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 is 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 $w\ell^2$, 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 $w\ell^2$ and a shear of one-half $w\ell$. 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.)

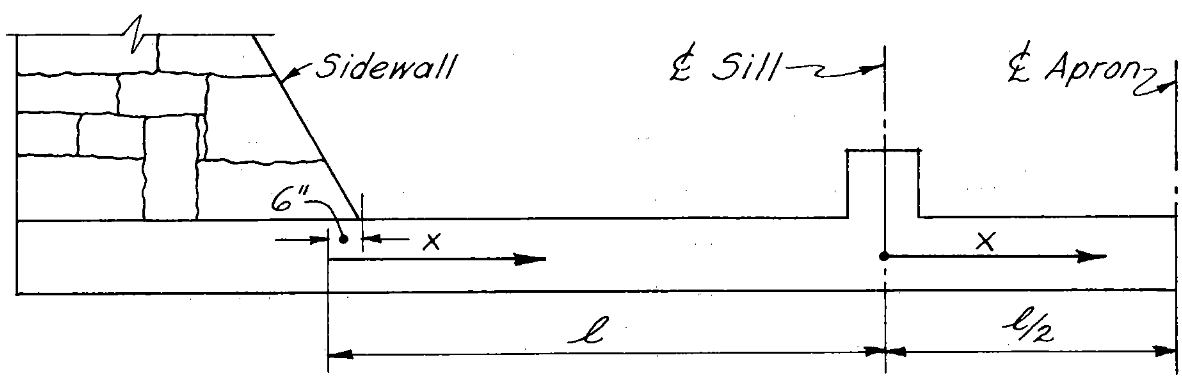


FIGURE 7.1

Moment and Shear Coefficients in Transverse Direction

$$M = C\,w\ell^2, \qquad V = C_1 w\ell$$

in which

w = load in lb/ft^2 = lb/ft on 12 in slice

ℓ = span length in ft

$\frac{x}{\ell}$	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"

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

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

h = 3.02 ft   Use 3'-0"

Height of Transverse Sill:

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

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

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

Height of Headwall = F + s = 6.0 + 1.0 = 7.0 ft

Height of Sidewall = 2.5 d_c + s = 2.5 · 1.83 + 1.0 = 5.58 ft   Use 6'-0"

Minimum Length of Apron: h ÷ F = 3 ÷ 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 · 3 + 2.0 = 11.0 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 = 3.4 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/ft^3 with 40 percent voids

Effective submerged wt = 100 − (1.0 − 0.4) 62.4 = 62.5 lbs/ft^3

Compacted dry wt = 120 lbs/ft^3 with 28 percent voids = W_1

Effective submerged wt = 120 − (1.0 − 0.28) 62.4 = 75.1 lbs/ft^3 = W_2

Angle of internal friction = ϕ = 30°.

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 · 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.)

