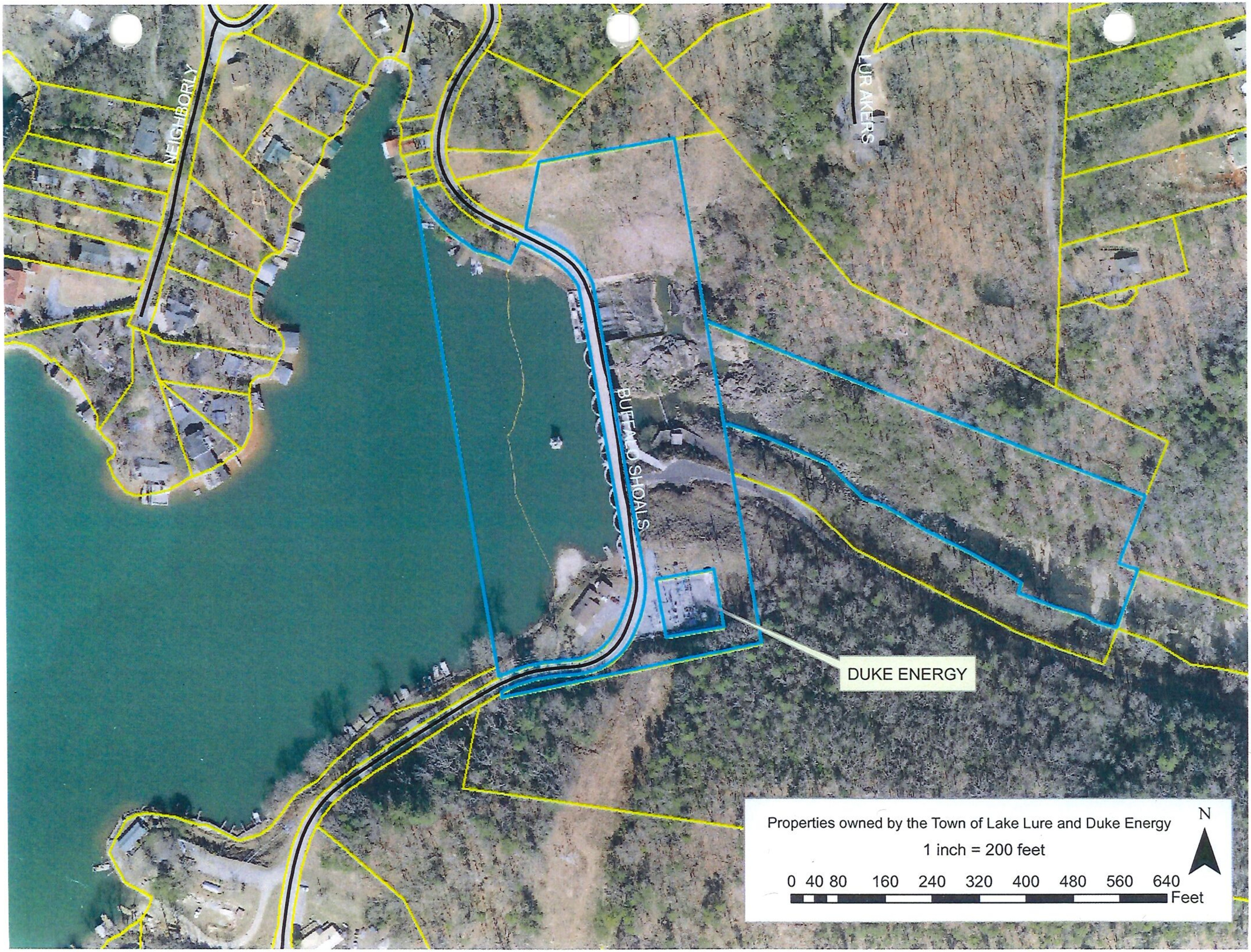
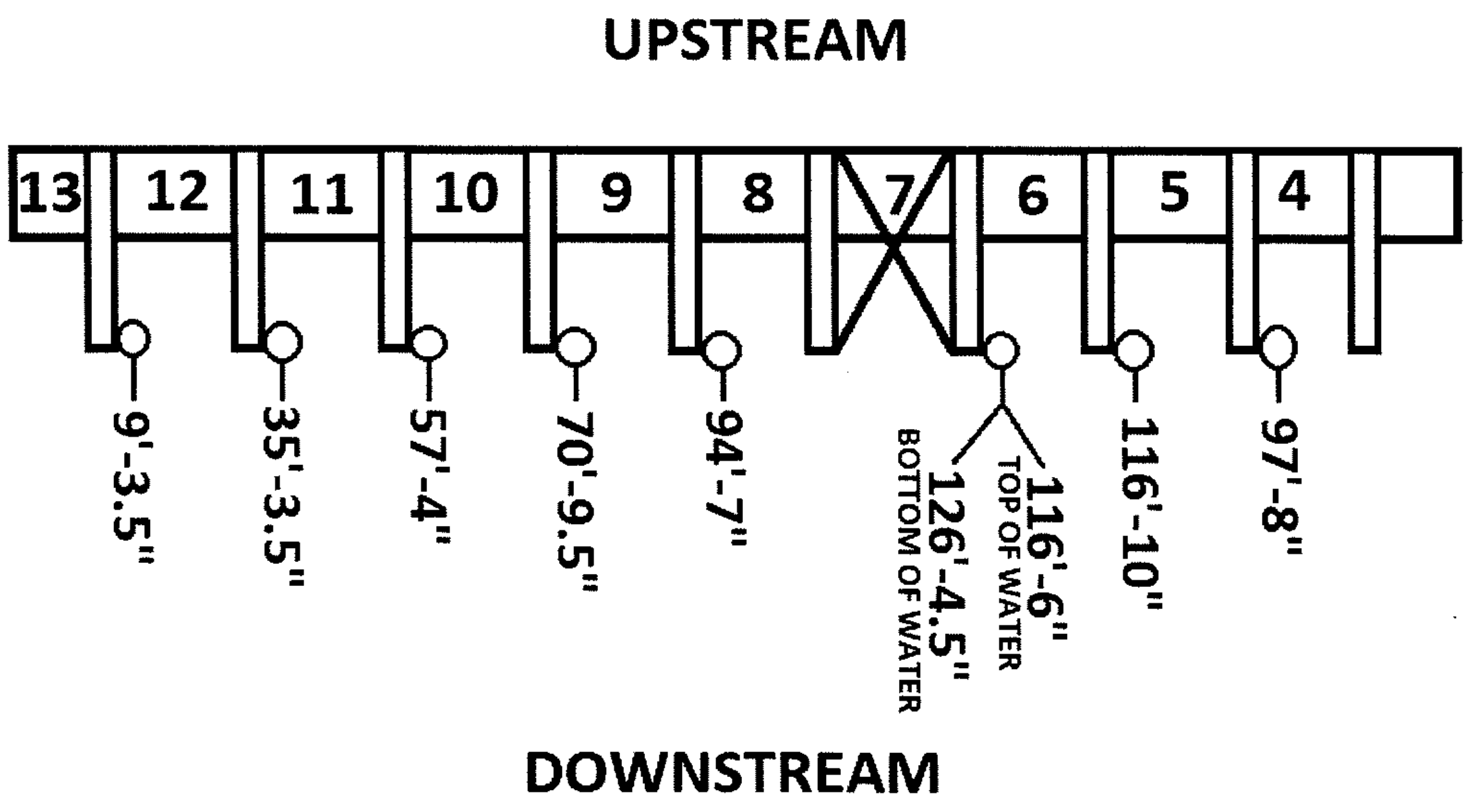
APPENDIX C

WORKING DRAWINGS, DETAILS, SKETCHES, AND MISCELLANEOUS DATA


NEIGHBORLY
LUR AKERS
BUFFALO SHOALS
DUKE ENERGY
Properties owned by the Town of Lake Lure and Duke Energy
1 inch = 200 feet
0 40 80 160 240 320 400 480 560 640
Feet
N

NOTE: All Measurments were taken from thr top of the Hand Rail on the Bridge Deck to the ground below and on the Left side of the Buttresses.



CHAPTER 3. ANALYSIS OF WALLS AND RETAINING STRUCTURES

Section 1. INTRODUCTION

1. SCOPE. Methods of determining earth pressures acting on walls and retaining structures are summarized in this chapter. Types of walls considered include concrete retaining walls and gravity walls that move rigidly as a unit, braced or tied bulkheads of thin sheeting that deflect according to the bracing arrangement, and double-wall cofferdams of thin sheeting to confine earth or rock fill.

2. RELATED CRITERIA. Additional criteria relating to the design and utilization of walls appear in the following sources:

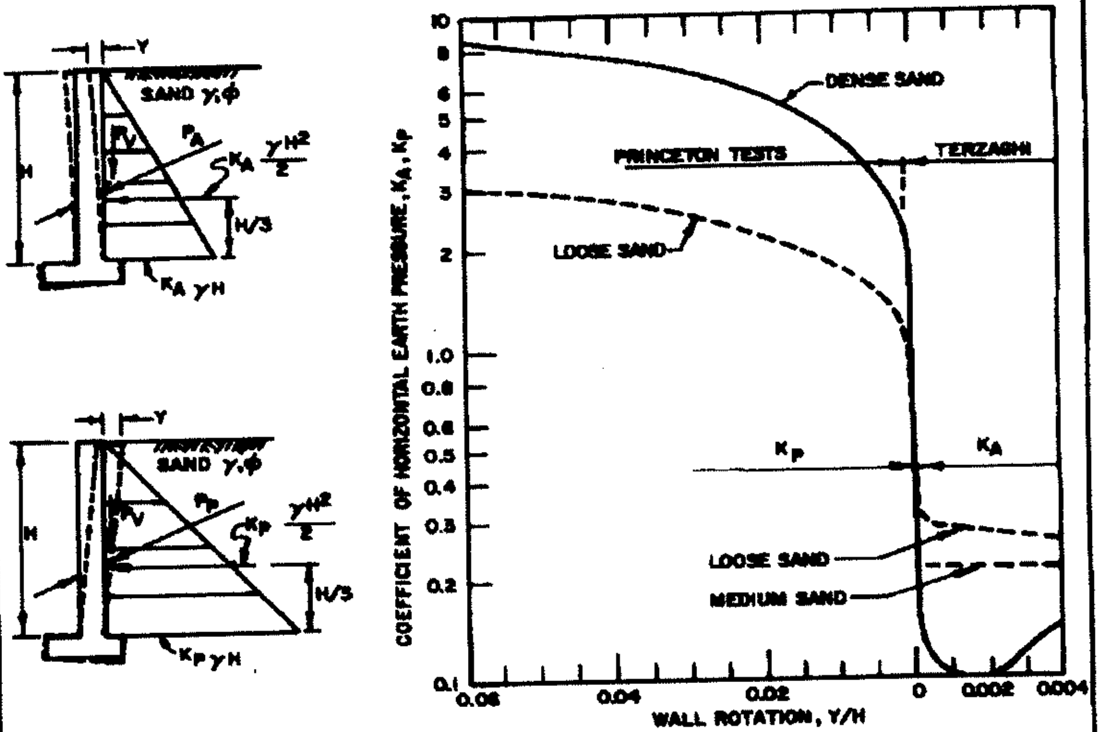
Subject	Source
Application of Bulkheads and Cofferdams to Waterfront Construction	NAVFAC DM-25.04
Structural Design of Retaining Walls	NAVFAC DM-2 Series

Section 2. COMPUTATION OF WALL PRESSURES

1. CONDITIONS. The pressure on retaining walls, bulkheads, or buried anchorages is a function of the relative movement between the structure and the surrounding soil.

a. Active State. Active earth pressure occurs when the wall moves away from the soil and the soil mass stretches horizontally sufficient to mobilize its shear strength fully, and a condition of plastic equilibrium is reached. (See Figure 1 from Reference 1, Excavations and Retaining Structures, by the Canadian Geotechnical Society.) The ratio of the horizontal component or active pressure to the vertical stress caused by weight of soil is the active pressure coefficient (K_a). The active pressure coefficient as defined above applies only to cohesionless soils.

b. Passive State. Passive earth pressure occurs when a soil mass is compressed horizontally, mobilizing its shear resistance fully (see Figure 1). The ratio of the horizontal component of passive pressure to the vertical stress caused by the weight of the soil is the passive pressure coefficient (K_p). The passive coefficient, as defined here, applies only the cohesionless soil. A soil mass that is neither stretched nor compressed is said to be in an at-rest state. The ratio of lateral stress to vertical stress is call the at-rest coefficient (K_o).



MAGNITUDES OF WALL ROTATION TO REACH FAILURE

SOIL TYPE AND CONDITION	ROTATION Y/H*	
	ACTIVE	PASSIVE
DENSE COHESIONLESS	.0005	.002
LOOSE COHESIONLESS	.002	.006
STIFF COHESIVE	.01	.02
SOFT COHESIVE	.02	.04

*Y = HORIZONTAL DISPLACEMENT
H = HEIGHT OF THE WALL

FIGURE 1
Effect of Wall Movement on Wall Pressures

2. COMPUTATION OF ACTIVE AND PASSIVE PRESSURES. See Figure 2 for formulas for active and passive pressures for the simple case on a frictionless vertical face with horizontal ground surface. Three basic conditions required for validity of the formulas are listed in Figure 2. Under these conditions the failure surface is a plane and the formulas represent pressures required for equilibrium of the wedge shaped failure mass.

