

6. STRUCTURAL DESIGN EXAMPLE

General. The following example deals with the stability and structural design only. It was assumed that the site location had been selected and that the hydrologic data indicated that the structure should be designed for a peak discharge of 610 cfs. The explanation of the methods and procedures of design are contained in the example or referenced to the corresponding sections of the Engineering Handbook.

Hydraulic Design (Design $Q = 610$ cfs) ($F = 12.0$ ft)

Weir dimensions: Try $h = 4'-0"$ and $L = 30'-0"$ and compute capacity by equation 3.5 (page 3.7).

$$Q = \frac{3.1 L h^{3/2}}{1.10 + 0.01F} = \frac{3.1 \cdot 30 \cdot 8}{1.22} = 610 \text{ cfs} \quad \text{OK}$$

Height of Transverse Sill (s): $s = h \div 3 = 4.0 \div 3 = 1.33' = 1'-4"$

Apron Length (L_B) and Height of Sidewall (J): (See ES-67, page 5.3)

$$J_{(\min)} = 2h = 2 \cdot 4 = 8.0', \quad L_{B(\min)} = 1.28 \cdot 12 = 15.36'$$

Use sidewall layout shown in the following layout drawing and determine combinations that will maintain 2 to 1 fill slope with 1.0 ft. of fill over upstream edge of headwall extension.

$$J = F + h + s + 1.0 - \left(\frac{L_B + 5''}{2} \right) - \frac{10''}{2}$$

$$L_B = 2 (F + h + s + 1.0 - J) - 0.417 - 0.833$$

$$L_B = 2 (12 + 4.0 + 1.33 + 1 - J) - 1.25$$

$$L_B = 35.42 - 2J$$

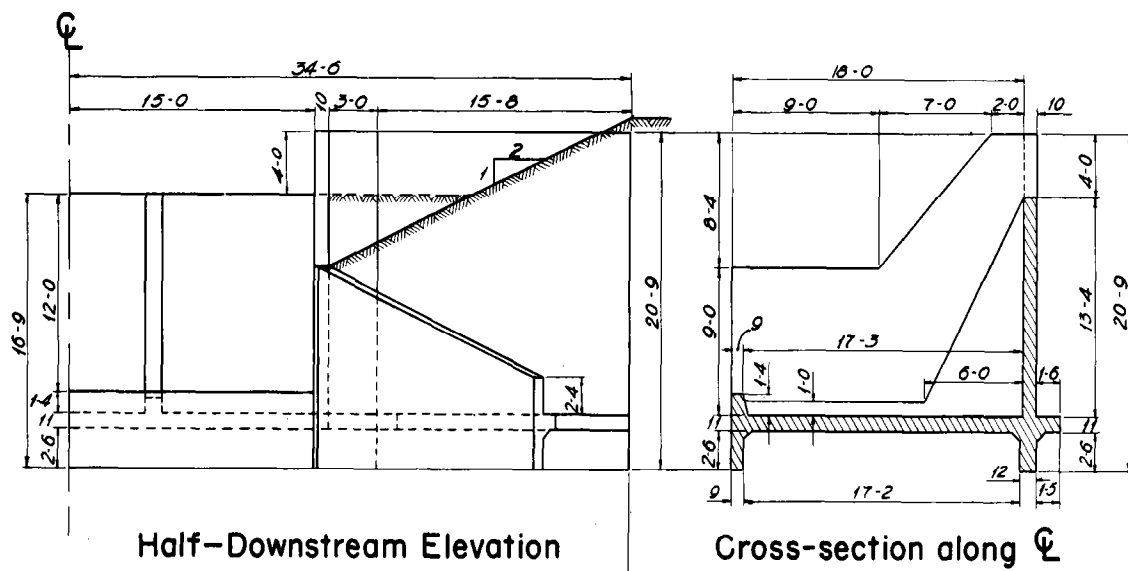
$$\text{When } J = 8.0 \text{ ft, } L_B = 19.42 \text{ ft}$$

$$\text{When } J = 9.0 \text{ ft, } L_B = 17.42 \text{ ft}$$

$$\text{When } J = 10.0 \text{ ft, } L_B = 15.42 \text{ ft}$$

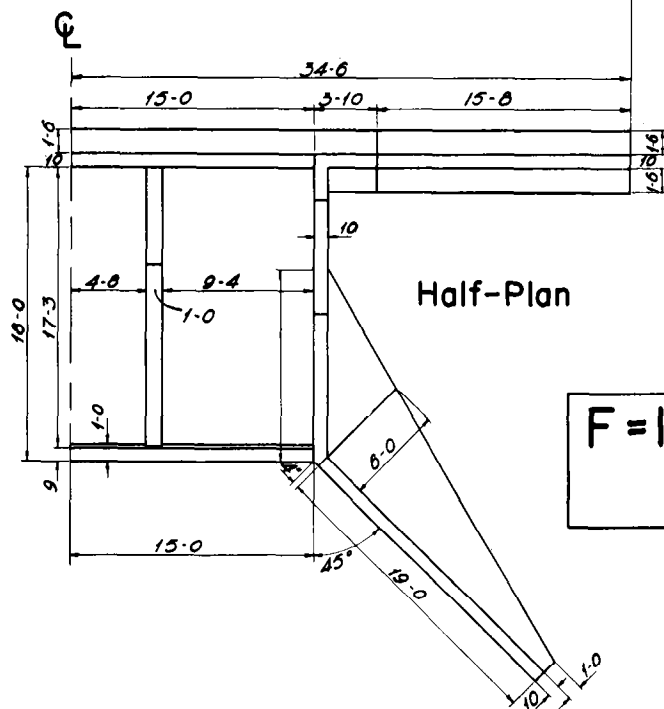
$$\text{Select and use } J = 9'-0", \quad L_B = 17'-3"$$

Shape of Sidewall: The sidewall will have a level top except near the headwall, where it will be shaped to confine the nappe. Plot the path of the upper nappe (See ES-68, page 6.3) from the crest elevation to the level portion of sidewall. Take $d_c = 2/3 h = 2.67$ ft.



Half-Downstream Elevation

Cross-section along $\mathbb{C}$



Half-Plan

$F=12-0, h=4-0, L=30-0$

Handbook Example

DROP SPILLWAYS: DIMENSIONLESS COORDINATES OF WATER SURFACES FOR AERATED NAPPE OVER WEIR WITH LEVEL, FLUSH APPROACH CHANNEL

EQUATIONS

Upper Surface of nappe

$$\frac{x}{d_c} = 2.12\sqrt{\frac{y}{d_c}} + 0.69 - 0.38 \text{ when } \frac{y}{d_c} > 0.3$$

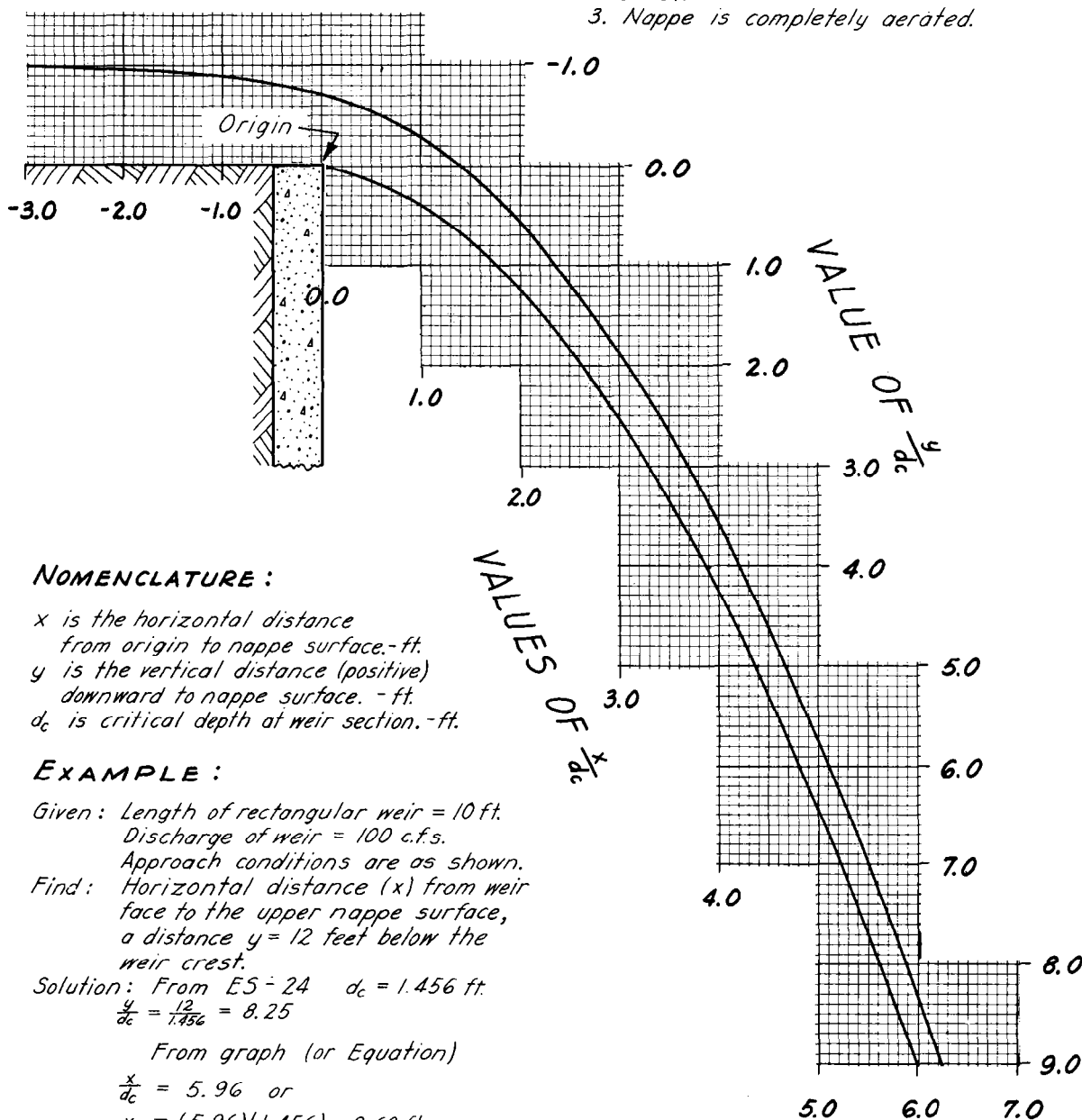
Lower Surface of nappe

$$\frac{x}{d_c} = 2.12\sqrt{\frac{y}{d_c}} - 0.38 \text{ when } \frac{y}{d_c} > 1.0$$

NOTE

The given equations define the nappe surfaces only when the flow conditions are as follows:

1. Approach Channel is level for a distance of $3.0 d_c$ upstream from the weir.
2. Approach Channel is flush with the weir crest.
3. Nappe is completely aerated.



NOMENCLATURE:

x is the horizontal distance from origin to nappe surface, -ft.
 y is the vertical distance (positive) downward to nappe surface, -ft.
 d_c is critical depth at weir section, -ft.

EXAMPLE:

Given: Length of rectangular weir = 10 ft.
 Discharge of weir = 100 c.f.s.
 Approach conditions are as shown.
 Find: Horizontal distance (x) from weir face to the upper nappe surface, a distance $y = 12$ feet below the weir crest.

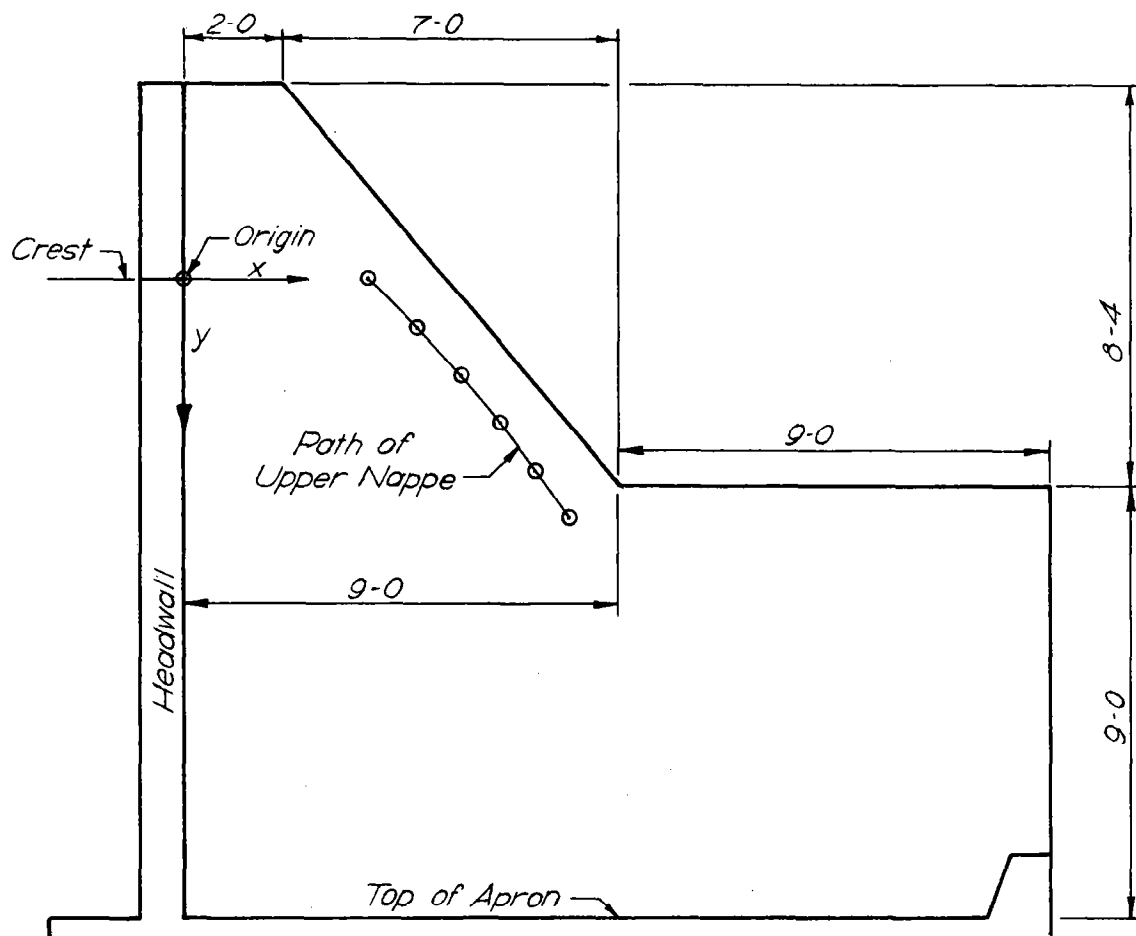
Solution: From ES-24 $d_c = 1.456$ ft.
 $\frac{y}{d_c} = \frac{12}{1.456} = 8.25$

From graph (or Equation)

$$\frac{x}{d_c} = 5.96 \text{ or}$$

$$x = (5.96)(1.456) = 8.68 \text{ ft.}$$

y	y/d _c	x/d _c	x
0	0	1.4	3.74
1	0.375	1.8	4.81
2	0.750	2.15	5.73
3	1.125	2.45	6.55
4	1.500	2.73	7.29
5	1.875	3.00	8.00



Downstream flow: (Tailwater depth above transverse sill, t)

Required depth, $t = 4.6$ ft (See fig. 5.1, page 5.2)

($d_c = 2.67$, $k = 1.25$)

Allowable velocity (max) = 3.0 fps

$s = 0.00095$ ft/ft, $n = 0.035$, sideslopes 3 to 1 or $z = 3$

Try channel bottom width, $b = 30$ ft (See ES-33, Section 5, Hydraulics)

$$\text{Hydraulic radius, } r = \frac{bd + zd^2}{b + 2d \sqrt{z^2 + 1}} = \frac{30 \cdot 4.6 + 3 \cdot 4.6^2}{30 + (2 \cdot 4.6 \sqrt{9 + 1})}$$

$$r = \frac{201.5}{59.1} = 3.41$$

See ES-34, Section 5, Hydraulics

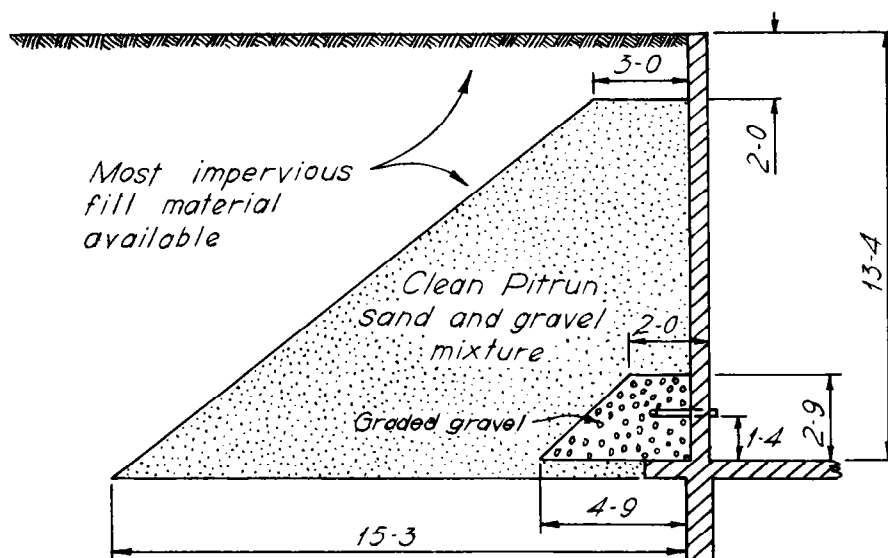
With $r = 3.41$, $s = 0.00095$, and $n = 0.035$, $v = 2.97$ fps OK

$t + s = \text{tailwater depth above apron} = 4.60 + 1.33 = 5.93$ ft

Use 6.0 ft for design purposes

Stability Design: The stability of the structure will be checked for a representative bay. The representative bay will extend $30.83 \div 6 = 5.14$ ft on both sides of the center line of a buttress.

Design of drain and drain filter above structure: Preliminary investigations indicated it would be advantageous to design a very good drain and filter upstream from the headwall to reduce uplift and to minimize the forces tending to slide the structure. Therefore, a drain similar to the one discussed on pages 6.2-13 and 6.2-14 of the Structural Design Section will be used. Use 4 in. Perforated Clay Pipe, with $1/4$ in. diameter perforation, four rows, 3 in. spacing. A.S.T.M. Designation C-211.



Criteria for Filter Gradation

Graded Gravel

$$\frac{85 \text{ percent size of gravel}}{\text{diameter of perforation}} \geq 1.0$$

Pitrun Sand and Gravel

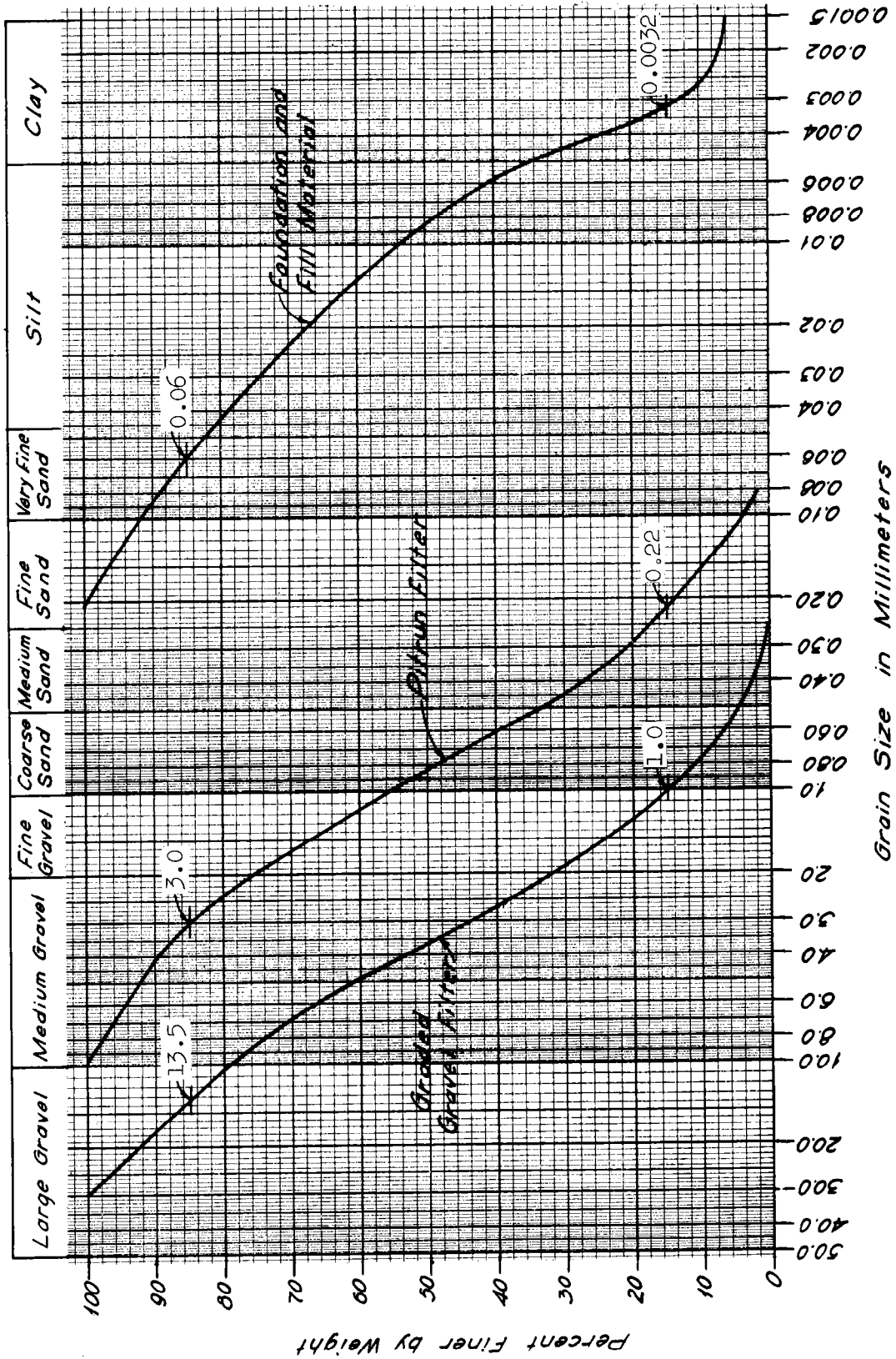
$$\frac{15 \text{ percent size of gravel}}{85 \text{ percent size of pitrun}} \leq 4 \text{ to } 5 \leq \frac{15 \text{ percent size of gravel}}{15 \text{ percent size of pitrun}}$$

and

$$\frac{15 \text{ percent size of pitrun}}{85 \text{ percent size of foundation}} \leq 4 \text{ to } 5 \leq \frac{15 \text{ percent size of pitrun}}{15 \text{ percent size of foundation}}$$

GRAIN SIZE CURVES

(U.S. Bureau of Soils Classification)



Check Filter Gradation: (See preceding page for grain sizes)

	15 percent size	85 percent size
Graded Gravel	1.0 mm	13.5 mm
Pitrun Sand and Gravel	0.22 mm	3.0 mm
Foundation and Fill Material	0.0032 mm	0.06 mm

Pipe perforations = $1/4$ in. diam. = $0.25 \cdot 25.4 = 6.35$ mm

$$\frac{\text{85 percent size of gravel}}{\text{diameter of perforations}} = \frac{13.5}{6.35} = 2.13 > 1.0 \quad \text{OK}$$

$$\frac{\text{15 percent size of gravel}}{\text{85 percent size of pitrun}} = \frac{1.0}{3.0} = 0.33 < 4.0 \quad \text{OK}$$

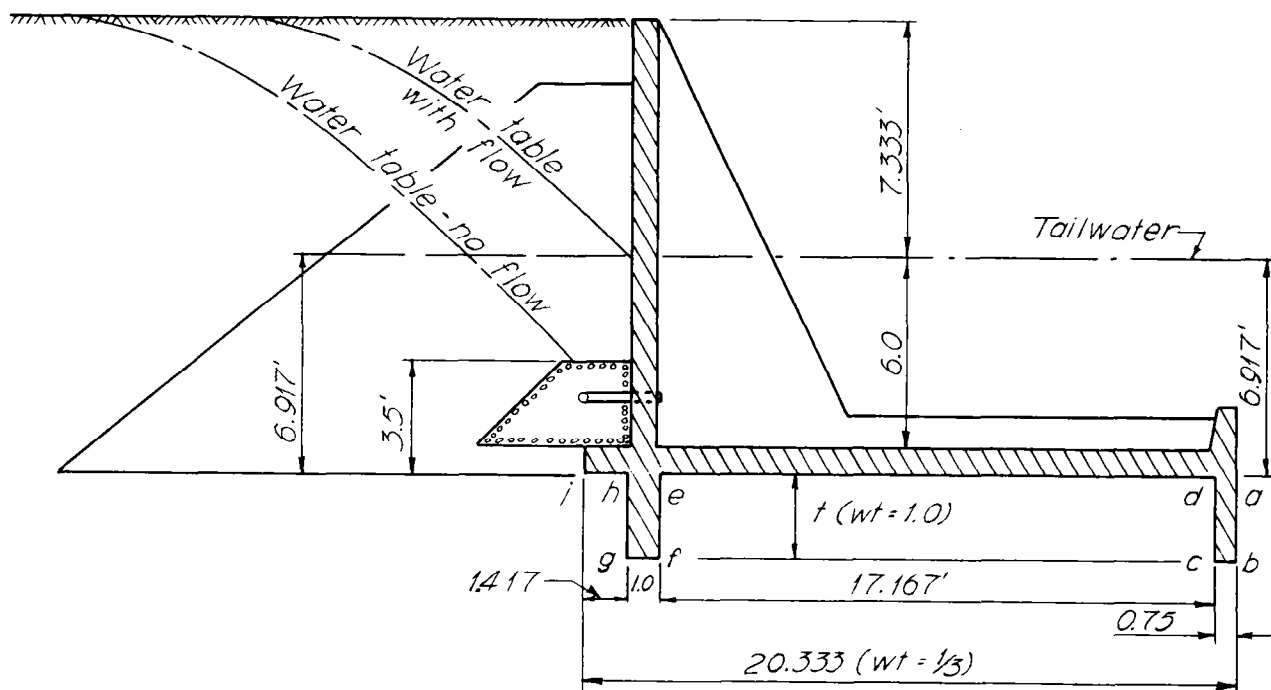
$$\frac{\text{15 percent size of gravel}}{\text{15 percent size of pitrun}} = \frac{1.0}{0.22} = 4.55 > 4.0 \quad \text{OK}$$

$$\frac{\text{15 percent size of pitrun}}{\text{85 percent size of foundation}} = \frac{0.22}{0.06} = 3.67 < 4.0 \quad \text{OK}$$

$$\frac{\text{15 percent size of pitrun}}{\text{15 percent size of foundation}} = \frac{0.22}{0.0032} = 68.8 > 4.0 \quad \text{OK}$$

The 15 percent size of any given material is a unique size for that material such that 15 percent by weight of the material is finer than it. Likewise, the 85 percent size is the size for which 85 percent by weight is finer.

