A DESIGN AND COST COMPARISON OF RETAINING WALLS FOR COAL STORAGE AT THE NEW MICHIGAN STATE COLLEGE POWER PLANT

Thesis for the Degree of B. S.

MICHIGAN STATE COLLEGE

D. J. Morfee — D. N. Worfel

1948

A Design and Cost Comparison of

Retaining Walls for Coal Storage at

The New Michigan State College

Power Plant

A Thesis Submitted to

The Faculty of

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of

AGRICULTURE AND APPLIED SCIENCE

by

D. J. Morfee

D. N. Worfel

Candidates for the Degree of

Bachelor of Science

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PREFACE

While the primary purpose of this thesis is to fulfill the requirements of the Michigan State College school of Engineering, it is also presented as a practical solution to an existing problem.

The authors have endeavored to include sufficient computations and working drawings to give the reader a clear concept of the procedures involved.

Investigation was made of unit weight, allowable soil pressures, angle of friction under various conditions of saturation and height of the filling material (coal), etc. Much discussion with persons familiar with the problem was necessary to more successfully and accurately produce a practical solution.

The authors wish to express their appreciation to those who have aided in this work. They stand fully indebted to Mr. William Bradley, faculty advisor and Evelyn Morfee, typist, who gave freely of their ideas and time.

June 1948

D.J.M.
D.N.W.

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INTRODUCTION

Due to the large increases in enrollment and the huge building program at Michigan State College since 1945, it was necessary for the college to build a new Power Plant. This plant is located south of the Red Cedar River and is to eventually substitute completely the existing plant.

In line with the expansion program, it seems reasonable to assume that all present planning should be done with utilization of space as the primary goal.

The storage of coal for the plant would consume a large area if merely piled on the ground with out the use of retaining walls. The coal would be spread out and would be carried to the hoppers from the pile with a great deal of inefficiency. An enclosure would also add beauty to the storage area.

With these aspects of the problem clearly in mind the required size is the next consideration.

As previously stated, the new plant will substitute the existing one. It will operate three boilers and have a maximum capacity of 360 tons per day. In this thesis, 270 tons per day and 42 days supply are the basis for design -- 270 being the maximum expected demand.

We will treat the back wall as a cantilever and then as a counterfort wall. The front wall will be semi-gravity and the side wall will be of three elevations. The front twenty feet of the side wall on the north side will be semi-gravity and of the same design as the front wall. The next twenty feet will be cantilever and the back twenty feet will be cantilever, and of the same design as the cantilever back wall. The side wall on the south side has only the front eighteen feet, the same design as the front wall. The rest of that side is the same as the north side.

Counterforts with steel angles on the edges will be used on the back wall. It is assumed that the shovel will not do damage with this precaution taken because it will be traveling vertically at sixty feet from the cab. Counterforts on the side wall, however, will not be used.

Expansion joints will be used at fifty foot intervals on the front and back walls. There will be no expansion joints in the side wall.

In determining the required capacity of the enclosure, it was found that many small piles of coal present less fire hazard than a few large piles. With large piles the fine and coarse particles separate and produce air channels and drafts which aid fire. It is suggested that the piles be kept of fairly equal height.

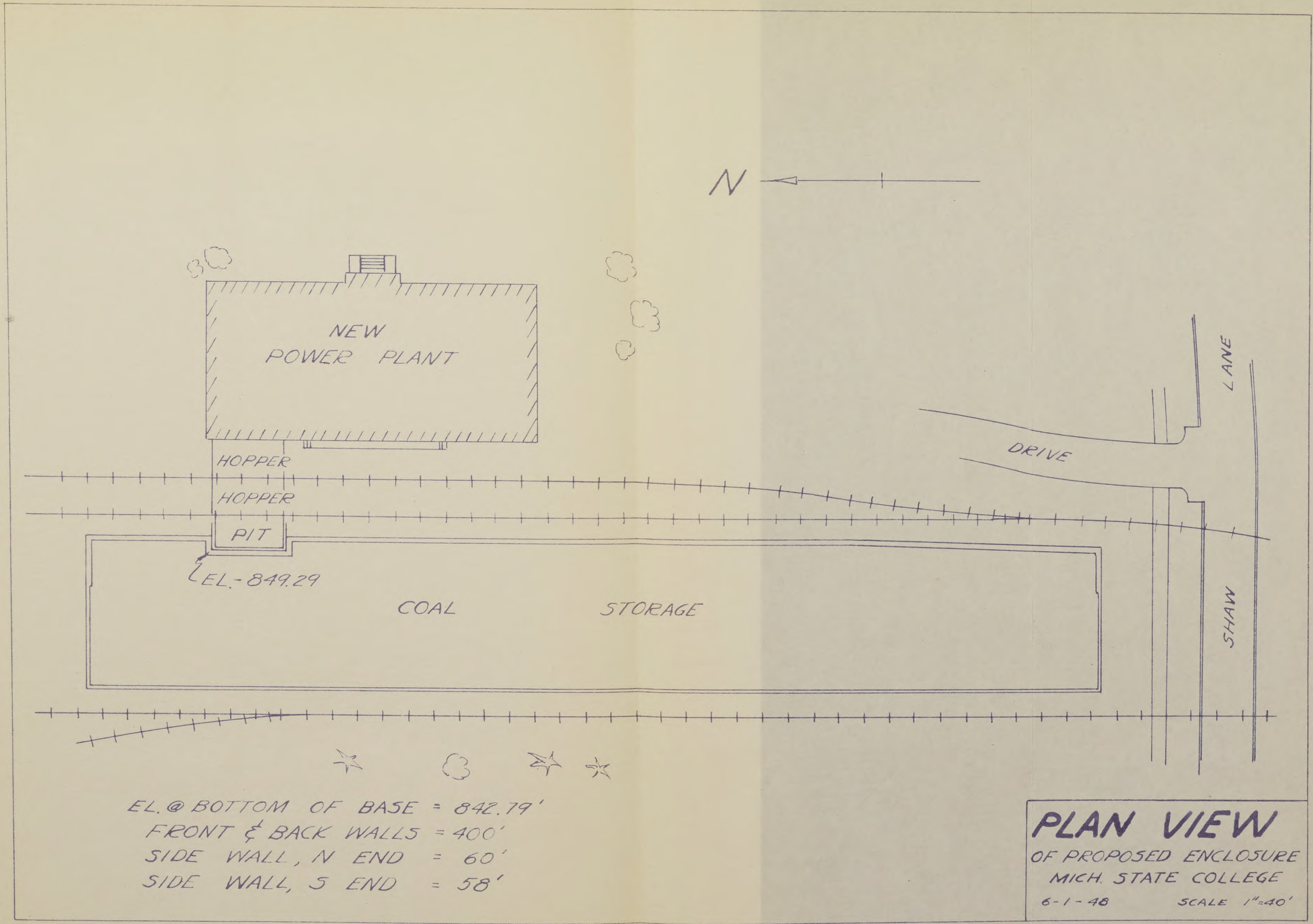
A ten foot portion of the wall should be omitted to permit the passage of a bulldozer. The bulldozer will allow a mixing of the coal when deemed necessary.

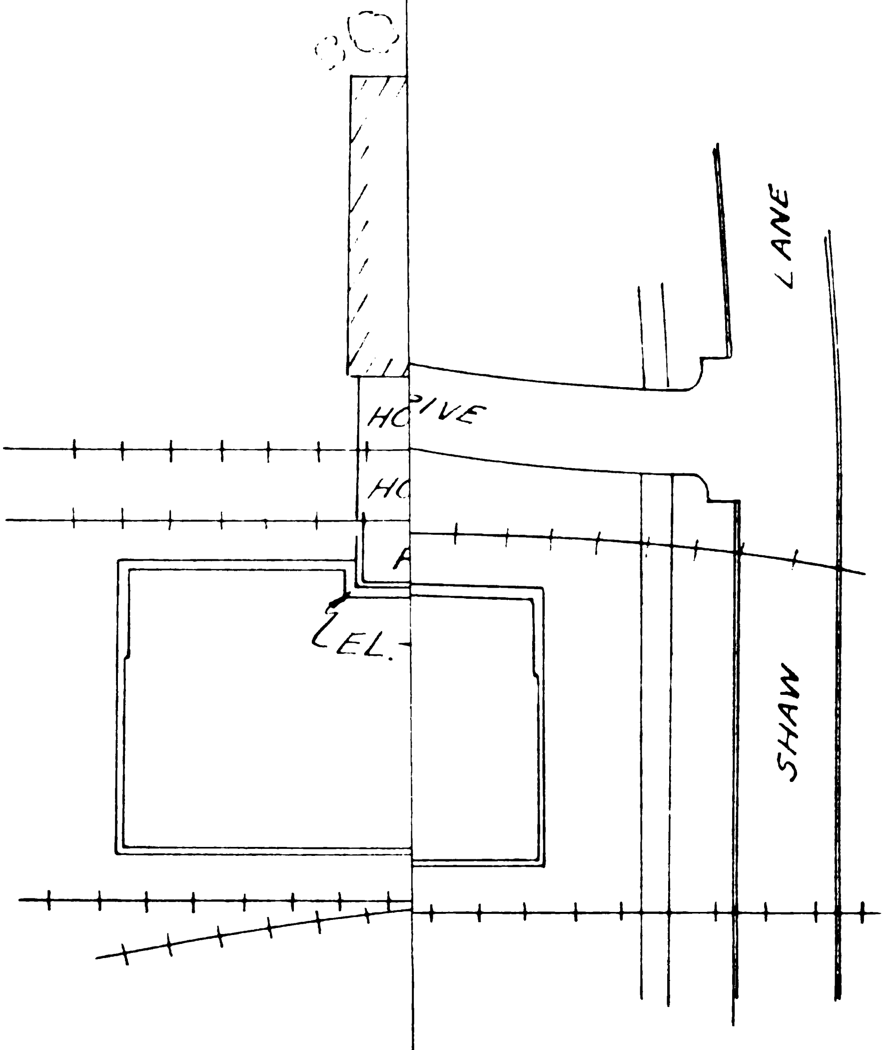
The back wall was designed at a height of twenty feet, overall, in both cantilever and counterfort design to allow a good cost comparison. The side and front walls will cost the same because their design is the same for both cases.

As a final pre-design consideration, let us trace the paths that the coal takes as it goes to the boilers.

It is brought into the area along the north-south tracks, east of the stadium and then onto the tracks close to the west side of the plant. As much coal as possible is dumped in a pit at the north-west corner of the plant and it is carried to overhead hoppers by a bucket escalator. The surplus is put in the storage area by a crane to be carried back to the pit when needed.

CAMPUS OF
MICHIGAN STATE COLLEGE
OF AGRICULTURE AND APPLIED SCIENCE
EAST LANSING, MICHIGAN
400 200 0 200 400 600 800 1000
CAMPUS LANDSCAPE ARCHITECT
HAROLD W LAUTNER
N
TO GRAND RAPIDS
CITY OF EAST LANSING
TO LANSING
TO DETROIT
ATHLETIC FIELD
WOMEN'S DORMITORIES
BEAUMONT TOWER
BEAL BOTANICAL GARDENS
POSSIBLE SITE FOR CHAPEL AND INTERNATIONAL HOUSE
HORTICULTURAL GARDENS
CLASSROOM
LIBRARY
MUSIC
MEN'S DORMITORIES
DEMONSTRATION HALL
PARKING
TENNIS COURTS
AUDITORIUM
STADIUM
TRACK
FIELD
PROPOSED SITE
KALAMAZOO STREET
QUONSET DORMS
MICHIGAN STATE POLICE
FACULTY HOUSING
APARTMENTS
SHAW LANE
FORESTRY NURSERY
GREENHOUSE SITE
WOODLAND AREA
PINETUM
FARM AREA
FARM LANE
HAGADORN ROAD
HARRISON ROAD
GRAND TRUNK RAILROAD
CAMPUS MAINTENANCE NURSERY
PERE MARQUETTE RAILROAD
LEGEND
PROPOSED ROADS & BUILDINGS
EXISTING ROADS & BUILDINGS


N
NEW
POWER PLANT
HOPPER
HOPPER
PIT
EL.- 849.29
COAL
STORAGE
DRIVE
LANE
SHAW
EL. @ BOTTOM OF BASE = 842.79'
FRONT & BACK WALLS = 400'
SIDE WALL, N END = 60'
SIDE WALL, S END = 58'
PLAN VIEW
OF PROPOSED ENCLOSURE
MICH. STATE COLLEGE
6-1-48
SCALE 1"=40'



EL. @ BOT

FRONT

SIDE W

SIDE W

PLAN VIEW

OF PROPOSED ENCLOSURE

MICH. STATE COLLEGE

6-1-48 SCALE 1"=40'

VOLUME REQUIREMENTS

of the

ENCLOSURE

In order to design a wall of sufficient strength, economical and still fill the desired requirements several factors had to be considered. First of all the volume was taken into consideration. The consumption of coal was placed at approximately 275 tons per day and a 42 day storage was required. It was found that 56 pounds per cubic foot was the best average value of the unit weight of bituminous coal.

In the book, "Walls, Bins, and Grain Elevators" by Milo S. Ketchum, an average value for the angle of repose was assumed as 35°. The volume then equals $\frac{275 \times 2000 \times 42}{56} = 412{,}500$ cubic feet of coal.

The cross sectional views are found for the 57 foot and the 58 foot widths on the following page. Also in this thesis are shown location and plan views which may be referred to.

The computations for the volumes were as follows:

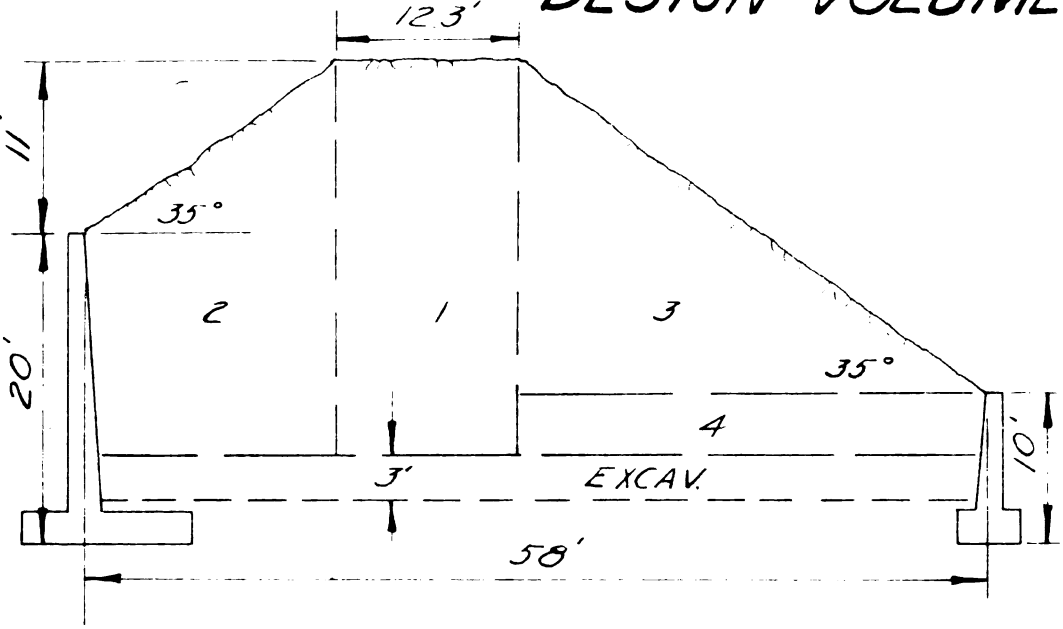
A. Cross Section Area for 58' Width.

(1)	12.3 x 25	= 307.5 Sq. Ft.
(2)	19.5 x 15.2	= 296.4
(3)	15 x 21	= 315.0
(4)	30 x 4	= 120.0
		1,038.9 Sq. Ft.
Excav.	3' x 57	= 171.0
		1,209.9 Sq. Ft.

B. Cross Section Area for a 57' Width.

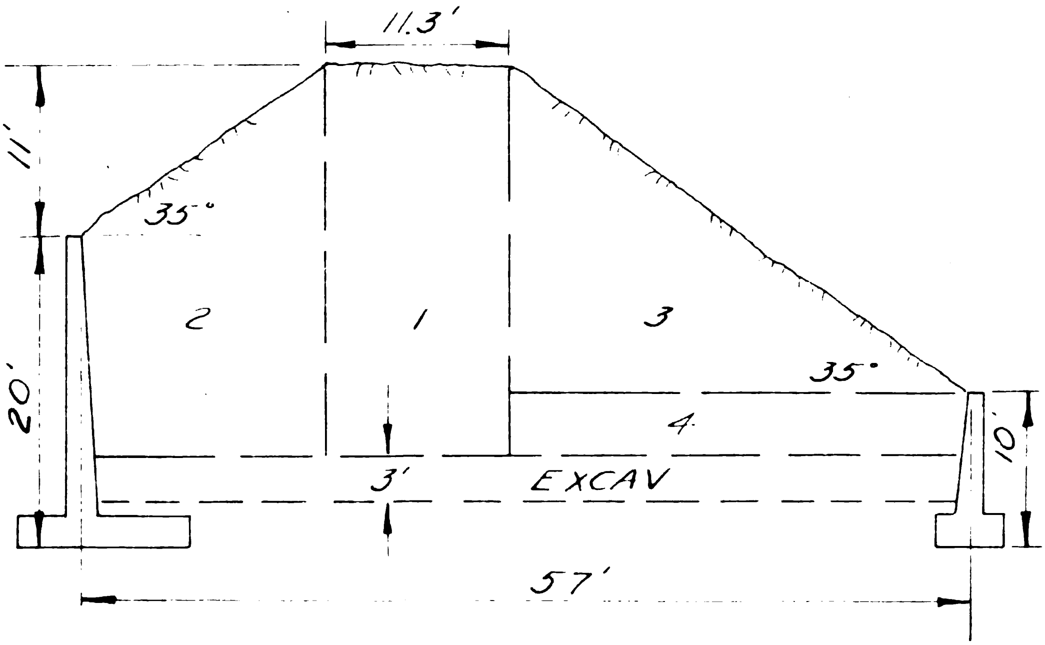
(1)	11.3 x 25	= 282.5 Sq. Ft.
(2)	19.5 x 15.2	= 296.4
(3)	15 x 21	= 315.0
(4)	30 x 4	= 120.0
		1,013.9 Sq. Ft.