The intensity of pressures applied depends on wall movements, as these control the degree of shear strength mobilization in surrounding soil. (See Figure 1 for the magnitude of the movement necessary for active condition to exist.) Wall friction and wall vertical movements also affect the passive and active pressures.

The effect of wall friction on active pressures is small and ordinarily is disregarded except in case of a settling wall where it can be very significant. The effect of wall friction on passive pressures is large, but definite movement is necessary for mobilization of wall friction. (See Table 1 for typical ultimate friction factors and adhesion between wall and backfill.) In the absence of specific test data, use these values in computations that include effects of wall friction.

Unless a wall is settling, friction on its back acts upward on the active wedge (angle [delta] is positive, see Figure 5), reducing active pressures. Generally, wall friction acts downward against the passive wedge (angle [delta] is negative), resisting its upward movement and increasing passive pressures.

a. Uniform Backfill, No Groundwater. Compute active and passive pressures by methods from Figure 2.

b. Sloping Backfill, No Groundwater, Granular Soil, Smooth Wall. Compute active and passive pressures by methods from Figure 3. Use Figure 4 to determine the position of failure surface for active and passive wedge.

c. Sloping Wall, Granular Soil With Wall Friction. Use Figure 5 (Reference 2, Tables for the Calculation of the Passive Pressure, Active Pressure and Bearing Capacity of Foundations, by Caquot and Kerisel) to compute active and passive earth pressure coefficients.

d. Sloping Backfill, Granular Soil with Wall Friction. Use Figure 6 (Reference 2) to compute active and passive earth pressure coefficient.

e. Uniform Backfill, Static Groundwater. Compute active earth and water pressures by formulas in Figure 7.

f. General Formula for Coefficients of Passive and Active Earth Pressure. Use Figure 8 for sloping wall with friction and sloping backfill.

g. Stratified Backfill, Sloping Groundwater Level. When conditions include layered soil, irregular surcharge, wall friction, and sloping groundwater level, determine active pressures by trial failure wedge. (See Figure 7.) Trial wedge is bounded by a straight failure plane or a series of straight segments at different inclination in each stratum. Commence the analysis with failure plane oriented at the angle shown in Figure 4.

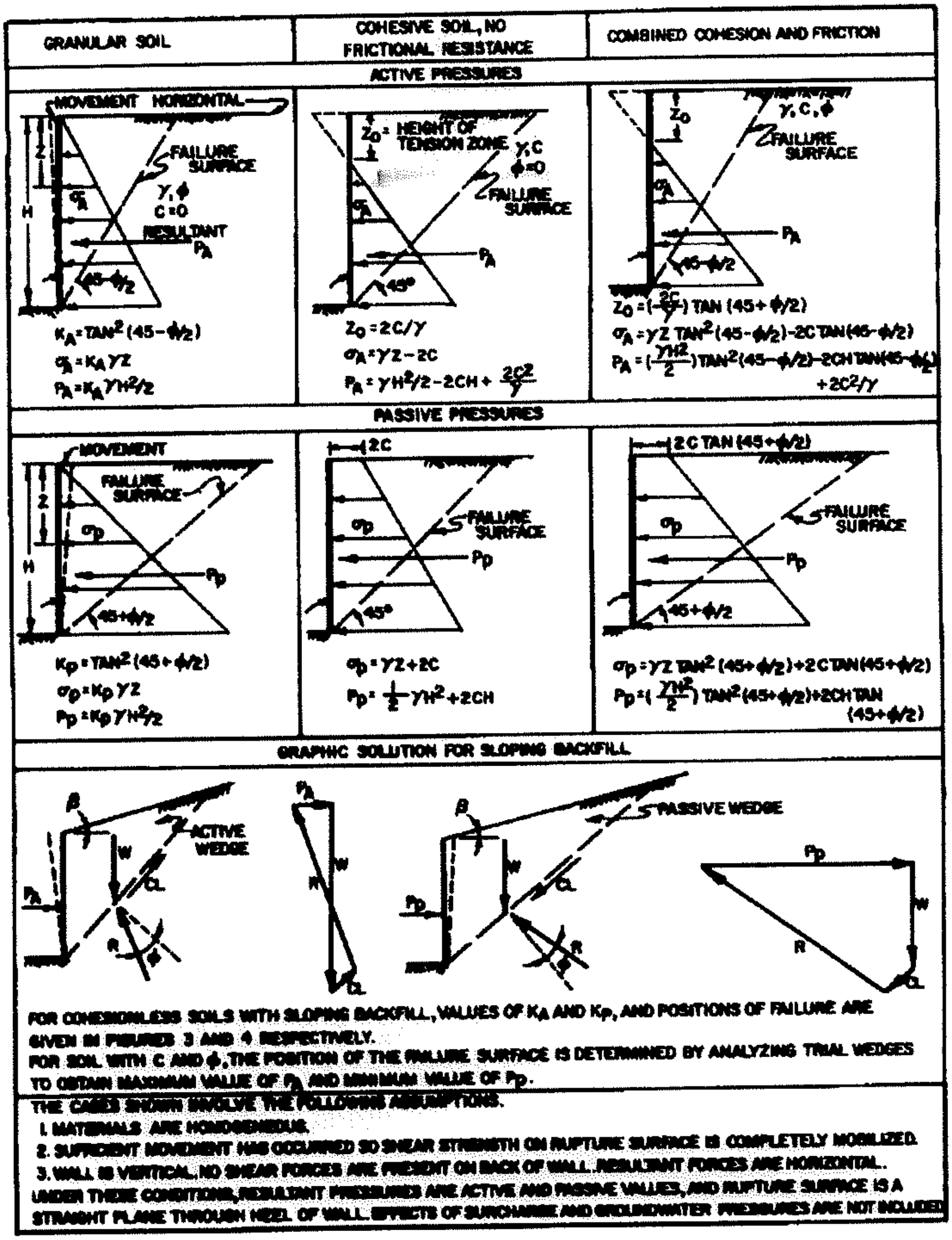
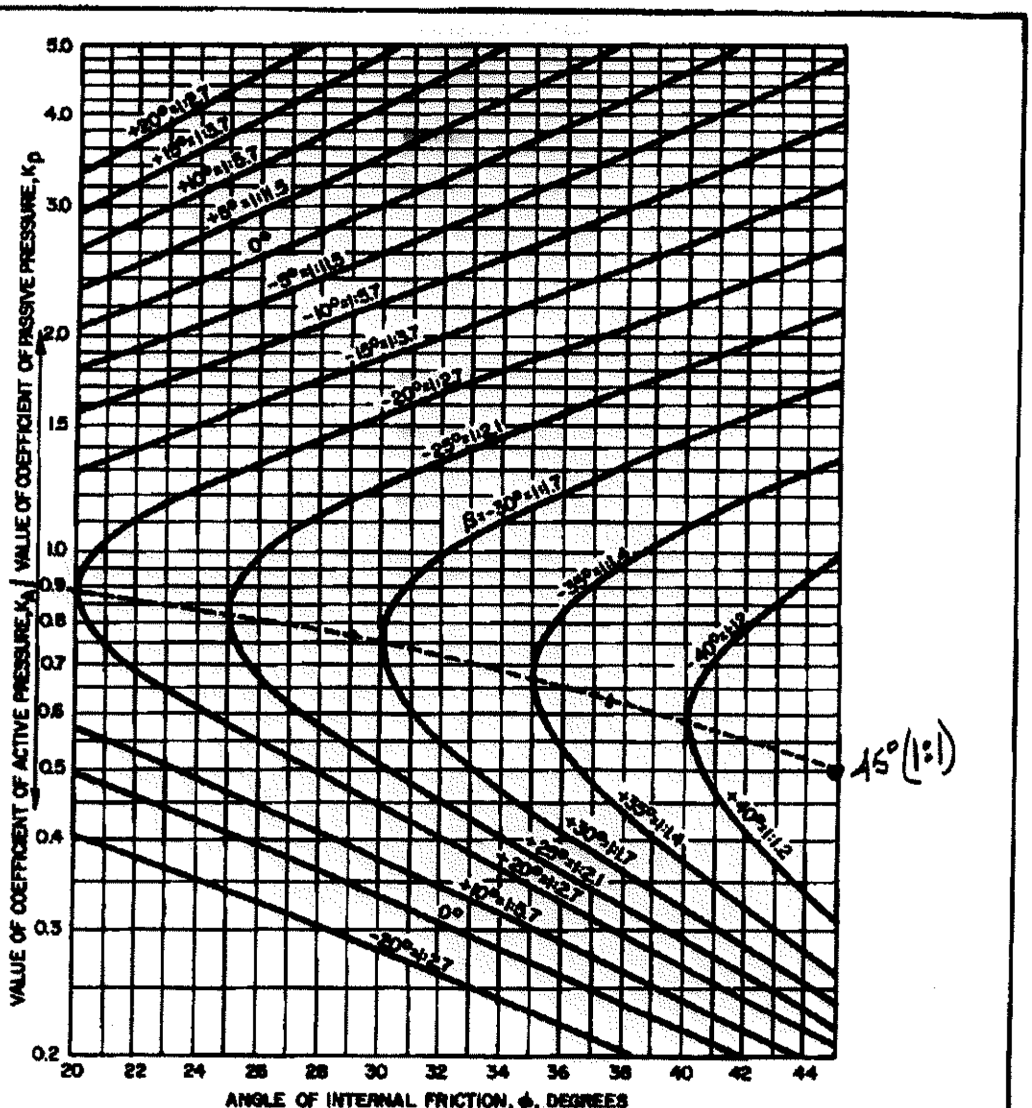


FIGURE 2
Computation of Simple Active and Passive Pressures

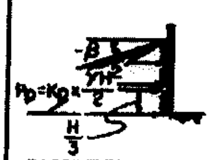
TABLE 1
Ultimate Friction Factors and Adhesion for Dissimilar Materials

Interface Materials	Friction factor, tan [delta]	Friction angle [delta] degrees
Mass concrete on the following foundation materials:		
Clean sound rock..............................	0.70	35
Clean gravel, gravel-sand mixtures, coarse sand...	0.55 to 0.60	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel..................	0.45 to 0.55	24 to 29
Clean fine sand, silty or clayey fine to medium sand..	0.35 to 0.45	19 to 24
Fine sandy silt, nonplastic silt.................	0.30 to 0.35	17 to 19
Very stiff and hard residual or preconsolidated clay..	0.40 to 0.50	22 to 26
Medium stiff and stiff clay and silty clay........	0.30 to 0.35	17 to 19
(Masonry on foundation materials has same friction factors.)		
Steel sheet piles against the following soils:		
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls.........................	0.40	22
Clean sand, silty sand-gravel mixture, single size hard rock fill...............................	0.30	17
Silty sand, gravel or sand mixed with silt or clay	0.25	14
Fine sandy silt, nonplastic silt.................	0.20	11
Formed concrete or concrete sheet piling against the following soils:		
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls.........................	0.40 to 0.50	22 to 26
Clean sand, silty sand-gravel mixture, single size hard rock fill...............................	0.30 to 0.40	17 to 22
Silty sand, gravel or sand mixed with silt or clay	0.30	17
Fine sandy silt, nonplastic silt.................	0.25	14
Various structural materials:		
Masonry on masonry, igneous and metamorphic rocks:		
Dressed soft rock on dressed soft rock..........	0.70	35
Dressed hard rock on dressed soft rock..........	0.65	33
Dressed hard rock on dressed hard rock..........	0.55	29
Masonry on wood (cross grain)....................	0.50	26
Steel on steel at sheet pile interlocks...........	0.30	17

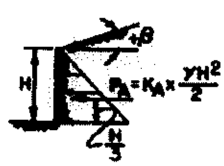
Interface Materials (Cohesion)	Adhesion c_a (psf)
Very soft cohesive soil (0 - 250 psf)	0 - 250
Soft cohesive soil (250 - 500 psf)	250 - 500
Medium stiff cohesive soil (500 - 1000 psf)	500 - 750
Stiff cohesive soil (1000 - 2000 psf)	750 - 950
Very stiff cohesive soil (2000 - 4000 psf)	950 - 1,300



$K_A = \left[\frac{\cos\phi}{1+\sqrt{\sin\phi(\sin\phi-\cos\phi\tan\beta)}}\right]^2$; $K_P = \left[\frac{\cos\phi}{1-\sqrt{\sin\phi(\sin\phi+\cos\phi\tan\beta)}}\right]^2$



