

7. MASONRY DROP SPILLWAYS

General. A comparative cost analysis will usually indicate that masonry construction should be confined to relatively low drop spillways. A reinforced concrete apron should be used in cases where masonry is selected as the construction material for the walls. A reinforced concrete apron is more durable than masonry and can be designed to resist uplift forces, whereas a masonry apron must resist uplift by its weight alone.

Design Procedures and Aids. The over-all dimensions of a masonry drop spillway and the loadings for the integral parts of the structure are determined in the same manner as for a reinforced concrete drop spillway. The required base widths for the gravity walls for various loads and loading conditions are shown on drawing ES-64, page 7.17.

The curves on this drawing are sufficiently accurate for the majority of conditions of loads and loadings. The top width of the wall has a negligible effect on the required base width. The weight of earth is not a factor in Case 2, drawing ES-64, page 7.17, and its effect is negligible in Case 1. In Case 3, for earth weights greater than 100 pounds per cubic foot, the curves are on the safe side. For weights of earth less than 100 pounds per cubic foot, the base width for Case 3 should be computed by the equation shown on drawing ES-64. The curves are not valid for a masonry weight other than 150 pounds per cubic foot.

To facilitate construction and to maintain smooth surfaces, the batter on the walls may be handled as follows:

The headwall extensions will have a vertical face on the downstream side and a minimum batter of 1 in 10 on the upstream side. The headwall extension will be designed for a differential pressure of 5 pounds per cubic foot acting in the downstream direction.

The headwall will have the same batter on the upstream side as the headwall extensions. The thickness of the headwall at crest elevation will be a function of the depth of the weir and the required batter on the headwall extensions.

The sidewalls and wingwalls will have vertical faces on the unexposed sides.

Reinforced Concrete Apron and Sill Design. The following design procedure should not be used where $F + h$ exceeds 12 feet.

The apron may be designed as a series of beams in both directions. In the transverse direction, the beams should be designed for two conditions: (1) simply supported at the sidewalls and continuous over the longitudinal sills, and (2) fixed at the sidewalls and continuous over the longitudinal sills. In the longitudinal direction, the beams should be considered as (1) simply supported at both supports, and (2) fixed at both supports.

The upward pressure on the base area may be taken as the weight of the structure, plus the weights of the water and earth on the structure, with the assumption that it is uniformly distributed over the base area. The headwall extensions beyond the unexposed faces of the sidewalls and the wingwalls should not be included in the structure weight or base area. The maximum net load produced with or without flow over the structure should be taken as the design load.

Due to the relatively short span in the longitudinal direction, which makes it uneconomical to attempt to reduce reinforcing steel areas as the moments decrease, it is not necessary to construct moment and shear diagrams. The steel in the top face of a 12-inch slice should be designed for a moment of one-eighth wl^2 , and should extend 1'-0" past the toe of the headwall batter and be bent into the upstream face of the toewall. The steel in the bottom face of a 12-inch slice should be designed for a moment of one-twelfth wl^2 and a shear of one-half wl . The bottom steel should be bent into the downstream faces of the toewall and cutoff wall. For convenience, the bottom steel should consist of two sets of bars spliced at about the midpoint between the toe of the headwall batter and the transverse sill.

It is usually advantageous to construct the moment and shear curves in the transverse direction. A table of moment and shear coefficients are given to facilitate the construction of these diagrams when the longitudinal sills divide the apron into three equal spans. (See drawing ES-56, page 4.27, if spans are not equal.)

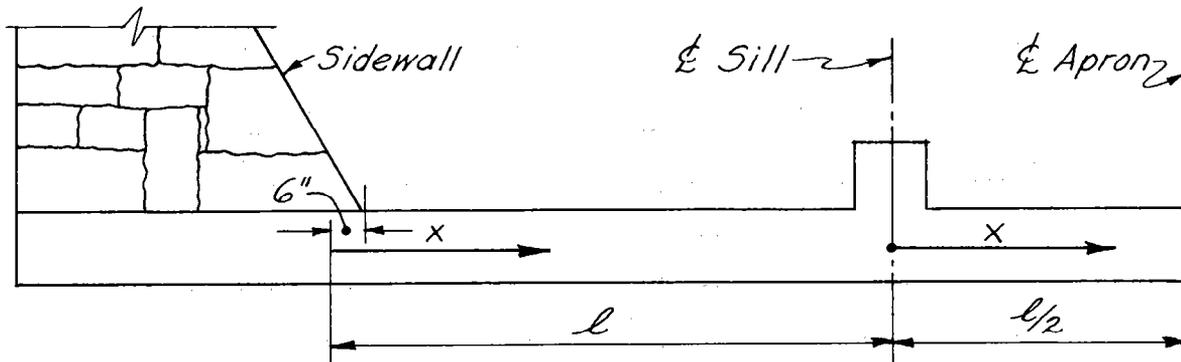


FIGURE 7.1

Moment and Shear Coefficients in Transverse Direction				
$M = C w l^2, \quad V = C_1 w l$				
in which				
$w = \text{load in lb/ft}^2 = \text{lb/ft on 12 in slice}$				
$l = \text{span length in ft}$				
$\frac{x}{l}$	Simply Supported at Sidewall		Fixed at Sidewall	
	C	C_1	C	C_1
0	0	-0.4	-0.0833	-0.5
0.1	+0.035	-0.3	-0.03833	-0.4
0.2	+0.060	-0.2	-0.00333	-0.3
0.3	+0.075	-0.1	+0.02167	-0.2
0.4	+0.080	0	+0.03667	-0.1
0.5	+0.075	+0.1	+0.04167	0
0.6	+0.060	+0.2	+0.03667	+0.1
0.7	+0.035	+0.3	+0.02167	+0.2
0.8	0	+0.4	-0.00333	+0.3
0.9	-0.045	+0.5	-0.03833	+0.4
Centerline Sill	-0.100	+0.6 -0.5	-0.0833	+0.5 -0.5
0.1	-0.055	-0.4	-0.03833	-0.4
0.2	-0.020	-0.3	-0.00333	-0.3
0.3	+0.005	-0.2	+0.02167	-0.2
0.4	+0.020	-0.1	+0.03667	-0.1
Centerline Apron	+0.025	0	+0.04167	0

- C denotes tension in the bottom face

- C_1 denotes shear in the downward direction.