DESIGN-VOLUME



FIFTY-EIGHT FOOT SECTION
LOOKING NORTH

FIFTY-SEVEN FOOT SECTION
LOOKING NORTH



Computations continued:

		1,013.9 Sq. Ft.
Excav. 3' x 56	=	168.0
		1,181.9 Sq. Ft.

Volumes:

A.	1,209.9 x 300	=	362,970
B.	1,181.9 x 100	=	118,190
			481,160 Cu. Ft.

This gives a value of 68,660 cubic feet margin which is sufficient to allow for angle of repose (35°) on each end.

It was found that a wall 20 feet high in back, 10 feet high in front, and 3 feet of excavation throughout would give the required volume. Allowance was made for the loss in volume due to the slope of the side wall. It was determined that about 25 feet above track level was the maximum convenient piling level. This factor limited the volume.

The wall is far enough below frost line, throughout, to be free from frost heave.

A STUDY OF THE POSSIBILITIES

OF USING A

CANTILEVER BACK WALL DESIGN

INTRODUCTION

After the height was determined to give sufficient volume, the type of design was the next consideration. The height of 20 feet eliminated the gravity wall because experience showed that greater cost was incurred. That left the other two main types, the cantilever and the counterfort. The first design was the cantilever and the second design was the counterfort.

Retaining walls have been designed for many, many years by many, many people with the result that there are several theories too numerous to mention. Used in this thesis were theories of Sutherland and Reese as found in their book, "Reinforced Concrete Design" with variations as noted. We attempted to use our own logical thinking to determine methods on controversial subjects.

From the "Joint Committee - Concrete and Reinforced Concrete" - 1940, the following values were taken:

Ultimate concrete stress	$= f_c' =$	2000 p.s.i.
Allowable concrete stress	$= f_c =$	800 p.s.i. $= .4\ f_c'$
Steel stress	$= f_s =$	18000 p.s.i.
Ratio $\frac{E_s}{E_c}$	$= n =$	15
	$R =$	139
Unit Shear	$= v =$	$.02\ f_c'$
Unit bond stress	$= u =$	100 p.s.i. (less than 150 if anchored)

A soil report for the new power plant was found in the Michigan State College engineering office and with this in mind and consideration for the depth required a unit force of somewhere between 4000 and 4250 pounds per square foot seemed like a good value for allowable soil pressure.

It should be noted that if this construction is contemplated a complete soil investigation should be carried out.

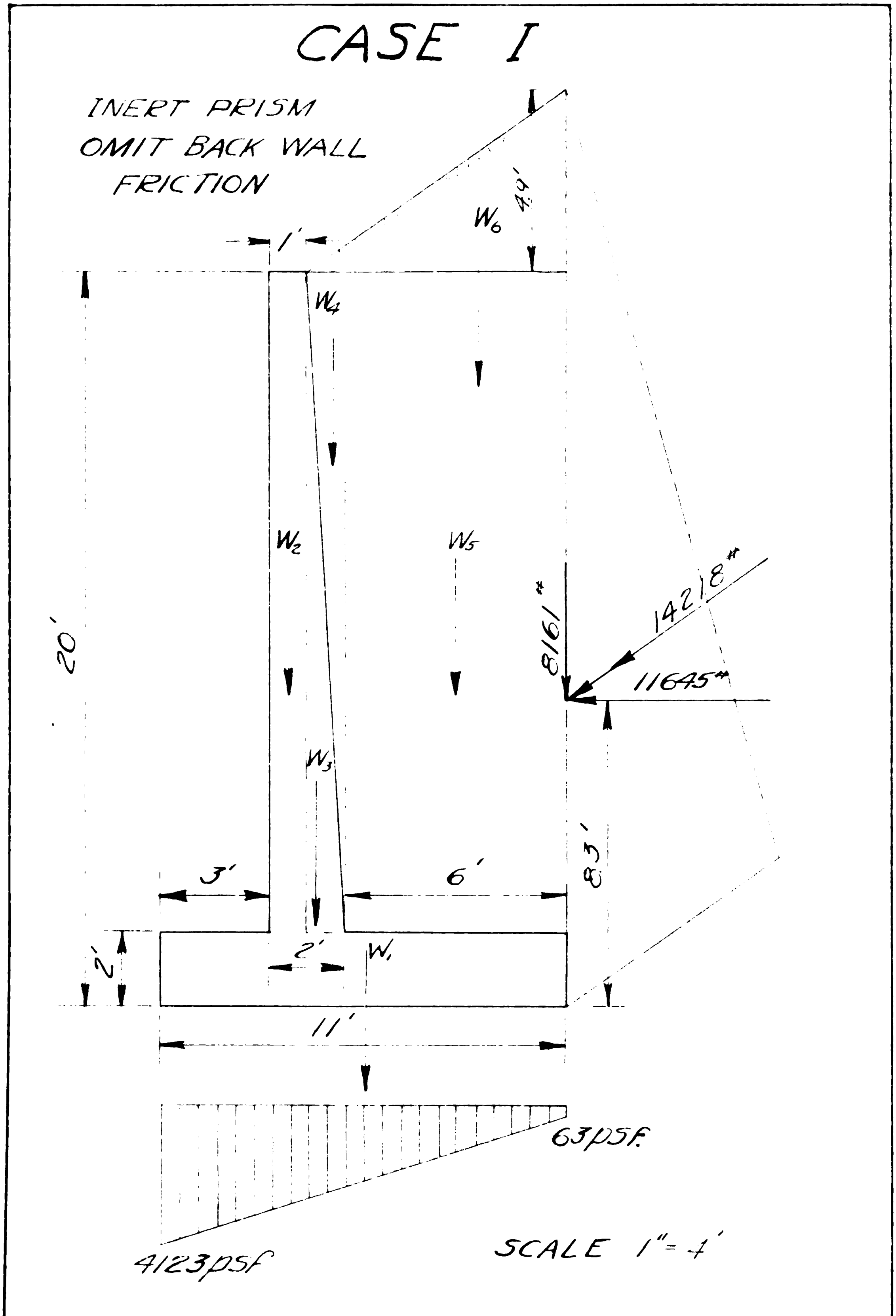
With the height in mind the wall was proportioned accordingly. Several dimensions were tried and only the final results will be found in the computations.

Three cases were decided upon, the worst of which form the basis for our calculations. Case one involved Rankine's design using the inert prism and omitting back wall friction.

Coulomb's theory with the Culmann construction considering an inert wedge with full back wall friction was the second case taken into consideration.

The third case was the same as the second, but an inert prism was used in place of an inert wedge.

It may be noted that there are, perhaps, more logical possibilities, but the three cases used should give approximately the same results.


CASE I
INERT PRISM
OMIT BACK WALL
FRICTION
1'
4.9'
W6
W4
W2
W5
W3
W1
20'
8161#
14218#
11645#
8.3'
3'
6'
2'
2'
11'
63 psf.
4123 psf
SCALE 1" = 4'

CASE I

With reference to the preceeding drawing the following computations can be easily followed. It can be seen that the resultant of the earth pressure acted in the middle one-third and that the soil pressures were within bounds of the assumed value. The safety factor against sliding was well below the required safety factor of two; therefore, a cut-off wall on the base was required.

Computations:

$$C = \cos\theta \left(\frac{\cos\theta \mp \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta \pm \sqrt{\cos^2\theta - \cos^2\phi}} \right)$$

$C = .819$

p bottom $= Cwh$

$= .819 \times 56 \times 24.9$

$= 1,142$ p.s.f.

$P = 1/2 \times 24.9 \times 1142 = 14,218\#$

$P_v = 8,161\#$

$P_h = 11,645\#$

$x = 1/3 \times 24.9 = 8.30'$

Base Pressure: Moment About Toe.

$W_1 = 11 \times 2 \times 150$	$= 3,300\#$	x	5.5'	$= 18,150'\#$
$W_2 = 1 \times 18 \times 150$	$= 2,700$	x	3.5	$= 9,450$
$W_3 = 1/2 \times 18 \times 150$	$= 1,350$	x	4.34	$= 5,859$
$W_4 = 1/2 \times 18 \times 56$	$= 504$	x	4.67	$= 2,354$
$W_5 = 6 \times 18 \times 56$	$= 6,048$	x	8.00	$= 48,384$
$W_6 = 7 \times 4.9 \times 1/2 \times 56$	$= 960$	x	8.67	$= 8,323$
	14,862#			92,520'#

Computations continued:

$$P_v = \frac{\begin{array}{r}14{,}862\# \\ 8{,}161\end{array}}{23{,}023\#} \times 11 = \frac{\begin{array}{r}92{,}520'\# \\ 89{,}771\end{array}}{182{,}291'\#}$$

$$P_h(d) = -\ 11645 \times 8.3 = \frac{-96{,}654}{85{,}637'\#}$$

$$x = \frac{85637}{23023} = 3.72'$$

$$5.50 - 3.72 = 1.78' = e$$

.05 within middle one-third

$$f = \frac{P}{A}\left(1 \pm \frac{6e}{b}\right)$$

$$= \frac{23023}{11}\left(1 \pm \frac{6 \times 1.78}{11}\right)$$

$$= 2093\left(1 \pm .97\right)$$

f toe = 4123 p.s.f.

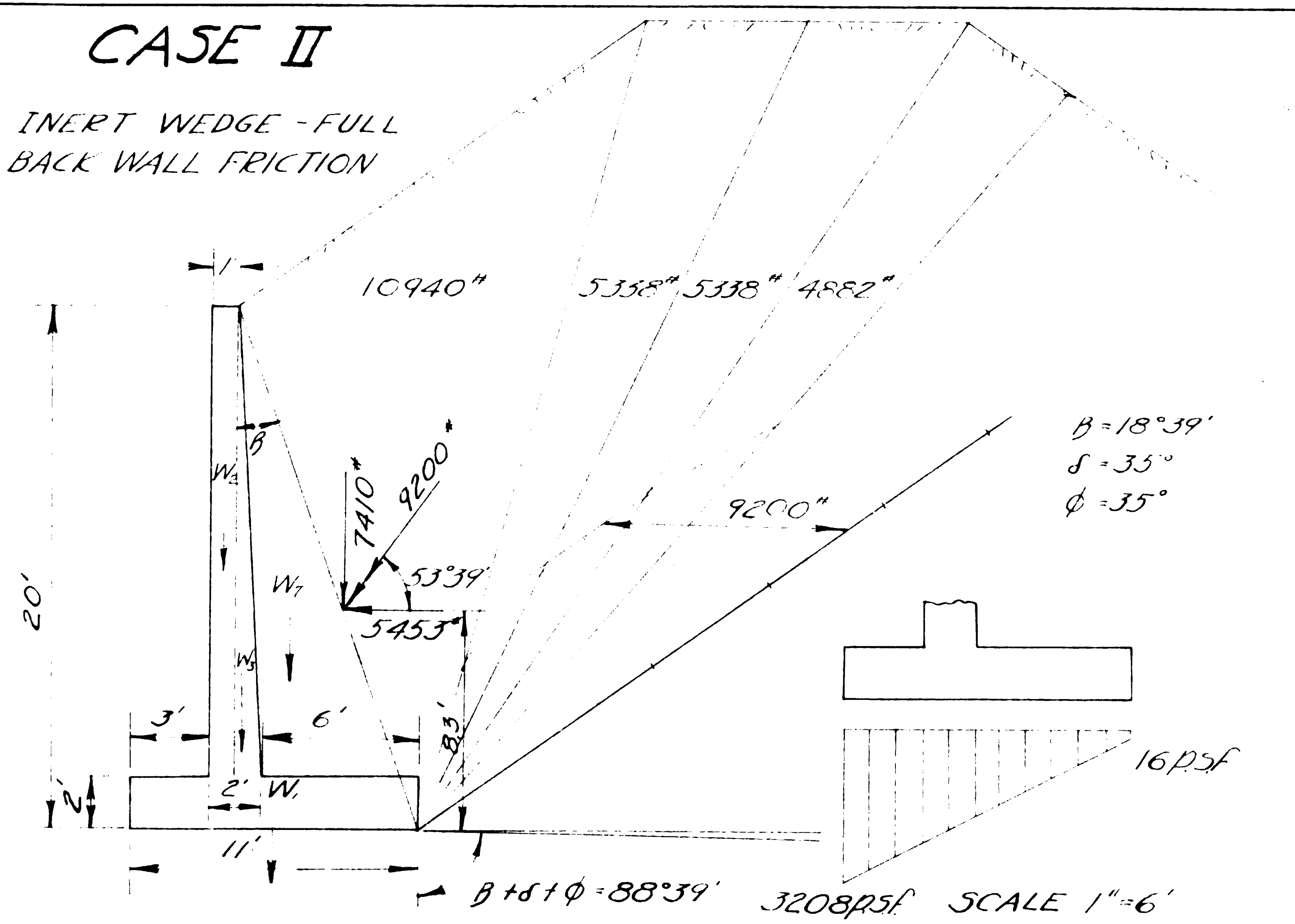
f heel = 63 p.s.f.

$$S.F. = \frac{23023 \times .499}{11645}$$

$= .99$ Therefore must anchor.

CASE II

INERT WEDGE - FULL
BACK WALL FRICTION



CASE II

Again the computations are easily followed with reference to the preceding drawing of the Culmann construction. The results again show that the resultant of the earth pressure fell within the middle one-third. This is an important factor because when the resultant acts at a point outside the middle one-third the cost of construction is greatly increased. This time the soil pressure at the toe was well below the maximum and the safety factor against sliding was higher but still required the cut off wall.

Base Pressure: Moment About Toe.

$W_1 = 11 \times 2 \times 150$	$= 3{,}300\#$	x	5.5	$= 18{,}150'\#$
$W_2 = 1 \times 18 \times 150$	$= 2{,}700$	x	3.5	$= 9{,}450$
$W_3 = 1/2 \times 18 \times 150$	$= 1{,}350$	x	4.34	$= 5{,}859$
$W_7 = 1/2 \times 18 \times 5.3 \times 56$	$= 2{,}671$	x	6.43	$= 17{,}175$
	$10{,}021\#$			$50{,}634'\#$
$P_v =$	$7{,}410$	x	8.09	$= 59{,}947$
	$17{,}731\#$			$110{,}581'\#$
	$P_h(d) = -5453 \times 8.3$			$= -45{,}260$
				$65{,}321'\#$

$$x = \frac{65321}{17731} = 3.684$$

$$5.50 - 3.68 = 1.82' = e$$

.017 within middle one-third

$$f = \frac{P}{A}\left(1 \pm \frac{6e}{b}\right)$$

$$f \text{ toe} = 1612 \times 1.99 = 3208 \text{ p.s.f.}$$

$$f \text{ heel} = 1612 \times .01 = 16 \text{ p.s.f.}$$

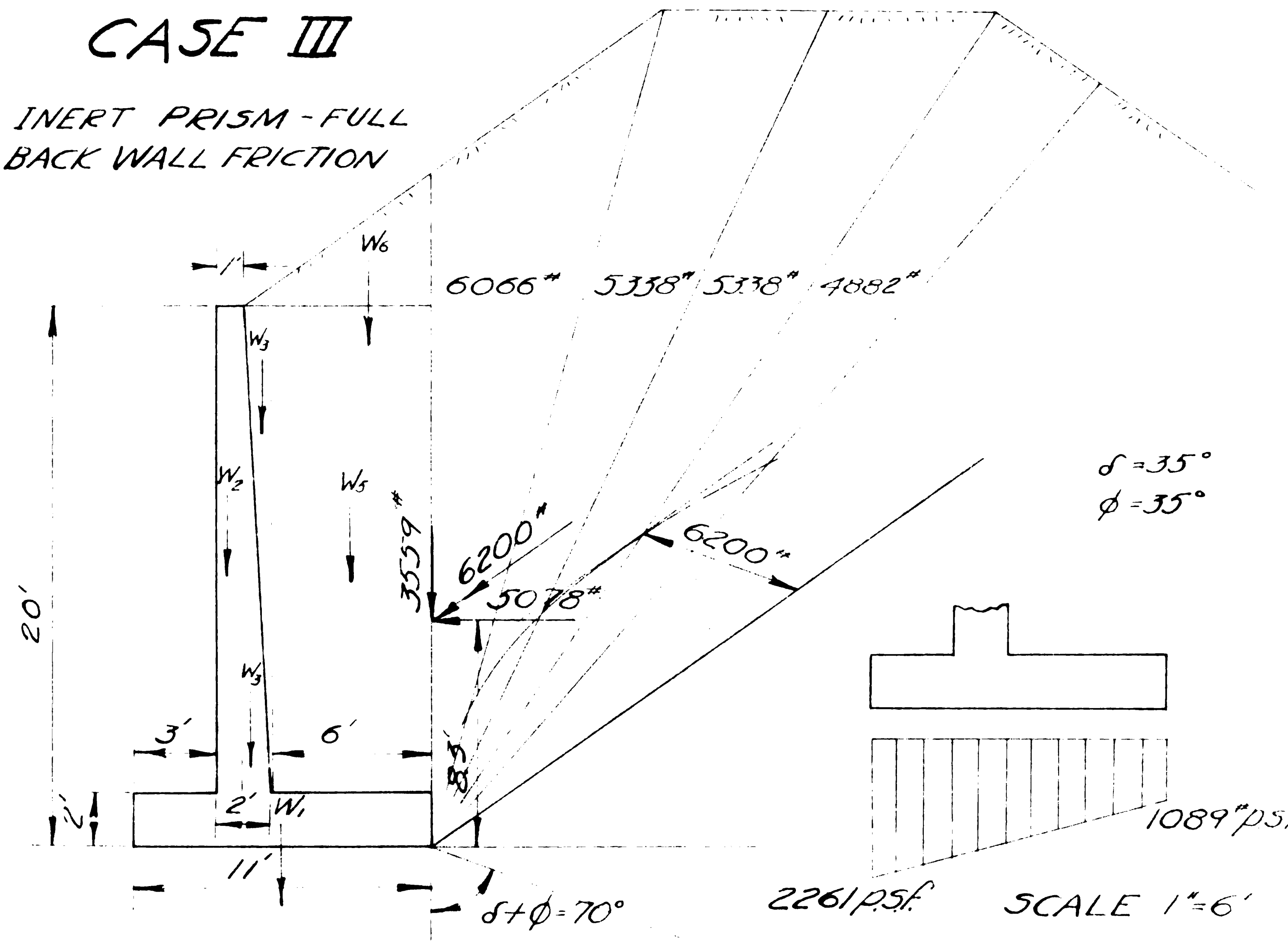
Computations continued:

$$S.F. = \frac{17731 \times .499}{5453}$$

$$= 1.6$$ Therefore must anchor.