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

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

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

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

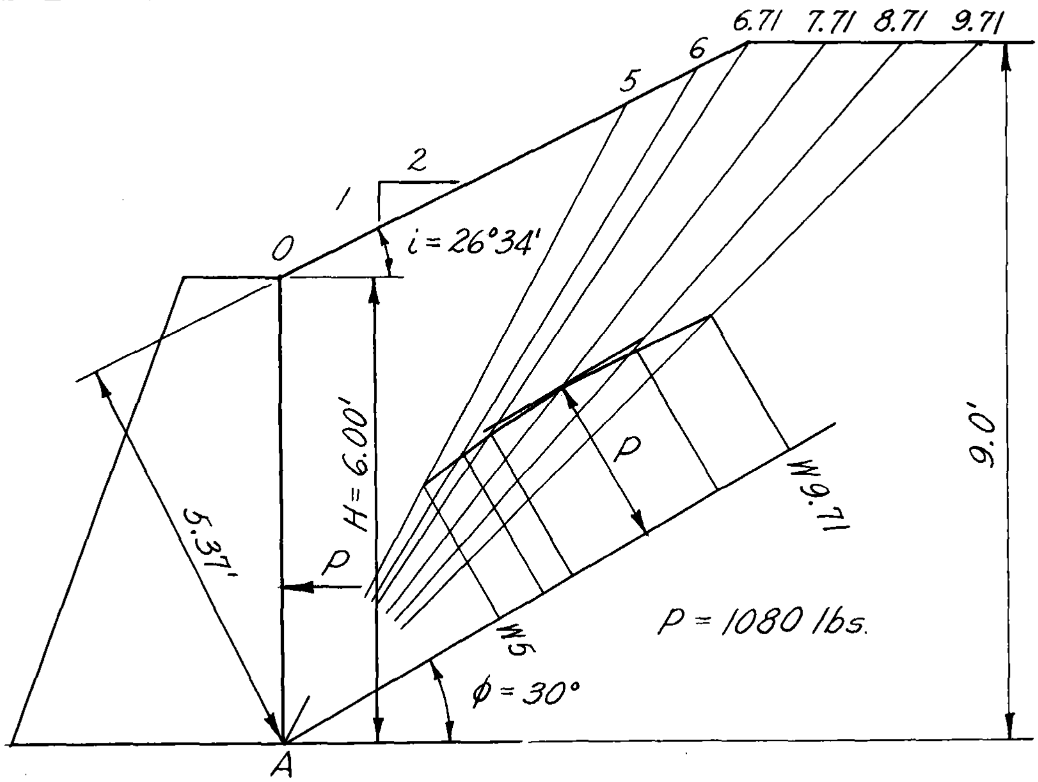
$$w = \frac{6}{343}\left\{\frac{1}{3}\left[3820 + 216\ (12.24 + 4.32 + 11.56) + 492\right]\right\}$$

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

From Case 1, drawing ES-64 (page 7.17), C = 4.2 ft <u>Use 4'-3"</u>

<u>Determine Base Width of Sidewall</u>

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



(See subsection 2.2.2 of Structural Design, Section 6)

Ave. ht. of fill = $\sqrt[3]{\dfrac{(6)^3 + (11)^3}{2}}$ = 9.2 ft <u>Use 9'-0"</u>

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

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

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

Determine Base Width of Headwall Extension

$H = 10.0$ ft, $w = 5$ lbs/ft^3

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

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

Determine Base Widths for Wingwall

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

When $H = 6.0$, $C = 3'\text{-}6"$, 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.

Weight of masonry = 150 lbs/ft^3

Weight of concrete = 150 lbs/ft^3

Weight of earth = 120 lbs/ft^3

		Volume
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 $= \dfrac{160{,}500}{274} = 585$ lbs/ft^2

Uplift

Without flow

Check neglecting cutoffs

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

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

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

With flow

Check neglecting cutoffs

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

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

Wt. of water on structure

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

Area at H_2O surface $= 17.0 \cdot 9.5 = 161.6$ ft^2

Gross H_2O volume on apron (approx.) $= \dfrac{108.8 + 161.6}{2} \cdot 3.4 = 460$ ft^3

Volume of sills $= 11.55$ ft^3

Net H_2O on apron $= 460 - 11 = 449$ ft^3

H_2O on headwall $= 1.83 \cdot 16 \cdot 1.55 = 45.4$ ft^3

H_2O on headwall batter $= (0.70 \cdot 1.83 \cdot 16) +$

$$\left(\frac{0.70 + 0.88}{2} \cdot 1.83 \cdot 5.5\right) = 28.5 \text{ ft}^3$$

Total volume of H_2O on structure $= 523$ ft^3

Wt. of $H_2O = 523 \cdot 62.4 = 32{,}700$ lbs

Total weight = 32,700 + 160,500 = 193,200 lbs

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

Check Sliding

Neglect sliding resistance of cutoff wall and toewall.

Neglect the weight and sliding resistance of the wingwalls and the headwall extensions beyond the unexposed faces of the sidewalls.