TABLE 7.1

The longitudinal sills may be designed as beams for two conditions; namely, simply supported at both ends and fixed at both ends. The design load should be taken as the maximum reaction at the sill found in the transverse apron design minus the weight of the sill.

The transverse sill together with the toewall may be designed as a beam for two conditions; namely, simply supported at both ends and fixed at both ends. Its loads are the concentrated loads from the longitudinal sills, and the uniform load equal to the base pressure minus the weight of the transverse sill and toewall.

Example 7.1

Design of Masonry Drop Spillway. Assume the design discharge is 225 cfs, $F = 6'-0''$, and that the site conditions limit the weir length to $16'-0''$.

Hydraulic Design

Weir Dimensions: $Q = 225$ cfs, $L = 16'-0''$

$$\text{Equation 3.6 (page 3.7)} \quad h = \left[\frac{Q(1.10 + 0.01 F)}{CL} \right]^{2/3}$$

$$h = \left[\frac{225(1.10 + 0.06)}{3.1 \cdot 16} \right]^{2/3} = (5.26)^{2/3}$$

$$h = 3.02 \text{ ft} \quad \text{Use } \underline{3'-0''}$$

Height of Transverse Sill:

$$s = \frac{1}{2} d_c$$

$$d_c = \sqrt[3]{\frac{Q^2}{L^2 g}} = \sqrt[3]{\frac{(225)^2}{(16)^2 \cdot 32.2}} = 1.83 \text{ ft}$$

$$s = \frac{1.83}{2} = 0.915 \text{ ft} \quad \text{Use } \underline{1'-0''}$$

$$\underline{\text{Height of Headwall}} = F + s = 6.0 + 1.0 = \underline{7.0 \text{ ft}}$$

$$\underline{\text{Height of Sidewall}} = 2.5 d_c + s = 2.5 \cdot 1.83 + 1.0 = 5.58 \text{ ft} \quad \text{Use } \underline{6'-0''}$$

Minimum Length of Apron: $h \div F = 3 \div 6 = 0.50$; From ES-67 (page 5.3)

$$L_B \div F = 2.28 (h \div F) + 0.52 = 1.66$$

$$L_B = 1.66 \cdot 6 = 9.96$$

Use $L_B = 9'-9''$ to maintain 2 to 1 fill slopes

Minimum Length of Headwall Extension = $3h + 2.0'$

$$3h + 2.0 = 3 \cdot 3 + 2.0 = 11.0 \text{ ft}$$

Note: To maintain 2 to 1 fill slopes, the length was increased to 12'-9''.

Tailwater Depth and Depth of Water on Apron

The tailwater depth from the downstream channel design is assumed to be 2.4 ft; therefore, the depth of water on the apron is equal to $2.4 + 1.0 = \underline{3.4 \text{ ft} = d_x}$.

Stability Design

The foundation and backfill material will be a sand-silt-clay mixture with the following characteristics:

Undisturbed dry wt = 100 lbs/ft³ with 40 percent voids

Effective submerged wt = $100 - (1.0 - 0.4) 62.4 = 62.5$ lbs/ft³

Compacted dry wt = 120 lbs/ft³ with 28 percent voids = W_1

Effective submerged wt = $120 - (1.0 - 0.28) 62.4 = 75.1$ lbs/ft³ = W_2

Angle of internal friction = $\phi = 30^\circ$.

The channel above the structure will be a graded channel to crest elevation. It is anticipated that the water table will be above the apron elevation after construction; therefore, an ample drain will be provided through the headwall with its outlet at the elevation of the top of the transverse sill, or 1.0 ft above the top of the apron. It is estimated that the drain will maintain the water-table elevation at the headwall at its outlet elevation during periods of no flow, and at about tailwater elevation during periods of flow. These assumptions make the period of no flow the critical period for seepage under the structure.

Determine Depth of Cutoff Wall and Toewall

Wt creep ratio = 5.5

Head causing seepage = 1.75 ft to bottom of apron

Required wt creep distance = $5.5 \cdot 1.75 = 9.63$ ft

Neglecting the horizontal creep distance

$4t = 9.63$ ft or $t = 2.41$ ft; therefore, use minimum depth of 2'-6".

Determine Base Width of Headwall

With flow (produces max.)

$y_0 = 7.0$, $y_1 = 3.6$, $y_2 = 3.4$, $W_1 = 120$, $W_2 = 75.1$, $d_x = 3.4$

The equation below gives the equivalent fluid pressure for a triangular load diagram.

$$w = \frac{6}{y_0^3} \left\{ \frac{1 - \sin \phi}{1 + \sin \phi} \left[31.2 y_0^2 (h - 0.5) + \frac{W_1 y_1}{2} (y_1 y_2 + \frac{y_1^2}{3} + y_2^2) + \frac{W_2 y_2^3}{6} \right] + 10.4 (y_2^3 - d_x^3) \right\}$$

(This computation can also be done as shown in Example 4.1, page 4.6).