PASSIVE PRESSURE

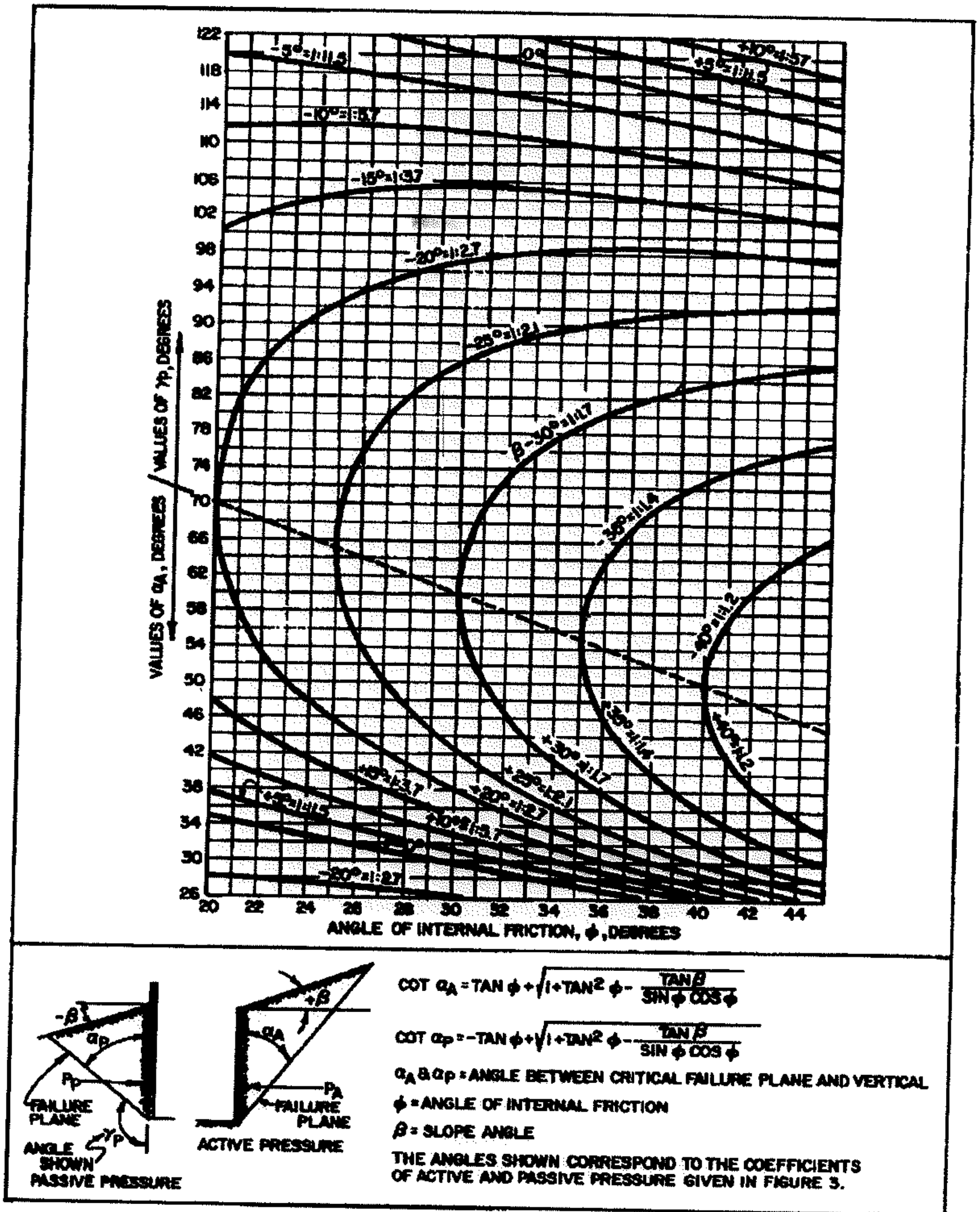


ACTIVE PRESSURE

K_A & K_P = COEFFICIENTS FOR COULOMB'S EQUATION FOR ACTIVE AND PASSIVE EARTH PRESSURE (NO SHEAR STRESS ON VERTICAL PLANES).

P_A = ACTIVE RESULTANT
P_P = PASSIVE RESULTANT
γ = UNIT WEIGHT OF SOIL
H = HEIGHT OF WALL
ϕ = ANGLE OF INTERNAL FRICTION
β = SLOPE ANGLE

FIGURE 3
Active and Passive Coefficients, Sloping Backfill
(Granular Soils)



$$\cot \alpha_A = \tan \phi + \sqrt{1 + \tan^2 \phi - \frac{\tan \beta}{\sin \phi \cos \phi}}$$

$$\cot \alpha_P = -\tan \phi + \sqrt{1 + \tan^2 \phi - \frac{\tan \beta}{\sin \phi \cos \phi}}$$

α_A & α_P = ANGLE BETWEEN CRITICAL FAILURE PLANE AND VERTICAL

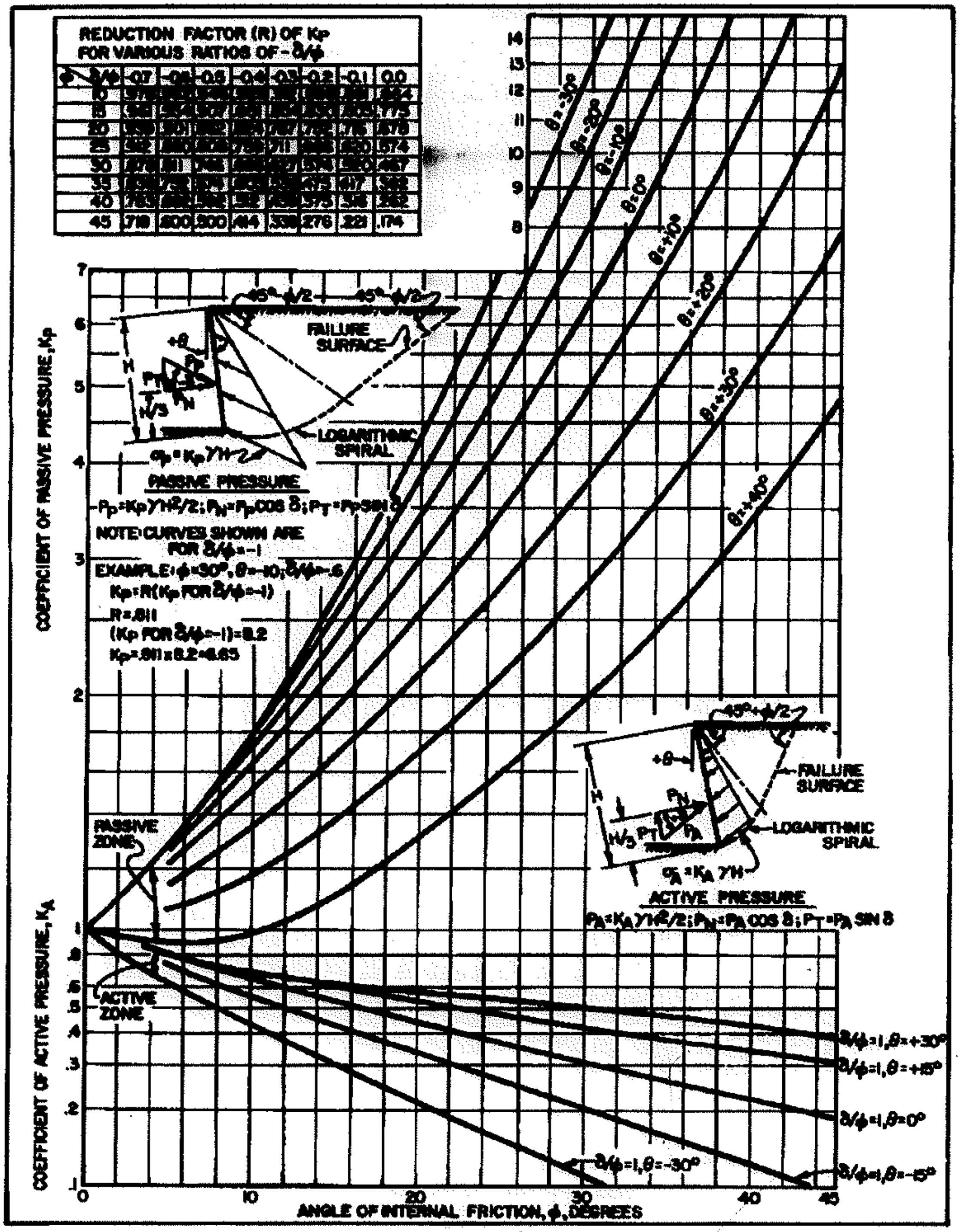
ϕ = ANGLE OF INTERNAL FRICTION

β = SLOPE ANGLE

THE ANGLES SHOWN CORRESPOND TO THE COEFFICIENTS OF ACTIVE AND PASSIVE PRESSURE GIVEN IN FIGURE 3.

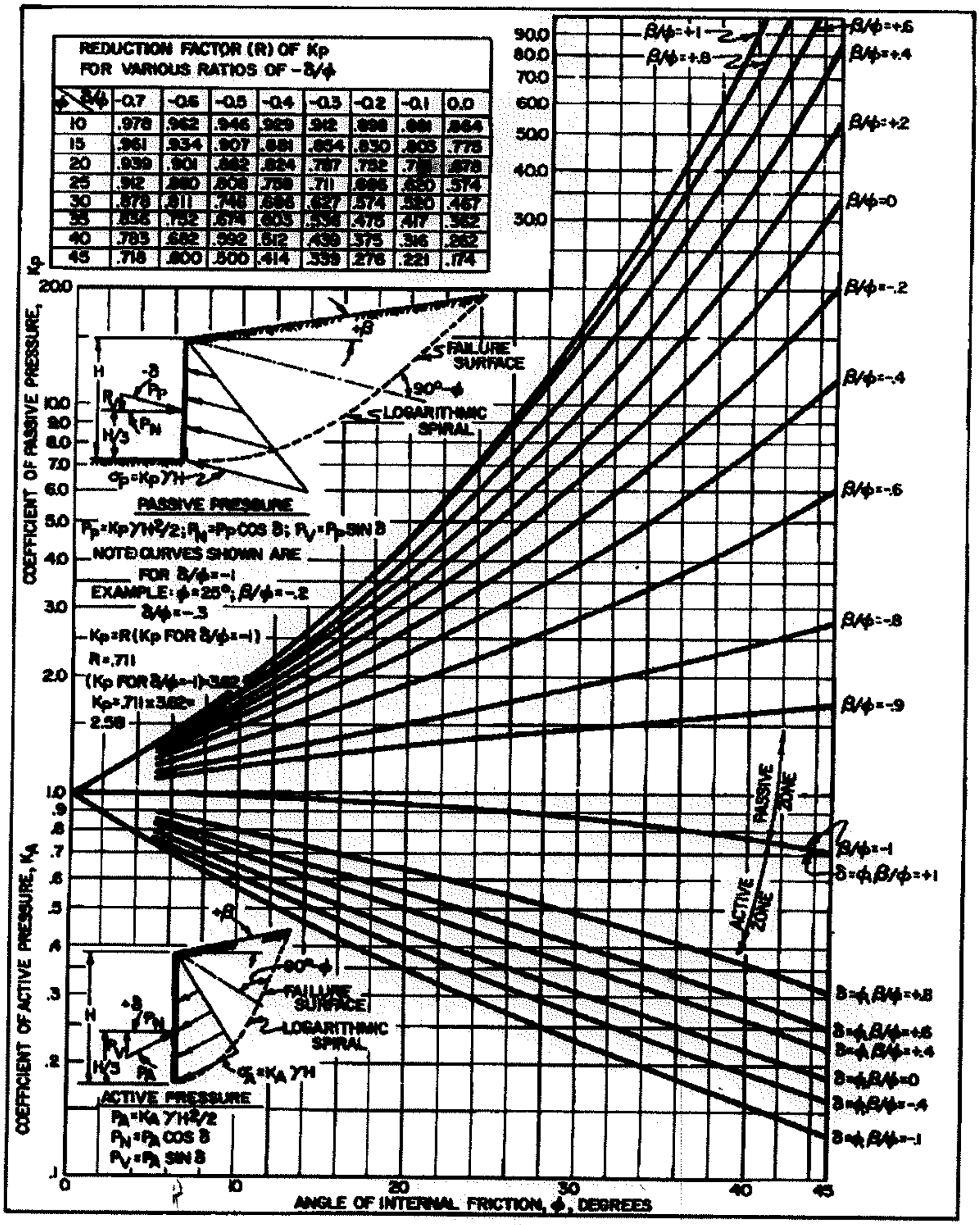
FIGURE 4

Position of Failure Surface for Active and Passive Wedges (Granular Soils)



φ \ δ/φ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
15	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
20	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
25	[illegible]	[illegible]	[illegible]	[illegible]	.711	[illegible]	[illegible]	[illegible]
30	[illegible]	.811	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
35	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
40	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
45	.718	.600	.500	.414	.338	.276	.221	.174

FIGURE 5
Active and Passive Coefficients with Wall Friction
(Sloping Wall)



REDUCTION FACTOR (R) OF Kp FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

FIGURE 6
Active and Passive Coefficients with Wall Friction (Sloping Backfill)

