APPENDIX D

STABILITY ANALYSES AND EVALUATIONS OF MAJOR COMPONENTS OF LAKE LURE DAM

STABILITY ANALYSES AND EVALUATIONS

STABILITY EVALUATION CRITERIA:

REQUIRED FACTORS OF SAFETY:

NORMAL LOADING - F.S ≥ 3.0

UNUSUAL LOADING - F.S. ≥ 2.0

EXTREME LOADING - F.S ≥ 1.0
(NORMAL LOADING + EARTHQUAKE)

STABILITY ANALYSES PARAMETERS:

REFERENCE LABORATORY TESTING (APPENDIX A)

UNIT WEIGHT OF CONCRETE - 17 SPECIMENS

RANGE OF UNIT WEIGHTS: 128 TO 150 pcf

MEAN VALUE = 139 pcf

AVERAGE VALUE = 142.6 pcf

$\boxed{\text{USE 140 pcf}}$

FILL & BACKFILL MATERIALS:

VERY COARSE TO MEDIUM SAND (SM-SW) WEATHERED FROM VERY COARSE-GRAINED SCHIST BEDROCK; DECOMPOSED VERY COARSE TO COARSE-GRAINED SCHIST AND GRAVEL, PEBBLES, AND BOULDERS WEATHERED FROM HARD TO HIGHLY WEATHERED BIOTITE GNEISS BEDROCK.

REFERENCE ATTACHED SHEET No. 2

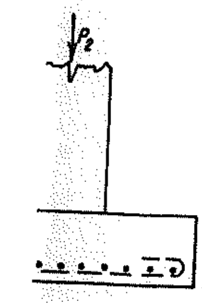
γ_{dry} = 100 pcf

γ_{moist} = 105 pcf

$\gamma_{sat.}$ = 112.4 pcf

γ_{sub} = 50 pcf

ANGLE OF INTERNAL FRICTION = 35°



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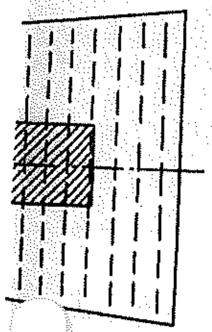


oting.

ned as inverted floor systems,
of the flat slab or beam and

MENTS

e, or reinforced concrete, is a
ther material possessing more
alls depend for their stability
walls) or on their own weight
terally supported material.
ly used for lateral supports of
the term "earth" will be used
terial is earth or some other

rting an earth fill, the surface
D. There is a tendency for a
such as *ACB*, to break away
ttle downward and move for-
ight of the block *ACB* tends
cting across *AB* and *CB*, by
friction along the plane *AB*.

SHEET No. 2 of 13

The internal frictional resistance may also include a cohesive quality in the soil. If cohesion is absent, the surface of rupture may be assumed to be an inclined plane such as *BD*; while if cohesion is present, the surface of rupture stands more nearly vertical at the surface of the ground than at the base of the wall, thus resulting in the curve *CB*. Usually this curved direction of the surface of rupture is influenced by a variation in moisture content. The more moisture, the less cohesion, and hence the flatter the angle of rupture. It should thus be seen at the outset that the removal of all possible moisture from the supported material reduces greatly the size of the prism of earth actually effective in pressing against the wall and thus increases the stability of the wall. Probably the most important rule in the design of retaining walls, therefore, is to *provide adequate drainage*.

Earth-pressure theories generally assume a granular mass possessing no cohesion, since this assumption is on the safe side of the usual conditions of soil possessing some cohesion. Tables 10 and 11 give weights, angles of repose, and values of internal friction

TABLE 10. ANGLES OF REPOSE AND WEIGHTS PER CUBIC FOOT FOR VARIOUS EARTHS [1]

Material	Slope	Angle of repose, deg	Weight, lb per cu ft
Sand, dry	2.8 :1 to 1.4 :1	20 to 35	90 to 110
Sand, moist	1.75:1 to 1:1	30 to 45	100 to 110
Sand, wet	2.8 :1 to 1.2 :1	20 to 40	110 to 120
Ordinary earth, dry	2.8 :1 to 1:1	20 to 45	80 to 100
Ordinary earth, moist	2.1 :1 to 1:1	25 to 45	80 to 100
Ordinary earth, wet	2.1 :1 to 1.75:1	25 to 30	100 to 120
Gravel, round to angular	1.75:1 to 0.9 :1	30 to 48	100 to 135
Gravel, sand and clay	1.8 :1 to 1.3 :1	20 to 37	100 to 115

$\gamma_{dry} = 100$ pcf
$\gamma_{moist} = 105$ pcf
$\gamma_{SAT} = 112.4$ pcf
$\gamma_{sub} = 50$ pcf

[1] CAIN, "Earth Pressure, Walls and Bins," p. 9.

TABLE 11. COEFFICIENTS OF INTERNAL FRICTION [1]

Kind of material	Tangent of angle of internal friction	Approximate corresponding: Angle, deg	Approximate corresponding: Slope	Authority
Coal, shingle, ballast, etc.	1.423	54	0.7 to 1	B. Baker
Bank sand	1.423	54	0.7 to 1	Goodrich
Riprap	1.097	48	0.9 to 1	Goodrich
Earth	1.097	48	0.9 to 1	B. Baker
Quicksand, 100 up	0.895	42	1.1 to 1	Goodrich
Clay	0.895	42	1.1 to 1	B. Baker
Quicksand, 50–100	0.750	37	1.3 to 1	Goodrich
Earth	0.750	37	1.3 to 1	Steel
Bank sand	0.750	37	1.3 to 1	Wilson
Sand, 50–100	0.549	29	1.8 to 1	Goodrich
Bank sand	0.549	29	1.8 to 1	Goodrich
Clay	0.474	25	2.1 to 1	Goodrich
Cinders	0.474	25	2.1 to 1	Goodrich
Gravel, ½-in.	0.474	25	2.1 to 1	Goodrich
Gravel, ¼-in.	0.350	19	2.9 to 1	Goodrich
Bank sand	0.350	19	2.9 to 1	Goodrich
Sand, 30–50	0.258	14	3.9 to 1	Goodrich
Sand, 20–30	0.179	10	5.6 to 0	Goodrich

$\phi_{FILL} = 35°$

[1] GOODRICH, *Trans. ASCE*, vol. 53, p. 301.

