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PROGRAM CRITERIA SPECIFICATIONS DOCUMENT, COMPUTER PROGRAM TWDA--ETC(U)
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(6) PROGRAM CRITERIA SPECIFICATIONS DOCUMENT.

COMPUTER PROGRAM TWDA FOR DESIGN AND ANALYSIS OF INVERTED-T RETAINING WALLS AND FLOODWALLS.

by

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PREFACE

This document presents criteria prepared as the basis for developing TWDA, a computer program for design and analysis of inverted-T retaining walls and floodwalls. Development of the program is a joint effort of the Computer-Aided Structural Design (CASD) Project of the U. S. Army Engineer Division, Lower Mississippi Valley (LMVD), and of the Computer-Aided Structural Engineering (CASE) Project of the Office, Chief of Engineers, U. S. Army (OCE).

Mr. William A. Price, Chief, Computer-Aided Design Group (CADG), Automatic Data Processing (ADP) Center, U. S. Army Engineer Waterways Experiment Station (WES), provided the overall design of the program and led the program development team.

Engineering criteria for TWDA were provided by the members of LMVD's CASD Committee and of the CASE Task Group on T-Walls and by other Corps personnel:

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This document was compiled by Mr. Agostinelli, Mr. Price, and Dr. Pace. It was published for LMVD.

A basic user's guide, a user's reference manual, and a validation report will also be published on TWDA by WES.

CONTENTS

Page

PREFACE i

MARGINAL NOTES viii

1. PROGRAM PURPOSE AND ORGANIZATION 1-1

1.1 PURPOSE OF PROGRAM TWDA 1-1

1.1.1 Computer-Aided Design 1-1

1.1.2 Criteria Document Scope (Structural, not Soils) 1-1

1.2 ORGANIZATION 1-1

1.2.1 Program Structure 1-1

1.2.2 Data Entry Procedure 1-1

1.2.3 Data Review Procedure 1-2

1.2.4 Restart Capability 1-3

1.2.5 Volume of Printout 1-3

1.2.6 Calculation Modules 1-3

2. DATA 2-1

2.1 GENERAL 2-1

2.2 BASIC DATA 2-1

2.2.1 Criteria Data 2-1

2.2.2 Wall Basic Data for Design 2-1

2.2.3 Soils 2-1

2.2.4 Loads 2-1

2.3 LOAD CASE DATA 2-1

2.3.1 Possible Factors for Describing Any ONE Load Case 2-1

2.3.2 Typical Application of Load Cases 2-4

3. STABILITY 3-1

3.1 INTRODUCTION TO STABILITY CRITERIA 3-1

3.2 CONFIGURATIONS 3-1

3.2.1 Design Load Configuration References 3-1

3.2.2 Base Contact Configurations 3-1

3.3 GENERAL STABILITY CAPABILITIES 3-3

3.3.1 Wall Dimension Variations 3-3

Page

3.3.2 Quantities For Cost Comparison 3-3

3.3.3 General Criteria 3-6

4. APPLIED PRESSURES 4-1

4.1 GENERAL BASIS OF EARTH PRESSURES 4-1

4.2 EARTH PRESSURE APPLICATION LOCATION 4-1

4.3 METHOD OF COMPUTATION OF EARTH PRESSURES 4-1

4.3.1 Active Earth Pressure 4-1

4.3.2 Fully Passive Pressures 4-3

4.3.3 At Rest Pressures 4-3

4.3.4 Forces Along Soil-Stem Interface 4-3

4.4 RESISTING FORCE DISCUSSION FOR OVERTURNING AND SLIDING 4-4

4.4.1 Precaution 4-4

4.4.2 Distribution for Overturning Calculations 4-4

4.4.3 Limiting Value for Sliding 4-5

4.4.4 Reduction of Excessive Required Passive Pressure 4-5

4.4.5 An Alternate Resisting Strut Force 4-5

4.5 HYDROSTATIC PRESSURES 4-7

4.6 SURCHARGE PRESSURES 4-7

4.6.1 Vertical Pressures 4-7

4.6.2 Horizontal Pressures 4-8

4.7 EFFECT OF SEEPAGE PRESSURES ON EFFECTIVE EARTH UNIT WEIGHT 4-9

5. OVERTURNING STABILITY 5-1

5.1 GENERAL 5-1

5.1.1 Terminology 5-1

5.1.2 Stability Requirements 5-1

5.1.3 Horizontal Pressures 5-1

5.2 ANALYSIS PROCEDURE 5-1

5.2.1 The Moment Center 5-1

5.2.2 Force Summations 5-1

5.2.3 Moment Summation 5-1

5.2.4 Sample Calculations 5-1

		Page
5.3	DESIGN PROCEDURE	5-1
5.3.1	The Basic Program Procedure	5-1
5.3.2	Verification	5-2
5.3.3	Setting Desired Values	5-2
5.3.4	Bearing Pressure Reference	5-2
5.4	RESULTANT RATIO	5-2
5.4.1	Definition	5-2
5.4.2	Limiting Values	5-2
5.4.3	Relationship to Percent Effective Area	5-3
5.5	UPLIFT	5-3
6.	SLIDING	6-1
6.1	AVAILABLE METHODS OF ANALYSIS	6-1
6.1.1	Shear Friction Method	6-1
6.1.2	Allowable Strength Equilibrium Method (I)	6-1
6.1.3	Allowable Strength Equilibrium Method (II)	6-1
6.1.4	Modified Shear Friction Sliding Method	6-1
6.2	DISCUSSION OF ANALYSIS METHODS	6-1
6.2.1	Shear Friction Method (ETL 1110-2-184)	6-1
6.2.1.1	Resistance Force for Uphill Sliding	6-2
6.2.1.2	Resisting Force for Downhill Sliding	6-2
6.2.1.3	Safety Factor	6-3
6.2.1.4	Apparent Deficiencies in ETL 1110-2-184 Shear Friction Formulas	6-3
6.2.1.5	Passive Resistance of the Wedge Adjacent to the Wall	6-3
6.2.1.6	Values for the Minimum Shear Friction Safety Factor	6-4
6.2.2	Allowable Strength Equilibrium Method (I)	6-4
6.2.3	Allowable Strength Equilibrium Method (II)	6-6
6.3	DISCUSSIONS COMMON TO ALL SLIDING METHODS	6-6
6.3.1	Assumed Failure Surfaces	6-6
6.3.2	Effect Due to Some of the Base Not Being in Compression With the Foundation	6-7
6.3.2.1	Portion of Base in Compression With Foundation (Percent Effective Base)	6-7

		Page
6.3.2.2	Uplift	6-8
6.3.2.3	Coefficient of Friction ϕ and Cohesion c to be Used in the Analysis	6-9
7.	BEARING PRESSURES	7-1
7.1	CALCULATION OF BEARING PRESSURES	7-1
7.1.1	The Procedure	7-1
7.1.2	Entire Base in Compression	7-1
7.1.3	Part of Base in Compression	7-1
7.2	MAXIMUM ALLOWABLE BEARING PRESSURES	7-3
7.2.1	Selection of Maximum Allowable Pressure	7-3
7.2.2	Data Structure	7-3
7.2.3	Method	7-3
8.	EARTHQUAKE ANALYSIS	8-1
8.1	INTRODUCTION	8-1
8.1.1	Seismic Coefficient	8-1
8.1.2	T-Wall (ER 1110-2-1806)	8-1
8.2	SELECTION OF SEISMIC COEFFICIENT	8-1
8.3	LOCAL SOIL CONDITIONS	8-2
8.4	INERTIA FORCE OF WALL	8-2
8.5	LATERAL COHESIONLESS EARTH PRESSURES DUE TO EARTHQUAKES	8-3
8.5.1	Active Earth Pressure Conditions	8-3
8.5.2	Passive Earth Pressure Conditions	8-5
8.5.3	At-Rest Earth Pressure Conditions	8-7
8.6	LATERAL COHESIVE EARTH PRESSURES DUE TO EARTHQUAKES	8-8
8.7	WATER PRESSURE DUE TO EARTHQUAKES	8-8
8.7.1	Method	8-8
8.7.2	Westergaard Theory	8-8
9.	STRUCTURAL DESIGN CRITERIA	9-1
9.1	GENERAL	9-1
9.1.1	Critical Sections and Alternate Pressure Diagrams	9-1
9.1.2	The Method of Analysis	9-1
9.1.2.1	Working Stress Design	9-1

		Page
	9.1.2.2 Ultimate Stress Design	9-1
	9.1.3 Pressures for Structural Design	9-2
9.2	REINFORCING STEEL DETAILS	9-2
	9.2.1 The Primary Reference	9-2
	9.2.2 Default Values for Concrete Clear Cover	9-2
	9.2.3 Compressive Reinforcement	9-2
	9.2.4 Placement of Reinforcing Steel	9-2
9.3	STEM LOADINGS	9-3
	9.3.1 General	9-3
	9.3.2 Alternate Loading	9-3
EXHIBIT A:	COMPARISON OF COULOMB AND INCREMENTAL WEDGE EARTH PRESSURE VALUES	A-1
	Procedures Followed	A-1
	Discussion of Comparisons	A-2
	Addendum A: Procedure for Obtaining Coulomb Active Earth Pressure at a Given Depth	A-4
	Addendum B: Lateral Earth Pressure on Walls Using Theory of Elasticity	A-6
	Addendum C: Active Force for One Wedge (Generic)	A-7
	Addendum D: Pressure Comparisons	A-9
	Addendum E: Calculation Worksheets for Addendum D	A-15
	Addendum F: Errors Inherent in Trial Wedge Method	A-63
EXHIBIT B:	DERIVATION OF EQUATIONS FOR SLIDING	B-1
EXHIBIT C:	EQUATIONS FOR EARTH PRESSURE UNDER A TRAPEZOIDAL BASE	C-1
EXHIBIT D:	COMPUTATION OF AT REST EARTH PRESSURE	D-1
EXHIBIT E:	FORMULAS FOR WORKING STRESS DESIGN OF CANTILEVER SLABS WITH AXIAL FORCE (ACCORDING TO APPENDIX B OF ACI 318-77)	E-1
EXHIBIT F:	FORMULAS FOR ULTIMATE STRENGTH DESIGN OF NONHYDRAULIC STRUCTURES WITH CANTILEVER SLABS WITH AXIAL FORCE (ACCORDING TO ACI STANDARD NO. 318-77)	F-1
EXHIBIT G:	COMPARISON OF BOUSSINESQ AND BOWLES SOLUTIONS FOR PRESSURES ON RETAINING WALLS DUE TO SURCHARGE LOADS	G-1
EXHIBIT H:	SAMPLE RETAINING WALL SHEAR FRICTION SLIDING ANALYSIS	H-1
EXHIBIT I:	SAMPLE FLOODWALL SLIDING ANALYSIS	I-1

		Page
EXHIBIT J:	SAMPLE FLOODWALL SLIDING ANALYSIS	J-1
EXHIBIT K:	SAMPLE OVERTURNING ANALYSIS FOR FLOODWALLS AND RETAINING WALLS	K-1
EXHIBIT L:	PASSIVE RESISTANCE FOR T-WALLS--EQUIVALENT RELATION FOR COULOMB THEORY AND ETL 1110-2-184	L-1

MARGINAL NOTES

Special marginal notes, in the form of a letter or group of letters enclosed in brackets, are used to identify the principal sources of the key items of criteria. These symbols include:

[F] EM 1110-2-2501, "Flood Walls," Jan 1948, with Change 3, 18 Jun 1962.

[FD] Draft manual for floodwalls (EC 1110-2-156, 17 Jun 1975).

[R] EM 1110-2-2502, "Retaining Walls," 29 May 1961, with Change 3, 25 Jan 1965.

[D] EM 1110-2-2200, "Gravity Dam Design," 25 Sep 1958.

[EL] ETL 1110-2-22, "Lock Gravity Walls," 19 Apr 1967.

[ED] ETL 1110-2-184, "Gravity Dam Design--Stability."

[E] ER 1110-2-1806, "Earthquake Design . . . Dams," 30 Apr 1977.

[ACI] ACI 318-71, "ACI Building Code . . . Concrete," with 1976 Supplement.

[WS] EM 1110-2-2101, "Working Stresses for Structural Design," 1 Nov 1963.

[HSR] EM 1110-2-2103, "Reinforcement . . . for Hydraulic Structures," 21 May 1971.

[ChS] OCE specifications (guidance from DAEN-CWE-DS personnel).

1. PROGRAM PURPOSE AND ORGANIZATION

1.1 PURPOSE OF PROGRAM TWDA

1.1.1 Program TWDA is to be a computer-aided structural design system for analysis and/or design of inverted-T cantilever walls founded on earth or rock. Multiple load cases will allow the wall to act as a floodwall or a retaining wall.

1.1.2 This program criteria specifications document is intended for use by structural engineers. The computer program that these criteria are for does not attempt to establish any soils design criteria; such data must be entered by the user after consultation with soils design engineers. There are no default values for soils criteria parameters, except as provided in the engineering manuals for structural design.

1.2 ORGANIZATION

1.2.1 Structure - The program will be a series of design or analysis modules,* each performing one specific step in the design or analysis process. These modules will be callable, in any logical sequence, from an executive command phase.** While in the executive phase, the user may call various procedures for data entry, data review, saving the current design status, restoring from an old status save, etc. This is illustrated in Figure 1-1.

1.2.2 Data Entry - The data entry procedure will be similar to that for program TGDA,+ except that the data phase may be incorporated into the command phase instead of being separate as in TGDA. Features will include:

a. Data are entered by naming the group and listing the values in that group, all on one line.

b. Default values may be requested by entering the letter "D" instead of a numerical value.

---

* A module is a subprogram that is controlled as one unit and that performs one complete aspect of the purpose of the entire program.

** The executive phase of this program is the central core of the user's flow of control. The user may enter data or start a module while in the executive phase.

\+ TGDA (three-girder tainter gate design/analysis) is a computer program (713-F3-RO-022) developed for LMVD's CASD Committee in 1976.

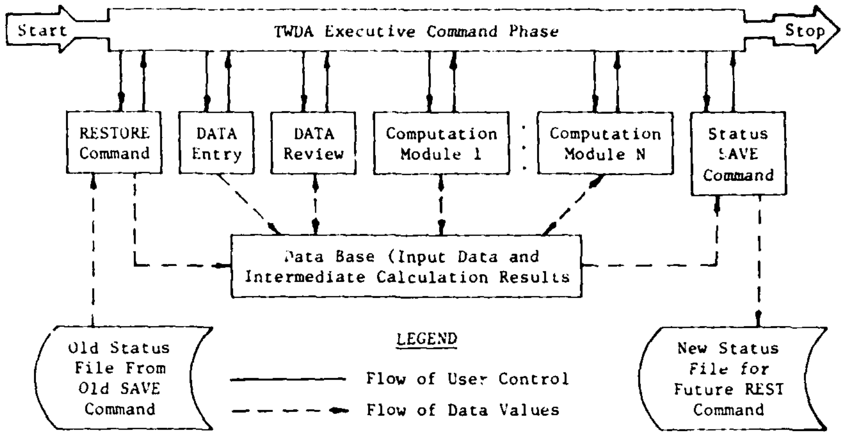


Figure 1-1. BASIC PROGRAM FLOWCHART

c. Values to be calculated will be identified to the program by typing the letter "C" instead of entering a value.

d. A value that is to be left unchanged from its previous state will be identified to the program by typing the letter "S."

e. The program will look for illogical and inconsistent data and will identify such items to the user for correction or use anyway.

f. The current status of items of input data or of all data values can be reviewed.

g. Multiple-level prompting is provided as in program TGDA, except that the minimum level will be less wordy than in program TGDA, for the more experienced users.

Thus, the program will accept several sets of input data, where the following sets contain only the changes to the data comprising the preceding sets. Repetitive data will remain unchanged.

1.2.3 Data Review - Data review will be available in two ways:

a. Input Data Review will be done as in the data input phase in program TGDA with the LOOK command.

b. Default Value Review will be done in a separate module. Unless reviewed with this option, default values will be set automatically by the user's selection of:

(1) Floodwall or retaining wall criteria.

(2) Hydraulic or nonhydraulic structure criteria.

Making the review of default values optional is expected to enable the experienced user to simplify and expedite his preliminary designs. In any case, the values will be printed out in the report file. The combination of a nonhydraulic floodwall, being illogical, will be rejected. Default values will always be taken from OCE publications; nonstandard values set by the user will be so labeled in the report file and verified interactively.

1.2.4 Restart Capability

a. In addition to the user-controlled SAVE files, the program will use an automatic UPDATE file that is reset after the completion of a command or a calculation module.

b. The RESTart command will restart the program from an old update or saved file.

1.2.5 Volume Of Printout - Printout will be of two types:

a. The printout to the user's time-sharing terminal will be restricted to the minimum needed for the user to make his decisions.

b. A full report of calculations made will be written to a report file that can be listed at a time-sharing terminal and/or sent to the high speed printer in the user's District office ADP Center.

1.2.6 Calculation Modules - A list of the major calculation modules includes:

a. SA - Stability analysis Active pressures for overturning and sliding, calculated along a vertical plane at end of heel.

(1) Coulomb's equations plus surcharge pressure equations assuming elastic soil.

(2) Incremental wedge methods (see paragraph 4.3.1b).

(3) As imputted.

b. FA - Foundation stability Analysis of completely defined wall (overturning, sliding, and bearing); uses module SA as needed.

c. FD - Foundation stability Design; uses modules SA and FA as needed.

d. SP - Stem Pressures for structural analysis. Same basis as module SA, except that the pressures are calculated at the stem face instead of at the end of the heel. This is for structural analysis of the stem. Heel, toe and key slabs will use pressures based on the stability analysis from modules FA or FD, as described in paragraph 9.1.1.

e. WA - Working stress structural Analysis.

f. WD - Working stress structural Design.

g. UA - Ultimate strength structural Analysis.

h. UD - Ultimate strength structural Design.

2. DATA

2.1 GENERAL

Data will be of two types, basic data and load case data. Basic data will be used as common to all load cases unless overridden by data for a particular load case. Load case data will consist of values applicable to only the one load case. Basic data will also include unchanging data such as wall dimensions.

2.2 BASIC DATA

2.2.1 Criteria Selection

(1) Floodwall or retaining wall?

(2) Hydraulic or nonhydraulic structure?

2.2.2 Wall (basic data for design, described in paragraph 3.3; may be set by groups for different wall types). Major items are listed below:

(1) Top of stem elevation and minimum thickness.

(2) Toe-side batter of stem.

(3) Heel-side top panel height and batter of stem.

(4) Heel-side bottom panel batter of stem.

(5) Minimum base slab thickness.

(6) Bottom of toe elevation or range of values.*

(7) Toe width or stem ratio.

(8) Base width, range of values.*

(9) Base slope, range of values.*

(10) Key depth, maximum value.*

(11) Key batter, toe-side face.

(12) Key location indicator (0 if at heel, 1 if at stem).

2.2.3 Soils data as illustrated in Figure 3-1 and described in Section 4.

2.2.4 Loads common to all load cases (except ones for which value(s) are reset in load case data), as described in paragraph 2.3.

