\,m$$

$$Curv := if\left(\left|\frac{1}{R_t} - \frac{1}{R_{ntrial}}\right| < CurvMin, CurvMin, \frac{1}{R_t} - \frac{1}{R_{ntrial}}\right) \qquad Curv = -0.018\,m^{-1}$$

$$M_{sidemin} := E_p \cdot I_p \cdot Curv \qquad M_{sidemin} = -17.21\,\frac{kN \cdot m}{m}$$

$$MaxConstructionM := if\left(\left|M_{sidemax}\right| > \left|M_{sidemin}\right|, \left|M_{sidemax}\right|, \left|M_{sidemin}\right|\right)$$

$$ConstructionControl := if\left(MaxConstructionM < M_y, \text{"OK"}, \text{"Reduce Construction Moment"}\right)$$

$$ConstructionControl = \text{"OK"}$$

Total Moment

$$M_{u_i} := \begin{array}{|c|}\hline \gamma_{EMax} \cdot -M_{sidemin} - \gamma_{EMin} \cdot M_E + \gamma_{LL} \cdot (M_{LL}) \\ \hline \gamma_{EMin} \cdot M_{sidemax} + \gamma_{EMax} \cdot M_E + \gamma_{LL} \cdot (M_{LL} + M_{Lane}) \\ \hline \end{array}$$

$$M_u = \begin{pmatrix} 30.67 \\ 22.65 \end{pmatrix} \frac{kN \cdot m}{m} \qquad M_U := \max(M_u)$$

$$M_U = 30.67 \frac{kN \cdot m}{m}$$

Check total moment against total capacity

$$M_n := \phi_b \cdot M_p \qquad M_n = 49.43 \frac{kN \cdot m}{m}$$

$$Status_{Flexure} := if(M_n > M_U, \text{"OK"}, \text{"Redesign"})$$

$$Status_{Flexure} = \text{"OK"}$$

COMBINED THRUST AND BENDING

Reduce thrust to reflect that peak thrust and moment do not occur at same location

$$T_{fSh} := \frac{0.67 \cdot \left(\gamma_{EMax} \cdot W_E + \gamma_{LL} \cdot W_{LL} + \gamma_{LL} \cdot W_{Lane}\right)}{2} \qquad T_{fSh} = 413 \frac{kN}{m}$$

$$T_{fCr} := \frac{0.5 \cdot \gamma_{EMax} \cdot W_E + \gamma_{LL} \cdot W_{LL} + 0.5\gamma_{LL} \cdot W_{Lane}}{2} \qquad T_{fCr} = 435 \frac{kN}{m}$$

$$T_{fRed} := \text{if}\left(T_{fCr} > T_{fSh}, T_{fCr}, T_{fSh}\right) \qquad T_{fRed} = 435 \frac{kN}{m}$$

$$\text{Combined}_i := \boxed{\begin{array}{c} \dfrac{T_{fRed}}{R_T} + \dfrac{8}{9} \cdot \dfrac{M_U}{M_n} \\ \hline \dfrac{T_{fRed}}{2 \cdot R_T} + \dfrac{M_U}{M_n} \end{array}} \qquad \text{Combined} = \begin{pmatrix} 0.90 \\ 0.80 \end{pmatrix}$$

$$\frac{T_{fRed}}{R_T} = 0.35$$

$$\frac{M_U}{M_n} = 0.62$$

$$\text{Indx} := \text{if}\left(\frac{T_{fRed}}{R_T} \geq 0.2, 1, 2\right) \qquad \text{Indx} = 1.00$$

$$\text{Status}_{Combined} := \text{if}\left(\text{Combined}_{Indx} < 1, \text{"OK"}, \text{"Redesign"}\right)$$

$$\text{Status}_{Combined} = \text{"OK"}$$

DESIGN SUMMARY

$MinimumStiffness = "OK"$

$Status_{Thrust} = "OK"$

$Status_{Bucklingtop} = "OK"$

$Status_{Bucklingbottom} = "OK"$

$FlexureCheck = "Required"$

$Status_{Flexure} = "OK"$

$Status_{Combined} = "OK"$

$Combined_{Indx} = 0.90$

$A = 7.74 \frac{mm^2}{mm}$

$I_u = 2395 \frac{mm^4}{mm}$

$I_p = 4790 \frac{mm^4}{mm}$

$M_p = 54.92 \frac{kN \cdot m}{m}$

$T_f = 616 \frac{kN}{m}$

$M_U = 30.67 \frac{kN \cdot m}{m}$

$ConstructionControl = "OK"$

$Topchord_{max} = 1.00$

$Topchord_{min} = 0.98$

Notes

- Circumferential stiffeners are used to meet minimum stiffness requirement
- Seam strength must also be checked and must be greater than T_f.

Design 31.2 ft Span by 19 ft Rise Elliptical Culvert, H = 2 ft

MathCad Units and Range Variables

$ksi := 1000 \cdot psi$

$k := 1000 \cdot lbf$

$i := 1, 2 .. 4$

Mathcad Terminology:
:= defines a term
= presents result of a calcuation

Installation Conditions

Depth of burial	$H := 2.0 \cdot ft$
Width of Structural Backfill	$W := 30 \cdot ft$
Live load	
Design Tandem	$P := 50 \cdot k$
Multiple presence factor	$mp := 1.2$
Tire length:	$L_o := 10 \cdot in$
Axle plus Wheel Width	$W_T := 7.67 \cdot ft$
Lane load	$Lane := 640 \cdot \frac{lbf}{ft}$
Lane load width	$Lane_W := 10 \cdot ft$

Culvert Geometry

Span	$S := 31.2 \cdot ft$
Rise	$R := 19.0 \cdot ft$
Upper Rise	$R_u := 9.5 \cdot ft$
Top Radius	$R_t := 21.0 \cdot ft$
Top Angle	$\theta_{top} := 80 \cdot deg$
Span/ Rise Ratio	$\frac{S}{R} = 1.64$

$$Topchord := \sin\left(\frac{\theta_{top}}{2}\right) \cdot R_t \cdot 2$$

$$Topchord = 27.00\,ft$$

Impact $I_{mp} := if\left(H < 8 \cdot ft, 1.33 - 0.33 \cdot \frac{H}{8 \cdot ft}, 1\right)$ $I_{mp} = 1.25$

Live load distribution with depth of fill $LLDF := 1.15$

Culvert Material Properties

$F_y := 33 \cdot ksi$ $E_p := 29000 \cdot ksi$

Soil Properties:

Structural Backfill (Sn95): Ms selected from table in Specifications based on vertical pressure:

Density $\gamma_s := 120 \cdot lbf \cdot ft^{-3}$

Friction angle (loose) $\phi := 36 \cdot deg$

Poisson's ratio $\nu := 0.3$

$p_{crown} := \gamma_s \cdot H$ $p_{crown} = 1.67\,psi$

$p_{side} := \gamma_s \cdot \left(H + \frac{R}{2}\right)$ $p_{side} = 9.58\,psi$

$M_{sCrown} := 2100 \cdot psi$

$M_{sSide} := 2950 \cdot psi$

$M_{sBottom} := 3050 \cdot psi$

Native soil: Soft Clay (See Table C2.3-1)

$M_{sN} := 750 \cdot MPa$

Design factors

Load factors	Earth	Max	$\gamma_{EMax} := 1.3$	Resistance factors	Thrust	$\phi_c := 0.7$
		Min	$\gamma_{EMin} := 0.9$		Bending	$\phi_b := 0.9$
	Live		$\gamma_{LL} := 1.75$		Soil	$\phi_s := 0.9$
					Buckling	$\phi_{bck} := 0.7$

Trial Section Properties

Structural Plate = 6 in. by 2 in. by 0.276 in.,
with circumferential stiffeners (2nd plate of same gage)

Basic plate: $I_u := 0.166 \cdot \frac{in^4}{in}$ $\quad A := 4.119 \cdot \frac{in^2}{ft}$

Stiffened plate: $I_p := 0.332 \cdot \frac{in^4}{in}$ $\quad M_p := 167 \cdot \frac{in \cdot k}{ft}$ $\quad M_y := 115 \cdot \frac{in \cdot k}{ft}$

MINIMUM STIFFNESS

$$FF_{max} := 30 \cdot \frac{in}{k}$$

$$FF := \frac{(2 \cdot R_t)^2 \cdot (1 - \sin(\phi))^3}{0.07 \cdot E_p \cdot I_p} \qquad FF = 26.40 \frac{in}{k}$$

MinimumStiffness := if(FF < FF_{max}, "OK" , "Stiffeners Required")

MinimumStiffness = "OK"

THRUST CAPACITY

Compute Vertical Arching Factor and Earth Load

1. $F_{W/S}$

$$K_{VAF_i} := \begin{array}{|c|} \hline 1.9 - 1.15 \cdot \dfrac{W}{S} \\ \hline 1.2 \\ \hline \end{array}$$

$K_{VAF} := \max(K_{VAF})$

$K_{VAF} = 1.20$

$$SoilRatio := if\left(\frac{M_{sSide}}{M_{sN}} < 100, \frac{M_{sSide}}{M_{sN}}, 100\right)$$

$$F_{WS} := 1.2 + 0.5 \cdot \log(SoilRatio)(K_{VAF} - 1.2)$$

$F_{WS} = 1.20$

2. $F_{S/R}$

$$F_{sr_i} := \begin{array}{|c|} \hline 1 - \dfrac{S}{R} \\ \hline 0 \\ \hline \end{array}$$

$F_{SR} := \max(F_{sr})$

$F_{SR} = 0.00$

3. $F_{H/S}$

$$hs_{lim_i} := \begin{array}{|c|} \hline 0.8 - 0.5 \cdot \dfrac{S}{R} \\ \hline 0.3 \\ \hline \end{array}$$

$HS_{lim} := \max(hs_{lim})$

$HS_{lim} = 0.30$

$$F_{hs_i} := \begin{array}{|c|} \hline 2.5 \cdot \left(HS_{lim} - \dfrac{H}{S}\right) \\ \hline 0 \\ \hline \end{array}$$

$F_{HS} := \max(F_{hs})$

$F_{HS} = 0.59$

4. VAF

$$VAF := F_{WS} + F_{SR} + F_{HS}$$

$VAF = 1.79$

5. Earth Load

$$K_{sp} := \mathrm{if}\left(0.172 + 0.019\cdot\frac{S}{R_u} < 0.5, 0.172 + 0.019\cdot\frac{S}{R_u}, 0.5\right) \quad K_{sp} = 0.23$$

$$W_{SP} := \gamma_s\cdot S\cdot\left(H + K_{sp}\cdot R_u\right) \qquad W_{SP} = 16\,\frac{k}{ft}$$

$$W_E := VAF\cdot W_{SP}$$

$$W_E = 28\,\frac{k}{ft}$$

6. Lane Load

$$W_{Lane} := Lane\cdot\left(\frac{Lane_W}{Lane_W + LLDF\cdot H}\right) \qquad W_{Lane} = 0.5\,\frac{k}{ft}$$

7. Live Load

$$L_L := L_o + LLDF\cdot H \qquad L_L = 3.13\,ft$$

$$L_W := W_T + LLDF\cdot H \qquad L_W = 9.97\,ft$$

$$W_{LL} := \frac{0.7\cdot mp\cdot I_{mp}\cdot P\cdot R_t}{L_L\cdot L_W} \qquad W_{LL} = 35\,\frac{k}{ft}$$

Total Factored Thrust

$$T_f := \frac{\gamma_{EMax}\cdot W_E + \gamma_{LL}\cdot W_{LL} + \gamma_{LL}\cdot W_{Lane}}{2} \qquad T_f = 50\,\frac{k}{ft}$$