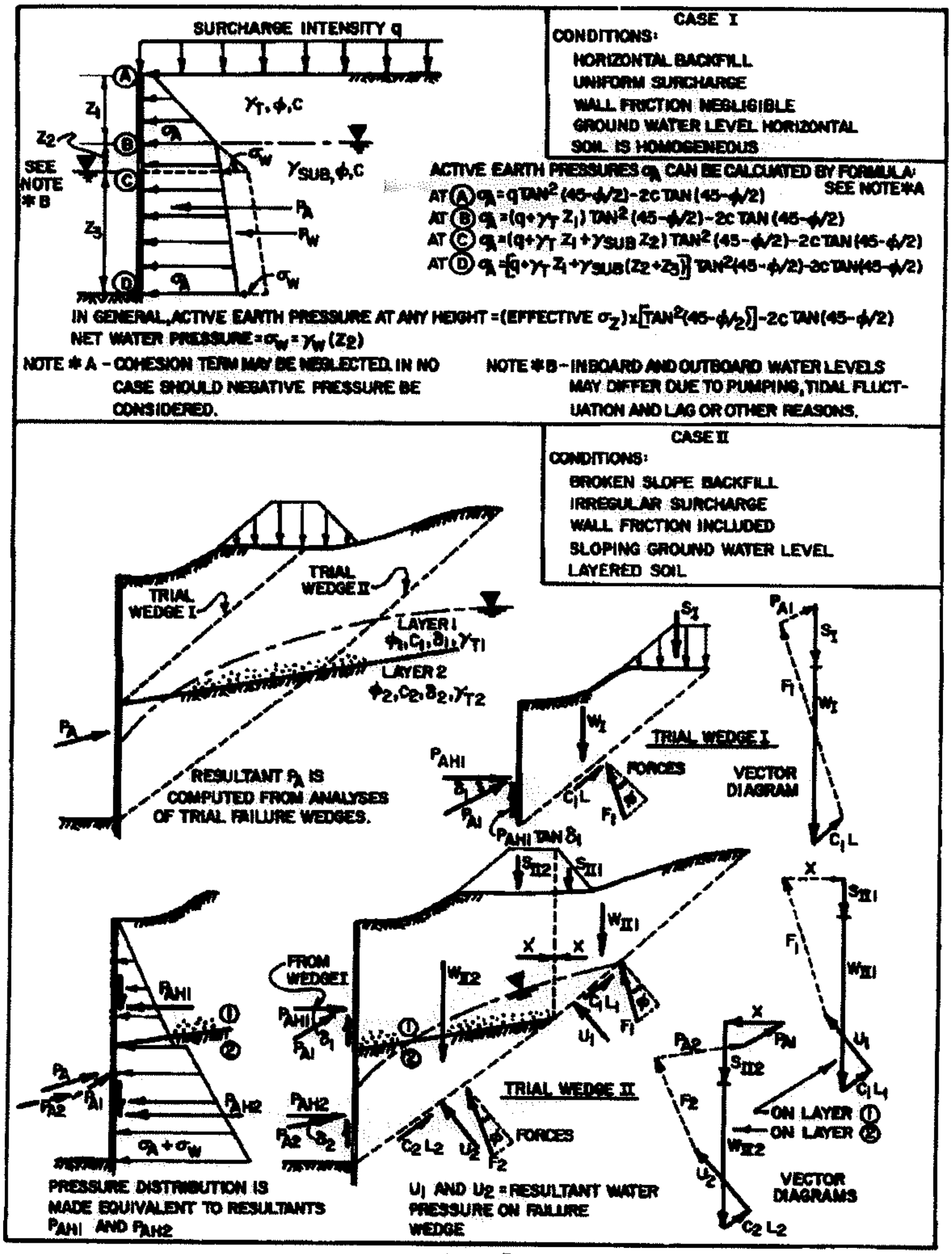
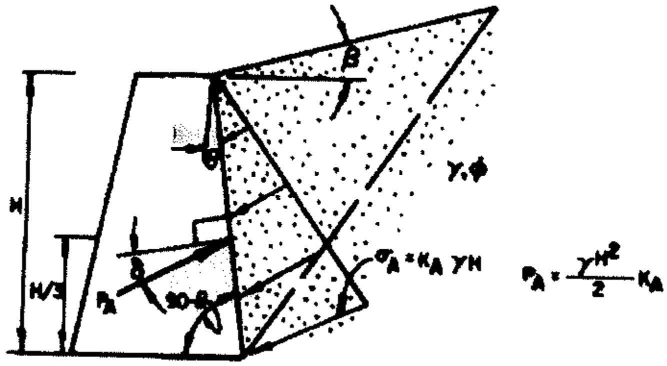
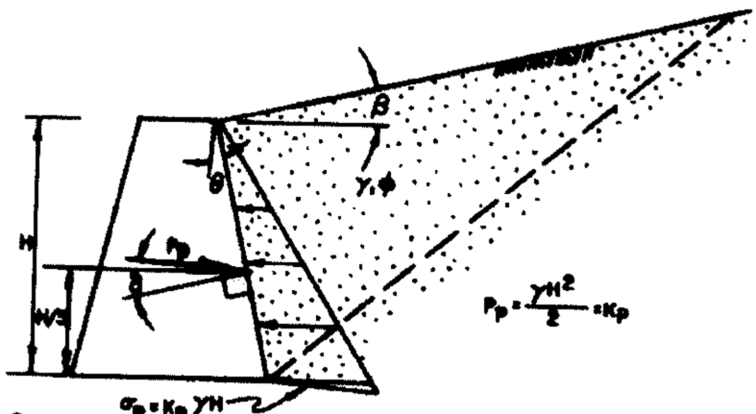


FIGURE 7
Computation of General Active Pressures



$$K_A = \frac{\cos^2(\phi-\theta)}{\cos^2\theta \cos(\theta+\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\theta+\delta)\cos(\theta-\beta)}}\right]^2}$$



$$K_p = \frac{\cos^2(\theta+\phi)}{\cos^2\theta \cos(\theta-\delta)\left[1-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi+\beta)}{\cos(\theta-\delta)\cos(\theta-\beta)}}\right]^2}$$

K_p VALUES ARE SATISFACTORY FOR $\delta \leq \phi/3$ BUT ARE UNCONSERVATIVE FOR $\delta > \phi/3$ AND THEREFORE SHOULD NOT BE USED.

FIGURE 8
Coefficients K_A and K_p for Walls with Sloping Wall and Friction, and Sloping Backfill

Compute resultant passive force by trial failure wedge analysis. (See Figure 9). When wall friction is included, compute pressures from a failing mass bounded by a circular arc and straight plane. Determine location of passive resultant by summing moments about toe of wall of all forces on that portion of the failing mass above the circular arc. Depending on complexity of cross section, distribute passive pressures to conform to location of resultant, or analyze trial failure surfaces at intermediate heights in the passive zone. When wall friction is neglected, the trial failure surface is a straight plane. See Figure 2.

(1) Simple Cross Section. For a simple cross section behind a wall, analyze the trial failure plane extending upward from the lowest point of the active zone on the wall. Determine the location of the active resultant by summing moments of all forces on the wedge about toe of wedge. Distribute active pressures to conform to the location of resultant.

(2) Complicated Cross Section. For complicated cross sections, analyze trial wedges at intermediate heights above the base of the active zone to determine pressure distribution in more detail. Force acting on an increment of wall height equals difference in resultant forces for wedges taken from the top and bottom of that increment.

3. EFFECT OF GROUNDWATER CONDITIONS. Include in pressure computations the effect of the greatest unbalanced water head anticipated to act across the wall.

a. General Conditions. For a major structure, analyze seepage and drainage effect by flow net procedures. Uplift pressures influencing wall forces are those acting on failure surface of active or passive wedge. Resultant uplift force on failure surface determined from flow net is applied in force diagram of the failure wedge. See vector U, the resultant water force, in Figures 7 and 9.

b. Static Differential Head. Compute water pressures on walls as shown in top panel of Figure 10.

c. Rainfall on Drained Walls. For cohesionless materials, sustained rainfall Increases lateral force on wall 20 to 40 percent over dry backfill, depending on backfill friction angle. The center panel of Figure 10 (Reference 3, Contribution to the Analysis of Seepage Effects in Backfills, by Gray) shows flow net set up by rainfall behind a wall with vertical drain. This panel gives the magnitude of resultant uplift force on failure wedge for various inclinations of failure plane to be used in analysis of the active wedge.

d. Seepage Beneath Wall. See bottom panel of Figure 10 (Reference 4, The Effect of Seepage on the Stability of Sea Walls, by Richart and Schmertmann) for correction to be applied to active and passive pressures in cohesionless material for steady seepage beneath a wall.