2.3 LOAD CASE DATA (for each individual load case)

2.3.1 Possible Factors for Describing Any ONE Load Case (in addition to or in place of basic data):

---

* Varies between limits set by user.

a. Water

(1) Water elevations over heel and over toe, unit weight of water (default = 62.5). See paragraph 3.2.1e for illustration of elevation.

(2) Seepage pressure according to descriptions in paragraph 3.3.3f.

b. Earth

(1) Earth geometry over heel, at stem, and over toe if different from basic data. Also soils properties data if different from soils basic data input.

(2) Earth pressures on wall (a) calculated from the earth elevations and K-value Coulomb theory, (b) calculated from the earth elevations and incremental wedge theory, or (c) as inputted separately. See paragraph 3.3.3h for more detail.

c. Horizontal Loads

(1) Trapezoidal (linearly varying distributed) loads, horizontal on stem (W_1 and W_3 through W_4 in Figure 2-1).

(2) Concentrated horizontal forces and their elevations (PH_1 and PH_2 in Figure 2-1).

d. Surcharges over heel and over toe, values and locations

(1) Distributed, over all or any part of cross section (W_H and W_T in Figure 2-1).

(2) Up to five vertical concentrated line loads parallel to wall, (P_{v1} through P_{v5} in Figure 2-1), plus force P_{v5} centered on the top of the stem and P_{vB} anywhere on the base.

e. Wind direction and magnitude, psf

f. Earthquake effect toward heel and toward toe (really two subcases)

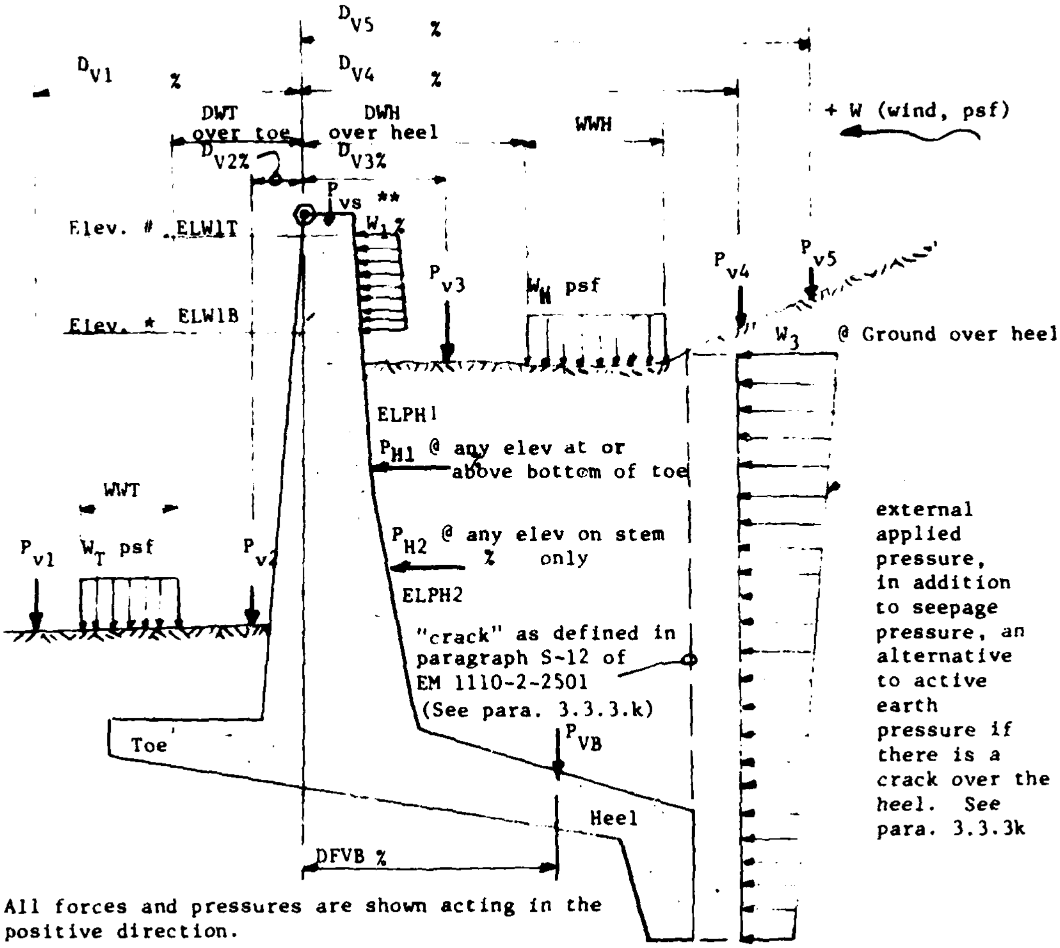
g. Design criteria

(1) Load factors for R/C Ultimate Strength Design or overstress factor for Working Stress Design.

(2) Allowable bearing capacity, range of values over ranges of allowable toe base elevations and base widths (see paragraph 7.2.2).

(3) Minimum creep ratio (see paragraph 3.3.3c for guidance).

(4) Minimum factor of safety against shear friction sliding (see paragraph 6.1a).



All forces and pressures are shown acting in the positive direction.

** Centered on top of stem

\# At or below top of stem

* At or above Ground over heel.

% Any value, on either side of the stem (either direction), + if over heel, - if over toe.

Note that forces P_{v1} through P_{v5} are applied at finished grade elevations while P_{vs} and P_{vb} are applied directly to the concrete.

Figure 2-1. ILLUSTRATION OF APPLIED LOADS

(5) Minimum safety factor for cohesion and (tan ϕ) data values used in sliding determination by allowable strength equilibrium methods (see paragraphs 6.1.2 and 6.1.3).

(6) Limiting value of overturning stability resultant ratio (paragraph 5.4.2).

(7) Reinforced concrete design parameters (see Exhibits E and F for items).

(8) Specification of "hydraulic" or "nonhydraulic" structure.

(9) Heel earth cover crack control (see paragraph 3.3.3k).

h. Loading Classification

(1) Long-term operation.

(2) Short-term operation.

(3) Normal operation plus earthquake.

2.3.2 Typical Application of Load Cases - Any load case may have any or all of the effects described in paragraph 2.3.1.

3. STABILITY

3.1 INTRODUCTION TO STABILITY CRITERIA

3.1.1 The criteria for stability are the most uncertain and among the most important data used in design of an inverted-T wall. A complete stability analysis includes the effects of overturning,* sliding,** base pressures,† and settlement. Settlement computations are beyond the scope of this program.

3.1.2 This computer program will lead the engineer/user through either design or analysis of an inverted-T wall with multiple load cases and a multilayered soil system with assorted surcharges. It will show the engineer a set of recommendations for design or analysis parameters and then use the user's decisions to complete the detailed analyses.

3.1.3 This computer program will design or analyze for shallow-seated stability limited to forces on the wall acting at or above the interface between the bottom of the wall base and the soil. The term "wall base" is used here to include any key. Deep-seated stability which includes the earth beneath the lowest point of the base or key is beyond the scope of the program.

3.2 CONFIGURATIONS

3.2.1 Design Load Configuration References

a. Paragraphs 2.3.1c and 2.3.1d describe surcharge types.

b. Figure 3-2 shows details of wall dimensioning.

c. Figure 3-4 shows location of edges of structural excavation.

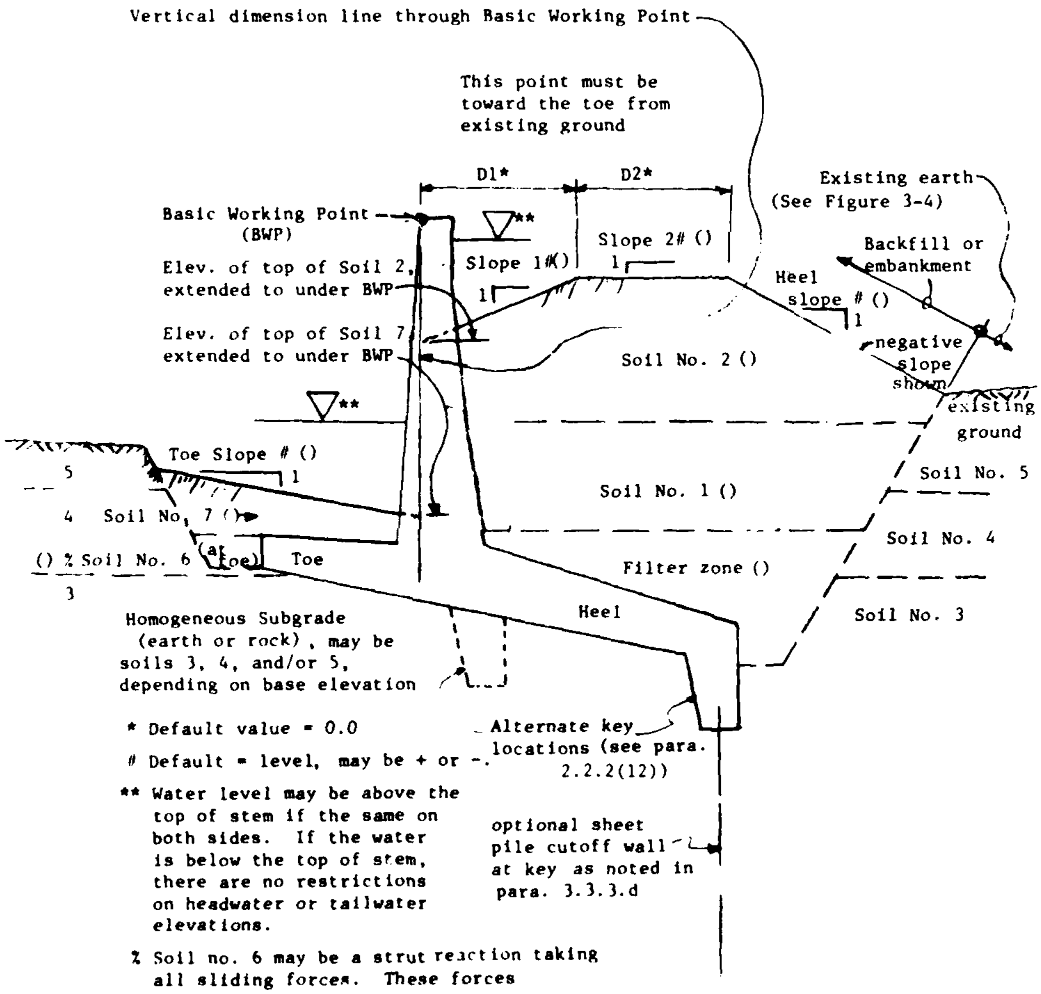
d. Paragraph 3.3.3d describes the effects of sheet pile cutoff walls.

e. Figure 3-1 illustrates the soils environment data.

3.2.2 Base Contact Configurations - This computer program works with a unit slice of wall, measured along the vertical plane through the basic working point shown in Figure 3-2. This is taken as being the toe side of the top of the stem.

---

* Discussed in Section 5.
** Discussed in Section 6.
† Discussed in Section 7.



* Default value = 0.0

\# Default = level, may be + or -.

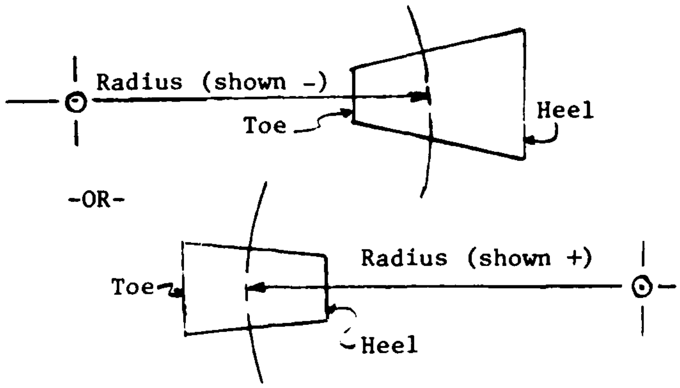
** Water level may be above the top of stem if the same on both sides. If the water is below the top of stem, there are no restrictions on headwater or tailwater elevations.

% Soil no. 6 may be a strut reaction taking all sliding forces. These forces will be printed out.

() Different load cases may include different values for backfill soils

Figure 3-1. FINISHED GRADE SOILS CONFIGURATION

a. General Case--Trapezoidal Base. In the general case of a curved wall segment, the base contact soil-structure interface is curved. The base plan used for calculations in this program uses straight-line approximations for the curved outer edges of the unit slice:



(See paragraphs 7.1.2b and 7.1.3b for formulas for base pressures.)

b. Usual Case--Straight-Line Wall Segment. The usual case of a straight-line wall utilizes the familiar expressions for pressures under a rectangular base contact area. See paragraphs 7.1.2a and 7.1.3a for formulas for these base pressures.

3.3 GENERAL STABILITY CAPABILITIES

3.3.1 Wall Dimension Variations - The heel side is taken as being the side with the greater driving force on the wall. Variable values are set to the default value, unless defined by the user or controlled by the various design routines. See Figure 3-1 for soils system description.

3.3.2 Quantities For Cost Comparison - Cost comparisons are used for identifying optimum designs and for design memorandum quantity takeoffs. Items considered include:

a. Structural Excavation below existing grade is shown in Figure 3-4. It will be calculated separately for each existing soil layer so that different unit prices can be used in each layer.

b. Structural Backfill, to either

(1) Existing grade or

(2) Finished grade if below existing grade. (Concrete volume not included.)

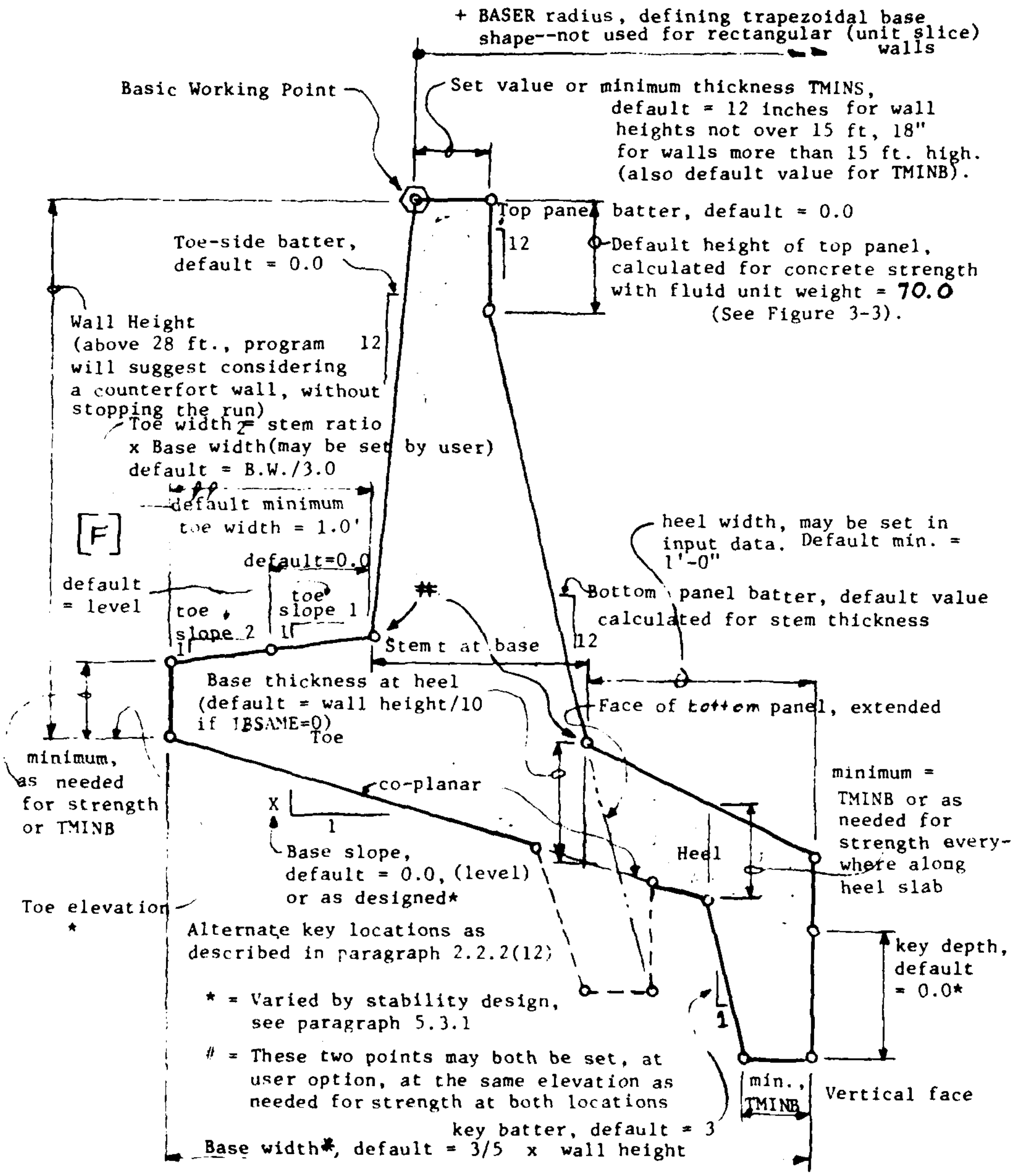
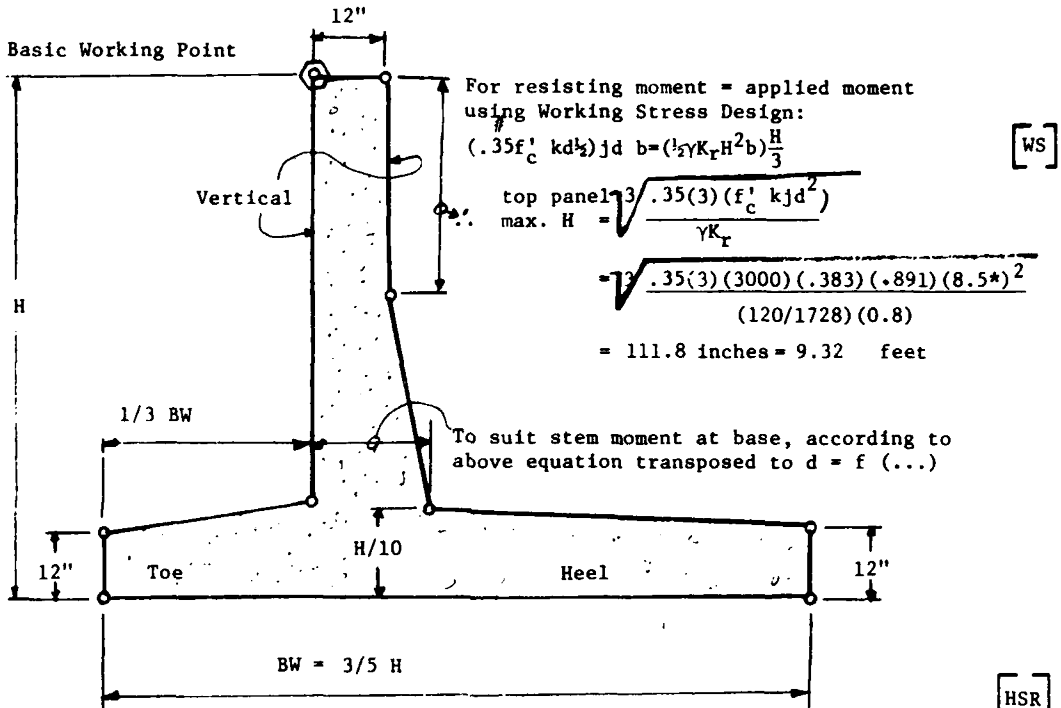