Determine required depth of cutoff wall and toewall: (See Piping, page 4.14)



Foundation Material (Firm clay)--20 percent sand, 45 percent silt, 35 percent clay

$$\text{Wt. Creep Ratio} = C_w = 2.3 \text{ (See page 4.16)}$$

It is estimated that the drain will lower the water table at the structure, as shown in the above sketch. The maximum differential head occurs during periods of no flow.

$$\text{Required Wt. Creep Dist.} = 2.3 \cdot 3.5 = 8.05 = 4t + (20.33 \div 3)$$

$$4t = 8.05 - 6.78 = 1.27, \quad t = 0.32, \quad \text{Use minimum of 2'-6"} \quad \underline{\hspace{1cm}}$$

$$\text{Actual Wt. Creep Dist.} = 4(2.5) + 6.78 = 16.78$$

$$\text{Safe differential head} = H = 16.78 \div 2.3 = 7.30 \text{ ft}$$

Compute Uplift (Refer to sketch in foregoing paragraph)

With flow

$$\text{Uplift} = 6.92 \cdot 62.4 = 432 \text{ lbs/ft}^2$$

$$\text{Total Uplift on bay} = 432 \cdot 20.33 \cdot 10.28 = 90,300 \text{ lbs}$$

$$\text{Wt. of bay} = 163,009 \text{ lbs (See base pressures, with flow)}$$

$$\text{Ratio} = \text{Wt.} \div \text{Uplift} = 163,009 \div 90,300 = 1.80 \quad \text{OK}$$

No flow

$$\text{Pressure at point (a)} = 0, \quad \text{Pressure at point (i)} = 3.5 \cdot 62.4 = 218 \text{ lbs/ft}^2$$

$$\text{Wt. Creep Dist.} = 16.78 \text{ ft, Increment of pressure} = \text{loss per unit of weighted creep dist.} = 218 \div 16.78 = 13.0 \text{ lbs/ft}^2$$

Point	Wt. Creep Dist. ft	Inc. of Pres. lbs/ft ²	Pres. at Point lbs/ft ²	Ave. Pres. between points lbs/ft ²	Base Length ft	Uplift per ft width lbs	Base Width ft	Total Uplift lbs
a	2.5	32.5	0					
b	0.25	3.2	32.5	34	0.75	26	10.28	267
c	2.5	32.5	35.7					
d	5.72	74.4	68.2	105	17.17	1802	10.28	18,500
e	2.5	32.5	142.6					
f	0.333	4.3	175.1	177	1.0	177	10.28	1,820
g	2.5	32.5	179.4					
h	0.472	6.1	211.9	215	1.42	304	10.28	3,120
i			218.0					
Total uplift on bay =								23,707

$$\text{Wt. of bay} = 88,959 \text{ lbs}$$

$$\text{Ratio} = \text{Wt.} \div \text{Uplift} = 88,959 \div 23,707 = 3.75$$

OK

Determine contact and total pressures on base: Take moments about downstream edge at elevation of bottom of apron of representative bay.

Before fill (No uplift)

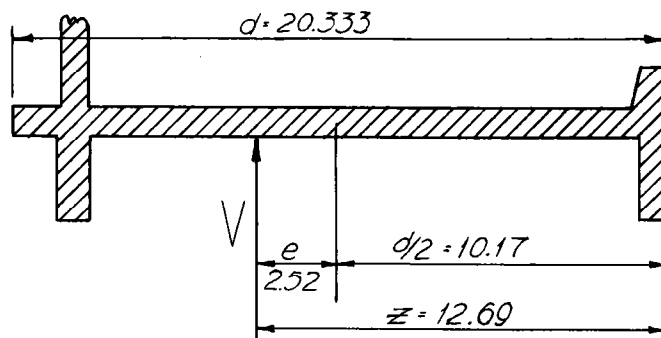
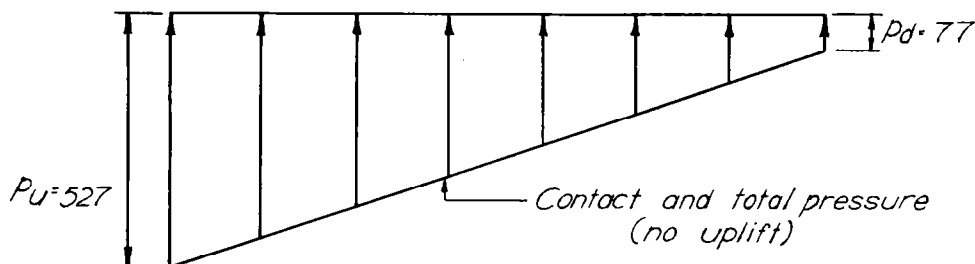
Part		Wt.	Arm	Moment
Apron	$10.28 \cdot 20.33 \cdot 0.917 \cdot 150$	28,700	10.17	292,000
Headwall	$10.28 \cdot 13.33 \cdot 0.833 \cdot 150$	17,100	18.42	315,000
C. O. Wall	$10.28 \cdot 2.5 \cdot 1.0 \cdot 150$	3,860	18.42	71,100
C. O. Fillets	$10.28 \cdot 0.25 \cdot 150$	386	18.42	7,100
Toewall	$10.28 \cdot 2.5 \cdot 0.75 \cdot 150$	2,890	0.375	1,086
Twl. Fillets	$10.28 \cdot 0.125 \cdot 150$	193	0.917	177
Trans. Sill	$10.28 \cdot 1.33 \cdot 0.875 \cdot 150$	1,800	0.440	792
Long. Sill	$17.09 \cdot 1.0 \cdot 1.0 \cdot 150$	2,560	9.44	24,200
Buttress	$12.33 \cdot 3.0 \cdot 1.0 \cdot 150$	5,550	16.00	88,800
		63,039	12.69	800,255

$$z = 800,255 \div 63,039 = 12.69 \text{ ft}, e = 12.69 - 10.17 = 2.52 \text{ ft}$$

$$\text{Area of base} = 10.28 \cdot 20.33 = 209.0 \text{ ft}^2$$

$$p = (V \div A) [1 \pm (6e \div d)] = (63,039 \div 209) [1 \pm (6 \cdot 2.52 \div 20.33)]$$

$$p_u = 302 (1 + 0.744) = \underline{527 \text{ lbs/ft}^2}, \quad p_d = 302 (1 - 0.744) = \underline{77 \text{ lbs/ft}^2}$$



After fill (No flow)

Backfill Properties (upstream from headwall) (compacted, clean pitrun sand and gravel)

Dry wt.	115 lbs/ft ³
Void ratio, e	0.35
Percent voids	$\left(\frac{e}{1+e} \cdot 100 \right) = 25.9$
Moist wt.	125 lbs/ft ³
ϕ	$35^\circ \quad \left(\frac{1 - \sin \phi}{1 + \sin \phi} \right) = 0.271$
Cohesion	0
Eff. submerged wt. =	$[115 - (1 - 0.259) 62.4] = 69 \text{ lbs/ft}^3$
Saturated wt. =	$115 + (0.259 \cdot 62.4) = 131.2 \text{ lbs/ft}^3$

For design purposes assume pitrun material to crest elevation.

Resolve Uplift into Trapezoidal Diagram, to simplify calculations.

Rate of change per ft of horizontal contact = $13.0 \div 3 = 4.33$
(See uplift, no flow)

Let x = unit uplift at downstream edge, y = unit uplift at upstream edge

$$\text{then } y = x + (4.33 \cdot 20.33) = x + 88$$

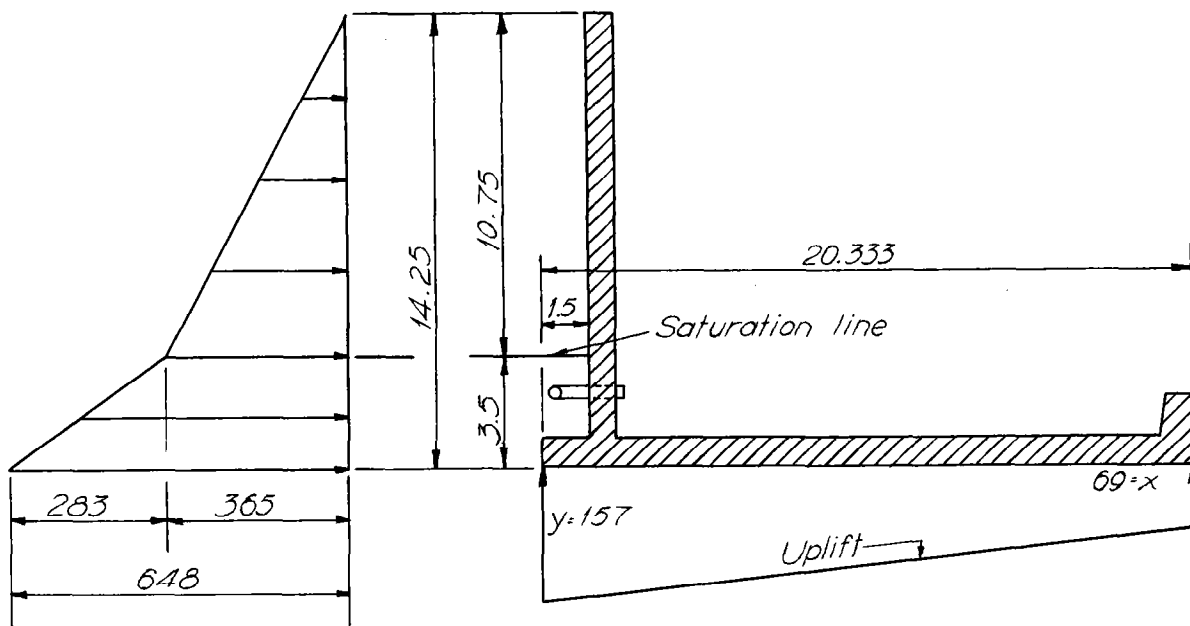
$$\text{Total uplift per ft width} = 23,707 \div 10.28 = 2305 \text{ lbs}$$

$$\left[(x + y) \div 2 \right] d = 2305 \text{ lbs}$$

$$(x + x + 88) \div 2 = 2305 \div 20.33 = 113 \text{ lbs/ft}^2$$

$$x = 113 - 44 = 69 \text{ lbs/ft}^2$$

$$y = 69 + 88 = 157 \text{ lbs/ft}^2$$



Determine horizontal pressuresAt 10.75 ft depth

$$\text{Vert. wt.} = 10.75 \cdot 125 = 1345 \text{ lbs}$$

$$\text{Unit equiv. fluid pres.} = \text{vert. wt.} (1 - \sin \phi \div 1 + \sin \phi)$$

$$p = 1345 (0.271) = 365 \text{ lbs/ft}^2$$

At 14.25 ft depth

$$\text{Vert. wt. (earth)} = 1345 + (3.5 \cdot 69) = 1345 + 242 = 1587 \text{ lbs}$$

$$\text{Unit equiv. fluid pres.} = 1587 (0.271) = 430 \text{ lbs/ft}^2$$

$$\text{Water pres.} = 3.5 \cdot 62.4 = 218 \text{ lbs/ft}^2$$

$$p = 430 + 218 = 648 \text{ lbs/ft}^2$$

Part	Force	Wt.	Arm	Moment
Concrete		63,039	12.69	800,255
Moist sand	$10.28 \cdot 10.75 \cdot 1.5 \cdot 125 =$	20,700	19.58	405,500
Sat. sand	$10.28 \cdot 2.58 \cdot 1.5 \cdot 131 =$	5,220	19.58	102,300
Hor. Pres.	$10.28 \cdot 0.5 \cdot 365 \cdot 10.75 = -20,200$	—	7.08	-143,000
Hor. Pres.	$10.28 \cdot 365 \cdot 3.5 = -13,130$	—	1.75	-23,000
Hor. Pres.	$10.28 \cdot 0.5 \cdot 283 \cdot 3.5 = -5,100$	—	1.17	-5,950
	-38,430	88,959		1,136,105
Uplift 	$10.28 \cdot 69 \cdot 20.33$	-14,400	10.17	-146,300
Uplift 	$10.28 \cdot 0.5 \cdot 88 \cdot 20.33$	-9,200	13.55	-124,700
		-23,600		-271,000
		+65,359	13.23	+865,105

$$z = 865,105 \div 65,359 = 13.23 \text{ ft}$$

$$e = 13.23 - 10.17 = 3.06 \text{ ft}$$

Contact Pressures

$$p_u = (65,359 \div 209) [1 + (6 \cdot 3.06 \div 20.33)]$$

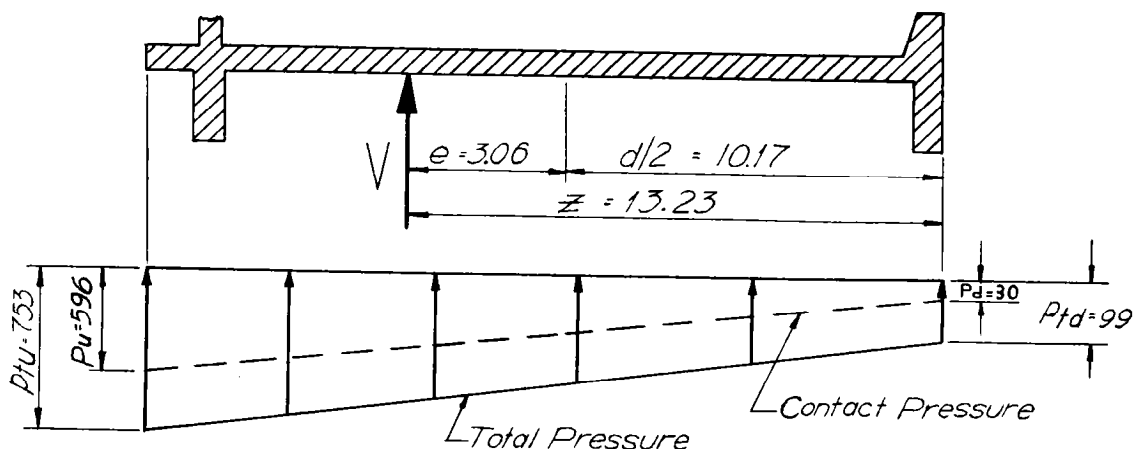
$$p_u = 313 (1 + 0.905) = 596 \text{ lbs/ft}^2$$

$$p_d = 313 (1 - 0.905) = 30 \text{ lbs/ft}^2 \quad \text{OK Positive pressures}$$

Total Pressures

$$p_{tu} = 596 + 157 = 753 \text{ lbs/ft}^2$$

$$p_{td} = 30 + 69 = 99 \text{ lbs/ft}^2$$

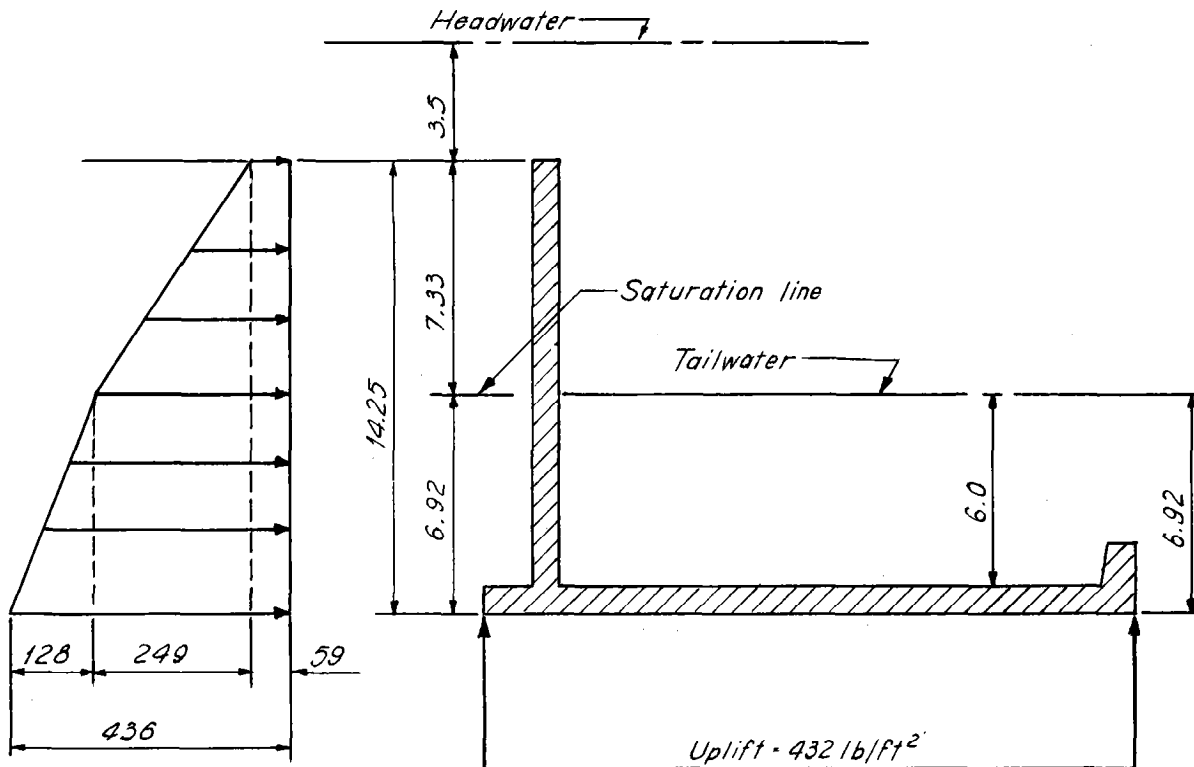


With flow (Design Discharge)

Depth of headwater above crest, use 3.5 ft

Depth of tailwater above bottom of apron = 6.92 ft

Uplift is uniform = 432 lbs/ft²

Determine Horizontal PressuresAt Crest (surcharge of headwater)

$$\text{Vert. Wt.} = 3.5 \cdot 62.4 = 218 \text{ lbs, } p = 218 \cdot 0.271 = 59 \text{ lbs/ft}^2$$

At 7.33 ft depth

$$\text{Vert. Wt.} = 218 + 125 \cdot 7.33 = 218 + 916 = 1134 \text{ lbs}$$

$$p = 1134 \cdot 0.271 = 308 \text{ lbs/ft}^2$$

At 14.25 ft depth

$$\text{Vert. Wt.} = 1134 + 6.92 \cdot 69 = 1134 + 477 = 1611 \text{ lbs}$$

$$p = 1611 \cdot 0.271 = 436 \text{ lbs/ft}^2$$

NOTE: Water pressures in both directions are equal and cancel one another.

Part		Force	Wt.	Arm	Moment
Concrete			63,039		800,255
Moist Sand	$10.28 \cdot 7.33 \cdot 1.5 \cdot 125$		14,150	19.58	277,000
Satur. Sand	$10.28 \cdot 6.92 \cdot 1.5 \cdot 131$		13,980	19.58	274,000
H ₂ O on Foot.	$10.28 \cdot 3.5 \cdot 1.5 \cdot 62.4$		3,370	19.58	66,000
H ₂ O on Apron	$9.28 \cdot 6.0 \cdot 17.25 \cdot 62.4$		59,900	9.38	562,000
H ₂ O on T. S.	$10.28 \cdot 4.67 \cdot 0.75 \cdot 62.4$		2,250	0.375	843
H ₂ O on Butt.	$1.0 \cdot 2.5 \cdot 2.43 \cdot 62.4$		379	12.81	4,850
H ₂ O on L. S.	$1.0 \cdot 11.25 \cdot 5.0 \cdot 62.4$		3,510	6.38	22,400
Hor. Pres.	$10.28 \cdot 59 \cdot 14.25$	8,650	—	7.13	— 61,600
Hor. Pres.	$10.28 \cdot 0.5 \cdot 249 \cdot 7.33$	9,390	—	9.36	— 87,900
Hor. Pres.	$10.28 \cdot 249 \cdot 6.92$	17,700	—	3.46	— 61,300
Hor. Pres.	$10.28 \cdot 0.5 \cdot 128 \cdot 6.92$	4,550	—	2.31	— 10,500
		40,290	+160,578		+1,786,048
Uplift	$10.28 \cdot 432 \cdot 20.33$		- 90,300	10.17	- 918,000
			+ 70,278	12.36	+ 868,048

$$z = 868,048 \div 70,278 = 12.36 \text{ ft}, e = 12.36 - 10.17 = 2.19 \text{ ft}$$

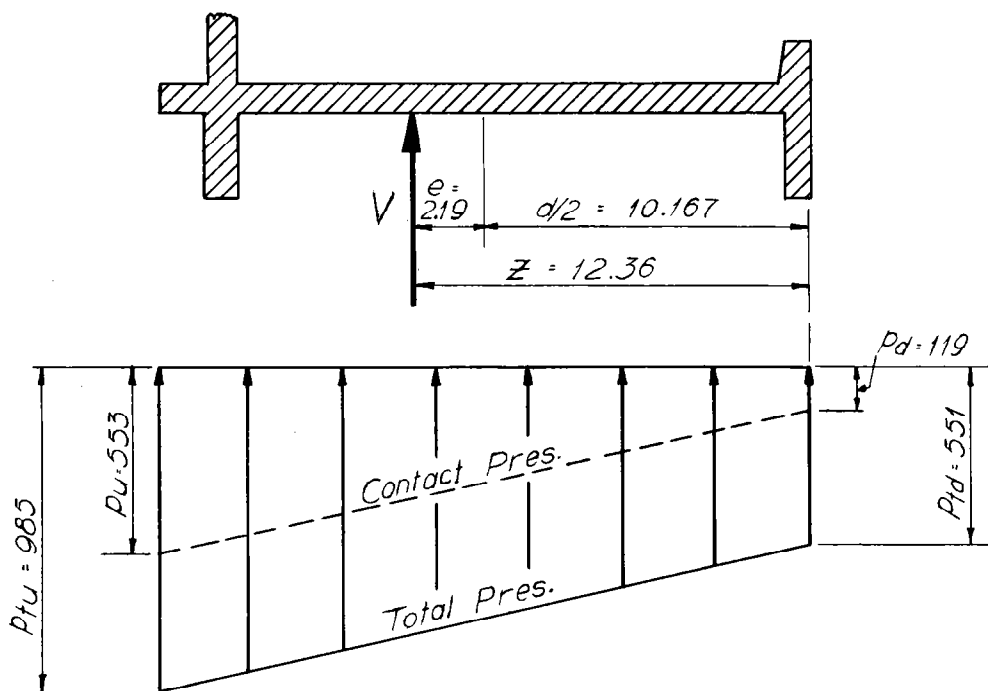
$$p_u = (70,278 \div 209) [1 + (6 \cdot 2.19 \div 20.33)]$$

$$p_u = 336 (1 + 0.646) = 553 \text{ lbs/ft}^2$$

$$p_d = 336 (1 - 0.646) = 119 \text{ lbs/ft}^2$$

$$p_{tu} = 553 + 432 = 985 \text{ lbs/ft}^2$$

$$p_{td} = 119 + 432 = 551 \text{ lbs/ft}^2$$



Check Sliding: Assume plane of sliding at elevation of bottom of cutoffs. Neglect passive resistance in front of toewall.