7.6

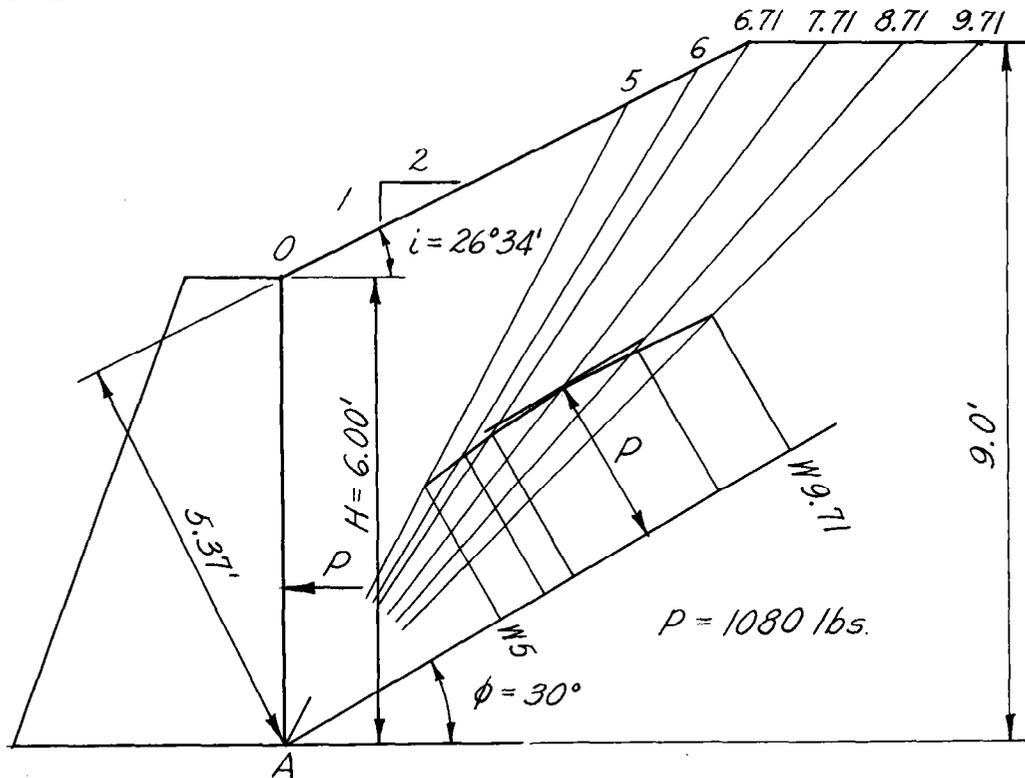
$$w = \frac{6}{343} \left\{ \frac{1}{3} \left[3820 + 216 (12.24 + 4.32 + 11.56) + 492 \right] \right\}$$

$$w = \frac{2}{343} (3820 + 6100 + 492) = \frac{2}{343} (10,412) = 60.8 \text{ lbs/ft}^3$$

From Case 1, drawing ES-64 (page 7.17), $C = 4.2 \text{ ft}$ Use 4'-3"

Determine Base Width of Sidewall

The construction below is a graphic solution for $w = 60.0 \text{ lbs/ft}^3$ and $H = 6.0$



(See subsection 2.2.2 of Structural Design, Section 6)

$$\text{Ave. ht. of fill} = \sqrt[3]{\frac{(6)^3 + (11)^3}{2}} = 9.2 \text{ ft} \quad \text{Use } 9'-0''$$

Slice	Weight of Slice	Accumulated Weight
0-A-5	$5.37 \cdot 2.5 \cdot 120 = 1612$	1612
5-A-6	$5.37 \cdot 0.5 \cdot 120 = 322$	1934
6-A-6.71	$5.37 \cdot 0.355 \cdot 120 = 229$	2163
6.71-A-7.71	$9.00 \cdot 0.50 \cdot 120 = 540$	2703
7.71-A-8.71	$9.00 \cdot 0.50 \cdot 120 = 540$	3243
8.71-A-9.71	$9.00 \cdot 0.50 \cdot 120 = 540$	3783

$$P = 1080 \text{ lbs} = \frac{wH^2}{2} = \frac{w(6)^2}{2} \therefore w = \frac{2 \cdot 1080}{36} = 60.0 \text{ lbs/ft}^3$$

From Case 2, drawing ES-64 (page 7.17), $C = 3.5$ Use 3'-6"

Determine Base Width of Headwall Extension

$$H = 10.0 \text{ ft}, \quad w = 5 \text{ lbs/ft}^3$$

$$\text{Minimum width} = b + (H \div 10) = 1.25 + 1.0 = 2.25 \text{ ft}$$

This is greater than shown on Case 3, drawing ES-64 (page 7.17), therefore, use $C = 2'-3"$.

Determine Base Widths for Wingwall

$H = 6.0$ ft and 2.0 ft. Use $w = 60 \text{ lbs/ft}^3$, the same as for the sidewall.

When $H = 6.0$, $C = 3'-6"$, same as sidewall.

When $H = 2.0$, $C = 1.2$ ft, from Case 2, drawing ES-64.

To maintain the same batter along the wall, use $2'-0"$.

Determine Weight of Structure excluding the wingwalls and the headwall extension beyond the unexposed faces of the sidewalls.

$$\text{Weight of masonry} = 150 \text{ lbs/ft}^3$$

$$\text{Weight of concrete} = 150 \text{ lbs/ft}^3$$

$$\text{Weight of earth} = 120 \text{ lbs/ft}^3$$

	<u>Volume</u>
Hdwl. Ext. $\frac{2.25 + 1.25}{2} \cdot 10 \cdot 2.75 \cdot 2$	= 96.30
Hdwl. (approx.) $\frac{4.25 + 1.55}{2} \cdot 7 \cdot 16.0$	= 325.00
Hdwl. (approx.) $(0.3 \cdot 1.5) + (0.75 \cdot 1.133) \frac{3}{4} \cdot 7 \cdot 2$	= 12.60
Sidewall $\frac{1.25 + 3.5}{2} \cdot 6 \cdot 8.5 \cdot 2$	= 242.00
Sidewall $2 \cdot 1.25 \cdot 6 \cdot 2$	= 30.00
Sidewall $2 \cdot \frac{1.7 \cdot 2.25}{2} \cdot \frac{2}{3} \cdot 6$	= 15.30
Long. sill $\frac{7.5 + 7.9}{2} \cdot 0.75 \cdot 1.0 \cdot 2.0$	= 11.55
Trans. sill $\frac{1.75}{2} \cdot 1.0 \cdot 15.0$	= 13.13
Apron $21.5 \cdot 12.75 \cdot 0.75$	= 205.60
Cutoff wall $21.5 \cdot 2.5 \cdot 0.75$	= 40.30
Cutoff fillet $21.5 \cdot 1/2 \cdot 1/4$	= 2.69

