would fail. The reliability of this slope is then said to be equal to 96.8 percent.

Based on the results of this study, it is concluded that

1. The probabilistic model developed here can be used to find a value of the probability of failure (or, the reliability) of a soil slope. This depends on the slope geometry and on the statistical values of the soil parameters.
2. The method can be applied to either deep or shallow failures. The kind of failure is reflected in the probability density functions of the coordinates of the center of the sliding surface.

REFERENCES

1. D. Athanasiou-Grivas. Reliability of Slopes of Particulate Materials. Ph.D. Thesis, Purdue Univ., Lafayette, IN, 1976.
2. L. Redulic. Ein Beitrag zur Bestimmung der Gleitsicherheit. Der Bauingenieur, No. 19/20, 1935.
3. O. K. Frölich. The Factor of Safety With Respect to Sliding of a Mass of Soil Along the Arc of a Logarithmic Spiral. Proc., 3rd International Conference on Soil Mechanics and Foundations Engineering, Switzerland, Vol. 2, 1953.
4. P. Lumb. Variability of Natural Soils. Canadian Geotechnical Journal, Vol. 3, No. 2, 1966.
5. M. E. Harr. Mechanics of Particulate Media—A Probabilistic Approach. McGraw-Hill, New York, 1977.
6. G. J. Hahn and S. S. Shapiro. Statistical Models in Engineering. Wiley, New York, 1967.
7. D. Athanasiou-Grivas and M. E. Harr. Stochastic Propagation of Rupture Surfaces Within Slopes. Proc., 2nd International Conference on Application of Statistics and Probability to Soil and Structure Engineering, Aachen, Germany, 1975.

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Soil-Culvert Interaction Method for Design of Metal Culverts

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A simple and rational method for the design of metal culverts, the soil-culvert interaction method, is described and compared to currently used design procedures. The principal advantage of the soil-culvert interaction method over those previously developed is that it provides a logical procedure for determining minimum required depth of cover, by consideration of the bending moments caused by live loads. Previously, minimum depths of cover have been determined empirically, using field experience. Values of minimum cover and maximum fill height determined using the soil-culvert interaction method are compared with values from published fill-height tables. The comparisons show that the soil-culvert interaction method gives values that are in good agreement with design experience for a wide range of corrugations and culvert diameters.

A simple method for design of metal culvert structures has been developed to provide rational procedures for designing culverts with deep or shallow cover. Design for deep cover is based on consideration of ring compression forces. Design for shallow cover is based on consideration of both ring compression forces and bending moments. The method, the soil-culvert interaction (SCI) method, is applicable to circular pipes, pipe arches, and arches constructed of corrugated steel or aluminum. It may be applied to structures having stiffening ribs that are curved to conform to the shape of the culvert barrel and attached to the barrel at frequent intervals. However, it is not applicable to soil bridge structures, which use straight ribs, fin plates, and sometimes strut to stiffen the upper part of the structure. The SCI method has been found to give values of maximum and minimum cover that are in good agreement with design experience as reflected in published fill-height tables and with the observed behavior of culverts in the field.

BASIS FOR SCI METHOD

The SCI design procedure is based on the results of finite element analyses, which modeled both the culvert structure and the surrounding backfill. Detailed results of the analyses and comparisons with field measurements were described by Duncan (1). Similar analyses were performed by Allgood and Takahashi (2), Abel and others (3), and Katona and others (4). The analyses on which the SCI method is based simulated the placement of backfill around and over the structure, and subsequent application of live loads on the surface of the backfill. Nonlinear and stress-dependent stress-strain relationships for the backfill soils were employed in the analyses. The results of these analyses were used to derive coefficients for ring compression forces and bending moments for design.

STEPS IN SCI DESIGN PROCEDURE

1. Calculate the rise/span ratio (R/S). The definitions of rise and span as used in this procedure are shown in Figure 1.
2. Calculate the maximum ring compression force

$$P = K_{p1}\gamma S^2 + K_{p2}\gamma HS + K_{p3}LL \qquad (1)$$

where

P = ring compression force (kN/m);
K_{p1} = ring compression coefficient or backfill, from Figure 2 (dimensionless);

Figure 1. Types of long-span metal culvert structures.