$\phi = 30^{\circ}$

Wt. of dry earth = 120 lbs/ft^3

Effective weight of submerged earth = 75.1 lbs/ft^3

Depth of submerged earth = 3.40 + 0.75 = 4.15 ft

Depth of dry earth = 7.75 − 4.15 = 3.6 ft

Downstream force

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

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

$p_2 = 62.4 \cdot 4.15 = + 259$ lbs (H_2O)

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

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

$p_5 = 62.4 \cdot 4.15 = - 259$ lbs (H_2O)

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

$p_2 = p_5$, therefore they cancel

ℓ = width of structure considered = 21.5 ft

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

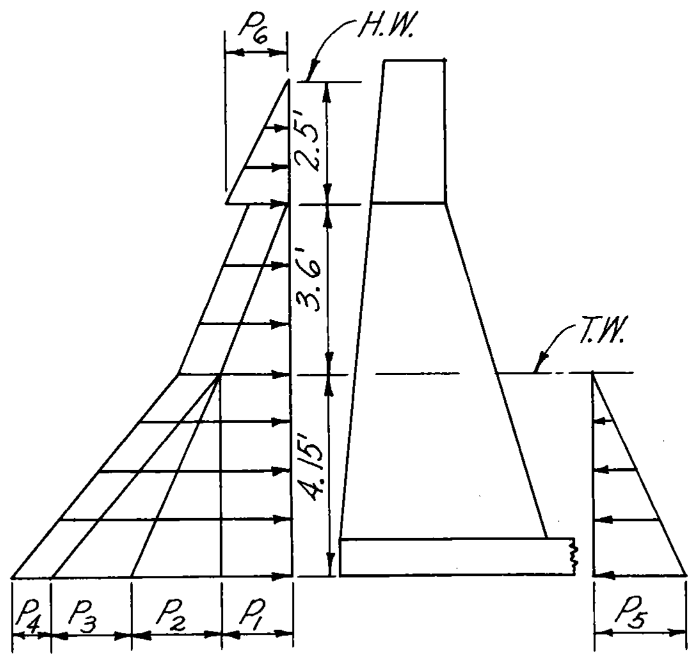
$$\text{D.F.} = 21.5\left[5.95\ (144) + 4.15\ (52) + 7.75\ (52)\right] + 5.5\ (2.5)(78)$$

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

Net wt. = (total wt. − uplift) = 193,200 − 71,000 = 122,200 lbs

Frictional resistance = f (net wt.) = 0.35 · 122,200 = 42,750 lbs

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



Apron Design

Determine design load

Without flow

Base area = 21.5 · 12.75 = 274.0 ft^2

Gross weight = 160,500 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

Gross weight = 193,200 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/ft^2

Longitudinal steel design (Use Class B concrete)