Stability Analyses Parameters (Cont'd):

Fill & Backfill Materials (Cont'd):

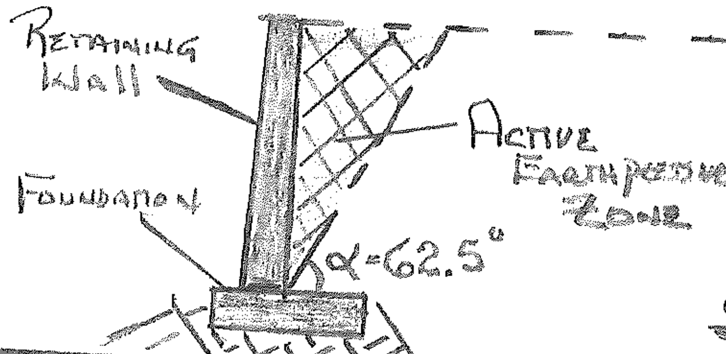
At-Rest Earthpressure Coefficient:

$$K_0 = 1 - \sin\phi = 1 - \sin 35^\circ$$
$$\underline{K_0 = 0.426}$$

Active Earthpressure Coefficient:

$$K_A = \tan^2\left(45^\circ - \frac{\phi}{2}\right) = \tan^2(45^\circ - 17.5^\circ)$$
$$\underline{K_A = 0.271}$$

Active Earthpressure Zone Intercept



$$\alpha = 45^\circ + \frac{\phi}{2}$$
$$\alpha = 45^\circ + \frac{35^\circ}{2}$$
$$\underline{\alpha = 62.5^\circ}$$

Foundation Bearing Materials:

Reference Sheet No. 4

Allowable Foundation Bearing Capacity, Q_a

Hard to Very Hard Sound Bedrock 50 to 100 tsf
Slightly Weathered to Hard Bedrock 25 to 75 tsf
Partially Weathered Rock (PWR) 20 to 50 tsf
Range of Internal Friction = 48° to 62°; $\phi_{ave} = 55^\circ$
Coefficient of Sliding Friction = $\tan\phi_A = \tan 45^\circ = 1.0$

SHEET 4 of 13

have been used instead of the Vierendeel trusses. Reinforced concrete interior walls restrict the use of basement space, and openings in these walls reduce the rigidity of the walls and introduce problems in design. Reinforced concrete trusses with diagonals also restrict the use of space, whereas Vierendeel trusses do not have this objectionable feature.

Bearing Power of Spread Foundations. Present methods for determining the bearing capacity of soils are not entirely satisfactory. The most common procedure consists of selecting values from tables of allowable or presumptive bearing capacities given in the local building codes. Such tables are commonly copied from other codes, and there may be little justification for the values given. In these tables, soils are usually divided into classes by indefinite terms, and a bearing value is given for each class. Actually, the bearing pressure which should be selected depends upon the amount of settlement which is permissible. The pressures under the foundations of buildings are rarely large enough to approach the actual strength of the soil except in the case of shallow foundations on soft clay.

For foundations supported on cohesive soils, the settlements of footings for a given unit pressure increase with the linear dimensions of the footings. The common assumption that the settlements of footings supported on a given soil will be equal if the unit pressures are equal is incorrect for cohesive soils. There would therefore be a different allowable pressure for each size of footing on a given cohesive soil if uniform settlement were required. However, some unequal or differential settlement between footings may not be serious, and the amount of differential settlement which may be permitted for the various footings, and which will not overstress the structural frame, should be determined.

The selection of allowable bearing values by examining only the soil at or near the surface on which the foundation is to rest may lead to serious errors. The soil below the bottom of a footing, for a depth of several times the width of the footing, contributes significantly to its settlement, and the underlying soil may be very different from that near the surface. Layers of peat, soft clay, and other soils of negligible or low bearing power at even greater depths may contribute considerably to settlement. Another factor which is commonly neglected in selecting allowable bearing values is the usual increase in bearing capacity with increase in the depth of the bearing surface below ground surface.

Allowable Bearing Capacities. The following table of allowable or presumptive surface bearing capacities, taken from the 1955 edition of the "Building Code of the City of Detroit," will serve as an example:

Foundation Material	Tons per Sq Ft
1. Hard sound rock	100
2. Soft rock PARTIALLY WEATHERED ROCK	~~12~~ 20
3. Hardpan overlying rock; very compact sandy gravel	10
4. Compact sandy gravel; very compact clay, sand, and gravel; very compact coarse or medium sand	6
5. Firm sandy gravel; compact clay, sand, and gravel; compact coarse or medium sand; very compact sand-clay soils; hard clay	5
6. Loose sandy gravel; firm coarse or medium sand	4
7. Loose coarse or medium sand; compact fine sand; compact sand-clay soils; stiff clay	3
8. Firm fine sand; compact inorganic silt; firm sand-clay soils; medium clay	2
9. Loose fine sand; firm inorganic silt	1½
10. Loose sand-clay soils; loose inorganic silt; soft clay	1 *

* In the case of loose organic silt and soft clay, the presumptive bearing value shall be substantiated by adequate supporting evidence.

Stability Analyses and Evaluations (Cont'd)

Normal Loading Condition Analyses of Section A:

Reference Working Drawing on Sheet No. 6

Determine Weight and Resultant Weight Locations:

$W_1 = V_1 \cdot \gamma_{concrete} = 313.5\ ft^3/ft \cdot 140\ pcf = 43{,}890\ \#/ft$ @ 6.33'

$W_2 = V_2 \cdot \gamma_c = 132\ ft^3/ft \cdot 140\ pcf = 18{,}480\ \#/ft$ @ 10.5'

$W_3 = V_3 \cdot \gamma_c = 148.5\ ft^3/ft \cdot 140\ pcf = 20{,}790\ \#/ft$ @ 13.0'

$W_4 = V_4 \cdot \gamma_c = 36\ ft^3/ft \cdot 140\ pcf = 5040\ \#/ft$ @ 17.5'

$88{,}200\ \#/ft$

$$\bar{X} = \frac{(43{,}800\# \cdot 6.33') + (18{,}480 \cdot 10.5') + (20{,}790 \cdot 13') + (5040 \cdot 17.5')}{88{,}200\ \#/ft}$$

$$\boxed{\bar{X} = \frac{829{,}764\ ft \cdot lbs/ft}{88{,}200\ \#/ft} = 9.4\ Feet}$$

Determination of Weight of Foundation:

$W_F = D_F \cdot L \cdot \gamma_c = 3' \cdot 20.5' \cdot 140\ pcf = 8{,}610\ \#/ft$

@ Mid-Point of Foundation = 10.25'

Total Weight of Wall & Foundation = $W_W + W_F = 88{,}200 + 8{,}610\ \#/ft$

$W_W + W_F = 96{,}810\ \#/ft$

Weight of Fill on Backslope of Wall:

$P_{FV} = 1/2 \cdot 9.5' \cdot 66' \cdot \left(\frac{105 \cdot 45 + 50 \cdot 21}{66'}\right) = 27{,}431.3\ \#/ft$

@ $2/3 \cdot 9.5' = 6.33\ ft$ from Toe

Total Weight of Wall, Foundation & Backfill:

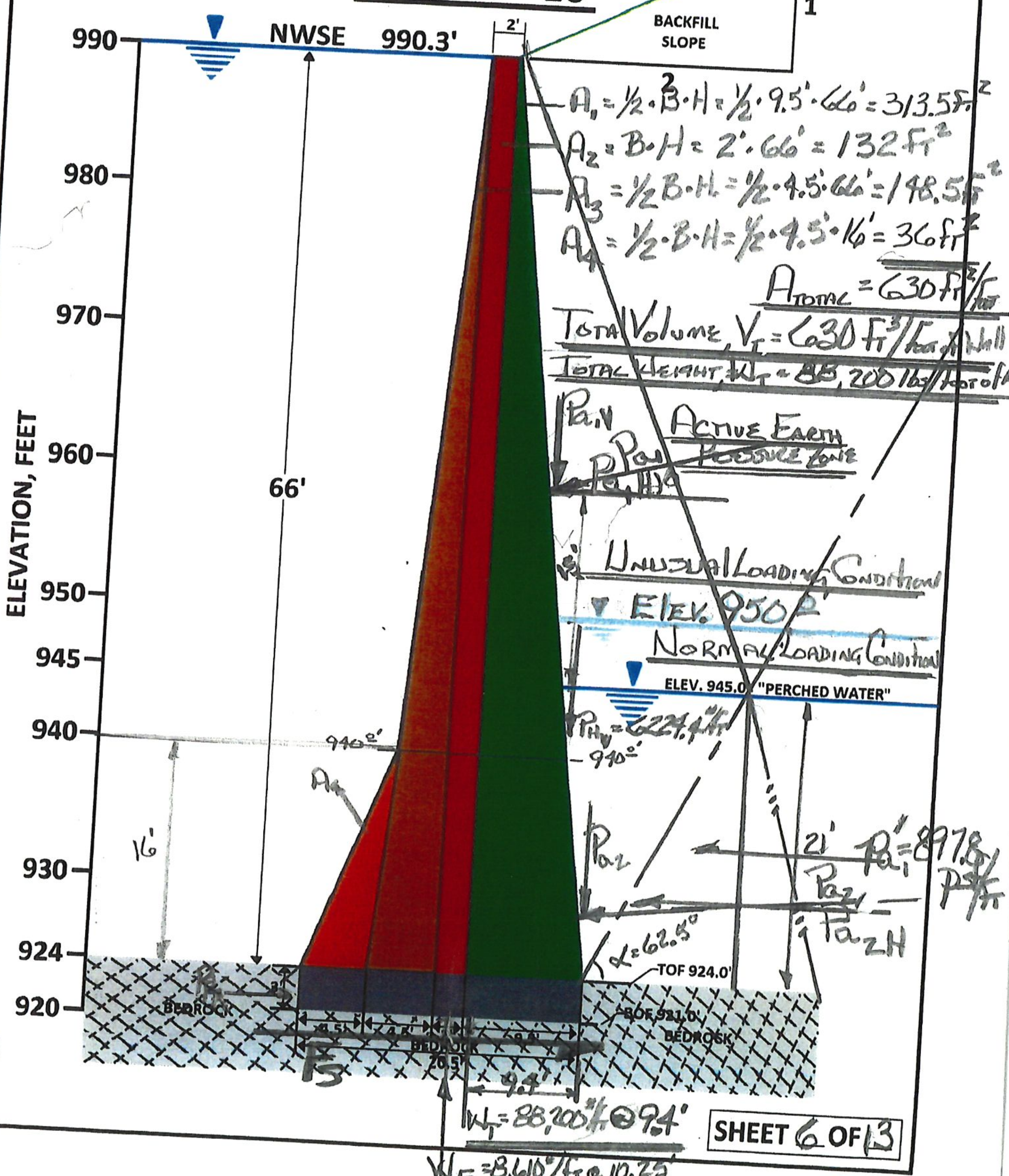
$W_{TOTAL} = W_W + W_F + P_{FV} = 124{,}241.3\ \#/ft$

Determine Sliding Resistance, F_S:

$F_S = W_{TOTAL} \cdot \tan \phi_F = 124{,}241.3\ \#/ft \cdot 1.0$

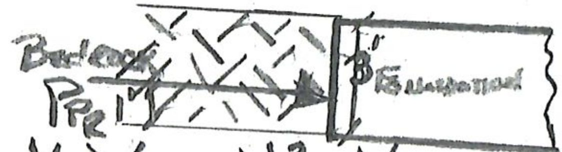
$F_S = 124.241.3\ \#/$foot of Wall

Stabilty Analysis
Left Abutment Wingwall
Section "A"
Scale: 1"= 10'



Stability Analyses & Evaluations (Cont'd)

Determine Passive Pressure Resistance of Bedrock:



$P_{P_R} = 1/2 \cdot \gamma_{rock} \cdot H^2 \cdot K_p$; K_p = Passive Pressure Coef.

$= \tan^2(45° + \phi/2)$

$P_{P_R} = 1/2 \cdot 145\ pcf \cdot (3')^2 \cdot 10.0$ $= \tan^2(45° + \frac{55°}{2})$

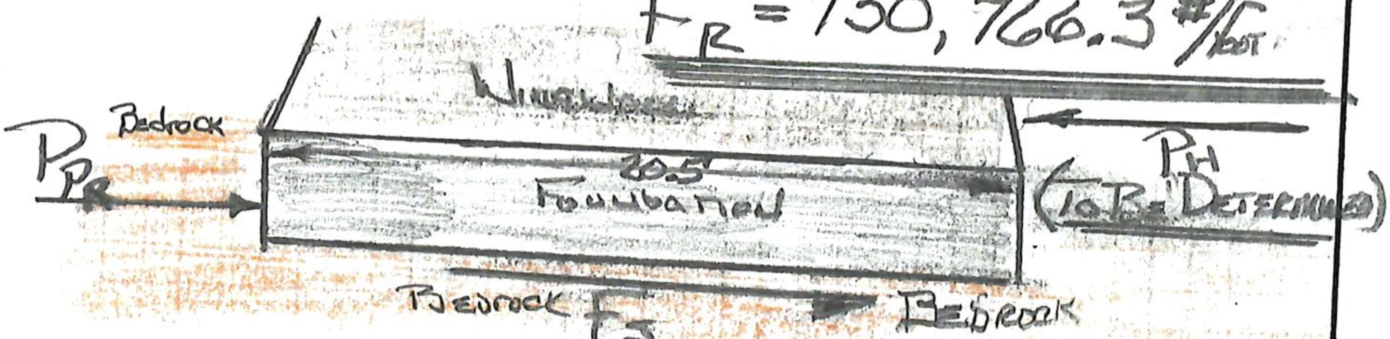
$= \tan^2(72.5°)$

$\underline{P_{P_R} = 6,525\ \#/ft}$ $K_p = 10.06$ (Use 10.0)

Determine Total Lateral Resistance, F_R:

$F_R = F_S$ (Sheet 5) $+ P_{P_R} = 124,241.3\ \#/ft$
$+ 6,525\ \#/Foot$

$\underline{F_R = 130,766.3\ \#/foot}$



$\underline{F_R = F_S + P_{P_R} = 130,766.3\ \#/\text{Foot of Wall}}$

Stability Analyses & Evaluations (Cont'd)

Determine Lateral Forces on Wingwall:

Reference Portions of Attached Chapter 3, NAVFAC DM-7.2 Foundations & Earth Structures

Reference Figure 3 on Page 7.2-64 to Determine Value of Coefficient of Active Earth Pressure for a Sloping Backfill. For a $\phi = 35°$ & a Fill Slope of +2:1; Ka = 0.38.