Figure 3-2. WALL CROSS SECTION SHOWING MAJOR VARIABLES

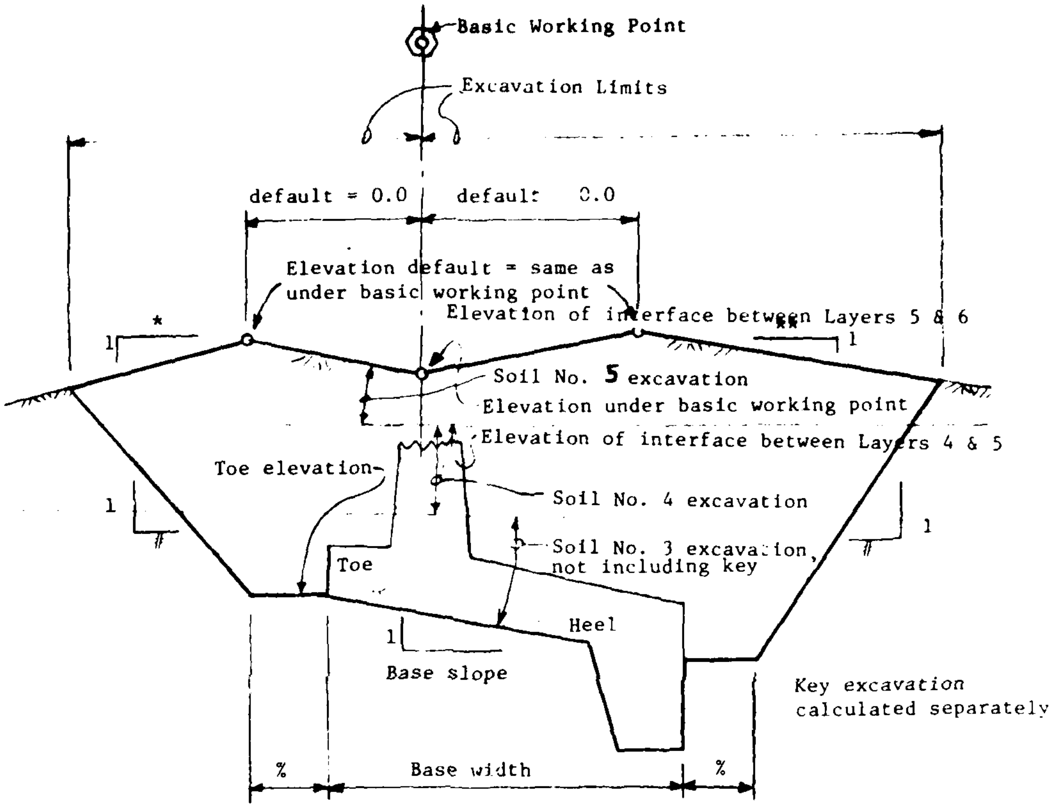


* Default value for clear cover over reinforcing = 3 inches (EM 1110-2-2103)

\# Default value of 0.35 is for hydraulic structures designed with WSD. Non-hydraulic structures designed by WSD would have a default value of 0.45. [WS]

Figure 3-3. WALL CROSS SECTION WITH ILLUSTRATION OF ALL DEFAULT VALUES FOR HYDRAULIC STRUCTURES

Default values are taken from EM 1110-2-2501, unless otherwise noted. [F]



* slope to toe
** slope to heel } default = level (shown -)

\# excavation side slope, default = 1:1

% excavation extra width, default = 2.0' each side

Figure 3-4. STRUCTURAL EXCAVATION

c. Volume of Concrete (stem, base, and key calculated and priced separately).

d. Embankment Above Existing Grade.

3.3.3 General Criteria

a. Incrementing Dimensions of Wall - Use a 3-inch increment for base width, toe base embedment, and key length. Use a 1/2-inch increment on concrete thicknesses. Use a 0.1-foot-horizontal to 1.0-foot-vertical increment on base slope. See also paragraph 5.3.1.

b. Limitations on Input - The computer program will allow complete flexibility for input. The specific values shown in this discussion are suggested values; therefore, the designer will be able to override them with his own values (the default limits are for design control as in paragraph 5.3.1):

(1) Minimum earth cover over top of heel: The default minimum value is (3 feet + exposed stem height/10), but not less than 5.0 feet, from paragraph S-12 of EM 1110-2-2501. [F]

(2) Elevation of bottom of base at end of tow: Controlled within limits established by the user. The usual highest elevation is to produce no cover over the toe. The usual minimum elevation is to produce an earth cover over the toe equal to one half the wall height. Note that as the toe elevation changes, so does the heel; and this may affect the heel cover discussed in paragraph 3.3.3b(1).

(3) Base slope: Controlled within limits established by the user. Usual limits are level and a 1:3 slope.

(4) Key length: Controlled within limits established by the user. Usual limits are no key and 0.8 times the stem height. [F]

(5) The stem ratio is set by the user. The default value is 0.33, from paragraph 1-09c of EM 1110-2-2501, except 0.25 when water is within 1.05 feet of the top of the stem. [F]

c. Hydraulic Gradient (1/creep ratio) at the toe, based on earth under the base. Minimum permissible creep ratios for boil control may be taken from page 5-5 of the draft manual for floodwalls (Loading No. 1 values are also in EM 1110-2-2501):

Type of Foundation Soil	Floodwall Loading No. 1 for Water at Top of Stem	Floodwall Loading No. 2 for Water 3 Feet Below Top of Stem
Granular (sand or gravel)	4.0	4.0 [FD]
Uniform sands and silts	3.0	3.0
Well-graded sandy silts	2.0 [F]	2.0
Lean, sandy and silty clays	1.8	2.0

d. Sheet Pile Cutoff Walls - Neglect bearing value and sliding resistance of cutoff walls. For boil control purposes only, comparing the creep ratio developed against the allowable minimum value for paragraph 3.3.3c, the program will use two items of input data:

(1) Effective length of sheet pile below the bottom of key, and

(2) A control parameter to select the location of the creep path portion between the bottom of the effective length of sheet pile and the end of the toe:

(a) One path along the toe-side face of the sheet pile, key, and the bottom of the base-soil interface.

(b) Or the other path being a single, straight line from the bottom of the effective length of sheet pile to the end of the toe.

The two creep paths are to provide minimum and maximum creep ratios.

e. Drains and Filters - These will be considered effective in shortening the seepage path, which may serve to increase the seepage uplift. In accordance with Engineering Manual recommendations, weep holes will not be considered; but the user may include them if he wants to, by coding his water elevations and/or pressures accordingly.

[R p. 6]
[F p. 15]

f. Hydrostatic Pressures - The user has three ways to control the calculation of line-of-creep hydrostatic pressures to be used for design. For analysis, the user may input his own set of pressures for any or all load cases, in which case these input pressures will be substituted for the pressures calculated for that load case(s). Note that these options do not apply to boil control as discussed in paragraph 3.3.3d:

(1) The first control has two options over how multiple load cases are handled:

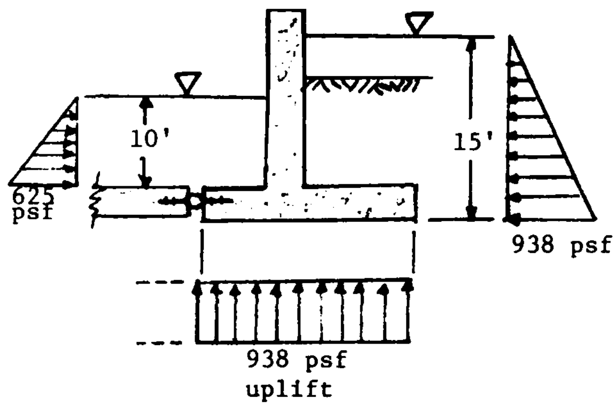
(a) Option 1: Each load case uses its own pressures according to the controls described in paragraphs 3.3.3f(2) and 3.3.3k. This is the default option.

(b) Option 2: All load cases use the pressures determined for the first load case listed in input data list.

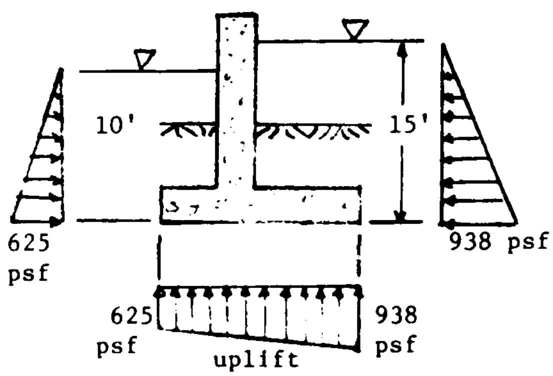
(2) The second control has four options that may be used for design or analysis:

(a) Option 1: The line of creep calculations are as described in EM 1110-2-2501 and as illustrated and discussed in detail in Exhibit H for sliding and Exhibit K for overturning. This is the default option for this control. Its action combines with the heel earth crack control (paragraph 3.3.3k) to determine how the pressures are determined.

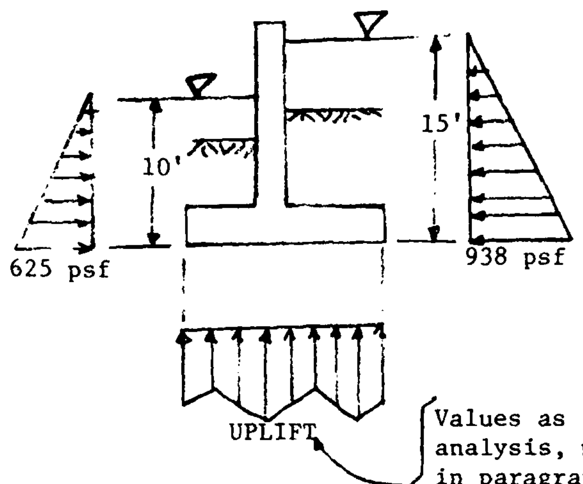
(b) Option 2 (perched water table): Any load case(s) will use the water elevation over the toe for weight and horizontal pressure above the toe only. Uplift will be hydrostatic, based on the water elevation over the heel. This would be selected by the user for a channel with an impervious floor:



(c) Option 3: Pressures will be those caused by the weight of water over the heel and toe. Uplift will be a linear variation between the heel and toe hydrostatic pressures. The user might select this option for a wall on rock: [D]



(d) Option 4: Water weight and horizontal pressures above the base will be hydrostatic pressures calculated from the input water elevations. Uplift pressures will be input data for analysis only; will be used as zero for design:



Values as inputted by user for analysis, may be zero as described in paragraph S-15e of EM 1110-2-2501. Will be taken as zero during design [F]

(3) The third control is the heel earth cover crack set of options described in paragraph 3.3.3k. It is not applicable to options (b), (c), and (d) under paragraph 3.3.3f(2).

g. Stability Design for Economy - Working within user-defined ranges of values for toe embedment, base slope, and key length, the program will determine the combination producing the least cost (calculated as described in paragraph 3.3.2). The least-cost combination will be offered to the user for acceptance or modification before the final analysis results are written to the report file and/or written to the user's time-shari[illegible]erminal.

h. Type of Analysis - Both Coulomb and incremental trial wedge analysis values of active earth pressures on a vertical plane along the end of the heel will be presented to the user. The user will then select the method to be used (or input his own values) for stability analysis.

Default is the Coulomb method. The pressures determined from the final stability analysis will be the ones used for structural design. The methods of calculation are presented later in this document:

(1) Coulomb: See paragraph 4.3.

(2) Incremental Wedge: See paragraph II of Exhibit A.

i. The program will assume, unless otherwise directed by the user, that the wall is a hydraulic structure within the meaning of EM 1110-1-2101. [WS]

j. Maximum Movement of floodwalls will be estimated according to the procedure described in paragraph 5-5 on page 5-7 of the draft manual for floodwalls. The values of C_1 and C_2 will be input data to be selected by the user. This estimate is to be optional. Default action is to omit it. [FD]

k. Cracks in Earth Cover Over Heel - The presence or absence of vertical cracks in the earth cover over the heel, as discussed on page S-9 of EM 1110-2-2501, will be controlled by the user, with the following options that will be effective for both overturning and sliding stability analyses: [F]

(1) Option 1: A crack, regardless of the depth of cover, the default action in accordance with paragraph S-15a on page S-18 of EM 1110-2-2501. This will preclude the application of active earth pressure at the heel, with the line of creep starting at the bottom of the crack. [F]

(2) Option 2: No crack, regardless of the depth of cover. This will cause the line of creep to start at the ground surface. This is the option used in the calculations in Exhibits, H, I, and J and is the default option for retaining walls.

4. APPLIED PRESSURES

4.1 GENERAL BASIS OF EARTH PRESSURES

This section includes the determination of active and passive earth pressures which are obtained from limiting conditions of equilibrium (failure criteria). Methods for approximating at rest earth pressures are also included.

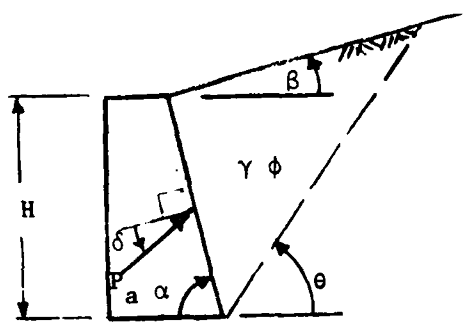
4.2 EARTH PRESSURE APPLICATION LOCATION

Earth pressures will be calculated along vertical planes at the ends of the base for stability determination and base slab design. Earth pressures for stem design will be calculated at the face of the stem.

4.3 METHOD OF COMPUTATION OF EARTH PRESSURES

4.3.1 Active Earth Pressure can be obtained by the Coulomb equation or by incremental trial wedge analysis, as discussed in paragraph 3.3.3h. Exhibit A describes the methods to be used and shows a comparison of the results from the two methods for nine load cases representative of a wide range of practical problems. The user can select either method or may input his own pressures. The default method is by Coulomb's equations.

a. Coulomb's equation for the fully active case in a one-layer soil is shown below. Exhibit A shows the extension to a multiple-layer system:



(Angles are shown +)

$$P_a = \left(\frac{\gamma H^2}{2}\right) K_a$$

where

$$K_a = \frac{\sin^2(\alpha + \phi)}{\sin^2 \alpha \sin(\alpha - \delta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \beta)}{\sin(\alpha - \delta)\sin(\alpha + \beta)}}\right]^2}$$

= derived from equation 3 on page 2 of EM 1110-2-2502, with β substituted for i and $(90 - \alpha)$ substituted for β.

When $\beta > \phi$, the slope is unstable and so equivalent values of K_a are unrealistic. Data that include a backfill slope angle β larger than ϕ will be rejected and the user will have to input his own value for K_a or use the incremental trial wedge option.

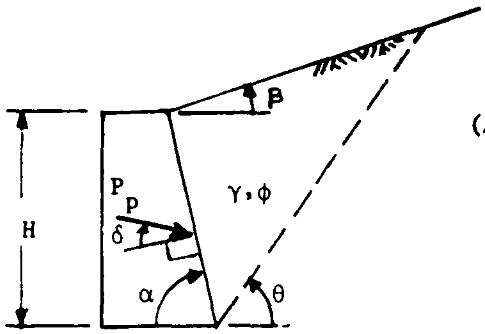
b. The Incremental Trial Wedge Method shown in Exhibit A must be applied with care to a multiple-layer soil system. When the trial wedge method is used in a multiple-layer soil system, more accurate (and more costly) results are obtained from using a failure surface with different slopes in each different type of soil. The weighted average single-plane slope used in Exhibit A costs significantly less money to execute but yields forces as much as 15 percent lower than the more accurate values. The 15 percent value is for an extreme case where the upper half of the backfill is sand ($\phi = 45°$, $\gamma = 120$ pcf) and the lower half is clay ($\phi = 0°$, $c = 300$ psf, $\gamma = 120$ pcf). For more usual sand ϕ values near 35°, the error is about 10 percent. The program will have the dual capacity of single-plane or multiple-plane analysis. Single-plane analysis will be used for design; both procedures will be available for analysis. The multiple-plane analysis procedure used in the above comparison is shown in Addendum F to Exhibit A.

c. Arching Active pressures are accounted for in accordance with paragraph 3g of EM 1110-2-2502. The program will provide for a "Correction Factor for Moment Arm" (CFMA) that will be used as an active-pressure moment multiplying factor: [R]

$$M = \text{active force} \times \text{moment arm} \times \text{CFMA}$$

This will act to increase moment but not horizontal force in both stability and stress analyses.

4.3.2 Fully Passive Pressures can be calculated from Coulomb's equation for homogeneous soil:



$$P_p = \left(\frac{\gamma H^2}{2}\right) K_p$$

where

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

= derived from equation 4 on page 2 of EM 1110-2-2502, with β substituted for i and (90 - α) substituted for β .

4.3.3 At Rest Pressures can be approximated by several methods. The at rest state is not a limiting condition and is not subject to a general analytical formulation. Established sources for at rest pressures include, but are not limited to, the finite element method, Jaky's equation, and as described in paragraph 3d on page 3 of EM 1110-2-2502. These methods are described briefly in Exhibit D. The program will not calculate at rest pressure coefficients as such, but the user may input his own horizontal earth pressure coefficients for the soil layers over the heel, to be used instead of Coulomb's active earth pressure coefficients. It will be assumed, unless overridden, that the distribution of at rest pressures is the same as for active pressures. [R]

4.3.4 Force Along Soil-Stem Interface is accounted for by the angle δ in the sketches in paragraphs 4.3.1a and 4.3.2. The default value for δ , to be used if an actual value has not been determine by the user, is zero. The value of δ may be set to any other value for any load case. [R]

4.4 RESISTING FORCE DISCUSSION FOR OVERTURNING AND SLIDING

4.4.1 Precaution - Passive pressure should never be used unless construction and maintenance of the backfill over the toe assure its effectiveness. The program will leave this decision to the user, providing for separate soil layers beyond and over the toe. See soil layers 6 and 7 in Figure 3-1.

4.4.2 Distribution for Overturning Calculations (see Exhibit K for a more detailed discussion):

a. Passive pressure distribution options for overturning analysis of walls with keys, on soil or rock foundations, are shown in part 1 of Figure 4-1. Option "a" in the figure is the default preferred distribution for floodwalls (EM 1110-2-2501, Plate 5). Option "c" in the figure is the default preferred distribution for retaining walls. The pressure values have no arbitrary upper limit since separate calculations for sliding account for passive pressure limiting values. If option "e" (strut) is selected, then all of the horizontal equilibrium force will be taken by the strut. [F]

b. Walls without keys (see part 2 of Figure 4-2), founded on either soil or rock, will be assumed to use a combined horizontal equilibrant force to resist the ΣH forces, broken into two parts:

(1) A concentrated force along the base of the wall, assumed to be mobilized first but limited to a maximum value of

$$N \tan \phi + cL$$

where N is the total net force normal to the base.