ℓ = 8.0 ft

<u>Top steel</u> $d = 8 - 2.5 = 5.5$ in

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

$A_s = 0.46$ in^2 <u>Use No. 5 at 8 ($A_s = 0.46$)</u>

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

<u>Bottom steel</u> $d = 9 - 3.5 = 5.5$ in

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

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

<u>Use No. 5 at 12 ($A_s = 0.31$, $\Sigma_o = 1.96$)</u>

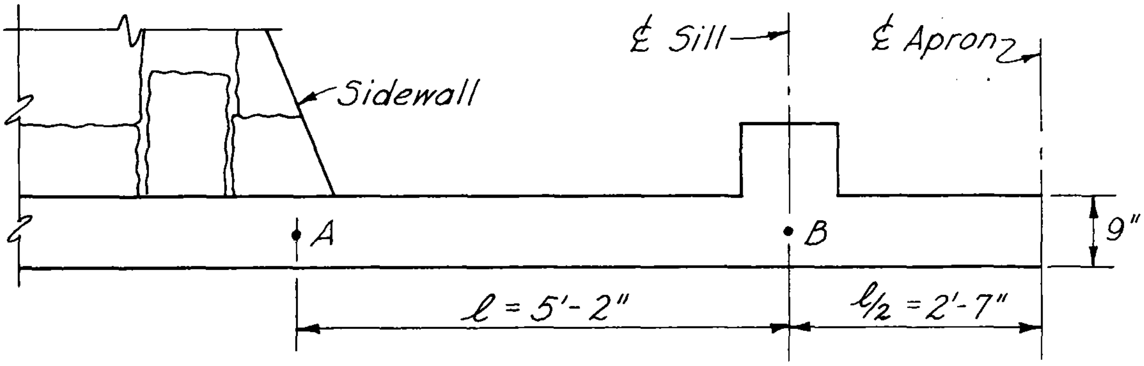
Splice two bars

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

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

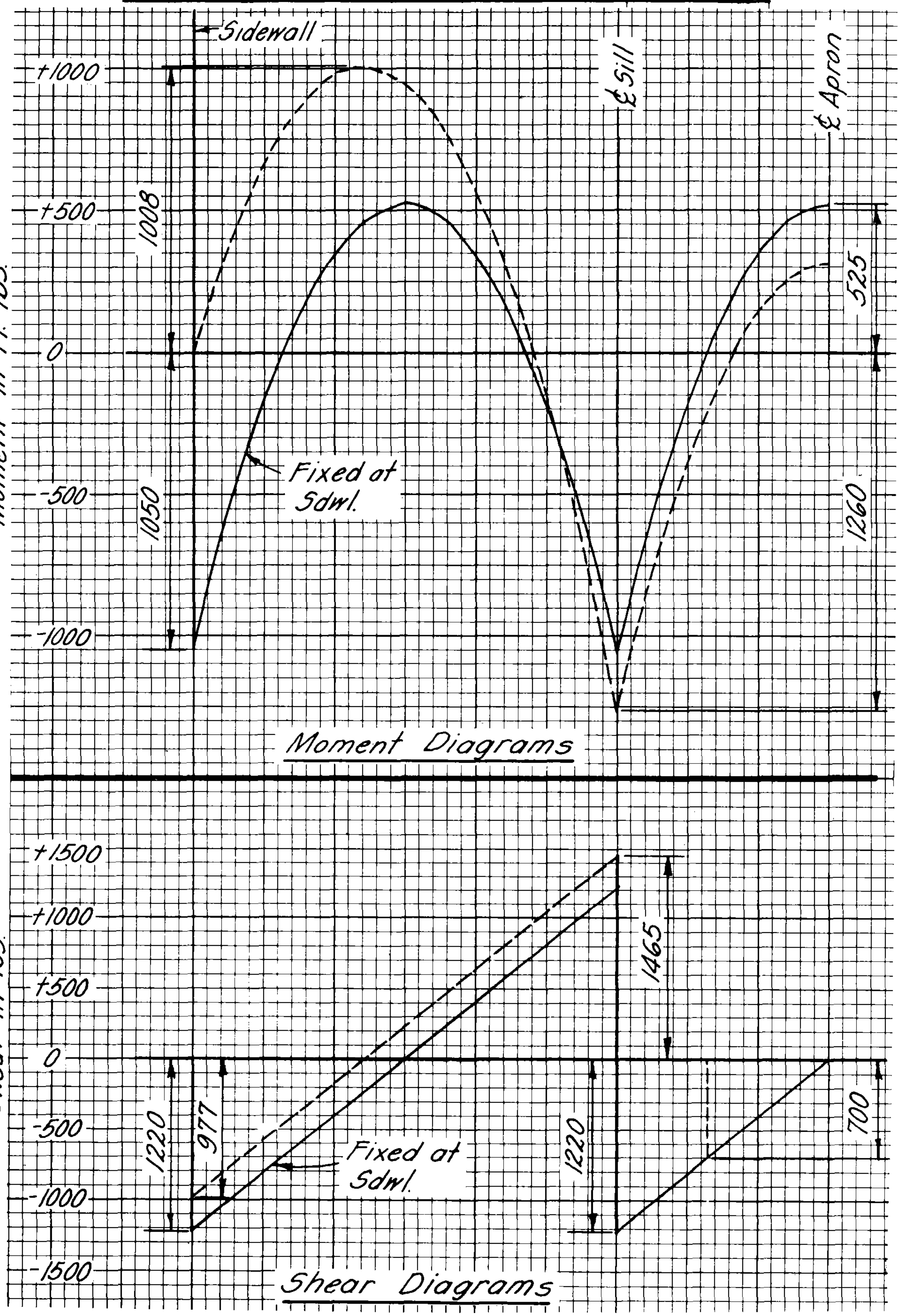


Moments

$M = Cw\ell^2, \quad w\ell^2 = 472\ (5.17)^2 = 12{,}600$

$\frac{x}{\ell}$	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
	$V_{AB} = -0.4\ w\ell$ $V_{AB} = -977$ lbs $V_{BA} = +0.6\ w\ell$ $V_{BA} = +1465$ lbs $R_B = 1.1\ w\ell$ $R_B = 2685$ lbs	$V_{AB} = V_{BA}$ $= 0.5\ w\ell$ $= 1220$ lbs

APRON DESIGN (TRANSVERSE DIRECTION)



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

$- M = 1050$ ft lbs at sdwl; $A_s = 0.135$ in^2

$- M = 1260$ ft lbs at sill; $A_s = 0.163$ in^2

$V = 1220$ lbs at sdwl; $\Sigma_o < 1.00$ in

$V = 1465$ lbs at sill; $\Sigma_o = 1.12$ in

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

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

$+ M = 1008$ ft lbs in end spans; $A_s = 0.133$ in^2

$+ M = 525$ ft lbs in center span; $A_s < 0.10$ in^2

$V = 977$ lbs in end spans; $\Sigma_o < 1.00$ in

$V = 700$ lbs in center span; $\Sigma_o < 1.00$ in

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

Longitudinal Sill Design

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

$d = 18 - 3.5 = 14.5$ in

Bottom steel

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

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

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

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

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

Net load = 478 − 439 = 39 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.

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

End fixed

$$-M = \left[Z\,(1 - Z)^2 + Z^2\,(1 - Z)\right] P\ell = Z\,(1 - Z)\,P\ell$$ (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 ($A_s = 0.61$, $\Sigma_o = 3.93$)