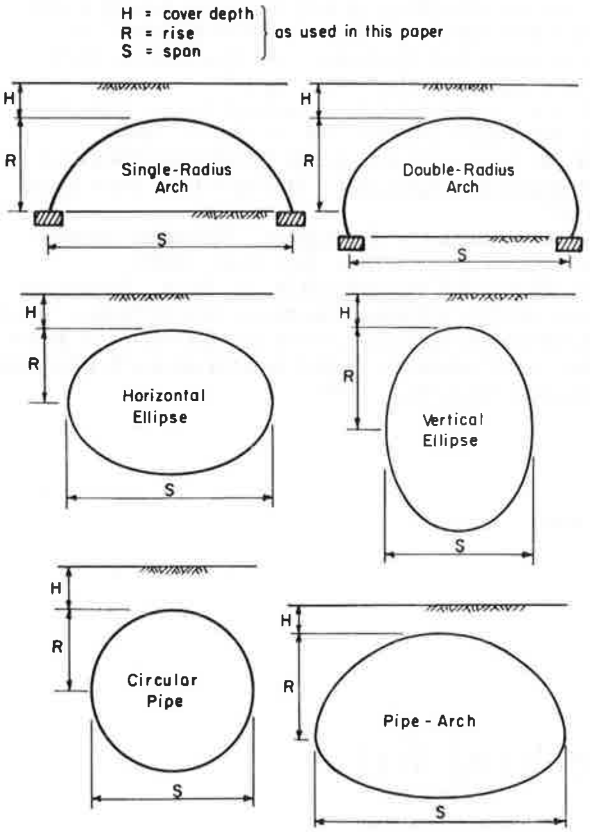
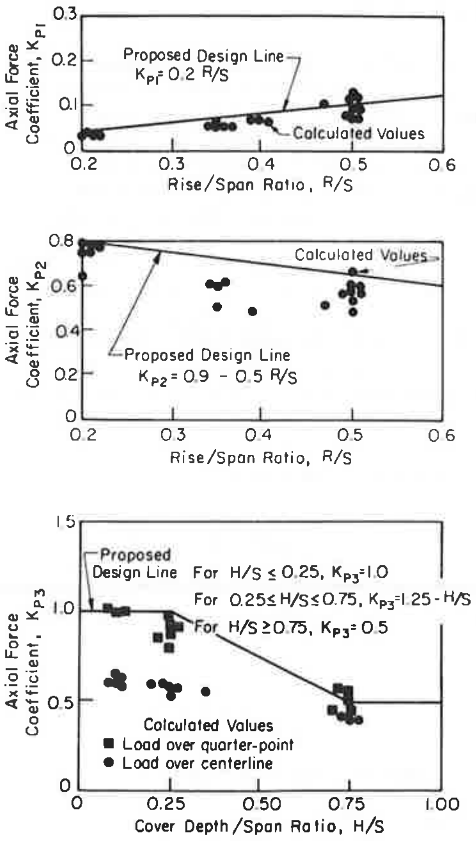


Figure 2. Axial force coefficients.



K_{p2} = ring compression coefficient for cover, from Figure 2 (dimensionless);
K_{p3} = ring compression coefficient for live load, from Figure 2 (dimensionless);
γ = unit weight of backfill (kN/m^3);
H = cover depth (m);
S = span (m); and
LL = live load (kN/m).

The table below gives values corresponding to H-20 traffic loading (1 m = 3.3 ft, 1kN/m = 74 lb/ft):

Cover Depth H (m)	Line Load LL (kN/m)	Cover Depth H (m)	Line Load LL (kN/m)
0.3	89	6.1	19
0.6	69	9.2	13
0.9	53	15.2	9
1.5	38	30.0	4
2.1	35	45.8	3
3.0	29	61.0	3
4.6	23		

Line load produces the same peak stress at depth H as do two HS-20 truck trailers that have single rear axles side by side on a two-lane road.

A section is chosen that has a seam strength sufficient to provide a factor of safety against seam compression failure that is equal to or greater than 1.50.

3. Calculate maximum bending moment at H = 0.

$$M_1 = K_{M1} R_B \gamma S^3 \quad (2)$$

where

M_1 = maximum bending moment at H = 0 (kN·m/m), which occurs at both the crown and the upper quarter-point;
K_{M1} = moment coefficient, from Figure 3 (dimensionless); and
R_B = moment reduction factor, from Figure 3 (dimensionless).