(2) A force due to passive pressure distribution as described for walls with keys in paragraph 4.4.2a. This force will be assumed to be mobilized after the concentrated force limit is reached and will have no arbitrary upper limit. See part 2 of Figure 4-1.

It is recognized that a wall without a key, that has an inclined base as shown in part 2 of Figure 4-1, falls into a "gray area" of design with respect to overturning, since the wall could behave in a manner similar to a wall with a key. If the user should have a wall without a key but with a sloping base and want the wall to be analyzed with the procedure described in paragraph 4.4.2a (walls with a key), an input value of 0.01 foot for the key length will cause the desired action but with a wall that essentially has no key.

4.4.3 Limiting Value for Sliding - The limiting value of passive pressure for the sliding analysis may be selected by the user from the following options (methods a and b supply default values as appropriate):

a. For Floodwalls:

$$P_p = \frac{\gamma H^2}{2} \tan^2\left(45 + \frac{\phi'}{2}\right) + 2H \tan\left(45 + \frac{\phi'}{2}\right) c'$$

where

$$\phi' = \tan^{-1}(\tan \phi/FS)$$
$$c' = c/(FS + 2c')$$

[F, from equation's 4a & 4b on p. 27, with paragraphs 1-19e on p. 29]

where ϕ and c are test values.

b. For Retaining Walls (EM 1110-2-2502, paragraph 3c):

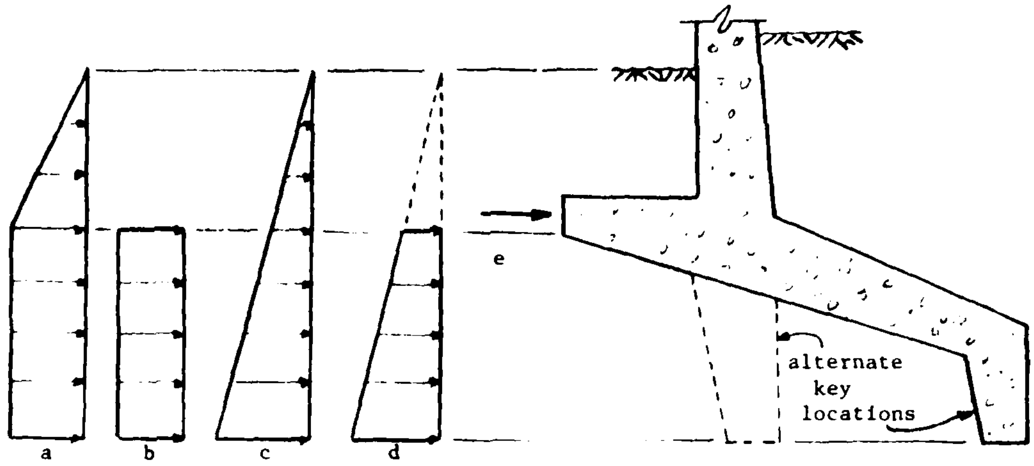
$$P_p \leq \frac{\gamma H^2}{2} \tan^2\left(45 + \frac{\phi'}{2}\right) + 2H \tan\left(45 + \frac{\phi'}{2}\right) c$$

Remember the warning about the uncertainty of cohesion given in the referenced EM paragraph. [F]

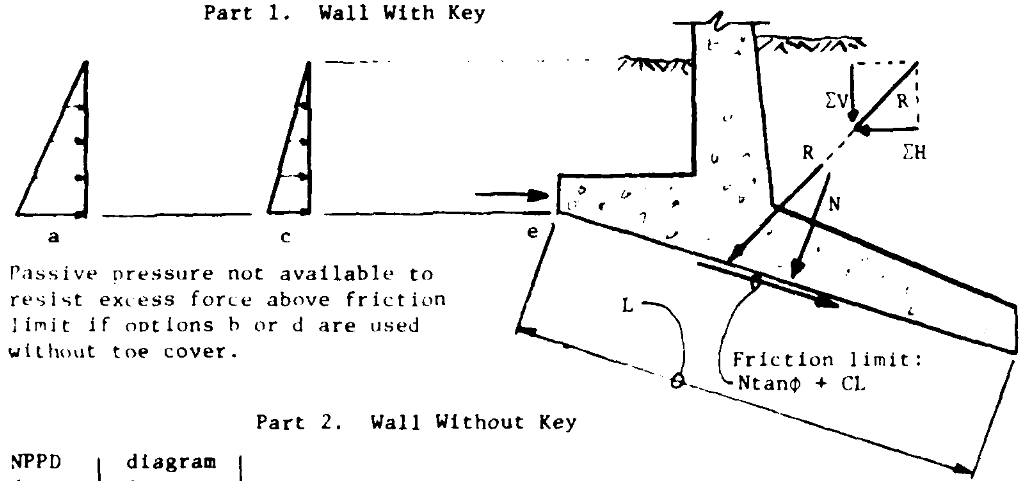
c. A User-Supplied Value - In the expressions in options a and b above, the factor, "$\tan^2(45 + \phi'/2)$" will be replaced by Coulomb's K_p as shown in paragraph 4.3.2 (adjusted for multilayer soil system).

4.4.4 Reduction of Excessive Required Passive Pressure from sliding stability is not possible through simple increase in base width, so the program will, in the design mode, increase base embedment, base slope, and/or key length instead. See paragraph 5.3.

4.4.5 An Alternate Resisting Strut Force will be provided as an option to act instead of passive pressure to provide horizontal restraint. This is to provide for structural features, such as channel bottom slabs, that would provide positive restraint. This is illustrated with NPPD = 5 in Figure 4-1.



Part 1. Wall With Key



Part 2. Wall Without Key

NPPD data value	diagram in parts 1 and 2	Description
1	a	As in Plate 5 of EM 1110-2-2501 (item b) (default distribution for floodwalls).
2	b	Alternate distribution for diagram a, when toe cover (Soil layers 6 & 7)# is assumed ineffective.*
3	c	Default distribution for retaining walls.
4	d	Alternate distribution for diagram c, when toe cover (Soil layers 6 & 7)# is assumed ineffective.*
5	e	Strut reaction, instead of passive pressure.

NOTE: Option e in diagrams (strut reaction) will cause all horizontal resistance to be taken by strut in both sliding and overturning calculations.

*Do not use passive pressure above base of toe unless construction and maintenance of backfill will assure its effectiveness.

See Figure 3-1

Figure 4-1. PASSIVE PRESSURE DISTRIBUTION OPTIONS

4.5 HYDROSTATIC PRESSURES

Hydrostatic pressures will be applied as explained in paragraphs 3.3.3f and 3.3.3k. In the computation of line-of-creep pressures, no creep losses are included along the portion of the base where the structure is not exerting a compressive pressure on the subgrade, except on the toe-side face of the key when this face is in passive contact with the subgrade. [D]

4.6 SURCHARGE PRESSURES

4.6.1 Vertical Pressures on the base will be computed from Boussinesq's equations:

a. The expression for vertical pressures under a point load (unit slice of a line load) is integrated from equation 35.1 on page 203 of Soil Mechanics in Engineering Practice, by Terzaghi and Peck, John Wiley & Sons, 1948:

$$p_v = \frac{3Q}{2\pi z^2}\left[\frac{1}{1 + \frac{r^2}{z^2}}\right]^{5/2}$$

where

p_v = vertical pressure, psf

Q = point load, lb

r = horizontal component of distance from load to point

z = vertical component of distance from load to point

b. The method to be used for computing vertical pressures under a distributed load is shown in Figure 17-7 and in the text between the figure and the end of paragraph 17.10 on pages 259-262 of Soil Engineering by M. G. Spangler, International Textbook Company, 1951: "...Equation 17-[illegible] and Fig. 17-4 may be used to determine the unit pressures at points in the soil which are not under the center of the area over which a uniform load is applied. The influence coefficient for the unit pressure under point O due to the uniform load on the area BCFH may be obtained from the coefficients for various rectangles, as follows: Rectangle BCFH = ACIO-ABPO-DFIO+DFHO. Multiplying the resulting coefficient by the uniform load on the rectangle BCFH gives the unit pressure under point O." Equation 17-8 from page 259 is shown below:

$$\frac{\sigma_z}{p} = \frac{1}{4\pi}\left[\frac{2ABH\sqrt{A^2+B^2+H^2}}{H^2(A^2+B^2+H^2)+A^2B^2}\cdot\frac{A^2+B^2+2H^2}{A^2+B^2+H^2} + \left(\sin^{-1}\frac{2ABH\sqrt{A^2+B^2+H^2}}{H^2(A^2+B^2+H^2)+A^2B^2}\right)\right]$$

where

A,B = width and length of loaded area

H = vertical distance from corner of rectangle A × B , down to the point where pressure is being calculated

p = uniformly distributed unit load

$_z$ = unit pressure at depth H

Figure 17-7 from page 261 is shown below:

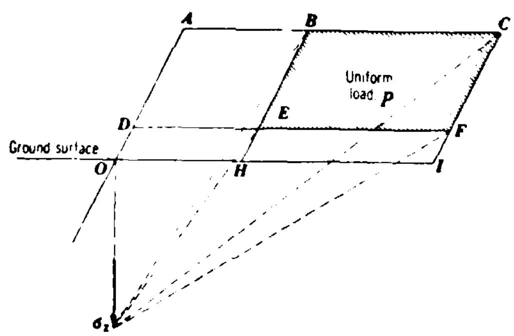


Fig. 17-7. Point in Soil Eccentric to Loaded Area

4.6.2 Horizontal Pressures will be computed using modified forms of the theory of elasticity equations described in Addendum B to Exhibit A. These pressures will be added to earth pressures computed according to Coulomb's equations. The incremental wedge method includes all surcharges in the trial wedge calculations. A study comparing the elasticity equations with the alternate concept of using Boussinesq's equations for vertical pressure and multiplying by the factor K_a is shown in Exhibit C.

4.7 EFFECT OF SEEPAGE PRESSURES ON EFFECTIVE EARTH UNIT WEIGHT

Seepage pressures tend to increase the effective unit weight of soil when the seepage flow is downward through the soil and to decrease the effective unit weight of soil when the seepage flow is upward through it. The program will calculate the effective unit weight as follows:

$$\gamma_{eff} = \gamma_b + \frac{\Delta H}{L} \gamma_w$$

for the wall side with the higher water elevation, or

$$\gamma_{eff} = \gamma_b - \frac{\Delta H}{L} \gamma_w$$

for the wall side with the lower water elevation, where

γ_b = buoyant unit weight of soil

γ_{eff} = effective unit weight of soil

γ_w = unit weight of water

$\Delta H/L$ = hydraulic gradient (reciprocal of creep ratio)

The user will have the option of using either effective unit weight or buoyant soil unit weight in the computation of active and passive pressures. Default action will be to use buoyant soil unit weight for both active and passive soil zones.

5. OVERTURNING STABILITY

5.1 GENERAL

5.1.1 Terminology is as described in paragraph 1-5 on page 8 of EM 1110-2-2501. [F]

5.1.2 Stability Requirements are grouped into sliding, overturning, and bearing. Sliding stability is considered in Section 6, overturning in Section 5, and bearing in Section 7 of this document.

5.1.3 Horizontal Pressures are calculated as described in Section 4. Actual and allowable bearing pressures are determined as described in Section 7.

5.2 ANALYSIS PROCEDURE

5.2.1 The Moment Center is taken as the toe end of the bottom of the base.

5.2.2 Force Summations in the horizontal and vertical directions will be made simultaneously so that resisting forces can be included when acting along a sloping base. See paragraph 4.4.2 and Exhibit K for a detailed discussion.

5.2.3 Moment Summation establishes the location of the resultant force, which then permits calculation of the earth bearing pressures as described in paragraph 7.1. Moments due to active earth will be multiplied by a "Correction Factor for Moment Arm," to shift the effective loading upward for the arching active case as discussed in paragraph 3g of EM 1110-2-2502. This will act to increase moment but not horizontal force. [R]

5.2.4 Sample Calculations are shown in Exhibit K.

5.3 DESIGN PROCEDURE

The design criteria and options are described in general terms in paragraph 3.3.3.

5.3.1 The Basic Program Procedure for both sliding and overturning stability can be diagrammed thus, in the form of nested FORTRAN DO loops:

User controls toe length parameter manually or sets stem ratio.

Toe elevation parameter is incremented by 3-inch units, between limits set by the user.

Base slope parameter is incremented by 0.1-foot-vertical or 1-foot-horizontal units, between limits set by the user.

Key length parameter is incremented by 3-inch units, between limits set by the user.

Program determines minimum base width (to nearest 3-inch unit) to satisfy resultant ratio, bearing pressure, sliding, creep ratio, and heel cover requirements.

If passive pressure from sliding stability check is too high, go to next key length; skip cost comparison.

Calculate construction cost; save design if cheaper than last saved design.

After the nested loops in the program have completed searching the ranges of possible parameter values within the limits set by the user, the design that yields the lowest estimated construction cost is presented to the user for his approval or revision.

5.3.2 By manipulating the limits, or by calling for just an analysis of any one combination of toe elevation, base slope, key length, and base width, he can satisfy himself that the full-range design is the best one. The analysis mode reports the estimated cost along with the pressure values and resultant ratio.

5.3.3 The user can set a desired value for any parameter by setting both the upper and lower limits for that parameter to the (one) desired value.

5.3.4 Section 7 describes how the program determines the actual and allowable bearing pressures under the base.

5.4 RESULTANT RATIO

5.4.1 Definition - The resultant ratio is defined as being the ratio of the horizontal distance from the end of the toe to the point of intersection of the resultant and the bottom of the base to the horizontal base width. [R]

5.4.2 Limiting Values

a. 0.33 minimum for:

(1) Floodwalls with water 3 feet below the top of the stem (EM 1110-2-2501 Loading No. 2). [F]

(2) Retaining walls loaded with active earth pressure and EM 1110-1-2101 Group I loads (EM 1110-2-2502). [R]

b. 0.25 minimum, recommended for:

(1) Floodwalls with water to the top of the stem (EM 1110-2-2501 Loading No. 1). [F]

(2) Retaining walls with at rest earth pressure on the stem or maintenance condition loading. This is interpreted to mean EM 1110-1-2101 Group II loads.

c. For an earthquake loading condition, the resultant may fall anywhere within the base provided the allowable foundation pressure for the earthquake loading condition is not exceeded. [D]

5.4.3 Relationship to Percent Effective Area - The relationship is simplified for this computer program by the consideration of only unit slices of wall.

a. The percent effective area is defined as simply the ratio of base area in compression to the total base area, times 100.

b. The relationship between percent effective area and resultant ratio is simple for the case of a rectangular base:

Percent Effective Area	Resultant Ratio
100	0.5
100	0.33
75	0.25
50	0.1667

There is a constant ratio of 300:1 between the values calculated from the two concepts for rectangular bases with resultant ratio values of 0.33 or less. For example, 75 ÷ 300 = 0.25.

c. The relationship for a trapezoidal base is more complex and is not as meaningful as it is for a rectangular base, so the derivation is not shown here.

5.5 UPLIFT

The action to be taken in the computations for overturning stability is modified if the resultant force falls outside the kern.

These changes in action affect uplift and bearing pressure computation, as described in Exhibit K.

6. SLIDING

6.1 AVAILABLE METHODS OF ANALYSIS

This program will use any of the following four methods for determining safety against sliding, as selected by the user:

6.1.1 Shear Friction Method - This method complies with guidance furnished in ETL 1110-2-184. It is the computer program default method for retaining walls. The method uses the passive pressure formula as given in paragraph 4.4.3b and the formulas for the horizontal components of sliding resistance which is mobilized along the assumed failure surface beneath the base of the wall, as given in paragraphs 6.2.1.1 and 6.2.1.2. Deficiencies associated with this method are discussed in paragraph 6.2.1.4. This method is illustrated in Exhibit H. [ED]

6.1.2 Allowable Strength Equilibrium Method (I) - This method complies with the provisions in EM 1110-2-2501, using different factors of safety on $\tan\phi$ and c applied to the driving and resisting forces. This method is the computer program default method for floodwalls. The passive resisting force is as presented in paragraph 4.4.3a. This method is illustrated in Exhibit I. [F]

6.1.3 Allowable Strength Equilibrium Method (II) - This method is the same as discussed in paragraph 6.1b above except the same factor of safety is used on $\tan\phi$ and c in both the driving and resisting forces. This method is illustrated in Exhibit J.

6.1.4 Modified Shear Friction Sliding Method - This method is presently incomplete and will be completed upon receipt of guidance from OCE. This sliding option, which will be based on a shear friction approach, will incorporate solutions to the deficiencies cited in paragraph 6.2.1.4 relating to the formulations presented in ETL 1110-2-184. In the interim, this option will be provided for in the structure of the computer program but not actually programmed.

6.2 DISCUSSION OF ANALYSIS METHODS

The first three methods listed in paragraph 6.1 are discussed in more detail in paragraphs 6.2.1 through 6.2.3.

6.2.1 Shear Friction Method (ETL 1110-2-184) - The nomenclature for Equations 6-1 and 6-2 is given below:

R = horizontal sliding resistance which can be mobilized along the critical path beneath the base of the wall

P_p = passive resistance of the earth or rock wedge adjacent to the wall

ΣH = net applied horizontal driving force

ΣV = summation of vertical applied forces above the assumed sliding plane which is below or at the base of the wall

A = area of the potential failure path which develops the unit shearing strength. (Any portion of the assumed failure plane at the base-foundation interface which is not in compression should be exluded from "A." However, if the assumed failure plane is not at the base-foundation interface but through the soil, no reduction in "A" should be made.)

ω = angle between the inclined failure path and a horizontal datum plane

c = cohesive strength = unit shearing strength at zero normal loading along the potential failure path beneath the base of the wall = test ultimate

ϕ = angle of internal friction of the foundation material (test ultimate value, degrees) or, where applicable, the angle of sliding friction of the wall on the subgrade

FS = factor of safety

The shear friction method is discussed in the following paragraphs.

6.2.1.1 Resistance Force for Uphill Sliding - The expression for "R" for uphill sliding resistance on a homogeneous (soil or rock) foundation material is taken from Equation No. 1 in ETL 1110-2-184. The angle ω is used with a positive sign in the equation

$$R = \Sigma V \left[\tan (\phi + \omega)\right] + \frac{cA}{\cos \omega \; (1 - \tan \phi \tan \omega)} \qquad (6\text{-}1)$$

See Exhibit B for derivation of Equation 6-1.