4. SURCHARGE LOADING. For the effects of surcharge loading, see Figures 7 and 9.

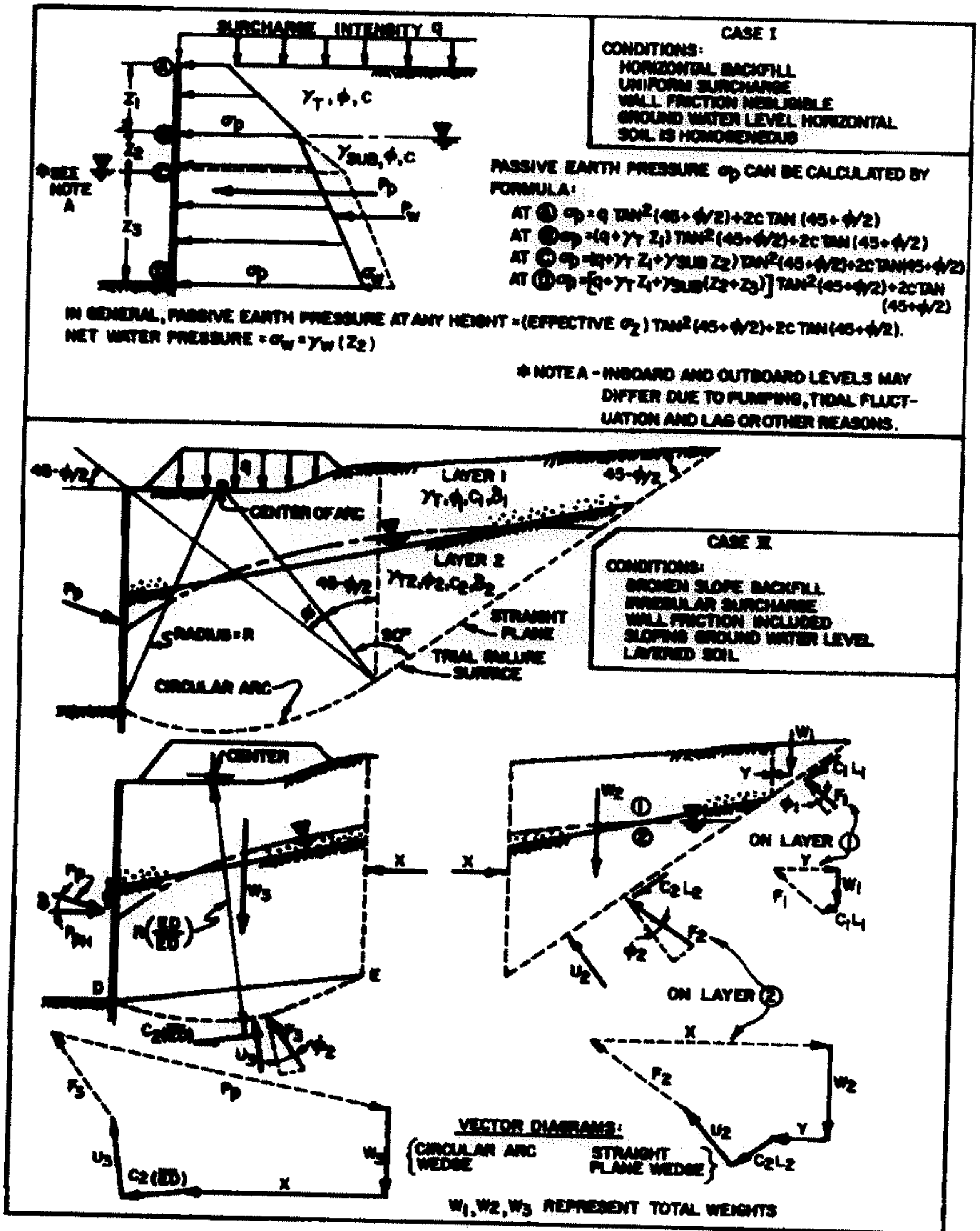


FIGURE 9
Computation of General Passive Pressures

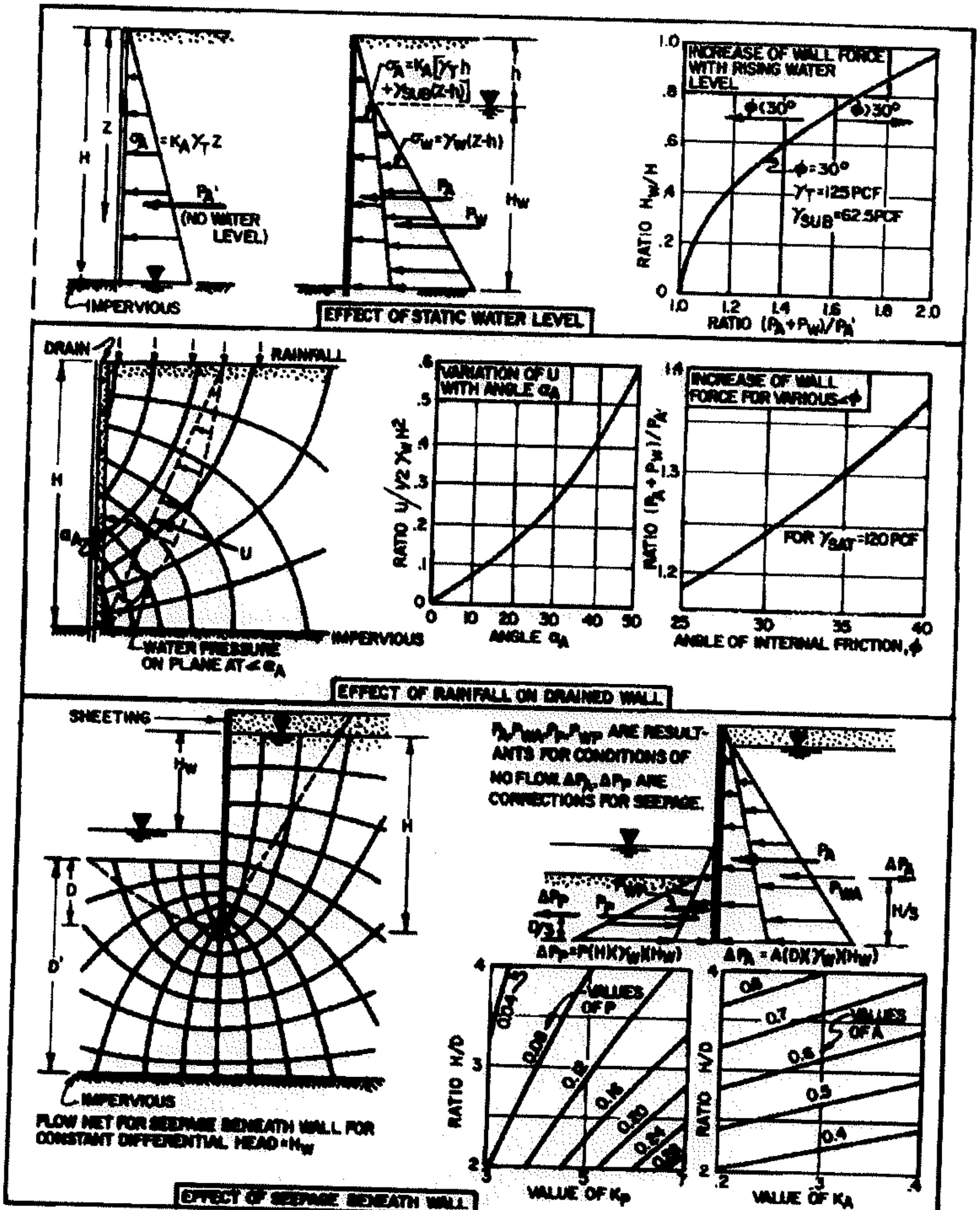


FIGURE 10
Effect of Groundwater Conditions on Wall Pressures

a. Point Load and Live Load. Use Figure 11 (Reference 5, Anchored Bulkheads, based on the work by Terzaghi) to compute lateral pressure on wall due to point load and line loads; this assumes an unyielding rigid wall and the lateral pressures are approximately double the values obtained by elastic equations. The assumption of an unyielding rigid wall is conservative and its applicability should be evaluated for each specific wall.

b. Uniform Loading Area. For uniform surcharge loading lateral stress can be computed by treating the surcharge as if it were backfill and multiplying the vertical stress at any depth by the appropriate earth pressure coefficient.

c. Uniform Rectangular Surcharge Loading. For the effect of this loading see Figure 12 (see Reference 6, Lateral Support Systems and Underpinning, 2 Volume 1, Design and Construction (Summary), by Goldberg, et al.). If the construction procedures are such that the wall will move during the application of live loads, then the pressure calculated from Figure 12 will be conservative.

d. Practical Considerations. For design purposes, it is common to consider a distributed surface load surcharge on the order of 300 psf to account for storage of construction materials and equipment. This surcharge is usually applied within a rather limited work area of about 20 feet to 30 feet from the wall and is also intended to account for concentrated loads from heavy equipment (concrete trucks, cranes, etc.) located more than about 20 feet away. If such equipment is anticipated within a few feet of the wall, it must be accounted for separately.

5. WALL MOVEMENT. For the effect of wall movement on the earth pressure coefficients, see Figure 1.

a. Wall Rotation. When the actual estimated wall rotation is less than the value required to fully mobilize active or passive conditions, adjust the earth pressure coefficients by using the diagram on the upper right hand corner of Figure 1. Relatively large movements are required to mobilize the passive resistance. A safety factor must be applied to the ultimate passive resistance in order to limit movements.

b. Wall Translation. Wall uniform translation required to immobilize ultimate passive resistance or active pressure is approximately equivalent to movement of top of wall based on rotation criteria given in Figure 1.

c. Internally Braced Flexible Wall. Sheeting on cuts rigidly braced at the top undergoes insufficient movement to produce fully active conditions, Horizontal pressures are assumed to be distributed in a trapezoidal diagram. (See Section 4.) The resultant force is higher than theoretical active force. For clays, the intensity and distribution of horizontal pressures depend on the stability number N_{o} = [gamma] H/c. (See Section 4.)

d. Tied Back Walls. Soil movement associated with prestressed tied back walls is usually less than with internally braced flexible walls, and design pressures are higher. (See Section 4.)

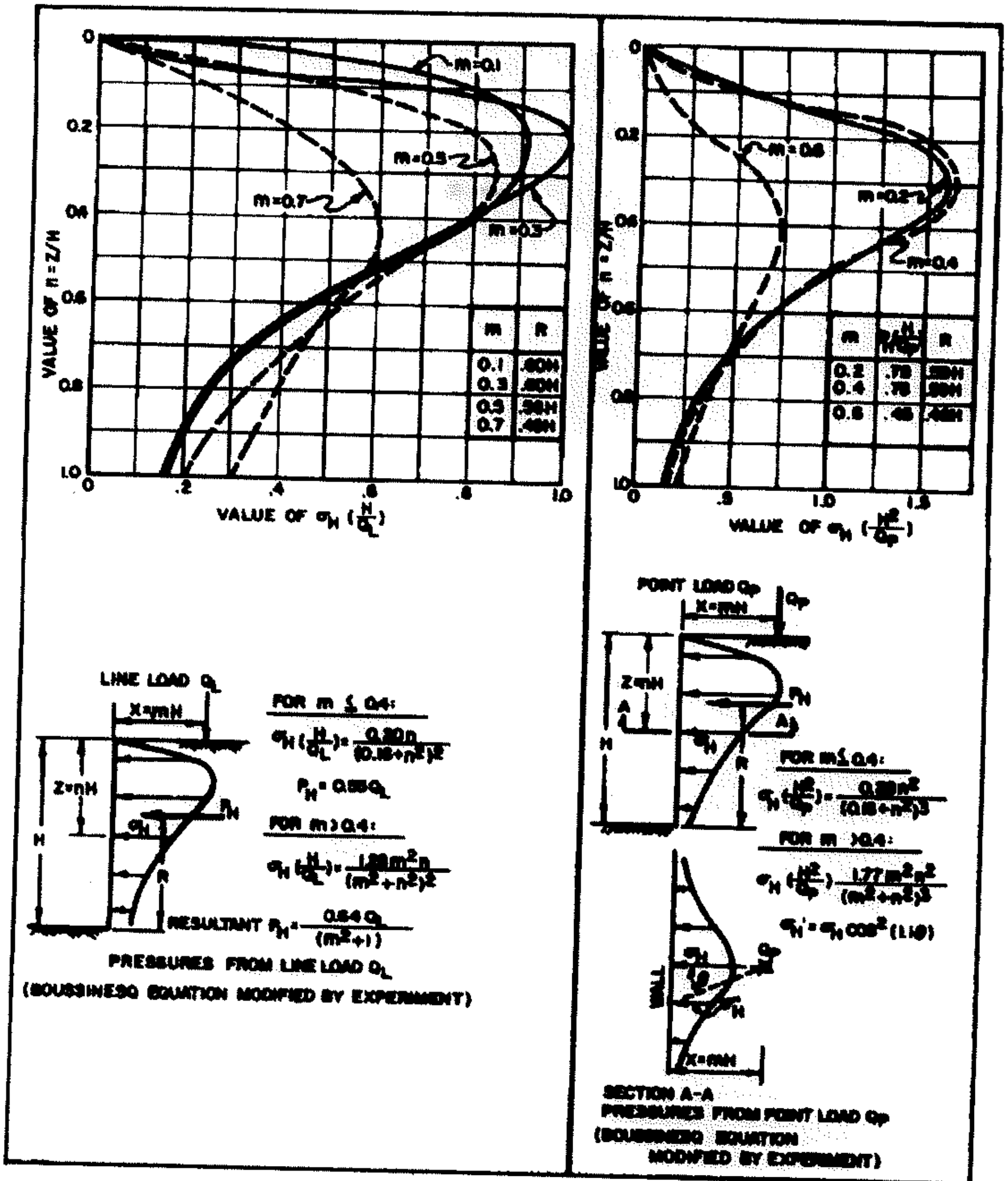


FIGURE 11
Horizontal Pressures on Rigid Wall from Surface Load

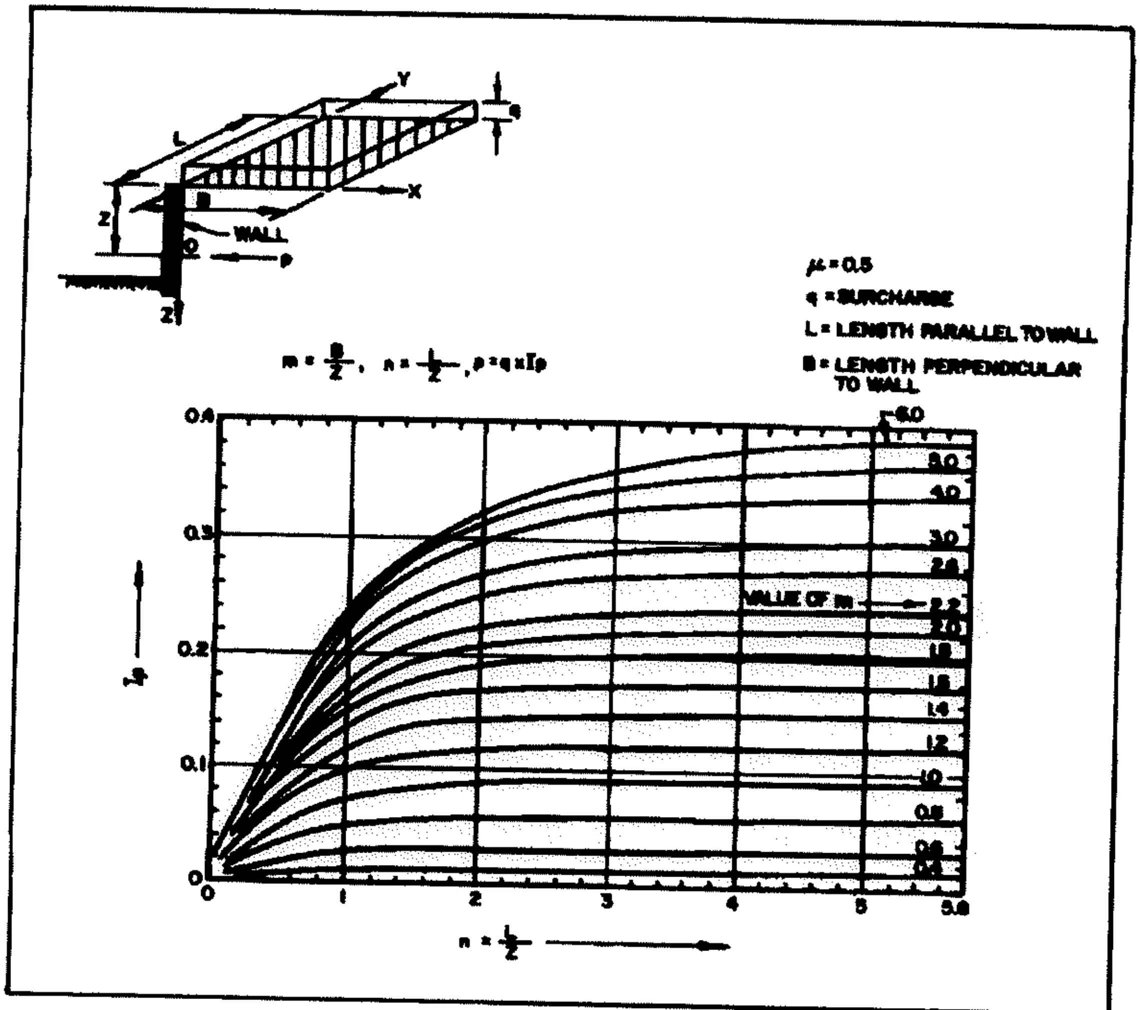


FIGURE 12
Lateral Pressure on an Unyielding Wall due to Uniform Rectangular Surface Load

e. Restrained Walls. If a wall is prevented from even slight movement, then the earth remains at or near the value of at-rest conditions. The coefficient of earth pressure at-rest, K_{o}, for normally consolidated cohesive or granular soils is approximately:

$$K_{o} = 1 - \sin [\theta]'$$

where: [theta]' = effective friction angle

Thus for [theta]' = 300, K_{o} = 0.5.