The value of the moment coefficient K_{M1} depends on the flexibility of the culvert section relative to the backfill, as defined by the flexibility number N_f:

$$N_f = (E_s S^3)/EI \quad (3)$$

where

N_f = flexibility number (dimensionless);
E_s = soil modulus, which depends on soil type, degree of compaction, and depth of overburden, from Figure 4 (kPa);
E = modulus of elasticity of metal culvert (mPa); and
I = moment of inertia of metal culvert (m^4/m).

The values of E_s shown in Figure 4 are based on the results of laboratory tests on over 100 different soils, which have been summarized by Wong and Duncan (5) and were selected to be representative of the behavior of the soils under the particular stress conditions that exist around flexible metal culverts.

The section should have sufficient moment capacity to withstand this bending moment and the corresponding axial force (calculated using Equation 1 for H = 0 and LL = 0) with a factor of safety against development of a plastic hinge (F_p) that is greater than or equal to 1.65.

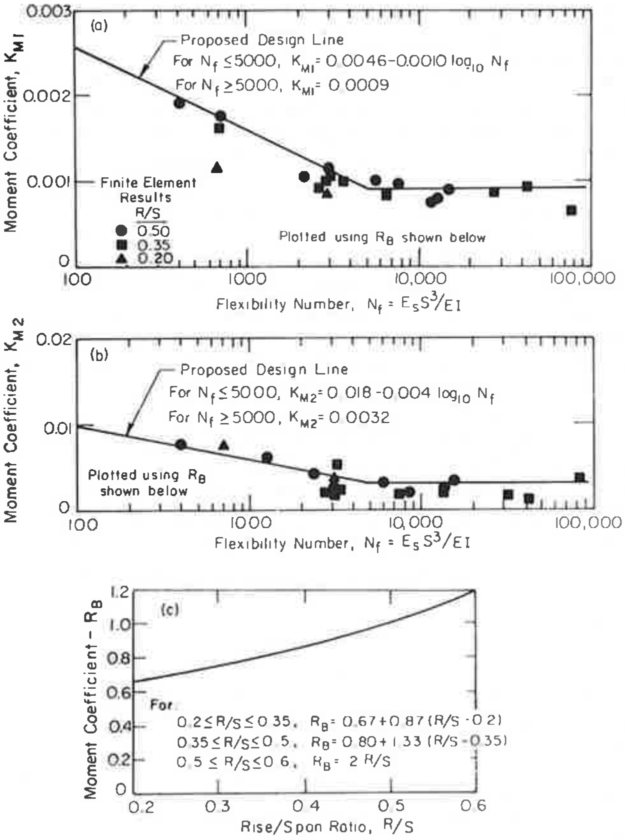
The value of E_p is calculated from the following equation

$$F_p = 0.5\, P_p/p\, [\sqrt{(M/M_p)^2 (P_p/p)^2 + 4} - (M/M_p)\, (P_p/p)] \quad (4)$$

where

F_p = factor of safety against formation of a plastic hinge, considering both axial force and moment;
P = axial force (kN/m);
P_p = fully plastic axial force, with no moment (kN/m);
M = bending moment (kN·m/m); and
M_p = fully plastic bending moment, with no axial force (kN·m/m).

Figure 3. Coefficients for backfill moments.



4. If the final depth of cover is greater than or equal to one-quarter of the span ($H \ge 0.25S$), bending need not be investigated for the final cover condition. If the final cover depth is less than one-quarter of the span ($H < 0.25S$), the bending moment due to both backfill and live load for the final cover condition are calculated using the following equation

$$M = M_1 - R_B K_{M2} \gamma S^2 H + R_L K_{M3} S\, LL \quad (5)$$

where

M = bending moment due to backfill and live load with cover depth H (kN·m/m);
M_1 = bending moment calculated previously for $H = 0$ (kN·m/m);
K_{M2} = moment coefficient, from Figure 3 (dimensionless);
R_L = moment reduction factor, from Figure 5 (dimensionless); and
K_{M3} = moment coefficient, from Figure 5 (dimensionless).

For purposes of determining K_{M2}, K_{M3}, and R_L, the value of N_f should be recalculated using a value of E_s corresponding to the final cover depth.