Properties of foundation material (Undisturbed and saturated)

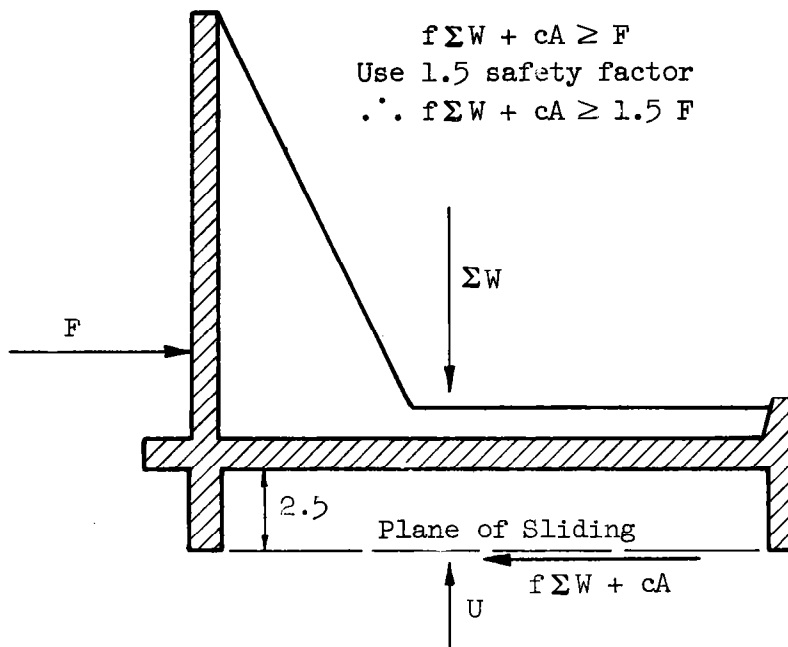
Angle of internal friction, $\phi = 12^\circ$, $\left(\frac{1 - \sin \phi}{1 + \sin \phi} \right) = 0.655$

Friction coefficient = $\tan \phi = 0.213 = f$

Void ratio = 0.65

Effective submerged wt. = 62.2 lbs/ft³

Cohesion, $c = 500$ lbs/ft² for sliding



No flow

Eff. wt. of soil between cutoffs = $2.5 \cdot 10.28 \cdot 17.17 \cdot 62.2 = 27,500$ lbs

Wt. of bay minus uplift (See base pressures) = 65,359 lbs

$\Sigma W = 92,859$ lbs

Area of sliding plane, $A = 17.17 \cdot 10.28 = 176.5$ ft²

$\therefore f \Sigma W + cA = (0.213 \cdot 92,859) + (500 \cdot 176.5) = 108,000$ lbs

Determine F (See contact pressure calculation for horizontal forces from crest to bottom of apron, pages 6.10 and 6.11.)

At 14.25 ft depth (just below apron)

Vert. wt. = 1587 lbs

$p = 1587 \cdot 0.655 = 1040$ lbs/ft²

At 16.75 ft depth

Vert. wt. = $1587 + (2.5 \cdot 62.2) = 1587 + 155 = 1742$ lbs

$p = 1742 \cdot 0.655 = 1140$ lbs/ft²

$$F = 10.28 \left[(365 \cdot 0.5 \cdot 10.75) + (1013 \cdot 0.5 \cdot 3.5) + (2180 \cdot 0.5 \cdot 2.5) \right]$$

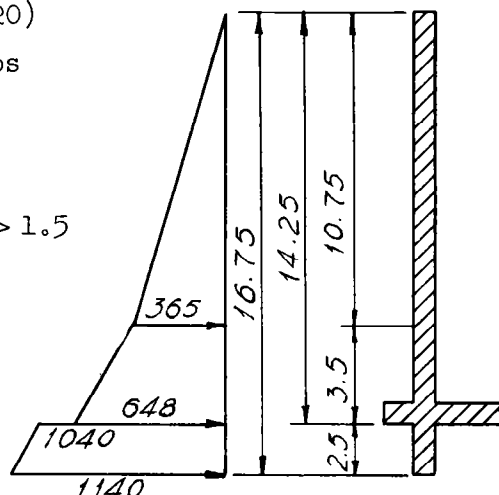
$$F = 10.28 (1960 + 1770 + 2720)$$

$$F = 10.28 \cdot 6450 = 66,300 \text{ lbs}$$

$$1.5F = 99,450 \text{ lbs}$$

OK Safe from sliding

$$\frac{f \Sigma W + cA}{F} = \frac{108,000}{66,300} = 1.63 > 1.5$$



With flow (Design discharge)

$$\text{Eff. wt. of soil} = 27,500 \text{ lbs}$$

$$\text{Wt. of bay - uplift} = 70,278 \text{ lbs}$$

$$\Sigma W = 97,778 \text{ lbs}$$

$$f \Sigma W + cA = 0.212 (97,778) + 500 (1765) = 20,720 + 88,250 = 108,270$$

Determine F (See contact pressure calculations, page 6.12 and 6.13)

At 14.25 ft depth (just below apron)

$$\text{Vert wt.} = 1611 \text{ lbs, } p = 1611 \cdot 0.655 = 1055 \text{ lbs/ft}^2$$

At 16.75 ft depth

$$\text{Vert. wt.} = 1611 + 155 = 1766 \text{ lbs, } p = 1766 \cdot 0.655 = 1157 \text{ lbs/ft}^2$$

$$F = 10.28 \left[(367 \cdot 0.5 \cdot 7.33) + (744 \cdot 0.5 \cdot 6.92) + (2212 \cdot 0.5 \cdot 2.5) \right]$$

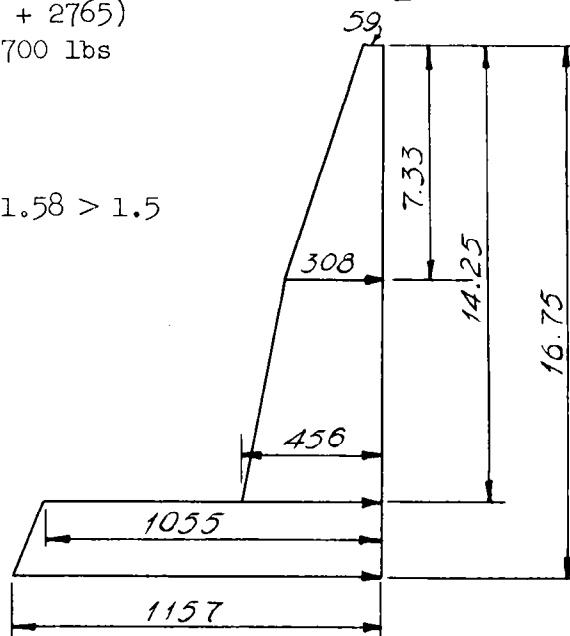
$$F = 10.28 (1345 + 2575 + 2765)$$

$$F = 10.28 \cdot 6685 = 68,700 \text{ lbs}$$

$$1.5F = 103,050 \text{ lbs}$$

OK Safe from sliding

$$\frac{f \Sigma W + cA}{F} = \frac{108,270}{68,700} = 1.58 > 1.5$$



Headwall Design: The headwall will be designed as a slab with fixed sides and bottom and a free top (see ES-6, Engineering Handbook, Section 6). To use this reference, it is necessary to resolve the actual load diagram into a triangular load diagram. This is done by making the cantilever moment at the base of the headwall equal for both load diagrams. The maximum load will occur with the design flow over the structure. (See load computed to determine contact pressures.)

Determine Equivalent Hydrostatic Pressure (w) for Triangular Load Diagram. (See actual load computed to determine contact pressures)

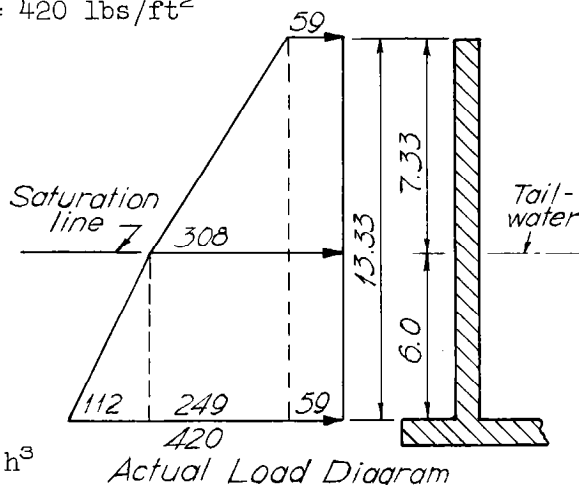
At 13.33 ft depth

$$\text{Vert. wt.} = 1134 + 6 (69) = 1134 + 414 = 1548 \text{ lbs}$$

$$p = 1548 (0.271) = 420 \text{ lbs/ft}^2$$

Taking Moments about base of headwall

Force	Arm	M
$59 \cdot 13.33 = 787$	6.67	5,250
$249 \cdot 0.5 \cdot 7.33 = 912$	8.44	7,700
$249 \cdot 6.0 = 1494$	3.0	4,482
$112 \cdot 0.5 \cdot 6.0 = 336$	2.0	672
M =		18,104



For Triangular Load Diagram

$$M = wh^3 \div 6 \text{ or } w = 6M \div h^3$$

$$w = 6 (18,104) \div (13.33)^3$$

$$w = 45.9 \text{ lbs/ft}^3 \quad \text{Use } 46.0 \text{ lbs/ft}^3$$

Slab Design (with two buttresses) (Class B Concrete)

$$b = 30.83 \div 3 = 10.28 \text{ ft, } a = 13.33 \text{ ft, } b/a = 0.77$$

Check minimum wall thickness of 10 in (d = 7.5 in)

$$M_{\max} = -0.025 \cdot 46 \cdot (13.33)^3 = 2730 \text{ ft lbs (see ES-6, sheet 1)}$$

$$\text{Req'd. } d = 3.8 \text{ in OK (See ES-45, Engr. Hbk., Sec. 6)}$$

$$V_{\max} = 0.27 \cdot 46 \cdot (13.33)^2 = 2210 \text{ lbs}$$

$$\text{Req'd. } d < 3.0 \text{ in OK (See ES-51, Engr. Hbk., Sec. 6)}$$

Vertical Steel (Unexposed face) d = 10 - 2.5 = 7.5 in

$$-M_x = 0.025 \cdot 109,300 = 2730 \text{ ft lbs, } A_s = 0.235 \text{ (from ES-45)}$$

$$V = 0.197 \cdot 8190 = 1610 \text{ lbs, } \Sigma O < 1.0 \text{ (from ES-44, Sec. 6)}$$

Use No. 5 at 15 ($A_s = 0.25$, $\Sigma O = 1.57$) (See ES-46, Sec. 6)

Find where No. 4 at 15 ($A_s = 0.16$) can be used

$$\text{Allowable } M = 1900 \text{ ft lbs (from ES-45)}$$

Allowable $-C_m = 1900 \div 109,300 = 0.0174$ which corresponds to a point $(2.3 \div 40) 13.33 = 0.77 \text{ ft}$ above top of apron. (See M_x moment curves)

∴ No. 5 bars may be cut $0.77 + 1.0 = 1.77$ ft. above apron and will extend into cutoff wall. Length of No. 5 bars $1.77 + 0.92 + 2.5 - 0.25 = 4.94$ ft.

Use 5'-0" and extend bars 1'-10" above apron

Set No. 4 bars on construction joint

Length = $(13'-4") - (0'-4") - (0'-3") = 12'-9"$

SHEAR AND MOMENT COEFFICIENTS

	$M_x = 0$	0	0	$x = 0$
	$M_y = +.005$	$+.001$	$-.008$	
	$M_x = +.002$	0	$-.0025$	$x = \frac{1}{4}$
	$M_y = +.008$	$+.002$	$-.0120$	
	$M_x = +.005$	$+.002$	$-.0035$	$x = \frac{1}{2}$
	$M_y = +.010$	$+.003$	$-.0180$	
	$M_x = +.007$	$+.003$	$-.003$	$x = \frac{3}{4}$
	$M_y = +.007$	$+.003$	$-.014$	
	$M_x = -.025$	$-.0155$	0	$x = a$
	$M_y = -.005$	$-.0030$	0	

$C_s = 0.27 @ 0.775a$

$C_s = 0.197$

$V = C_s w a^2 = C_s (46) (13.33)^2$

$V = 8190 C_s$

$M = C_m w a^3 = C_m (46) (13.33)^3$

$M = 109,300 C_m$

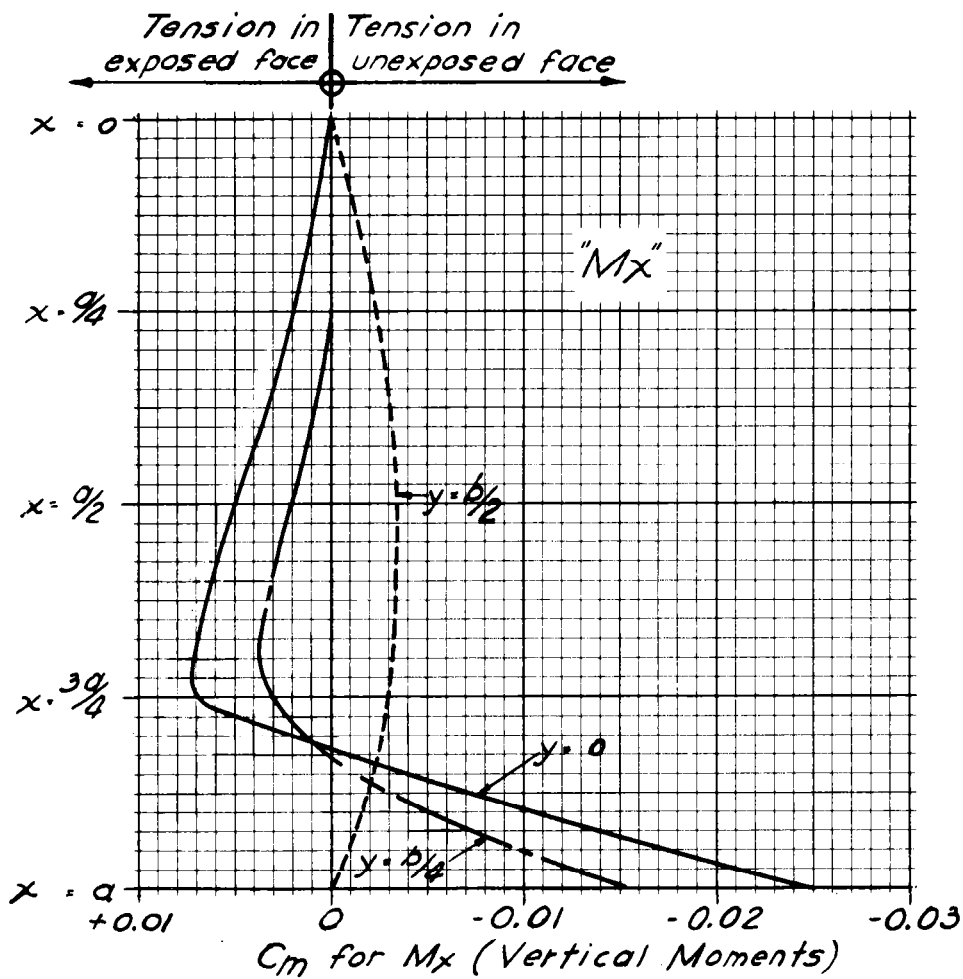
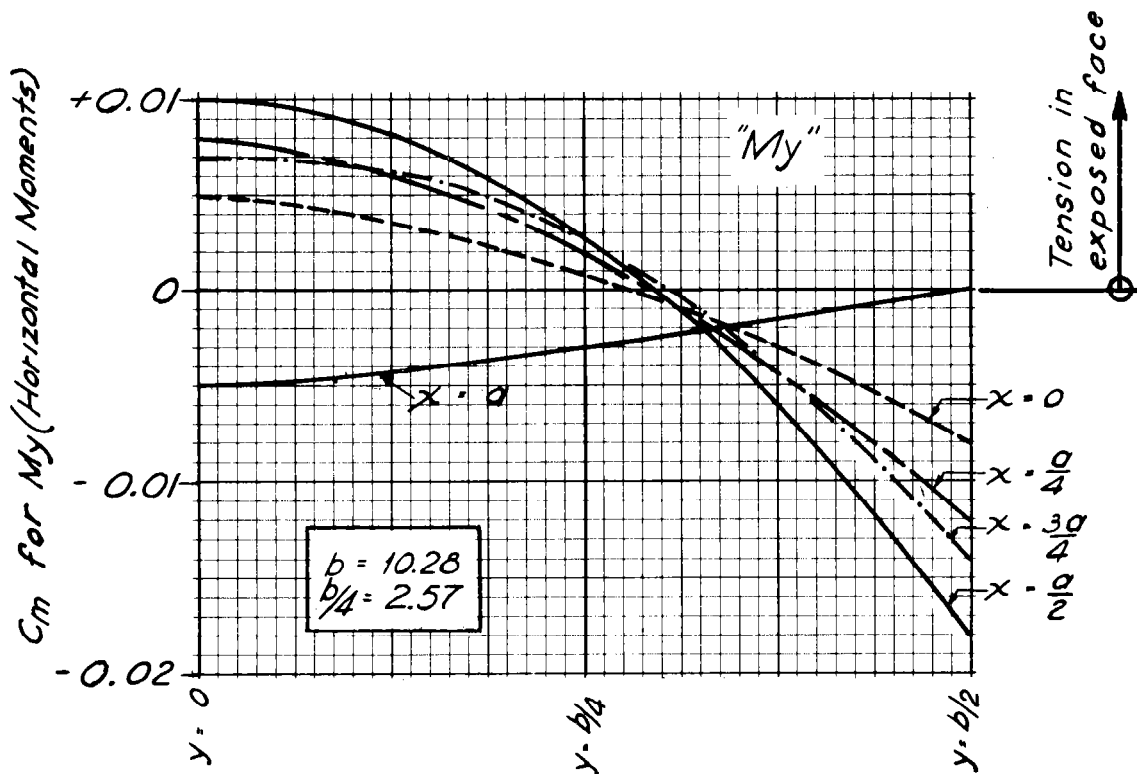
Horizontal Steel (Unexposed face) $d = 10 - 2.0 - 5/8 - 1/4 = 7 \frac{1}{8}"$

$-M_y = 0.018 \cdot 109,300 = 1970$ ft lbs, Req'd. $A_s = 0.18$ in²

$V = 0.27 \cdot 8190 = 2210$ lbs, Req'd. $\Sigma_o = 1.2$ in

Use No. 4 at 12 Splice at center line of structure and extend into headwall extension stub.

HEADWALL MOMENT COEFFICIENT CURVES



Horizontal Steel (Exposed face) $d = 7 \frac{1}{2}$ in

$$+ M_y = 0.01 \cdot 109,300 = 1093 \text{ ft lbs, Req'd. } A_s < 0.10 \text{ in}^2$$

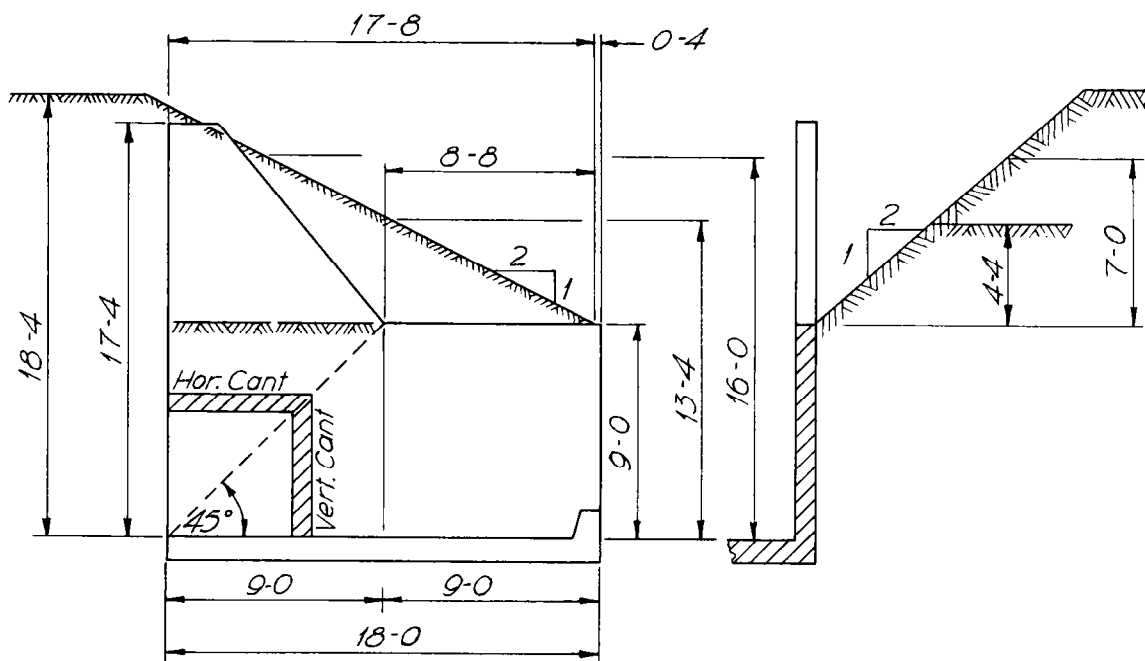
Use No. 5 at 15 ($A_s = 0.25$) to meet temp. requirements.
Splice at center line of structure and extend into headwall extension stub.

Vertical Steel (Exposed face) $d = 10 - 2 - 1/2 - 1/4 = 7 \frac{1}{4}$ in

$$+ M_x = 0.0075 \cdot 109,300 = 820 \text{ ft lbs, Req'd. } A_s < 0.10 \text{ in}^2$$

Use No. 4 at 15 ($A_s = 0.16$) Min. for 10 in. wall. Extend into cutoff wall and cut 1'-7" above top of apron. Length of bars = 4'-9". Lap on No. 4 set on construction joint. Length = 12'-9"

Sidewall Design: The sidewall is designed as vertical and horizontal beams divided as shown in the sketch below by the 45° line.