CASE III

INERT PRISM - FULL
BACK WALL FRICTION



CASE III

Case three was constructed similar to Case two with the exception that the inert wedge was replaced by the inert prism. This time the resultant on the base moved closer to the center giving a more even earth pressure which, of course, was well below the 4250 pounds per square foot maximum. The sliding factor was higher than the previous two cases but still not the required two.

Computations:

Base Pressure: Moment About Toe.

$W_1 = 11 \times 2 \times 150$	$= 3,300\#$	x	5.5'	$= 18,150'\#$
$W_2 = 1 \times 18 \times 150$	$= 2,700$	x	3.5	$= 9,450$
$W_3 = 1/2 \times 18 \times 150$	$= 1,350$	x	4.34	$= 5,859$
$W_4 = 1/2 \times 18 \times 56$	$= 504$	x	4.67	$= 2,354$
$W_5 = 6 \times 18 \times 56$	$= 6,048$	x	8.00	$= 48,384$
$W_6 = 7 \times 4.9 \times 1/2 \times 56$	$= 960$	x	8.67	$= 8,323$
	14,862#			92,520'#
P_v	$= 3,559$	x	11	$= 39,149$
	18,421#			131,669'#
$P_h(d) = -5078 \times 8.3$				$= -42,147$
				89,522'#

$$x = \frac{89522}{18421} = 4.86'$$

$$5.50 - 4.86 = .64' = e$$

1.19 within middle one-third

$$f = \frac{P}{A}\left(1 \pm \frac{6e}{b}\right)$$

$$= \frac{18421}{11}\left(1 \pm .35\right)$$

f toe $= 1675 \times 1.35 = 2261$ p.s.f.

f heel $= 1675 \times .65 = 1089$ p.s.f.

Computations continued:

$$S.F. = \frac{18421 \times .499}{5078} = 1.81$$

Therefore must anchor.

SUMMARY

It was necessary to design the wall for the worst case in order to be assured that the wall would not fail. From the chart that follows it can be noted immediately that Case one was by far the worst case. The sliding factor was very low, high toe pressure and a large horizontal component of thrust were all indications that Case one should be designed.

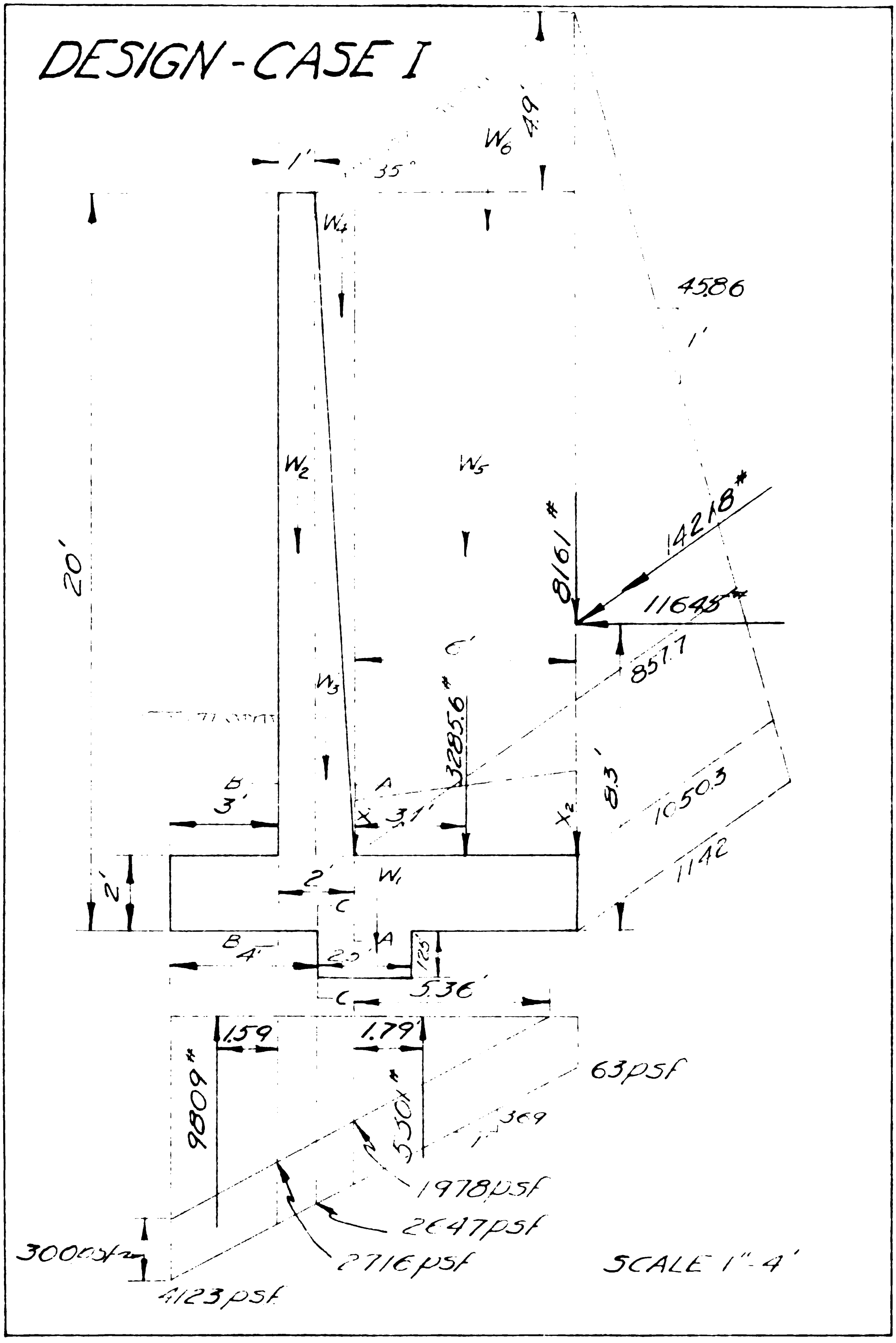
SUMMARY

	P TOE	P HEEL	H COMP OF THRUST	M_o	M_R	FACTOR $\frac{M_R}{M_o}$	SLIDING FACTOR
CASE I	4,123	63	11,645	96,654	182,291	1.88	.99
CASE II	3,208	16	5,453	45,260	110,581	2.44	1.60
CASE III	2,661	1,089	5,078	42,147	131,669	3.12	1.81

DESIGN FOR CASE I

OVERTURNING FACTOR SHOULD BE 2 BUT 1.88 SEEMS WITHIN REASON CONSIDERING THAT THE EARTH WEIGHT ACTING DOWN HAS NOT BEEN CONSIDERED.

ACTUAL DESIGN


DESIGN - CASE I
4.9'
W_6
1'
35°
4586
1'
W_4
W_2
W_5
20'
8161#
14218#
11645#
857.7
6'
W_3
3285.6#
8.3'
1050.3
B
A
3'
X
3.1'
X_2
1142
2'
2'
W_1
C
A
B
4'
2.5
1.25
5.36'
C
1.59
1.79
9809#
63psf
5501#
369
1978psf
2647psf
2716psf
3000psf
SCALE 1" - 4'
4123psf

Next, was a closer study of the cantilever wall. If the dimensions that have been assumed are correct the design is a simple process. In this design the authors followed through the computations numerous times until the following computations were derived with the dimensions as noted on the previous page. The wall design was in sections: toe, heel, cut-off, etc., always correlating the results to assure uniformity.

Again, it can be said that the computations contain no involved mathematics and can be easily followed by merely referring to the drawings of the wall of Case one.

HEEL DESIGN

The heel was the first consideration. From the book, "Concrete, Plain and Reinforced", Vol. 1, by Taylor, Smulski and Thompson, it was noted that the entire pressure triangle is considered and that the force acts through the centroid of the triangle, parallel to the surface and thus to the wall where its components are taken. This reasoning seemed very practical and was used in our designs of the cantilever walls.

Shear and moments were taken at the section noted and the results used to compute the depths. The formulas used are those accepted by the 1940 "Joint Committee on Concrete and Reinforced Concrete".

The results obtained were reasonable and conformed to our already accepted values and dimensions. Note the fact that special anchorage on the reinforcing bars was not required.

Computations:

$$x_1 = \frac{1142 \times 18.7}{24.9} \times .574 = 492.3\# \text{ /Sq.Ft.}$$

$$x_2 = \frac{1142 \times 22.9}{24.9} \times .574 = 602.9\# \text{ /Sq.Ft.}$$

$$P_v = \frac{492.3 \quad 602.9}{2} \times 6 = 3285.6\#$$

$$x = \frac{6}{3}\left(\frac{492.3 \quad 2 \quad 602.9}{1095.2}\right) = 3.1'$$

Computations continued:

Maximum Shear and Moment on Section A-A.

	$P_v = -$ 3,285.6#	x	3.1'		= -10,185'#
	$W_5 = -$ 6,048	x	3.0		= -18,144
	$W_6 = -$ 960	x	4.0		= - 3,840
	-10,293.6#				-32,169'#
Earth P	= 5,301.0	x	1.79		= +9,489
V	= 4,992.6#			M	= 22,680'#

$$d = \frac{V}{bjv} = \frac{4992.6}{40x.875x12} = 11.9"$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{22680}{139}} = 12.8"$$

Therefore, approximately 16" is required but keep base 24" due to expected large shear on toe.

$$v = \frac{V}{bjd} = \frac{4992.6}{12x.875x21} = 22.6 \text{ p.s.i.}$$

No special anchorage required.

$$R = \frac{M}{bd^2} = \frac{22680}{21^2} = 51 < 139$$

Therefore f_c is very low.

$$A_s = \frac{M}{f_s jd} = \frac{22680}{18000x.875x21} = .069 \text{ sq.in. per in.}$$

Use 3/4 inch square bars @ 6" c/c.
Gives .073 sq. inch.

$$u = \frac{Vb}{\Sigma o} = \frac{22.6 \text{ x } 6}{3} = 45.2 \text{ p.s.i.} < 100 \text{ p.s.i.}$$

No special anchorage required.

TOE DESIGN

By taking a section B-B the toe was very readily attacked and in much the same manner the values were obtained. The earth above the toe was not considered, thus giving a larger shear and a safer design.

The values obtained are of good nature and conform again to the accepted values. One difference may be noted in the fact that the bars are anchored.

Computations:

$$P = 3\left(\frac{3823 \quad 2716}{2}\right) = 9809\#$$

$$x = \frac{3}{3}\left(\frac{2716 \quad 3823 \times 2}{6539}\right)$$

$$x = \frac{10362}{6539} = 1.59'$$

Maximum Shear and Moment on Section B-B.

$$V = 9809\#$$

$$M = 9809 \times 1.59 = 15{,}596'\#$$

$$d = \frac{V}{vjb} = \frac{9809}{60 \times .875 \times 12} = 15.6'' \qquad \text{OK}$$

The steel must be anchored.

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{15596}{139}} = 10.6''$$

$$v = \frac{V}{bjd} = \frac{9809}{12 \times .875 \times 21} = 44.5 \text{ p.s.i.}$$

Therefore special anchorage is required.

$$R = \frac{M}{bd^2} = \frac{15596}{21^2} = 35 \qquad (f_c \text{ is very low})$$

Computations continued:

$$A_s = \frac{M}{f_s jd} = \frac{15596}{18000x.875x21} = .047 \text{ sq.in. per in.} \quad \text{Bottom}$$

Use 5/8" square bars @ 6-1/2" c/c.

Gives .0477 sq. inch.

$$u = \frac{44.5 \times 6.5}{3} = 97 \text{ p.s.i.} < 100$$

CUT-OFF WALL (Or Anchorage)

The purpose of the cut-off wall was to bring the factor of safety against sliding up to a universally accepted value of two. There are several places that the cut-off wall could have been located, one of which is locating it under the stem. It was located four feet from the toe, giving the following calculations and results.

Computations:

$$C = \frac{1 + \sin 35^{\circ}}{1 - \sin 35^{\circ}} = 3.7$$

@ C-C, $wh = 2647$ p.s.f.

Possible resistance $= 3.7 \times 2647 = 9794$ p.s.f.

To make sliding factor 2:

$$R = 2 \times 11645 - .499 \times 23023$$

$R = 11{,}802\#$ required.

Min. $h = \dfrac{11802}{9794} = 1.2$ Make $h = 1'\text{-}3''$ for convenience.

$$V = 11{,}802\#$$

$$M = 7.5 \times 11802 = 88{,}515 \text{ lb.in.}$$

$$d = \frac{V}{bjv} = \frac{11802}{12x.875x40} = 28.1''$$

Use 2'-6" for convenience.

If no tension reinforcement:

$$d = \frac{6x88515}{12x60} = 27''$$

STEM DESIGN

By using the same line of reasoning as used in the heel design, the stem was designed. The assumed widths of 12" at the top and 24" at the bottom were confirmed, leaving only the steel left to be found.

The stem schedule that follows the calculations was attained in order to find the necessary steel at each three feet of the stem height. By using these values the curve was plotted and by the use of same, it was then easily found where the steel could be cut off and still give the desired strength.

Computations:

$$P = 1/2 \times 857.7 \times 18.7 = 8{,}019.5\#$$

$$V = 8019.5 \times .819 = 6{,}568\#$$

Moment at Bottom of Stem.

$$M = \frac{6568 \times 18.7 \times 12}{3} = 491{,}286 \text{ in. lbs.}$$

$$d = \frac{V}{bjv} = \frac{6568}{12 \times .875 \times 40} = 15.6"$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{491286}{139 \times 12}} = 17.2"$$

Make stem 12" @ top and 24" @ bottom.

@ any level:

$$V_x = 22.93\ h^2$$

$$M_x = 7.64\ h^3$$

Computations continued:

$$u = \frac{vb}{\Sigma_o} = \frac{40x7}{4} = 70 \text{ p.s.i.}$$

$$\text{Embedment} = L = \frac{f_s}{4u} \times D = \frac{18000}{4x100} = 45''$$

Hook Bottom

Temperature Steel:

(.002 of concrete area)

.002 x 12 x 18 = .44 Sq.in. per ft. of height.