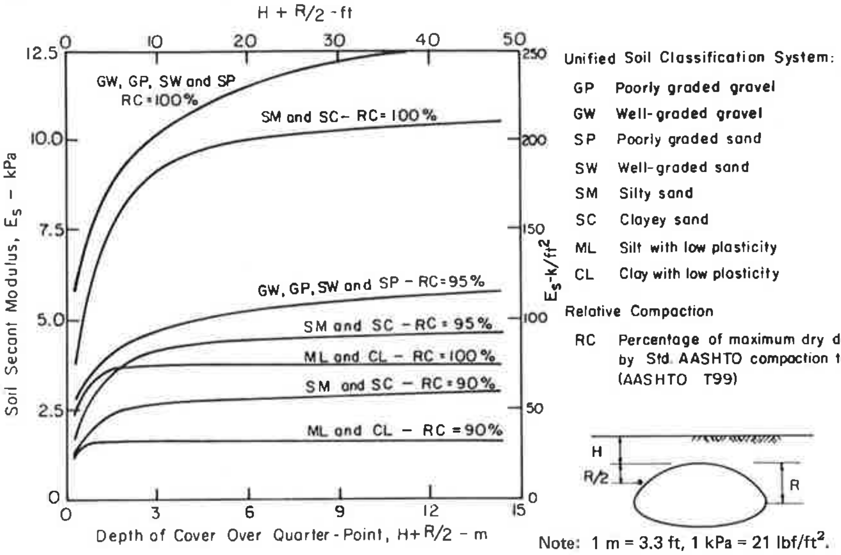
As for bending due to backfill loads at $H = 0$, the section chosen should have sufficient moment capacity to provide a factor of safety against development of a plastic hinge (F_p) greater than or equal to 1.65.

5. For arch structures, consideration must also be given to footing size, to ensure that the horizontal or vertical bearing pressures do not exceed the allowable values for the supporting soil. Similarly, for pipe arches, consideration must be given to the bearing pressures at the haunch.

MINIMUM COVER DEPTHS

Consideration of bending moments due to backfill loads and live loads in the SCI design method provides a rational means of establishing minimum cover depths

Figure 4. Approximate secant modulus values for various types of backfill.



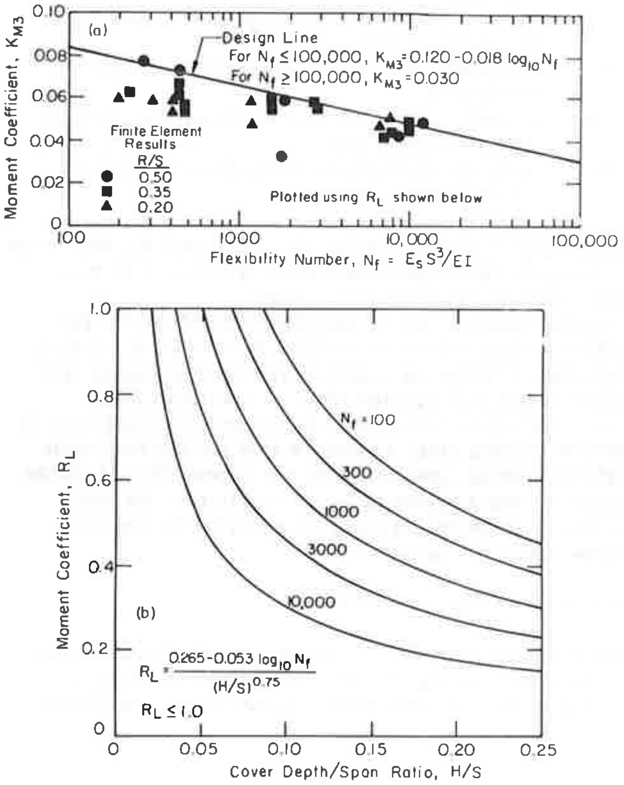
Note: 1 m = 3.3 ft, 1 kPa = 21 lbf/ft².

for live loads. As the depth of cover increases, the factors of safety against yield and development of a plastic hinge under live loads increase. By calculating factors of safety for a range of cover depths, it is possible to determine the minimum acceptable cover depth for a given culvert, backfill, and live load.

Minimum cover depths calculated using the SCI design method depend on a number of factors:

1. Culvert size or diameter,
2. Size of corrugation,
3. Metal thickness,
4. Yield stress of metal,
5. Backfill soil type,
6. Relative compaction of backfill, and
7. Magnitude of live load (only the HS-20 live load is considered in this paper).

Figure 5. Coefficients for live-load moments.