Backfill Properties (Compacted)

Dry wt.	= 100 lbs/ft ³	specific gravity of solids = 2.65
Void ratio (e)	= 0.65	
Percent voids	= 39.4	
Moist wt.	= 110 lbs/ft ³	
Percent moisture	= 10	
ϕ (moist)	= $25^\circ \left(\frac{1 - \sin \phi}{1 + \sin \phi} \right)$	= 0.406
Cohesion (c)	= consider as 0	

Equivalent fluid pressure on vertical cantilever section

Maximum ht. of surcharge on 2 to 1 slope = 4.33 ft
 $w = 70 \text{ lbs/ft}^3$ (See graphical solution)

Equivalent fluid pressures on horizontal cantilever section

Assumed average ht. of surcharge on 2 to 1 slope = 7.0 ft
 $w = 78 \text{ lbs/ft}^3$ (See graphical solution)

Vertical Steel (Unexposed face) $d = 10 - 2.5 = 7.5 \text{ in}$

Use ES-4 $M = 121.5 \cdot 70 = 8505 \text{ ft lbs}$, Req'd. $A_s = 0.77 \text{ in}^2$
 $V = 40.5 \cdot 70 = 2835 \text{ lbs}$, Req'd. $\Sigma_o = 1.46 \text{ in}$

Use No. 7 at 9 ($A_s = 0.80$, $\Sigma_o = 3.67$)

Find where No. 4 at 9 ($A_s = 0.26$) can be used

Allowable $M = 3010 \text{ ft lbs}$ (from ES-45)
 $3010 \div 70 = 43.2$; (from ES-4, EH, Sec. 6) distance from top of wall to point where No. 4 at 9 will carry moment = 6'-3" which corresponds to point 2'-9" above top of apron. Length of vertical leg of No. 7 at 9 = $2.75 + 1.00 + 0.67 = 4.42 \text{ ft}$ (Use 4'-6")

Horizontal Steel (Unexposed face) $d = 10 - 2 - 7/8 - 1/2 = 6 \frac{5}{8} \text{ in}$

Find points of maximum shear and moments

$$V = w (H - x) x ; \frac{dV}{dx} = H - 2x = 0$$

$$\therefore x = \frac{H}{2}$$

$$V_{\max} = 78 (4.5) 4.5 = 1580 \text{ lbs}$$

$$M = w (H - x) \frac{x^2}{2} ; \frac{dM}{dx} = Hx - \frac{3}{2} x^2 = 0$$

$$\therefore x = \frac{2}{3} H$$

$$M_{\max} = 78 (3) \frac{36}{2} = 4210 \text{ ft lbs}$$

Req'd. $A_s = 0.42 \text{ in}^2$, Req'd. $\Sigma_o < 1.0 \text{ in}$

Use No. 7 at 15 ($A_s = 0.48$, $\Sigma_o = 2.20$)

Find where No. 4 at 15 ($A_s = 0.16$) is sufficient

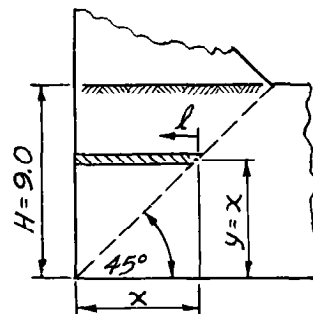
$M = 1660 \text{ ft lbs}$ (from ES-45, EH, Sec. 6)

$$M = w (H - x) \frac{l^2}{2} ; 1660 = 78 \cdot 3 (l^2 \div 2);$$

$$l^2 = \frac{1660 \cdot 2}{78 \cdot 3} = 14.2 \text{ ft}^2$$

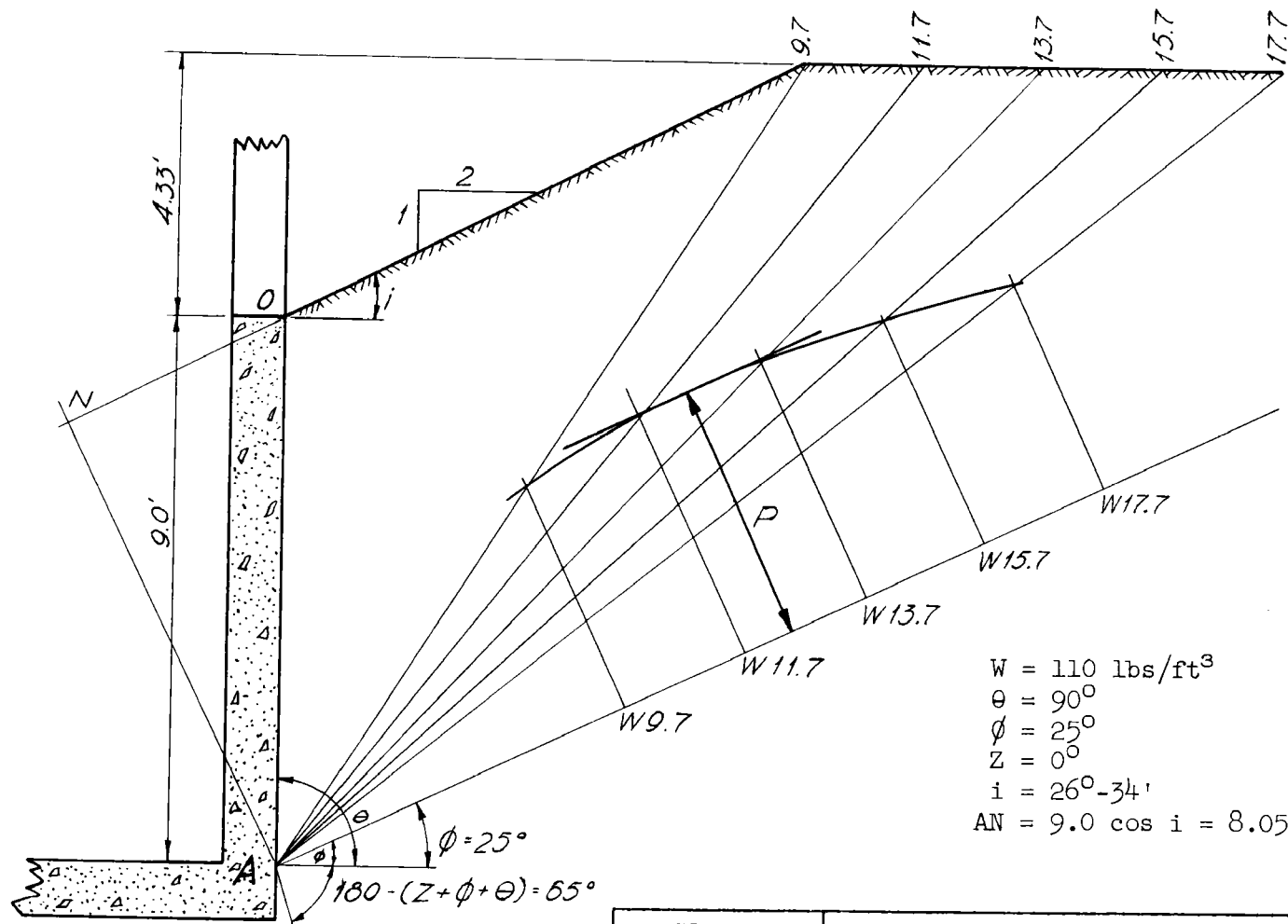
$l = 3.77 \text{ ft}$ which corresponds to a point $6 - 3.77 = 2.23 \text{ ft}$ from downstream face of headwall.

Length of leg of bar = $(2'-3") + (1'-0") + (0'-7 \frac{1}{2}")$
 $= 3'-10 \frac{1}{2} \text{ in.}$ (Use 4'-0")

Steel in Exposed Face

Horizontal Steel--Use No. 5 at 15 (temp. steel) place nearer face.

Vertical Steel--Use No. 4 at 15 (tie steel)



$$\begin{aligned}
 W &= 110 \text{ lbs/ft}^3 \\
 \theta &= 90^\circ \\
 \phi &= 25^\circ \\
 Z &= 0^\circ \\
 i &= 26^\circ - 34' \\
 AN &= 9.0 \cos i = 8.05
 \end{aligned}$$

$$P = 2870 \text{ lbs} = \frac{WH^2}{2}$$

$$w = \frac{5740}{(9)^2} = 70.9 \text{ lbs/ft}^3$$

Use 70.0 lbs/ft³

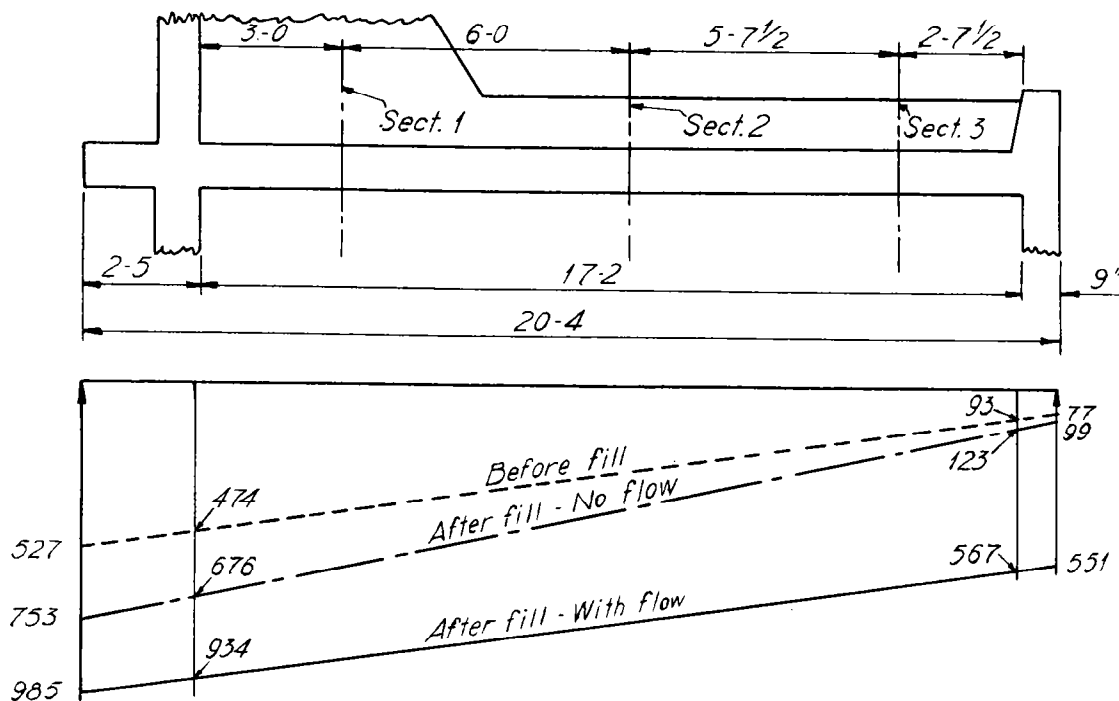
Slice	Weight of Slice	Acc Wt
A-0-9.7	$8.05 \cdot 0.5 \cdot 9.7 \cdot 110 = 4,290$	4,290
A-9.7-11.7	$13.33 \cdot 1.0 \cdot 110 = 1,467$	5,757
A-11.7-13.7	$13.33 \cdot 1.0 \cdot 110 = 1,467$	7,224
A-13.7-15.7	$13.33 \cdot 1.0 \cdot 110 = 1,467$	8,691
A-15.7-17.7	$13.33 \cdot 1.0 \cdot 110 = 1,467$	10,158

See Paragraph 2.2.2 of Structural Design

Apron Design: The apron will be designed as a series of beams in the transverse direction, considered restrained at the sidewalls and continuous over the longitudinal sills. The apron will be designed for the following loading conditions: (1) before backfill has been placed, (2) after backfill has been placed with no flow, (3) after backfill has been placed with design flow.

Upward Total Pressures on Base

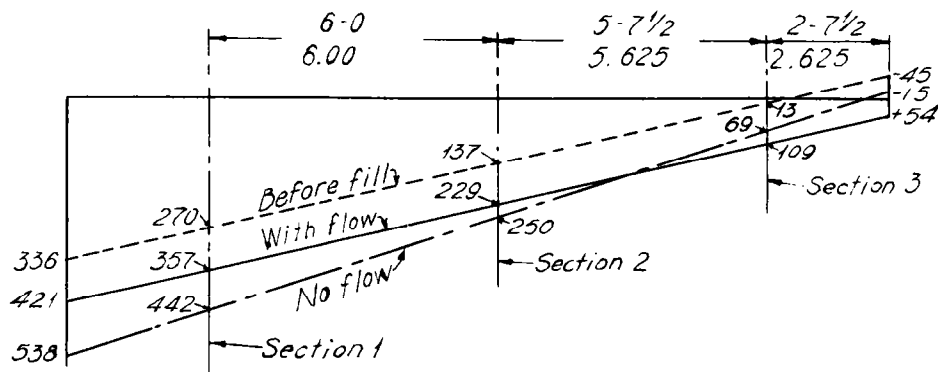
(See determination of contact and total pressures)



Net Loads on Apron (Total Pressure less wt. of concrete and H₂O)

$$\text{Wt. of concrete/ft}^2 \text{ of apron} = 11/12 \cdot 150 = 138 \text{ lbs}$$

$$\text{Wt. of H}_2\text{O on apron/ft}^2 = 62.4 \cdot 6.0 = 375 \text{ lbs}$$



Due to the wide variation in loads across the apron, the apron was divided into three approximately equal parts. The stress analysis will be made at the mid-sections of these parts at Sections 1, 2, and 3.

At Section 1

Before fill $l = 30.83 \div 3 = 10.28 \text{ ft}$, $l^2 = 105.7 \text{ ft}^2$

Sidewall Moment, $M_{ab} = 0$

Load on apron, $w = 270 \text{ lbs/ft}^2$

$$M_{ba} = \frac{1}{10} w l^2 = 27 \cdot 105.7 = 2850 \text{ ft lbs (See ES-56, page 4.27)}$$

$$V_{ab} = -\frac{1}{2} w l + \frac{1}{l} (M_{ab} + M_{ba}) = - (135 \cdot 10.28) + (2850 \div 10.28)$$

$$V_{ab} = -1388 + 277 = -1111 \text{ lbs}$$

$$V_{ba} = w l + V_{ab} = 2776 - 1111 = +1665 \text{ lbs}$$

$$V_{bc} = -\frac{1}{2} w l = -1388 \text{ lbs}$$

$$R_a = 1111 \text{ lbs (down)}$$

$$R_b = 1665 + 1388 = 3053 \text{ lbs (down)}$$

After fill--No flow $w = 442 \text{ lbs/ft}^2$

Sidewall Moment, $= -M_{ab}$ (at Q of apron)

$$M_{ab} = -\frac{6 \cdot 70 \cdot (3.42)^2}{2} - \frac{3.42 \cdot 70 (3.42)^2}{6}$$

$$M_{ab} = -2460 - 467 = -2927 \text{ ft lbs}$$

$$M_{ba} = \frac{1}{5} M_{ab} + \frac{1}{10} w l^2$$

$$M_{ba} = -585 + 44.2 (105.7) = -585 + 4670 = +4085 \text{ ft lbs}$$

$$V_{ab} = -\frac{1}{2} w l + \frac{1}{l} (M_{ab} + M_{ba}) = - (221 \cdot 10.28) + \frac{1}{10.28} (-2927 + 4085)$$

$$V_{ab} = -2270 + 113 = -2157 \text{ lbs}$$

$$V_{ba} = w l + V_{ab} = 4540 - 2157 = +2383 \text{ lbs}$$

$$V_{bc} = -\frac{1}{2} w l = -2270 \text{ lbs}$$

$$R_a = 2157 \text{ lbs (down)}$$

$$R_b = 2383 + 2270 = 4653 \text{ lbs (down)}$$

After fill--With flow $w = 357 \text{ lbs/ft}^2$

$$M_{ab} = -2927 + (3 \cdot 62.4 \cdot 3 \cdot 1.92) + (3 \cdot 62.4 \cdot 1.5 \cdot 1.42)$$

$$M_{ab} = -2927 + 1078 + 399 = -1450 \text{ ft lbs}$$

$$M_{ba} = (-1450 \div 5) + (35.7 \cdot 105.7)$$

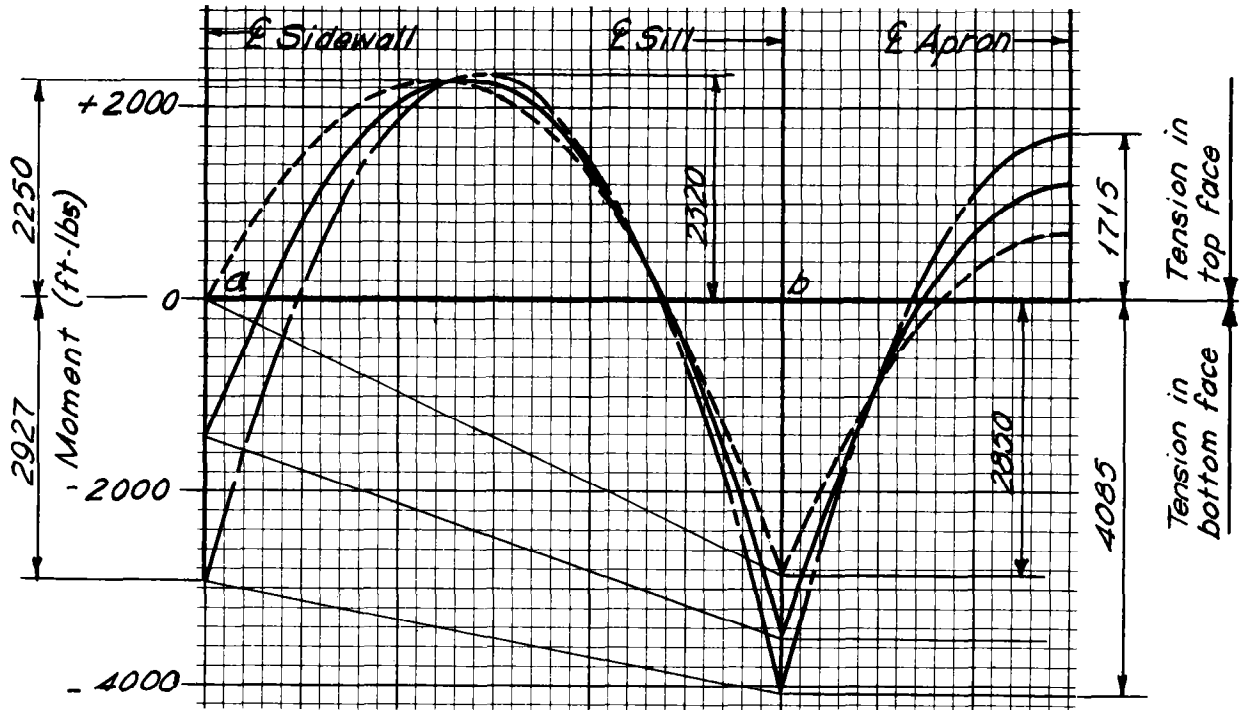
$$M_{ba} = -290 + 3770 = +3480 \text{ ft lbs}$$

$$V_{ab} = (-178 \cdot 10.28) + \frac{1}{10.28} (-1450 + 3470)$$

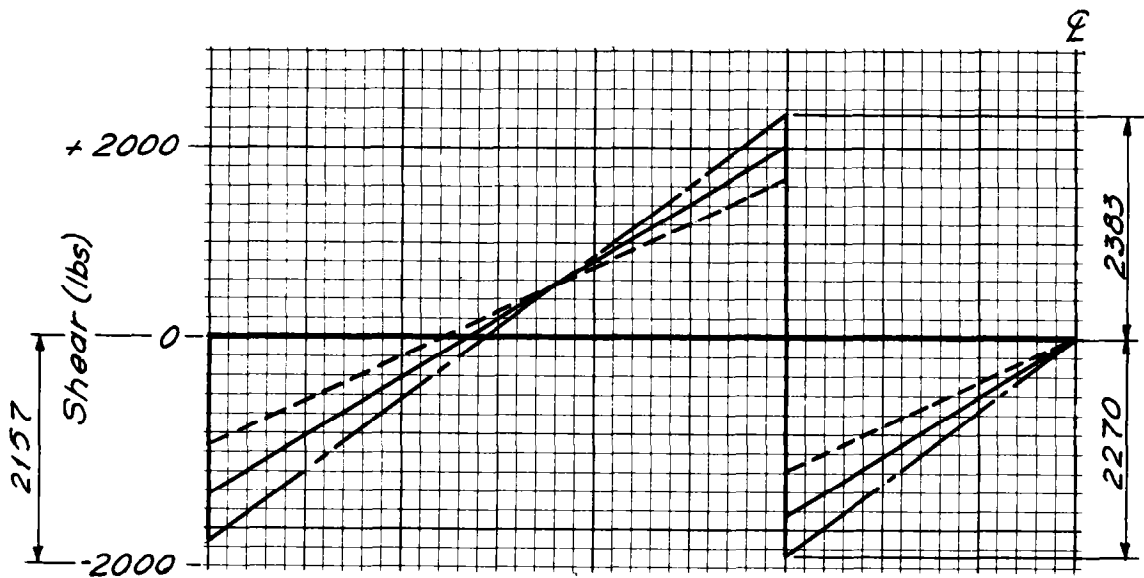
$$V_{ab} = -1830 + 196 = -1634 \text{ lbs}$$

APRON DESIGN (Section 1)

Before fill -----
 After fill - no flow -----
 After fill - with flow _____



MOMENT DIAGRAMS



SHEAR DIAGRAMS

$$V_{ba} = w\ell + V_{ab} = 3670 - 1634 = 2036 \text{ lbs}$$

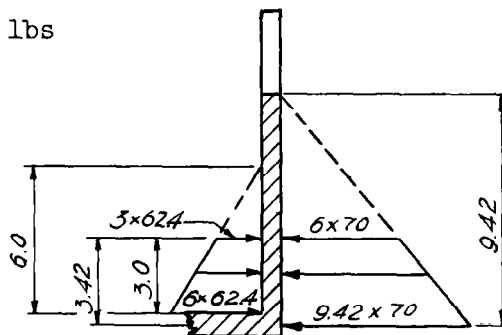
$$V_{bc} = -\frac{1}{2} w\ell = -1830 \text{ lbs}$$

$$R_a = 1634 \text{ lbs (down)}$$

$$R_b = 2036 + 1830 = 3866 \text{ lbs (down)}$$

Simple Moments

$$M_Q = \frac{1}{8} w\ell^2 = \frac{1}{8} w (10.28)^2 = 13.2 w$$



	$\frac{M_x}{M_Q}$	Before fill $w = 270$	No flow $w = 442$	With flow $w = 357$
at Q	1.0	3560	5830	4710
at 0.4ℓ	0.96	3420	5600	4520
at 0.3ℓ	0.84	2990	4900	3960
at 0.2ℓ	0.64	2280	3730	3020
at 0.1ℓ	0.36	1280	2100	1700

In determining the steel requirements, the horizontal compression thrust in the apron due to the load on the sidewall will be taken into account.
(See ES-45, EH, Sec. 6)

Magnitudes of Horizontal Compression Thrust (N)

Before fill, $N = 0$

$$\text{After fill--No flow, } N = \left(\frac{(6 \cdot 70) + (9.42 \cdot 70)}{2} \right) 3.42 = 1850 \text{ lbs}$$

$$\text{After fill--With flow, } N = 1850 - (4.5 \cdot 62.4 \cdot 3.0)$$

$$N = 1850 - 842 = 1008 \text{ lbs}$$

At Section 2

Before fill $\ell = 10.28 \text{ ft}$, $\ell^2 = 105.7 \text{ ft}^2$

Sidewall Moment, $M_{ab} = 0$

Load on apron = 137 lbs/ft^2

$$M_{ba} = \frac{1}{10} w\ell^2 = 13.7 \cdot 105.7 = 1450 \text{ ft lbs}$$

$$V_{ab} = -\frac{1}{2} w\ell + \frac{1}{\ell} (M_{ab} + M_{ba}) = -\frac{1}{2} \cdot 137 \cdot 10.28 + (1450 \div 10.28)$$

$$V_{ab} = -704 + 141 = -563 \text{ lbs}$$

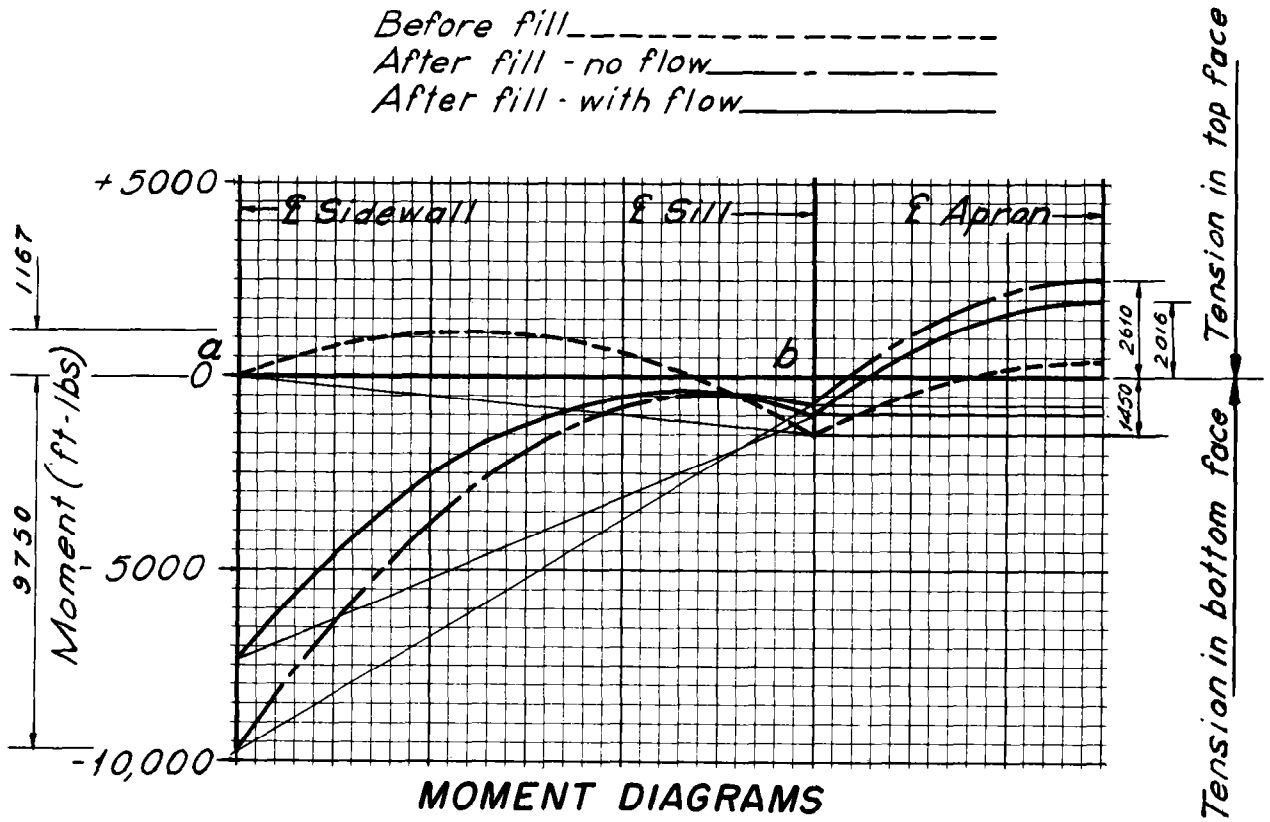
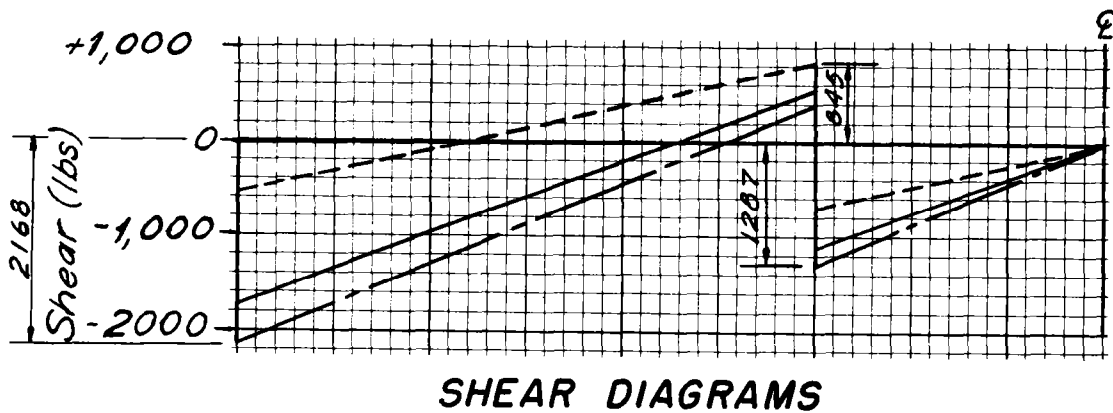
$$V_{ba} = w\ell + V_{ab} = 1408 - 563 = +845 \text{ lbs}$$

$$V_{bc} = -\frac{1}{2} w\ell = -704 \text{ lbs}$$

$$R_a = 563 \text{ lbs (down)}$$

$$R_b = 704 + 845 = 1549 \text{ lbs (down)}$$

APRON DESIGN (Section 2)