	Volume
Toewall $16.5 \cdot 2.5 \cdot 0.75$	= 30.93
Toewall fillet $16.5 \cdot 1/2 \cdot 1/4$	= 2.06
(Concrete and Masonry)	1027.46 cu ft
Earth on batter = $0.7 \cdot 3.5 \cdot 21.5$	= 52.6 cu ft
Weight = $1027.46 \cdot 150 + 52.6 \cdot 120 = 160,500$ lbs	
Corresponding base area = $21.5 \cdot 12.75 = 274.0$ ft ²	
Ave. pressure on base = $\frac{160,500}{274} = 585$ lbs/ft ²	

UpliftWithout flow

Check neglecting cutoffs

$$\text{Uplift per ft width of structure} = \frac{1.75 \cdot 62.4}{2} \cdot 12.75 = 696 \text{ lbs/ft}$$

$$\text{Total uplift} = 696 \cdot 21.5 = 14,980 \text{ lbs}$$

$$\text{Ratio of } \frac{\text{Wt}}{\text{Uplift}} = \frac{160,500}{14,980} = 10.70 \quad \text{OK}$$

With flow

Check neglecting cutoffs

$$\text{Uplift per ft width of structure} = 4.15 \cdot 62.4 \cdot 12.75 = 3300 \text{ lbs/ft}$$

$$\text{Total uplift} = 3300 \cdot 21.5 = 71,000 \text{ lbs}$$

Wt. of water on structure

$$\text{Open apron area} = 7.5 \cdot 14.5 = 108.8 \text{ ft}^2$$

$$\text{Area at H}_2\text{O surface} = 17.0 \cdot 9.5 = 161.6 \text{ ft}^2$$

$$\text{Gross H}_2\text{O volume on apron (approx.)} = \frac{108.8 + 161.6}{2} \cdot 3.4 = 460 \text{ ft}^3$$

$$\text{Volume of sills} = 11.55 \text{ ft}^3$$

$$\text{Net H}_2\text{O on apron} = 460 - 11 = 449 \text{ ft}^3$$

$$\text{H}_2\text{O on headwall} = 1.83 \cdot 16 \cdot 1.55 = 45.4 \text{ ft}^3$$

$$\begin{aligned} \text{H}_2\text{O on headwall batter} &= (0.70 \cdot 1.83 \cdot 16) + \\ &\quad \left(\frac{0.70 + 0.88}{2} \cdot 1.83 \cdot 5.5 \right) = 28.5 \text{ ft}^3 \end{aligned}$$

$$\text{Total volume of H}_2\text{O on structure} = 523 \text{ ft}^3$$

$$\text{Wt. of H}_2\text{O} = 523 \cdot 62.4 = 32,700 \text{ lbs}$$

$$\text{Total weight} = 32,700 + 160,500 = 193,200 \text{ lbs}$$

$$\text{Ratio of } \frac{\text{Wt}}{\text{Uplift}} = \frac{193,200}{71,000} = 2.72 \quad \text{OK}$$

Check Sliding

Neglect sliding resistance of cutoff wall and toewall.

Neglect the weight and sliding resistance of the wingwalls and the head-wall extensions beyond the unexposed faces of the sidewalls.

$$\phi = 30^\circ$$

$$\text{Wt. of dry earth} = 120 \text{ lbs/ft}^3$$

$$\text{Effective weight of submerged earth} = 75.1 \text{ lbs/ft}^3$$

$$\text{Depth of submerged earth} = 3.40 + 0.75 = 4.15 \text{ ft}$$

$$\text{Depth of dry earth} = 7.75 - 4.15 = 3.6 \text{ ft}$$

Downstream force

$$p_a = wH \left(\frac{1 - \sin \phi}{1 + \sin \phi} \right) = wH \left(\frac{1 - 0.5}{1 + 0.5} \right) = \frac{wH}{3}$$

$$p_1 = 120 \cdot 3.6 \cdot \frac{1}{3} = + 144 \text{ lbs (dry earth)}$$

$$p_2 = 62.4 \cdot 4.15 = + 259 \text{ lbs (H}_2\text{O)}$$

$$p_3 = 75.1 \cdot 4.15 \cdot \frac{1}{3} = + 104 \text{ lbs (submerged earth)}$$

$$p_4 = 62.4 \cdot 2.5 \cdot \frac{1}{3} = + 52 \text{ lbs (surcharge due to headwater)}$$

$$p_5 = 62.4 \cdot 4.15 = - 259 \text{ lbs (H}_2\text{O)}$$

$$p_6 = 62.4 \cdot 2.5 = + 156 \text{ lbs (headwater on headwall extension)}$$

$p_2 = p_5$, therefore they cancel

$$l = \text{width of structure considered} = 21.5 \text{ ft}$$

$$\begin{aligned} \text{Downstream force} = & \left[\frac{3.6 p_1}{2} + 4.15 p_1 + \frac{4.15 p_3}{2} + 7.75 p_4 \right] \\ & + (l - 16) \left(\frac{2.5 p_6}{2} \right) \end{aligned}$$

$$\text{D.F.} = 21.5 \left[5.95 (144) + 4.15 (52) + 7.75 (52) \right] + 5.5 (2.5)(78)$$