6.2.1.2 Resisting Force for Downhill Sliding - The expression for "R" for downhill sliding resistance on a homogeneous (soil or rock) foundation material is taken from Equation No. 2 in ETL 1110-2-184. The angle ω is used with a positive sign in the equation

$$R = \Sigma V \left[\tan (\phi - \omega)\right] + \frac{cA}{\cos \omega \; (1 + \tan \phi \tan \omega)} \qquad (6\text{-}2)$$

6.2.1.3 Safety Factor - Sliding stability can be evaluated by following the shear friction formula

$$S_{s-f} = \frac{R + P_p}{\Sigma H}$$

which relates the total available resistance ($R + P_p$) to the net applied force (ΣH) which tends to induce sliding. The full test values of ϕ and c are used in these calculations so that the forces are evaluated at their limiting values. This is the default method for retaining walls and is illustrated in Exhibit H. Passive pressure of earth is calculated using the formula shown in paragraph 4.4.3b or a user-supplied value.

6.2.1.4 Apparent Deficiencies in ETL 1110-2-184 Shear Friction Formulas - The apparent deficiencies in the shear friction safety factor as given in ETL 1110-2-184 are going to be discussed in a WES Miscellaneous Paper which is in the process of being written. These deficiencies are summarized as follows:

a. Any normal component of the passive pressure to the plane of assumed sliding is not considered in the frictional resistance (N tan ϕ). This deficiency exists only when considering inclined failure planes.

b. The safety factor in ETL 1110-2-184 is computed for an inclined plane using horizontal force components. In reality, the safety factor should be considered in a direction consistent with the inclination of the failure plane. The vectors which drive and resist the movement of the structure are along the failure plane; therefore, the safety factor should be the ratio of the resisting to driving forces in the direction of the inclined plane.

c. The frictional, cohesive, and passive resistances are assumed to develop at the same rate. In reality, these resistances do not develop at the same rate. It will therefore never be possible to have a total resistance equal to the sum of their maximums.

6.2.1.5 Passive Resistance of the Wedge Adjacent to the Wall - Passive resistance of soil is calculated as described in paragraph 4.4.3b.

A theoretical value for the passive resistance offered by a homogeneous rock wedge is

$$P_p = N \tan (\phi + w) + \frac{cA}{(\cos \omega)\ (1 - \tan \phi \tan \omega)} \qquad (6\text{-}3)$$

where

w = effective (submerged) weight of the rock wedge above the inclined plane of resistance (plus any surcharge loads)

ϕ = angle of internal friction or, if applicable, the angle of sliding friction

ω = angle between the inclined failure plane and the horizontal datum plane (for earth $\omega = 45 - \phi/2$ usually)

c = unit cohesion along the failure plane

A = area of the inclined failure plane along the base of the wedge

a. This simple formulation is valid for only a homogeneous foundation. For layered foundations, passive resistance will be calculated for each layer and superimposed. However, if fissures, seams, cracks, etc., are present, the user can have the program determine the force required to resist sliding and manually evaluate the rock subgrade for its ability to withstand this force. See paragraph 4.4.5.

b. Equation 6-3 above for passive resistance of a rock wedge can be directly transformed into the usual Coulomb equation for passive earth pressure by introducing assumptions and limitations which are consistent with the Coulomb theory (see Exhibit L for more details).

c. When the presence of a strut is indicated by the input data (item in NPPD being set to 5, see Figure 4-1), the resisting force is calculated as being equal to the net driving force multiplied by the input minimum safety factor (FSMIN) against sliding.

6.2.1.6 Values for the Minimum Shear Friction Safety Factor - Paragraph 7b(3) of EM 1110-2-2502 suggests a value of 1.5 for walls on rock. The minimum shear friction safety factor for a seismic load case should be two thirds of the safety factor for the normal load case, but not less than 1.15. [R] [ChS]

6.2.2 Allowable Strength Equilibrium Method (I) - Paragraphs 1-16 [F] through 1-19 of EM 1110-2-2501 describe a method for evaluating safety against sliding for floodwalls, based on applying a user-selected value of factor of safety on the $\tan \phi$ and c values in the equation for resisting and driving forces. Paragraph S-15d provides an additional description and allows the use of the assumed failure surfaces shown in paragraph 6.3.1 of this document. The procedure includes the following steps, as illustrated in Exhibit I.

a. Assume a trial value of factor of safety FS.

b. Calculate the allowable values for cohesion c' and friction ϕ' from Equations 4a and 4b in paragraph 1-17 of EM 1110-2-2501. They are formulated as follows:

$$c' = \frac{c}{FS + 2q}$$

where

$$q = c' \text{ (see paragraph 1-19e of EM 1110-2-2501)}$$

$$c' = \frac{c}{FS + 2c'}$$

and $$\phi' = \tan^{-1}\left(\frac{\tan \phi}{FS}\right)$$

The equation for c' can be rearranged to read

$$2(c')^2 + FS(c') - c = 0$$

With FS positive, this equation yields one positive real value of c' and one negative value. The positive one is used:

$$c' = \frac{\sqrt{FS^2 + 8c} - FS}{4}$$

where c and c' are in tsf.

c. Calculate K_A and K_p from ϕ'.

d. Sum all driving forces ΣD_ω parallel to the assumed failure plane below the base of the wall. ΣD_ω includes the net hydrostatic forces, if any, acting on the wall. The driving force contributed by the soil is calculated as a function of c' and ϕ', but the effect is neglected if its value is negative since the soil cannot pull on the structure.

e. Sum the resisting forces ΣR_ω parallel to the assumed failure plane below the base of the wall. The resisting forces consist of passive earth forces and friction and/or cohesion forces developed along the assumed failure plane below the base of the wall. ΣR_ω is formulated as a function of ϕ' and c'.

f. Two equations (ΣD_ω and ΣR_ω) then exist, each of which can be solved for any assumed safety factor FS. The expression $\Sigma D_\omega - \Sigma R_\omega = 0$ is solved by an iterative procedure with assumed safety factors. The actual safety factor for the structure is the one computed where $\Sigma D_\omega - \Sigma R_\omega = 0$.

6.2.3 Allowable Strength Equilibrium Method (II) - This method and its procedure are identical with the Allowable Strength Equilibrium Method (I), except that the factor of safety for cohesion FS_c is taken as being equal to the factor of safety for friction FS_ϕ. In this method,

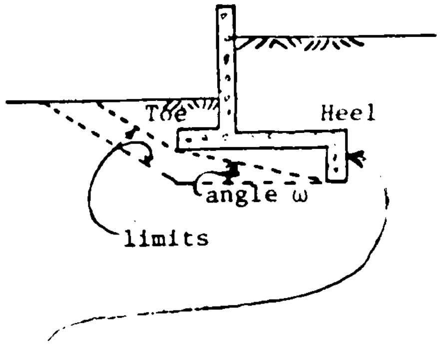
$$c' = \frac{c}{FS}$$

and

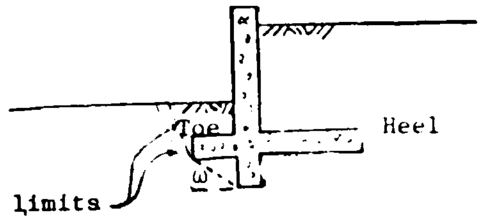
$$\tan \phi' = \frac{\tan \phi}{FS}$$

6.3 DISCUSSIONS COMMON TO ALL SLIDING METHODS

6.3.1 Assumed Failure Surfaces - The program will try a series of failure surfaces, each one made up of two planes, varying between the limiting cases shown below unless the user exercises his option of specifying a single value for the angle ω:



OR



This position is indicated in program by KFLAG=0 in the input data.

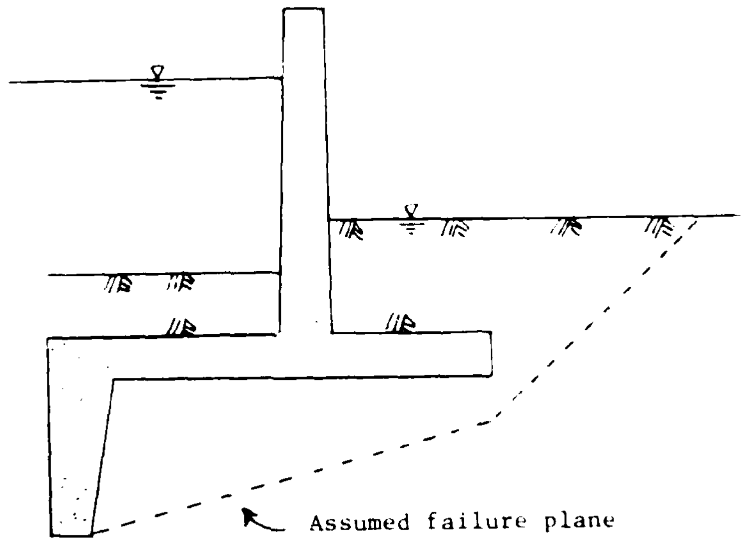
(The angle ω is defined as the angle from the horizontal, up to the assumed sliding failure plane.)

The situation shown above is still being studied by OCE and LMVD.

6.3.2 Effect Due to Some of the Base Not Being in Compression With the Foundation

6.3.2.1 Portion of Base in Compression With Foundation (Percent Effective Base) - The total base-foundation interface of the T wall may not be in compression with the foundation. If any part of the surface under consideration is along the base-foundation interface and is not in contact with the foundation, this portion should be neglected when obtaining the effective base area to resist sliding. However if the assumed failure surface is not along the base-foundation interface but through the soil, no reduction in the area to resist sliding is made.

6.3.2.2 Uplift



When the resultant falls outside the kern, a portion of the base of a T wall will not be in compression, thus creating a crack which can result in an increase in uplift pressures. This condition will affect the sliding stability analysis when the assumed sliding plane acts along the soil-structure interface below the base of the wall. (For this condition the program will have to recycle back through the line-of-creep calculations until the creep path assumptions match the final part of the base that is in contact with the foundation.) For example, consider a wall without a key and with a horizontal base. When the resultant falls outside the kern and the assumed sliding plane is along the interface between the base of the structure and the soil foundation, uplift pressures will be computed assuming no creep loss for the portion of the foundation not in compression. For the condition where the resultant falls outside the kern but the assumed sliding plane is through the soil, for example a wall with a key positioned at the extreme end of the heel, no increase in uplift pressure will be considered because the soil does not lift and form a crack as is the case at the soil-structure interface. Another reason the uplift forces are not affected for this condition is because they are forces inside the soil-structure free body and therefore do not affect the overall sliding stability.

6.5.2.3 Coefficients of Friction ϕ and Cohesion c to be Used in the Analysis - The ϕ and c values should be consistent with the material being sheared. A plane of failure through the soil should use the ϕ and c of the soil. For any of the failure planes along the soil-structure interface, use the ϕ and c for sliding friction at the interface.

7. BEARING PRESSURES

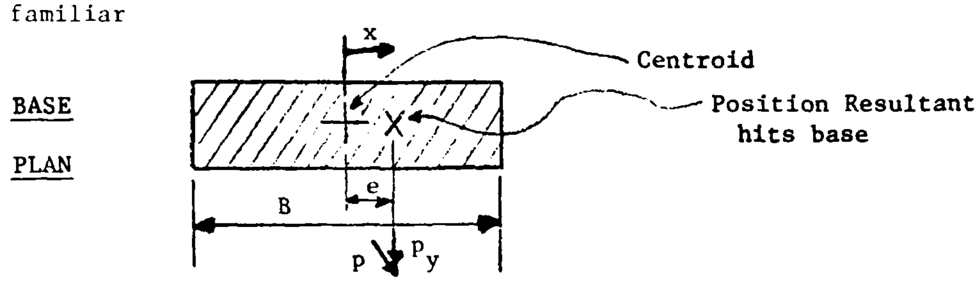
7.1 CALCULATION OF BEARING PRESSURES

Bearing pressures are calculated during overturning analysis from the vertical resultant (see paragraph 5.2.2 for summation of vertical forces) and its location (see paragraph 5.2.3 for summation of moments). The formulas for earth bearing pressure are different for a base in compression over its entire area and for a base with only part of its area in compression.

7.1.1 The Procedure is to first calculate the minimum pressure, assuming that the entire base is in compression. If the minimum pressure calculates to a positive value, then this was the appropriate assumption. If, however, the minimum pressure calculates to a negative value, then the assumption was not appropriate and the pressures must be recalculated with the assumption that only part of the base is in compression.

7.1.2 Entire Base in Compression - The formation and calculations for obtaining base pressures are straightforward for rectangular or trapezoidal bases; use $f = (P_y/A) \pm (Mc/I)$ (no biaxial moment). The properties of the base (A, I, c) are for the total base.

a. For a unit slice of a rectangular base, the formula becomes the familiar



$$f_{psf} = \frac{P_y}{B} \pm \frac{P_y ex}{\left(\frac{B^3}{12}\right)} = \frac{P_y}{B} \pm \frac{6P_y e}{B^2}$$

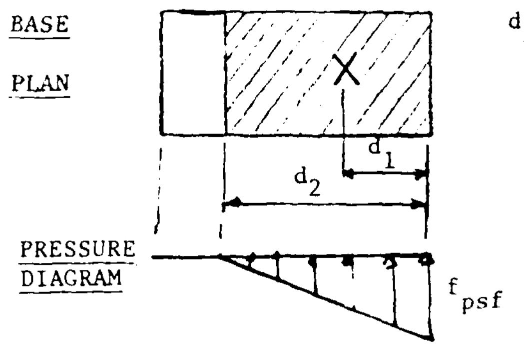
b. For a trapezoidal base, the formula becomes as shown in paragraph C-1 of Exhibit C.

7.1.3 Part of Base in Compression - The calculation of bearing pressure when only part of the base is in compression is a two-step process: First, find the location of the zero-pressure edge of the part in compression; then, calculate the maximum pressure:

a. For a unit slice of a rectangular base, the proceIure is

d_1 = known location of resultant

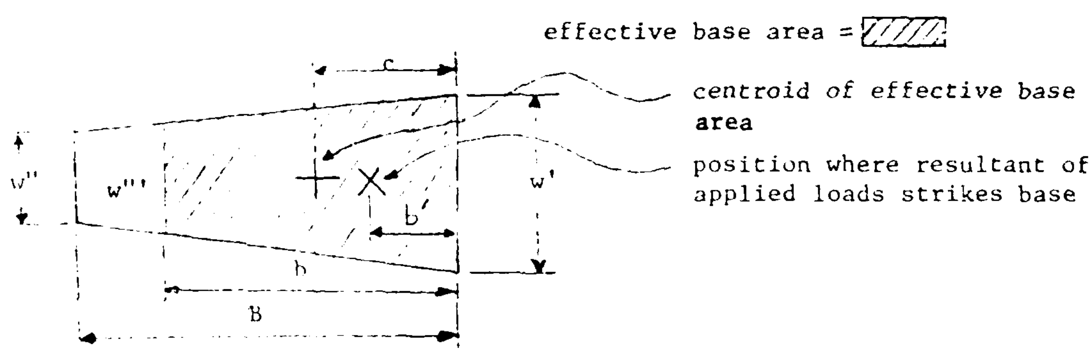
d_2 = width of base in compression



First, locate the zero-pressure point as being at $d_2 = 3d_1$, for the triangular distribution. Then, calculate the maximum pressure

$$f = \frac{2P_y}{d_2} = \frac{2}{3}\frac{P_y}{d_1}$$

b. For a trapezoidal base, the procedure is summarized below. The general figure is also given below (resultant outside kern, toward wide end):



the procedure is as follows:

(1) An incremental search across the base is done to find the zero-pressure point and the corresponding effective base width. The corresponding width must satisfy the expression

$$f = \frac{P_v}{A} + \frac{M(b - c)}{I}$$

(2) The maximum stress is

$$f = \frac{6R}{b(w''' + 2w')}$$

where w''' is computed using the effective base width. The details of the procedure are given in paragraph C-2 of Exhibit C.

7.2 MAXIMUM ALLOWABLE BEARING PRESSURES

7.2.1 Selection of Maximum Allowable Pressure - The determination of criteria for selecting allowable bearing pressures is beyond the scope of this document and of the computer program that it describes. The procedure in the program is based on linear double interpolation between limiting values read in as data.

7.2.2 Data Structure - The wall may bear on one or more of the existing soil layers (soils layers 3, 4, or 5 in Figure 3-1). There will be four allowable bearing pressure data values for each of these soil layers, as shown in the tabulation below:

Soil Layer Number	At Top of Layer		At Bottom of Layer	
	For Narrowest Base Expected	For Widest Base Expected	For Narrowest Base Expected	For Widest Base Expected
3	psf	psf	psf	psf
4	psf	psf	psf	psf
5	psf	psf	psf	psf

The "narrowest" and "widest" base widths consistent with these allowable bearing pressure values will also be input data, separately from any data parameter used to control the stability analysis action.

7.2.3 Method - The program will interpolate linearly between the top and bottom elevation of each soil layer to the elevation of the point being checked and between the "narrowest base" and the "widest base" values to the actual base width being checked. This check will be made at the highest and lowest base elevations in each soil layer.

8. EARTHQUAKE ANALYSIS [ChS]

8.1 INTRODUCTION

The seismic coefficient method of analysis is used in the following earthquake force computations:

a. The earthquake--induced inertial forces of the wall, plus that portion of the adjacent earth and/or water which is assumed to act as an added mass with the wall, is computed as the product of mass times acceleration (F = Ma).

b. The dynamic horizontal earth pressure magnitude and resulting force are approximated by the Mononobe-Okabe method.* [F]

c. The hydrodynamic force is obtained by Westergaard's theory. [R p. 6]

8.1.1 The seismic coefficient represents the ratio of an assumed acceleration of the wall to the acceleration of gravity. The earthquake forces are superimposed onto the static forces for the loading condition being analyzed for an earthquake (usually the normal operating condition) in order to obtain the total loads for the earthquake loading case. A static solution is then performed for the stability analysis and structural design.

8.1.2 Where a T wall is used as part of a dam, and where failure of the wall could result in loss of life or extensive property damage, then the final design of the T wall should be in accordance with ER 1110-2-1806. [E]

8.2 SELECTION OF SEISMIC COEFFICIENT

8.2.1 Values for the seismic coefficient should be selected carefully. [E] The most important consideration is the proximity of the structure to known earthquake epicenters and/or faults capable of generating an earthquake. Minimum seismic coefficient values for various sections of the Unite[illegible]s can be found in the seismic zone maps of Appendix B of ER 1110-2-[illegible] The seismic coefficient, when multiplied by the acceleration of gravity, gives the bedrock acceleration (also used as

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* Seed, H. B. and Whitman, R. V. 1970. "Design of Earth Retaining Structures for Dynamic Loads," _Proceedings, ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures_, pp 103-147.

structure acceleration) to be used in the analysis.

8.2.2 These maps can be used as a guide, in the absence of more accurate data, for determining the peak acceleration that should be used in the calculation of lateral pressures generated by earthquakes.