After fill--Now flow $w = 250 \text{ lbs/ft}^2$

$$M_{ab} = -\frac{wl^3}{6} = -\frac{70 (9.42)^3}{6} = -9750 \text{ ft lbs (at } Q \text{ apron)}$$

$$M_{ba} = \frac{1}{5} M_{ab} + \frac{1}{10} wl^2$$

$$M_{ba} = -1950 + (25 \cdot 105.7) = -1950 + 2640 = +690 \text{ ft lbs}$$

$$V_{ab} = -\frac{1}{2} wl + \frac{1}{l} (M_{ab} + M_{ba}) = (-125 \cdot 10.28) + \frac{1}{10.28} (-9750 + 690)$$

$$V_{ab} = -1287 - 881 = -2168 \text{ lbs}$$

$$V_{ba} = wl + V_{ab} = 2573 - 2168 = +405 \text{ lbs}$$

$$V_{bc} = -\frac{1}{2} wl = -1287 \text{ lbs}$$

$$R_a = 2168 \text{ lbs (down)}$$

$$R_b = 405 + 1287 = 1692 \text{ lbs (down)}$$

After fill-- With flow $w = 229 \text{ lbs/ft}^2$

$$M_{ab} = -9750 + \frac{62.4 (6)^2}{2} \cdot 2.42 = -9750 + 2720 = -7030 \text{ ft lbs}$$

$$M_{ba} = \frac{1}{5} (-7030) + \frac{1}{10} (229)(105.7) = -1406 + 2420 = +1014 \text{ ft lbs}$$

$$V_{ab} = -\frac{1}{2} wl + \frac{1}{l} (M_{ab} + M_{ba}) = -\frac{1}{2} (229)(10.28) + \frac{1}{10.28} (-7030 + 1014)$$

$$V_{ab} = -1177 - 585 = -1762 \text{ lbs}$$

$$V_{ba} = wl + V_{ab} = 2354 - 1762 = +592 \text{ lbs}$$

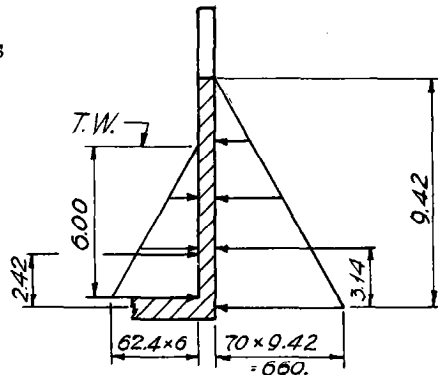
$$V_{bc} = -\frac{1}{2} wl = -1177 \text{ lbs}$$

$$R_a = 1762 \text{ lbs (down)}$$

$$R_b = 592 + 1177 = 1769 \text{ lbs (down)}$$

Simple Moments

$$M_Q = \frac{1}{8} wl^2 = 13.2 w$$



	$\frac{M_x}{M_Q}$	Before fill $w = 137$	No flow $w = 250$	With flow $w = 229$
at Q	1.0	1820	3300	3030
at $0.4l$	0.96	1747	3165	2910
at $0.3l$	0.84	1530	2770	2540
at $0.2l$	0.64	1166	2110	1940
at $0.1l$	0.36	655	1190	1090

Magnitude of Horizontal Compression Thrust (N)

Before fill, $N = 0$

$$\text{After fill--No flow, } N = \frac{70 \cdot (9.42)^2}{2} = 3100 \text{ lbs}$$

$$\text{After fill--With flow, } N = 3100 - \frac{62.4 (6)^2}{2} = 3100 - 1120$$

$$N = 1980 \text{ lbs}$$

At Section 3 $l = 10.28 \text{ ft}$, $l^2 = 105.7 \text{ ft}^2$

Before fill

$$\text{Sidewall Moment} = M_{ab} = 0$$

$$\text{Load on apron} = 13 \text{ lbs/ft}^2$$

$$M_{ba} = \frac{1}{10} w l^2 = 1.3 (105.7) = 138 \text{ ft lbs}$$

$$V_{ab} = -\frac{1}{2} w l + \frac{1}{l} (M_{ab} + M_{ba}) = -\frac{1}{2} (13)(10.28) + \frac{1}{10.28} (138)$$

$$V_{ab} = -67 + 13 = -54 \text{ lbs}$$

$$V_{ba} = w l + V_{ab} = 134 - 54 = +80 \text{ lbs}$$

$$V_{bc} = -\frac{1}{2} w l = -67 \text{ lbs}$$

$$R_a = 54 \text{ lbs (down)}$$

$$R_b = 80 + 67 = 147 \text{ lbs (down)}$$

After fill--No flow $w = 69 \text{ lbs/ft}^2$

$$M_{ab} = -9750 \text{ ft lbs (Same as Section 2)}$$

$$M_{ba} = \frac{1}{5} M_{ab} + \frac{1}{10} w l^2 = -1950 + 730 = -1220 \text{ ft lbs}$$

$$V_{ab} = -\frac{1}{2} w l + \frac{1}{l} (M_{ab} + M_{ba}) = -\frac{1}{2} (69)(10.28) + \frac{1}{10.28} (-9750 - 1220)$$

$$V_{ab} = -354 - 1068 = -1422 \text{ lbs}$$

$$V_{ba} = w l + V_{ab} = 708 - 1422 = -714 \text{ lbs}$$

$$V_{bc} = -\frac{1}{2} w l = -354 \text{ lbs}$$

$$R_a = 1422 \text{ lbs (down)}$$

$$R_b = 714 - 354 = 360 \text{ lbs (up)}$$

After fill--With flow $w = 109 \text{ lbs/ft}^2$

$$M_{ab} = -7030 \text{ ft lbs (Same as Section 2)}$$

$$M_{ba} = \frac{1}{5} M_{ab} + \frac{1}{10} w l^2 = -1406 + 1152 = -254 \text{ ft lbs}$$

$$V_{ab} = -\frac{1}{2} w l + \frac{1}{l} (M_{ab} + M_{ba}) = -\frac{1}{2} (109)(10.28) + \frac{1}{10.28} (-7030 - 254)$$

$$V_{ab} = -560 - 709 = -1269 \text{ lbs}$$

$$V_{ba} = w l + V_{ab} = 1120 - 1269 = -149 \text{ lbs}$$

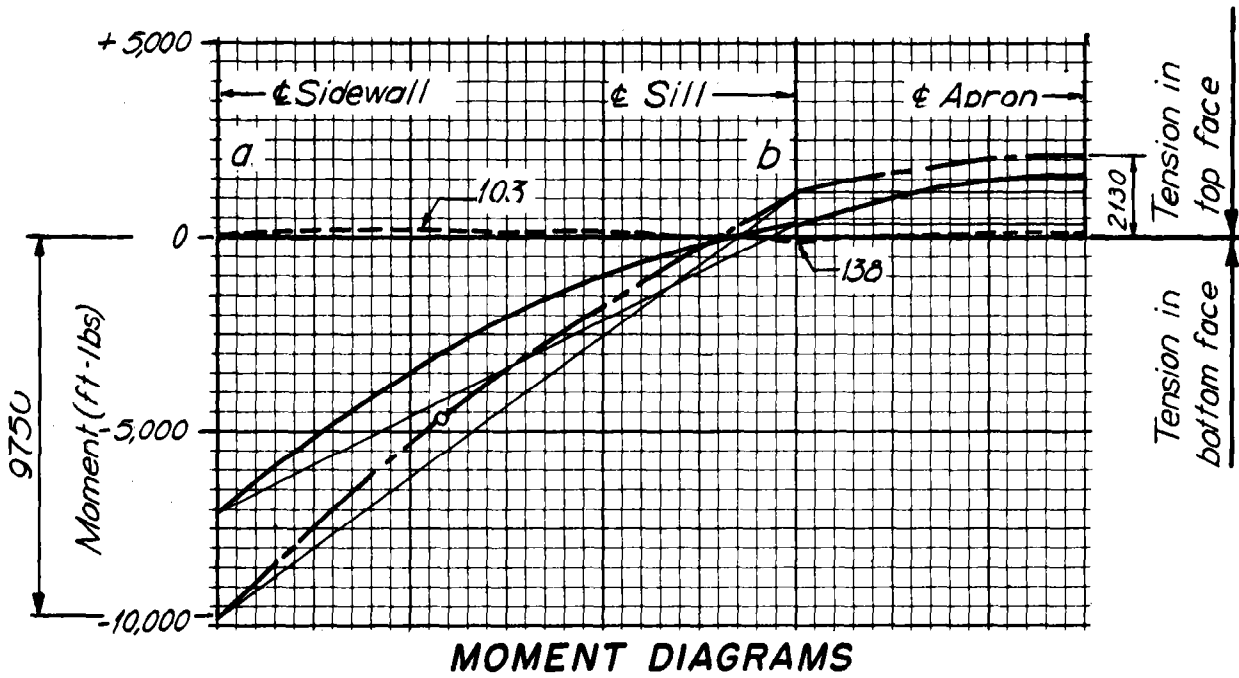
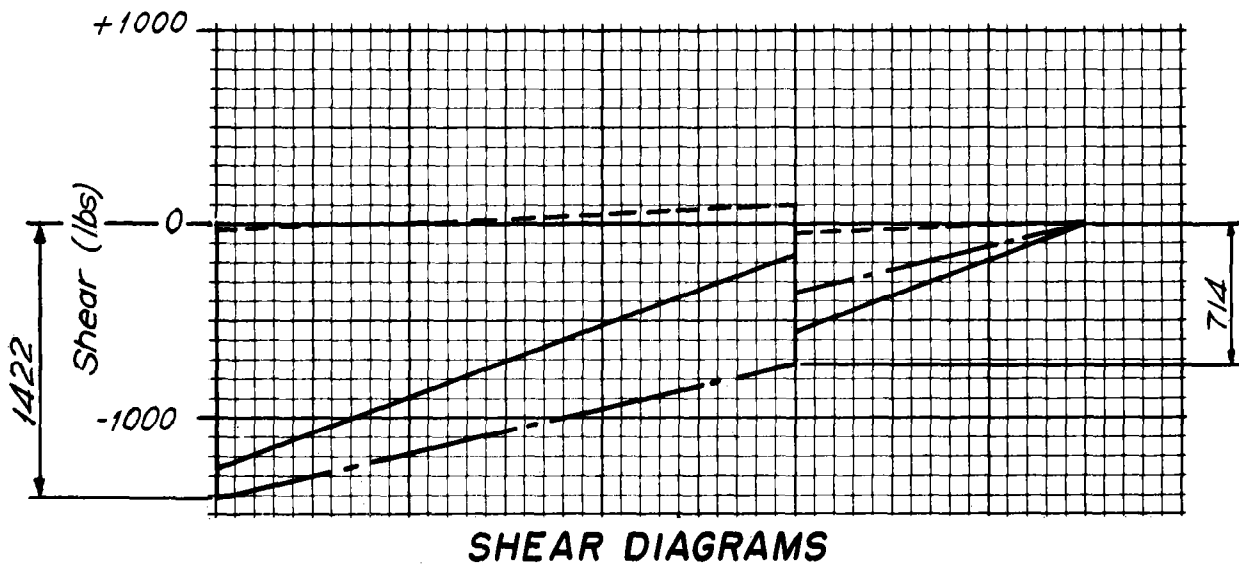
$$V_{bc} = -\frac{1}{2} w l = -560 \text{ lbs}$$

$$R_a = 1269 \text{ lbs (down)}$$

$$R_b = 560 - 149 = 411 \text{ lbs (down)}$$

APRON DESIGN (Section 3)

Before fill-----
 After fill-no flow-----
 After fill-with flow-----



Simple Moments

$$M_Q = \frac{1}{8} w l^2 = 13.2 w$$

	$\frac{M_x}{M_Q}$	Before fill $w = 13$	No flow $w = 69$	With flow $w = 109$
at Q	1.0	172	910	1440
at 0.4l	0.96	165	873	1380
at 0.3l	0.84	145	765	1210
at 0.2l	0.64	110	582	920
at 0.1l	0.36	62	328	518

Values of N are the same for Sections 2 and 3

Steel Requirements: Because of nature of design assumptions and to facilitate the placing of the steel during construction, the apron will be reinforced the same at all sections and will be designed to meet the maximum stress requirement in any section.

Bottom Steel (Transverse) $d = 11 - 3 \frac{1}{2} = 7 \frac{1}{2}$ in

At Sidewall

$M = 9750$ ft lbs (Sections 2 and 3, No flow governs)

$V = 2168$ lbs, $N = 3100$ lbs

$M_s = M + (Nd'' \div 12)$ (See ES-45, EH, Sec. 6) $d'' = 6 - 3.5 = 2.5$ in

$M_s = 9750 + (3100 \cdot 2.5 \div 12) = 9750 + 646 = 10,396$ ft lbs

$A = 0.94$ in² (See ES-45)

Req'd. $A_s = A - (N \div f_s) = 0.94 - (3.1 \div 20) = 0.94 - 0.16 = 0.78$ in²

Req'd. $\Sigma_o = 1.11$ in (See ES-44, EH, Sec. 6)

Use steel from sidewall--No. 7 at 9 ($A_s = 0.80$, $\Sigma_o = 3.67$)

At Sill

$M = 4085$ ft lbs--(Section 1, No flow governs)

$V = 2383$ lbs, $N = 1850$ lbs

$M_s = M + (Nd'' \div 12) = 4085 + (1850 \cdot 2.5 \div 12)$

$M_s = 4085 + 385 = 4470$ ft lbs

$A = 0.39$ in² (ES-45)

Req'd. $A_s = A - (N \div f_s) = 0.39 - (1.85 \div 20) = 0.39 - 0.09 = 0.30$ in²

Req'd. $\Sigma_o = 1.21$ in (ES-44)

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

Find where No. 5 at 12 ($A_s = 0.31$) can be used in end spans.

(Section 3, No flow governs) $N = 3100$ lbs

$A = A_s + (N \div f_s) = 0.31 + (3.1 \div 20) = 0.31 + 0.16 = 0.47$ in²

$M_s = 5330$ ft lbs (ES-45)

$M = M_s - (Nd'' \div 12) = 5330 - (3100 \cdot 2.5 \div 12)$

$M = 5330 - 662 = 4668$ ft lbs which corresponds to a point

$11.5/30 \cdot 10.28 = 3.94$ ft from the Q of sidewall

Length of horizontal legs of No. 7 bar

3.94 = distance from Q sidewall to theoretical cutoff point

$+0.25$ = distance from Q sidewall to back of bar

$+1.00$ = for contingencies

$5.19'$ total (Use $5'-3"$) (lap)

Length of No. 5 bars = $(31'-4") - 2 (5'-3") + 2 (1'-7") = 24'-0"$

Top Steel (Transverse) $d = 10 - 2.5 = 7.5$ in

End Span

$M_s = M = 2250$ ft lbs (Section 1, Before fill governs as $N = 0$)

$A_s = A = 0.192$ in²

$V = 1450$ lbs (Section 1, No flow governs)

$\Sigma_o < 1.0$ in

Center Span

$M = 2610$ ft lbs (Section 2, No flow governs) $N = 3100$

$M_s = M + (Nd'' + 12) = 2610 + 646 = 3256$ ft lbs

$A = 0.281$, $A_s = A - (N + f_s) = 0.281 - 0.15 = 0.13$ in²

Use No. 4 at 12 ($A_s = 0.20$, $\Sigma_o = 1.57$) Length = $31'-0"$

Longitudinal Steel

Bottom Face

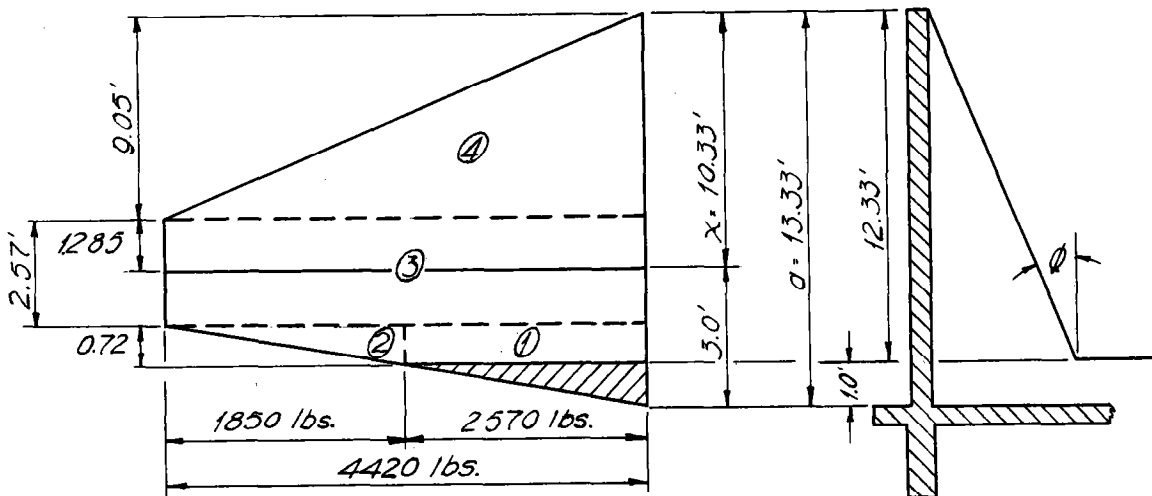
Temp. steel--Use No. 4 at 15, Length = $21'-0"$, horizontal leg = $20'-0"$, vertical leg = $1'-0"$ in downstream face of toewall

Top Face

Temp. steel--Use No. 5 at 12, Length = $20'-6"$, horizontal leg = $19'-6"$, vertical leg = $1'-0"$ in upstream face of toewall

Buttress Design: The buttress will be designed as a vertical cantilever beam for a load equal to the sum of the shears from the adjacent headwall slabs as explained on page 4.26.

Determine Load (See Headwall Design, page 6.16)



From Headwall Design

$$V_{\max} = 0.27 \cdot 8190 = 2210 \text{ lbs/ft (from one side only)}$$

$$\text{Max. load on buttress} = 2 \cdot 2210 = 4420 \text{ lbs/ft (both sides)}$$

$$x = 0.775 a = 0.775 \cdot 13.33 = 10.33 \text{ ft}$$

$$b/4 = 10.28 \div 4 = 2.57 \text{ ft, } b/8 = 1.285 \text{ ft}$$

$$\text{Load at top of sill elevation} = (1.0 + 1.72) 4420 = 2570 \text{ lbs}$$

Determine cantilever M and V at top of sill elevation

	Force	Arm	Moment
1	$0.72 \cdot 2570 = 1,850$	0.36	665
2	$0.72 \cdot 0.5 \cdot 1850 = 666$	0.48	319
3	$2.57 \cdot 4420 = 11,360$	2.0	22,720
4	$9.05 \cdot 0.5 \cdot 4420 = 20,000$	7.30	146,000
	33,876		169,704

$$V = 33,876 \text{ lbs}$$

$$M = 169,704 \text{ ft lbs}$$

Determine base width of buttress

$$\text{Shear } V = 33,876 \text{ lbs, } b = 12$$

$$v = V \div bjd \quad \therefore d = V \div bjv$$

For Class B Concrete

$$v = 90 \text{ lbs/in}^2, j = 0.875, k = 0.375, f_c = 1200 \text{ lbs/in}^2$$

$$\text{From ES-51, EH, Sec. 6, } d = 36 \text{ in}$$

$$\text{Moment } M = 169,704 \text{ ft lbs} = 2.37 \cdot 10^6 \text{ in lbs}$$

$$M_c = f_c b k j d^2 / 2$$

$$d = \sqrt{\frac{2M_c}{f_c b k j}} = \sqrt{\frac{2 \cdot 2.37 \cdot 10^6}{1.2 \cdot 10^3 \cdot 12 \cdot 0.375 \cdot 0.875}} = 29.4 \text{ in}$$

Shear governs

$$\text{Required base width} = 36 - 7.5 = 28.5 \text{ in}$$

A 6'-0" base width will be used for the following reasons: (1) to reduce the span length of the longitudinal sills, (2) to allow room for an aeration hole through the buttress at a higher elevation, (3) to reduce steel area requirement and thus facilitate placement of tension steel for buttress.

Steel RequirementsFor cantilever moment

$$A_s = M (\text{in ft kips}) \div ad, a = 1.44 \text{ (for Class B)}$$

$$d = (6 \cdot 12) + 7.5 = 79.5 \text{ in}$$

$$A_s = 169.7 \div (1.44 \cdot 79.5) = 1.48 \text{ in}^2$$

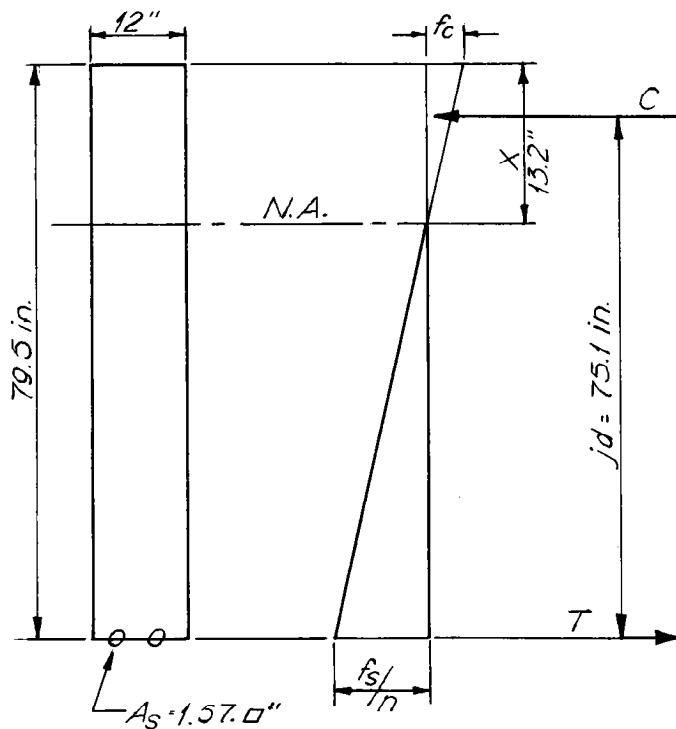
Use 2 No. 8 bars ($A_s = 1.57$) in unexposed face of headwall.

In both faces of buttress

Use Temp. steel requirement

No. 5 at 12 (both faces, both directions)

Compute compressive stresses in buttress



Locate neutral axis--Try $x = 10$

$$\begin{array}{rclcl}
 10 \cdot 12 & = & 120 & \cdot & 5 & = & 600 \\
 1.57 \cdot 10 & = & \underline{15.7} & \cdot & 79.5 & = & \underline{1250} \\
 & & 135.7 & & & &)1850(13.64 \\
 3.64 \cdot 12 & = & \underline{43.7} & \cdot & 11.82 & & \underline{516} \\
 & & 179.4 & & & &)2366(13.2 \\
 -0.44 \cdot 12 & = & \underline{-5.3} & \cdot & 13.42 & & \underline{-71} \\
 & & 174.1 & & & & 2295(13.2 = x
 \end{array}$$

$$jd = d - x/3 = 79.5 - 4.4 = 75.1$$

Compute vertical stresses

$$C = T = \frac{M}{jd} = \frac{2.37 \cdot 10^6}{75.1} = 31,600 \text{ lbs}$$

$$C = f_c/2 \cdot 13.2 \cdot 12 = 31,600$$

$$f_c = \frac{31,600 \cdot 2}{13.2 \cdot 12} = 399 \text{ psi}, \quad f_s = \frac{31,600}{1.57} = 20,100 \text{ psi}$$

Correct vertical compressive stress to stress parallel to face of buttress. $\tan \phi = 6 \div 12.33 = 0.487$; $\phi = 25^{\circ}58'$; $\cos \phi = 0.899$.
 Max. f_c parallel to face $= 399 \div \cos^2 \phi = 399 \div 0.81 = 493$ psi OK
 Allowable stress $= 0.225 f'_c (1.3 - 0.03 \frac{h}{d})$
 $h = (12.33^2 + 6^2)^{1/2} = 13.71$

Allowable stress $= 0.225 \cdot 3000 [1.3 - (0.03)(13.71 \div 1)] = 600$ psi
 (See Section 1107, ACI Code)

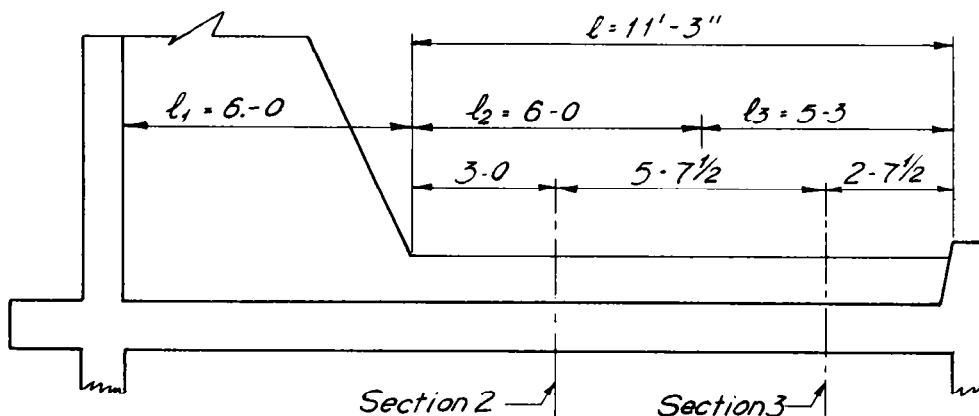
Longitudinal Sill Design: The longitudinal sills will be designed as beams considered fixed at the toe of the buttress and both fixed and simply supported at the transverse sill. The loads on the longitudinal sills will be taken as a uniform load, equal in magnitude and opposite in direction to the reactions at the sills as determined in the apron design at Section 2 and 3, plus or minus the weight of the longitudinal sill.