Check Capacity for Hoop Thrust

Factored Axial Resistance $\quad R_T := \phi_c\cdot F_y\cdot A \qquad R_T = 95\,\frac{k}{ft}$

$$Status_{Thrust} := \mathrm{if}\left(R_T > T_f, \text{"OK"}, \text{"Redesign"}\right)$$

$$Status_{Thrust} = \text{"OK"}$$

Check Buckling Capacity

Stiffened top arc with live load thrust

$$R_h := \frac{11.4}{\left(11 + \frac{S}{H}\right)} \qquad R_h = 0.43$$

$$C_n := 0.55 \qquad K_b := \frac{(1 - 2\cdot\nu)}{(1 - \nu)^2} \qquad K_b = 0.82$$

$$R_b := \left[1.2\cdot\phi_{bck}\cdot C_n\cdot\left(E_p\cdot I_p\right)^{\frac{1}{3}}\cdot\left(\left(\phi_s\cdot M_{sCrown}\cdot K_b\right)\right)^{\frac{2}{3}}\right]\cdot R_h \qquad R_b = 67\,\frac{k}{ft}$$

$$T_f = 50\,\frac{k}{ft}$$

$$Status_{Bucklingtop} := if\left(R_b > T_f, \text{"OK"}, \text{"Redesign"}\right)$$

$$Status_{Bucklingtop} = \text{"OK"}$$

Unstiffened bottom arc without live load thrust

$$T_E := \frac{\gamma_{EMax}\cdot W_E}{2}$$

$$H_{bot} := H + 0.75\cdot R$$

$$R_h := \frac{11.4}{\left(11 + \frac{S}{H_{bot}}\right)} \qquad R_h = 0.88$$

$$C_n := 0.55$$

$$R_{bE} := \left[1.2\cdot\phi_{bck}\cdot C_n\cdot\left(E_p\cdot I_u\right)^{\frac{1}{3}}\cdot\left(\left(\phi_s\cdot M_{sBottom}\cdot K_b\right)\right)^{\frac{2}{3}}\right]\cdot R_h \qquad R_{bE} = 141\,\frac{k}{ft}$$

$$T_E = 18\,\frac{k}{ft}$$

$$Status_{Bucklingbottom} := if\left(R_b > T_E, \text{"OK"}, \text{"Redesign"}\right)$$

$$Status_{Bucklingbottom} = \text{"OK"}$$

FLEXURAL CAPACITY

Bending stiffness factor

$$S_B := \frac{\phi_s \cdot M_{sSide} \cdot S^3}{E_p \cdot I_p} \qquad S_B = 14472$$

Earth Load Moment

$$K_{e_i} := \begin{array}{|l|} \hline 0.05 \cdot \left(1 - \frac{S_B}{S_B + 400}\right) \\ \hline 0.0025 \\ \hline \end{array}$$

$$K_E := \max(K_e)$$

$$K_E = 0.0025$$

$$M_E := \gamma_s \cdot S^2 \cdot H \cdot K_E \qquad M_E = 7.01 \frac{in \cdot k}{ft}$$

Live Load Moment

$$K_{ll_i} := \begin{array}{|l|} \hline 0.02 \cdot \left(1.05 - \frac{S_B}{S_B + 800}\right) \\ \hline 0.001 \\ \hline \end{array}$$

$$K_{LL} := \max(K_{ll})$$

$$K_{LL} = 0.0020$$

$$M_{LL} := 2 \cdot W_{LL} \cdot R_t \cdot K_{LL} \qquad M_{LL} = 36.35 \frac{in \cdot k}{ft}$$

Lane Load Moment - Compute with same formula as earth load moment

$$K_{Lane_i} := \begin{array}{|l|} \hline 0.05 \cdot \left(1 - \frac{S_B}{S_B + 400}\right) \\ \hline 0.0025 \\ \hline \end{array}$$

$$K_{Lane} := \max(K_{Lane})$$

$$K_{Lane} = 0.0025$$

$$M_{Lane} := W_{Lane} \cdot S \cdot K_{Lane} \qquad M_{Lane} = 0.49 \frac{in \cdot k}{ft}$$

Check if total live load moment is greater than 15% of plastic moment capacity

$$FlexureCheck := if\left[\gamma_{LL} \cdot (M_{LL} + M_{Lane}) > 0.15 \cdot (\phi_b \cdot M_p), \text{"Required"}, \text{"Not Required"}\right]$$

FlexureCheck = "Required"

Construction Moment

Maximum allowed extension of top chord

$Topchord_{max} := 1.002$ times original top chord

Select R_{ntrial} to give $\delta_{chord} = 0.0$ $\qquad R_{ntrial} := 21.254 \cdot ft$

$CurvMin := 0.0015 \cdot ft^{-1}$

$$\delta_{chord} := 2 \cdot R_{ntrial} \cdot \sin\left(\frac{\theta_{top} \cdot R_t}{2 \cdot R_{ntrial}}\right) - Topchord_{max} \cdot Topchord \qquad \delta_{chord} = -0.0001\,ft$$

$$Curv := if\left(\left|\frac{1}{R_t} - \frac{1}{R_{ntrial}}\right| < CurvMin, CurvMin, \frac{1}{R_t} - \frac{1}{R_{ntrial}}\right) \qquad Curv = 0.0015\,ft^{-1}$$

$$M_{sidemax} := E_p \cdot I_p \cdot Curv \qquad M_{sidemax} = 14.44\,\frac{in \cdot k}{ft}$$

Minimum allowed top chord

$Topchord_{min} := 0.995$ times original top chord

Select R_{ntrial} to give $\delta_{chord} = 0.0$ $\qquad R_{ntrial} := 20.40 \cdot ft$

$$\delta_{chord} := 2 \cdot R_{ntrial} \cdot \sin\left(\frac{\theta_{top} \cdot R_t}{2 \cdot R_{ntrial}}\right) - Topchord_{min} \cdot Topchord \qquad \delta_{chord} = -0.0002\,ft$$

$$Curv := if\left(\left|\frac{1}{R_t} - \frac{1}{R_{ntrial}}\right| < CurvMin, CurvMin, \frac{1}{R_t} - \frac{1}{R_{ntrial}}\right) \qquad Curv = 0.0015\,ft^{-1}$$

$$M_{sidemin} := E_p \cdot I_p \cdot Curv \qquad M_{sidemin} = 14.44\,\frac{in \cdot k}{ft}$$

$$MaxConstructionM := if\left(\left|M_{sidemax}\right| > \left|M_{sidemin}\right|, \left|M_{sidemax}\right|, \left|M_{sidemin}\right|\right)$$

$$ConstructionControl := if\left(MaxConstructionM < M_y, \text{"OK"}, \text{"Reduce Construction Moment"}\right)$$

$$ConstructionControl = \text{"OK"}$$

Total Moment

$$M_{u_i} :=$$

$$\boxed{\begin{array}{c} \gamma_{EMax} \cdot -M_{sidemin} - \gamma_{EMin} \cdot M_E + \gamma_{LL} \cdot (M_{LL}) \\ \hline \gamma_{EMin} \cdot M_{sidemax} + \gamma_{EMax} \cdot M_E + \gamma_{LL} \cdot (M_{LL} + M_{Lane}) \end{array}}$$

$$M_u = \begin{pmatrix} 38.53 \\ 86.57 \end{pmatrix} \frac{in \cdot k}{ft} \qquad M_U := \max(M_u)$$

$$M_U = 86.57 \frac{in \cdot k}{ft}$$

Check total moment against total capacity

$$M_n := \phi_b \cdot M_p \qquad M_n = 150.30 \frac{in \cdot k}{ft}$$

$$Status_{Flexure} := if(M_n > M_U, \text{"OK"}, \text{"Redesign"})$$

$$Status_{Flexure} = \text{"OK"}$$

COMBINED THRUST AND BENDING

Reduce thrust to reflect that peak thrust and moment do not occur at same location

$$T_{fSh} := \frac{0.67 \cdot \left(\gamma_{EMax} \cdot W_E + \gamma_{LL} \cdot W_{LL} + \gamma_{LL} \cdot W_{Lane}\right)}{2} \qquad T_{fSh} = 33 \frac{k}{ft}$$

$$T_{fCr} := \frac{0.5 \cdot \gamma_{EMax} \cdot W_E + \gamma_{LL} \cdot W_{LL} + 0.5 \gamma_{LL} \cdot W_{Lane}}{2} \qquad T_{fCr} = 40 \frac{k}{ft}$$

$$T_{fRed} := \mathrm{if}\left(T_{fCr} > T_{fSh}, T_{fCr}, T_{fSh}\right) \qquad T_{fRed} = 40 \frac{k}{ft}$$

$$\mathrm{Combined}_i := \begin{array}{|c|} \hline \frac{T_{fRed}}{R_T} + \frac{8}{9} \cdot \frac{M_U}{M_n} \\ \hline \frac{T_{fRed}}{2 \cdot R_T} + \frac{M_U}{M_n} \\ \hline \end{array} \qquad \mathrm{Combined} = \begin{pmatrix} 0.94 \\ 0.79 \end{pmatrix} \qquad \frac{T_{fRed}}{R_T} = 0.42$$

$$\frac{M_U}{M_n} = 0.58$$

$$\mathrm{Indx} := \mathrm{if}\left(\frac{T_{fRed}}{R_T} \geq 0.2, 1, 2\right) \qquad \mathrm{Indx} = 1.00$$

$$\mathrm{Status}_{Combined} := \mathrm{if}\left(\mathrm{Combined}_{Indx} < 1, \text{"OK"}, \text{"Redesign"}\right)$$

$$\mathrm{Status}_{Combined} = \text{"OK"}$$

DESIGN SUMMARY

MinimumStiffness = "OK"

$Status_{Thrust}$ = "OK"

$Status_{Bucklingtop}$ = "OK"

$Status_{Bucklingbottom}$ = "OK"

FlexureCheck = "Required"

$Status_{Flexure}$ = "OK"

$Status_{Combined}$ = "OK"

$Combined_{Indx} = 0.94$

$$A = 4.12 \frac{in^2}{ft} \qquad T_f = 50 \frac{k}{ft}$$

$$I_u = 0.1660 \frac{in^4}{in} \qquad M_U = 86.57 \frac{in \cdot k}{ft}$$

$$I_p = 0.3320 \frac{in^4}{in}$$

$$M_p = 167.00 \frac{in \cdot k}{ft}$$

ConstructionControl = "OK"

$Topchord_{max} = 1.00$

$Topchord_{min} = 0.99$

Notes

- Circumferential stiffeners are used to meet minimum stiffness requirement
- Seam strength must also be checked and must be greater than T_f.