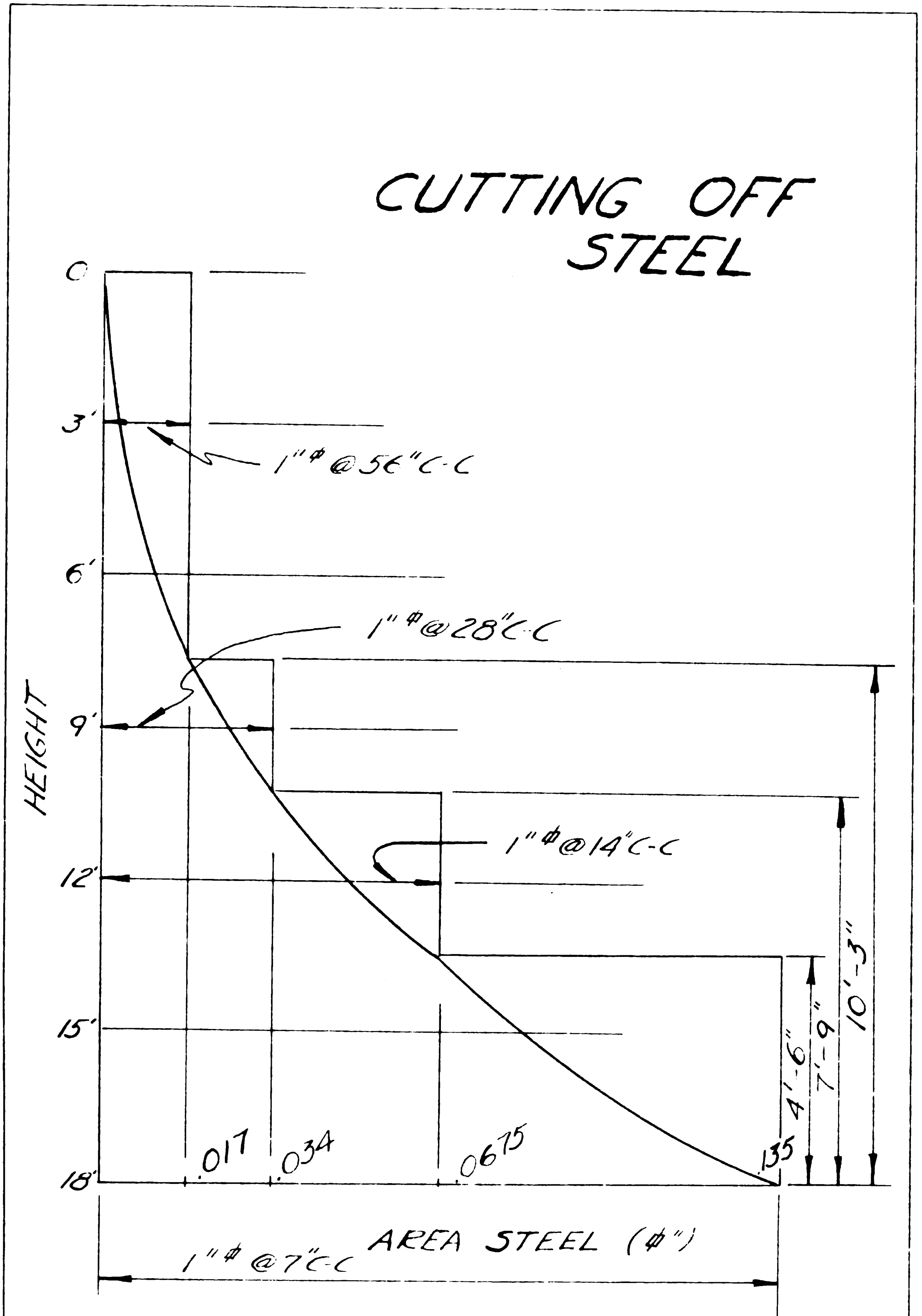
$$\underbrace{\left\{ 1/2'' \ \phi \ @ \ 9'' \ c/c \right\}}_{\text{Front}} + \underbrace{\left\{ 1/2'' \ \phi \ @ \ 12'' \ c/c \right\}}_{\text{Back}} = .46 \text{ Sq. in.}$$

Key:

A general formula of $\frac{3}{2} \frac{V}{A} = K f'_c$ where K ranges from .02 to .12 could be used here. Generally, keys are designed up to 9" and this, with the steel and concrete friction, take the shear present.

STEM SCHEDULE

h	V	d	v	M	R	A_s	RODS
3'	206	11"	1+	206	1+	.00119	
6'	825	13"	6+	1,650	9+	.00806	
9'	1857	15"	11+	5,570	24+	.0236	SEE SKETCH
12'	3302	17"	18+	13,202	45+	.0493	
15'	5159	19"	25+	25,785	71+	.0862	
18'	7429	21"	33+	44,556	101	.135	1"□@7"=.143□"


CUTTING OFF
STEEL
0
3'
6'
9'
12'
15'
18'
HEIGHT
1"φ @ 56" C-C
1"φ @ 28" C-C
1"φ @ 14" C-C
.017
.034
.0675
.135
4'-6"
7'-9"
10'-3"
AREA STEEL (φ")
1"φ @ 7" C-C

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JOINTS

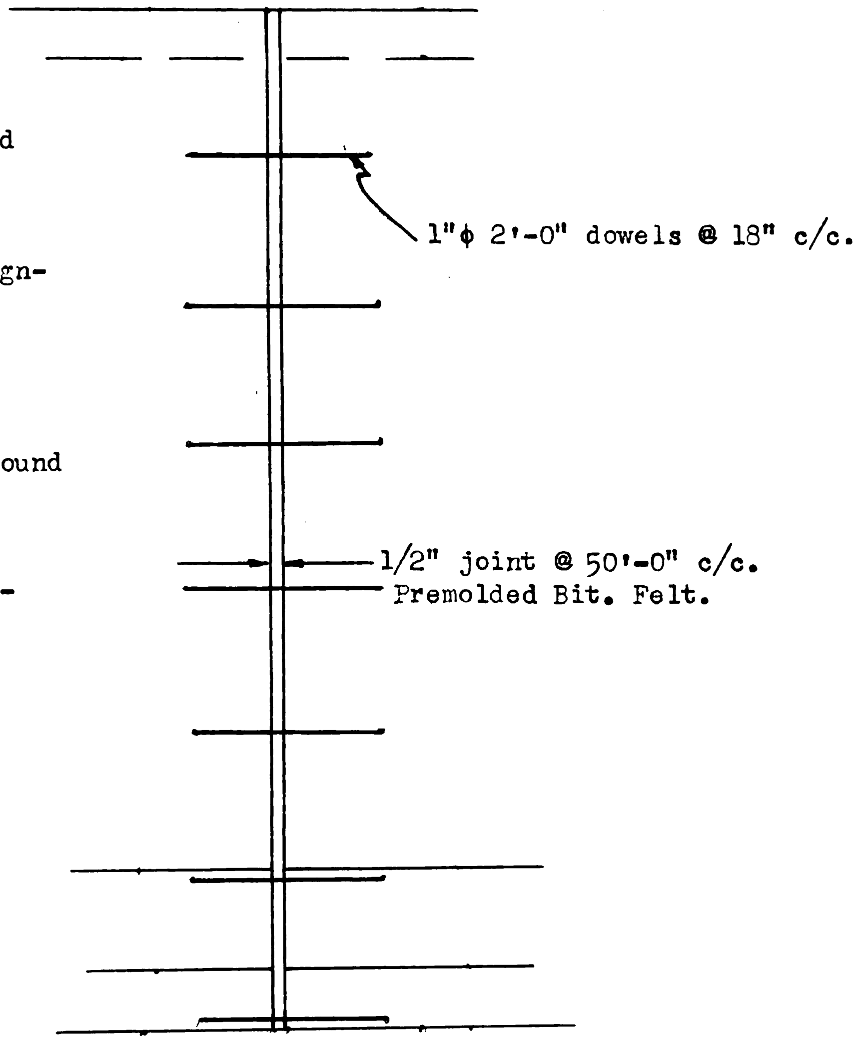
Coefficient expansion for reinforced concrete equals .000006 for 1° F. Therefore, maximum expected expansion for a 50 ft. length equals .21". (Assume 60° F. change of temperature). In this design a .5" expansion is allowed.

At each joint use 1" round steel dowels encased in tubes to hold wall in alignment.

Space dowels as shown.

Encase dowels in 1-1/8" round tubes.

Joint will be of 1/2" premolded bituminous felt.



Expansion Joint - Construction Suggestions:

1. Stop steel at joint form.

2. Place dowell tubes before concrete hardens.

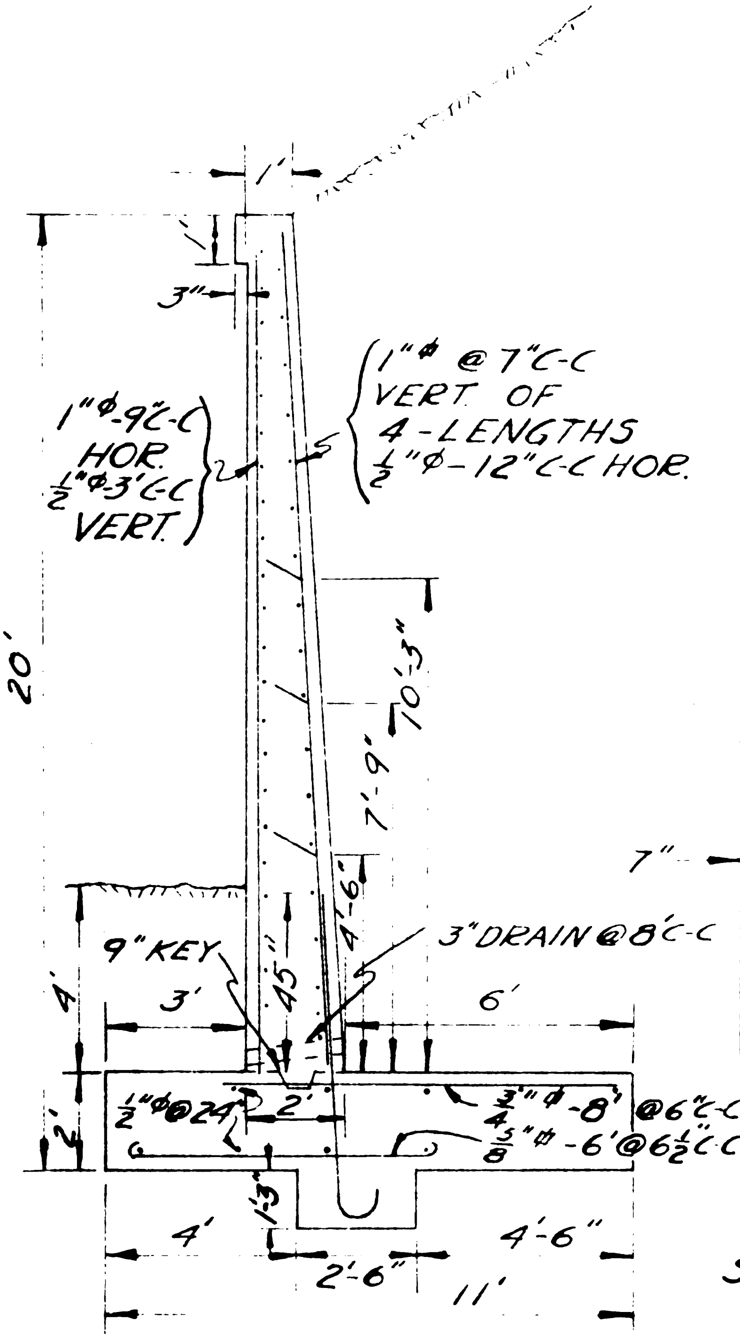
3. Copper plating can be used to prevent water flow through joint. This will do away with wall staining from bituminous material.

CONCLUSION

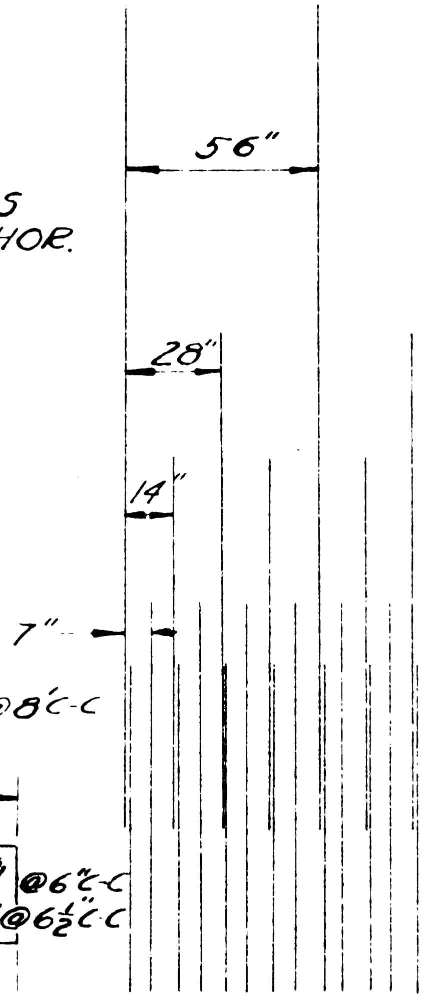
The finished cantilever wall shown on the following page was properly designed and is safe against destruction under the conditions previously mentioned. The economy is good.

Before definite acceptance or rejection is in order the whole picture must be drawn. Thus, we move on to the other conditions.

CANTILEVER WALL



SCALE - 1" = 4'



STEM STEEL PLACING

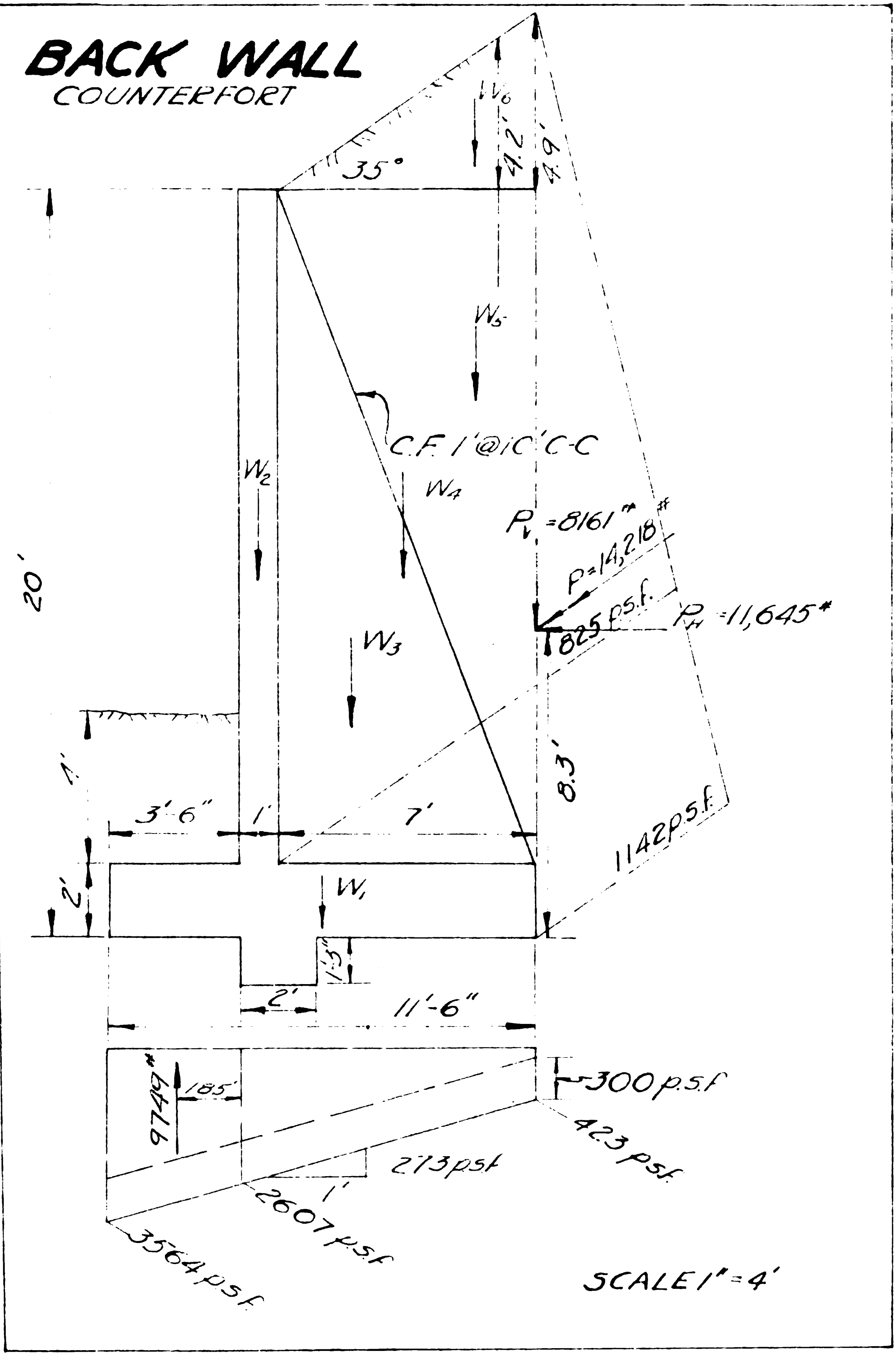
SCALE 1" - 4'8"

STUDY OF THE POSSIBILITIES

OF USING A

COUNTERFORT BACKWALL

DESIGN


BACK WALL
COUNTERFORT
35°
W_6
4.2'
4.9'
W_5
C.F. 1'@10' C-C
W_2
W_4
P_V = 8161#
P = 14,218#
825 p.s.f.
P_H = 11,645#
W_3
20'
1'
8.3'
3'-6"
1'
7'
1142 p.s.f.
2'
W_1
1'-3"
2'
11'-6"
9749#
1.85'
300 p.s.f.
423 psf.
273 psf
1'
2607 p.s.f.
3564 p.s.f.
SCALE 1" = 4'

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With only slight variation the preliminary investigation of the counterfort wall was the same as for the cantilever wall. Sliding, overturning, and excessive soil pressures would cause failure as before, but also it was determined that failure could occur through tearing of the counterfort from the stem, tearing of the counterfort from the base, and bending of the stem both parallel and perpendicular to the base.

The wall was made 20 feet high as before (see diagram). This repitition allowed for a better cost comparison. The base was again placed far enough below ground to defeat frost action. The base was made 11-1/2 feet long, the stem was made 12 inches thick, counterforts 12 inches thick (1/20 of the height) and spaced at 10 feet. These all conform to general practice.

To determine whether the wall would fail as a unit, Rankine's theories were used. To be consistent with the previous design, variations as set forth by Taylor, Smulski and Thompson were employed where necessary.

The resultant of the pressures on the base fell within the middle one-third (required in retaining wall design) and the soil pressure at the toe was below the determined maximum of 4,250 p.s.f. The sliding factor of safety did not equal two and thus anchorage was required.

Expansion joints were employed as before.