Reaction at Sill from Apron Design

	Section 2	Section 3
Before fill	1549 (down)	147 (down)
No flow	1692 (down)	360 (up)
With flow	1769 (down)	411 (down)

From inspection of the reactions, it is evident that both the no-flow and with-flow conditions must be considered because of the reversal of direction at Section 3.

Determination of Loads



Wt. of longitudinal sill/ft length $= 1.0 \cdot 1.0 \cdot 1.0 \cdot 150 = 150$ lbs

Loads at Section 2

No flow, load $= 1692 - 150 = 1542$ lbs/ft (up)

With flow, load $= 1769 - 150 = 1619$ lbs/ft (up)

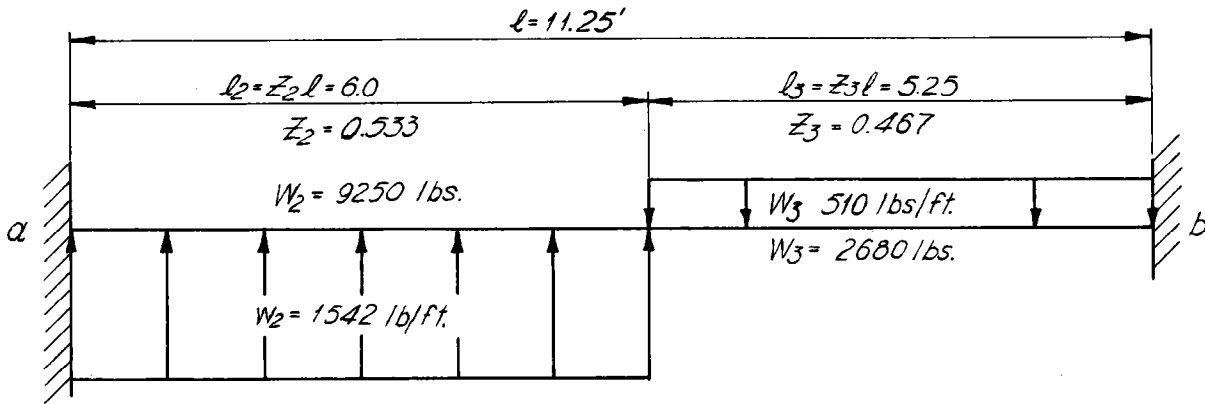
Loads at Section 3

No flow, load = $360 + 150 = 510 \text{ lbs/ft}$ (down)

With flow, load = $411 - 150 = 261 \text{ lbs/ft}$ (up)

Moments and Shears

No flow--both ends fixed



(See ES-32, EH, Sec. 6)

$$M_{ab}^F = -\frac{1}{12} l z_2 (6 - 8z_2 + 3z_2^2) w_2 + \frac{1}{12} l z_3^2 (4 - 3z_3) w_3$$

$$M_{ab}^F = -\frac{1}{12} (11.25)(0.533)(6 - 4.26 + 0.85)(9250) + \frac{1}{12} (11.25)(0.218)(2.60)(2680)$$

$$M_{ab}^F = -11,970 + 1425 = \underline{-10,545 \text{ ft lbs}}$$

$$M_{ba}^F = -\frac{1}{12} l z_3 (6 - 8z_3 + 3z_3^2) w_3 + \frac{1}{12} l z_2^2 (4 - 3z_2) w_2$$

$$M_{ba}^F = -\frac{1}{12} (11.25)(0.467)(2.91)(2680) + \frac{1}{12} (11.25)(0.284)(2.40)(9250)$$

$$M_{ba}^F = -3420 + 5920 = \underline{+2500 \text{ ft lbs}}$$

$$V_{ab}^S = R_a^S = \frac{1}{l} \left[w_3 z_3 l \cdot \frac{1}{2} - w_2 (z_3 l + z_2 l \cdot \frac{1}{2}) \right]$$

$$V_{ab}^S = \frac{1}{11.25} \left[(2680 \cdot 2.62) - (9250 \cdot 8.25) \right]$$

$$V_{ab}^S = \frac{1}{11.25} (7040 - 76,300) = \underline{-6150 \text{ lbs}}$$

$$V_{ba}^S = R_b^S = w_2 - w_3 - V_{ab}^S = 9250 - 2680 - 6150 = \underline{+420 \text{ lbs}}$$

$$V_{ab}^F = V_{ab}^S + \frac{1}{l} (M_{ab}^F + M_{ba}^F) = -6150 + \frac{1}{11.25} (-10,545 + 2500)$$

$$V_{ab}^F = -6150 - 715 = \underline{-6865 \text{ lbs}}$$

$$V_{ba}^F = V_{ba}^S + \frac{1}{l} (M_{ab}^F + M_{ba}^F) = + 420 - 715 = \underline{- 295 \text{ lbs}}$$

$$M_c^S = R_b^S z_3 l + \frac{1}{2} W_3 z_3 l = (420 \cdot 5.25) + (1340 \cdot 5.25) = \underline{9240 \text{ lbs}}$$

Simple Moments (See ES-1, EH, Sec. 6)

$$\text{At Section 2, } M_c = \frac{1}{8} W_2 z_2 l = + 6940 \text{ ft lbs}$$

$$\text{At Section 3, } M_c = \frac{1}{8} W_3 z_3 l = - 1760 \text{ ft lbs}$$

	M_x/M_c	for span $z_2 l$	for span $z_3 l$
at $0.5z l$	1.0	+6940	-1760
at $0.4z l$	0.96	+6660	-1690
at $0.3z l$	0.84	+5830	-1480
at $0.2z l$	0.64	+4440	-1127
at $0.1z l$	0.36	+2500	- 634

Plot moments and shears as explained in ES-32, EH, Sec. 6.

No flow--Simply supported at b

Satisfy this condition by correcting moments and shears when both ends are fixed.

With both ends fixed

$$M_{ab}^F = - 10,545 \text{ ft lbs, } M_{ba}^F = + 2500 \text{ ft lbs}$$

$$V_{ab}^F = - 6865 \text{ lbs, } V_{ba}^F = - 295 \text{ lbs}$$

Simply supported at b

$$M_{ba} = 0$$

$\therefore M_{ab} = M_{ab}^F + 0.5 (- M_{ba}^F)$, for 0.5 carryover factor and reason for minus sign, refer to any good reference on structural mechanics.

$$M_{ab} = - 10,545 - 1250 = - 11,795 \text{ ft lbs}$$

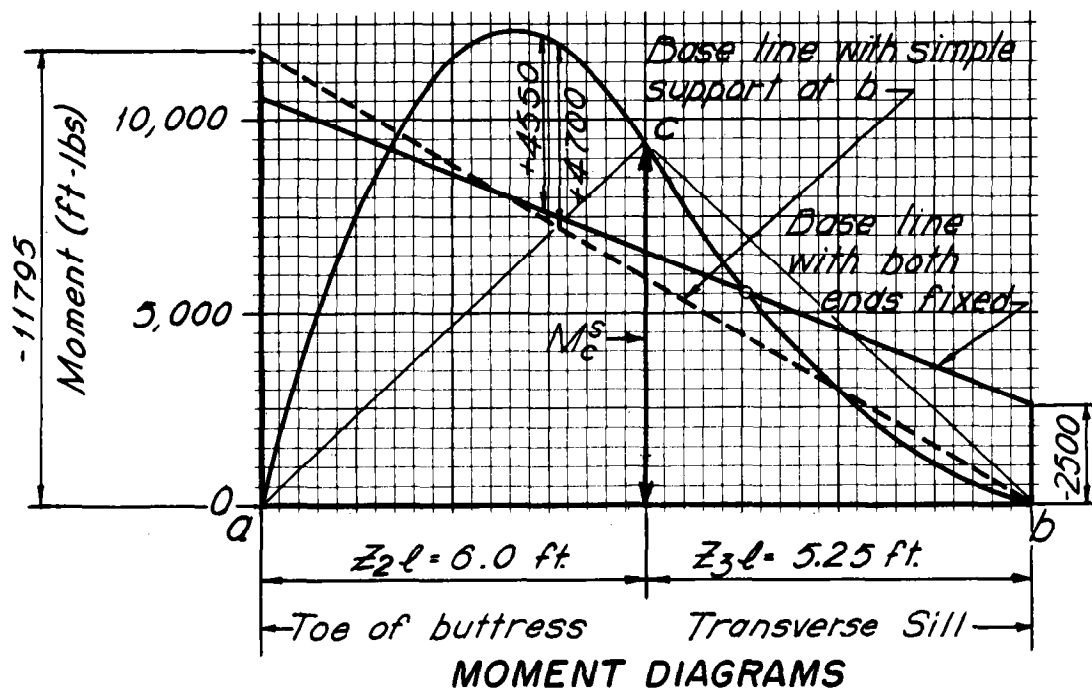
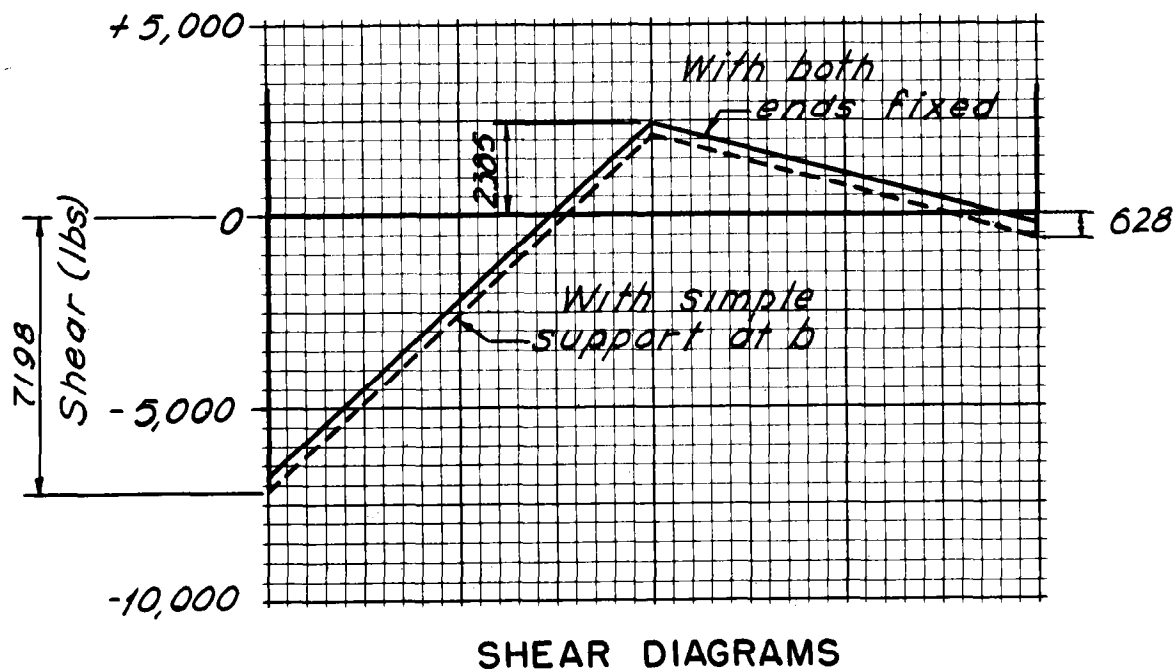
$$V_{ab} = V_{ab}^F + \frac{1}{l} (- 1.5 M_{ba}^F) = - 6865 + \frac{1}{11.25} (- 3750)$$

$$V_{ab} = - 6865 - 333 = \underline{- 7198 \text{ lbs}}$$

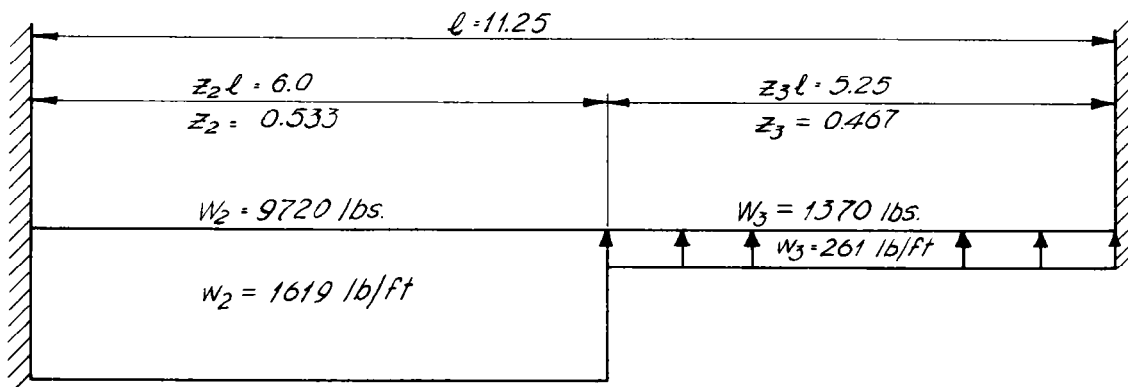
$$V_{ba} = V_{ba}^F - 333 = - 295 - 333 = - 628 \text{ lbs}$$

LONGITUDINAL SILL (No flow)

+ indicates tension in top face
- indicates tension in bottom face



With flow--both ends fixed



$$M_{ab}^F = -\frac{1}{12} l z_2 (6 - 8z_2 + 3z_2^2) w_2 - \frac{1}{12} l z_3^2 (4 - 3z_3) w_3$$

$$M_{ab}^F = -\frac{1}{12} (11.25)(0.533)(2.59)(9720) - \frac{1}{12} (11.25)(0.218)(2.60)(1370)$$

$$M_{ab}^F = -12,600 - 728 = \underline{-13,328 \text{ ft lbs}}$$

$$M_{ba}^F = +\frac{1}{12} l z_3 (6 - 8z_3 + 3z_3^2) w_3 + \frac{1}{12} l z_2^2 (4 - 3z_2) w_2$$

$$M_{ba}^F = +\frac{1}{12} (11.25)(0.467)(2.91)(1370) + \frac{1}{12} (11.25)(0.284)(2.40)(9720)$$

$$M_{ba}^F = 1746 + 6220 = \underline{+7966 \text{ ft lbs}}$$

$$V_{ab}^S = \frac{1}{l} \left[(-w_3 z_3 l \cdot \frac{1}{2}) - w_2 (z_3 l + \frac{1}{2} z_2 l) \right]$$

$$V_{ab}^S = \frac{1}{11.25} \left[(-1370 \cdot 2.62) - (9720 \cdot 8.25) \right]$$

$$V_{ab}^S = \frac{1}{11.25} (-3600 - 80,100) = \underline{-7440 \text{ lbs}}$$

$$V_{ba}^S = w_2 + w_3 - V_{ab}^S = 9720 + 1370 - 7440 = \underline{+3650 \text{ lbs}}$$

$$V_{ab}^F = V_{ab}^S + \frac{1}{l} (M_{ab}^F + M_{ba}^F) = -7440 + \frac{1}{11.25} (-13,328 + 7966)$$

$$V_{ab}^F = -7440 - 476 = \underline{-7916 \text{ lbs}}$$

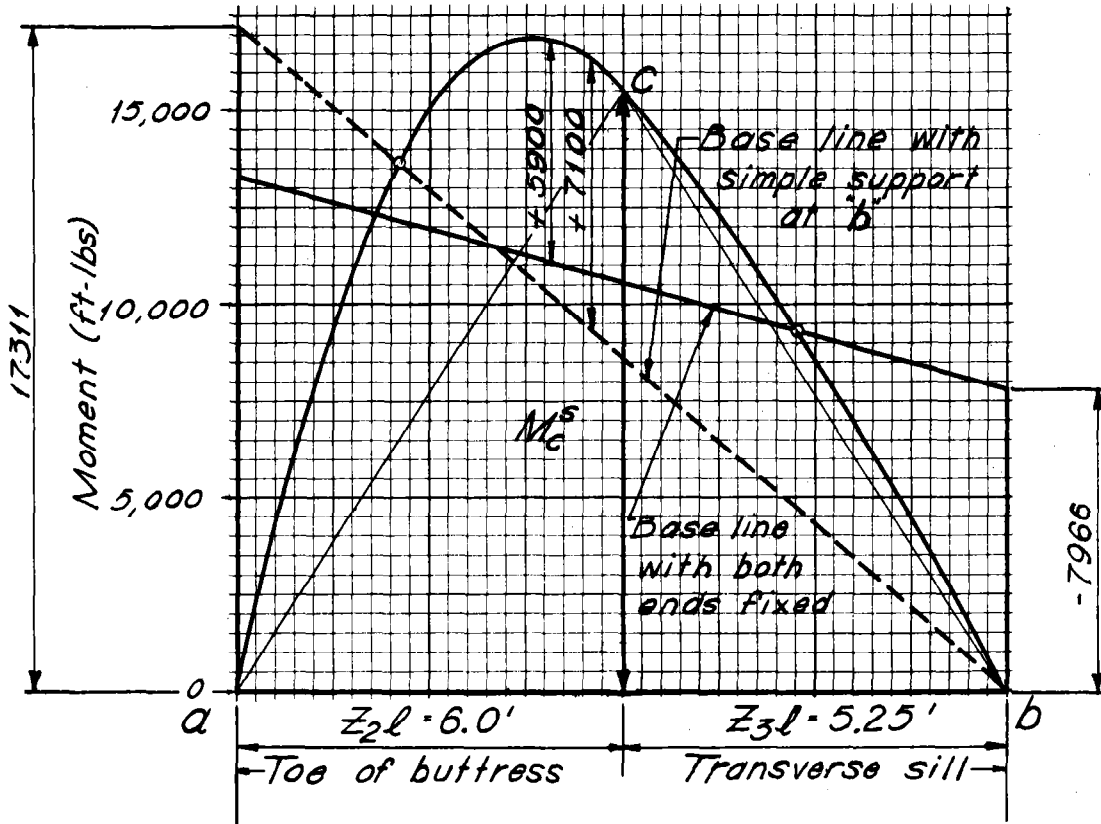
$$V_{ba}^F = V_{ba}^S - 480 = 3650 - 476 = \underline{+3174 \text{ lbs}}$$

$$M_c^S = V_{ba}^S z_3 l - \frac{1}{2} w_3 z_3 l = (3650 \cdot 5.25) - (\frac{1}{2} \cdot 1370 \cdot 5.25)$$

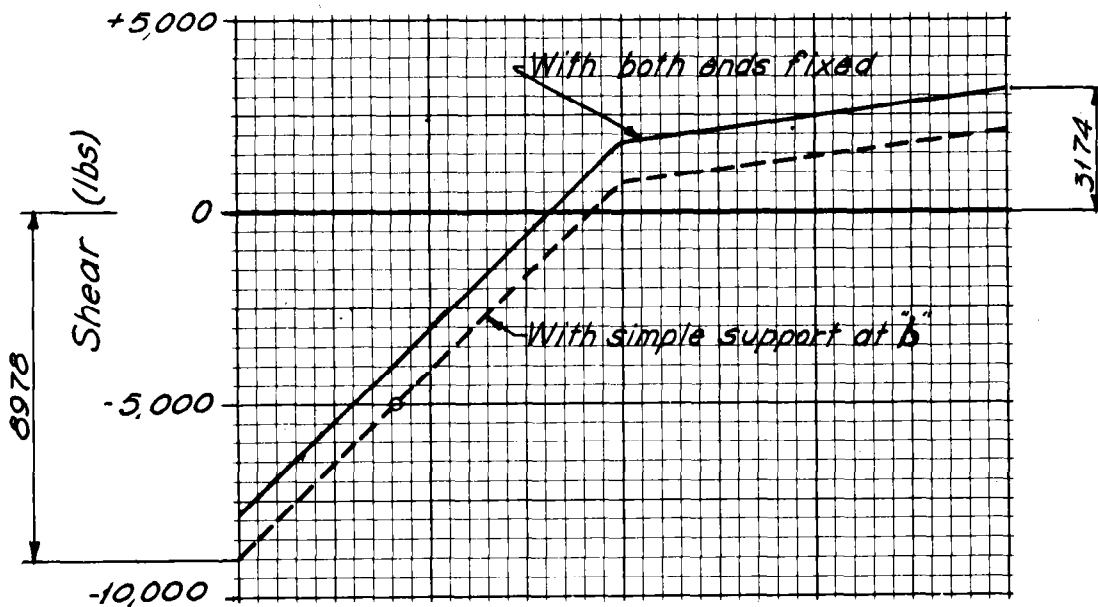
$$M_c^S = 19,160 - 3600 = \underline{15,560 \text{ ft lbs}}$$

LONGITUDINAL SILL (With flow)

+ indicates tension in top face
- indicates tension in bottom face



MOMENT DIAGRAMS



SHEAR DIAGRAMS

Simple Moments

$$\text{At Section 2, } M_Q = \frac{1}{8} W_2 z_2 \ell = 7290 \text{ ft lbs}$$

$$\text{At Section 3, } M_Q = \frac{1}{8} W_3 z_3 \ell = 900 \text{ ft lbs}$$

	M_x/M_Q	At Section 2	At Section 3
at $0.5z\ell$	1.0	7290	900
at $0.4z\ell$	0.96	6990	863
at $0.3z\ell$	0.84	6120	755
at $0.2z\ell$	0.64	4660	576
at $0.1z\ell$	0.36	2620	324

With flow--Simply supported at b

Correcting fixed end moments and shears

$$M_{ba} = 0$$

$$M_{ab} = M_{ab}^F + 0.5 (-M_{ba}^F) = -13,328 - 3983 = \underline{-17,311 \text{ ft lbs}}$$

$$V_{ab} = V_{ab}^F + \frac{1}{\ell} (-1.5 M_{ba}^F) = -7916 + \frac{1}{11.25} (-1.5 \cdot 7966)$$

$$V_{ab} = -7916 - 1062 = -8978 \text{ lbs}$$

$$V_{ba} = V_{ba}^F - 1062 = 3174 - 1062 = \underline{+2112 \text{ lbs}}$$

Steel in bottom face $d = 11 + 12 - 3 - \frac{3}{8} - \frac{3}{4} = 18 \frac{7}{8} \text{ in}$ (Use 19 in)

At Buttress

$$M_{ab} = -17,311, V_{ab} = 8978 \text{ (With flow, simple support at b)}$$

$$\text{Req'd. } A_s = 0.58 \text{ in}^2 \text{ (ES-45, EH, Sec. 6) Req'd. } \Sigma_o = 1.79 \text{ in (ES-44)}$$

$$\text{Use 1 No. 7 bar } (A_s = 0.60, \Sigma_o = 2.75)$$

$$\text{Extend bar } \frac{8.3}{20} (6.0) + 1.0 = 3.49 \text{ ft Say 3'-6" into sill and 2'-0" under buttress. Length} = 5'-6".$$

At Sill

$$M_{ba}^F = 7966, V_{ab}^F = 3174, \text{ With flow, both ends fixed.}$$

$$\text{Req'd. } A_s = 0.273 \text{ in}^2, \text{ Req'd. } \Sigma_o < 1.0 \text{ in}$$

$$\text{Use 1 No. 5 bar } (A_s = 0.31, \Sigma_o = 1.96) \text{ Length of horizontal}$$

$$\text{leg} = \left(\frac{15}{20} \cdot 5.25\right) + 1.0 + 0.583 \text{ ft (No flow, both ends fixed}$$

$$\text{governs) Use 5'-6" Vertical leg} = 1'-0" \text{ in downstream face of toewall. Total length} = 6'-6".$$

Steel in top face Use $d = 19$ in

$M = 7100$, $V = 5000$ (With flow, simple support at b governs)

Req'd. $A_s = 0.235 \text{ in}^2$, Req'd. $\Sigma_o = 1.45$ in ($u = 210 \text{ lbs/in}^2$)

Use 1 No. 5 bar ($A_s = 0.31$, $\Sigma_o = 1.96$) Start bar 2 in. from downstream edge of transverse sill and extend it about 1'-0" under buttress. Length = $(0'-7") + (11'-3") + (1'-0") = 12'-10"$

Use 13'-0". Use L bars No. 4 at 18 for tie bars. Both legs 1'-6", Length = 3'-0"

Transverse Sill Design: The transverse sill, together with the toewall, will be designed as a rectangular beam with its thickness equal to that of the toewall. The design moment at the ends of the beam will be determined, considering the beam as fixed at both ends. The design moment at mid-span will be determined, considering the beam to be partially restrained at the ends and assuming the end moments to be equal to one-half the fixed end moments. The loads will be the total base pressure minus the weight of water and/or concrete and the reactions from the longitudinal sills.