Design 14.3 m Span by 8.6 m Elliptical Culvert, H = 2 m

MathCad Units and Range Variables

$kN := 224.8 \cdot lbf$ $MPa := 145.1379 \cdot \frac{lbf}{in^2}$ $i := 1, 2 .. 4$

Mathcad Terminology:
:= defines a term
= presents result of a calcuation

$kPa := \frac{MPa}{1000}$ $GPa := 1000 \cdot MPa$

Installation Conditions

Depth of burial	$H := 2.0 \cdot m$
Width of Structural Backfill	$W := 14.3 \cdot m$
Live load	
Design Tandem	$P := 222 \cdot kN$
Multiple presence factor	$mp := 1.2$
Tire length:	$L_o := 250 \cdot mm$
Axle + Wheel Width	$W_T := 2300 \cdot mm$
Lane load	$Lane := 9.3 \cdot \frac{kN}{m}$
Lane load width	$Lane_W := 3 \cdot m$

Culvert Geometry

Span	$S := 14.3 \cdot m$
Rise	$R := 8.6 \cdot m$
Upper Rise	$R_u := 4.3 \cdot m$
Top Radius	$R_t := 9.6 \cdot m$
Top Angle	$\theta_{top} := 80 \cdot deg$
Span/ Rise Ratio	$\frac{S}{R} = 1.66$

$$Topchord := \sin\left(\frac{\theta_{top}}{2}\right) \cdot R_t \cdot 2$$

$$Topchord = 12.34 m$$

Impact $I_{mp} := if\left(H < 2.44 \cdot m, 1.33 - 0.33 \cdot \frac{H}{2.44 \cdot m}, 1\right)$ $I_{mp} = 1.06$

Live load distribution with depth of fill $LLDF := 1.15$

Culvert Material Properties

$F_y := 227.6 \cdot MPa$ $E_p := 200 \cdot GPa$

Soil Properties:

Structural Backfill (Sn95): Ms selected from table in Specifications based on vertical pressure:

Density $\gamma_s := 18.84 \cdot kN \cdot m^{-3}$

Friction angle (loose) $\phi := 36 \cdot deg$

Poisson's ratio $\nu := 0.3$

$p_{crown} := \gamma_s \cdot H$ $p_{crown} = 38 kPa$ $M_{sCrown} := 18.1 \cdot MPa$

$p_{side} := \gamma_s \cdot \left(H + \frac{R}{2}\right)$ $p_{side} = 119 kPa$ $M_{sSide} := 23.1 \cdot MPa$

$M_{sBottom} := 24 \cdot MPa$

Native soil: Soft Clay (See Table C2.3-1)

$M_{sN} := 5 \cdot MPa$

Design factors

Load factors — Earth — Max: $\gamma_{EMax} := 1.3$

Min: $\gamma_{EMin} := 0.9$

Live: $\gamma_{LL} := 1.75$

Resistance factors

Thrust: $\phi_c := 0.7$

Bending: $\phi_b := 0.9$

Soil: $\phi_s := 0.9$

Buckling: $\phi_{bck} := 0.7$

Trial Section Properties

Structural Plate = 150 mm by 50 mm by 7.112 mm,
with circumferential stiffeners of same structural plate

Basic plate: $I_u := 2717 \cdot \frac{mm^4}{mm}$ $A := 8.72 \cdot \frac{mm^2}{mm}$

Stiffened plate: $I_p := 5434 \cdot \frac{mm^4}{mm}$ $M_p := 62.08 \cdot \frac{kN \cdot m}{m}$ $M_y := 42.79 \cdot \frac{kN \cdot m}{m}$

MINIMUM STIFFNESS

$$FF_{max} := 0.17 \cdot \frac{mm}{N}$$

$$FF := \frac{(2 \cdot R_t)^2 \cdot (1 - \sin(\phi))^3}{0.07 \cdot E_p \cdot I_p} \qquad FF = 0.34 \frac{mm}{N}$$

MinimumStiffness := if(FF < FF_{max}, "OK" , "Stiffeners Required")

MinimumStiffness = "Stiffeners Required"

NOTE: Use longitudinal stiffeners in addition to circumferential stiffeners (Longitudinal stiffeners not designe in this example)

THRUST CAPACITY

Compute Vertical Arching Factor and Earth Load

1. $F_{W/S}$

$$K_{VAF_i} := \begin{array}{|c|}\hline 1.9 - 1.15\cdot\frac{W}{S} \\ \hline 1.2 \\ \hline\end{array}$$

$$K_{VAF} := \max(K_{VAF})$$

$$K_{VAF} = 1.20$$

$$SoilRatio := if\left(\frac{M_{sSide}}{M_{sN}} < 100, \frac{M_{sSide}}{M_{sN}}, 100\right)$$

$$F_{WS} := 1.2 + 0.5\cdot\log(SoilRatio)(K_{VAF} - 1.2)$$

$$F_{WS} = 1.20$$

2. $F_{S/R}$

$$F_{sr_i} := \begin{array}{|c|}\hline 1 - \frac{S}{R} \\ \hline 0 \\ \hline\end{array}$$

$$F_{SR} := \max(F_{sr})$$

$$F_{SR} = 0.00$$

3. $F_{H/S}$

$$hs_{lim_i} := \begin{array}{|c|}\hline 0.8 - 0.5\cdot\frac{S}{R} \\ \hline 0.3 \\ \hline\end{array}$$

$$HS_{lim} := \max(hs_{lim})$$

$$HS_{lim} = 0.30$$

$$F_{hs_i} := \begin{array}{|c|}\hline 2.5\cdot\left(HS_{lim} - \frac{H}{S}\right) \\ \hline 0 \\ \hline\end{array}$$

$$F_{HS} := \max(F_{hs})$$

$$F_{HS} = 0.40$$

4. VAF

$$VAF := F_{WS} + F_{SR} + F_{HS}$$

$$VAF = 1.60$$

5. Earth Load

$$K_{sp} := \text{if}\left(0.172 + 0.019\cdot\frac{S}{R_u} < 0.5, 0.172 + 0.019\cdot\frac{S}{R_u}, 0.5\right) \quad K_{sp} = 0.24$$

$$W_{SP} := \gamma_s \cdot S \cdot \left(H + K_{sp}\cdot R_u\right) \qquad W_{SP} = 811\,\frac{kN}{m}$$

$$W_E := VAF \cdot W_{SP}$$

$$W_E = 1298\,\frac{kN}{m}$$

6. Lane Load

$$W_{Lane} := Lane\cdot\left(\frac{Lane_W}{Lane_W + LLDF\cdot H}\right) \qquad W_{Lane} = 5.3\,\frac{kN}{m}$$

7. Live Load

$$L_L := L_o + LLDF\cdot H \qquad L_L = 2.55\,m$$

$$L_W := W_T + LLDF\cdot H \qquad L_W = 4.60\,m$$

$$W_{LL} := \frac{0.7\cdot mp\cdot I_{mp}\cdot P\cdot R_t}{L_L\cdot L_W} \qquad W_{LL} = 162\,\frac{kN}{m}$$

Total Factored Thrust

$$T_f := \frac{\gamma_{EMax}\cdot W_E + \gamma_{LL}\cdot W_{LL} + \gamma_{LL}\cdot W_{Lane}}{2} \qquad T_f = 990\,\frac{kN}{m}$$

Check Capacity for Hoop Thrust

Factored Axial Resistance $R_T := \phi_c\cdot F_y\cdot A$ $\qquad R_T = 1390\,\frac{kN}{m}$

$$Status_{Thrust} := \text{if}\left(R_T > T_f, \text{"OK"}, \text{"Redesign"}\right)$$

$$Status_{Thrust} = \text{"OK"}$$

Check Buckling Capacity

Stiffened top arc with live load thrust

$$R_h := \frac{11.4}{\left(11 + \frac{S}{H}\right)} \qquad R_h = 0.63$$

$$C_n := 0.55 \qquad K_b := \frac{(1 - 2\cdot\nu)}{(1 - \nu)^2} \qquad K_b = 0.82$$

$$R_b := \left[1.2\cdot\phi_{bck}\cdot C_n\cdot\left(E_p\cdot I_p\right)^{\frac{1}{3}}\cdot\left(\left(\phi_s\cdot M_{sCrown}\cdot K_b\right)\right)^{\frac{2}{3}}\right]\cdot R_h \qquad R_b = 1676\,\frac{kN}{m}$$

$$T_f = 990\,\frac{kN}{m}$$

$$Status_{Bucklingtop} := if\left(R_b > T_f, \text{"OK"}, \text{"Redesign"}\right)$$

$$Status_{Bucklingtop} = \text{"OK"}$$

Unstiffened bottom arc without live load thrust

$$T_E := \frac{\gamma_{EMax}\cdot W_E}{2}$$

$$H_{bot} := H + 0.75\cdot R$$

$$R_h := \frac{11.4}{\left(11 + \frac{S}{H_{bot}}\right)} \qquad R_h = 0.90$$

$$C_n := 0.55$$

$$R_{bE} := \left[1.2\cdot\phi_{bck}\cdot C_n\cdot\left(E_p\cdot I_u\right)^{\frac{1}{3}}\cdot\left(\left(\phi_s\cdot M_{sBottom}\cdot K_b\right)\right)^{\frac{2}{3}}\right]\cdot R_h \qquad R_{bE} = 2296\,\frac{kN}{m}$$

$$T_E = 844\,\frac{kN}{m}$$

$$Status_{Bucklingbottom} := if\left(R_b > T_E, \text{"OK"}, \text{"Redesign"}\right)$$

$$Status_{Bucklingbottom} = \text{"OK"}$$

FLEXURAL CAPACITY

Bending stiffness factor $S_B := \dfrac{\phi_s \cdot M_{sSide} \cdot S^3}{E_p \cdot I_p}$ $S_B = 55939$

Earth Load Moment

$$K_{e_i} := \begin{array}{|l|} \hline 0.05 \cdot \left(1 - \dfrac{S_B}{S_B + 400}\right) \\ \hline 0.0025 \\ \hline \end{array}$$

$K_E := \max(K_e)$

$K_E = 0.0025$

$M_E := \gamma_s \cdot S^2 \cdot H \cdot K_E$ $M_E = 19.26 \dfrac{kN \cdot m}{m}$

Live Load Moment

$$K_{ll_i} := \begin{array}{|l|} \hline 0.02 \cdot \left(1.05 - \dfrac{S_B}{S_B + 800}\right) \\ \hline 0.001 \\ \hline \end{array}$$

$K_{LL} := \max(K_{ll})$

$K_{LL} = 0.0013$

$M_{LL} := 2 \cdot W_{LL} \cdot R_t \cdot K_{LL}$ $M_{LL} = 3.98 \dfrac{kN \cdot m}{m}$

Lane Load Moment - Compute with same formula as earth load moment

$$K_{Lane_i} := \begin{array}{|l|} \hline 0.05 \cdot \left(1 - \dfrac{S_B}{S_B + 400}\right) \\ \hline 0.0025 \\ \hline \end{array}$$

$K_{Lane} := \max(K_{Lane})$

$K_{Lane} = 0.0025$

$M_{Lane} := W_{Lane} \cdot S \cdot K_{Lane}$ $M_{Lane} = 0.19 \dfrac{kN \cdot m}{m}$

Check if total live load moment is greater than 15% of plastic moment capacity

$$\text{FlexureCheck} := \text{if}\left[\gamma_{LL} \cdot (M_{LL} + M_{Lane}) > 0.15 \cdot (\phi_b \cdot M_p), \text{"Required"}, \text{"Not Required"}\right]$$

FlexureCheck = "Not Required"

Construction Moment

Maximum allowed extension of top chord

$Topchord_{max} := 1.005$ times original top chord

Select R_{ntrial} to give $\delta_{chord} = 0.0$ $\qquad R_{ntrial} := 9.9 \cdot m$

$CurvMin := 0.005 \cdot m^{-1}$

$$\delta_{chord} := 2 \cdot R_{ntrial} \cdot \sin\left(\frac{\theta_{top} \cdot R_t}{2 \cdot R_{ntrial}}\right) - Topchord_{max} \cdot Topchord \qquad \delta_{chord} = 0.0003\,m$$

$$Curv := if\left(\left|\frac{1}{R_t} - \frac{1}{R_{ntrial}}\right| < CurvMin, CurvMin, \frac{1}{R_t} - \frac{1}{R_{ntrial}}\right) \qquad Curv = 0.005\,m^{-1}$$

$$M_{sidemax} := E_p \cdot I_p \cdot Curv \qquad M_{sidemax} = 5.44\,kN \cdot \frac{m}{m}$$