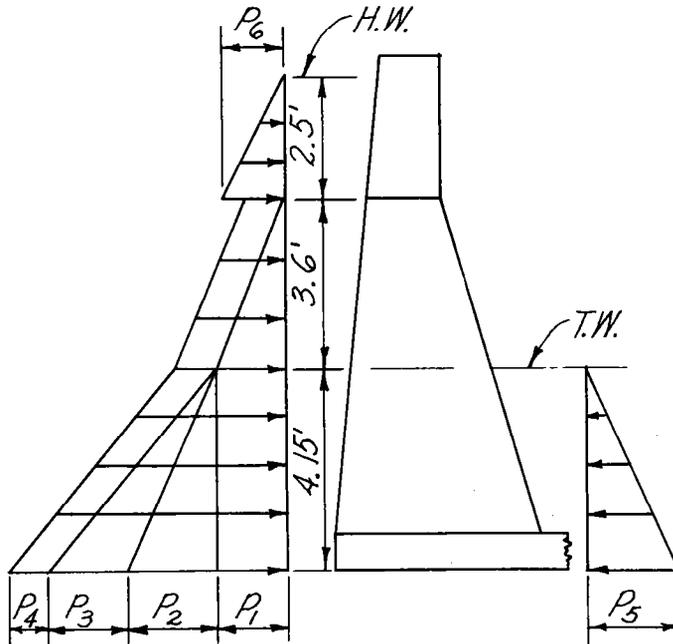
$$\text{D.F.} = 21.5 (1476) + 1073 = 31,725 + 1073 = 32,800 \text{ lbs}$$

$$\text{Net wt.} = (\text{total wt.} - \text{uplift}) = 193,200 - 71,000 = 122,200 \text{ lbs}$$

$$\text{Frictional resistance} = f (\text{net wt.}) = 0.35 \cdot 122,200 = 42,750 \text{ lbs}$$

$$\text{Ratio} = \frac{42,750}{32,800} = 1.30$$

OK



Apron Design

Determine design load

Without flow

$$\text{Base area} = 21.5 \cdot 12.75 = 274.0 \text{ ft}^2$$

$$\text{Gross weight} = 160,500 \text{ lbs (see page 7.8)}$$

$$\text{Net load} = \frac{160,500}{274} - (0.75 \cdot 150) = 585 - 113 = 472 \text{ lbs/ft}^2$$

With flow

$$\text{Gross weight} = 193,200 \text{ lbs (see page 7.9)}$$

$$\text{Net load} = \frac{193,200}{274} - 113 - (3.4 \cdot 62.4) = 705 - 325 = 380 \text{ lbs/ft}^2$$

Use 472 lbs/ft²

Longitudinal steel design (Use Class B concrete)

$$l = 8.0 \text{ ft}$$

Top steel $d = 8 - 2.5 = 5.5$ in

$$M = \frac{1}{8} w l^2 = \frac{1}{8} \cdot 472 \cdot (8.0)^2 = 3776 \text{ ft lbs}$$

$$A_s = 0.46 \text{ in}^2 \quad \underline{\text{Use No. 5 at 8 } (A_s = 0.46)}$$

Length = 10'-3" with vertical leg = 1'-3" bent into upstream face of toewall.

Bottom steel $d = 9 - 3.5 = 5.5$ in

$$M = \frac{1}{12} w l^2 = \frac{1}{12} \cdot 472 \cdot (8.0)^2 = 2520 \text{ ft lbs}, A_s = 0.30 \text{ in}^2$$

$$V = \frac{1}{2} w l = \frac{1}{2} \cdot 472 \cdot 8.0 = 1890 \text{ lbs}, \quad \Sigma_o = 1.30 \text{ in}$$

$$\underline{\text{Use No. 5 at 12 } (A_s = 0.31, \Sigma_o = 1.96)}$$

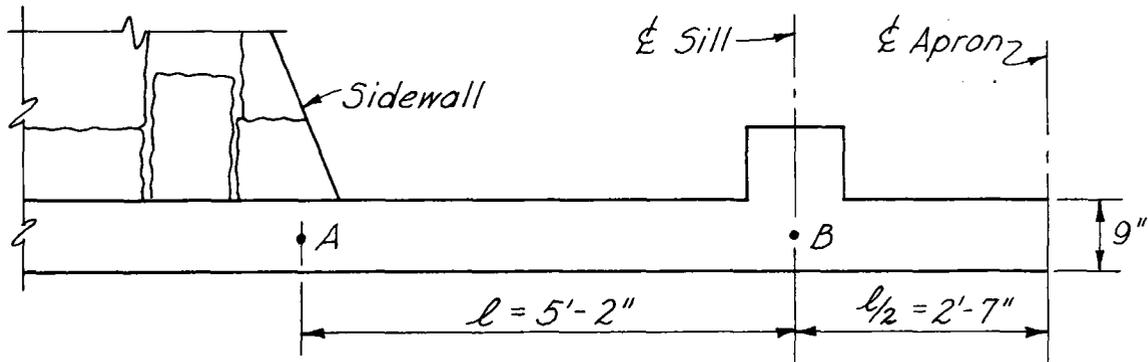
Splice two bars

Length = 12'-0" with vertical leg = 2'-6" bent into downstream face of cutoff wall.

Length = 5'-3" with vertical leg = 1'-3" bent into downstream face of toewall.