For over-consolidated soils and compacted soils the range of K_{o} may be on the order of 1.0. In cohesionless soils, full at-rest pressure will occur only with the most rigidly supported wall. In highly plastic clays, soil may creep, and if wall movement is prevented, at-rest conditions may redevelop even after active pressures are established.

f. Basement and Other Below Grade Walls. Pressure on walls below grade may be computed based on restraining conditions that prevail, type of backfill, and the amount of compaction.

6. EFFECT OF CONSTRUCTION PROCEDURES.

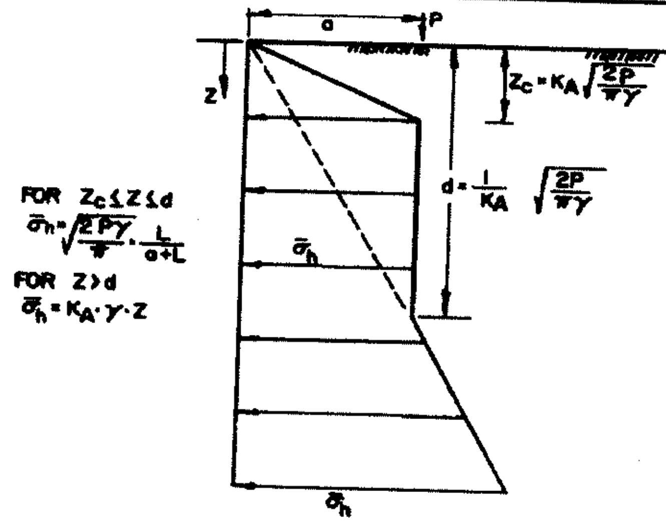
a. Staged Construction. As earth pressures are influenced by wall movement, it is important to consider each stage of construction, especially with regard to brace placement and its effects.

b. Compaction. Compaction of backfill in a confined wedge behind the wall tends to increase horizontal pressures beyond those represented by active or at-rest values. For guidance on horizontal pressure computations associated with the compaction of granular soil, see Figure 13 (after Reference 7, Retaining Wall Performance During Backfilling, by Ingold).

Clays and other fine-grained soils, as well as granular soils, with considerable amount of clay and silt (>/=15%) are not normally used as backfill material. Where they must be used, the earth pressure should be calculated on the basis of "at-rest" conditions or higher pressure with due consideration to potential poor drainage conditions, swelling, and frost action.

c. Hydraulic Fills. Active pressure coefficients for loose hydraulic fill materials range from about 0.35 for clean sands to 0.50 for silty fine sands. Place hydraulic fill by procedures which permit runoff of wash water and prevent building up large hydrostatic pressures. For further guidance see discussion on dredging in DM-7.3, Chapter 3.

7. EARTHQUAKE LOADING. The pressure during earthquake loading can be computed by the Coulomb theory with the additional forces resulting from ground acceleration. For further guidance on the subject see Reference 8, Design of Earth Retaining Structures for Dynamic Loads, by Seed and Whitman. A synopsis of some material from this Reference follows:



P (ROLLER LOAD) = $\frac{\text{DEAD WT. OF ROLLER + CENTRIFUGAL FORCE}}{\text{WIDTH OF ROLLER}}$

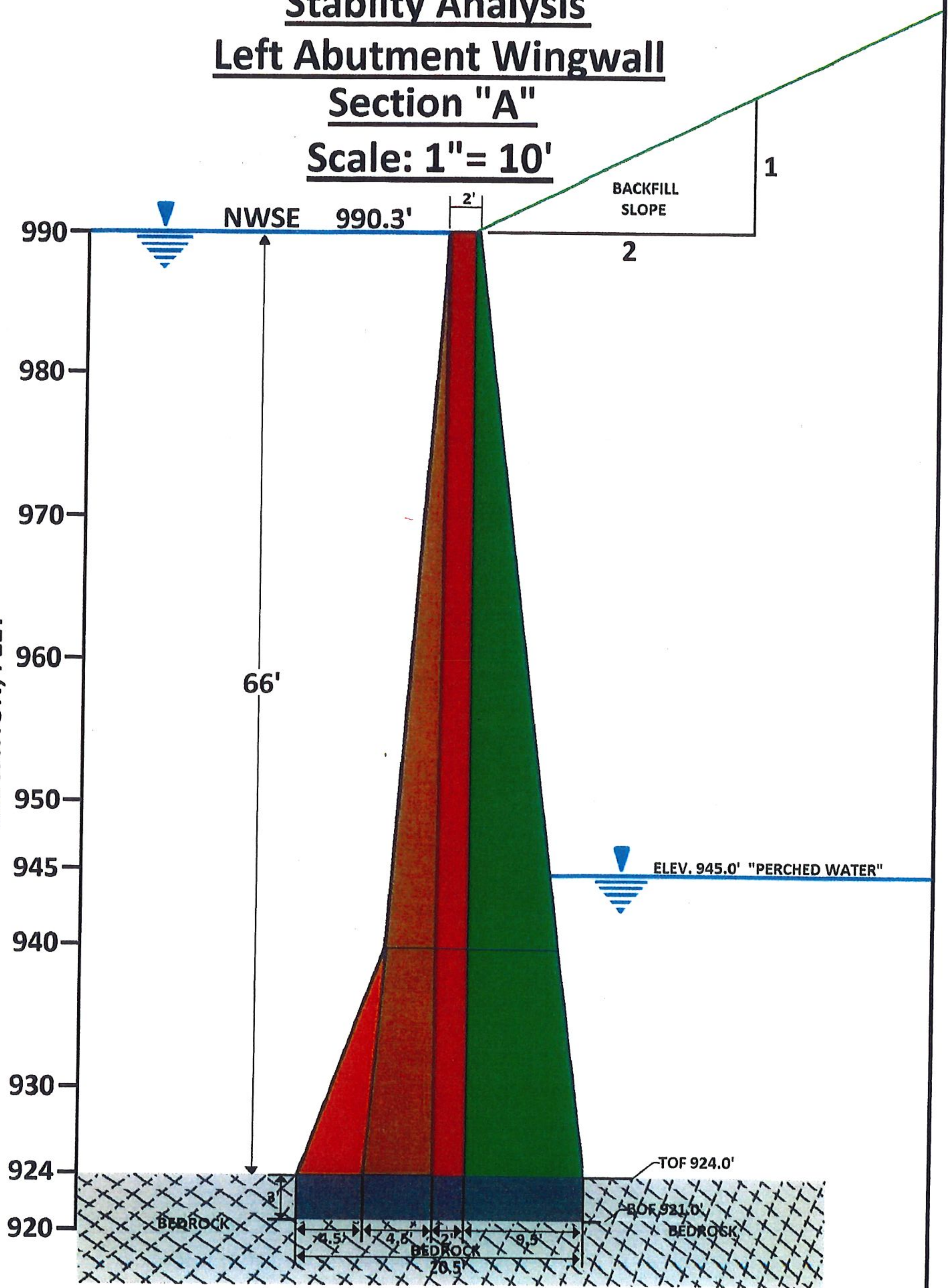
a : DISTANCE OF ROLLER FROM WALL

L: LENGTH OF ROLLER

USE FIGURES 2, 3, 5 OR 6 FOR K_A

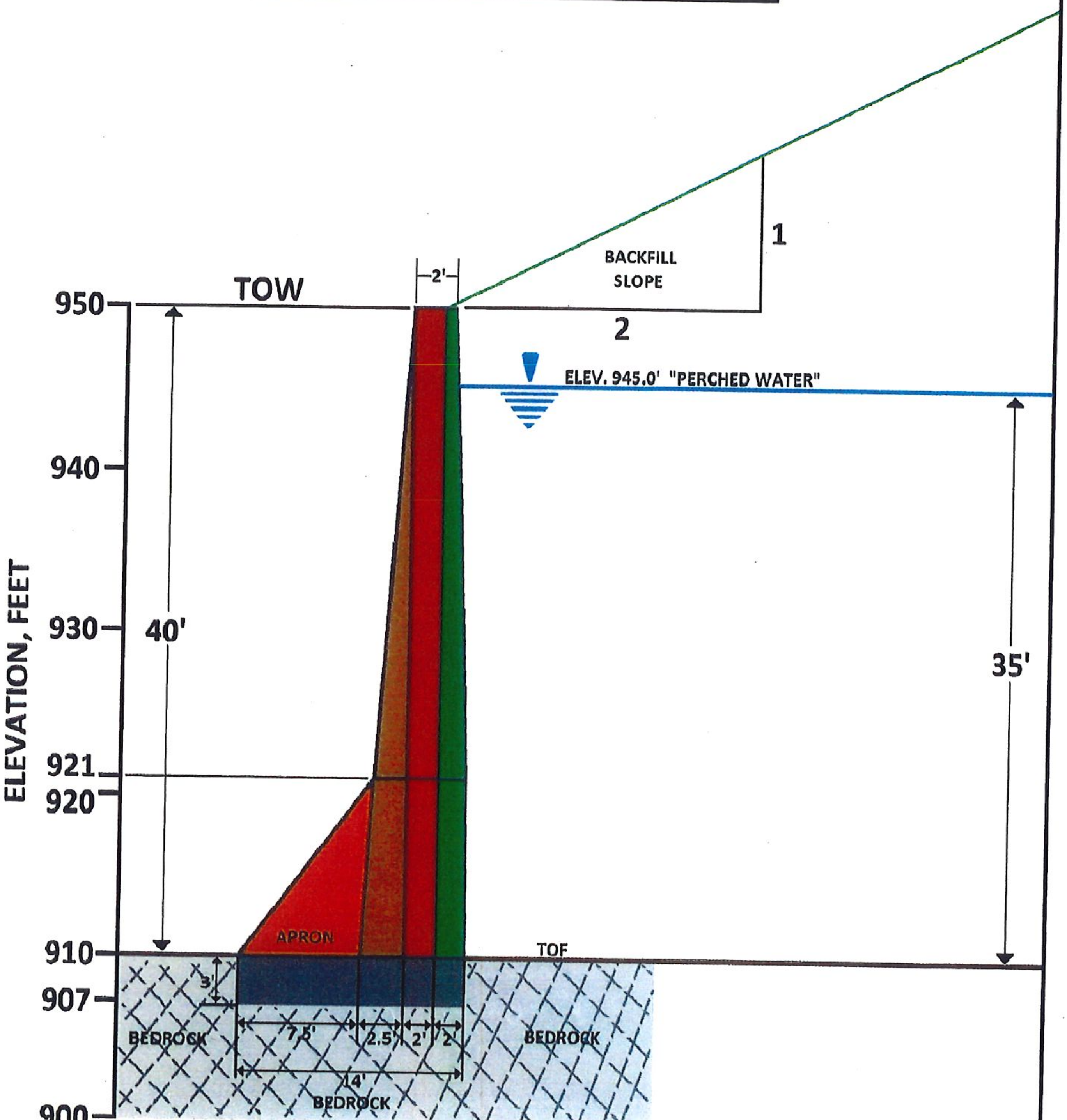
FIGURE 13
Horizontal Pressure on Walls from Compaction Effort