Again, if actual construction is contemplated (to the present knowledge of the authors it is not) more thorough study of the subsoil should be conducted.

Computations:

$C = .819$

$P_{top} = .819 \times 4.9 \times 56 = 224.7$ p.s.f.

$P_{bot} = .819 \times 24.9 \times 56 = 1,142.0$ p.s.f.

$P = 1/2 \times 24.9 \times 1142 = 14,218\#$

$P_h = .819 \times 14218 = 11,645\#$

$P_v = .574 \times 14218 = 8,161\#$

$x = 1/3 \times 24.9 = 8.30'$

Base Pressure: Moment About Toe:

$W_1 = 11.5 \times 2 \times 150 \times 10$	$= 34,500\#$	x	$5.75'$	$= 198,375'\#$
$W_2 = 1 \times 18 \times 150 \times 10$	$= 27,000$	x	4.00	$= 108,000$
$W_3 = 3.5 \times 18 \times 150 \times 1$	$= 9,450$	x	6.834	$= 64,581$
$W_4 = 7 \times 18 \times 56 \times 9$	$= 63,504$	x	8.000	$= 508,032$
$W_5 = 3.5 \times 18 \times 56 \times 1$	$= 3,528$	x	9.167	$= 32,341$
$W_6 = 3.5 \times 4.9 \times 56 \times 10$	$= 9,604$	x	9.167	$= 88,040$
	$147,586\#$			$999,369'\#$
$P_v \times 10$	$= 81,610$	x	11.5	$= 938,515$
	$229,196\#$			$1,937,884'\#$
$P_h(d) = -11645 \times 10 \times 8.3$				$= -966,535$
				$971,349'\#$

$$x = \frac{971,349}{229,196} = 4.24'$$

$5.75 - 4.24 = 1.51' = e$

.41 within middle one-third

Computations continued:

$$f = \frac{P}{A}\left(1 \pm \frac{6e}{b}\right)$$

$$f = \frac{229,196}{11.5 \times 10}\left(1 \pm \frac{6 \times 1.51}{11.5}\right)$$

$$f = 1993\ (1 \pm .788)$$

f toe = 3564 p.s.f.

f heel = 423 p.s.f.

Overturning Factor of Safety:

$$F.S. = \frac{1937884}{966535} = 2$$

Sliding Factor of Safety:

$$F.S. = \frac{229196 \times .499}{116450} = .98$$ Therefore must anchor.

VERTICLE SLAB

As explained in the cantilever backwall design, the unit pressure on any section of the wall caused by the backfill was found by the pressure triangle. The pressure is represented by a force acting through the centroid of the triangle parallel to the backfill surface at a point common to the backwall and the line of action of the force.

The horizontal component of this unit pressure equals w and the moment designed for at any interior panel is given by the formula, $M = 1/12\ wL^2$. For a panel bordering an expansion joint, or an end panel $M = 1/10\ wL^2$. L is the clear span. To be consistent the wall was designed for its entire length as an exterior panel, thus given additional strength.

By correlating the results obtained in the following chart the steel design was accomplished.

Computations:

$$M = 1/10\ wL^2$$

$$M = 1/10 \times 676.1 \times 9^2 = 5476.4'\# \text{ or } 65{,}717''\#$$

$$V = \frac{wL}{2} = \frac{676.1 \times 9}{2} = 3{,}042\#$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{65717}{139 \times 12}} = 6.3''$$

$$d = \frac{V}{vjd} = \frac{3042}{40 \times .875 \times 12} = 7.2''$$

$$A_s = \frac{M}{f_s jd} = \frac{65717}{18000 \times .9 \times 9} = .451 \text{ sq.in. per ft.} \qquad \text{Assumed } j = .9$$

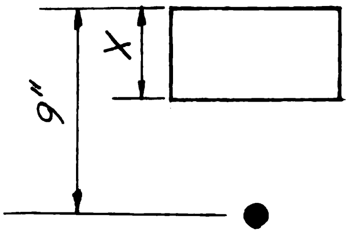
Use 5/8 inch round bars @ 8" c/c.
Gives .465 sq. inches.

Computations continued:

$$u = \frac{V}{\Sigma_o jd} = \frac{3042}{\pi\ x .625 x 1.5 x .9 x 9} = 127.5 \text{ p.s.i.}$$

Therefore must anchor.

Check j:



$$x\ (12)\ \frac{x}{2} = 9 - x\ (.465\ (15)\ )$$

$$6x^2 = 9 - 6.975x$$

$$x^2 + 1.163x = 9$$

$$(x + .5815)^2 = 9.338$$

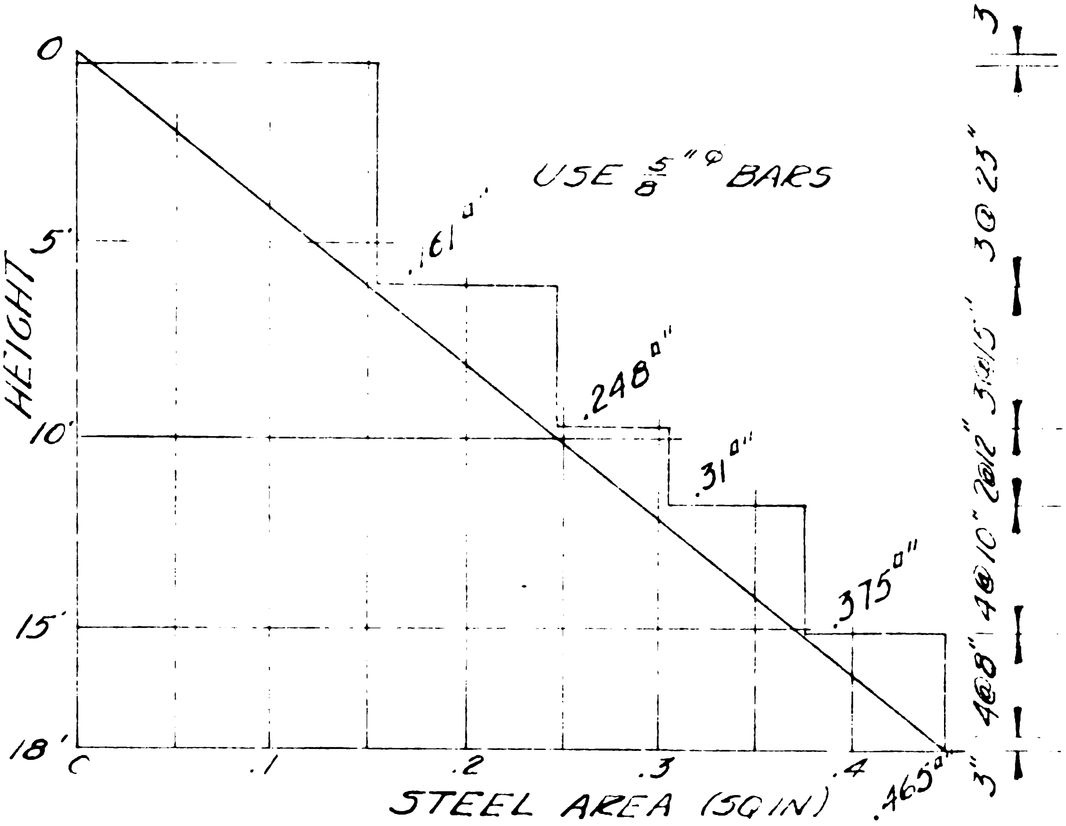
$$x + .5815 = 3.055$$

$$x = 2.47$$

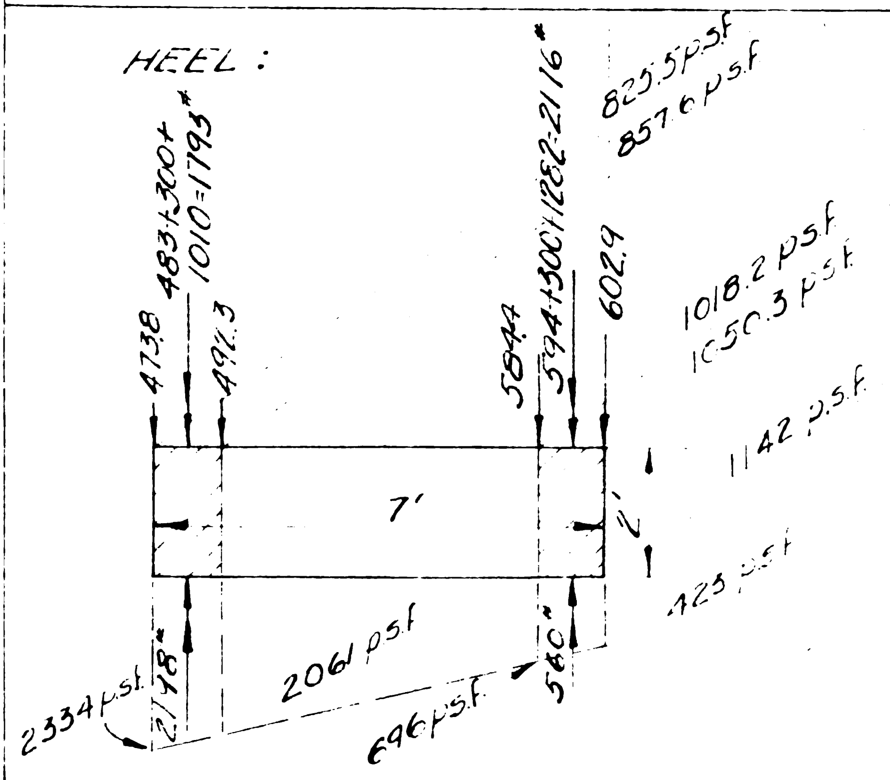
$$j = \frac{9 - 1/3 \times 2.47}{9} = .909$$

OK Assumed .9

VERTICAL SLAB STEEL:



HEEL:



HEEL SLAB

The moments and shear on the heel were found by the same formula as used for the verticle wall. The shear at the extremity of the heel and at the back wall are different and were found to be of opposite direction. Note that there is a point where this shear is zero. From the graph it was found that the steel should be placed as shown by the diagram that follows. The heel design was conducted as shown in the computations.

Computations:

One foot on right end of heel:

$$M = 1/10\ wL^2$$

$$M = 1/10 \times 1616 \times 9^2 = 13{,}090'\# \text{ or } 157{,}080\ ''\#$$

$$V = \frac{wL}{2} = \frac{1616 \times 9}{2} = 7{,}272\#$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{157080}{139 \times 12}} = 9.7''$$

$$d = \frac{V}{bjd} = \frac{7272}{12 \times .875 \times 40} = 17.3'' \qquad \text{OK}$$

$$A_s = \frac{M}{f_s jd} = \frac{157080}{18000 \times .9 \times 21} = .462 \text{ sq.in. per ft.}$$

Assumed $j = .9$

Use 1/2 inch square bars @ 6-1/2" c/c.
Gives .462 sq. inches.

One foot on left end of heel:

$$M = 1/10\ wL^2$$

$$M = 1/10 \times 405 \times 9^2 = 3{,}281'\# \text{ or } 39{,}372''\#$$

$$V = \frac{wL}{2} = \frac{405 \times 9}{2} = 1{,}823\#$$

Computations continued:

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{39372}{139x12}} = 4.9''$$

$$d = \frac{V}{bjd} = \frac{1823}{12x.875x40} = 4.3''$$

$$A_s = \frac{M}{f_s jd} = \frac{39372}{13000x.9x21} = .1157 \text{ sq.in. per ft.}$$

Assumed $j = .9$

Use 1/2 inch square bars @ 25" c/c.
Gives .120 sq. inches.

Check j :

$$x\ (12)\ \frac{x}{2} = 21 - x\ (.461\ (15)\ )$$

$$6x^2 = 21 - .6915x$$

$$x^2 + 1.153 = 3.5$$

$$(x + .5765)^2 = 3.5 + .332$$

$$x + .5765 = 1.96$$

$$x = .461$$

$$j = \frac{21 - .461}{21} = .97$$

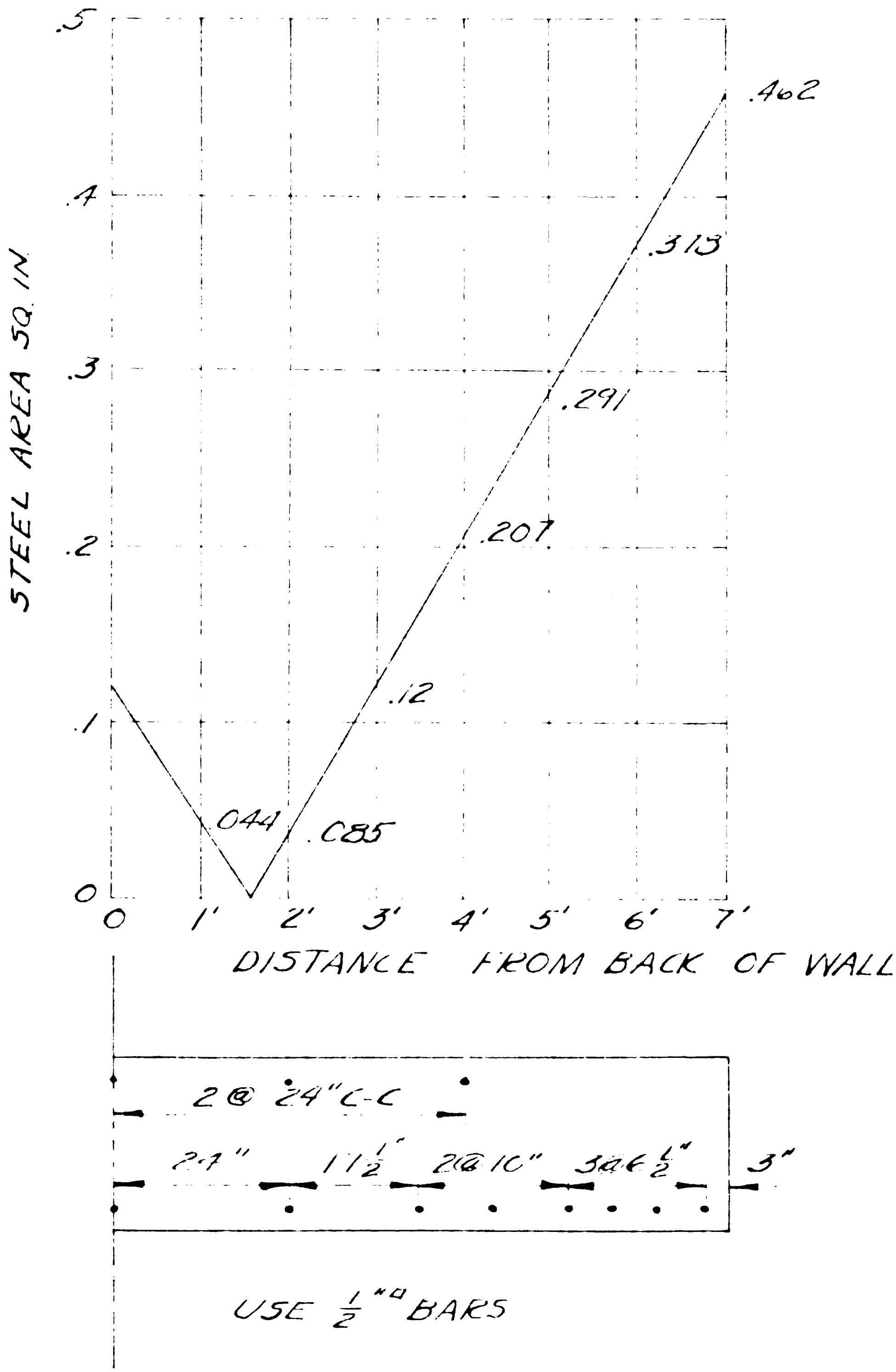
OK Assumed .9

$$u = \frac{V}{\Sigma_o\ jd} = \frac{7272}{2x1.846x.875x21} = 108.2 \text{ p.s.i.}$$

Therefore must anchor.

HEEL

STEEL PLACEMENT



TOE SLAB

The shear and moment on the toe were figured neglecting the effect of the earth above the toe. This earth will not be present during construction and therefore was not taken into account. Note that special anchorage of the steel was required. In the base design a value of j was assumed and then was checked and found to be all right.

Computations:

$$V = \left(\frac{3264 \quad 2307}{2}\right) 3.5 = 9{,}749\#$$

$$M = 9749 \times 1.85 = 18{,}036'\#$$

$$d = \frac{V}{bjd} = \frac{9749}{12\text{x}.875\text{x}60} = 15.5'' \qquad \text{Therefore must anchor.}$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{18036}{139}} = 11.4''$$

$$A_s = \frac{M}{f_s jd} = \frac{18036}{18000\text{x}.9\text{x}21} = .053 \text{ sq.in. per in.}$$

Assumed $j = .9$

Use 5/8 inch round bars @ 5-1/2" c/c.
Gives .0564 sq. inches.

$$u = \frac{V}{\Sigma_o \; jd} = \frac{9749}{\pi \text{x}.625\text{x}2.182\text{x}.875\text{x}21} = 124 \text{ p.s.i.}$$

Therefore must anchor.

Check j:

$$x\ (12)\frac{x}{2} = 21 - x\ (.676\ (15))$$

$$6x^2 = 21 - 10.14x$$

$$x^2 + 1.69x = 3.5$$

$$(x + .845)^2 = 3.5 + 7.14$$

$$x = .4034$$

$$j = \frac{21 - .4034}{21} = .98 \qquad \text{OK Assumed } .9$$

COUNTERFORTS

The tendency of the counterfort to tear was the next consideration. This tendency was checked by steel bars vertically placed in the counterfort 3 inches from the back and running from the top of the wall to the back of the heel. These bars were anchored where necessary. The d used in figuring the area of the steel was measured from the front of the wall at the base.