Reference Figure 6 on Page 7.2-67 to Determine Value of Coefficient of Active Pressure for a Wall Friction Angle, β to φ Angle of 0.76 The Ka = 0.38.

Reference Figure 5 on Page 7.2-66 to Determine Value of Coefficient of Active Pressure for a Wall Slope Angle ~~θ~~ = 8° & $\phi = 35°$ the K_a Value = 0.38.

Since the K_a Value for All Conditions is 0.38, the Appropriate Ka Value for this Wall Condition & φ Value is 0.38

Determine the Active Earth Pressure from the Top of Wall to the "Perched" Water Level.

$$P_{A_1} = \tfrac{1}{2} \cdot \gamma_{moist} \cdot H^2 \cdot K_a = \tfrac{1}{2} \cdot 105\text{pcf} \cdot (45')^2 \cdot 0.38$$

$$P_{a_1} = 40{,}398.8\ \#/\text{foot}$$

$$P_{a_{Horz}} = P_{a_1} \cdot \cos\phi = 40{,}398.8 \cdot \cos 35°$$

$$\underline{P_{a_{1Hor.}} = 33{,}092.7\ \#/\text{foot}}$$ Acting @ Elev. 960.0' or 15' Above "Perched Water"

$$P_{a_{1Vert}} = P_{a_1} \cdot \sin 35° = 40{,}398.8\ \#/\text{ft} \cdot \sin 35°$$

$$\underline{P_{a_{1Vert}} = 23{,}171.8\ \#/\text{foot}}$$

Reference Sheet 6 to See the Location of These Forces.

BY: BDM **MARKS ENTERPRISES OF NC, PLLC** **DATE:** 11-07-2017
PROJECT: PHASE II LAKE LURE DAM INSPECTION **PROJECT No.** ME-16-006 **SHEET No.** 9 **of** 13

STABILITY ANALYSES & EVALUATIONS (CONT'D)

DETERMINE THE LATERAL EARTH PRESSURES ACTING ON THE WINGWALL BELOW THE "PERCHED WATER" ELEVATION 945.0'. THIS PORTION OF THE WALL IS ACTED UPON BY TWO (2) LATERAL EARTH PRESSURE SYSTEMS ARE AS FOLLOWS: 1) THE IMPACT OF THE MOIST SOILS ABOVE THE PERCHED WATER ELEVATION; AND 2) THE INFLUENCE OF THE SUBMERGED SOIL MASS BELOW THE PERCH WATER ELEVATION.

The impact of the moist soils above the perched water elevation is determined by the following Equation:

$$Pa_1 = \tfrac{1}{2} \cdot \gamma_{moist} \cdot H \cdot K_a = \tfrac{1}{2} \cdot 105 \cdot 45' \cdot 0.38$$

$$Pa_1 = 897.8\ \#/ft^2$$

$$Pa_{1H} = 897.8\ \#/ft^2 \cdot 21' \cdot \cos 35°$$

$$\underline{Pa_{1H} = 15{,}444.1\ \#/foot}$$

$$Pa_{1V} = 897.8\ \#/ft \cdot 21' \cdot \sin 35°$$

$$\underline{Pa_{1V} = 10{,}814.1\ \#/ft}$$

Both of these forces act @ 10.5 feet above the Top of Foundation Elevation.

The Equation for the submerge soil beneath the perched water elevation is as follows:

$$Pa_2 = \tfrac{1}{2}\, \gamma_{sub} \cdot H_2^2 \cdot Ka = \tfrac{1}{2} \cdot 50\,pcf \cdot (21')^2 \cdot 0.38$$

$$Pa_2 = 4{,}189.5\ \#/ft \ @\ \tfrac{1}{3} \cdot 21' = 7' \text{ Above } \overline{TOF}$$

$$\underline{Pa_{2H} = Pa_2 \cdot \cos 35° = 3431.8\ \#/foot}$$

$$\underline{Pa_{2V} = Pa_2 \sin 35° = 2403\ \#/foot}$$

Pa_{1V} & Pa_{2V} Shall Be Added to the Vertical Component (Weight of Wall, Weight of Backfill, and Weight of Foundation) for Sliding Resistance)

STABILITY ANALYSES & EVALUATIONS (CONT'D)

DETERMINATION of FACTORS of SAFETY:

FACTOR of SAFETY AGAINST SLIDING FAILURES:

$$F.S._{SLIDING} = \frac{\sum \text{RESISTING FORCES}}{\sum \text{DRIVING FORCES}}$$

$$\sum \text{RESISTING FORCES} = \left[W_{WALL} + W_F + W_{BACKFILL} + P_{a_{1V}} + P_{a_{2V}}\right] \cdot \tan\phi_F + P_{P_R}$$

$$\sum F_R = \left[89{,}200\ \#/ft + 8{,}610\ \#/ft + 27{,}431.3\ \#/ft + 23{,}171.8 + 10{,}814.1\ \#/ft + 2403\ \#/ft\right] \cdot \tan 45° + 6{,}525\ \#/foot$$

$$\sum F_R = \left[152{,}020.2\right] \cdot 1.0 + 6{,}525\ \#/ft$$

$$\underline{\underline{\sum F_R = 158{,}545.2\ \#/ft}}$$

$$\sum \text{DRIVING FORCE} = \left[P_{a_{1H}} + P_{a_{2H}} + P'_{a_{1H}}\right]$$

$$\sum F_H = \left[33{,}092.7\ \#/ft + 3{,}431.8\ \#/ft + 15{,}444.1\ \#/foot\right]$$

$$\underline{\underline{\sum F_H = 51{,}968.6\ \#/ft}}$$

$$F.S._{SLIDING} = \frac{158{,}545.2}{51{,}968.6}$$

THIS F.S. IS SUITABLE

$$\boxed{F.S._{SLIDING} \doteq 3.05 > 3.00}$$

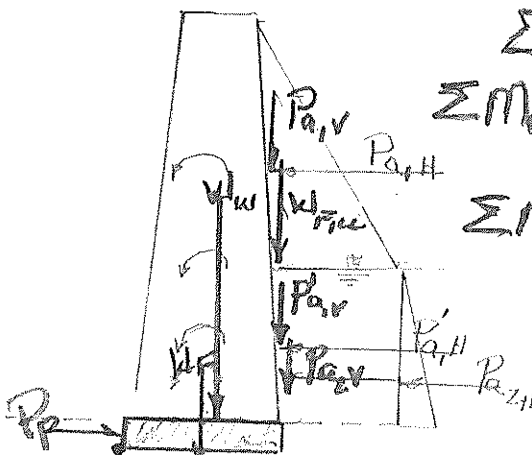
HOWEVER, THE WINGWALL MUST BE MONITORED CLOSELY

Stability Analyses and Evaluations (Cont'd)