Minimum allowed top chord

$Topchord_{min} := 1.00$ times original top chord

Select R_{ntrial} to give $\delta_{chord} = 0.0$ $\qquad R_{ntrial} := 9.6 \cdot m$

$$\delta_{chord} := 2 \cdot R_{ntrial} \cdot \sin\left(\frac{\theta_{top} \cdot R_t}{2 \cdot R_{ntrial}}\right) - Topchord_{min} \cdot Topchord \qquad \delta_{chord} = 0.00\,m$$

$$Curv := if\left(\left|\frac{1}{R_t} - \frac{1}{R_{ntrial}}\right| < CurvMin, CurvMin, \frac{1}{R_t} - \frac{1}{R_{ntrial}}\right) \qquad Curv = 0.005\,m^{-1}$$

$$M_{sidemin} := E_p \cdot I_p \cdot Curv \qquad M_{sidemin} = 5.44\,\frac{kN \cdot m}{m}$$

$$MaxConstructionM := if\left(\left|M_{sidemax}\right| > \left|M_{sidemin}\right|, \left|M_{sidemax}\right|, \left|M_{sidemin}\right|\right)$$

$$ConstructionControl := if\left(MaxConstructionM < M_y, \text{"OK"}, \text{"Reduce Construction Moment"}\right)$$

$$ConstructionControl = \text{"OK"}$$

Total Moment

$$M_{u_i} := \begin{array}{|c|}\hline \gamma_{EMax} \cdot -M_{sidemin} - \gamma_{EMin} \cdot M_E + \gamma_{LL} \cdot (M_{LL}) \\ \hline \gamma_{EMin} \cdot M_{sidemax} + \gamma_{EMax} \cdot M_E + \gamma_{LL} \cdot (M_{LL} + M_{Lane}) \\ \hline \end{array}$$

$$M_u = \begin{pmatrix} -17.44 \\ 37.23 \end{pmatrix} \frac{kN \cdot m}{m} \qquad M_U := \max(M_u)$$

$$M_U = 37.23 \frac{kN \cdot m}{m}$$

Check total moment against total capacity

$$M_n := \phi_b \cdot M_p \qquad M_n = 55.87 \frac{kN \cdot m}{m}$$

$$Status_{Flexure} := if(M_n > M_U, "OK", "Redesign")$$

$$Status_{Flexure} = "OK"$$

COMBINED THRUST AND BENDING

Reduce thrust to reflect that peak thrust and moment do not occur at same location

$$T_{fSh} := \frac{0.67 \cdot \left(\gamma_{EMax} \cdot W_E + \gamma_{LL} \cdot W_{LL} + \gamma_{LL} \cdot W_{Lane}\right)}{2} \qquad T_{fSh} = 663 \frac{kN}{m}$$

$$T_{fCr} := \frac{0.5 \cdot \gamma_{EMax} \cdot W_E + \gamma_{LL} \cdot W_{LL} + 0.5\gamma_{LL} \cdot W_{Lane}}{2} \qquad T_{fCr} = 566 \frac{kN}{m}$$

$$T_{fRed} := \mathrm{if}\left(T_{fCr} > T_{fSh}, T_{fCr}, T_{fSh}\right) \qquad T_{fRed} = 663 \frac{kN}{m}$$

$$\text{Combined}_i := \begin{array}{|c|} \hline \dfrac{T_{fRed}}{R_T} + \dfrac{8}{9} \cdot \dfrac{M_U}{M_n} \\ \hline \dfrac{T_{fRed}}{2 \cdot R_T} + \dfrac{M_U}{M_n} \\ \hline \end{array} \qquad \text{Combined} = \begin{pmatrix} 1.07 \\ 0.90 \end{pmatrix} \qquad \frac{T_{fRed}}{R_T} = 0.48 \qquad \frac{M_U}{M_n} = 0.67$$

$$\text{Indx} := \mathrm{if}\left(\frac{T_{fRed}}{R_T} \geq 0.2, 1, 2\right) \qquad \text{Indx} = 1.00$$

$$\text{Status}_{Combined} := \mathrm{if}\left(\text{Combined}_{Indx} < 1, \text{"OK"}, \text{"Redesign"}\right)$$

$$\text{Status}_{Combined} = \text{"Redesign"}$$

DESIGN SUMMARY

MinimumStiffness = "Stiffeners Required"

$Status_{Thrust}$ = "OK"

$Status_{Bucklingtop}$ = "OK"

$Status_{Bucklingbottom}$ = "OK"

FlexureCheck = "Not Required"

$Status_{Flexure}$ = "OK"

$Status_{Combined}$ = "Redesign"

$Combined_{Indx} = 1.07$

$A = 8.72 \frac{mm^2}{mm}$

$I_u = 2717 \frac{mm^4}{mm}$

$I_p = 5434 \frac{mm^4}{mm}$

$M_p = 62.08 \frac{kN \cdot m}{m}$

$T_f = 990 \frac{kN}{m}$

$M_U = 37.23 \frac{kN \cdot m}{m}$

ConstructionControl = "OK"

$Topchord_{max} = 1.00$

$Topchord_{min} = 1.00$

Notes

- Circumferential stiffeners are used to meet minimum stiffness requirement
- Seam strength must also be checked and must be greater than T_f.

Design 9.2 m Span by 9.0 m Rise Pear Shaped Culvert, H = 8 m

MathCad Units and Range Variables

$kN := 224.8 \cdot lbf$ $\quad MPa := 145.1379 \cdot \frac{lbf}{in^2}$ $\quad i := 1, 2..4$

Mathcad Terminology:
:= defines a term
= presents result of a calcuation

$kPa := \frac{MPa}{1000}$ $\quad GPa := 1000 \cdot MPa$

Installation Conditions

Depth of burial $H := 8.0 \cdot m$

Width of Structural Backfill $W := 4.6 \cdot m$

Live load

Design Tandem $P := 222 \cdot kN$

Multiple presence factor $mp := 1.2$

Tire length: $L_o := 250 \cdot mm$

Axle + Wheel Width $W_T := 1800 \cdot mm$

Lane load $Lane := 9.3 \cdot \frac{kN}{m}$

Lane load width $Lane_W := 3 \cdot m$

Impact $I_{mp} := if\left(H < 2.44 \cdot m, 1.33 - 0.33 \cdot \frac{H}{2.44 \cdot m}, 1\right)$ $\quad I_{mp} = 1.00$

Live load distribution with depth of fill $LLDF := 1.15$

Culvert Geometry

Span $S := 9.2 \cdot m$

Rise $R := 9.0 \cdot m$

Upper Rise $R_u := 3.0 \cdot m$

Top Radius $R_t := 6.7 \cdot m$

Top Angle $\theta_{top} := 67 \cdot deg$

Span/ Rise Ratio $\frac{S}{R} = 1.02$

Side Radius $R_{side} := 7.4 \cdot m$

$Topchord := \sin\left(\frac{\theta_{top}}{2}\right) \cdot R_t \cdot 2$

$Topchord = 7.40\, m$

Culvert Material Properties

$F_y := 227.6 \cdot MPa$ $\quad E_p := 200 \cdot GPa$

Soil Properties:

Structural Backfill (Sn95): Ms selected from table in Specifications based on vertical pressure:

Density $\gamma_s := 18.84 \cdot kN \cdot m^{-3}$

Friction angle (loose) $\phi := 36 \cdot deg$

Poisson's ratio $\nu := 0.3$

$p_{crown} := \gamma_s \cdot H$ $\quad p_{crown} = 151\, kPa$ $\quad M_{sCrown} := 24.6 \cdot MPa$

$p_{side} := \gamma_s \cdot \left(H + \frac{R}{2}\right)$ $\quad p_{side} = 235\, kPa$ $\quad M_{sSide} := 28.8 \cdot MPa$

$M_{sBottom} := 30 \cdot MPa$

Native soil: Soft Clay (See Table C2.3-1)

$M_{sN} := 5 \cdot MPa$

Design factors

Load factors	Earth	Max	$\gamma_{EMax} := 1.3$	Resistance factors	Thrust	$\phi_c := 0.7$
		Min	$\gamma_{EMin} := 0.9$		Bending	$\phi_b := 0.9$
	Live		$\gamma_{LL} := 1.75$		Soil	$\phi_s := 0.9$
					Buckling	$\phi_{bck} := 0.7$

Trial Section Properties

Structural Plate = 150 mm by 50 mm by 6.324 mm,
without circumferential stiffeners

Basic plate: $I_u := 2395 \cdot \frac{mm^4}{mm}$ $\quad A := 7.74 \cdot \frac{mm^2}{mm}$

Stiffened plate: $I_p := 2395 \cdot \frac{mm^4}{mm}$ $\quad M_p := 27.46 \cdot \frac{kN \cdot m}{m}$ $\quad M_y := 19.11 \cdot \frac{kN \cdot m}{m}$

MINIMUM STIFFNESS

Top Plates

$$FF_{max} := 0.17 \cdot \frac{mm}{N}$$

$$FF := \frac{(2 \cdot R_t)^2 \cdot (1 - \sin(\phi))^3}{0.07 \cdot E_p \cdot I_p} \qquad FF = 0.37 \frac{mm}{N}$$

$$MinimumStiffness := if(FF < FF_{max}, "OK", "Stiffeners Required")$$

$$MinimumStiffness = "Stiffeners Required"$$

Side Plates

$$FF_{max} := 0.17 \cdot \frac{mm}{N}$$

$$FF_{side} := \frac{(2 \cdot R_{side})^2 \cdot (1 - \sin(\phi))^3}{0.07 \cdot E_p \cdot I_p} \qquad FF_{side} = 0.46 \frac{mm}{N}$$

$$MinimumStiffness_{side} := if(FF < FF_{max}, "OK", "Increase Side Stiffness")$$

$$MinimumStiffness_{side} = "Increase Side Stiffness"$$

NOTE: Use heavier gage for side plates
or adjust flexibility requirement, or increase control
of side compaction

THRUST CAPACITY

Compute Vertical Arching Factor and Earth Load

1. $F_{W/S}$

$$K_{VAF_i} := \begin{array}{|c|}\hline 1.9 - 1.15 \cdot \dfrac{W}{S} \\ \hline 1.2 \\ \hline \end{array}$$

$$K_{VAF} := \max(K_{VAF})$$

$$K_{VAF} = 1.32$$

$$SoilRatio := if\left(\frac{M_{sSide}}{M_{sN}} < 100, \frac{M_{sSide}}{M_{sN}}, 100\right)$$

$$F_{WS} := 1.2 + 0.5 \cdot \log(SoilRatio)(K_{VAF} - 1.2)$$

$$F_{WS} = 1.25$$

2. $F_{S/R}$

$$F_{sr_i} := \begin{array}{|c|}\hline 1 - \dfrac{S}{R} \\ \hline 0 \\ \hline \end{array}$$

$$F_{SR} := \max(F_{sr})$$

$$F_{SR} = 0.00$$

3. $F_{H/S}$

$$hs_{lim_i} := \begin{array}{|c|}\hline 0.8 - 0.5 \cdot \dfrac{S}{R} \\ \hline 0.3 \\ \hline \end{array}$$

$$HS_{lim} := \max(hs_{lim})$$

$$HS_{lim} = 0.30$$

$$F_{hs_i} := \begin{array}{|c|}\hline 2.5 \cdot \left(HS_{lim} - \dfrac{H}{S}\right) \\ \hline 0 \\ \hline \end{array}$$

$$F_{HS} := \max(F_{hs})$$

$$F_{HS} = 0.00$$

4. VAF

$$VAF := F_{WS} + F_{SR} + F_{HS}$$

$$VAF = 1.25$$

5. Earth Load

$$K_{sp} := \text{if}\left(0.172 + 0.019\cdot\frac{S}{R_u} < 0.5, 0.172 + 0.019\cdot\frac{S}{R_u}, 0.5\right) \quad K_{sp} = 0.23$$

$$W_{SP} := \gamma_s\cdot S\cdot\left(H + K_{sp}\cdot R_u\right) \qquad W_{SP} = 1506\,\frac{kN}{m}$$

$$W_E := VAF\cdot W_{SP}$$

$$W_E = 1879\,\frac{kN}{m}$$

6. Lane Load

$$W_{Lane} := Lane\cdot\left(\frac{Lane_W}{Lane_W + LLDF\cdot H}\right) \qquad W_{Lane} = 2.3\,\frac{kN}{m}$$