Similar computations were carried out at various sections and by correlating the results into the graph shown, the required steel at any point and the steel cut-off was determined.

It was also necessary to add steel to tie the counterfort to the verticle and to the heel slabs. This was done as shown in the computations taking care of the embeddedment required.

To assist in the protection of the counterforts against the coal scoop it was decided that 4" x 4" x 1/4 x 19'-3" angles (or their equivalent) should be placed on the back of the counterforts.

Computations:

$$V = 1/2 \times 676 \times 18 \times 10 = 60{,}840\#$$

$$M = 1/3 \times 18 \times 60840 = 365{,}040'\#$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{365040}{139}} = 52.2" \quad \text{OK} \quad 86.3" \text{ available.}$$

$$A_s = \frac{M}{f_s jd} = \frac{365040 \times 12}{18000 \times .9 \times 86.5} = 3.12 \text{ sq. in.}$$

Use 1-1 inch round bar
and 4-7/8 inch round bars, spaced as shown on final drawing.

Gives 3.185 sq. inches.

Computations continued:

This spacing decreases d by a small amount. Excess of A_s is sufficient, however, to take care of this. This is shown below. (Also see final drawing of counterfort wall).

$$\text{New } d = \frac{1.985 \times 3 + 1.2 \times 5.7}{3.185} = 4.017'' \qquad 89.5 - 4.02 = 85.5''$$

$$\text{New } A_s = \frac{M}{f_s jd} = \frac{365040}{18000 x .9 x 85.5} = 3.163 \text{ sq.in.}$$

OK, since this is less than that present.

$$u = \frac{V}{\Sigma_o jd} = \frac{60840}{x4x.875x85.5} = 64.7 \text{ p.s.i OK}$$

Steel Cut-Off (6' from top of base):

$$V = \frac{12 \times 10 \times 451}{2} = 27{,}060\#$$

$$M = 27060 \times 4 = 108{,}240'\#$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{108240}{139}} = 28'' \qquad \text{OK} \qquad 60.4'' \text{ available.}$$

$$A_s = \frac{M}{f_s jd} = \frac{108240 \times 12}{18000 x .9 x 60.4} = 1.33 \text{ sq.in.}$$

Use 2 bars 1 inch round, spaced at 6" c/c.
Gives 1.57 sq. inches.

$$u = \frac{V}{\Sigma_o jd} = \frac{27060}{2x.875x60.4x\pi} = 81.5 \text{ p.s.i.} \qquad \text{OK}$$

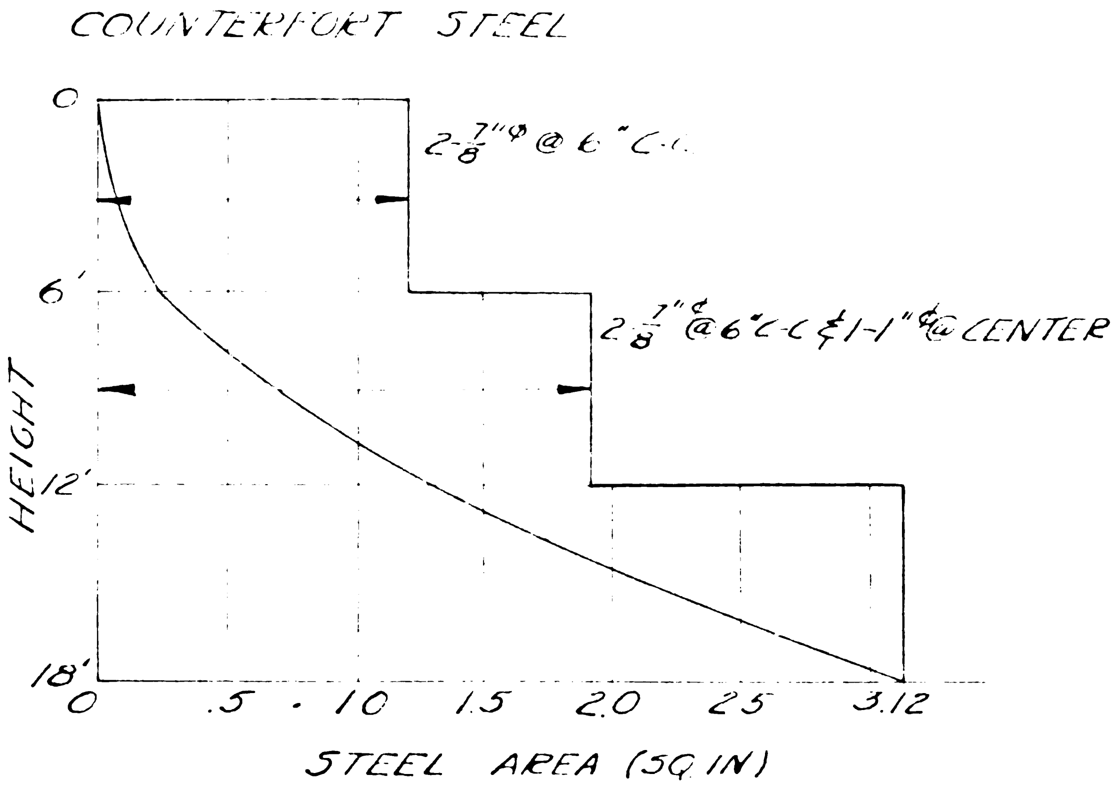
Steel Cut-Off (12' from top of base):

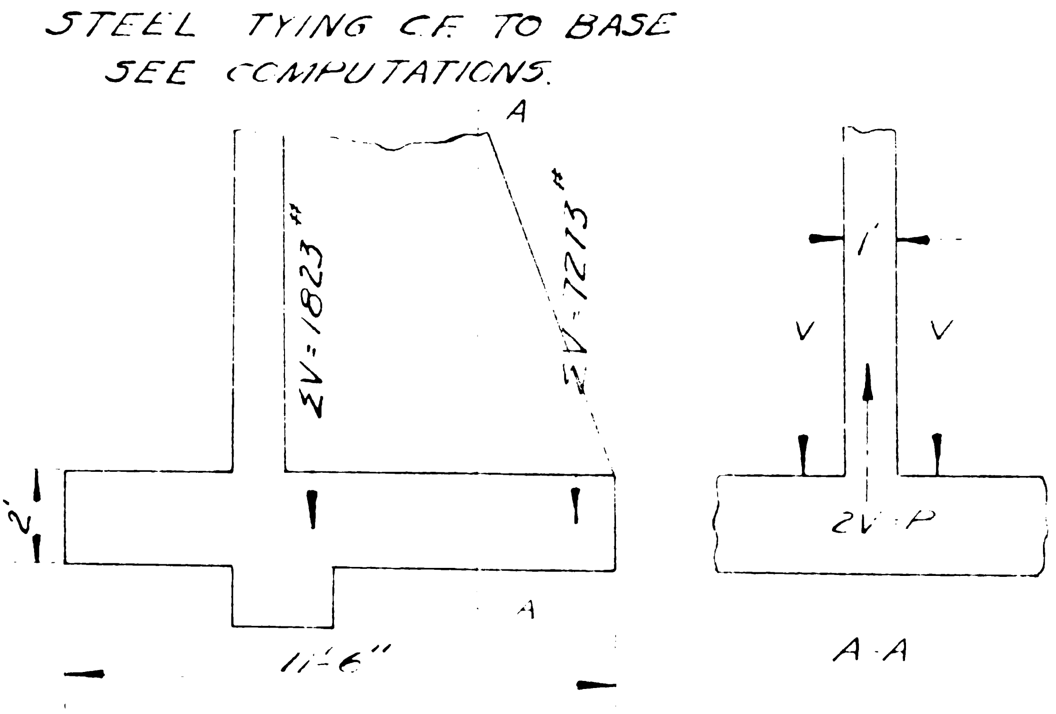
$$V = 1/2 \times 275 \times 6 \times 10 = 8{,}250\#$$

$$M = 8250 \times 2 = 16{,}500'\#$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{16500}{139}} = 10.8'' \qquad \text{OK} \qquad 34.3'' \text{ available.}$$

$$A_s = \frac{M}{f_s jd} = \frac{16500 \times 12}{18000 x .9 x 60.4} = .2 \text{ sq.in.}$$


COUNTERFORT STEEL
0
6'
12'
18'
HEIGHT
2-7/8"φ @ 6" C-C
2-7/8"φ @ 6" C-C & 1-1"φ @ CENTER
0
.5
1.0
1.5
2.0
2.5
3.12
STEEL AREA (SQ IN)


STEEL TYING C.F. TO BASE
SEE COMPUTATIONS.
A
ΣV=1823#
ΣV=1213#
2'
A
11'-6"
1'
V
V
2V=P
A-A

Computations continued:

Steel Tying Vertical Slab to Counterfort:

$$V = 676 \times 9 = 6{,}084\#$$

$$A_s = \frac{6084}{18000} = .338 \text{ sq.in. per ft.}$$

Assume hook on every bar.

$$A = \frac{.338 \times 8}{12 \times 2} = .113 \text{ sq.in.}$$

Use 3/8 in.sq. bars.
Gives .14 sq. inches.

$$L = \frac{f_s d}{4u} = \frac{18000 \times 3/8}{4 \times 100} = 16.8''$$

Steel Tying Counterfort to Base:

At Heel:

$$A_s = \frac{14546}{18000} = .808 \text{ sq.in.}$$

$$A = \frac{12 \times .5}{6.5} = .83$$

Use 2-1/2 in.sq. bars on every horizontal bar.

$$L = \frac{f_s d}{4u} = \frac{18000 \times 1/2}{4 \times 100} = 22.5''$$

Key:

Again the general formula $\frac{3}{2}\ \frac{V}{A} = K\ f'_c$ could be used. A 9 inch key will be used to take the shear that is present.

CUT-OFF WALL (Or Anchorage)

Since the safety factor was too small (must equal two) a cut-off wall was required (see drawing). This involved the passive pressure given by the formula:

$$P = wh\left(\frac{1+\sin\phi}{1-\sin\phi}\right)$$

where wh equals the soil pressure at the face of the wall and ϕ equals the internal angle of friction of the coal. The required dimensions of the wall were found by the method shown in the computations.

Computations:

$$P = wh\left(\frac{1+\sin\phi}{1-\sin\phi}\right) = 3.7\ wh$$

$$= 2607 \times 3.7 = 9{,}646 \text{ p.s.f.}$$

To make S.F. = 2

$$R = 2 \times 10 \times 11645 - .499 \times 229196$$

$$R = 118{,}531\#$$

$$\text{Min. } h = \frac{118531}{10 \times 9646} = 1.22'$$ Use 1'-3" for convenience.

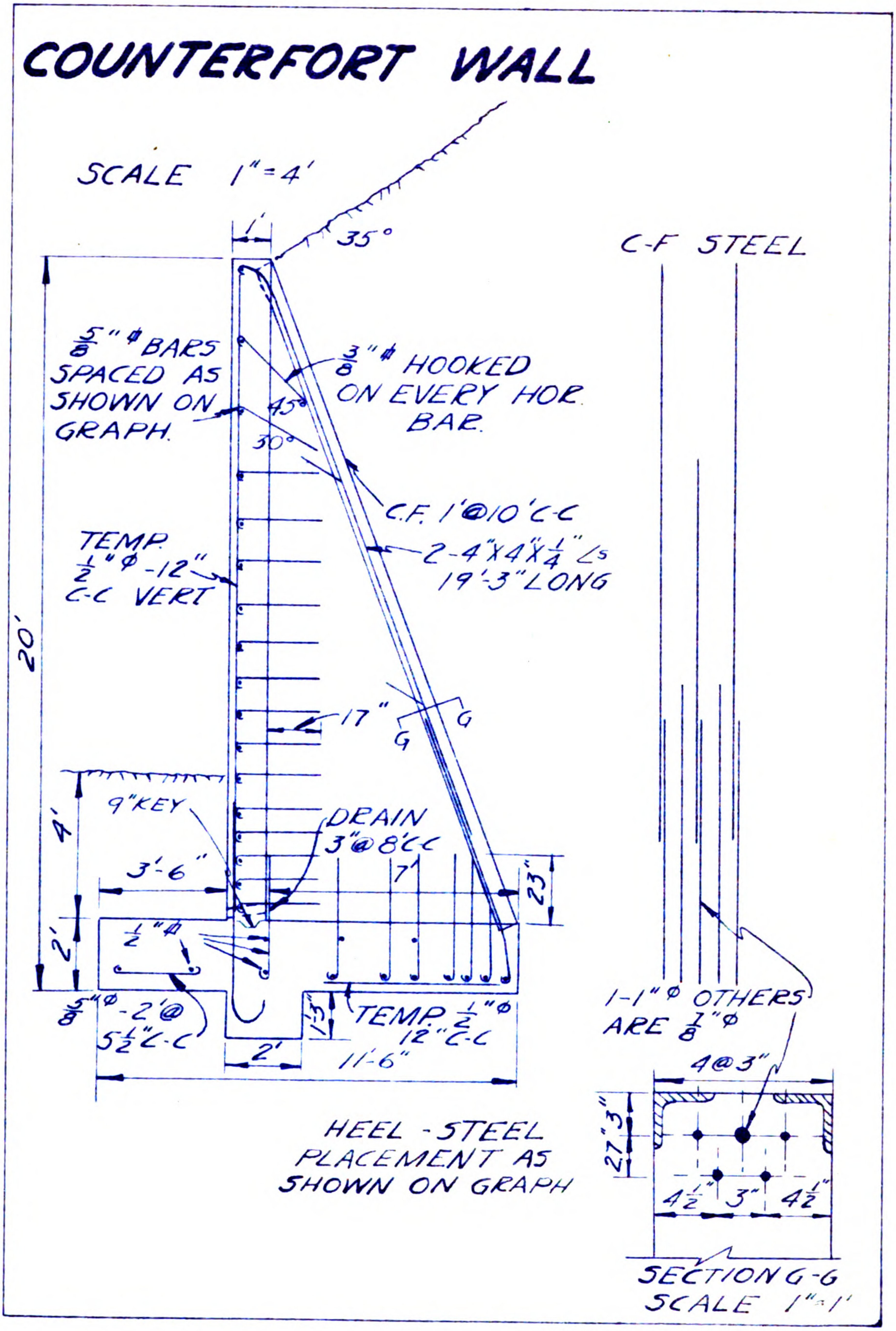
At front of wall:

$$V = 118{,}531\#$$

$$d = \frac{V}{bjv} = \frac{118531}{12 \times .875 \times 60} = 19''$$

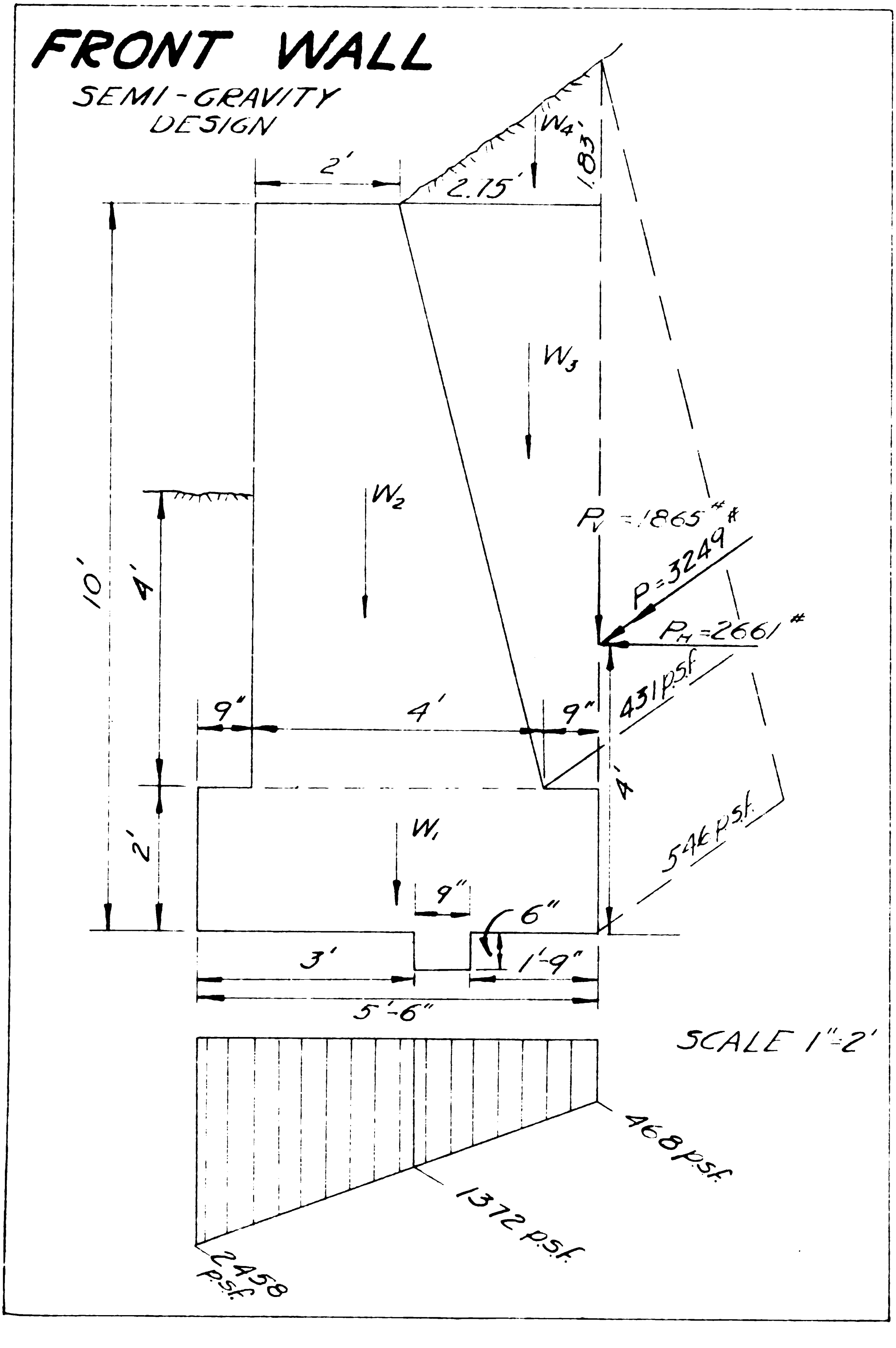
SUMMARY

By the previous computations the assumed dimensions were checked and found to be suitable. The back wall was designed cantilever and now counterfort. A cost comparison could now be made but it would be of little significance. It was decided that the front and side walls should be included and then a cost comparison be computed. The variables, however, have been designed and the rest will be constant in the comparison.