Factor of Safety Against Overturning:

$$F.S._{OVERTURNING} = \frac{M_{RESISTING}}{M_{OVERTURNING}}$$

$\Sigma M_{OVERTURNING}$:



SKETCH- NOT-TO-SCALE

$$\Sigma M_O = P_{a_1H} \cdot l_1 + P'_{a_1H} \cdot l_2 + P_{a_{2H}} \cdot l_3$$

$$\Sigma M_O = 33{,}092.7\#/ft \cdot 39' + 15{,}444.1\#/ft \cdot 13.5' + 3431.8\#/ft \cdot 10'$$

$$\Sigma M_O = 1{,}290{,}615.3 \text{ ft-lbs/ft} + 208{,}495.4 + 34{,}318 \text{ ft-lbs/foot}$$

$$\boxed{\Sigma M_O = 1{,}533{,}428.7 \text{ ft-lbs/ft}}$$

$\Sigma M_{RESISTING}$:

$$\Sigma M_R = W_W \cdot l_1 + W_F \cdot l_2 + W_{Fill} \cdot l_3 + P_{a_1V} \cdot l_4 + P'_{a_1V} \cdot l_5 + P_{a_{2V}} \cdot l_6 \cdot P_P \cdot l_7$$

$$\Sigma M_R = 88{,}764\#/ft \cdot 10.6' + 8{,}610\#/ft \cdot 10.25' + 27{,}431.3\#/ft \cdot 14.2' + 23{,}171.8 \cdot 18' + 10{,}814.1\#/ft \cdot 20' + 2{,}403 \cdot 20.4' + 6{,}525\#/ft \cdot 1.0'$$

$$\Sigma M_R = 940{,}898.4 \text{ ft-lbs/ft} + 88{,}252.5 \text{ ft-lbs/ft} + 389{,}524.5 \text{ ft-lbs/ft} + 417{,}092.4 \text{ ft-lbs/ft} + 216{,}282 \text{ ft-lbs/foot} + 49{,}021.2 + 6525\#/ft$$

$$\Sigma M_R = 2{,}107{,}596 \text{ ft-lbs/foot}$$

w/o Contribution of Foundation Bearing Capacity

SHEET 12 of 13

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n yield
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TABLE 1002(a)—ALLOWABLE STRESSES IN CONCRETE

Description		Allowable stresses: For any strength of concrete in accordance with Section 502	For strength of concrete shown below 28 day: $f_c' =$ 2500 psi	$f_c' =$ 3000 psi	$f_c' =$ 4000 psi	$f_c' =$ 5000 psi
Modulus of elasticity ratio: n		$\frac{29,000,000}{w^{1.5}\,33\sqrt{f_c'}}$				
For concrete weighing 145 lb per cu ft (see Section 1102)	n		10	9	8	7
Flexure: f_c						
Extreme fiber stress in compression	f_c	$0.45f_c'$	1125	1350	1800	2250
Extreme fiber stress in tension in plain concrete footings and walls	f_c	$1.6\sqrt{f_c'}$	80	88	102	113
Shear: v (as a measure of diagonal tension at a distance d from the face of the support)						
Beams with no web reinforcement*	v_c	$1.1\sqrt{f_c'}$	55*	60*	70*	78*
Joists with no web reinforcement	v_c	$1.2\sqrt{f_c'}$	61	66	77	86
Members with vertical or inclined web reinforcement or properly combined bent bars and vertical stirrups	v	$5\sqrt{f_c'}$	250	274	316	354
Slabs and footings (peripheral shear, Section 1207)*	v_c	$2\sqrt{f_c'}$	100*	110*	126*	141*
Bearing: f_c						
On full area		$0.25f_c'$	625	750	1000	1250
On one-third area or less†		$\boxed{0.375f_c'}$	938	$\boxed{1125}$	1500	1875

*For shear values for lightweight aggregate concrete see Section 1208.
†This increase shall be permitted only when the least distance between the edges of the loaded and unloaded areas is a minimum of one-fourth of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonably concentric area greater than one-third but less than the full area shall be interpolated between the values given.

AREA = $\frac{20.5'}{3} = 6.83\ ft^2/ft$

Factor of Safety Against Overturning With Minimum Ingagement of Foundation Bearing.

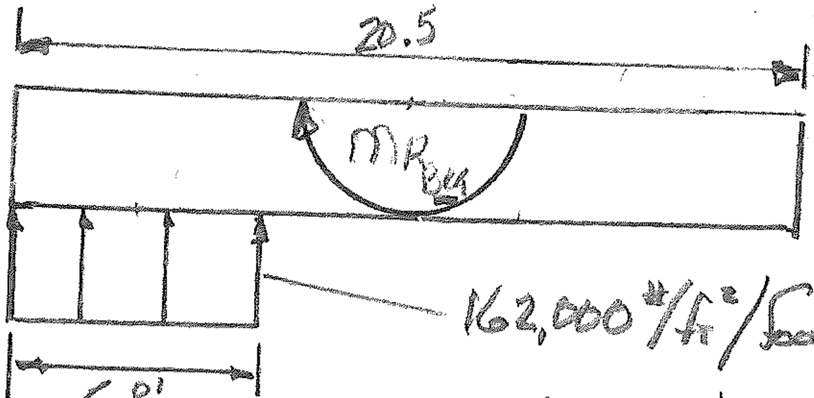
Allowable Concrete Bearing Capacity For Less Than or Equal to 1/3 of The Bearing Area = $0.375 \cdot f'_c$

f'_c From Lab Tests on Concrete Core Samples

$$f'_c = 3000 \text{ psi}$$

$$q_{all.} = 0.375 \cdot 3000 \text{ psi} = 1125 \text{ psi}$$

$$q_{all} = 162{,}000 \text{ \#/ft}^2\text{/ft of Wall}$$



$$MR_{Bearing} = (20.5' - 3.4') \cdot 162{,}000 \cdot 6.8'$$
$$= 18{,}837{,}360 \text{ ft-lbs/foot}$$

$$MR_{Total} = 18{,}837{,}360 + 2{,}107{,}596 \text{ ft-lbs/ft}$$

$$MR_{Total} = 20{,}944{,}956 \text{ ft-lbs/foot}$$

$$F.S._{OT} = \frac{20{,}944{,}956}{1{,}533{,}4287} = 13.7 \gg 3.0$$

Good!!