7. Live Load

$$L_L := L_o + LLDF\cdot H \qquad L_L = 9.45\,m$$

$$L_W := W_T + LLDF\cdot H \qquad L_W = 11.00\,m$$

$$W_{LL} := \frac{0.7\cdot mp\cdot I_{mp}\cdot P\cdot R_t}{L_L\cdot L_W} \qquad W_{LL} = 12\,\frac{kN}{m}$$

Total Factored Thrust

$$T_f := \frac{\gamma_{EMax}\cdot W_E + \gamma_{LL}\cdot W_{LL} + \gamma_{LL}\cdot W_{Lane}}{2} \qquad T_f = 1234\,\frac{kN}{m}$$

Check Capacity for Hoop Thrust

Factored Axial Resistance $\quad R_T := \phi_c\cdot F_y\cdot A \qquad R_T = 1234\,\frac{kN}{m}$

$$Status_{Thrust} := \text{if}\left(R_T > T_f, \text{"OK"}, \text{"Redesign"}\right)$$

$$Status_{Thrust} = \text{"OK"}$$

Check Buckling Capacity

Stiffened top arc with live load thrust

$$R_h := \frac{11.4}{\left(11 + \frac{S}{H}\right)} \qquad R_h = 0.94$$

$$C_n := 0.55 \qquad K_b := \frac{(1 - 2\cdot\nu)}{(1 - \nu)^2} \qquad K_b = 0.82$$

$$R_b := \left[1.2\cdot\phi_{bck}\cdot C_n\cdot\left(E_p\cdot I_p\right)^{\frac{1}{3}}\cdot\left(\left(\phi_s\cdot M_{sCrown}\cdot K_b\right)\right)^{\frac{2}{3}}\right]\cdot R_h \qquad R_b = 2338\,\frac{kN}{m}$$

$$T_f = 1234\,\frac{kN}{m}$$

$$Status_{Bucklingtop} := if\left(R_b > T_f, "OK", "Redesign"\right)$$

$$Status_{Bucklingtop} = "OK"$$

Unstiffened bottom arc without live load thrust

$$T_E := \frac{\gamma_{EMax}\cdot W_E}{2}$$

$$H_{bot} := H + 0.75\cdot R$$

$$R_h := \frac{11.4}{\left(11 + \frac{S}{H_{bot}}\right)} \qquad R_h = 0.98$$

$$C_n := 0.55$$

$$R_{bE} := \left[1.2\cdot\phi_{bck}\cdot C_n\cdot\left(E_p\cdot I_u\right)^{\frac{1}{3}}\cdot\left(\left(\phi_s\cdot M_{sBottom}\cdot K_b\right)\right)^{\frac{2}{3}}\right]\cdot R_h \qquad R_{bE} = 2789\,\frac{kN}{m}$$

$$T_E = 1221\,\frac{kN}{m}$$

$$Status_{Bucklingbottom} := if\left(R_b > T_E, "OK", "Redesign"\right)$$

$$Status_{Bucklingbottom} = "OK"$$

FLEXURAL CAPACITY

Bending stiffness factor $S_B := \frac{\phi_s \cdot M_{sSide} \cdot S^3}{E_p \cdot I_p}$ $S_B = 42137$

Earth Load Moment

$$K_{e_i} := \begin{array}{|l|} \hline 0.05 \cdot \left(1 - \frac{S_B}{S_B + 400}\right) \\ \hline 0.0025 \\ \hline \end{array}$$

$K_E := \max(K_e)$

$K_E = 0.0025$

$M_E := \gamma_s \cdot S^2 \cdot H \cdot K_E$ $M_E = 31.89 \frac{kN \cdot m}{m}$

Live Load Moment

$$K_{ll_i} := \begin{array}{|l|} \hline 0.02 \cdot \left(1.05 - \frac{S_B}{S_B + 800}\right) \\ \hline 0.001 \\ \hline \end{array}$$

$K_{LL} := \max(K_{ll})$

$K_{LL} = 0.0014$

$M_{LL} := 2 \cdot W_{LL} \cdot R_t \cdot K_{LL}$ $M_{LL} = 0.22 \frac{kN \cdot m}{m}$

Lane Load Moment - Compute with same formula as earth load moment

$$K_{Lane_i} := \begin{array}{|l|} \hline 0.05 \cdot \left(1 - \frac{S_B}{S_B + 400}\right) \\ \hline 0.0025 \\ \hline \end{array}$$

$K_{Lane} := \max(K_{Lane})$

$K_{Lane} = 0.0025$

$M_{Lane} := W_{Lane} \cdot S \cdot K_{Lane}$ $M_{Lane} = 0.05 \frac{kN \cdot m}{m}$

Check if total live load moment is greater than 15% of plastic moment capacity

$$\text{FlexureCheck} := \text{if}\left[\gamma_{LL} \cdot (M_{LL} + M_{Lane}) > 0.15 \cdot (\phi_b \cdot M_p), \text{"Required"}, \text{"Not Required"}\right]$$

FlexureCheck = "Not Required"

Construction Moment

Maximum allowed extension of top chord

$Topchord_{max} := 1.0$ times original top chord

Select R_{ntrial} to give $\delta_{chord} = 0.0$ $R_{ntrial} := 6.7 \cdot m$

$CurvMin := 0.005 \cdot m^{-1}$

$$\delta_{chord} := 2 \cdot R_{ntrial} \cdot \sin\left(\frac{\theta_{top} \cdot R_t}{2 \cdot R_{ntrial}}\right) - Topchord_{max} \cdot Topchord \qquad \delta_{chord} = 0.0000\,m$$

$$Curv := if\left(\left|\frac{1}{R_t} - \frac{1}{R_{ntrial}}\right| < CurvMin, CurvMin, \frac{1}{R_t} - \frac{1}{R_{ntrial}}\right) \qquad Curv = 0.005\,m^{-1}$$

$$M_{sidemax} := E_p \cdot I_p \cdot Curv \qquad M_{sidemax} = 2.40\,\frac{kN \cdot m}{m}$$

Minimum allowed top chord

$Topchord_{min} := 0.98$ times original top chord

Select R_{ntrial} to give $\delta_{chord} = 0.0$ $R_{ntrial} := 5.778 \cdot m$

$$\delta_{chord} := 2 \cdot R_{ntrial} \cdot \sin\left(\frac{\theta_{top} \cdot R_t}{2 \cdot R_{ntrial}}\right) - Topchord_{min} \cdot Topchord \qquad \delta_{chord} = 0.00\,m$$

$$Curv := if\left(\left|\frac{1}{R_t} - \frac{1}{R_{ntrial}}\right| < CurvMin, CurvMin, \frac{1}{R_t} - \frac{1}{R_{ntrial}}\right) \qquad Curv = -0.024\,m^{-1}$$

$$M_{sidemin} := E_p \cdot I_p \cdot Curv \qquad M_{sidemin} = -11.42\,\frac{kN \cdot m}{m}$$

$$MaxConstructionM := if\left(\left|M_{sidemax}\right| > \left|M_{sidemin}\right|, \left|M_{sidemax}\right|, \left|M_{sidemin}\right|\right)$$

$$ConstructionControl := if\left(MaxConstructionM < M_y, \text{"OK"}, \text{"Reduce Construction Moment"}\right)$$

$$ConstructionControl = \text{"OK"}$$

Total Moment

$$M_{u_i} :=$$

$$\boxed{\begin{array}{l} \gamma_{EMax} \cdot -M_{sidemin} - \gamma_{EMin} \cdot M_E + \gamma_{LL} \cdot (M_{LL}) \\ \gamma_{EMin} \cdot M_{sidemax} + \gamma_{EMax} \cdot M_E + \gamma_{LL} \cdot (M_{LL} + M_{Lane}) \end{array}}$$

$$M_u = \begin{pmatrix} -13.47 \\ 44.10 \end{pmatrix} \frac{kN \cdot m}{m} \qquad M_U := \max(M_u)$$

$$M_U = 44.10 \frac{kN \cdot m}{m}$$

Check total moment against total capacity

$$M_n := \phi_b \cdot M_p \qquad M_n = 24.71 \frac{kN \cdot m}{m}$$

$$Status_{Flexure} := if(M_n > M_U, \text{"OK"}, \text{"Redesign"})$$

$$Status_{Flexure} = \text{"Redesign"}$$

COMBINED THRUST AND BENDING

Reduce thrust to reflect that peak thrust and moment do not occur at same location

$$T_{fSh} := \frac{0.67 \cdot \left(\gamma_{EMax} \cdot W_E + \gamma_{LL} \cdot W_{LL} + \gamma_{LL} \cdot W_{Lane}\right)}{2} \qquad T_{fSh} = 827 \frac{kN}{m}$$

$$T_{fCr} := \frac{0.5 \cdot \gamma_{EMax} \cdot W_E + \gamma_{LL} \cdot W_{LL} + 0.5 \gamma_{LL} \cdot W_{Lane}}{2} \qquad T_{fCr} = 622 \frac{kN}{m}$$

$$T_{fRed} := \text{if}\left(T_{fCr} > T_{fSh}, T_{fCr}, T_{fSh}\right) \qquad T_{fRed} = 827 \frac{kN}{m}$$

$$\text{Combined}_i := \begin{array}{|c|} \hline \dfrac{T_{fRed}}{R_T} + \dfrac{8}{9} \cdot \dfrac{M_U}{M_n} \\ \hline \dfrac{T_{fRed}}{2 \cdot R_T} + \dfrac{M_U}{M_n} \\ \hline \end{array} \qquad \text{Combined} = \begin{pmatrix} 2.26 \\ 2.12 \end{pmatrix} \qquad \frac{T_{fRed}}{R_T} = 0.67 \qquad \frac{M_U}{M_n} = 1.78$$

$$\text{Indx} := \text{if}\left(\frac{T_{fRed}}{R_T} \ge 0.2, 1, 2\right) \qquad \text{Indx} = 1.00$$

$$\text{Status}_{Combined} := \text{if}\left(\text{Combined}_{Indx} < 1, \text{"OK"}, \text{"Redesign"}\right)$$

$$\text{Status}_{Combined} = \text{"Redesign"}$$

DESIGN SUMMARY

$MinimumStiffness$ = "Stiffeners Required"

$MinimumStiffness_{side}$ = "Increase Side Stiffness"

$Status_{Thrust}$ = "OK"

$Status_{Bucklingtop}$ = "OK"

$Status_{Bucklingbottom}$ = "OK"

$FlexureCheck$ = "Not Required"

$Status_{Flexure}$ = "Redesign"

$Status_{Combined}$ = "Redesign"

$Combined_{Indx} = 2.26$

$$A = 7.74\,\frac{mm^2}{mm} \qquad T_f = 1234\,\frac{kN}{m}$$

$$I_u = 2395\,\frac{mm^4}{mm} \qquad M_U = 44.10\,\frac{kN\cdot m}{m}$$

$$I_p = 2395\,\frac{mm^4}{mm}$$

$$M_p = 27.46\,\frac{kN\cdot m}{m}$$

$ConstructionControl$ = "OK"

$Topchord_{max} = 1.00$

$Topchord_{min} = 0.98$

Notes

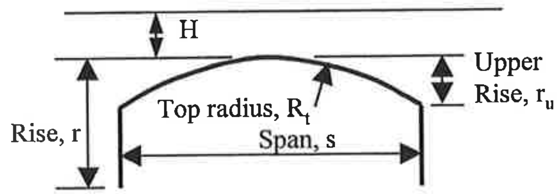
- Circumferential stiffeners are used to meet minimum stiffness requirement
- Seam strength must also be checked and must be greater than T_f.