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

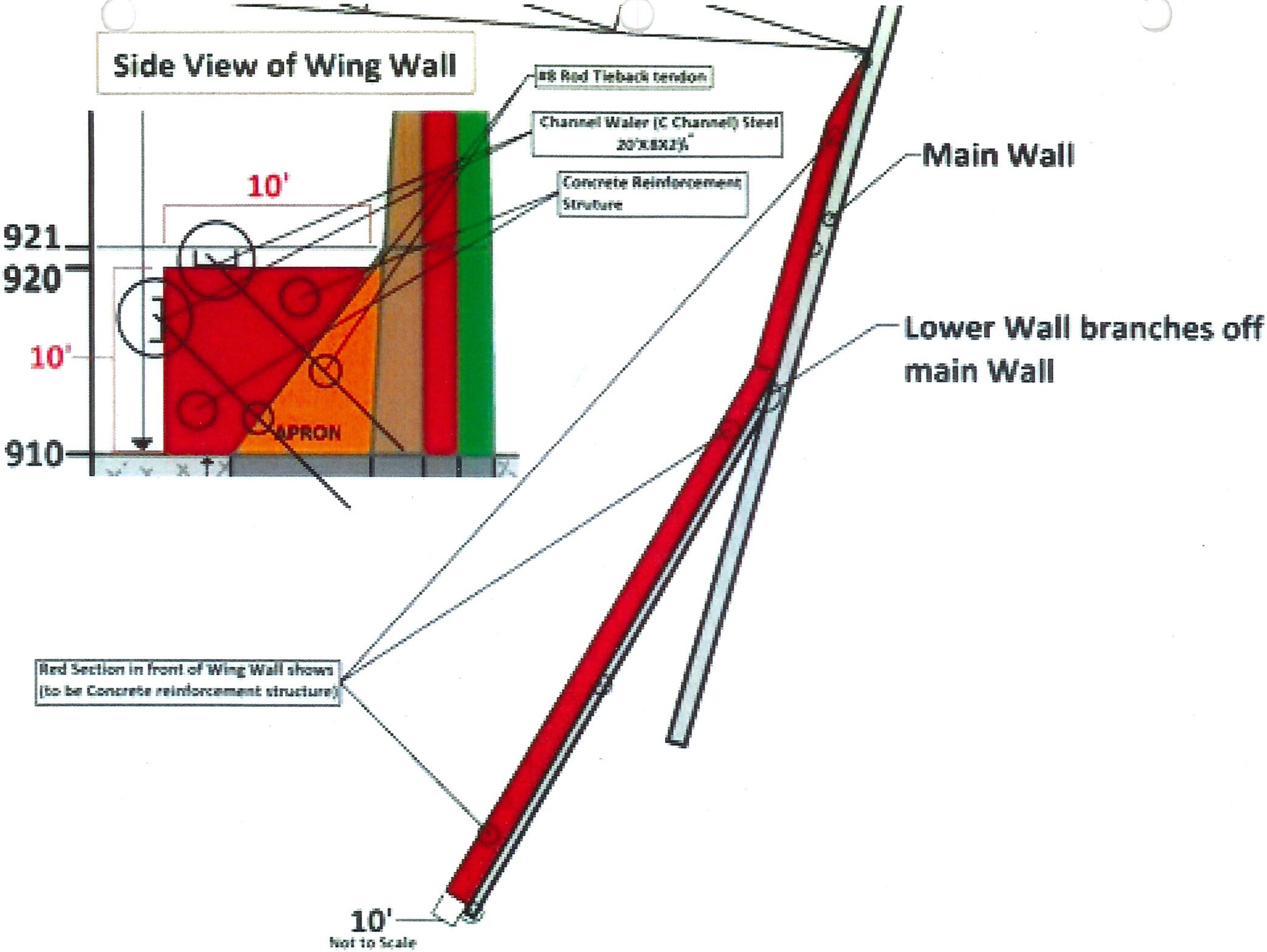


SHEET __ OF __

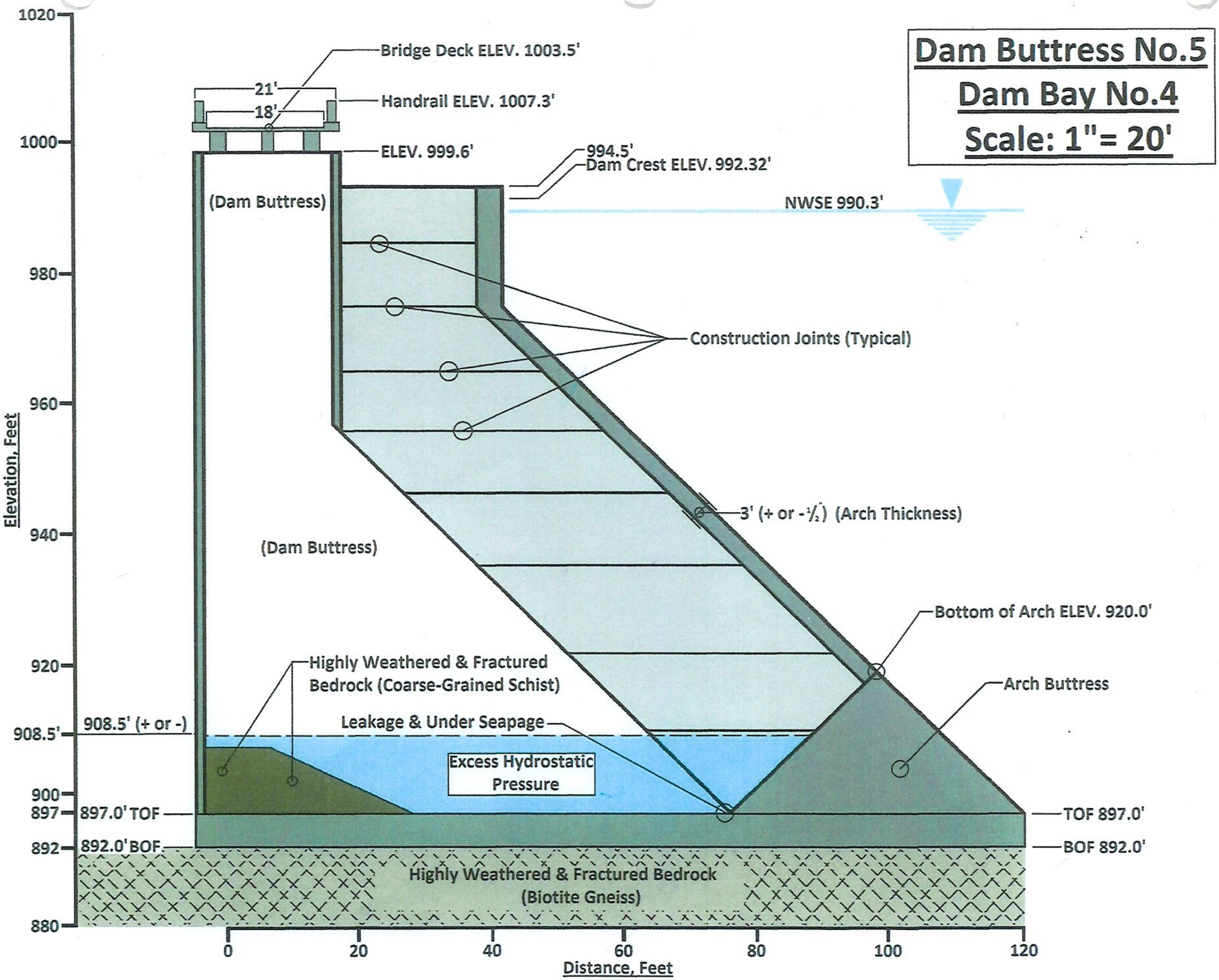
Stabilty Analysis (Cont'd)
Left Abutment Wingwall
Section "B"
Scale: 1"= 10'

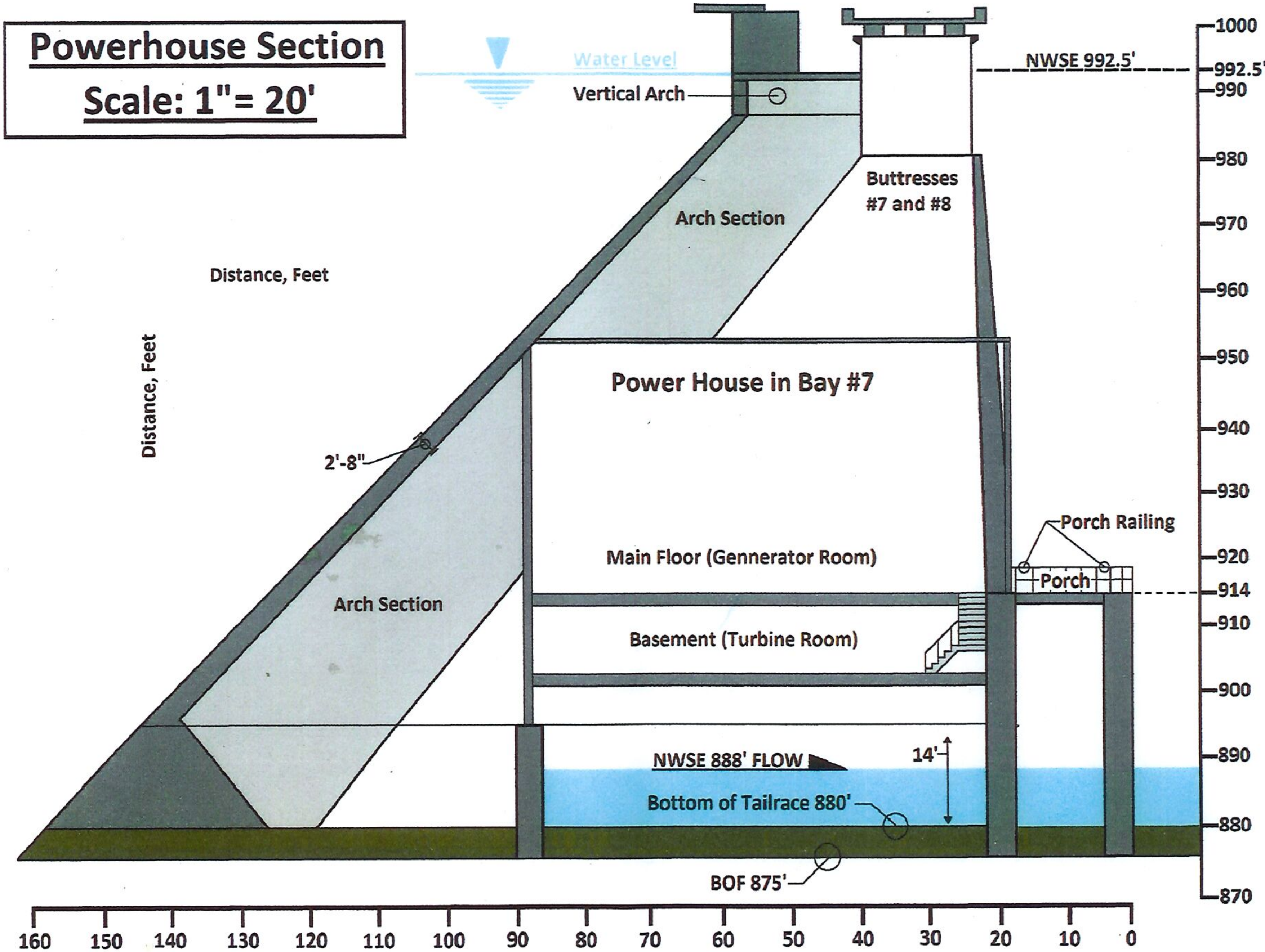


SHEET __ OF __



Side View of Wing Wall
#8 Rod Tieback tendon
Channel Waler (C Channel) Steel 20'X8X2½"
Concrete Reinforcement Struture
10'
921
920
10'
910
APRON
Main Wall
Lower Wall branches off main Wall
Red Section in front of Wing Wall shows (to be Concrete reinforcement structure)
10'
Not to Scale