STABILITY ANALYSES & EVALUATIONS

LEFT ABUTMENT WINGWALL ANALYSES

UNUSUAL LOADING CONDITIONS

EXCESSIVE SPILLWAY DISCHARGE RESULTS IN ELEVATED "PERCHED TAILRACE"

REFERENCE SHEET 2 "SECTION A"

NOTE: DUPLICATE VALUES SUCH AS WEIGHT OF WALL WILL NOTE BE RE-CALCULATED BUT COPIED FROM "NORMAL LOADING CONDITION" CALCS.

WEIGHT OF WALL SECTION A = 88,200#/foot

$\bar{X}$ = 9.4 feet from Toe of Wall

WEIGHT OF FOUNDATION = 8,610#/foot

$\bar{X}$ = 10.25 feet from Toe (℄ of Fdn)

WEIGHT OF UPPER WET BACKFILL = ½·6'·40'·105pcf

W_{FILL_1} = 12,600#/foot

$\bar{Y}$ = 4 feet from Fill Transition

WEIGHT OF LOWER SUBMERGED FILL = ½·4'·26'·50pcf

W_{FILL_2} = 2600#/foot

$\bar{X}$ = 2.5' from Toe of Wall

PASSIVE EARTH PRESSURE @ HEEL OF WALL:

P_P = 6,525#/foot ; $\bar{Y}$ = 1.0' above BOF

TOTAL WEIGHT, $W_T = W_W + W_{FDN} + W_{FILL}$ = 88,200#/ft + 8,610#/ft + 12,600#/ft + 2,600#/ft

W_T = 112,000#/foot

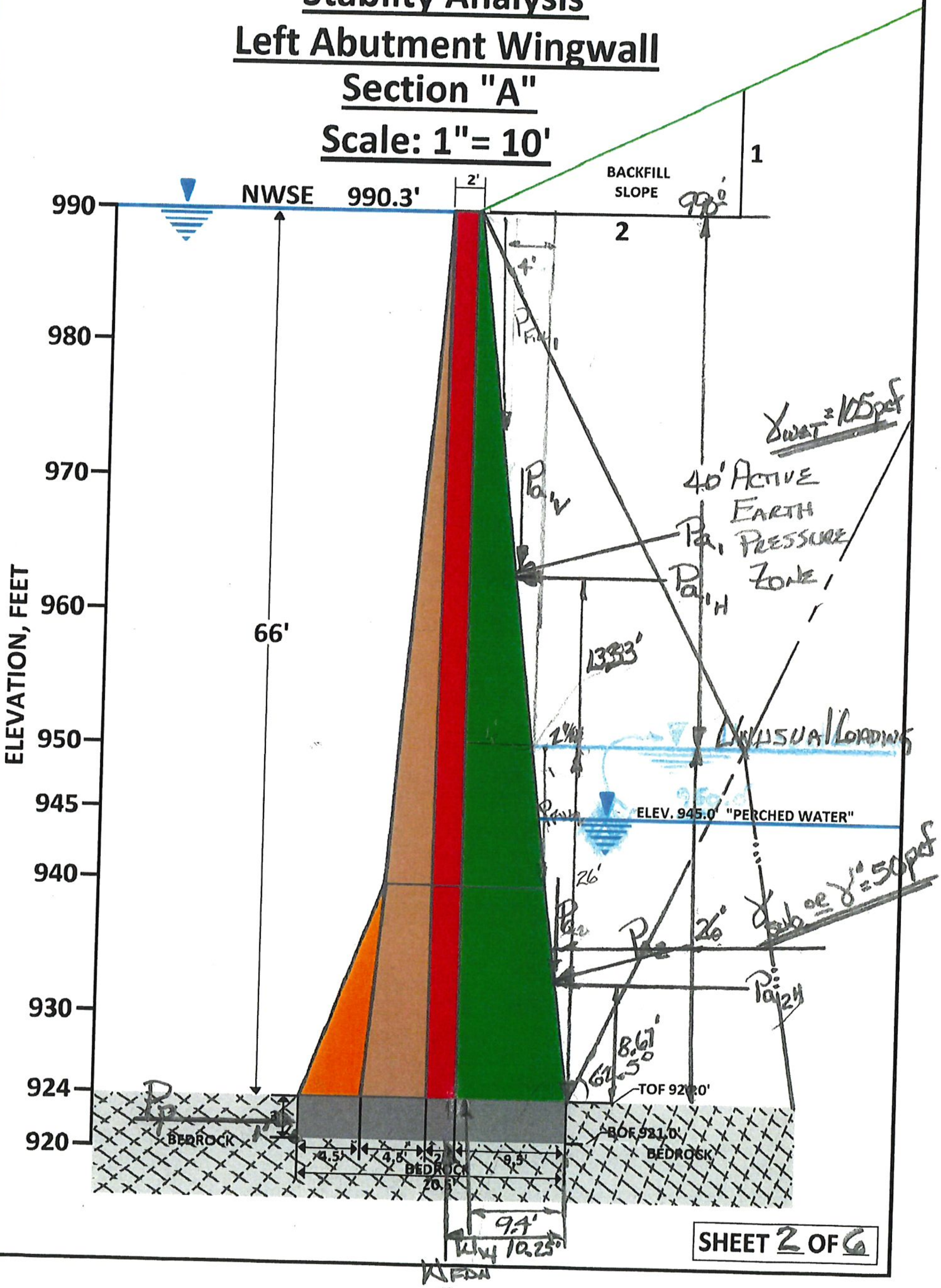
SLIDING RESISTANCE, $F_S = W_T \cdot \tan\phi_C + P_P$

NOTE: WALL SHEAR MUST BE ADDED TO WEIGHT & SLIDING RESISTANCE

F_S = 112,000·tan45° + 6,525#/foot

F_S = 118,525#/foot

Stabilty Analysis
Left Abutment Wingwall
Section "A"
Scale: 1"= 10'



SHEET 2 OF 6

STABILITY ANALYSES & EVALUATIONS (CONT'D)

ACTIVE EARTH PRESSURE DETERMINATIONS:

$P_{a_1} = \frac{1}{2} \cdot \gamma_{moist} \cdot H^2 \cdot K_a = \frac{1}{2} \cdot 105\,pcf \cdot (40)^2 \cdot 0.38$