Transverse steel design

$$l = 5'-2" = 5.17 \text{ ft}$$

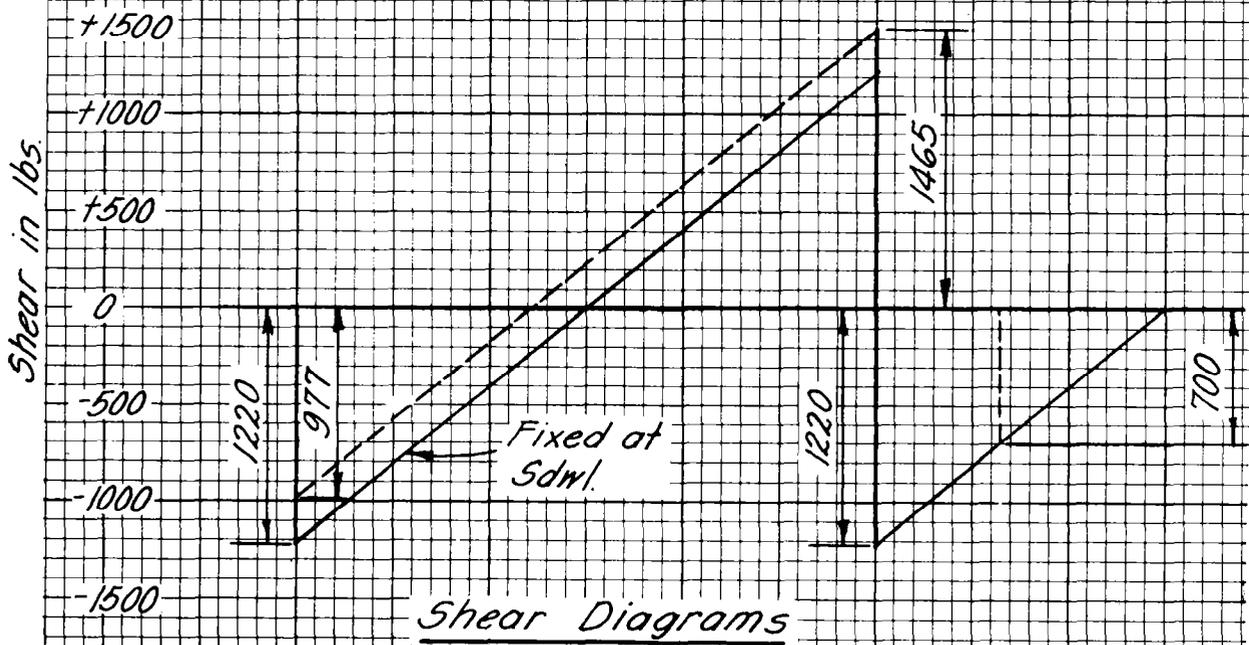
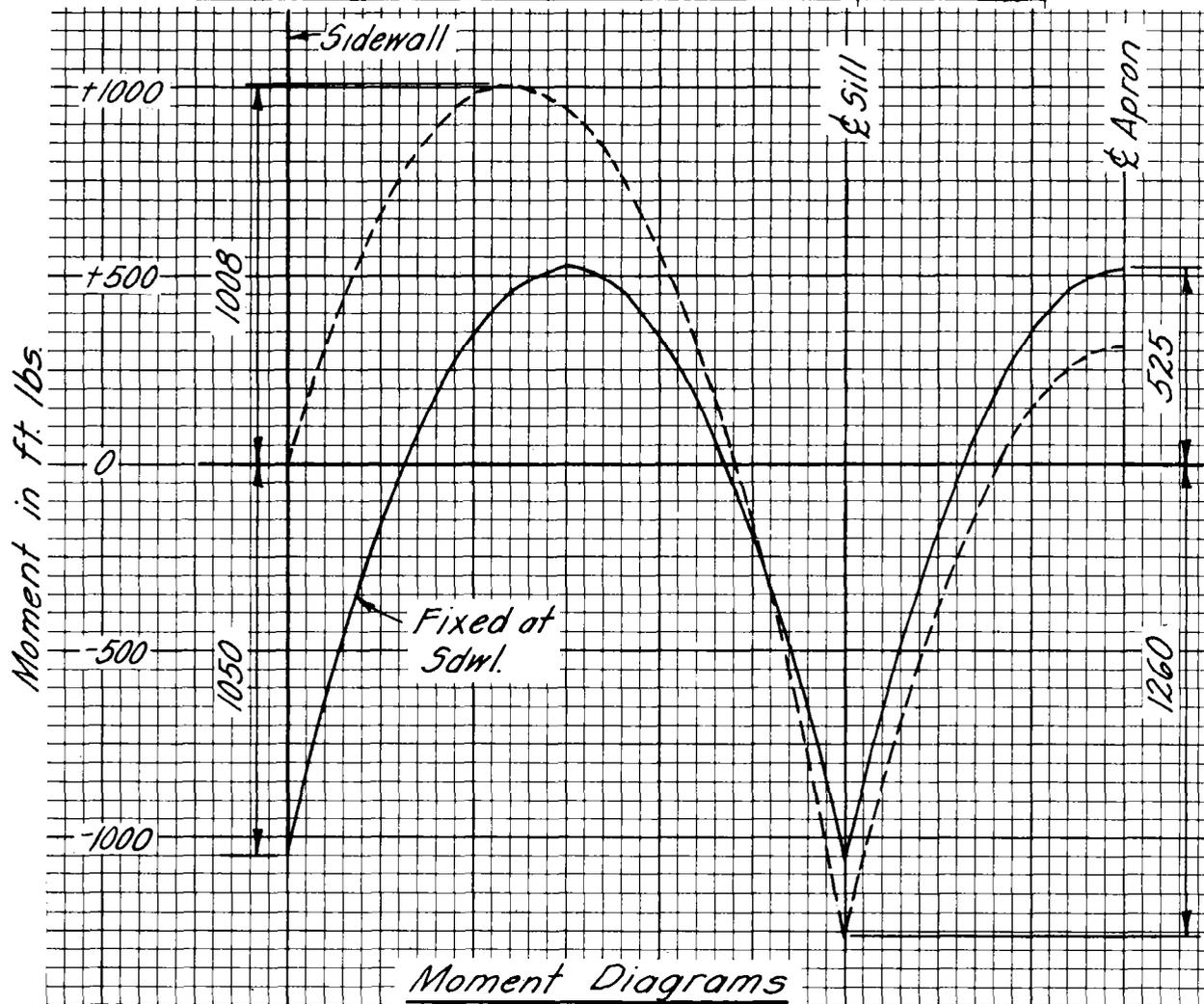