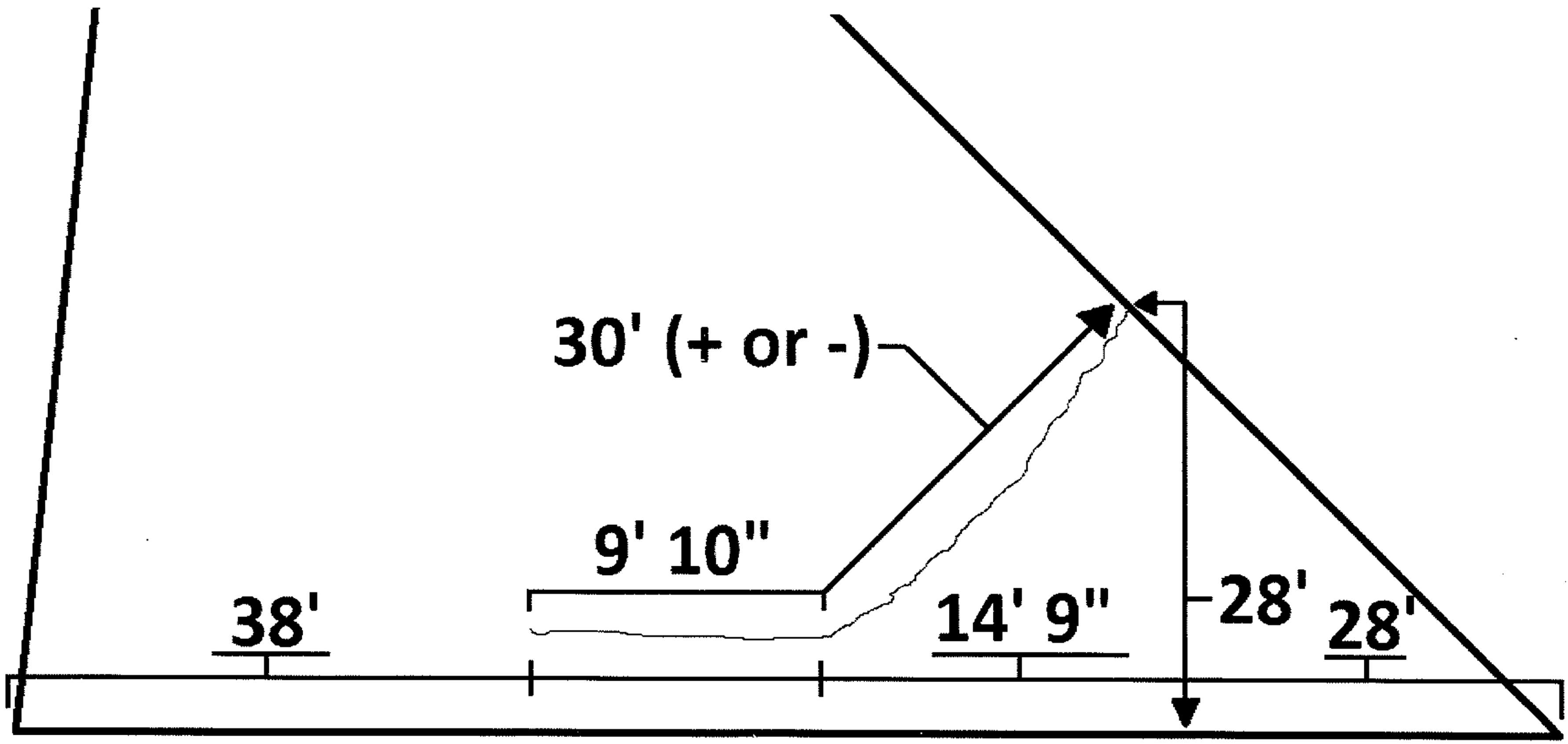

Dam Buttress No.5
Dam Bay No.4
Scale: 1"= 20'
Bridge Deck ELEV. 1003.5'
Handrail ELEV. 1007.3'
21'
18'
ELEV. 999.6'
994.5'
Dam Crest ELEV. 992.32'
NWSE 990.3'
(Dam Buttress)
Construction Joints (Typical)
3' (+ or -½) (Arch Thickness)
(Dam Buttress)
Bottom of Arch ELEV. 920.0'
Arch Buttress
Highly Weathered & Fractured Bedrock (Coarse-Grained Schist)
908.5' (+ or -)
Leakage & Under Seapage
Excess Hydrostatic Pressure
897.0' TOF
892.0' BOF
TOF 897.0'
BOF 892.0'
Highly Weathered & Fractured Bedrock (Biotite Gneiss)
Elevation, Feet
1020
1000
980
960
940
920
908.5'
900
897
892
880
0
20
40
60
80
100
120
Distance, Feet

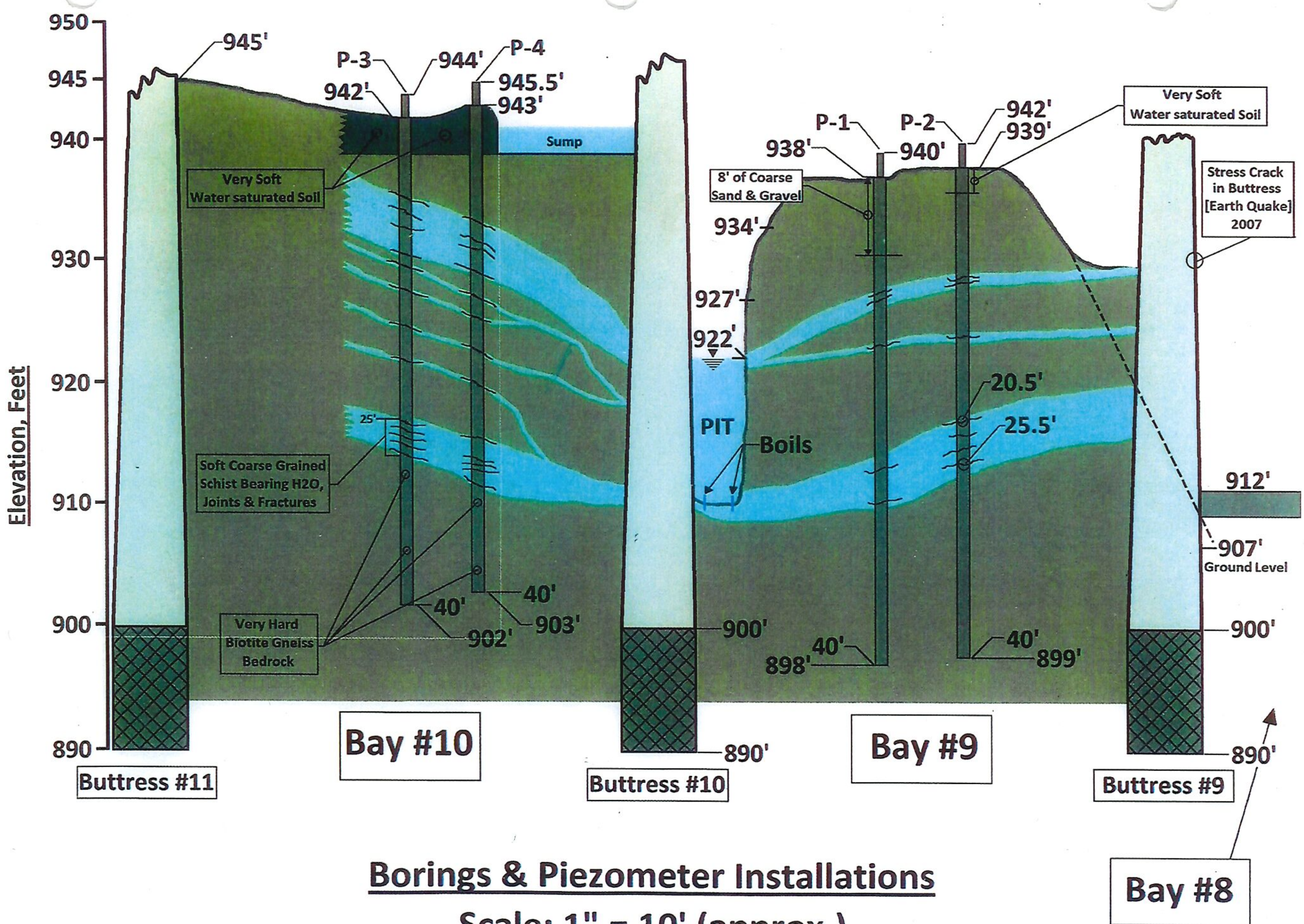


Powerhouse Section
Scale: 1"= 20'
Water Level
Vertical Arch
NWSE 992.5'
Buttresses
#7 and #8
Arch Section
Distance, Feet
Distance, Feet
Power House in Bay #7
2'-8"
Porch Railing
Main Floor (Gennerator Room)
Porch
Arch Section
Basement (Turbine Room)
NWSE 888' FLOW
14'
Bottom of Tailrace 880'
BOF 875'
1000
992.5'
990
980
970
960
950
940
930
920
914
910
900
890
880
870
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0

Buttress #9 in Bay #8

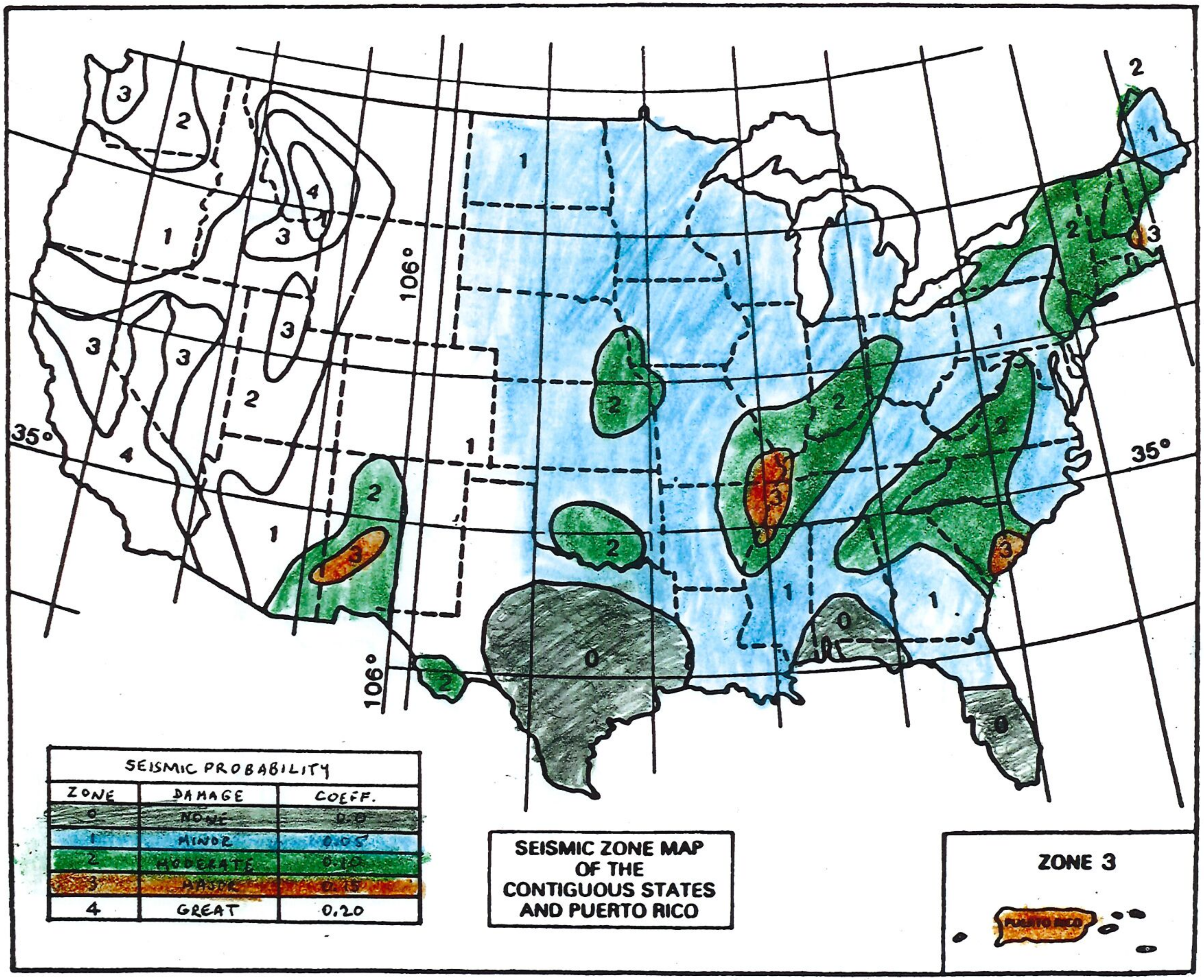
Image Showing a large Stress Fracture





Borings & Piezometer Installations

Scale: 1" = 10' (approx.)


SEISMIC ZONE MAP
OF THE
CONTIGUOUS STATES
AND PUERTO RICO
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
ZONE 3
PUERTO RICO
35°
106°

)E Chapter 4
ıg, Mixing, and Placing
· Precast Concrete Floor
tions of Mortars by Pneu-
;Report of ACI Committee
: Beams and Girders for
9, 1960)
y Committee 611, 4th Edi-
ɜd by Committee 317, 2nd
ɪr direction of Committee

PART III — CONSTRUCTION REQUIREMENTS

CHAPTER 5 — CONCRETE QUALITY

500—Notation

f_c' = compressive strength of concrete (see Section 301)
F_{sp} = ratio of splitting tensile strength to the square root of compressive strength

501—Concrete quality

(a) For the design of reinforced concrete structures, the value f_c' shall be used in determining stresses in Part IV-A and strengths in Part IV-B.

(b) All plans submitted for approval or used for any project shall clearly show the specified strength, f_c', of concrete at the specified age for which each part of the structure was designed.

(c) Concrete that is to be subject to freezing temperatures while wet shall have a water-cement ratio not exceeding 6 gal. per bag and it shall contain entrained air.*

(d) Concrete that will be exposed to sulfate-containing or other chemically aggressive solutions shall be proportioned in accordance with "Recommended Practice for Selecting Proportions for Concrete (ACI 613)" and "Recommended Practice for Selecting Proportions for Structural Lightweight Concrete (ACI 613A)."

502—Methods of determining the proportions of concrete

(a) The determination of the proportions of cement, aggregate, and water to attain the required strengths shall be made by one of the following methods, but lower water-cement ratios may be required for conformance with Sections 501(c) and (d).

Method 1 — Without preliminary tests

Where preliminary test data on the materials to be used in the con-

TABLE 502(a)—MAXIMUM PERMISSIBLE WATER-CEMENT RATIOS FOR CONCRETE (METHOD NO. 1)

Specified compressive strength at 28 days, psi f_c'	Maximum permissible water-cement ratio†			
	Non-air-entrained concrete		Air-entrained concrete	
	U.S. gal. per 94-lb bag of cement	Absolute ratio by weight	U.S. gal. per 94-lb bag of cement	Absolute ratio by weight
2500	7¼	0.642	6¼	0.554
3000	6½	0.576	5¼	0.465
3500	5¾	0.510	4½	0.399
4000	5	0.443	4	0.354

†Including free surface moisture on aggregates.

*Detailed recommendations for quality of concrete and requirements for air content for various exposures are given in "Recommended Practice for Selecting Proportions for Concrete" (ACI 613).