$P_{a_1} = 31{,}920\ \#/foot$

$P_{a_{1H}} = P_{a_1} \cdot \cos\phi = 31{,}920\ \#/foot \cdot 0.819$

$\underline{P_{a_1 H} = 26{,}417\ \#/foot}$

$P_{a_{1V}} = P_{a_1} \cdot \sin\phi = 31920 \cdot 0.574$

$\underline{P_{a_1 V} = 18{,}322.0\ \#/foot}$

$P_{a_1} = \frac{1}{2} \cdot \gamma_{moist} \cdot H \cdot K_a = \frac{1}{2} \cdot 105\,pcf \cdot 40' \cdot 0.38$

$P_{a_1} = 798\ \#/foot$

$P'_{a_{1H}} = 798\ \#/foot \cdot 26' \cdot \cos 35°$

$\underline{P'_{a_1 H} = 16{,}995.8\ \#/foot}$

$P'_{a_V} = P_{a_1} \cdot 26' \cdot \sin 35°$

$\underline{P'_{a_1 V} = 11{,}900.6\ \#/foot}$

$P_{a_2} = \frac{1}{2} \cdot \gamma_{sub} \cdot H^2 \cdot K_a = \frac{1}{2} \cdot 50\,pcf \cdot (26')^2 \cdot 0.38$

$\underline{P_{a_2} = 6{,}422\ \#/foot}$

NOTE: Add $P_{a_2 V}$; $P'_{a_1 V}$ & $P_{a_2 V}$ To Vertical Component for Sliding Resistance

$P_{a_2 H} = P_{a_2} \cdot \cos\phi = 6{,}422\ \#/foot \cdot \cos 35°$

$P_{a_2 H} = 5260.6\ \#/foot$

$P_{a_2 V} = 6422\ \#/foot \cdot \sin\phi$

$\underline{P_{a_2 V} = 3683.5\ \#/foot}$

STABILITY ANALYSES & EVALUATIONS (CONT'D)

TOTAL VERTICAL WEIGHT COMPONENT:

TOTAL WEIGHT COMPONENT, $W_{TOTAL} = W_f + Pa_{1V} + P'_{a1V} + Pa_{2V}$

$$W_{TOTAL} = 112{,}000 + 18{,}322 \text{ \#/ft} + 11{,}900.6 + 3683.5 \text{ \#/foot}$$

$$W_{TOTAL} = 145{,}906.1 \text{ \#/foot}$$

TOTAL SLIDING RESISTANCE, $F_{S_{TOTAL}}$:

$$F_{S_{TOTAL}} = W_{TOTAL} \cdot \tan\phi_f + P_p$$

$$F_{S_{TOTAL}} = 145{,}906.1 \cdot 1.0 + 6525 \text{ \#/foot}$$

$$F_{S_{TOTAL}} = 152{,}431.1 \text{ \#/foot}$$

TOTAL DRIVING FORCE, P_H:

$$P_H = Pa_{1H} + P'_{a1H} + Pa_{2H} = 26{,}417 + 16{,}995.8 + 6422 \text{ \#/foot}$$

$$P_H = 49{,}834.8 \text{ \#/foot}$$

$$F.S._{SLIDING} = \frac{152{,}431.1}{49{,}834.8} = 3.06$$

$$F.S._{SLIDING} \gg 2.0 \text{ VERY GOOD}$$

STABILITY ANALYSES & EVALUATIONS (CONT'D)

FACTOR of SAFETY AGAINST OVERTURNING:

$$F.S._{OVERTURNING} = \frac{M_{RESISTING}}{M_{OVERTURNING}}$$

$$M_{RESISTING} = \boxed{W_W \cdot l_1 + W_{FDN} \cdot l_2 + P_P \cdot l_3 + P_{FILL_1} \cdot l_4 + P_{FILL_2} \cdot l_5 + P_{a_{1V}} \cdot l_6 + P'_{a_{1V}} \cdot l_7 + P_{a_{2V}} \cdot l_8}$$

$$M_R = (88{,}200 \cdot 10.6') + (8610 \cdot 10.25') + (6525 \cdot 1') + (12{,}600 \cdot 13') + (2600 \cdot 18') + (18{,}322 \cdot 16') + (3683.5 \cdot 19') =$$

$$M_R = 1{,}673{,}422.5 \text{ FT-lbs/FOOT of WALL}$$

$$M_O = P_{a_{1H}} \cdot l_1 + P'_{a_{1H}} \cdot l_2 + P_{a_{3H}} \cdot l_3$$

$$M_O = (26{,}417 \cdot 39.33') + (16995.8 \cdot 13') + (5260.6 \cdot 8.67')$$

$$M_O = 1{,}305{,}535.4 \text{ FT-lbs/FOOT}$$

$$F.S._{OT} = \frac{1{,}673{,}422.5}{1{,}305{,}535.4} = 1.28$$

W/O FOUNDATION BEARING CAPACITY

Stability Analyses & Evaluations (Cont'd)

Contributing Allowable Bearing Capacity Resisting Overturning:

$f'_c = 3000 psi$; $q_{ALL} = 0.375 \cdot f'_c$

$q_{ALL} = 1125 psi$

$M_{R_{BEARING}} = 162,000 \#/ft^2 \cdot 6.8' \cdot (20.5' - 3.4')$

$M_{R_{BEARING}} = 18,837,360$ ft·lbs/foot

$M_{R_{TOTAL}} = 18,837,360 + 1,673,422.5$

$M_R = 20,510,782.5$ ft·lbs/foot

$$F.S._{OT} = \frac{20,510,782.5}{1,305,535.4}$$

$F.S. = 15.7 >>> 2.0$

Very Good!!

Stability Analyses & Evaluations (Cont'd)