COUNTERFORT WALL
SCALE 1"=4'
1'
35°
5/8"φ BARS SPACED AS SHOWN ON GRAPH.
3/8"φ HOOKED ON EVERY HOR. BAR.
45°
30°
C.F. 1'@10'C-C
2-4"X4"X1/4" ∠s 19'-3" LONG
TEMP. 1/2"φ-12" C-C VERT.
20'
17"
G
G
4'
9" KEY
DRAIN 3"@8'C-C
3'-6"
7'
23"
2'
1/2"φ
5/8"φ-2'@ 5 1/2"C-C
1'-3"
TEMP. 1/2"φ 12"C-C
2'
11'-6"
HEEL-STEEL PLACEMENT AS SHOWN ON GRAPH
C-F STEEL
1-1"φ OTHERS ARE 7/8"φ
4@3"
27"
3"
4 1/2"
3"
4 1/2"
SECTION G-G
SCALE 1"=1'

FRONT WALL DESIGN

(Semi-Gravity)


FRONT WALL
SEMI-GRAVITY
DESIGN
2'
2.75'
W_4
1.83'
W_3
W_2
P_V = 1865 #
P = 3249
P_H = 2661 #
431 p.s.f.
10'
4'
9"
4'
9"
4'
2'
W_1
546 p.s.f.
9"
6"
3'
1'-9"
5'-6"
SCALE 1" = 2'
468 p.s.f.
1372 p.s.f.
2458 p.s.f.

To allow the shell to easily clear the front wall it was, of necessity, designed so that the top of the front wall was well below the crane. The wall, being ten feet high with four feet above the ground, could have been designed either gravity or cantilever.

It was decided that a semi-gravity wall would be the most practical solution. This wall must be stable in the same respects as all other walls. Excessive soil pressures, sliding, overturning, and shear or tearing of members would cause failure as before.

Expansion joints were placed every fifty feet as in the previous designs. The only steel required in this semi-gravity wall was that shown in the stem. This steel eliminates the tendency of the stem to tear away from the base.

The toe and heel did not need investigation because of the limited dimensions involved. The remainder of the design was repitition of the foregoing design and it is believed that the reader can follow the computations without further discussion.

Computations:

$C = .819$

$P_{top} = .819\ wh$

$= .819 \times 56 \times 1.9$

$= 87\#$

$P_{bot} = .819 \times 56 \times 11.9$

$= 546\#$

$P = 1/2 \times 11.9 \times 546 = 3{,}249\#$

$P_h = .819 \times 3249 = 2{,}661\#$

$P_v = .574 \times 3249 = 1{,}865\#$

Base Pressures: Moment About Toe:

$W_1 = 5.5 \times 2 \times 150$	$= 1{,}650\#$	x	2.75'	$= 4{,}538'\#$
$W_2 = 3 \times 8 \times 150$	$= 3{,}600$	x	2.30	$= 8{,}280$
$W_3 = 1/2 \times 3.5 \times 8 \times 56$	$= 784$	x	4.53	$= 3{,}552$
$W_4 = 1/2 \times 2.75 \times 1.93 \times 56$	$= 149$	x	4.58	$= 682$
	6,183#			17,052'#
P_v	$= 1{,}865$	x	5.50	$= 10{,}257$
	8,048#			27,309'#
	$P_h(d) = -2661 \times 4$			$= -10{,}644$
				16,665'#

$$x = \frac{16665}{8048} = 2.07'$$

$2.75 - 2.07 = .68' = e$

.24' within middle one-third

Computations continued:

Soil Pressures:

$$f = \frac{P}{A}\left(1 \pm \frac{6e}{b}\right)$$

$$= \frac{8048}{5.5}\left(1 \pm \frac{6 \times .68}{6}\right)$$

f toe $= 1463 \times 1.68 = 2{,}458$ p.s.f.

f heel$= 1463 \times .32 = 468$ p.s.f.

Sliding Factor of Safety:

$$F.S. = \frac{8048 \times .499}{2661} = 1.51$$

Therefore must anchor.

Cut-Off

Passive $P = 3.7$ wh

@ face wh $= 1{,}372$ p.s.f.

Possible $R = 1372 \times 3.7 = 5{,}076\#$

To make S.F. $= 2$

$R = 2 \times 2661 - .499 \times 8048$

$R = 1{,}297\#$

Min. $h = \frac{1297}{5076} = .26'$ Make $h = 6"$ for convenience.

On face of cut-off:

$V = 1{,}297\#$

$M = 1297 \times 3 = 3{,}891\#"$

$d = \sqrt{\frac{6M}{bf}} = \sqrt{\frac{6 \times 3891}{12 \times 60}} = 5.7"$ Make $d = 9"$ for convenience.

Shear of Cut-off:

$$\frac{3}{2}\ \frac{V}{A} = \frac{3 \times 1297}{2 \times 12 \times 9} = 18 \text{ p.s.i.}$$

Computations continued:

Stem Steel Design:

$d = 43.8''$

Shear at base of stem:

$p = .819 \times 56 \times 9.4$

$= 431$ p.s.f.

$V = 1/2 \times 431 \times 9.4$

$= 2{,}026\#$

$M = 2026 \times 2.7 = 5{,}470$ ft.lbs. or $65{,}640$ in.lbs.

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{65640}{139 \times 12}} = 6.3''$$ 43.8" available.

$$A_s = \frac{M}{f_s jd} = \frac{65640}{18000 \times .9 \times 43.8} = .09 \text{ sq.in. per ft.}$$

Use 3/8 inch round bars @ 14-1/2" c/c.

Bond:

$$u = \frac{V}{\Sigma_o\, jd} = \frac{2026}{.375\text{x} \quad \text{x}.829\text{x}.875\text{x}43.8} = 54.7 \text{ p.s.i.} \qquad \text{OK}$$

Anchorage:

$$L = \frac{f_s d}{4u} = \frac{18000 \times 3/8}{4 \times 100} = 16.8''$$

Anchor 17" as shown in sketch.

Temperature steel:

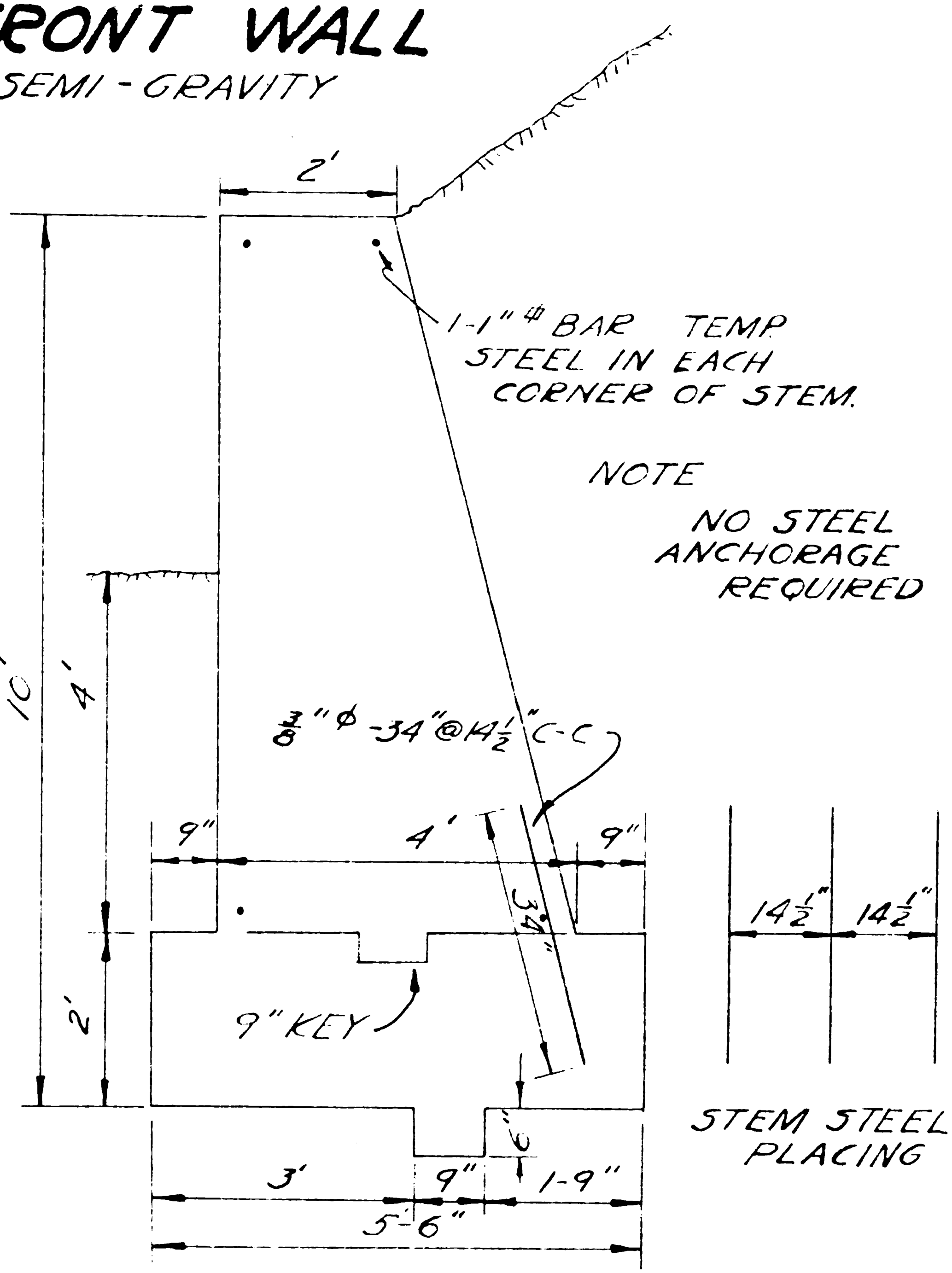
Use 1" sq. bars in the corners to prevent cracking.

This is the only steel required in the front wall.

This design will be used for the front 15' of the north side wall and the front 13' on the south side wall.

FRONT WALL

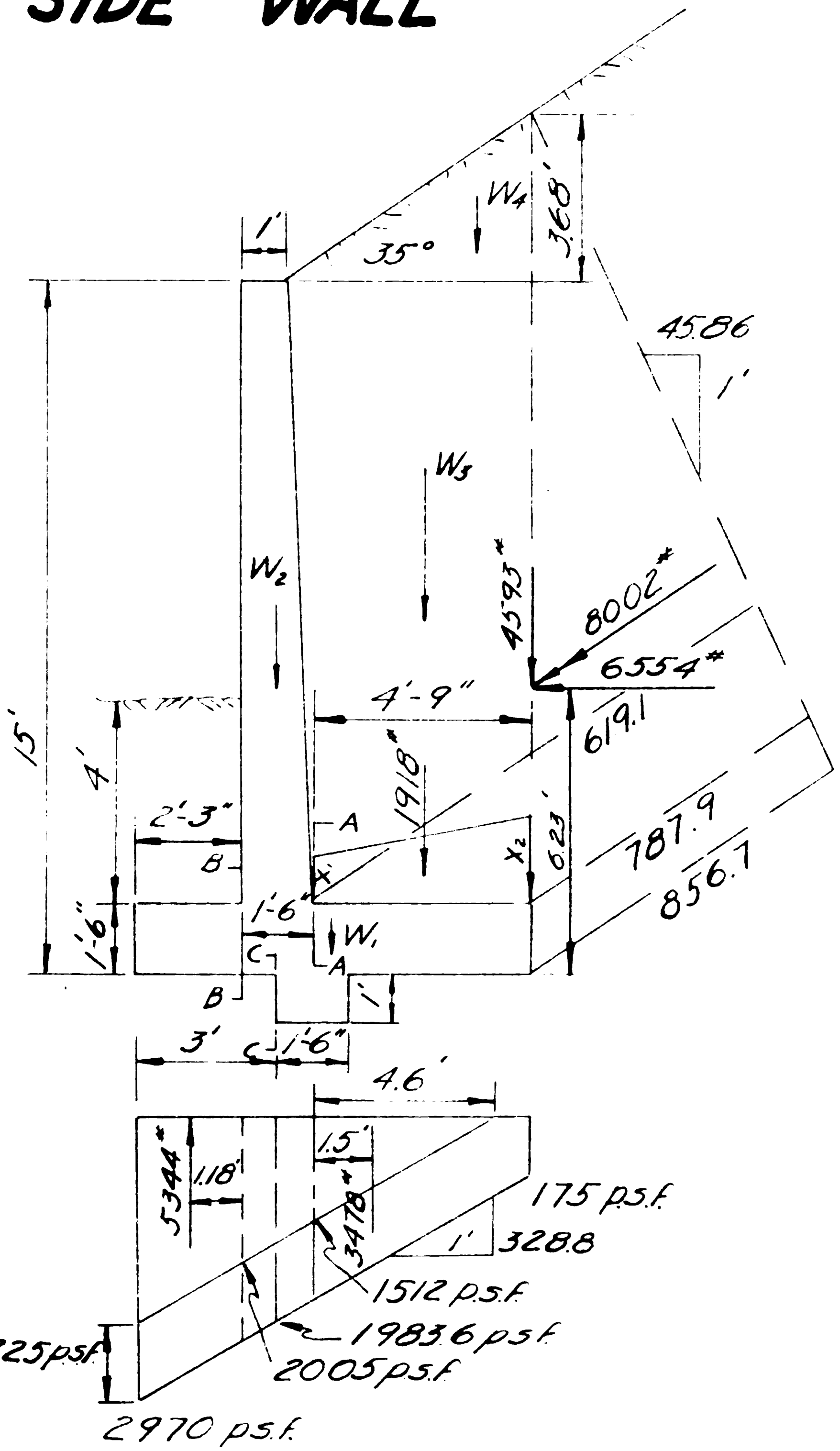
SEMI - GRAVITY



SCALE 1" = 2'

SIDE WALL DESIGN

SIDE WALL



SCALE 1" = 4'

The design of the side walls involved a decision as to the best method to change the elevation of the wall above the ground from 14 feet at the back to 4 feet at the front. A three elevation system was the outcome. The north side (being 60 feet in length) has the front 20 feet of semi-gravity design the same as the front wall. The back section of 20 feet was made cantilever and the design was the same as the cantilever back wall.

The south side (being 58 feet in length) has a similar arrangement, but the front section as described above was made only 18 feet.

It was the 20 foot long center section that had to be investigated. The design was carried out according to Rankine's principles with variations as needed. Expansion joints were not used. The height was made 15 feet making a step, both up and down, of 5 feet. This height made a gravity design uneconomical. Counterforts would give poor design because of probable damage to them by the coal scoop. The only remaining and economical solution was the cantilever wall, and, using constants and knowledge gained from the previous cantilever design, the side wall design continued. The following is a discussion of the principles and computations involved in each portion of the wall.

Computations:

$p = .819 \times 56 \times 18.68 = 856.7$ p.s.f.

$P = 1/2 \times 856.7 \times 18.68 = 8002\#$

$P_h = 6{,}554\#$

$P_v = 4{,}593\#$

$x = 1/3 \times 18.68 = 6.23'$

Base Pressures: Moments About Toe:

$W_1 = 8.5 \times 1.5 \times 150$	$= 1{,}912.5\#$	x	4.25'	$= 8{,}128'\#$
$W_2 = 13.5 \times 1.25 \times 150$	$= 2{,}531.3$	x	2.88	$= 7{,}290$
$W_3 = 13.5 \times 5.0 \times 56$	$= 3{,}780.0$	x	6.0	$= 22{,}680$
$W_4 = 1/2 \times 5.25 \times 3.68 \times 56$	$= 541.0$	x	6.75	$= 3{,}652$
	$8{,}764.8\#$			$41{,}750'\#$
P_v	$= 4{,}593.0$	x	8.5	$= 39{,}041$
	$13{,}357.8\#$			$80{,}791'\#$
$P_h(d) = -\ 6554 \times 6.23$				$= -40{,}831$
				$39{,}960'\#$

$$x = \frac{39960}{13357.8} = 2.99'$$

$4.25 - 2.99 = 1.26 = e$

.16 within middle one-third.

$$f = \frac{P}{A}\left(1 \pm \frac{6e}{b}\right)$$

$$= \frac{13358}{8.5}\left(1 \pm \frac{6 \times 1.26}{8.5}\right)$$

f toe = 2970 p.s.f.

f heel = 175 p.s.f.

$$S.F. = \frac{13358 \times .499}{6554} = 1.02$$ Therefore must anchor.