Top steel $d = 47$ in

Ends simply supported

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

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

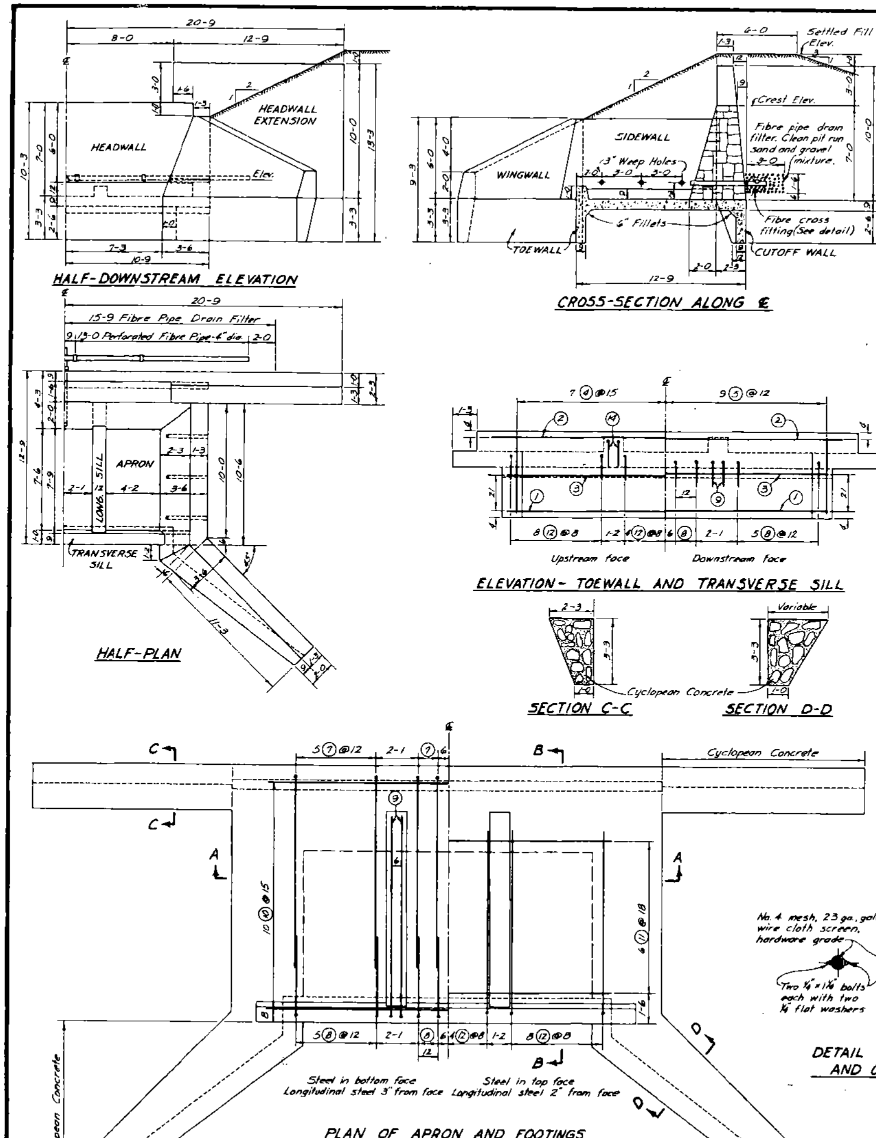
Use two No. 6 bars ($A_s = 0.88$)

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"



NOTES

The structure is symmetrical about its centerline.

Apron designed for Class B concrete – $f_c = 1200$ psi, $f_s = 20{,}000$ psi, $n = 10$

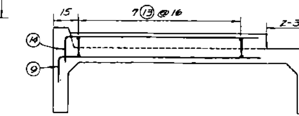
All reinforcing steel shall meet ASTM Designation A 305.

The base pour for the headwall extensions and wingwalls shall be cyclopean concrete and shall be poured at the same time as the apron to insure bond between the two types of concrete.

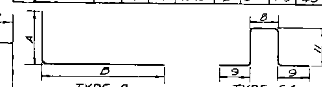
The first course of masonry shall be imbedded at least one inch in the base concrete pour.

STEEL SCHEDULE

LOCATION	MARK	SIZE	QUAN.	LENGTH	TYPE	A	B	TOTAL FT.
Toewall	1	5	2	16-0	St.	—	—	32-0
Trans. Sill	2	6	2	17-6	St.	—	—	35-0
Toewall	3	4	2	16-0	St.	—	—	32-0
Twl. & Trans. Sill	4	5	13	3-9	St.	—	—	48-9
Twl. & Trans. Sill	5	4	17	3-9	St.	—	—	63-9
Cutoff Wall	6	4	2	16-3	St.	—	—	32-6
Apron & C.O. Wall	7	5	14	12-0	2	9-6	2-6	168-0
Apron & Toewall	8	5	14	5-3	2	4-0	1-3	73-6
Longitudinal Sill	9	6	4	11-3	2	10-0	1-3	45-0
Apron	10	4	10	18-0	St.	—	—	180-0
Apron	11	4	6	16-6	St.	—	—	99-0
Apron & Toewall	12	5	23	10-3	2	9-0	1-3	235-9
Longitudinal Sill	13	4	14	4-0	54	See detail		56-0
Longitudinal Sill	14	7	4	10-9	2	9-6	1-3	43-0