Minimum cover depths were determined for a range of culvert sizes and types so that these could be compared with minimum cover depths from published fill-height tables. These latter values are based on experience with field performance and thus provide a basis for determining if values calculated by the SCI design method are reasonable, because any method that gives values of minimum cover that differ greatly from those derived from long field experience must be considered not to be reflective of actual field behavior.

Minimum cover depths for 152 × 51-mm (6 × 2-in) corrugated steel circular pipe are shown in Table 1, together with those published by the U.S. Department of Transportation (DOT), Federal Highway Administration (FHWA), Bureau of Public Roads (BPR), American Iron and Steel Institute (AISI), and National Corrugated Steel Pipe Association (NCSPA). The criteria used in the SCI calculations were chosen to correspond closely to the conditions specified for the published fill-height tables: The value used for the yield stress of steel was 228 MPa (33 000 lb/in^2) in all cases. The unit weight of the backfill used in the SCI calculations was 19.6 KN/m^3 (125 lb/ft^3), compared to 18.8 kN/m^3 (120 lb/ft^3) specified for DOT, FHWA, BPR, AISI, and NCSPA fill-height tables. The relative compaction used in the SCI calculations was 90 percent of standard American Association of State Highway and Transportation Officials (AASHTO), as compared to 95 percent for the DOT, FHWA, and BPR tables and 85 percent for the AISI and NCSPA tables.

The values shown for the SCI method in Table 2 were calculated using the procedures outlined previously, except when the calculations indicated that a minimum cover depth less than 0.3 m (1 ft) would be acceptable. In those cases the minimum cover depth was made equal to 0.3 m, in accordance with experience and conventional practice.

It may be noted that the minimum cover depths calculated by the SCI method decrease with increasing metal thickness, except when the minimum is equal to 0.3 m. For example, for 4.5-m (15-ft) diameter pipes, the minimum cover depths calculated by the SCI method vary from 0.75 m (2.5 ft) for t = 2.77 mm (0.109 in or 12 gage) to 0.3 m for t = 7.1 mm (0.280 in or 1 gage). The other fill-height tables indicate a minimum cover depth of 0.6 m (2.0 ft) for all metal thicknesses. It is reasonable that minimum cover depth should decrease as metal thickness increases. One of the advantages of the SCI method over the use of experience alone for establishing

Table 1. Minimum cover depths for steel structural plate circular pipe HS-20 live load (152 x 51-mm corrugation).

		Minimum Depth of Cover (m)			
Diameter (m)	Source	2.8 mm Thick (12 gage)	4.3 mm Thick (8 gage)	5.5 mm Thick (5 gage)	7.1 mm Thick (1 gage)
1.5	SCI method	0.3	0.3	0.3	0.3
	DOT, FHWA, BPR	0.3	0.3	0.3	0.3
	AISI, NCSPA	0.3	0.3	0.3	0.3
3.0	SCI method	0.4	0.3	0.3	0.3
	DOT, FHWA, BPR	0.6	0.6	0.6	0.6
	AISI, NCSPA	0.5	0.5	0.5	0.5
4.6	SCI method	0.8	0.5	0.4	0.3
	DOT, FHWA, BPR	0.6	0.6	0.6	0.6
	AISI, NCSPA	0.6	0.6	0.6	0.6
6.1	SCI method		0.8	0.6	0.5
	DOT, FHWA, BPR			0.9	0.9
	AISI, NCSPA		0.8	0.8	0.8
7.6	SCI method				0.8
	DOT, FHWA, BPR				
	AISI, NCSPA				1.1

Notes: 1 m = 3.3 ft, 1 mm = 0.04 in.

Table 2. Minimum cover depths for corrugated steel circular pipe HS-20 live load (76 x 25-mm corrugation).

		Minimum Depth of Cover (m)			
Diameter (m)	Source	1.6 mm Thick (16 gage)	2.8 mm Thick (12 gage)	3.4 mm Thick (10 gage)	4.3 mm Thick (8 gage)
1.2	SCI method	0.3	0.3	0.3	0.3
	DOT, FHWA, BPR	0.3	0.3	0.3	0.3
	AISI, NCSPA	0.3	0.3	0.3	0.3
	US Steel	0.3	0.3	0.3	0.3
2.1	SCI method	0.5	0.4	0.3	0.3
	DOT, FHWA, BPR	0.3	0.3	0.3	0.3
	AISI, NCSPA	0.3	0.3	0.3	0.3
	US Steel	0.3	0.3	0.3	0.3
3.0	SCI method		0.6	0.5	0.4
	DOT, FHWA, BPR		0.6	0.6	0.6
	AISI, NCSPA		0.5	0.5	0.5
	US Steel		0.6	0.6	0.6

Notes: 1 m = 3.3 ft, 1 mm = 0.04 in.