Moments

$$M = Cwl^2, \quad wl^2 = 472 (5.17)^2 = 12,600$$

$\frac{x}{l}$	Simply supported at sidewall	Fixed at sidewall
At sdwl.	0	- 1050
0.1	+ 441	- 483
0.2	+ 756	- 42
0.3	+ 945	+ 273
0.4	+ 1008	+ 462
0.5	+ 945	+ 525
0.6	+ 756	+ 462
0.7	+ 441	+ 273
0.8	0	- 42
0.9	- 567	- 483
At sill	- 1260	- 1050
0.1	- 693	- 483
0.2	- 252	- 42
0.3	+ 63	+ 273
0.4	+ 252	+ 462
Centerline apron	+ 315	+ 525
	$V_{AB} = - 0.4 wl$ $V_{AB} = - 977 \text{ lbs}$ $V_{BA} = + 0.6 wl$ $V_{BA} = + 1465 \text{ lbs}$ $R_B = 1.1 wl$ $R_B = 2685 \text{ lbs}$	$V_{AB} = V_{BA}$ $= 0.5 wl$ $= 1220 \text{ lbs}$

APRON DESIGN (TRANSVERSE DIRECTION)



Bottom steel $d = 9 - 3.5 - 0.25 - 0.25 = 5.0$ (assuming No. 4 bars)

$$- M = 1050 \text{ ft lbs at sdwl}; \quad A_s = 0.135 \text{ in}^2$$

$$- M = 1260 \text{ ft lbs at sill}; \quad A_s = 0.163 \text{ in}^2$$

$$V = 1220 \text{ lbs at sdwl}; \quad \Sigma_o < 1.00 \text{ in}$$

$$V = 1465 \text{ lbs at sill}; \quad \Sigma_o = 1.12 \text{ in}$$

Use No. 4 at 15 ($A_s = 0.16$, $\Sigma_o = 1.26$) continuous across apron.

Top steel $d = 8 - 2 \frac{1}{2} - \frac{3}{8} - \frac{1}{4} = 4 \frac{7}{8}$ in (assuming No. 4 bars)

$$+ M = 1008 \text{ ft lbs in end spans}; \quad A_s = 0.133 \text{ in}^2$$

$$+ M = 525 \text{ ft lbs in center span}; \quad A_s < 0.10 \text{ in}^2$$

$$V = 977 \text{ lbs in end spans}; \quad \Sigma_o < 1.00 \text{ in}$$

$$V = 700 \text{ lbs in center span}; \quad \Sigma_o < 1.00 \text{ in}$$

Use No. 4 at 18 ($A_s = 0.13$, $\Sigma_o = 1.05$) continuous across apron.

Longitudinal Sill Design

$$w = \text{Max } R_B = 2685 \text{ lbs/ft}, \quad \ell = 8.0 \text{ ft}$$

$$d = 18 - 3.5 = 14.5 \text{ in}$$

Bottom steel

$$- M = \frac{1}{12} w \ell^2 = \frac{1}{12} \cdot 2685 (8)^2 = 14,320 \text{ ft lbs}; \quad A_s = 0.645 \text{ in}^2$$

$$V = \frac{1}{2} w \ell = 10,740 \text{ lbs}; \quad \Sigma_o = 2.84 \text{ in}$$

Use two No. 6 bars ($A_s = 0.88$, $\Sigma_o = 4.71$)

Length = 11'-3" with vertical leg = 1'-3" bent into downstream face of toewall.

Top steel

$$M = \frac{1}{8} w \ell^2 = \frac{1}{8} \cdot 2685 (8)^2 = 21,500 \text{ ft lbs}; \quad A_s = 0.98 \text{ in}^2$$

Use two No. 7 bars ($A_s = 1.20$)

Length = 10'-9" with vertical leg 1'-3" bent into upstream face of toewall.

Transverse Sill Design

Uniform load

$$\text{Load up} = 585 \cdot 0.75 = 439 \text{ lbs/ft}$$

$$\text{Load down} = 150 \cdot 0.75 \cdot 4.25 = 478 \text{ lbs/ft}$$

$$\text{Net load} = 478 - 439 = 39 \text{ lbs/ft}$$

(Due to the fact that this load is small and acts in the opposite direction of the concentrated loads, it will be neglected.)

Concentrated loads

Loads = 10,740 lbs (V from longitudinal sill design) acting at 1/3 points.

$$\text{Bottom steel } d = (4'-3") - 4" = 3'-11" = 47 \text{ in}$$

End fixed

$$-M = \left[Z(1-Z)^2 + Z^2(1-Z) \right] P\ell = Z(1-Z)P\ell \text{ (See ES-17, Engineering Handbook, Section 6 on Structural Design.)}$$

$$Z = 1/3, \quad P = 10,740, \quad \ell = 15.5$$

$$-M = 1/3(2/3)(10,740)(15.5) = 37,000 \text{ ft lbs}$$

$$A_s = \frac{M}{ad} = \frac{37.0}{1.44 \cdot 47} = 0.55 \text{ in}^2$$

$$V = P = 10,740 \text{ lbs}$$

$$\Sigma_o = \frac{8V}{7ud} = \frac{8 \cdot 10,740}{7 \cdot 300 \cdot 47} = 0.87$$

Use two No. 5 bars ($A_s = 0.61$, $\Sigma_o = 3.93$)

$$\text{Top steel } d = 47 \text{ in}$$

Ends simply supported

$$M = \frac{\ell}{3} P = \frac{15.5}{3} \cdot 10,740 = 55,500 \text{ ft lbs}$$

$$A_s = \frac{M}{ad} = \frac{55.5}{1.44 \cdot 47} = 0.82 \text{ in}^2$$

Use two No. 6 bars ($A_s = 0.88$)