Long Span Concrete Culvert - Load Calculations

English Units

Input

Geometry



Outside span $s_o := 460 \cdot in$

Rise $r := 138 \cdot in$

Upper rise $r_u := 54.18 \cdot in$

Top Radius $R_t := 486 \cdot in$

Thick. $Side_t := 14 \cdot in$ $Arch_t := 12 \cdot in$

Soil & Backfill

Initial Values: $Lat_1 := 0$ $Lat_2 := 0.3$

Backfill type: $Soil := \text{"Sn95"}$

$Lat_1 := if(Soil = \text{"Sn95"}, 0.05, 0)$

Soil unit weight $\gamma_s := 120 \cdot \frac{lbf}{ft^3}$

$Lat_1 := if(Soil = \text{"Sn90"} \vee Soil = \text{"Si95"}, .025, Lat_1)$

$Lat_2 := if(Soil = \text{"Sn95"} \vee Soil = \text{"Sn90"} \vee Soil = \text{"Si95"}, 0.4, Lat_2)$

Depth of fill $H := 2.0 \cdot ft$

$Lat_2 := if(Soil = \text{"Sn85"} \vee Soil = \text{"Si90"} \vee Soil = \text{"Cl95"}, 0.4, Lat_2)$

Lateral pressure coefficient

Final Values $Lat_1 = 0.050$ $Lat_2 = 0.40$

$K_H := Lat_2 + Lat_1 \cdot \frac{H}{3.28 \cdot ft}$ $K_H := if(K_H < 0.6, K_H, 0.6)$ $K_H = 0.43$

- No groundwater above footings of culvert
- Native soil: dense granular

Live load

Design Tandem, Load per axle $P := 25000 \cdot lbf$

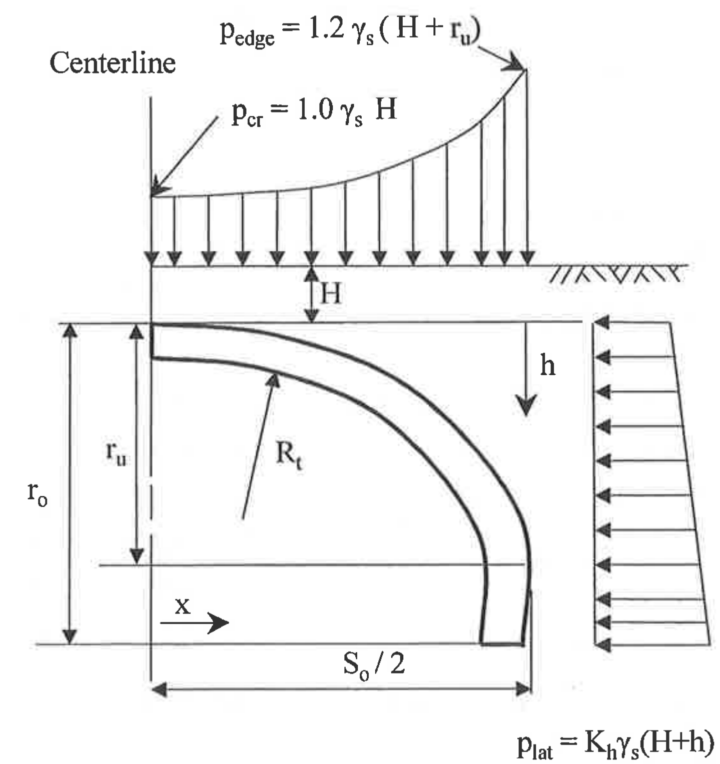
Multiple presence factor $mp := 1.2$

Impact Factor

$$I_0 := 1 + 0.33 \cdot \left(\frac{8 - \frac{H}{ft}}{8} \right)$$

$I_1 := 1$ $IM := max(I)$ $IM = 1.25$

Earth Loads



Vertical Pressure

Soil prism load

$$K_{VAF} := 0.172 + 0.019 \cdot \frac{s_o}{r_u} \qquad K_{VAF} = 0.33$$

$$W_{sp} := \gamma_s \cdot s_o \cdot \left(H + K_{VAF} \cdot r_u\right) \qquad W_{sp} = 16123\,\frac{lbf}{ft}$$

Soil pressure over crown

$$p_{cr} := \gamma_s \cdot H \qquad p_{cr} = 240\,\frac{lbf}{ft^2} \qquad p_{cr} = 20.00\,\frac{lbf}{in \cdot ft}$$

Soil pressure at edge

$$p_{edge} := 1.2 \cdot \gamma_s \cdot \left(H + r_u\right) \qquad p_{edge} = 938\,\frac{lbf}{ft^2} \qquad p_{edge} = 78.18\,\frac{lbf}{in \cdot ft}$$

Lateral Pressure

Top

$$p_{lattop} := K_H \cdot \gamma_s \cdot H \qquad p_{lattop} = 8.61\,\frac{lbf}{in \cdot ft}$$

Bottom

$$p_{latbot} := K_H \cdot \gamma_s \cdot (H + r) \qquad p_{latbot} = 58.12\,\frac{lbf}{in \cdot ft}$$

Total lateral load

$$P_{Lat} := \frac{\left(p_{lattop} + p_{latbot}\right)}{2} \cdot r \qquad P_{Lat} = 4604\,\frac{lbf}{ft}$$

Note: Frame model used in analysis will be based on centerline dimensions. To assure that all load on culvert is placed on the model the pressures need to be scaled up by the ratio of the outside dimensions to the centerline dimensions.

Vertical Pressures:

$Scv := \dfrac{s_o}{s_o - Side_t}$ $\quad Scv = 1.03$ $\quad p_{cr} := p_{cr} \cdot Scv$ $\quad p_{cr} = 20.63 \dfrac{lbf}{in \cdot ft}$

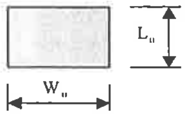
$p_{edge} := p_{edge} \cdot Scv$ $\quad p_{edge} = 80.63 \dfrac{lbf}{in \cdot ft}$

Lateral Pressures

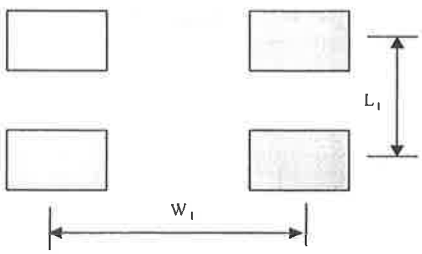
$Scl := \dfrac{r}{r - 0.5 \cdot Arch_t}$ $\quad Scl = 1.05$ $\quad p_{lattop} := p_{lattop} \cdot Scl$ $\quad p_{lattop} = 9.00 \dfrac{lbf}{in \cdot ft}$

$p_{latbot} := p_{latbot} \cdot Scl$ $\quad p_{latbot} = 60.76 \dfrac{lbf}{in \cdot ft}$

Live Load



a) Single tire



b) Multiple tires

Live load distribution factor: $LLDF := 1.15$

Wheel length: $L_o := 10 \cdot in$

Wheel width $W_o := 20.in$

Wheel spacing $W_1 := 72 \cdot in$

Axle spacing (width) $L_1 := 48 \cdot in$

Distribution width $D_w := 40 \cdot in$

Single wheel load* $P_w := \frac{P \cdot mp \cdot IM}{2}$

*includes multiple presence and impact factors

$P_w = 18713\,lbf$

At depth of structure:

Patch width for single wheel:	$WS_0 := W_o + LLDF \cdot H + D_w$	$WS_0 = 87.60\,in$
Patch width for multiple wheels: i.e. wheels interact.	$WS_1 := (W_o + W_1) + LLDF \cdot H + D_w$	$WS_1 = 159.60\,in$
Depth of interaction:	$H_{wint} := \frac{(W_1 - W_o - D_w)}{LLDF}$	$H_{wint} = 0.87\,ft$
Effective patch width:	$W_s := if(H < H_{wint}, WS_0, WS_1)$	$W_s = 159.60\,in$
	$P_{wa} := if(H < H_{wint}, P_w, 2P_w)$	$P_{wa} = 37425\,lbf$
Patch length for single wheel:	$LS_0 := (L_o + LLDF \cdot H)$	$LS_0 = 37.60\,in$
Patch length for multiple wheels: i.e. wheel pressures interact.	$LS_1 := [(L_o + L_1) + LLDF \cdot H]$	$LS_1 = 85.60\,in$
Depth of interaction:	$H_{lint} := \frac{(L_1 - L_o)}{LLDF}$	$H_{lint} = 2.754\,ft$
Effective patch length:	$L_s := if(H < H_{lint}, LS_0, LS_1)$	$L_s = 37.60\,in$
	$P_{wb} := if(H < H_{lint}, P_{wa}, 2P_{wa})$	$P_{wb} = 37425\,lbf$

<u>Effective live load pressure on structure:</u>

$$p := \frac{P_{wb}}{W_s \cdot L_s} \qquad p = 74.84 \frac{lbf}{in \cdot ft}$$

Length of effective pressure

$$L_s := if(L_s < s_o, L_s, s_o) \qquad L_s = 37.60\,in$$

Total live load $P := L_s \cdot p$ $\qquad P = 2814 \frac{lbf}{ft}$

Notes: 1. This analysis assumes that the length dimension is parallel to the direction of travel and that the direction of travel is across the culvert span.
2. If H is less than H_{lint}, than the structure must be loaded with two loads of magnitude, P, and length L_s, spaced a distance L_1.