Table 3. Minimum cover depths for corrugated steel circular pipe HS-20 live load (68 x 13-mm corrugation).

		Minimum Depth of Cover (m)			
Diameter (m)	Source	1.6 mm Thick (16 gage)	2.8 mm Thick (12 gage)	3.4 mm Thick (10 gage)	4.4 mm Thick (8 gage)
1.2	SCI method	0.3	0.3	0.3	0.3
	DOT, FHWA, BPR	0.3	0.3	0.3	0.3
	AISI, NCSPA	0.3	0.3	0.3	0.3
1.5	SCI method		0.3	0.3	0.3
	DOT, FHWA, BPR		0.3	0.3	0.3
	AISI, NCSPA		0.3	0.3	0.3
1.8	SCI method			0.4	0.3
	DOT, FHWA, BPR			0.3	0.3
	AISI, NCSPA			0.3	0.3
2.1	SCI method				0.4
	DOT, FHWA, BPR				0.3
	AISI, NCSPA				0.3

Notes: 1 m = 3.3 ft, 1 mm = 0.04 in.

Table 4. Minimum cover depths for corrugated aluminum circular pipe HS-20 live load (68 x 13-mm corrugation).

		Minimum Depth of Cover (m)			
Diameter (m)	Source	1.6 mm Thick (16 gage)	2.8 mm Thick (12 gage)	3.4 mm Thick (10 gage)	4.3 mm Thick (8 gage)
0.6	SCI method, KACS[a]	0.3	0.3	0.3	
	DOT, FHWA, BPR	0.3	0.3	0.3	0.3
1.2	SCI method, KACS[a]		0.4	0.4	0.3
	DOT, FHWA, BPR		0.3	0.3	0.3
1.8	SCI method, KACS[a]			0.5	0.4
	DOT, FHWA, BPR				0.3

Notes: 1 m = 3.3 ft, 1 mm = 0.04 in.
The values contained in the published fill-height tables (9) were established using the SCI method.

[a] KACS—Kaiser Aluminum and Chemical Sales, Inc.

minimum cover depths is that it provides a means of evaluating the benefits of increased metal thickness.

By comparing the values of minimum cover calculated using the SCI method with the values from the published fill-height tables, it may be seen that the calculated values are in good agreement with design experience. Because the values given by DOT, FHWA, BPR, AISI, and NCSPA tables are the same for all metal thicknesses in a given diameter, they are controlled by the requirements for the lightest gage, which requires the greatest depth of cover. The values of minimum cover calculated by the SCI method are in close agreement with the others for the lightest gage shown for each diameter, and smaller minimum cover depths are permitted by the SCI method for heavier gages.

Similar comparisons for 76 × 25-mm (3 × 1-in) corrugated steel pipe are shown in Table 2. In addition to values of minimum cover from the sources mentioned previously, those published by the U.S. Steel Corporation (10) are also shown in Table 2. Minimum cover depths for 68 × 13-mm (2⅔ × ½-in) corrugated steel pipe are given in Table 3, and values for 68 × 13-mm corrugated aluminum pipe are given in Table 4. In each case, the values given by the SCI method are in reasonable agreement with design experience as represented by the published fill-height tables.

MAXIMUM COVER DEPTHS

The factor of safety against seam compression failure calculated by the SCI design method provides a means of establishing maximum permissible cover depths for culverts. As shown in Table 5, similar values are used by DOT, FHWA, BPR (6), AISI (7), and NCSPA (8) design procedures, which also include criteria for buckling. However the buckling criterion is usually less critical than seam compression failure. DOT, FHWA, and BPR fill-height tables are also based on a limiting deflection equal to 5 percent of the nominal diameter, which controls the maximum fill heights in some cases.

Table 5. Criteria used in culvert design procedures.