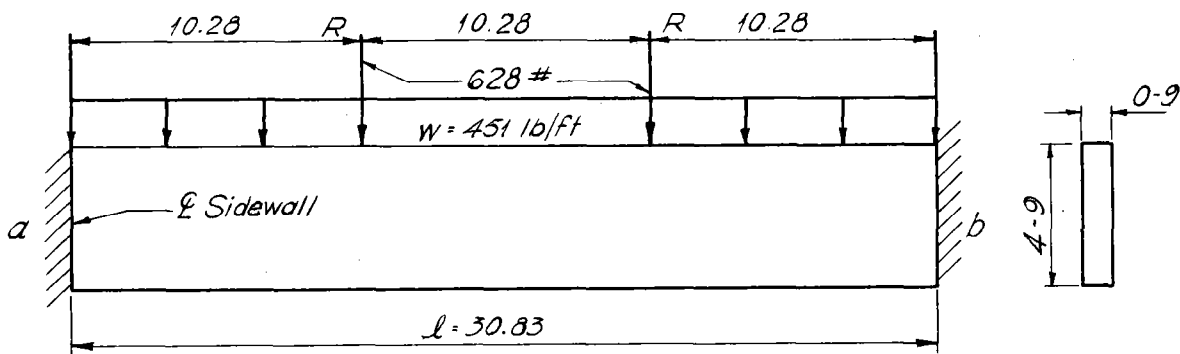
After fill--No flow

Total base pressures/ft length = $0.75 (99 + 123) \div 2 = 83 \text{ lbs/ft (up)}$
(See apron design)

Wt. of concrete = $4.75 \cdot 0.75 \cdot 1.0 \cdot 150 = 534 \text{ lbs/ft (down)}$

Uniform load per ft = $534 - 83 = 451 \text{ lbs/ft (down)}$

Concentrated load = R_b from longitudinal sill = $R = 628 \text{ lbs (down)}$



Moments and Shears

$$V_{ab} = -V_{ba} = \frac{wl}{2} + R = (451)(30.83) \frac{1}{2} + 628 = 6950 + 628$$

$$V_{ab} = 7578 \text{ lbs}$$

$$M_{ab} = -M_{ba} = \frac{1}{12} wl^2 + \frac{2}{9} Rl = \frac{1}{12} (451)(30.83)^2 + \frac{2}{9} (628)(30.83)$$

$$M_{ab} = -M_{ba} = 35,700 + 4300 = 40,000 \text{ ft lbs}$$

Simple Moments

From Concentrated load at point of load

$$M = 628 \cdot 10.28 = 6460 \text{ ft lbs}$$

From Uniform Load

$$M_C = \frac{1}{8} w l^2 = \frac{1}{8} \cdot 451 (30.83)^2 = 53,600 \text{ ft lbs}$$

$$\text{at } 0.4l, M = 0.96 \cdot 53,600 = 51,400 \text{ ft lbs}$$

$$0.3l, M = 0.84 \cdot 53,600 = 45,000 \text{ ft lbs}$$

$$0.2l, M = 0.64 \cdot 53,600 = 34,300 \text{ ft lbs}$$

$$0.1l, M = 0.36 \cdot 53,600 = 19,300 \text{ ft lbs}$$

After fill--With flow

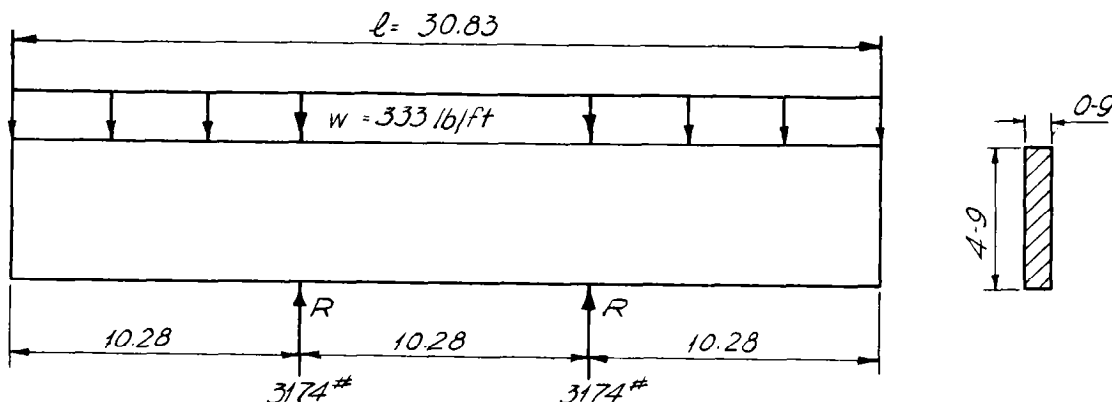
$$\text{Total base pressure} = 0.75 (551 + 567) \frac{1}{2} = 419 \text{ lbs/ft (up)}$$

$$\text{Wt. of concrete} = 534 \text{ lbs (down)}$$

$$\text{Wt. of H}_2\text{O on sill} = 62.4 \cdot 4.67 \cdot 0.75 = 218 \text{ lbs (down)}$$

$$\text{Uniform load/ft} = 534 + 218 - 419 = 333 \text{ lbs/ft (down)}$$

$$\text{Concentrated load} = R_b = R = 3174 \text{ lbs (up)}$$

Moments and Shears

$$V_{ab} = -V_{ba} = \frac{w l}{2} - R = (333 \cdot 15.41) - 3174 = 1966 \text{ lbs}$$

$$M_{ab} = -M_{ba} = \frac{1}{12} w l^2 - \frac{2}{9} R l = \left[\frac{1}{12} \cdot 333 \cdot (30.83)^2 \right] - \left[\frac{2}{9} (3176) (30.83) \right]$$

$$M_{ab} = -M_{ba} = 26,400 - 21,800 = 4,600 \text{ ft lbs}$$

Simple MomentsFrom Concentrated load at point of load

$$M = 3176 \cdot 10.28 = 32,600 \text{ ft lbs}$$

From Uniform load

$$M_C = \frac{1}{8} w l^2 = \frac{1}{8} \cdot 333 (30.83)^2 = 39,600 \text{ ft lbs}$$

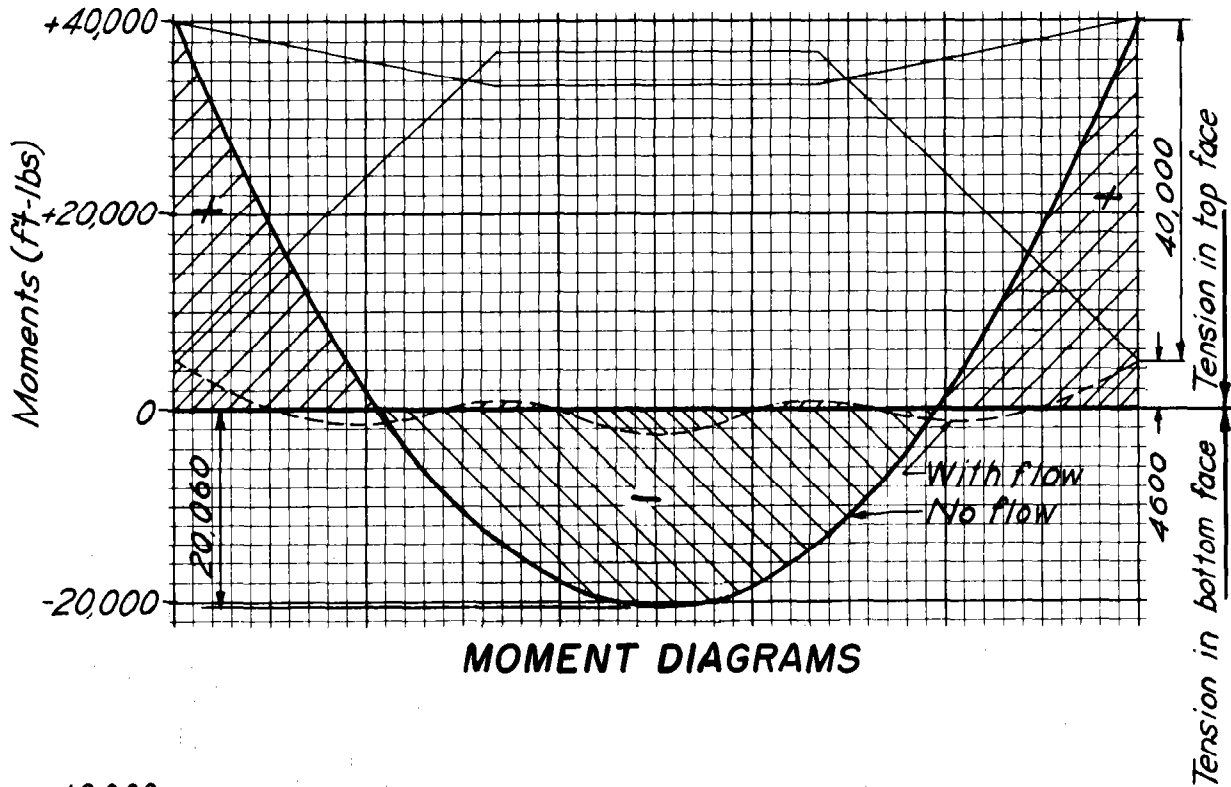
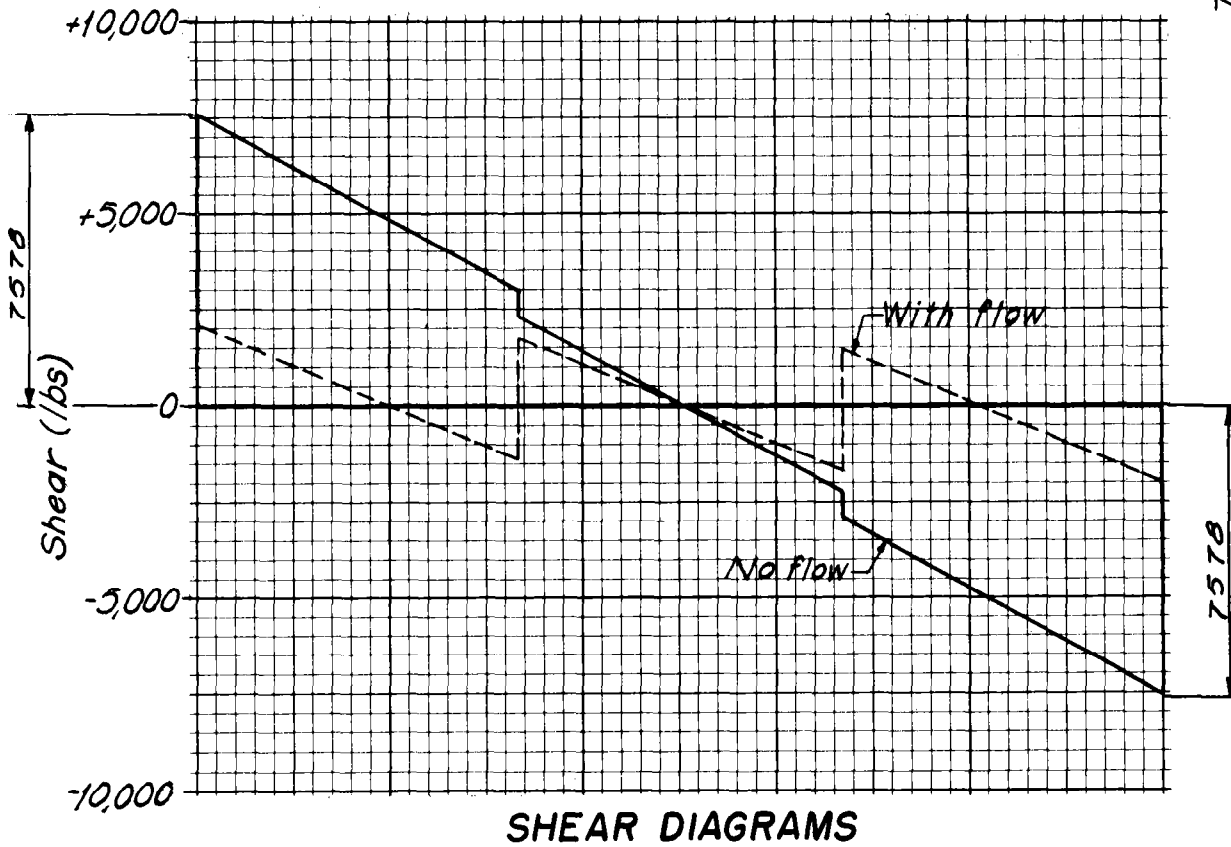
$$\text{at } 0.4l, M = 0.96 \cdot 39,600 = 38,000 \text{ ft lbs}$$

$$0.3l, M = 0.84 \cdot 39,600 = 33,300 \text{ ft lbs}$$

$$0.2l, M = 0.64 \cdot 39,600 = 25,300 \text{ ft lbs}$$

$$0.1l, M = 0.36 \cdot 39,600 = 14,260 \text{ ft lbs}$$

TRANSVERSE SILL



Steel in top face $d = 57" - 4" = 53$ in

$M = 40,000$ ft lbs, $V = 7578$ lbs (No flow governs)

Req'd. $A_s = M \div ad = 40.0 \div (1.44 \cdot 53) = 0.524$ in²

Req'd. $\Sigma O < 1.0$ in. Use 2 No. 5 bars ($A_s = 0.61$, $\Sigma O = 3.93$)

Find where 2 No. 3 bars ($A_s = 0.22$) can be used

$M = ad A_s = 1.44 \cdot 53 \cdot 0.22 = 16.8$ ft kips which corresponds to a point $5.7/50 \cdot 30.83 = 3.52$ ft from $\bar{C}$ of sidewall.

Length of horizontal leg of No. 5 bars $= 3.52 + 0.25 + 1.0 = 4.77$ ft

Use 4'-9". Make vertical leg 2'-0". Total length = 6'-9"

Length of No. 3 bars $= (30'-0") + 2 (0'-8") + 2 (1'-0") - 2 (4'-9")$
 $= 23'-10"$, Use 24'-0" with 1'-1" lap

Steel in bottom face Increase moment 50 percent of fixed end moment

Moment $= 20.06 + (0.5 \cdot 40) = 40.06$ ft kips; shear $= 4600$ lbs (max)

$A_s = M \div ad = 40.06 \div (1.44 \cdot 53) = 0.532$ sq in

$\Sigma O = 8 V \div 7 u d = (8 \cdot 4600) \div (7 \cdot 300 \cdot 53) = 36,800 \div 111,300$
 $= 0.33$ in

Use 2 No. 5 bars ($A_s = 0.61$, $\Sigma O = 3.93$). Extend bars 0'-6" into sidewall on both ends then length $= 31'-0"$

Other Steel

Vertical Steel

Use No. 3 at 12 in downstream face (*Apron bar*)

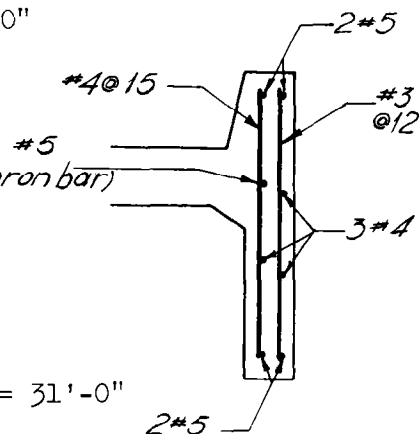
Use No. 4 at 15 in upstream face

Length $= (4'-9") - 2 (0'-3") = 4'-3"$

Horizontal Steel

Use No. 4 in both faces

Length $= (30'-0") + (0'-6") + (0'-6") = 31'-0"$



Wingwall Design: The wingwall will be designed independently of the rest of the structure and as a series of cantilever beams. The wingwall makes an angle of 45° with the sidewall.

Length of Wingwall $= (J - s - 1.0) 2.83$

Length $= (9.0 - 1.33 - 1.0) 2.83 = 18.85$ ft Use 19'-0"

Required width of footing to prevent overturning

Fill Characteristics

Moist wt. $= W = 110$ lbs/ft³

Angle of internal friction $= \phi = 25^\circ$

Friction angle of soil $= Z = \phi = 25^\circ$

Angle between horizontal and vertical wall $= \theta = 90^\circ$

Angle between horizontal and fill slope $= i = 19^\circ-27'$

Try 6.0 ft width of footing at upstream end

See equation 6.2-1 (EH, Sec. 6)

$$P = \frac{wH^2}{2} \frac{\sin^2 (\theta - \phi)}{\sin^2 \theta \sin (\theta + Z) \left[1 + \sqrt{\frac{\sin (Z + \phi) \sin (\phi - i)}{\sin (\theta + Z) \sin (\theta - i)}} \right]^2}$$

$$P = \frac{wH^2}{2} \frac{\sin^2 65^\circ}{\sin^2 90^\circ \cos 25^\circ \left[1 + \sqrt{\frac{\sin 50^\circ \sin 5^\circ - 33'}{\cos 25^\circ \sin 70^\circ - 33'}} \right]^2}$$

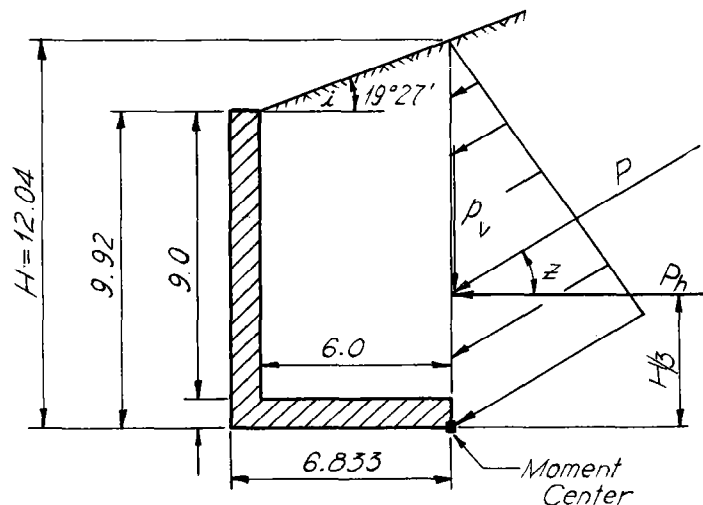
$$P = \frac{wH^2}{2} \frac{0.821}{(1.0)(0.906) \left[1 + \sqrt{\frac{0.766 \cdot 0.0967}{0.906 \cdot 0.943}} \right]^2} = \frac{wH^2}{2} (0.54)$$

$$P = 55 \cdot (12.04)^2 \cdot 0.54 = 4300 \text{ lbs}$$

$$P_h = P \cos Z = 4300 \cdot 0.906 = 3900 \text{ lbs}$$

$$P_v = P \sin Z = 4300 \cdot 0.423 = 1850 \text{ lbs}$$

$$H = 9.92 + 6 \tan i = 9.92 + 2.12 = 12.04, \quad H/3 = 4.01$$



Taking Moments

Part	Weight	Force	Arm	Moment
Footing $6.0 \cdot 0.92 \cdot 150 =$	828		3.0	2,480
Wall $9.92 \cdot 0.83 \cdot 150 =$	1,230		6.42	7,900
Earth $\square 6.0 \cdot 9.0 \cdot 110 =$	5,940		3.0	17,820
Earth $\triangle 3.0 \cdot 2.12 \cdot 110 =$	700		2.0	1,400
P_v	1,850		0.0	0
P_h		3,900	4.01	15,640
	10,548			45,240

$z = 45,240 \div 10,548 = 4.29 \text{ ft}$ ∴ stable from overturning as resultant falls within the middle third of base ($6.83 \cdot 2/3 = 4.55 \text{ ft}$)

Try 1.0 ft width of footing at downstream end and check overturning at midsection.

Footing at midsection = $(6.0 + 1.0) \div 2 = 3.5$ ft wide

Base width at midsection = $3.5 + 0.83 = 4.33$ ft

$H = [(9.0 + 2.33) \div 2] + 0.92 + 3.5 \tan i$

$H = 5.67 + 0.917 + 1.23 = 7.82$ ft

Ht. of wall above footing = 5.67 ft

$P = (wH^2 \div 2) 0.54 = [(110 \cdot 7.82^2) \div 2] 0.54 = 1810$ lbs

$P_h = P \cos Z = 1810 \cdot 0.906 = 1640$ lbs

$P_v = P \sin Z = 1810 \cdot 0.423 = 765$ lbs

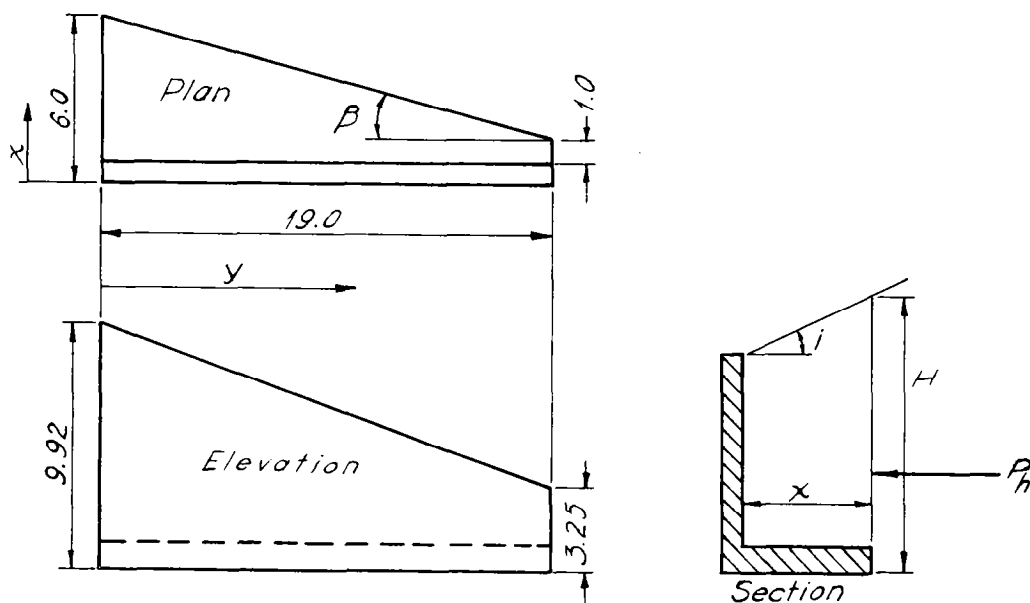
Taking Moments

Part	Weight	Force	Arm	Moment
Footing $3.5 \cdot 0.92 \cdot 150$	482		1.75	844
Wall $6.58 \cdot 0.833 \cdot 150$	822		3.92	3,220
Earth $\square 3.5 \cdot 5.67 \cdot 110$	2,180		1.75	3,820
Earth $\triangle 1.75 \cdot 1.23 \cdot 110$	238		0.412	98
P_v	765		0.0	0
P_h		1,640	2.61	4,280
	4,487		2.73	12,262

$4.33 \cdot \frac{2}{3} = 2.88 > 2.73$; hence, OK safe from overturning

Sliding

To prevent the wingwall from sliding, the wingwall footing will be tied into the apron and reinforced to carry the horizontal moment set up by the earth pressures on the wingwall, tending to rotate the wingwall about its junction with the sidewall.



Determine Horizontal Moment

From overturning calculations

$$P_h = P \cos Z = 0.906 P$$

$$P = (wH^2 \div 2) 0.54$$

$$\therefore P_h = 0.906 (110 H^2 \div 2) 0.54 = 26.92 H^2$$

$$H = 9.92 + x \cos i - \left(\frac{9.92 - 3.25}{19.0} \right) y = 9.92 + 0.353 x - 0.351 y$$

$$x = 6 - \left(\frac{6.0 - 1.0}{19} \right) y = 6 - 0.263 y$$

$$H = 9.92 + 2.12 - 0.093 y - 0.351 y = 12.04 - 0.444 y$$

$$\therefore P_h = 26.92 (12.04 - 0.444 y)^2$$

$$P_h = 3902 - 287.8 y + 5.31 y^2$$

$$\text{Total } P_h = P_{ht} = P_h dy = \int_0^{19} (3902 - 287.8 y + 5.31 y^2) dy$$

$$M = P_h y dy = \int_0^{19} (3902 - 287.8 y + 5.31 y^2) y dy$$

$$M = \left[\frac{3902 y^2}{2} - \frac{287.8 y^3}{3} + \frac{5.31 y^4}{4} \right]_0^{19}$$

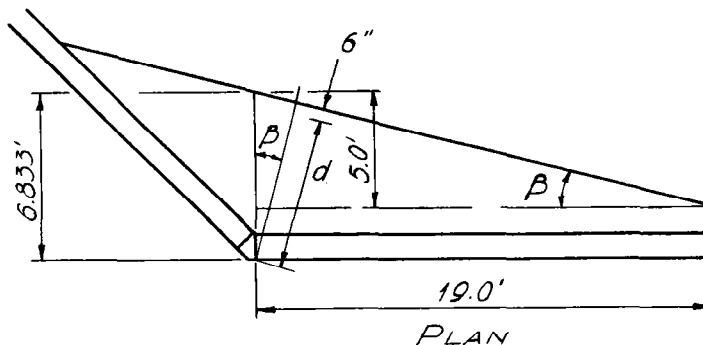
$$M = 704,300 - 662,600 + 173,000 = 214,700 \text{ ft lbs}$$

Determine effective depth

$$\tan \beta = 5 \div 19 = 0.263 \quad \therefore \beta = 14^\circ - 45'$$

$$\cos \beta = 0.967$$

$$d = (12 \div 6.83 \cos \beta) - 6 = 70.6 \text{ in} \quad (\text{Use } 70.0 \text{ in})$$



$$\text{Req'd. steel } (A_s = M (\text{ft kip}) \div ad)$$

$$A_s = 214.7 \div (1.44 \cdot 70) = 2.13 \text{ in}^2$$

Use 2 No. 10 bars ($A_s = 2.54$) place one bar 4 in. from edge and the other 10 in. from edge.