8.2.3 Structures that are in Seismic Zones 3 or 4, and which will endanger life or cause substantial property damage if they fail, and are on soil in which a liquefaction potential exists, should be analyzed by dynamic or pseudodynamic procedures (ER 1110-2-1806). Liquefaction potential for a material is defined as being when the relative density of the in-place material is less than 70 percent. Embankments constructed by hydraulic fill methods are usually found to be susceptible to liquefaction. [E]

8.3 LOCAL SOIL CONDITIONS

Soil foundations may modify the intensity of structure motions. A practical method for including the effects of soil foundations on structure motions is to vary the structure acceleration by a soil-structure-resonant factor of 1 to 1.5. This factor depends on the degree of similarity between the natural period of vibration of the retaining structure and the natural period of vibration of the soil foundation. A review of anticipated and actual damage patterns as a function of soil conditions suggests that the forces and damaging effects included in different types of structures are maximized when there is a similarity in the natural periods of the structure and the ground on which it rests. When the natural period of vibration is not properly established for a soil foundation, the seismic coefficient should be multiplied by 1.5.* [E]

8.4 INERTIA FORCE OF WALL

The inertia force of the wall mass is computed by multiplying the selected seismic coefficient by the weight of the wall. It is applied at the center of gravity. This force is obtained by multiplying the mass by acceleration as follows:

$$F = ma = ma\ (g/g) = (a/g)\ W = \alpha' W$$

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* Uniform Building Code, 1976 Edition, p. 135.

Also, that portion of the backfill above the heel or toe of the wall which is not included as part of the Coulomb wedge is included as an inertial force acting at its centroid.

8.5 LATERAL COHESIONLESS EARTH PRESSURES DUE TO EARTHQUAKES

The dynamic earth pressure magnitude is approximated by the Mononobe-Okabe method. This assumes that the soil developes a Coulomb wedge behind the wall and that the ground acceleration is uniform within the wedge. The effect of the earthquake motions can be represented by forces $k_h W$ and $k_v W$ where W is the weight of the sliding wedge and $k_h g$ and $k_v g$ are the horizontal and vertical components of the earthquake acceleration at the base of the wall. (See Seed and Whitman.)

8.5.1 Active Earth Pressure Conditions - The active pressure force P_{ae} due to an earthquake is computed, in effect, by the Coulomb theory except that the additional inertia forces $k_h W$ and $k_v W$, as shown in Figure 8-1 are included in the computations. Determining the critical sliding surface and the active pressure corresponding to thi urface leads to the equations as given in Figure 8-1. These are th juations used to compute the active earth pressure in this prog am.

a. The influence of the vertical acceleration coefficient k_v on the earth pressure coefficient K_{ae} depends on the corresponding component of the horizontal acceleration coefficient k_h. For most earthquakes, the horizontal components are considerably greater than the vertical components. To study the influence of k_v, the vertical acceleration coefficient is taken as $2/3k_h$. For a value of k_h of 0.1, a value of k_v of $2/3k_h$ equal to 0.067 can cause increases or decreases in K_{ae} of about 2 percent. For a value of k_h of 0.3, a value of k_v of 0.20 can cause increases or decreases in K_{ae} of less than 5 percent. From this it seems reasonable to conclude that the influence of k_v can be neglected for practical purposes. (See Seed and Whitman.) Neglecting k_v, the expression for P_{ae} becomes

$$P_{ae} = \frac{\gamma H^2}{2} (K_{ae})$$

b. For applying the increased earth pressure due to an earthquake acceleration, ΔP_{ae} is computed by using the following expression:

$$\Delta P_{ae} = \frac{\gamma H^2}{2} (\Delta K_{ae})$$

where

$$\Delta K_{ae} = K_{ae} - K_a$$

and

$$P_{ae} = P_a + \Delta P_{ae}$$

where P_a is the total active static force. P_a is applied at 1/3H (see Figure 8-1) above the base and ΔP_{ae} is applied at 2/3H above the base. The distribution of ΔP_{ae} is assumed to vary linearly from a maximum at the top of the wall to zero at the base. This is important in the design of sections along the structure.

$$P_{ae} = \frac{\gamma H^2}{2} (1 - k_v) K_{ae}$$

where

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

$$\theta = \tan^{-1} \left(\frac{k_h}{1 - k_v} \right)$$

k_h = horizontal ground acceleration

k_v = vertical ground acceleration

γ = unit weight of soil

H = height of vertical plane AB

ϕ = angle of friction of soil

$\delta = \beta$

β = slope of ground surface behind wall

α = 90 degrees

c. The critical sliding surface was considered as in Coulomb theory and the active pressure corresponding to this surface leads to the expression for K_{ae}. It is possible to simplify the mathematical calculation for K_{ae} such that it can be obtained directly from Coulomb's active pressure coefficient, as described by Seed and Whitman.

d. For the more general case, the variation of ΔK_{ae} with a_h, shown in Figure 8-2, depends greatly on the slope angle of the backfill β.

e. The wedge's critical sliding surface along an irregular backfill will be approximated by a single line or a continuous series of straight lines, as the user decides. The increase in horizontal earth pressure due to the earthquake will be formulated and computed by multiplying the mass times the acceleration for the various triangular or trapezoidally shaped earth masses. The force will act through the centroid of the mass and in the most critical direction for the condition being considered.

8.5.2 Passive Earth Pressure Conditions - The increase or decrease in

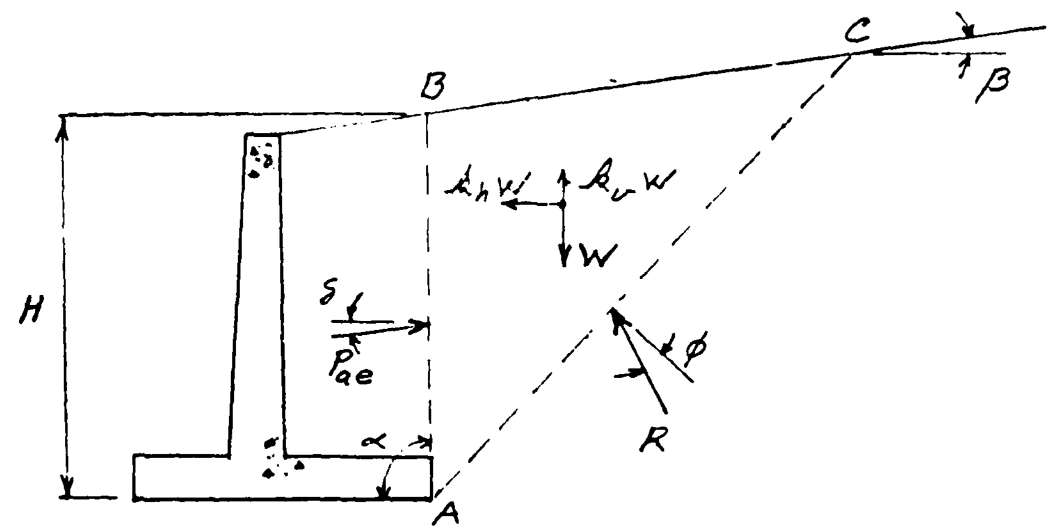
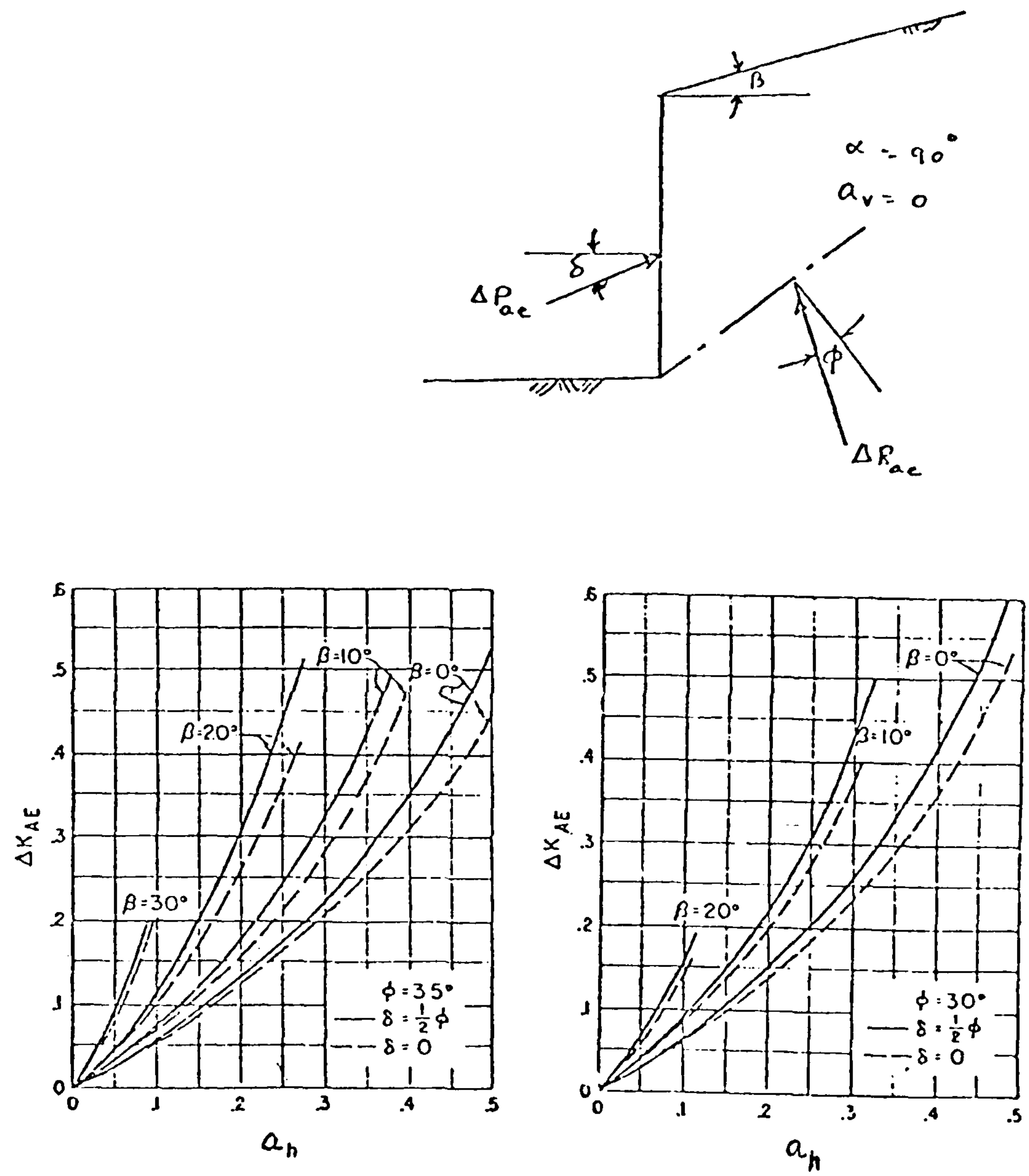


Figure 8-1. ACTIVE WEDGE DURING EARTHQUAKE



Example = A_h = 0.10g, β = 0.0, Ø = 30°, δ = 0.0° :

ΔK_{ac} = 0.07 from second graph.

Figure 8-2. VARIATION OF DYNAMIC EARTH PRESSURE COEFFICIENT WITH THE HORIZONTAL GROUND ACCELERATION A_h

a fully passive earth pressure due to an earthquake is computed as follows:

$$\Delta P_{pe} = \frac{\gamma H^2}{2} (\Delta K_{pe})$$

where

$$\Delta K_{pe} = K_{pe} - K_p$$

where

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

and

K_p = static passive pressure coefficient for the fully passive case (see 4.1.4b)

For the case where passive earth pressure is used as a stabilizing force, a reduction in the passive earth pressure due to an earthquake acceleration is assumed at the same instant the fill pressure behind the structure is increased. If a reduction in K_p has been used for computing an effective K_p for the static case, this same reduction in ΔK_{pe} is used. ΔP_{pe} is applied at $2/3H$ above the base. The pressure distribution of ΔP_{pe} is the same as assumed for the active earth pressure condition.

8.5.3 <u>At Rest Earth Pressure Conditions</u> - The increase in an at rest earth pressure due to an earthquake is approximated by the Mononobe-Okabe method. The change in the active earth pressure coefficient ΔK_{ae} is first computed as described in paragraph 8.5.1, and then multiplied by the ratio K_r/K_a to obtain the change in the at rest earth pressure coefficient ΔK_{re}. The change in at rest earth pressure is then computed as follows:

$$\Delta P_{re} = \frac{\gamma H^2}{2} (\Delta K_{re})$$

Only the horizontal component of the earthquake acceleration is considered. ΔP_{re} is applied at two thirds the height of the fill above the base. The pressure distribution of ΔP_{re} is the same as assumed for the active earth pressure condition.

8.6 LATERAL COHESIVE EARTH PRESSURES DUE TO EARTHQUAKES

The computation of the dynamic earth pressure for cohesive soils is beyond the scope of this computer program. A nonlinear finite element analysis to account for inelastic strains in the soil could possibly be used for critical cases.

8.7 WATER PRESSURE DUE TO EARTHQUAKES

8.7.1 Method - If the backfill over the toe or the heel is saturated to some level, the dynamic soil force should be determined by using a combination of the Mononobe-Okabe and Westergaard theories. When water exists on both sides of the structure for the loading condition being analyzed, simultaneous increases and decreases in water pressure on opposite sides of the structure are included in the analysis. To include the dynamic effect of water in a saturated backfill using Westergaard theory, the computation is divided into two parts. For that part of the soil below the saturation line, the increase of the force from the earth pressure is computed by the Mononobe-Okabe method using the buoyant soil weight. The increase of the force due to the water in the backfill is computed by the Westergaard theory, and 100 percent of this is used and applied with the other forces as shown in diagram (5) of Figure 8-3. If water is the only medium which acts on the wall, the Westergaard theory is applicable to determine the pressure and its distribution on the wall.

8.7.2 Westergaard Theory - By the Westergaard theory, the dynamic water pressure down to depth y below the surface for a total water depth h is expressed by Equation 3 on page 5 of EM 1110-2-2200 as [D]

$$P_{e_2} = \frac{2}{3} C_e \alpha y \sqrt{hy}$$

The additional moment at depth y due to P_{e_2} is given by

$$M_e = \frac{4}{15} C_e \alpha y^2 \sqrt{hy}$$

with

$$C_e = \frac{51}{\sqrt{1 - 0.72\left(\frac{h}{1000\ t_e}\right)^2}}$$

where g is acceleration of gravity (32.2 ft/sec^2).

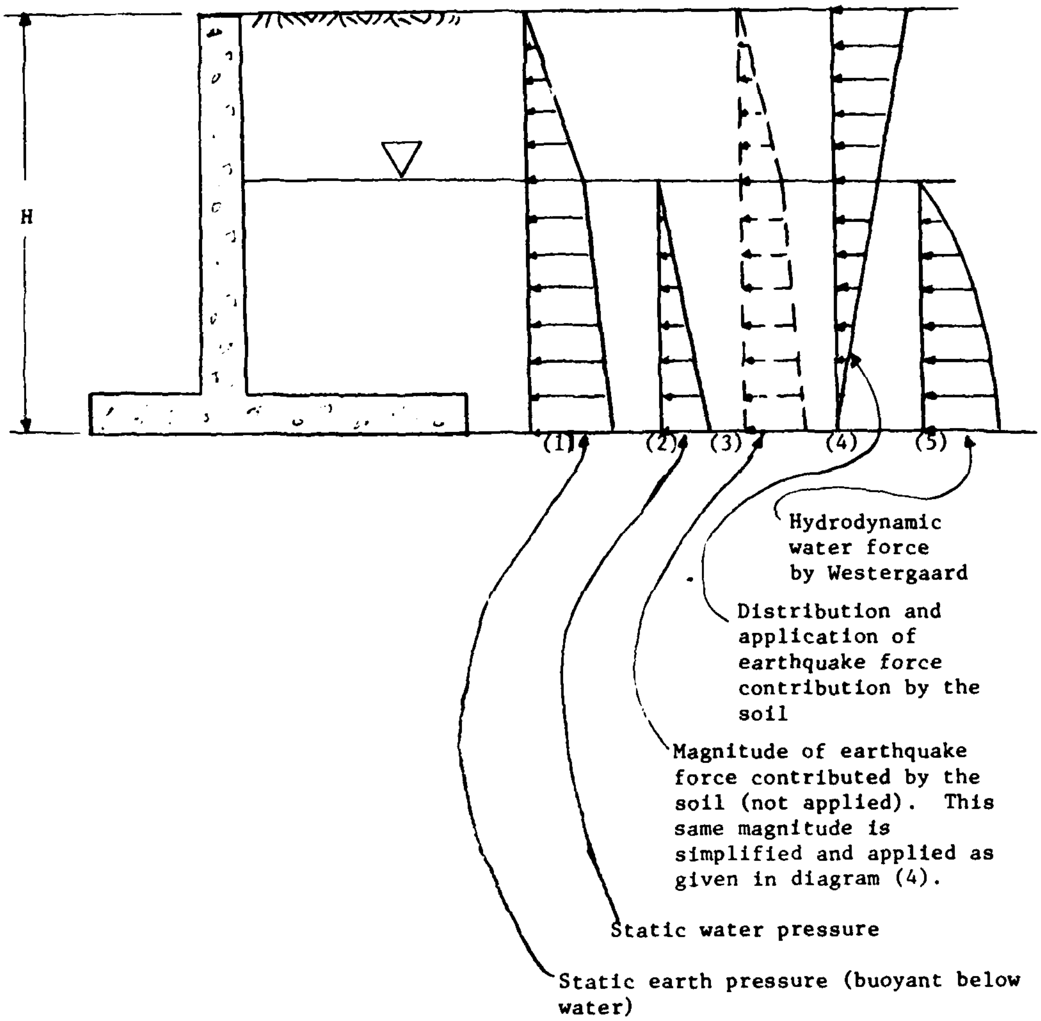


Figure 8-3. DYNAMIC PRESSURE VARIATIONS

9. STRUCTURAL DESIGN CRITERIA

9.1 GENERAL

9.1.1 Critical Sections and Alternate Pressure Diagrams will be selected as described in paragraphs 1-12, Chapter 1, of EM 1110-2-2501 as amended by Paragraph S-21 of the March 1961 Supplement to EM 1110-2-2501, unless the user elects to use the overturning stability pressures exactly as calculated. [F] The default procedure will be as described in EM 1110-2-2501. All wall components will be designed for flexure, shear, and deflection. The heel slab sill also be designed for these effects plus axial tension caused by passive pressure on the key (if the key is under the heel). The stem will be designed for flexure, shear, and deflection, plus the axial compression caused by the weight of concrete. Critical sections for shear will be at the face of the stem in the heel, at the bottom of the slab in the key, at a distance d from the face of stem in the toe, and at a distance d above the top of the slab in the stem. The key and/or toe should be checked as deep beam brackets if they are short enough compared to their thicknesses.

9.1.2 The Method of Analysis will be chosen by the user from the two options of Ultimate Strength Design (USD) or Working Stress Design (WSD), both in accordance with American Concrete Institute Standard No. 318-77 (ACI 318-77) with notation from the 1971 edition (ACI 318-71 with 1976 Supplement). This standard will be modified by the requirements of EM 1110-1-2101, "Working Stresses for Structural Design," for WSD. The label "hydraulic" or "nonhydraulic" will be set as described in paragraph 1.2.3.2 and used to select alternate sets of default values for analysis parameters. These values may be changed by input data.