Left Abutment Wingwall Analyses

Extreme Loading Condition

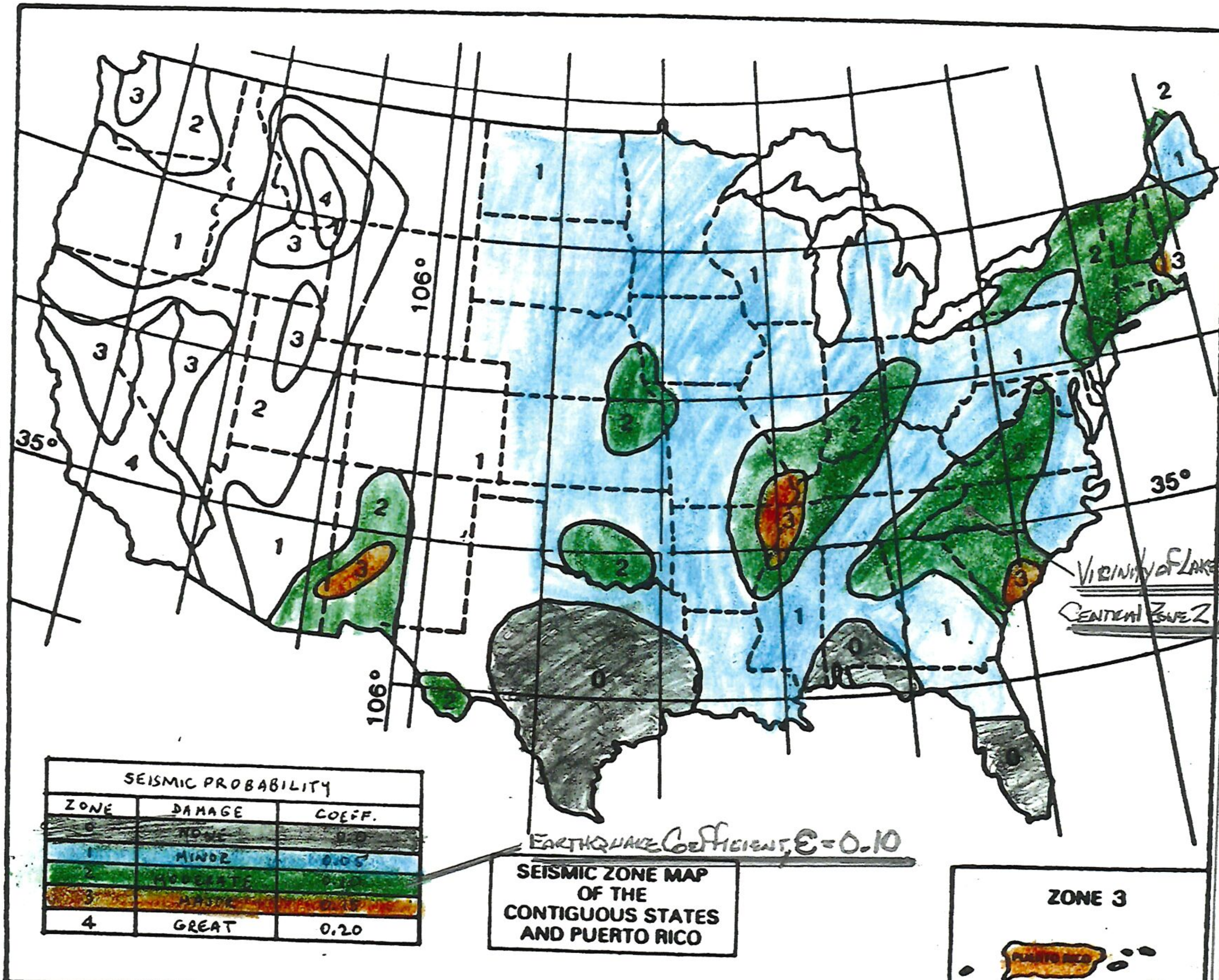
Siesmic or Earthquake Loading

"Performance Substantiated Stability of Lake Lure Dam & Its Appurtenances"

Lake Lure Dam & Its Appurtenances, as well as the Hydroelectric Generation Facilities and its appurtenances were subjected to Recent Performance Based Siesmic Loading Event.

Ironically, the Completion of the On-Going Dam Safety Inspection of Lake Lure Dam and its Appurtenances will be close to the 10-Year Anniversary of the Seismic Event. Comparatively the Earthquake of 2007 is comparable to a Current Full-Scale Test when considering Geologic Aging and/or Siesmic Event Analyses & Evaluations. Although the Author has Discovered Evidence of Earthquake Damage Not Previously Documented, Dr. Marks Has Elected to Evaluate Lake Lure Dam By the Earthquake Coefficient Pseudo-Dynamic Analysis Procedure. This procedure involves application of Earthquake Loading Coefficients to Normal Loading Conditions to Model Earthquake Effects.

Reference Sheet 2 of this Analysis to Find Earthquake Coefficients for the Continental United States of America.



SEISMIC PROBABILITY		
ZONE	DAMAGE	COEFF.
0	NONE	0.0
1	MINOR	0.05
2	MODERATE	0.10
3	MAJOR	0.15
4	GREAT	0.20

SEISMIC ZONE MAP OF THE CONTIGUOUS STATES AND PUERTO RICO

Stability Analyses & Evaluations (Cont'd)

Pseudostatic Earthquake Analysis
Section A
Left Abutment Wingwall

Factored Normal Loading

Active Earth Pressure Forces:

Normal Loads:

$P_{Ha_1} = 33{,}092.7$ #/foot

$P'_{Ha_1} = 15{,}444.1$ #/foot

$P_{Ha_2} = 3{,}431.8$ #/foot

$P_{H_{TOTAL}} = 51{,}968.6$ #/foot

Factored Load:

$F_{HOET} = (1+\epsilon) \cdot P_{H_{TOTAL}} = 1.10 \cdot 51{,}968.6$ #/ft

$F_{HOET} = 57{,}165.5$ #/foot

Factor of Safety Against Sliding:

$$F.S.'_{\substack{\text{Sliding}\\ \text{Pseudostatic}}} = \frac{\Sigma F_R}{F_{HOET}} = \frac{158{,}595.2}{57{,}165.5}$$

$$F.S.'_{\substack{\text{Sliding}\\ \text{Pseudostatic}}} = 2.77 >> 1.0$$

Very Good!!

STABILITY ANALYSES & EVALUATIONS (CONT'D)

PSEUDOSTATIC ANALYSIS - OVERTURNING

NORMAL OVERTURNING MOMENT:

$\Sigma M_o = 1,533,428.7$ ft-lbs/foot

FACTORED OVERTURNING MOMENT:

$\Sigma M_o' = \Sigma M_o \cdot 1.10 = 1,686,771.6$

PSEUDOSTATIC $F.S._{OT}$:

$$F.S._{OT}' = \frac{\Sigma M_R}{\Sigma M_o'} = \frac{2,107,596 \text{ ft-lbs/foot}}{1,686,771.6}$$

$$F.S._{OT}' = 1.25 > 1.0$$

ADEQUATE - BUT MUST BE MONITORED

REFER TO NORMAL LOADING ANALYSES

RECOMMENDATIONS:

1) REMEDIATION ACTIVITIES:
   a. INSTALL REINFORCED 4" ϕ WEEP HOLES
   b. INSTALL DRAINAGE & LEAKAGE AUTOMATIC FLOW DETECTION DEVICE
   c. MONITOR FLOW DAILY

2) LONG-TERM STABILIZATION: [MAJOR STORM OR LONG TERM (20 YRS)]
   a. ADDITIONAL REINFORCED WEEP HOLES
   b. ROCK-ANCHOR TIE-BACK SYSTEM.