Design of Vertical Wall

Take Z , angle of friction between earth and concrete = 0
(See equation 6.2-3, EH, Sec. 6)

$$P = \frac{wH^2}{2} \frac{\cos^2 \phi}{\left(1 + \sqrt{\frac{\sin \phi \sin (\phi - i)}{\sin (\theta - i)}}\right)^2} = \frac{wH^2}{2} \frac{\cos^2 25^\circ}{\left(1 + \sqrt{\frac{\sin 25^\circ \sin 5^\circ - 33'}{\sin 70^\circ - 33'}}\right)^2}$$

$$H = 9.0 \text{ ft}$$

$$P = \frac{110 (9)^2}{2} \frac{0.821}{\left(1 + \sqrt{\frac{0.423 \cdot 0.0967}{0.943}}\right)^2} = 4455 \cdot 0.562 = 2510 \text{ lbs}$$

$$M = P \frac{H}{3} = 2510 \cdot 3 = 7530 \text{ ft lbs, } V = P = 2510 \text{ lbs}$$

Assuming the same equivalent fluid pressure, w , at all points along the wall, build up M and A_s curves ($d = 7 \frac{1}{2}$ in)

$$P = wH^2 \div 2 \quad \therefore w = 2P \div H^2$$

$$w = 2 \cdot 2510 \div 81 = 62.0 \text{ lbs/ft}^3$$

$$M = wH^3 \div 6 = 10.33 H^3$$

H	H ³	M	A _s
9.0	729	7530	0.67
8.0	512	5300	0.47
7.0	343	3550	0.31
6.0	216	2240	0.193
5.0	125	1290	0.108
4.0	64	662	<0.1
3.0	27	279	<0.1

$$d = 7.5 \text{ in}$$

Vertical Steel (Unexposed face)At Upstream End

$$M = 7530 \text{ ft lbs, } V = 2510 \text{ lbs}$$

$$\text{Req'd. } A_s = 0.67 \text{ in}^2, \text{ Req'd. } \Sigma O = 1.29 \text{ in (M will govern)}$$

$$\text{Use No. 6 at 8 (} A_s = 0.66, \Sigma O = 3.53 \text{ in)}$$

$$\text{Find where No. 4 at 8 (} A_s = 0.29 \text{) can be used. From } A_s \text{ curve}$$

$$H = 6.9 \text{ ft, } \therefore \text{Length of No. 6 bar} = 9.0 - 6.9 + 3.42 - 0.25 + 1.0 = 6.27; \text{ Use 6'-3" and place 3" above bottom of toewall.}$$

Find point along wall where No. 6 at 12 ($A_s = 0.44$) can be used

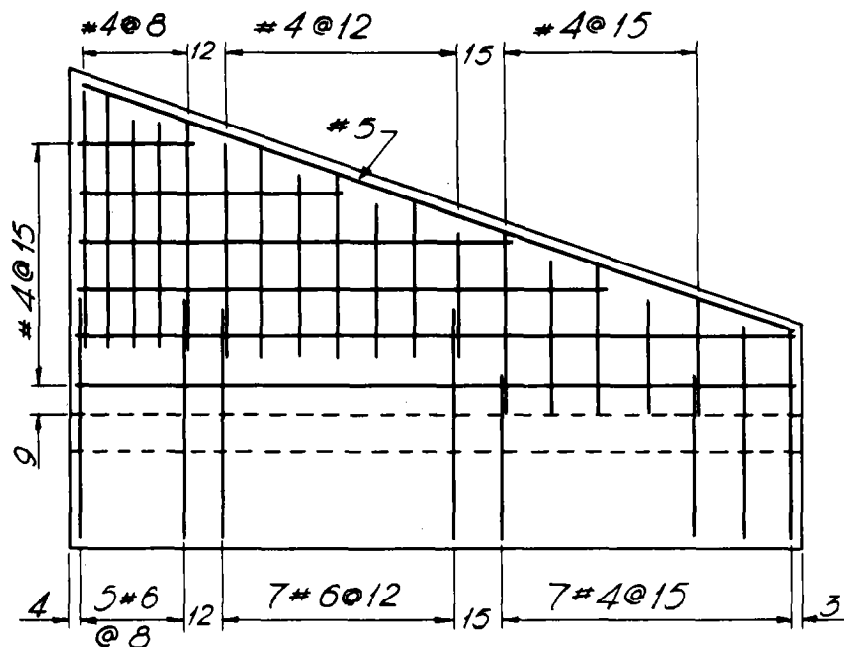
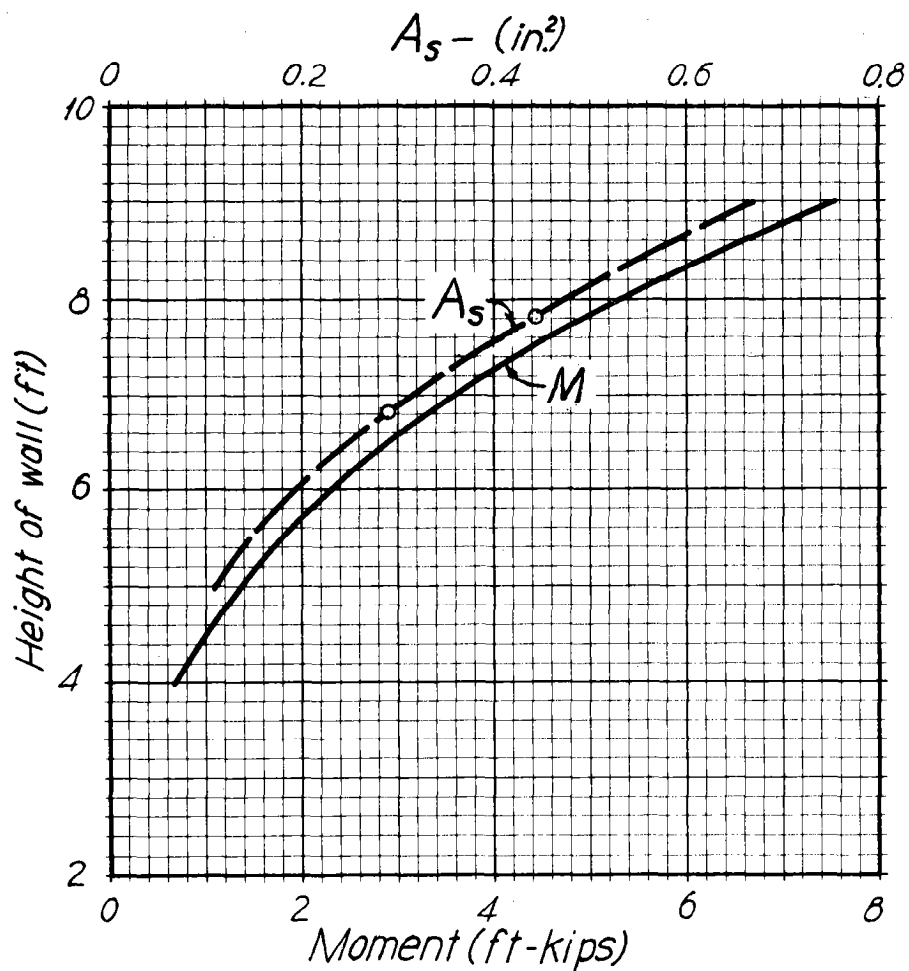
$$\text{From } A_s \text{ curve } H = 7.85$$

$$\text{Dist. from upstream end} = (9.0 - 7.85) \left[\frac{19}{9.0 - 2.33} \right] = 3.28 \text{ ft}$$

$$\text{Find where No. 4 at 12 (} A_s = 0.20 \text{) can be used.}$$

WINGWALL DESIGN

Unexposed face of wingwall



$$H = 6.1 \therefore \text{Length of No. 6 bars} = 7.85 - 6.1 + 3.42 - 0.25 + 1.0 \\ = 5.92. \quad \underline{\text{Use } 6'-0''}$$

Find point along wall where No. 4 at 15 ($A_s = 0.16$) can be used

From A_s curve $H = 5.70$

$$\text{Dist. from upstream end} = (9.0 - 5.70) \left[\frac{19.0}{9.0 - 2.33} \right] = 9.40 \text{ ft}$$

Horizontal Steel (Unexposed face)

Temp. steel--Use No. 4 at 15 (Start 9 in. above footing)

Steel in Exposed face

Horizontal steel--Use No. 5 at 15 Temp. steel

Vertical steel--Use No. 4 at 15 Tie steel. Set on construction joint.

Design of Footing

Load on upstream section (See page 4.12)

$$p = \frac{V}{A} \left[1 \pm \frac{6e}{d} \right]$$

V = vertical weight, A = area of base, e = eccentricity, d = length of section, $e = z - (d \div 2) = 4.29 - (6.83 \div 2) = 0.873$

$$p_1 = \frac{10,548}{6.83} \left[1 + \frac{6(0.873)}{6.83} \right]$$

$$p_1 = 1546 (1 + 0.766) = 2730 \text{ lbs/ft}^2$$

$$p_2 = 1546 (1 - 0.766) = 362 \text{ lbs/ft}^2$$

$$p_3 = 362 + (6 \div 6.83)(2730 - 362)$$

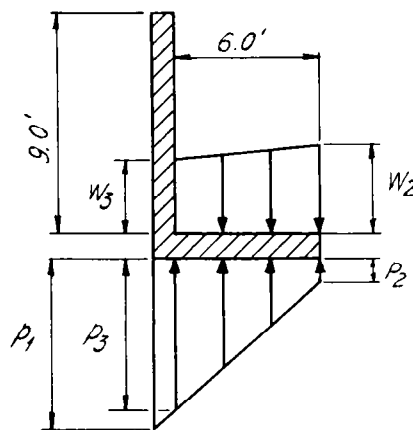
$$p_3 = 362 + 2080 = 2442 \text{ lbs/ft}^2$$

$$W_2 = (11.12 \cdot 110) + (0.92 \cdot 150)$$

$$W_2 = 1360 \text{ lbs/ft}^2$$

$$W_3 = (9.0 \cdot 110) + (0.92 \cdot 150)$$

$$W_3 = 1128 \text{ lbs/ft}^2$$



Moment and Shear

$$M = \frac{W_3 \cdot (6.0)^2}{2} + \frac{(W_2 - W_1)}{2} \frac{2}{3} (6.0)^2 - \frac{p_2(6.0)^2}{2} - \frac{(p_3 - p_2)}{2} \frac{1}{3} (6.0)^2$$

$$M = \frac{1128 \cdot 36}{2} + \frac{232 \cdot 36}{3} - \frac{362 \cdot 36}{2} - \frac{2080 \cdot 36}{6}$$

$$M = 20,300 + 2780 - 6510 - 12,480 = + 4090 \text{ ft lbs}$$

$$V = \frac{(W_2 + W_3)}{2} 6.0 - \frac{(P_2 + P_3)}{2} 6.0 = (W_2 + W_3 - P_2 - P_3) 3.0$$

$$V = 3.0 (1360 + 1128 - 362 - 2442) = 316 \cdot 3 = 948 \text{ lbs}$$

$$d = 7 \frac{1}{2} \text{ in}$$

Req'd. $A_s = 0.36 \text{ in}^2$. Use No. 6 at 12 ($A_s = 0.44$) to midsection then use No. 4 at 12.; Bent into downstream face of toewall.

Use No. 4 at 15 parallel to wall as tie steel.

Design of Headwall Extension

Due to the fact that the headwall extension is articulate, and the earth pressures on both sides of the wall will be about equal, the wall will be reinforced with the minimum steel requirements.

Vertical Steel (both faces)

Use No. 4 at 15

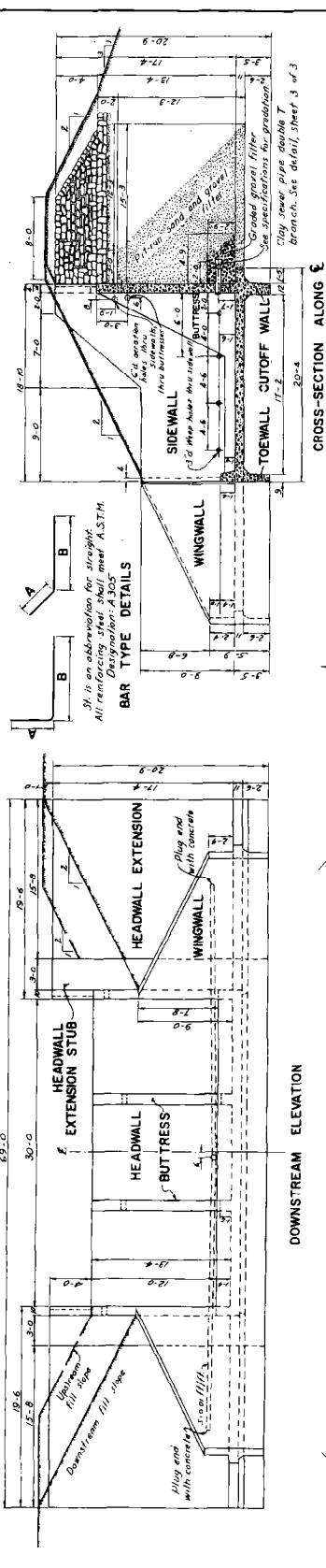
$$\begin{aligned} \text{Length of bars in first pour} &= (2'-6") + (0'-11") + (0'-4") + \\ &\quad (1'-3") - (0'-3") = 4'-9" \text{ (provides 1'-3" lap)} \end{aligned}$$

Horizontal Steel

Use No. 4 at 15, Length = 15'-3"

Steel in footing (1 to wall)(both faces)

Use No. 4 at 15, Length = 3'-6"

[illegible][illegible]

PLAN

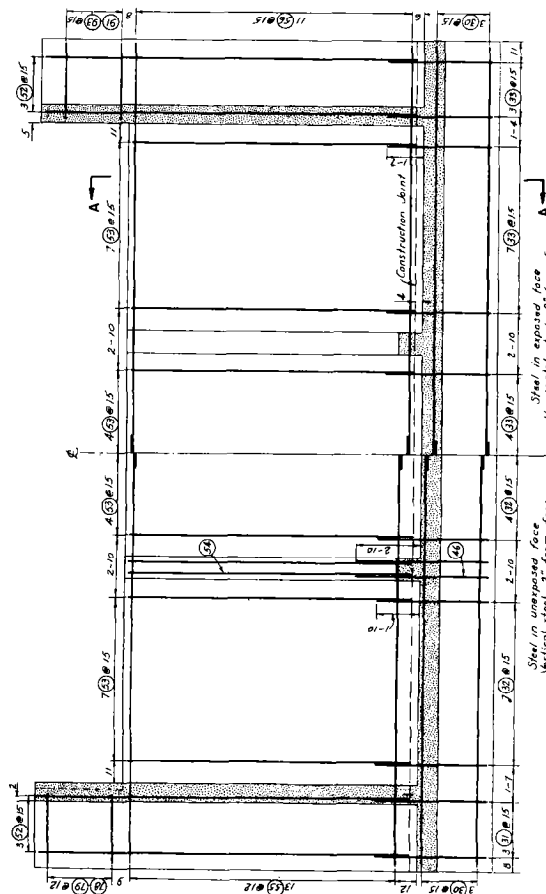
SECTION OF WEEP HOLE AND DRAIN FILTER

NOTE: Structure is symmetrical about E, except clay drain pipe outlet and filter. Dimensions are spaced every 1/4". All fillets are 1/4".

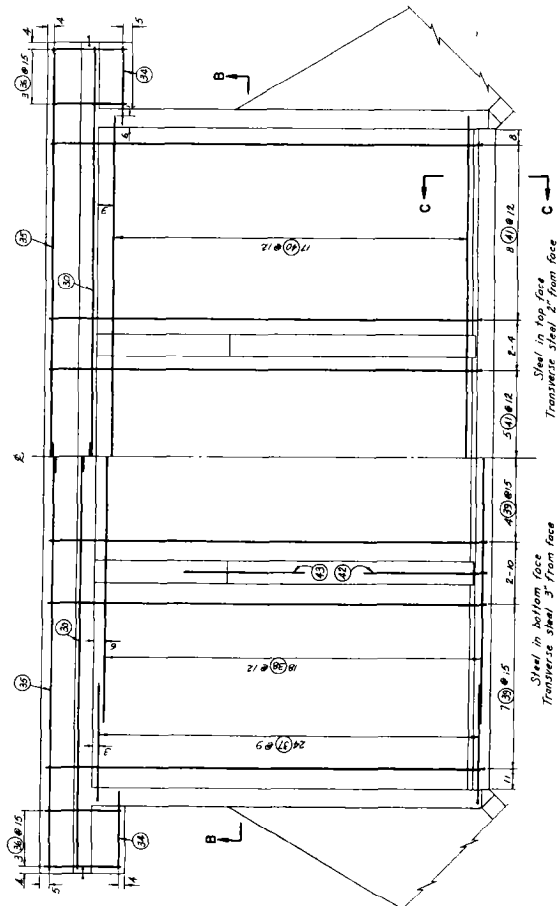
Class B Concrete

$f'_c = 3,000$ (ksi) min
 $f'_t = 1,000$ (ksi) min
 $f'_e = 1,000$ (ksi) min

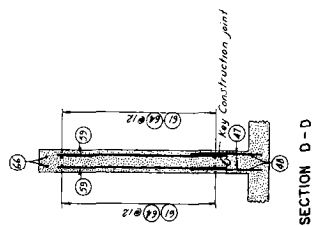
$F = 12-0$, $H = 4-0$, $L = 30-0$



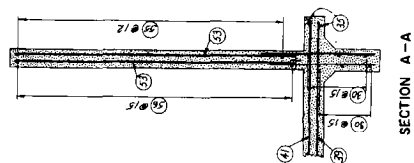
SECTIONAL ELEVATION OF HEADWALL AND HEADWALL EXTENSION STUB



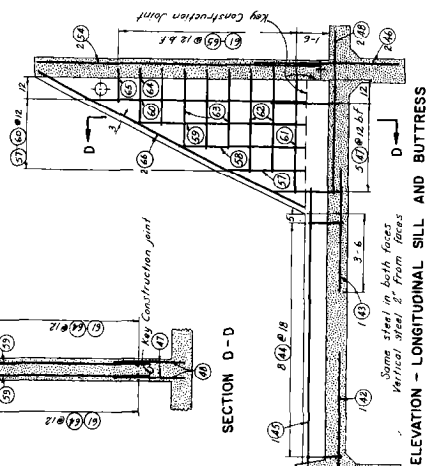
PLAN OF APRON AND HEADWALL EXTENSION STUB FOOTING



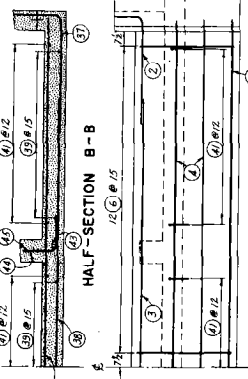
SECTION D-D



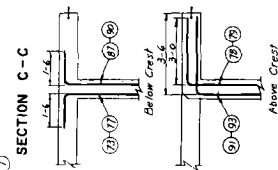
SECTION A-A



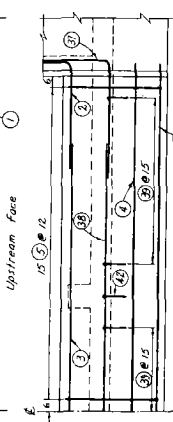
ELEVATION - LONGITUDINAL SILL AND BUTRESS



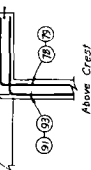
HALF-SECTION B-B



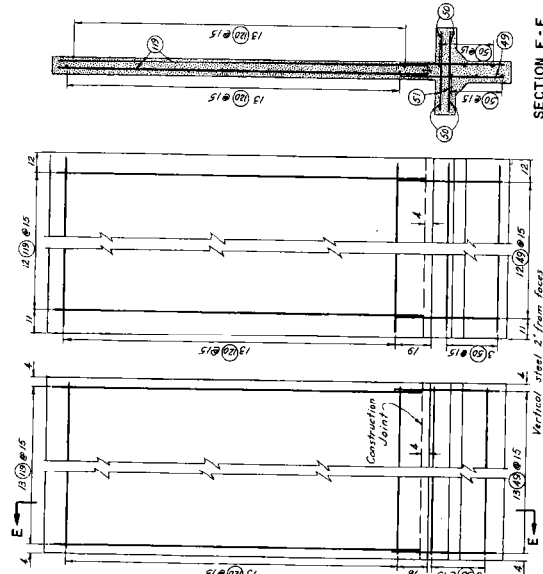
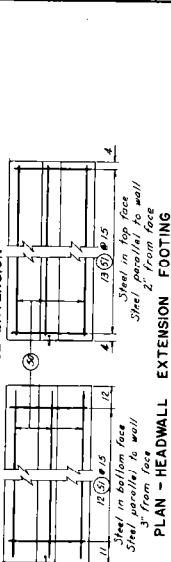
SECTION C-C



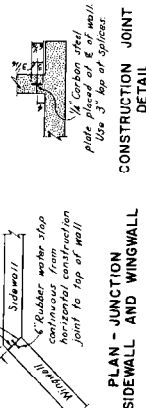
TOEWALL AND TRANSVERSE SILL

PLAN - JUNCTION OF HEADWALL,
SIDEWALL AND HEADWALL
EXTENSION STUB

U.S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	
DESIGNED BY H. M. C. F. R. M. A.	CHECKED BY H. M. C. F. R. M. A.
DATE 7-2-53	SCALE 1" = 10'
DRAWING NO. S-80	
SHEET 2 OF 2	



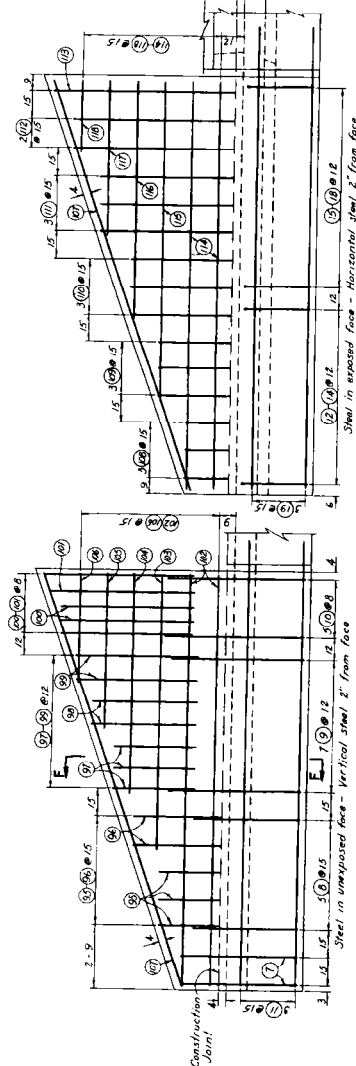
PLAN - JUNCTION HEADWALL EXTENSION AND HEADWALL EXTENSION STUB



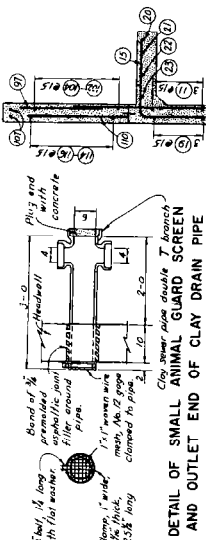
PLAN - JUNCTION SIDEWALL AND WINGWALL

CONSTRUCTION JOINT DETAIL

U.S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - DESIGN SECTION	ENGINEERING DESIGN DESIGNED BY CHECKED BY DATE	CONSTRUCTION JOINT DESIGNED BY CHECKED BY DATE
	C.A.R.	C.A.R.
	MACFARM	MACFARM
	ES - 80	ES - 80
	SHEET 3 OF 3	SHEET 3 OF 3



ELEVATION - WINGWALL



SECTION F-F

PLAN - WINGWALL FOOTING

Steel perpendicular to wall 2" from top face