Quantity	Value Used by Procedure Shown DOT, FHWA, BPR	AISI, NCSPA	SCI Method
Yield stress for steel, MPa	228	228	228
Yield stress for aluminum, MPa	166	Not used	166
Unit weight of backfill, kN/m^3	18.8	18.8	19.6
Relative compaction, % of Std. AASHTO max. dry density	95	85	90
Vertical load on culvert	Equal to weight of overlying soil	86% of weight of overlying soil	130% of weight of overlying soil
Factor of safety on seam strength	3.33	2.0	1.5
Factor of safety on development of a plastic hinge	Not used	Not used	1.65[a]
Factor of safety on yield stress or buckling stress	2.0[b]	2.0[b]	1.1[c]
Modulus of soil reaction-E', MPa	9.7	Not used	Not used
Soil stiffness coefficient-k	0.22	Not used	Not used
Limiting deflection	5%	Not used	Not used

Notes: 1 MPa = 145 lbf/in^2, 1 kN/m^3 = 6.4 lb/ft^3.

[a] For cover depth less than one-fourth of span.
[b] For axial stress only flexural stress not considered.
[c] For combined axial and flexural stress, with elastic design.

Table 6. Maximum fill heights for steel structural plate circular pipe HS-20 live load (152 x 51-mm corrugation).

Diameter (m)	Source	Maximum Fill Height (m)[a] 2.8 mm Thick (12 gage)	4.3 mm Thick (8 gage)	5.5 mm Thick (5 gage)	7.1 mm Thick (1 gage)
1.5	SCI method	22	42	59	71
	DOT, FHWA, BPR	13	25	32	38
	AISI, NCSPA	25	48	62	80
3.0	SCI method	10	20	27	36
	DOT, FHWA, BPR	7	12	15[b]	16[b]
	AISI, NCSPA	12	24	31	40
4.6	SCI method	6	12	18	23
	DOT, FHWA, BPR	4	8	11	13
	AISI, NCSPA	8	16	21	27
6.1	SCI method		9	13	17
	DOT, FHWA, BPR			9	11
	AISI, NCSPA		11[c]	14[c]	18[c]
7.6	SCI method				13
	DOT, FHWA, BPR				
	AISI, NCSPA				12[c]

Notes: 1 m = 3.3 ft, 1 mm = 0.04 in.

[a] Controlled by seam strength except as noted.
[b] Controlled by deflection criterion.
[c] Controlled by buckling criterion.

Table 7. Maximum fill heights for aluminum structural plate circular pipe HS-20 live load (229 x 64-mm corrugation with steel bolts).

Diameter (m)	Source	Maximum Fill Height (m)[a] 2.5 mm Thick	3.8 mm Thick	5.1 mm Thick	6.4 mm Thick
2.0	SCI method, KACS[b]	8	15	21	24
	DOT, FHWA, BPR			14	17
2.7	SCI method, KACS[b]	6	10	15	18
	DOT, FHWA, BPR			10	12
3.7	SCI method, KACS[b]	4	8	10	13
	DOT, FHWA, BPR			8	9
4.6	SCI method, KACS[b]		6	8	10
	DOT, FHWA, BPR			6	7

Notes: 1 m = 3.3 ft, 1 mm = 0.04 in.

[a] All values controlled by seam strength. Values for SCI and KACS determined using backfill unit weight = 22.0 kN/m^3. Values for DOT, FHWA, BPR determined using backfill unit weight = 19.6 kN/m^3.
[b] The values contained in the published fill height tables were established using the SCI method.

Calculated values of maximum fill height for steel structural plate pipe are given in Table 6. It may be noted that the values calculated using the SCI method are somewhat smaller than those calculated using the AISI and NCSPA procedure. The differences are due to differences in the vertical loads on the culverts and the factors of safety on seam strength. In some cases the maximum fill heights calculated using the AISI and NCSPA procedure are determined by buckling considerations, and in these cases the values are closer to those calculated by the SCI procedure.

The values calculated by the SCI method are consistently larger than those determined by the DOT, FHWA, and BPR procedure. The differences are due to differences in vertical load on the culvert and the factor of safety on seam strength, and also the deflection criterion used by the DOT, FHWA, and BPR procedure.

Maximum fill heights for aluminum structural plate pipe are given in Table 7. The values published by Kaiser Aluminum and Chemical Sales, Inc. (KACS) (9) were calculated using the SCI method and, therefore, are the same as the SCI values in all cases. The values calculated using the DOT, FHWA, and BPR procedure are smaller, for the same reasons discussed in reference to steel structural plate.