ELEVATION- LONGITUDINAL SILL

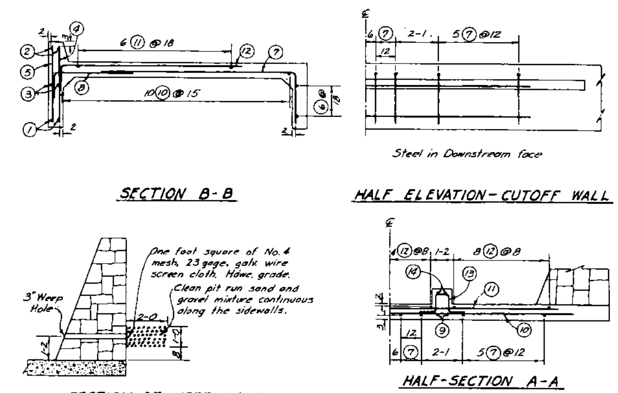


TYPE 2

TYPE 54

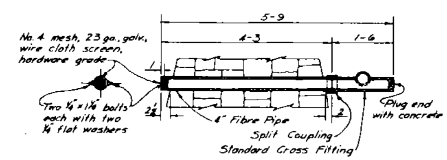
St. is an abbreviation for straight.

BAR TYPE DETAILS



QUANTITIES

Reinforced Concrete		11.6 Cu. Yd.
Cyclopean Concrete		10.0 Cu. Yd.
Masonry		47.2 Cu. Yd.
Reinforcing Steel		
#4	463.25 ft.	309.45 lbs.
#5	558.00 ft.	581.99 lbs.
#6	80.00 ft.	120.16 lbs.
#7	43.00 ft.	87.89 lbs.
Perforated fibre pipe 4" diam		26'-0"
Standard fibre pipe 4" diam.		4'-3"
Standard fibre pipe cross fitting 4" diam.		1
Split fibre couplings		5
No. 4 mesh, 23 gage, galv. wire screen cloth		7.0 Sq. ft.
1/4" × 1 1/4" machine bolts with nuts		2
1/4" flat washers		4
Drain filter material		6.75 Cu. Yd.