HEEL DESIGN

Shear and moments were taken at the section noted and the results used to compute the depth required and the steel needed. In many cases the size and steel used was larger than needed to make the actual construction convenient. In this design the reinforcing bars did not have to be anchored. The unit forces acting on the heel form a trapezoid (as shown) and were found as in the heel design of the cantilever back wall.

Computations:

$$X_1 = 355.4$$

$$X_2 = 452.3$$

$$x = \frac{4.6}{3}\left(\frac{452.3 + 710.8}{807.7}\right) = 2.2'$$

Maximum Shear and Moment on Section A-A:

$W_3 =$	-3,780#	x	3.25'	= -12,285'#
$W_4 =$	- 541	x	3.52	= - 1,904
$P_v =$	-1,918	x	2.55	= - 4,891
Earth P =	+3,478	x	1.53	=+5,321
	-2,761#			-13,759'#

$$d = \frac{V}{vjb} = \frac{2761}{40x.875x12} = 6.6"$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{13759}{139}} = 9.95"$$

$$v = \frac{2761}{12x.875x15} = 17$$ No special anchorage.

$$R = \frac{M}{bd^2} = \frac{13759}{15^2} = 61.1 < 139\ f_c$$ is low.

$$A_s = \frac{M}{f_s jd} = \frac{13759}{18000x.875x15} = .0583$$ Sq.in. per in.

Use 3/4 inch round bars @ 7-1/2" c/c.
Gives .0587 Sq. inches.

Computations continued:

$$u = \frac{vb}{\Sigma o} = \frac{17 \times 7.5}{3/4} = 54 < 100$$ No special anchorage.

TOE DESIGN

Working about the section shown, the toe design was accomplished. The assumed values for the dimensions were below the required value in each case. It may be noted that anchorage of the bars was required.

Computations:

$$P = \left(\frac{2745 + 2005}{2}\right) \times 2.25 = 5{,}344\#$$

$$x = \frac{2.25}{3} \left(\frac{2005 + 2 \times 2745}{4750}\right) = 1.18'$$

Maximum Shear and Moment on Section B-B:

$$V = 5{,}344\#$$

$$M = 5344 \times 1.18' = 6{,}306'\# \text{ per ft. or } ''\# \text{ per in.}$$

Keep toe 1.5' thick to match heel.

$$v = \frac{V}{bjd} = \frac{5344}{12 \times .875 \times 15} = 33.9 \text{ p.s.i.}$$

$$R = \frac{M}{bd^2} = \frac{6306}{15^2} = 28 \qquad (f_c \text{ very low})$$

$$A_s = \frac{M}{f_s jd} = \frac{6306}{18000 \times .875 \times 15} = .0267 \text{ Sq.in.}$$

Use 3/8 inch round bars @ 4" c/c.
Gives .0275 Sq. inches.

$$u = \frac{vb}{\Sigma_o} = \frac{34 \times 7}{\pi \times 1/2} = 115 > 100 \quad \text{Special anchorage.}$$

CUT-OFF WALL

The method used to design the cut-off wall for the side wall did not deviate from the method used for the cantilever and counterfort backwall and the front wall. The purpose of this wall was to increase the sliding factor of safety by decreasing the tendency of the wall to move over the ground.

The location of this wall is usually designed as shown but may be elsewhere under the base.

Computations:

Passive Earth Pressure $= wh \dfrac{1 + \sin 35^\circ}{1 - \sin 35^\circ} = 3.7\ wh$

At C-C, $wh = 1983.6$

Possible Res. $= 3.7 \times 1983.6 = 7{,}339$ p.s.f.

To make sliding factor $= 2$

$R = 2 \times 6554 - .499 \times 13358 = 6442\#$

Min. $h = \dfrac{6442}{7339} = .878'$ Use 1' for convenience.

On Section C-C:

$V = 6{,}442$

$M = 6442 \times 6 = 38{,}652''\#$

$d = \dfrac{V}{bjv} = \dfrac{6442}{12x.875x40} = 15.3''$

If no tension reinforcement:

$d = \sqrt{\dfrac{6 \times 38652}{12 \times 60}} = 17.9''$

Make 1'-6" for convenience.

STEM DESIGN

To discuss in detail the procedures used for the stem would be mere repitition. Any inquiry developing out of the following computations should be referred to the stem design of the cantilever back wall. The results, of course, were different due to the variations in dimensioning. As a result, anchorage and area of steel differed.

Computations:

Maximum Shear and Moment on Section D-D:

$$V = 1/2 \times 13.85 \times 507 = 3{,}511\#$$

$$d = \frac{V}{bjv} = \frac{3511}{12 \times .875 \times 40} = 8.36'' \quad \text{OK}$$

$$M = 3511 \times 1/3 \times 13.85 = 16{,}210'\# \text{ per ft. or } ''\# \text{ per in.}$$

$$d = \sqrt{\frac{16210}{139}} = 10.8'' \quad \text{OK}$$

Keep stem:

12" top

18" bottom

@ any level:

$$V_x = 18.78\ h^2$$

$$M_x = 6.26\ h^3$$

$$u = \frac{vb}{\Sigma_o} = \frac{21 \times 6.5}{3/4\ \pi} = 58 \text{ p.s.i.}$$

Embedment

$$L = \frac{f_s d}{4u} = \frac{18000 \times 3/4}{4 \times 100} = 33.75''$$

Hook Bottom

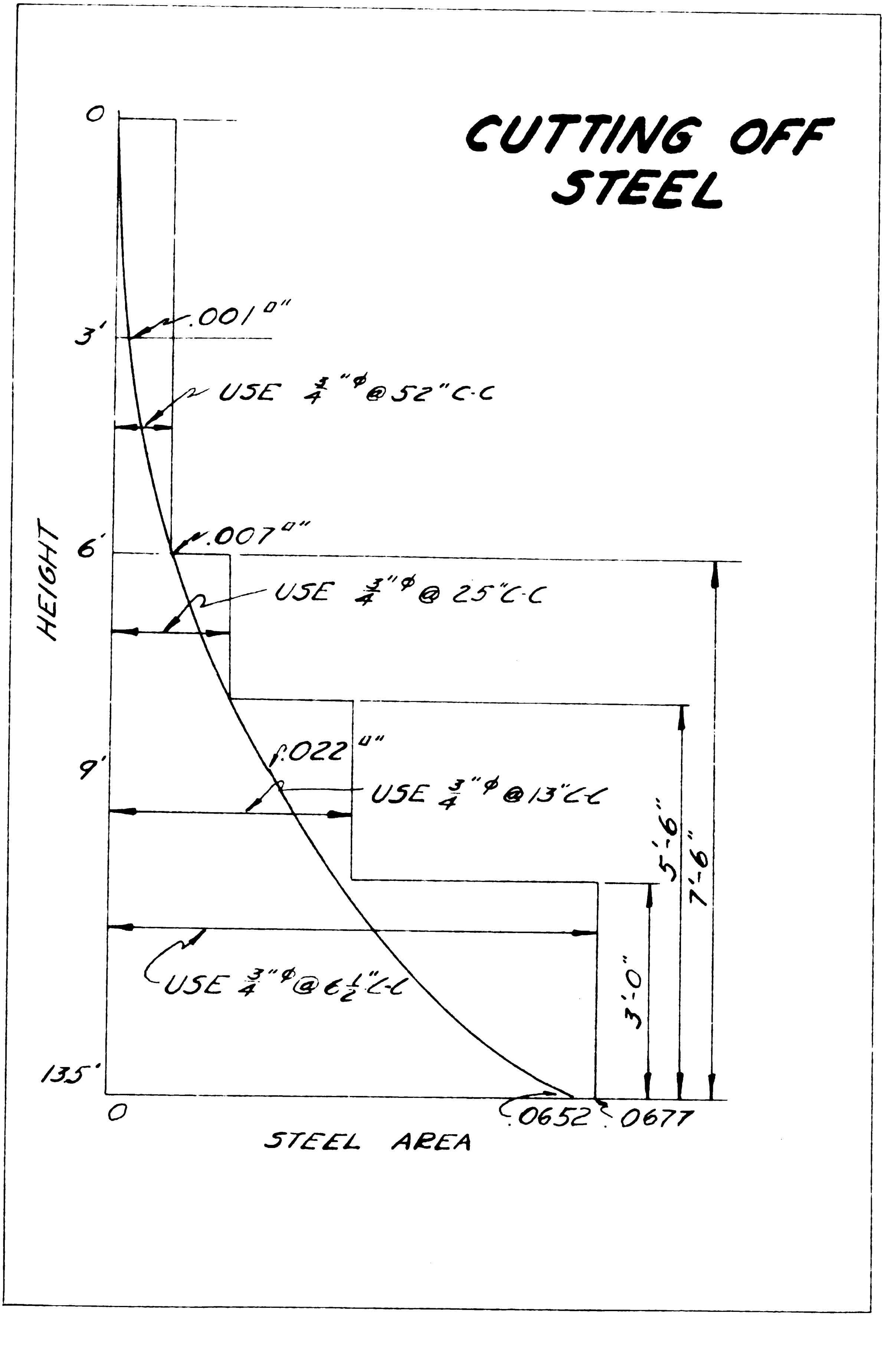
Temperature Steel:

$$.002 \times 12 \times 15 = .36 \text{ Sq.in. per ft. of height.}$$

$$\left\{\underset{\text{Front}}{1/2''\ \phi\ @\ 12''\ c/c}\right\} + \left\{\underset{\text{Back}}{1/2''\ \phi\ @\ 15''\ c/c}\right\} = .36 \text{ Sq.in.}$$

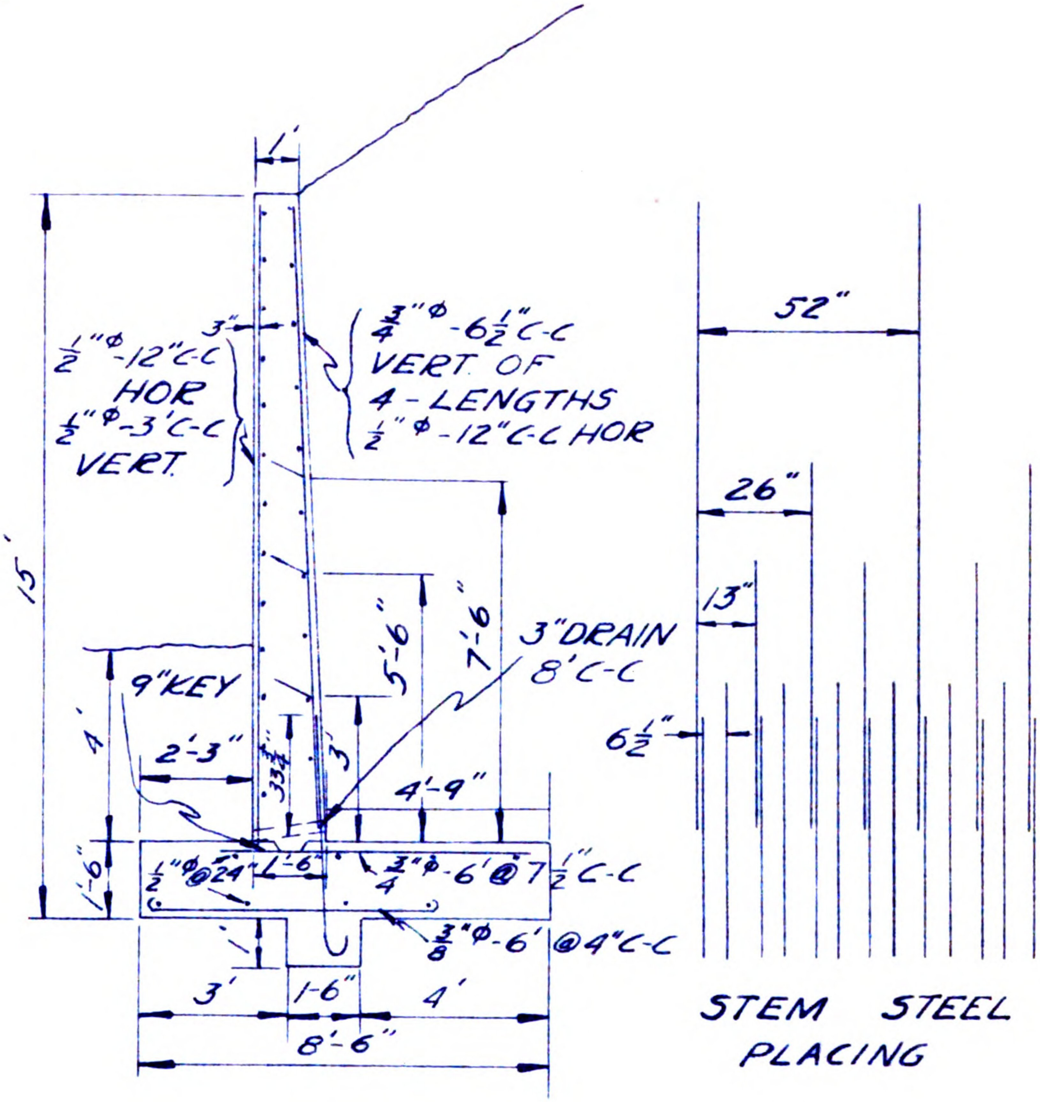
STEM SCHEDULE

h	V	d	v	M.	R	A_s	RODS
3'	169	10.3"	1+	169	1+	.00104	
6'	676	11.7"	5+	1,352	9+	.00734	
9'	1521	13.0"	11+	4,564	27	.0223	SEE SKETCH
13.5'	3424	15.0"	21+	15,400	68	.0652	$\frac{3}{4}$"ϕ @ $6\frac{1}{2}$" - .0677


CUTTING OFF STEEL
0
3'
6'
9'
135'
HEIGHT
.001 □"
.007 □"
.022 □"
USE 3/4"φ @ 52" C-C
USE 3/4"φ @ 25" C-C
USE 3/4"φ @ 13" C-C
USE 3/4"φ @ 6 1/2" C-C
5'-6"
7'-6"
3'-0"
0
.0652
.0677
STEEL AREA

SIDE WALL

SCALE 1" = 4'



STEM STEEL PLACING

NOTE:
THE ABOVE DESIGN WAS USED FOR THE CENTER 20' OF THE SIDE WALL (NORTH & SOUTH)

COST COMPARISON

The following cost comparison was derived from prices quoted in the "Engineering News Record" magazine of February 5, 1948, and is given as a comparison of the two designs heretofore presented.

The results obtained should not be taken as the cost of the construction, but merely as a comparison. Only the three main items were considered here: (1) Steel in Place, (2) Concrete in Place, and, (3) Excavation.

Concrete in Place

Location	Design	Volumn Cu.Yds.	Unit Price	Total
Back Wall	Cantilever	772.2	$40.00	$30,888
Back Wall	Counterfort	737.7	40.00	29,508
Front Wall	Semi-Gravity	530.6	40.00	21,224
Side Wall (North)	Combination	87.87	40.00	3,515
Side Wall (South)	Combination	85.25	40.00	3,410

Steel in Place

Location	Design	Weight	Unit Price	Total
Back Wall	Cantilever	74,394.1#	$.10	$ 7,439.41
Back Wall	Counterfort	50,356.9	.10	5,035.69
Front Wall	Semi-Gravity	5,733.6	.10	573.36
Side Wall (North)	Combination	5,237.9	.10	523.79
Side Wall (South)	Combination	5,208.9	.10	520.89

Excavation

Area	Depth	Volumn	Unit Price	Total
2,655 Sq. Yds.	1 Yd.	2,655 cu.yds.	$.90	$ 2,389.50

Cost of Cantilever Design

Location	Steel	Concrete	Total
Back Wall	\$7,439.40	\$30,888.00	\$38,327.40
Side Wall	1,044.68	6,925.00	7,969.68
Front Wall	573.36	21,224.00	21,797.36
Excavation	--	--	2,389.50
		GRAND TOTAL	\$70,483.94

Cost of Counterfort Design

Location	Steel	Concrete	Total
Back Wall	\$5,035.69	\$29,508.00	\$34,543.69
Side Wall	1,044.68	6,925.00	7,969.68
Front Wall	573.36	21,224.00	21,797.36
Counterfort Angles	1,056.00	--	1,056.00
Excavation	--	--	2,389.50
		GRAND TOTAL	\$67,756.23

DISCUSSION

The reader should not be falsely lead to believe that the results just obtained in the cost comparison are all conclusive.

To illustrate this let us briefly follow through the assumptions and results obtained.

As previously stated, the unit prices are four months old and if construction is contemplated, new prices must be used. The foregoing is merely a comparison of the major variables and constants involved in design and construction. The variables are the volume of concrete and steel needed for the cantilever and the counterfort back wall. The form area for concrete will vary. It is important to note that this is not brought out by the comparison, and will tend to increase the counterfort cost.

The side walls, front wall, and labor for the side and front walls, remains relatively constant. These constants, however, were included to give a better picture of the magnitude of the work and, also, the variation in the magnitude effects the unit prices.

The final prices given in the comparison are surprisingly close. It has been an accepted theory that twenty feet is the dividing line between cantilever and counterfort design and this design seems to substantiate that theory. The volumes of concrete in the two back walls was practically equal and the steel required for the cantilever wall was slightly greater.

The counterfort wall will require more labor and as the labor cost fluctuates, so will a comparison of the costs of construction vary. No conclusion has been made here as each design has its own place. This comparison concludes the study undertaken in this thesis of retaining walls.