9.1.2.1 Working Stress Design procedures to be used will be in accordance with the "Alternate Design Method" described in paragraph 8.10 of ACI 318-77. The equations and default parameters to be used are shown in Exhibit E.

9.1.2.2 Ultimate Strength Design equations and default parameters are shown in Exhibit F for nonhydraulic structures. These equation and parameters for hydraulic structures will be issued when completed and approved by OCE.

9.1.3 Pressures for Structural Design will be as described in paragraph 1.2.6d. Stems will be designed for pressures selected as described in paragraph 9.3. Heel, toe, and key slabs will be designed for pressures derived from the overturning stability analysis described in paragraph 5.2. The program is to be organized to permit use of load factors in ACI Ultimate Strength Design (USD) procedures, but implementation of this will be postponed until after completion of current research in USD of hydraulic structures. Stability analysis and working stress design will be for the summation of the actual loads.

9.2 REINFORCING STEEL DETAILS

9.2.1 The Primary Reference is Chapter 7 of ACI Standard 318-71 with the 1976 Supplement. This is supplemented and amended for hydraulic structures by EM 1110-2-2103, "Details of Reinforcement--Hydraulic Structures," whenever the EM is more restrictive. [HSR]

9.2.2 Default Values for Concrete Clear Cover over reinforcing steel will be according to the following table:

	Structure Type	
	Ordinary	*Hydraulic [HSR]*
Stem, heel-side face	2 inches	3 inches
Base, top face	2 inches	3 inches
Base, bottom face	3 inches	4 inches

9.2.3 Compressive Reinforcement will not be used, for two reasons:

a. The bar cover requirement, especially for hydraulic structures, forces any compressive reinforcement so close to the neutral axis that it is very inefficient.

b. Consideration of the implications of ACI 318-71 paragraph 9.12.5 and the corresponding Commentary leads to the belief that lateral reinforcement would be required to keep compressive reinforcement from popping out of the surface of the slab. [ACI]

9.2.4 Placement of Reinforcing Steel - The user will have specified in the input data the maximum bar size and the minimum bar spacing. The program will check whether one such layer will be sufficient for the required area of steel per foot. If not, the program will place as much steel as possible in the first (outer) layer and the remaining steel in the second layer (and a third layer if needed). The program will then check to see if the resulting actual effective depth d is sufficient. If it is not sufficient, the program will increase the concrete thickness as needed. The user will always have the option of looping back through a new stability analysis or design. The program will not attempt to place and cut off bars, but will show the required steel area and bond requirements per foot at suitable intervals across the base and up the stem.

9.3 STEM LOADINGS

9.3.1 General - Earth and other pressures for stem structural analyses will be calcualted as described in Section 4, applied directly to the face of stem as described in paragraph 4.2. Thus, surcharges located over the base will be considered to be vertical loads for stability analysis but will be considered to cause horizontal pressures applicable to stem stress analysis.

9.3.2 Alternate Loading - It will be possible for the user to substitute his own stem design pressures for the ones calculated by the program. Such a substitution will not change the pressures used for design of the base slabs.

EXHIBIT A: COMPARISON OF COULOMB AND INCREMENTAL WEDGE EARTH PRESSURE VALUES

This exhibit is a report by Dr. Michael W. O'Neill, Assistant Professor of Civil Engineering at the University of Houston. The report presents the calculation procedures he will use in a subroutine to calculate active earth pressures that will be a part of modules SA and SP of TWDA.

Procedures Followed

I. Coulomb Active Earth Pressure Diagram and Total Active Force:

A. Following the procedure described in Addendum A to this Exhibit, compute the active Coulomb earth pressure immediately above and immediately below each change of stratum, except for the surface stratum line (for which calculations are not made above the line) and a horizontal line drawn through the base of the wall (for which calculations are not made below the line).

B. Obtain the horizontal component of the earth pressure by multiplying each earth pressure computed in Step A by the cosine of the angle of wall friction at the depth at which the calculations are being made (Equation 2 of Addendum A). Add hydrostatic pressure if appropriate.

C. If surcharge loads exist, obtain the added lateral pressures due only to the surcharge at the depths described in Step A and at intermediate depths where required by a high stress gradient using the equations presented in Addendum B. Add these pressures to those obtained in Step B and connect the points to give a continuous pressure distribution.

D. Integrate the horizontal pressure diagram constructed in Step C from the top to the base of the wall to obtain the total active Coulomb horizontal wall force P_{AHC}.

II. Trial Wedge Active Earth Pressure Diagram and Total Active Force:

A. Establish several nodes along the wall. Each node will be a point for which the active force acting above the point will be computed.

Five equally spaced nodes were used in this comparative study.

B. For each ith node, lay off several straight lines (rays) at 5- to 10-degree intervals extending to the ground surface. See Addendum C. Compute the active force on the wall for the wedge defined by each ray, the wall, and the ground surface, including the weights of all surcharge loads between the wall and the point of intersection of the ray with ground surface, using the method shown in Addendum C.

C. Select the active force for Node i as the maximum value obtained from all rays through Node i. Denote the horizontal component P_{AHTi}.

D. Determine the increment of active force acting between Nodes i and i-1 as $P_{AHTi} - P_{AHTi-1}$. Divide the result by the vertical distance between Nodes i and i-1. The resulting pressure is the average pressure between Nodes i and i-1, which is assumed to be constant between the nodes.*

E. Obtain the total active horizontal trial wedge wall force P_{AHT} by integrating the pressure diagram obtained in the previous steps from the top of the wall to the base.

Discussion of Comparisons

For Cases 1, 3, and 6, the Coulomb and trial wedge procedures should give identical results. The differences obtained in the pressure diagrams and total horizontal active forces are due to minor computational errors in the graphical procedure for obtaining the trial wedge values and to the means of displaying the results (i.e., step functions for trial wedge as opposed to continuous functions for Coulomb). By increasing the number of nodes, the trial wedge solutions will converge to the Coulomb solutions.

For Cases 2 and 7, the Coulomb procedure yields higher values of active pressure primarily due to the method of handling surcharge. In the trial wedge approach, the surcharge weight contributes to and is

* Wu, T. H., Soil Mechanics, 1st ed., Allyn and Bacon, 1966, p 278.

included in a plastic failure model (the wedge). In the Coulomb analysis, the pressures due to the backfill are computed assuming plastic failure conditions, but the surcharge effects utilize elastic theory to compute the added pressures due to surcharge. Combination of the plastic and elastic methods in the latter case produces an inconsistency which yields an overestimate of the lateral stresses.

For Cases 4, 5 and 8 (and for Case 9 as well), use of the Coulomb active pressure coefficients in a $\phi = 0$ soil (clay) is equivalent to using Rankine-Rosae coefficients. That is, the Coulomb equations as commonly employed offer no advantage over the Rankine equations for use with purely cohesive soil and, in fact, lead to an inaccurate estimate of the active stresses within the zone of purely cohesive soil. When the backfill surface is horizontal, Coulomb active pressures in the clay are too high (Cases 4 and 5), and when the backfill surface slopes upwards, the Coulomb active pressures are too low.* Further, the standard Coulomb equation for active pressure coefficients does not contain a provision for specifying wall adhesion as a function of soil cohesion. Only an angle of soil-wall friction can be specified. Thus, not only are the Coulomb equations inaccurate for pure clays ($\phi = 0$), they are also inconvenient to use since wall adhesion must be converted to an equivalent angle of wall friction, a process for which few criteria exist.

For Case 9, the Coulomb approach is quite approximate since the active earth pressure equations do not permit modeling of a broken backfill surface. The degree of correlation with the trial wedge approach is dependent on the engineer's choice of the equivalent backfill slope, a factor obtainable only through subjective means.

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* When [illegible], the Coulomb and Rankine active pressure coefficients are complex numbers. The value used in computations is the real part of that number, which is always approximately equal to unity regardless of the slope of the surface.

Addendum A

Procedure for obtaining Coulomb active earth pressure at a given depth (Symbols explained on following sheet)

1. Compute:

$$K_a = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha \sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2} \quad \ldots\ldots(1)$$

For clay backfill, wall adhesion is considered by using a value of δ of 10^{o} and $\phi = 0$. For this special case, when $\alpha = 90^{o}$:

$$K_a = \frac{1}{\sin(90-\delta)\left[1+\sqrt{\frac{\sin(\delta)\sin(-\beta)}{\sin(90-\delta)\sin(90+\beta)}}\right]^2}$$

and when a horizontal backfill exists $\beta = 0$, so that

$$K_a = \frac{1}{\sin(90-\delta)}$$

NOTE: The above coefficient approximates the Rankine value.

NOTE: The properties ϕ and δ are those exactly at the depth under consideration.

2. Compute the active pressure (horizontal component):

$$p_h = \left[\bar{p}_\gamma K_a - 2c\sqrt{K_a}\right]\cos\delta + \gamma_w z_w \quad \ldots\ldots(2)$$

NOTE: Again, c and δ are the properties of the soil and interface exactly at depth under consideration.

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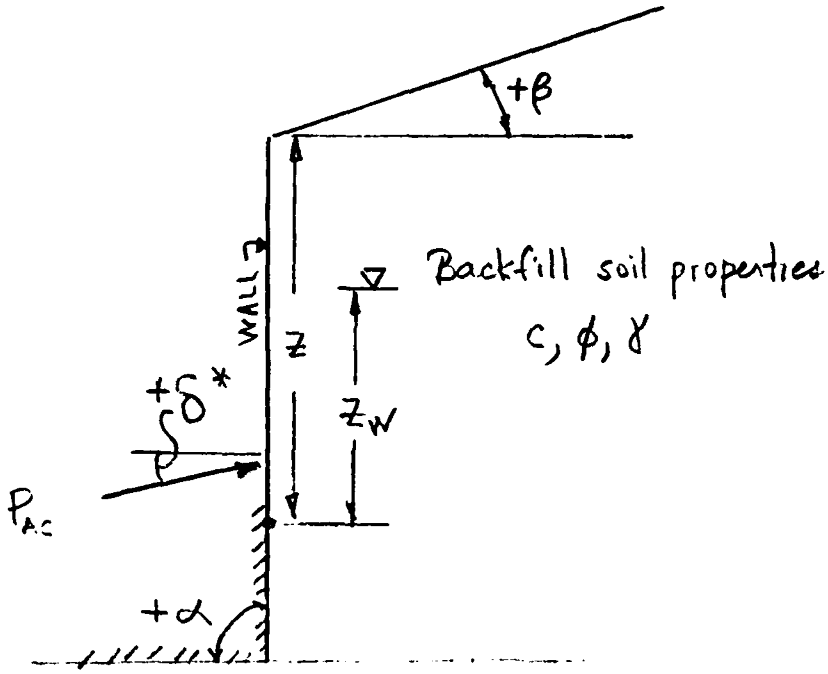
[1] Bowles, J. E., Foundation Analysis and Design; McGraw-Hill, 1968, p. 270.

[2] Ibid, p. 284



* δ = angle of soil-wall friction (obliquity of active force)

$$\bar{p}_v = \gamma\,(\text{averaged}) \cdot Z - \gamma_w \cdot Z_w$$

γ_w = unit weight of water.

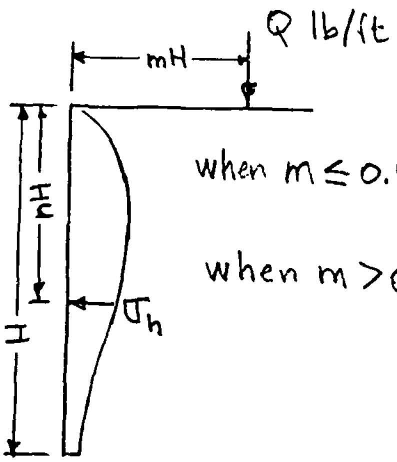
NOMENCLATURE FOR COULOMB EQUATIONS

Addendum B

Lateral earth pressure on walls using theory of elasticity[1]

To the lateral pressures calculated from the use of the Coulomb equation the following lateral pressures are added:

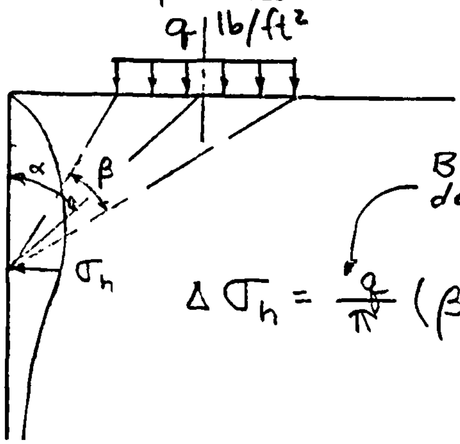
Case A: Line Load



when $m \le 0.4$ $\quad \Delta\sigma_h = \dfrac{Q}{H}\,\dfrac{0.203n}{(0.16+n^2)^2}$

when $m > 0.4$ $\quad \Delta\sigma_h = \dfrac{4}{\pi}\,\dfrac{Q}{H}\,\dfrac{m^2 n}{(m^2+n^2)^2}$

Case B: Strip Load

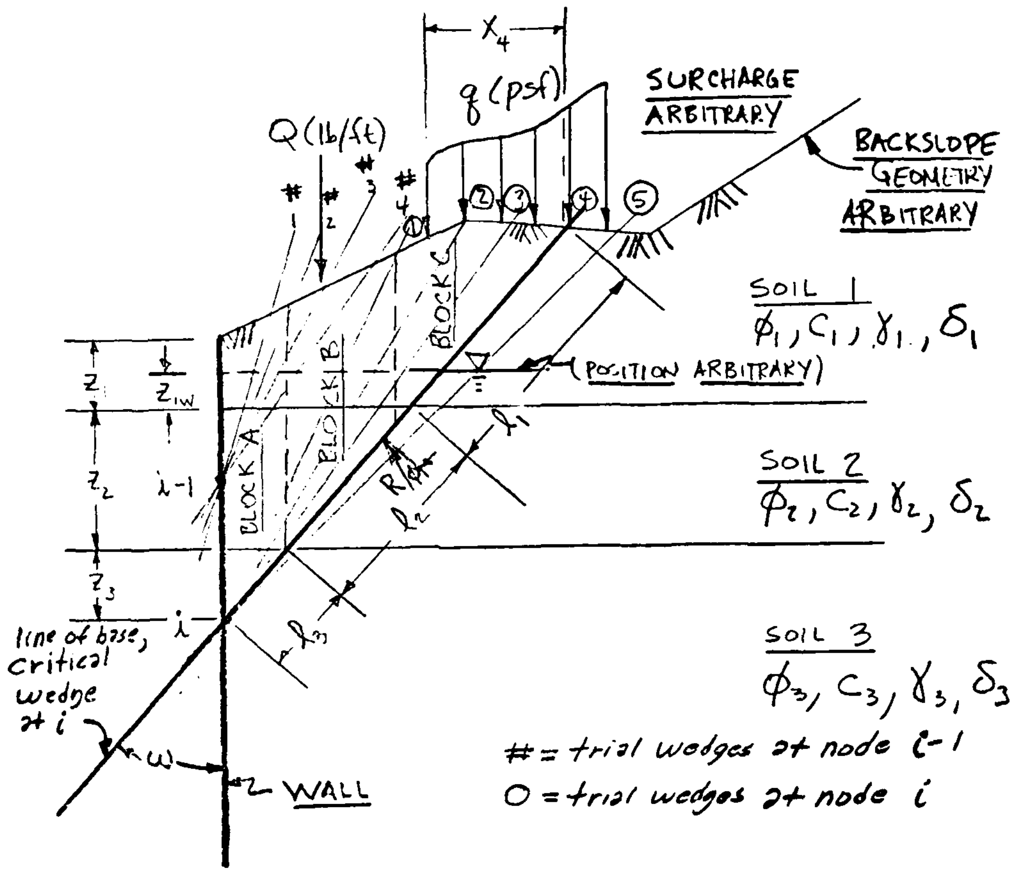


Bowles' factor of 2 deleted by WESKD, NOV 78

$$\Delta\sigma_h = \frac{q}{\pi}\left(\beta + \sin\beta\sin^2\alpha - \sin\beta\cos^2\alpha\right)$$

[1] Bowles, J. E., Foundation Analysis and Design, McGraw-Hill, 1968, pp. 297-303.

Active force for one wedge (generic)



EXAMPLE COMPUTATIONS FOR NODE i, Ray 4.

Weight (W) :

$$W = \text{weight}^* \text{ of Block A} + \text{weight}^* \text{ of Block B} + \text{weight}^* \text{ of Block C} + Q + q\,(X_4)$$

Cohesion (C) :

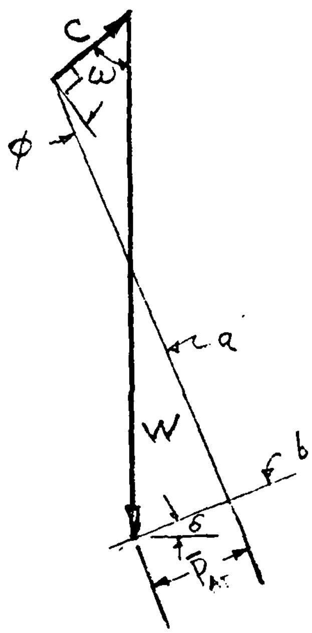
$$C = c_1 \ell_1 + c_2 \ell_2 + c_3 \ell_3$$

*Buoyant below water table. Weights of Blocks (including effects of buoyancy) denoted by W_A, W_B, W_C.

$$\phi = \frac{W_A \phi_3 + (W_B + Q)\phi_2 + (W_C + q X_4)\phi_1}{W_A + W_B + W_C + Q + q X_4}$$

$$\delta = \frac{z_1 \delta_1 + z_2 \delta_2 + z_3 \delta_3}{z_1 + z_2 + z_3}$$

Determination of Active Force for Node i, Ray 4:[1][2]



Lay off vectors W and C, whose magnitude and directions are known.

Lay off Line a, which is the vector of the lateral soil reaction force against the wedge (R). Only its direction is known. Head is at tail of C vector.

Lay off Line b, which is the vector of the desired active force ($\bar{P}_{AT}$). Tail is at head of W vector.

Intersection of a and b defines magnitude of $\bar{P}_{AT}$.

Compute P_{AHTi4} (horizontal component of total active wall force, Node i, Ray 4)

$$P_{AHTi4} = \bar{P}_{AT} \cos\delta + \gamma_\omega \left(z_{1w} + z_2 + z_3\right)^2 (0.5)$$

where γ_ω = unit weight of water.