CALCULATIONS TO EVALUATE REINFORCING REQUIREMENTS FOR CONSPAN REINFORCED CONCRETE CULVERT - Depth of Fill = 2.0 ft (0.6 m)

DIMENSIONS AND MATERIAL STRENGTHS

Horizontal span of culvert.. $S_i := 36 \cdot ft$

Crown radius.. $r_t := 72 \cdot ft$

Clear cover over reinforcement.. $t_b := 1.5 \cdot in$

Width of section being designed.. $b := 12 \cdot in \cdot ft^{-1}$

Design compressive strength of concrete.. $f_{cp} := 6 \cdot ksi$

Yield strength of steel reinforcement.. $f_y := 65 \cdot ksi$

Maximum developable stirrup material strength (not greater than f_y or anchorage strength).... $f_v := 60 \cdot ksi$

Spacing of circumferential reinforcement.. $s := 2 \cdot in$

Circumferential reinforcemenT provided in one (n=1) or multiple (n=2) layers........................ $n := 1$

Reinforcement Type.. $R_{type} := 2$

1 = smooth wire or plain bars

2 = welded smooth wire fabric with 8 in. maximum spacing of longitudinals

3 = welded deformed wire fabric, deformed wire, deformed bars or any reinforcement with stirrups

Load factor for selfweight.. $\gamma_{SW} := 1.35$

Load factor for earth pressure.. $\gamma_E := 0.9, 1.35$

Load factor for live load.. $\gamma_L := 1.35$

Resistance factor for flexure.. $\phi_f := 0.95$

Resistance factor for radial tension.. $\phi_r := 0.90$

Resistance factor for diagonal tension.. $\phi_v := 0.90$

Radial tension process factor.. $F_{rp} := 1.0$

Diagonal tesion process factor.. $F_{vp} := 1.0$

Crack control factor.. $F_{cr} := 0.9$

DESIGN FORCES (See next sheet for definitions and units)

Array index: $i := 1, 2 .. 17$

Design Forces From Frame Analysis:

$$M_{uactual} := \begin{pmatrix} 0.00 \\ -208.05 \\ -431.82 \\ -669.29 \\ -796.93 \\ -927.41 \\ -1060.47 \\ -804.51 \\ -580.65 \\ -381.99 \\ -203.63 \\ -41.14 \\ 108.06 \\ 246.50 \\ 376.38 \\ 473.09 \\ 497.71 \end{pmatrix} \quad N_u := \begin{pmatrix} 22.33 \\ 21.98 \\ 21.63 \\ 21.28 \\ 21.09 \\ 20.87 \\ 21.67 \\ 20.21 \\ 19.05 \\ 18.13 \\ 17.36 \\ 16.74 \\ 16.22 \\ 15.78 \\ 15.42 \\ 15.05 \\ 14.98 \end{pmatrix} \quad M_s := \begin{pmatrix} 0.00 \\ -129.55 \\ -276.57 \\ -438.81 \\ -528.03 \\ -620.41 \\ -715.65 \\ -532.10 \\ -375.01 \\ -238.77 \\ -119.33 \\ -13.13 \\ 82.00 \\ 168.13 \\ 247.01 \\ 304.89 \\ 319.81 \end{pmatrix} \quad N_s := \begin{pmatrix} 16.21 \\ 15.95 \\ 15.68 \\ 15.41 \\ 15.26 \\ 15.09 \\ 15.61 \\ 14.60 \\ 13.79 \\ 13.16 \\ 12.65 \\ 12.24 \\ 11.90 \\ 11.62 \\ 11.39 \\ 11.17 \\ 11.12 \end{pmatrix} \quad V_{vuactual} := \begin{pmatrix} -10.84 \\ -11.75 \\ -12.54 \\ -13.23 \\ -13.55 \\ -13.83 \\ 12.34 \\ 10.77 \\ 9.53 \\ 8.53 \\ 7.75 \\ 7.11 \\ 6.58 \\ 6.16 \\ 5.80 \\ 2.96 \\ 0.36 \end{pmatrix}$$

Forces from frame analysis:

Moments are in.-k/ft
Thrusts and shears are k/ft

$M_{uactual}$ = factored moment with proper sign + for tension on inside, - for tension on outside
M_u = moment with all signs positive.
N_u = factored thrust, + is compression
$V_{vuactual}$ = factored shear with proper sign
V_{vu} = factored shear with all signs positive
M_s = service load moment
N_s = service load thrust

Add Units to arrays and convert moments and shears to all positive signs:

$$M_u := M_{uactual} \cdot (in \cdot k) \cdot ft^{-1}$$

$$M_{u_i} := \text{if}\left(M_{u_i} < 0, -M_{u_i}, M_{u_i}\right)$$

$$N_u := N_u \cdot k \cdot ft^{-1}$$

$$V_{vu} := V_{vuactual} \cdot k \cdot ft^{-1}$$

$$V_{vu_i} := \text{if}\left(V_{vu_i} < 0, -V_{vu_i}, V_{vu_i}\right)$$

$$M_{vu_i} := M_{u_i}$$

$$N_{vu_i} := N_{u_i}$$

$$M_s := M_s \cdot in \cdot k \cdot ft^{-1}$$

$$M_{s_i} := \text{if}\left(M_{s_i} < 0, -M_{s_i}, M_{s_i}\right)$$

$$N_s := N_s \cdot k \cdot ft^{-1}$$

Note: Structure and all loads are symmetric, only Nodes 1 to 17 are presented in analysis.

Node 1 is base of leg
Node 7 is corner of segment
Node 17 is crown

Wall Thickness and Depth to Centroid from Compression Face

$$i = \begin{array}{|c|}\hline 1 \\ \hline 2 \\ \hline 3 \\ \hline 4 \\ \hline 5 \\ \hline 6 \\ \hline 7 \\ \hline 8 \\ \hline 9 \\ \hline 10 \\ \hline 11 \\ \hline 12 \\ \hline 13 \\ \hline 14 \\ \hline 15 \\ \hline 16 \\ \hline 17 \\ \hline \end{array} \qquad h := \begin{pmatrix} 14.00 \\ 14.00 \\ 14.00 \\ 14.31 \\ 15.91 \\ 20.04 \\ 23.24 \\ 20.84 \\ 16.14 \\ 13.30 \\ 12.21 \\ 12.0 \\ 12.00 \\ 12.00 \\ 12.00 \\ 12.00 \\ 12.00 \end{pmatrix} \qquad d := \begin{pmatrix} 12.02 \\ 12.02 \\ 12.02 \\ 12.33 \\ 13.93 \\ 18.06 \\ 21.26 \\ 18.96 \\ 14.16 \\ 11.32 \\ 10.23 \\ 10.02 \\ 10.02 \\ 10.02 \\ 10.02 \\ 10.02 \\ 10.02 \end{pmatrix}$$

Add units:

$$h := h \cdot in$$

$$d := d \cdot in$$

1.1 Reinforcement for Flexural Strength

Define the compressive strength per inch of thickness as........................ $g := 0.85 \cdot b \cdot f_{cp}$

$g = 5.10\,\mathrm{ksi}$

Required area of flexural steel

$$A_{sf_i} := \frac{1}{f_y} \cdot \left[g \cdot \phi_f \cdot d_i - N_{u_i} - \sqrt{g \cdot \left[g \cdot \left(\phi_f \cdot d_i\right)^2 - N_{u_i} \cdot \left(2 \cdot \phi_f \cdot d_i - h_i\right) - 2 \cdot M_{u_i} \right]} \right]$$

$A_{sf} =$ (in²/ft)

	1
1	-0.210
2	0.081
3	0.403
4	0.730
5	0.780
6	0.675
7	0.641
8	0.528
9	0.514
10	0.397
11	0.165
12	-0.094
13	0.021
14	0.259
15	0.488
16	0.663
17	0.708

$\frac{in^2}{ft}$

1.2 Minimum Flexural Reinforcement

Minimum reinforcement area....

$$A_{smin_i} := \text{for } j \in i$$
$$\left(\text{if}(i < 7, 0.002 \cdot 12 \cdot 14, .002 \cdot 12 \cdot 12) \cdot in^2 \cdot ft^{-1}\right)$$

Governing Reinforcement......... $A_{sf_i} := \text{if}\left(A_{smin_i} > A_{sf_i}, A_{smin_i}, A_{sf_i}\right)$

$A_{sf} =$

	1
1	0.336
2	0.336
3	0.403
4	0.730
5	0.780
6	0.675
7	0.641
8	0.528
9	0.514
10	0.397
11	0.288
12	0.288
13	0.288
14	0.288
15	0.488
16	0.663
17	0.708

$\frac{in^2}{ft}$

1.3 Maximum Flexural Reinforcement without Stirrups (Radial Tension)

Radius of the inside layer of reinforcement.................. $r_s := r_t + t_b$ .... $r_s = 865.50\,in$

Size factor for radial tension, fixed value for large span culverts.. $F_{rt} := 0.8$

Radial tension index:

$$M_{max} := -\min(M_{uactual}) \cdot in \cdot k \cdot ft^{-1}$$

$$a_i := \text{for } j \in i \quad \text{if}\left(\frac{-M_{max}}{in \cdot k \cdot ft^{-1}} \neq M_{uactual_i}, 0, i\right)$$

$nndx := \max(a)$ $\quad nndx = 7.00$

Parameters at Critical Radial Tension Section

$M_{max} = 1.06 \times 10^3\, in \cdot k \cdot ft^{-1}$ $\quad N_{u_{nndx}} = 21.67 k \cdot ft^{-1}$ $\quad d_{nndx} = 21.26\,in$

$$R_{rt} := \frac{M_{max} - 0.45 \cdot N_{u_{nndx}} \cdot d_{nndx}}{1.2 \cdot b \cdot d_{nndx} \cdot \phi_r \cdot r_s \cdot \sqrt{f_{cp} \cdot psi} \cdot F_{rt} \cdot F_{rp}}$$

$R_{rt} = 0.06$

Maximum Reinforcement Area

$$\beta_1 := \text{if}\left[f_{cp} < 4 \cdot ksi, 0.85, \text{if}\left[f_{cp} > 8ksi, 0.65, 0.85 - 0.05 \cdot \left(\frac{f_{cp}}{ksi} - 4\right)\right]\right]$$

$\beta_1 = 0.75$

$$A_{smax} := \frac{1}{f_y} \cdot \left(\frac{55 \cdot \phi_f \cdot \beta_1 \cdot d \cdot f_{cp}}{87 + f_y \cdot ksi^{-1}} - 0.75 \cdot N_u\right)$$

$A_{smax} =$ ($\frac{in^2}{ft}$)

	1
1	3.17
2	3.18
3	3.18
4	3.28
5	3.73
6	4.92
7	5.82
8	5.18
9	3.82
10	3.02
11	2.72
12	2.67
13	2.67
14	2.68
15	2.68
16	2.69

Evaluate limits on Maximum Reinforcement

$\text{RadialTension} := \text{if}\left(R_{rt} < 1, \text{"ok"}, \text{"Stirrups Required"}\right)$

$\text{RadialTension} = \text{"ok"}$

$\text{MaxCompression}_i := \text{if}\left(A_{sf_i} < A_{smax_i}, \text{"ok"}, \text{"NotOK"}\right)$

MaxCompression =

	1
1	"ok"
2	"ok"
3	"ok"
4	"ok"
5	"ok"
6	"ok"
7	"ok"
8	"ok"
9	"ok"
10	"ok"
11	"ok"
12	"ok"
13	"ok"
14	"ok"
15	"ok"
16	"ok"

Note: If maximum compression is "NotOK" then the options for redesign include:
- increase concrete strength
- increase depth of section
- design section as a compression member with ties

1.4 Flexural Reinforcement Requirements for Crack Width Control

Crack control coefficients........ $B_1 := \left(\dfrac{t_b \cdot s}{2 \cdot n \cdot in^2}\right)^{0.333}$ .... $B_1 = 1.145$

$C_1 := \text{if}(R_{type} \geq 3, 1.9, \text{if}(R_{type} \geq 2, 1.5, 1.0))$ .... $C_1 = 1.50$

Service level load eccentricity.............................. $e_i := \dfrac{M_{s_i}}{N_{s_i}} + d_i - \dfrac{h_i}{2}$

Lower bound enforced for $^e/_d$ ratio........................ $edratio_i := \text{if}\left(\dfrac{e_i}{d_i} > 1.15, \dfrac{e_i}{d_i}, 1.15\right)$

Flexural design parameter................................ $j_i := \text{if}\left(0.9 < 0.74 + \dfrac{edratio_i}{10}, 0.9, 0.74 + \dfrac{edratio_i}{10}\right)$

Flexural design parameter................................ $i_{cr_i} := \left(1 - \dfrac{j_i}{edratio_i}\right)^{-1}$

Moment-thrust contribution factor......................... $K_i := \dfrac{1}{i_{cr_i} \cdot j_i} \cdot \left[M_{s_i} + N_{s_i} \cdot \left(d_i - \dfrac{h_i}{2}\right)\right]$

Required area of flexural reinforcement steel for crack width control at service load design based upon **Equation B.7**... $A_{scr_i} := \dfrac{B_1 \cdot psi^{-1}}{30000 \cdot \phi_f \cdot d_i \cdot F_{cr}} \cdot \left[K_i - C_1 \cdot (h_i)^2 \cdot \sqrt{f_{cp} \cdot psi}\right]$