Steel in Toewall

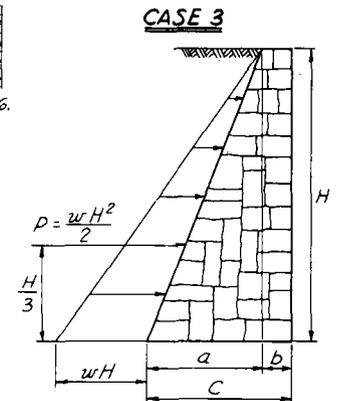
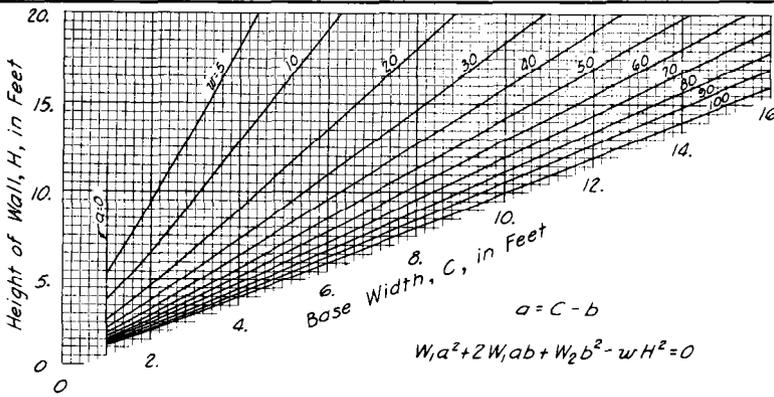
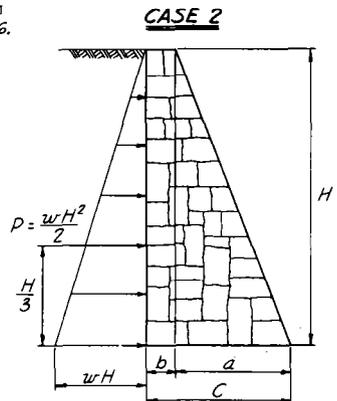
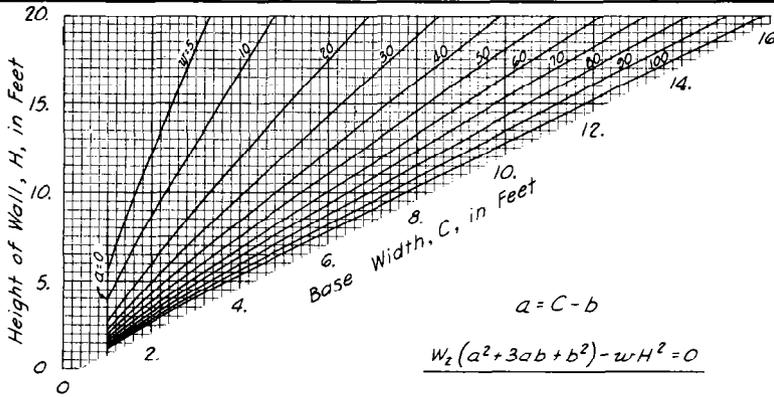
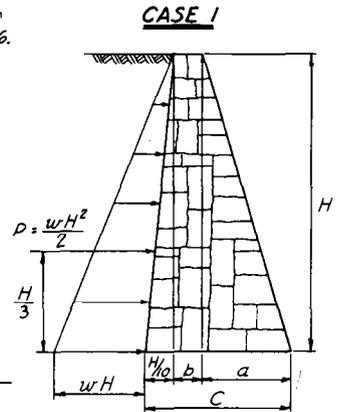
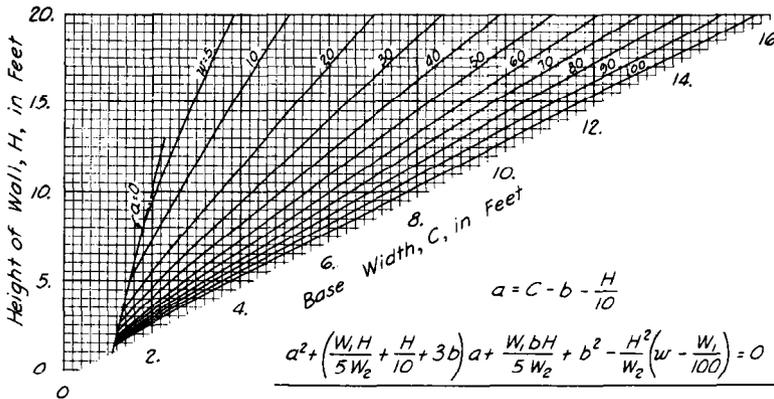
Vertical steel

Upstream face - Use No. 5 at 15; length = 3'-9"

Downstream face - Use No. 4 at 12; length = 3'-9"



DROP SPILLWAYS: REQUIRED BASE WIDTH FOR GRAVITY WALLS WITH VARIOUS LOADS AND LOADINGS



NOTES

The curves are plotted for $W_1 = 100 \text{ lbs/ft}^3$, $W_2 = 150 \text{ lbs/ft}^3$, $b = 1.0 \text{ ft}$ and are sufficiently accurate for reasonable variations of these values.

If more accurate results are desired for such variations, use the actual values of b , W_1 , and W_2 and the appropriate equations shown above.

SYMBOLS

a = required batter in ft.
 b = top width of wall in ft.
 C = required base width of wall in ft.
 H = height of wall in ft.
 W_1 = weight of earth in lbs/ft^3
 W_2 = weight of masonry in lbs/ft^3
 w = equivalent fluid pressure in lbs/ft^2
 P = horizontal earth pressure per foot length of wall.

REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
 SOIL CONSERVATION SERVICE
 Robert M. Salter, Chief
 ENGINEERING STANDARDS UNIT

STANDARD DWG. NO.

ES-64

SHEET 1 OF 1

DATE 2-2-52