CONCLUSION

The SCI design procedure provides a rational method for determination of both minimum depths of cover and maximum fill heights. Values of minimum cover calculated using the SCI method compare well with design experience as reflected in values from published fill-height tables. The advantage of the SCI method is that it provides a rational procedure for including the effects of all the variables that affect minimum cover, namely diameter, corrugation size, metal thickness, yield stress, backfill type, degree of compaction, and magnitude of live load. The ability to account for the effects of these factors in a rational way is especially important for long-span culverts, where cover depths are often small.

Values of maximum cover calculated using the SCI method are somewhat smaller than those calculated using the AISI and NCSPA procedure. They are considerably larger than those calculated using the DOT, FHWA, and BPR procedure, especially in cases where the latter values are determined by consideration of calculated deflections.

Although all the examples of the use of the SCI method in this paper are for circular pipes, the method is also applicable to pipe-arch and arch structures. It is particularly useful for design of long-span culvert structures, for which considerations of performance under live load with shallow cover are of prime importance.

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REFERENCES

1. J. M. Duncan. Behavior and Design of Long-Span Metal Culvert Structures. Paper presented at the Technical Session on Soil-Structure Interaction for Shallow Foundations and Buried Structures, ASCE National Convention, San Francisco, Oct. 1977.
2. J. R. Allgood and S. K. Takahashi. Balanced Design and Finite Element Analysis of Culverts. HRB, Highway Research Record 413, 1972, pp. 45-56.
3. J. R. Abel, R. Mark, and R. Richards. Stresses Around Flexible Elliptic Pipes. Journal of the Soil Mechanics and Foundations Division, Proc., ASCE, Vol. 99, No. SM7, 1973, pp. 509-526.
4. M. G. Katona, J. B. Forrest, R. J. Odello, and J. R. Allgood. Computer Design and Analysis of Pipe Culvert. U.S. Naval Civil Engineering Laboratory, Port Hueneme, Internal Technical Rept., CA 51-040, FHWA 3-1-1170, 1974.
5. K. S. Wong and J. M. Duncan. Hyperbolic Stress-Strain Parameters for Nonlinear Analyses of Stresses and Movements in Earth Masses. University of California, Berkeley, Geotechnical Engineering Rept., TE 74-3, 1974.
6. E. W. Wolf and M. Townsend. Corrugated Metal Pipe: Structural Design Criteria and Recommended Installation Practice. U.S. Bureau of Public Roads, U.S. Government Printing Office, 1970.
7. Handbook of Steel Drainage and Highway Construction Products, 2nd Ed. American Iron and Steel Institute, New York, 1971.
8. Corrugated Steel Pipe. National Corrugated Steel Pipe Association, Chicago.
9. Aluminum Storm Sewers. Kaiser Aluminum and Chemical Sales, Inc., Oakland, CA, 1976.
10. Fill Heights for 3- by 1-in. Corrugated Steel Pipe in 'Good' Backfill. United States Steel Corp., Pittsburgh, 1967.

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Analysis of Long-Span Culverts by the Finite Element Method

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The long-span culvert is a synergistic unit composed of a corrugated metal liner and a compacted soil envelope that surrounds the liner. Conceptually, the system is very simple and, therefore, economically attractive as a bridge substitute. Analytically, however, the system is not simple because of the modeling difficulties associated with soil-structure interaction. Using the finite element method, this study investigates the influence of fundamental modeling assumptions on the behavior of long-span culverts. Two basic modeling assumptions are examined: large deformation theory versus small deformation theory and monolith structure versus incremented structure. In addition, the sensitivity of the following parameters are determined: compaction loads, soil stiffness, liner gage, liner shape, and special features of manufacturers. Results are shown graphically by comparing crown displacement histories between parametric families. Comparisons of maximum moment and thrust are also reported. Based on these studies, recommendations for analytical modeling techniques are summarized. The intent of this study is to provide a foundation for other studies. A systematic investigation of modeling assumptions and parameter sensitivity is a necessary step toward an analytical model for long-span culverts.

The long span is an arch or closed-shaped corrugated metal liner surrounded by compacted soil, where the horizontal span measures from 5 to 15 m (15 to 50 ft) or more. A primary use is to serve as a bridge substitute. To date, more than 600 long-span systems have been installed, and manufacturers estimate a cost savings from 30 to 75 percent over comparable conventional bridge structures. In view of the current bridge repair and replacement problem in the United States