$A_{scr_i} := \text{if}\left[\left(A_{scr_i}\right) < 0, 0, A_{scr_i}\right]$

$e_i =$ (in)	$edratio_i =$	$j_i =$	$i_{cr_i} =$	$A_{scr_i} =$ ($in^2 \cdot ft^{-1}$)
5.02	1.15	0.86	3.90	0.00
13.14	1.15	0.86	3.90	0.00
22.66	1.89	0.90	1.91	0.00
33.65	2.73	0.90	1.49	0.36
40.58	2.91	0.90	1.45	0.39
49.15	2.72	0.90	1.49	0.00
55.49	2.61	0.90	1.53	0.00
44.99	2.37	0.90	1.61	0.00
33.28	2.35	0.90	1.62	0.00
22.81	2.02	0.90	1.81	0.00
13.56	1.33	0.87	2.93	0.00
5.09	1.15	0.86	3.90	0.00
10.91	1.15	0.86	3.90	0.00
18.49	1.85	0.90	1.95	0.00
25.71	2.57	0.90	1.54	0.05
31.32	3.13	0.90	1.40	0.34
32.78	3.27	0.90	1.38	0.41

Select limiting area based on cracking & flexure:.. $A_{s_i} := \text{if}\left(A_{scr_i} > A_{sf_i}, A_{scr_i}, A_{sf_i}\right)$

$i2 := 1, 2..2$

Governing positive reinforcement

$$A_{smpos_i} := \text{if}\left(M_{uactual_i} > 0, A_{s_i}, 0\right)$$

$$A_{sg_1} := \max\left(A_{smpos}\right) \qquad A_{sg_1} = 0.71\,\frac{in^2}{ft}$$

Governing Negative reinforcement

$$A_{smneg_i} := \text{if}\left(M_{uactual_i} < 0, A_{s_i}, 0\right)$$

$$A_{sg_2} := \max\left(A_{smneg}\right) \qquad A_{sg_2} = 0.78\,\frac{in^2}{ft}$$

$A_s =$ (in²/ft)

	1
1	0.34
2	0.34
3	0.40
4	0.73
5	0.78
6	0.67
7	0.64
8	0.53
9	0.51
10	0.40
11	0.29
12	0.29
13	0.29
14	0.29
15	0.49
16	0.66
17	0.71

Depth to tension reinforcement at governing locations:

User input is required

$$d_{g_1} := d_{17}$$

$$d_{g_2} := d_3$$

$$A_{sg} = \begin{pmatrix} 0.71 \\ 0.78 \end{pmatrix} \frac{in^2}{ft}$$

$$d_g = \begin{pmatrix} 10.02 \\ 12.02 \end{pmatrix} in$$

1.5 Shear Strength Calculations

Moment for $M/_{Vd}$ ratio .... $M_{nu_i} := M_{vu_i} - N_{vu_i} \cdot \left(\frac{4 \cdot h_i - d_i}{8} \right)$ $M_{nu_i} := \mathrm{if}\left(M_{nu_i} < 0, 0, M_{nu_i}\right)$

Reinforcement ratio............................... $\rho_{i2} := \mathrm{if}\left(0.02 < \frac{A_{sg_{i2}}}{b \cdot d_{g_{i2}}}, 0.02, \frac{A_{sg_{i2}}}{b \cdot d_{g_{i2}}} \right)$ $\rho_{i2} =$

0.0059
0.0054

$\rho_{g_i} := \mathrm{if}\left(M_{uactual_i} > 0, \rho_2, \rho_1\right)$

Factor for depth of section .... $F_{d_i} := 0.8 + \frac{1.6 \cdot in}{d_i}$ $F_{d_{i2}} := \mathrm{if}\left(F_{d_{i2}} > 1.3, 1.3, F_{d_{i2}}\right)$

Thrust factor .... $F_{n_i} := 1 + \frac{N_{vu_i} \cdot psi^{-1}}{24000 \cdot h_i}$

F_c factor for shear capacity calculation........... $F_{c_i} := 1 + \frac{d_i}{r_t + 0.5 \cdot h_i}$ $F_{c_i} := \mathrm{if}\left(i < 7, 1, \mathrm{if}\left(i > 26, 1, F_{c_i}\right)\right)$

$M_{nu} =$ ($\frac{in \cdot k}{ft}$), $\rho_g =$, $F_d =$, $F_n =$, $F_c =$

	M_{nu} (in·k/ft) 1	ρ_g 1	F_d 1	F_n 1	F_c 1
1	0	$5.89 \cdot 10^{-3}$	0.93	1.01	1.00
2	87	$5.89 \cdot 10^{-3}$	0.93	1.01	1.00
3	313	$5.89 \cdot 10^{-3}$	0.93	1.01	1.00
4	550	$5.89 \cdot 10^{-3}$	0.93	1.01	1.00
5	666	$5.89 \cdot 10^{-3}$	0.91	1.00	1.00
6	765	$5.89 \cdot 10^{-3}$	0.89	1.00	1.00
7	866	$5.89 \cdot 10^{-3}$	0.88	1.00	1.02
8	642	$5.89 \cdot 10^{-3}$	0.88	1.00	1.02
9	461	$5.89 \cdot 10^{-3}$	0.91	1.00	1.02
10	287	$5.89 \cdot 10^{-3}$	0.94	1.00	1.01
11	120	$5.89 \cdot 10^{-3}$	0.96	1.00	1.01
12	0	$5.89 \cdot 10^{-3}$	0.96	1.00	1.01
13	31	$5.41 \cdot 10^{-3}$	0.96	1.00	1.01
14	172	$5.41 \cdot 10^{-3}$	0.96	1.00	1.01
15	303	$5.41 \cdot 10^{-3}$	0.96	1.00	1.01
16	402	$5.41 \cdot 10^{-3}$	0.96	1.00	1.01
17	427	$5.41 \cdot 10^{-3}$	0.96	1.00	1.01

Shear capacity at critical section.................. $V_{b_i} := \phi_v \cdot b \cdot d_i \cdot F_{vp} \cdot \sqrt{f_{cp} \cdot psi} \cdot \left(1.1 + 63 \cdot \rho_{g_i}\right) \cdot \frac{F_{d_i} \cdot F_{n_i}}{F_{c_i}}$

$$MVD_i := if\left(\frac{M_{nu_i}}{V_{vu_i} \cdot d_i} > 3, 3, \frac{M_{nu_i}}{V_{vu_i} \cdot d_i}\right)$$

$$V_{c_i} := if\left(V_{b_i} \cdot \frac{4}{MVD_i + 1} > 4.5 \cdot \sqrt{f_{cp} \cdot psi} \cdot d_i, 4.5 \cdot \sqrt{f_{cp} \cdot psi} \cdot d_i, \frac{V_{b_i} \cdot 4}{MVD_i + 1}\right)$$

Diagonal tension index................................. $R_{dt_i} := \frac{V_{vu_i}}{V_{c_i}}$

Evaluate Diagonal Tension Strength $DT_{strength_i} := if\left(R_{dt_i} > 1, \text{"Strength Exceeded"}, \text{"OK"}\right)$

$V_b =$ ($\frac{k}{ft}$)

	1
1	13.88
2	13.88
3	13.88
4	14.18
5	15.76
6	19.82
7	22.43
8	20.27
9	15.72
10	13.01
11	11.96
12	11.76
13	11.51
14	11.51
15	11.51
16	11.51
17	11.51

$MVD_i =$	$V_{c_i} =$ ($\frac{k}{ft}$)	$R_{dt_i} =$
0.00	50.28	0.22
0.62	34.32	0.34
2.08	18.05	0.69
3.00	14.18	0.93
3.00	15.76	0.86
3.00	19.82	0.70
3.00	22.43	0.55
3.00	20.27	0.53
3.00	15.72	0.61
2.97	13.10	0.65
1.51	19.05	0.41
0.00	41.91	0.17
0.47	31.31	0.21
2.78	12.18	0.51
3.00	11.51	0.50
3.00	11.51	0.26
3.00	11.51	0.03

$DT_{strength} =$

	1
1	"OK"
2	"OK"
3	"OK"
4	"OK"
5	"OK"
6	"OK"
7	"OK"
8	"OK"
9	"OK"
10	"OK"
11	"OK"
12	"OK"
13	"OK"
14	"OK"
15	"OK"
16	"OK"
17	"OK"

Check if Circumferential Reinforcement Can Be Increased to Improve Shear Strength

$$A_{sinc_i} := \text{if}\left[R_{dt_i} > 1, \frac{0.01587 \cdot V_{vu_i}}{\phi_v \cdot F_{vp} \cdot \sqrt{f_{cp} \cdot psi}} \cdot \left(\frac{F_{c_i}}{F_{d_i} \cdot F_{n_i}}\right) - ((0.01746)) \cdot d_i, A_{s_i}\right]$$

$$\rho_{inc_i} := \frac{A_{sinc_i}}{b \cdot d_i} \qquad A_{sinc_i} := \text{if}\left(\rho_{inc_i} > 0.02, 10^5 \cdot in^2 \cdot ft^{-1}, A_{sinc_i}\right)$$

$$DT_{inc_i} := \text{if}\left(\rho_{inc_i} > 0.02, \text{"Stirrups Requ'd"}, \text{"OK"}\right)$$

If increased reinforcement ratio is greater than 2% than stirrups must be used

Governing Design $\quad A_{si_i} := \text{if}\left(M_{uactual_i} > 0, A_{sinc_i}, 0\right) \quad A_{sinside} := \max(A_{si})$

$A_{so_i} := \text{if}\left(M_{uactual_i} < 0, A_{sinc_i}, 0\right) \quad A_{soutside} := \max(A_{so})$

$A_{sinc} =$ ($\frac{in^2}{ft}$), $A_{si_i} =$ ($\frac{in^2}{ft}$), $A_{so_i} =$ ($\frac{in^2}{ft}$), $\rho_{inc_i} =$, $DT_{inc} =$

	A_{sinc} (1)	A_{si_i}	A_{so_i}	ρ_{inc_i}	DT_{inc} (1)
1	0.336	0.00	0.00	0.002	"OK"
2	0.336	0.00	0.34	0.002	"OK"
3	0.403	0.00	0.40	0.003	"OK"
4	0.730	0.00	0.73	0.005	"OK"
5	0.780	0.00	0.78	0.005	"OK"
6	0.675	0.00	0.67	0.003	"OK"
7	0.641	0.00	0.64	0.003	"OK"
8	0.528	0.00	0.53	0.002	"OK"
9	0.514	0.00	0.51	0.003	"OK"
10	0.397	0.00	0.40	0.003	"OK"
11	0.288	0.00	0.29	0.002	"OK"
12	0.288	0.00	0.29	0.002	"OK"
13	0.288	0.29	0.00	0.002	"OK"
14	0.288	0.29	0.00	0.002	"OK"
15	0.488	0.49	0.00	0.004	"OK"
16	0.663	0.66	0.00	0.006	"OK"
17	0.708	0.71	0.00	0.006	"OK"

Design Summmary:

Flexural Criteria Only $\quad A_{sg} = \begin{pmatrix} 0.71 \\ 0.78 \end{pmatrix} \frac{in^2}{ft}$

Flexure, crack, and diagonal tension criteria $\quad A_{sinside} = 0.71 \frac{in^2}{ft} \quad A_{soutside} = 0.78 \frac{in^2}{ft}$

Note - If As is listed as 10^5 in^2/ft, the shear strength cannot be adequately increased by increasing the circumferential reinforcement. Stirrup reinforcement, a thicker section or increased concrete strength are possible adjustements to the design.