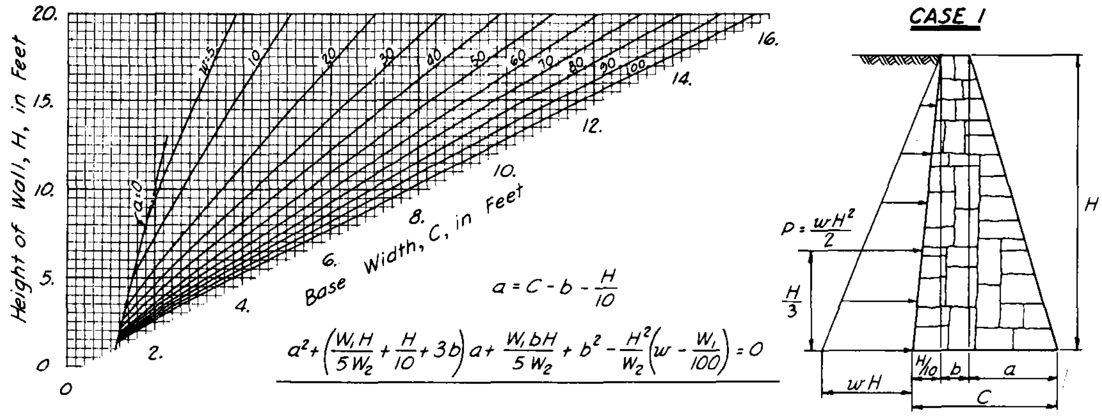
DETAIL OF SMALL ANIMAL GUARD SCREEN AND OUTLET END OF 4" FIBRE PIPE

Masonry Drop Spillway
$F = 6\text{-}0,\ h = 3\text{-}0,\ L = 16\text{-}0$

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

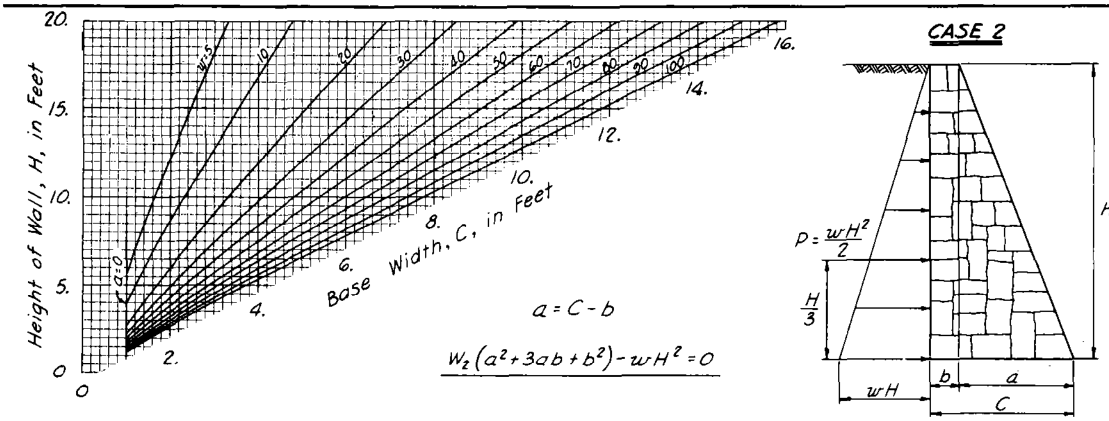
REFERENCE	
ENGINEERING APPROVAL	CARTOGRAPHIC APPROVAL
DESIGNED BY	DRAWN BY T.A.O.
CHECKED BY	STANDARD DWG. NO. ES-71
DATE 4-5-52	SHEET 1 OF 1 SHEETS

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



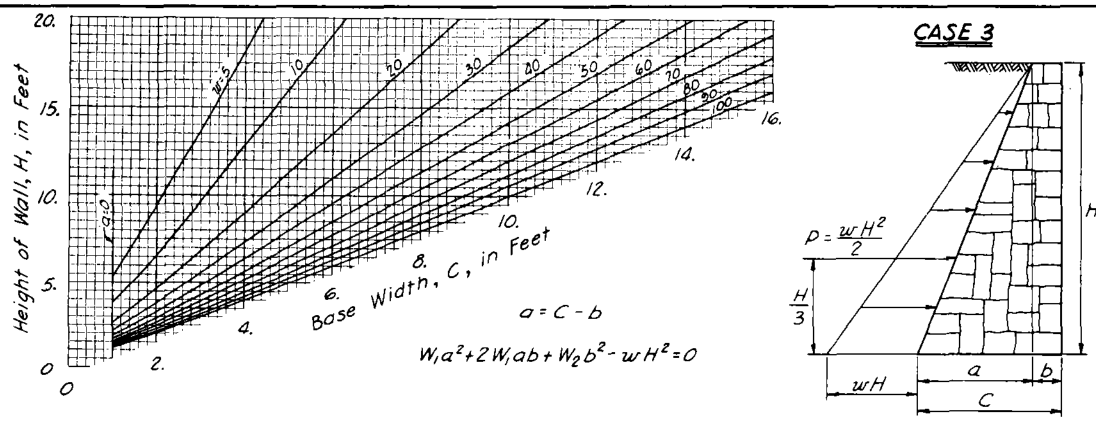
$$a = C - b - \frac{H}{10}$$

$$a^2 + \left(\frac{W_1 H}{5 W_2} + \frac{H}{10} + 3b\right)a + \frac{W_1 b H}{5 W_2} + b^2 - \frac{H^2}{W_2}\left(w - \frac{W_1}{100}\right) = 0$$



$$a = C - b$$

$$W_2(a^2 + 3ab + b^2) - wH^2 = 0$$



$$a = C - b$$

$$W_1 a^2 + 2 W_1 ab + W_2 b^2 - wH^2 = 0$$

NOTES

The curves are plotted for $W_1 = 100$ lbs/ft^3, $W_2 = 150$ lbs/ft^3, $b = 1.0$ 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.^{3}
- 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