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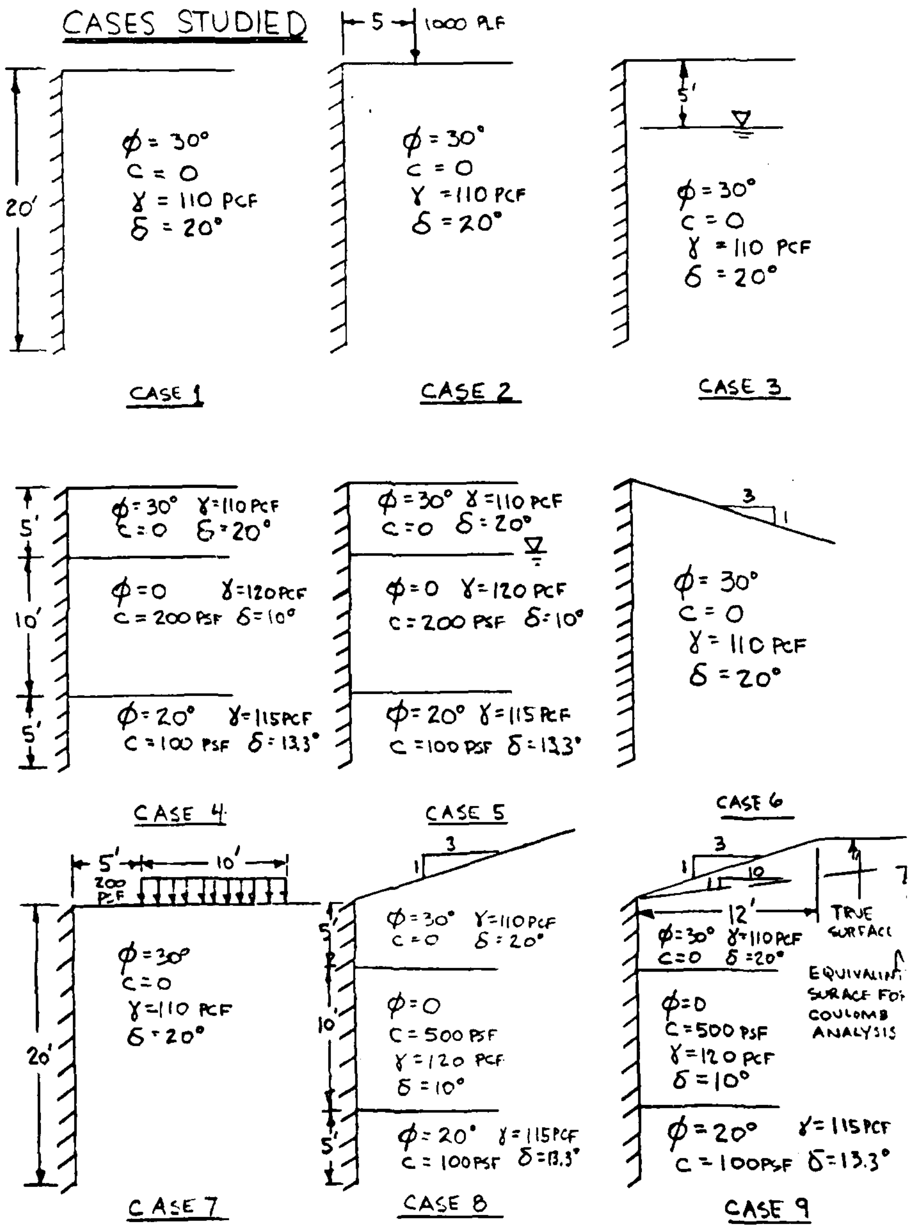
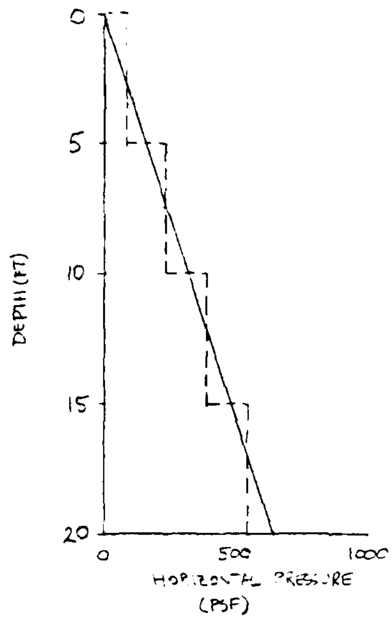
[1] Bowles, J. E., Foundation Analysis and Design, McGraw-Hill, 1968, pp. 291-295 (Basic Vector Diagram and Mechanical Procedures)

[2] Terzaghi, K., Theoretical Soil Mechanics, Wiley, 1943, pp. 95-99 (Stratified and Cohesive Backfill Considerations).

Addendum D

Pressure comparisons





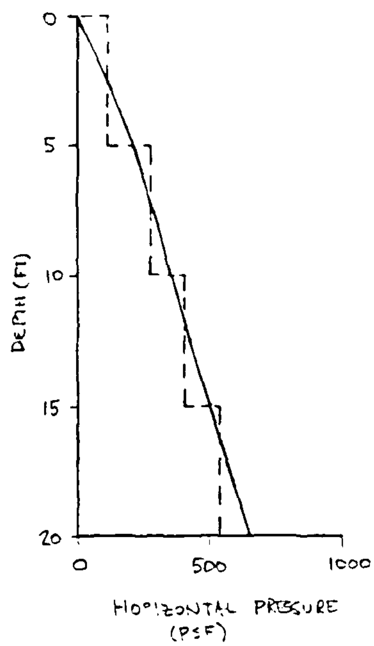
CASE 1 -

HORIZONTAL STRESSES

P_{AHT} = 6215 lb *
P_{AHC} = 6140 lb *

——— COULOMB
– – – – TRIAL WEDGE

* THEORETICALLY, SHOULD BE IDENTICAL. DISCREPANCY LIES IN MINOR GRAPHICAL CONSTRUCTION ERRORS



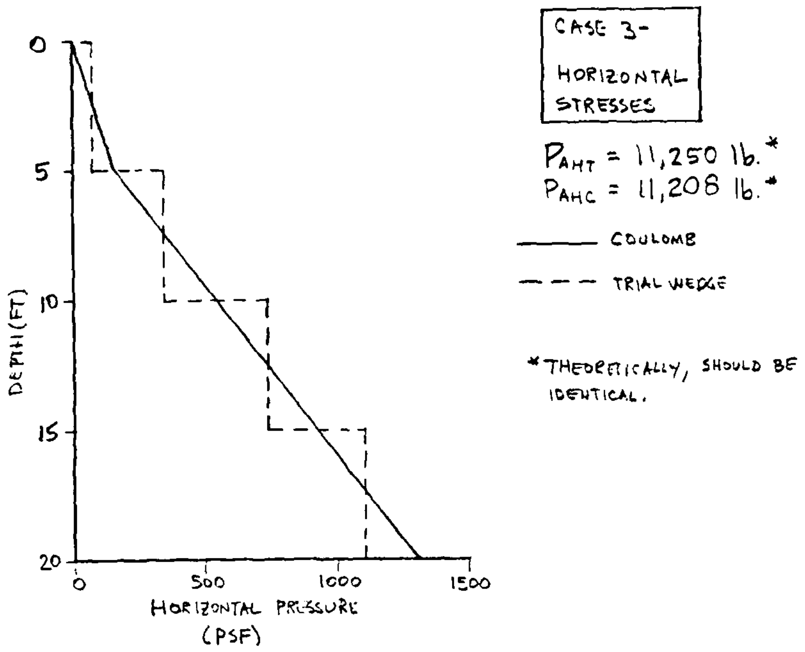
CASE 2 -

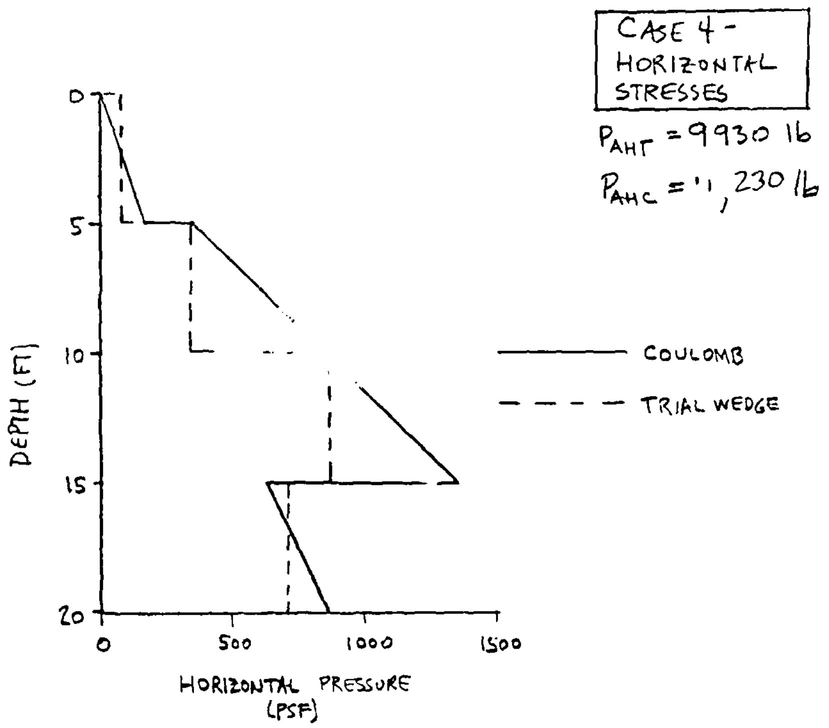
HORIZONTAL STRESSES

P_{AHT} = 6625 lb *
P_{AHC} = 6643 lb *

——— COULOMB
– – – TRIAL WEDGE

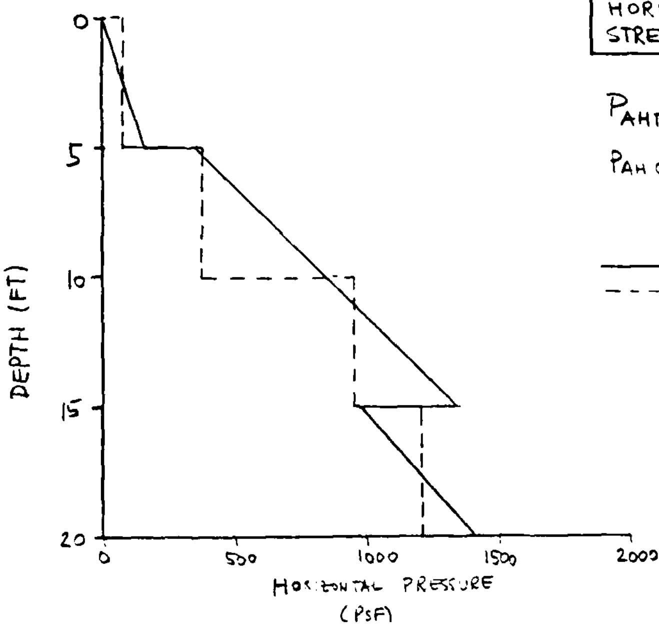
* THEORETICALLY, THESE FORCES SHOULD NOT BE EQUAL.


CASE 3-
HORIZONTAL
STRESSES
P_{AHT} = 11,250 lb.*
P_{AHC} = 11,208 lb.*
COULOMB
TRIAL WEDGE
*THEORETICALLY, SHOULD BE IDENTICAL.
DEPTH (FT)
0
5
10
15
20
0
500
1000
1500
HORIZONTAL PRESSURE (PSF)


CASE 4 -
HORIZONTAL
STRESSES
P_{AHT} = 9930 lb
P_{AHC} = 11,230 lb
COULOMB
TRIAL WEDGE
DEPTH (FT)
0
5
10
15
20
0
500
1000
1500
HORIZONTAL PRESSURE (PSF)

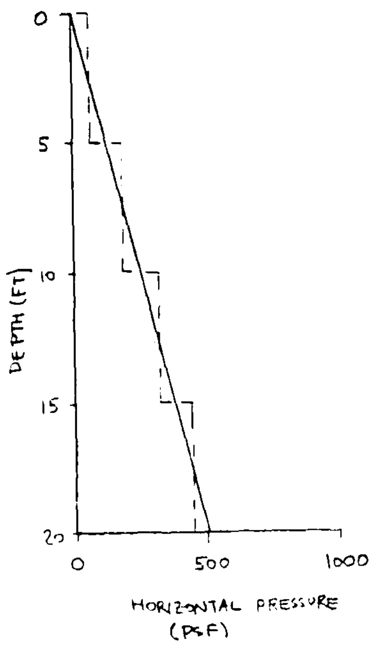
CASE 5 –
HORIZONTAL
STRESSES

P_{AHT} = 12,470 lb.
P_{AHC} = 13, 863 lb.

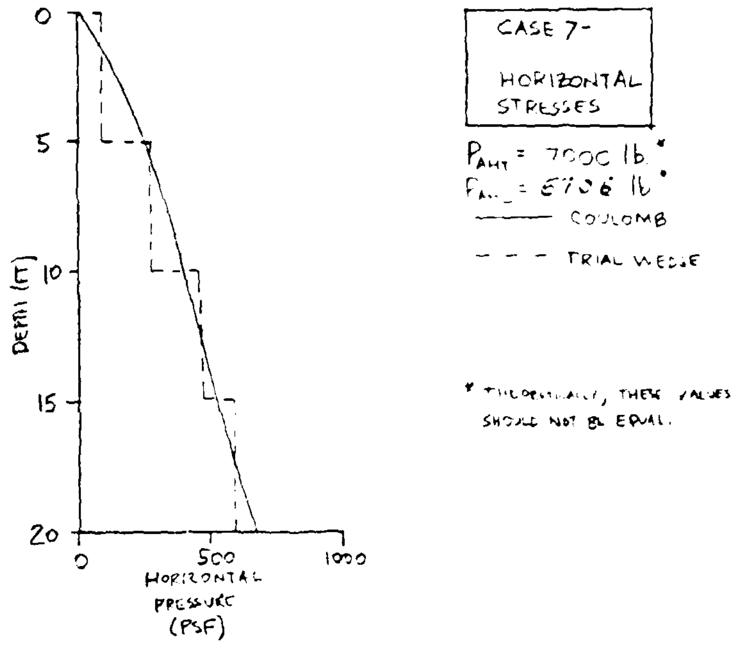


CASE 6 –
HORIZONTAL
STRESSES

P_{AHT} = 5120 lb*
P_{AHC} = 5100 lb*

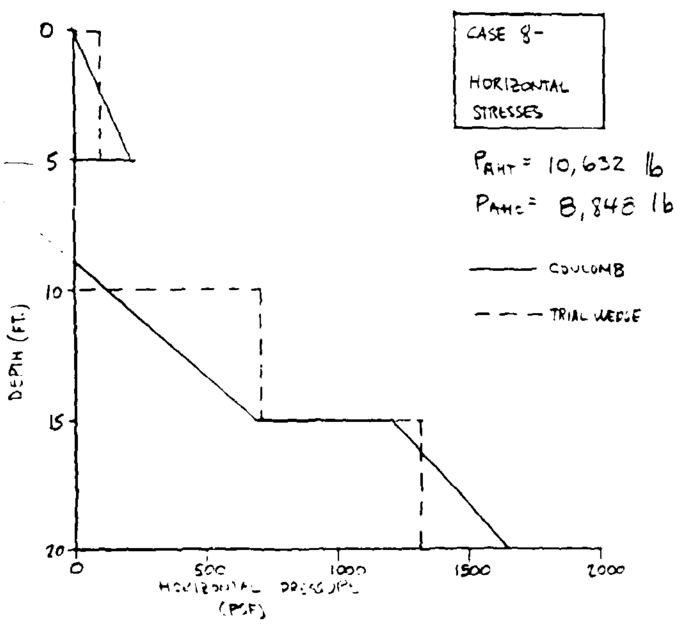


* DIFFERENCES DUE ONLY TO NUMERICAL ERRORS. THEORETICALLY, THESE VALUES ARE IDENTICAL.



P_{AHT} = 7000 lb.*
P_{AHC} = [illegible] lb*

* THEORETICALLY, THESE VALUES SHOULD NOT BE EQUAL.



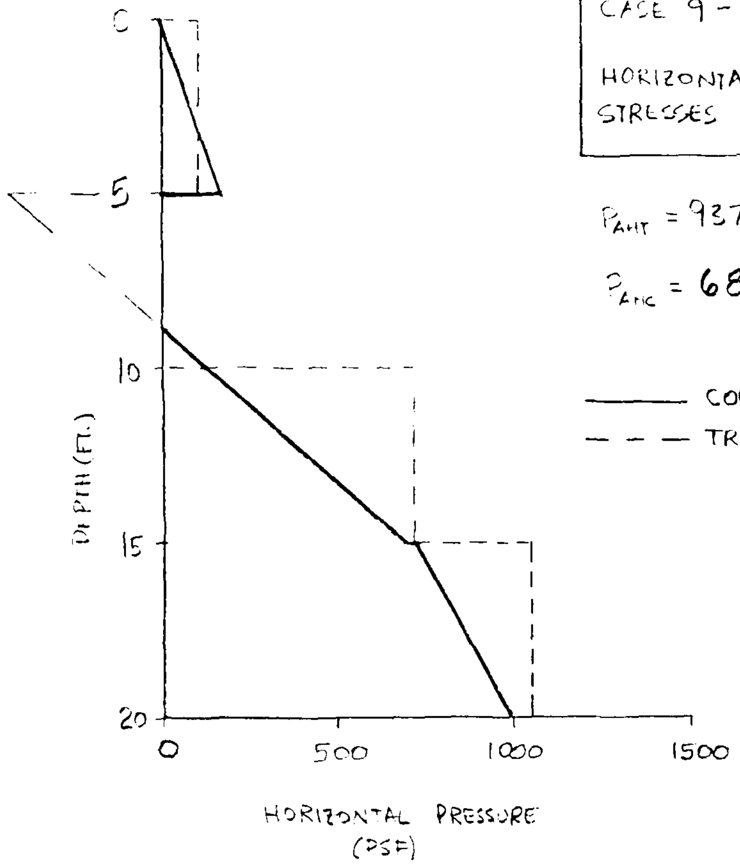
P_{AHT} = 10,632 lb
P_{AHC} = 8,848 lb

CASE 9 -

HORIZONTAL STRESSES

P_{AHT} = 9370 lb.

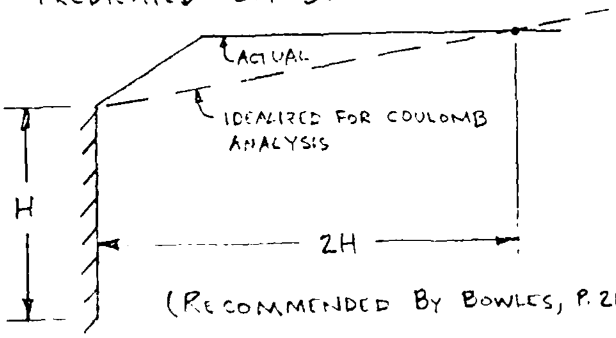
P_{AHC} = 6800 lb.



——— COULOMB *

— — — TRIAL WEDGE

* PREDICATED ON SURFACE SLOPE AS FOLLOWS:



(RECOMMENDED BY BOWLES, P. 285)

Addendum E

Calculation worksheets for Addendum D:

Cases 1, 2, 3

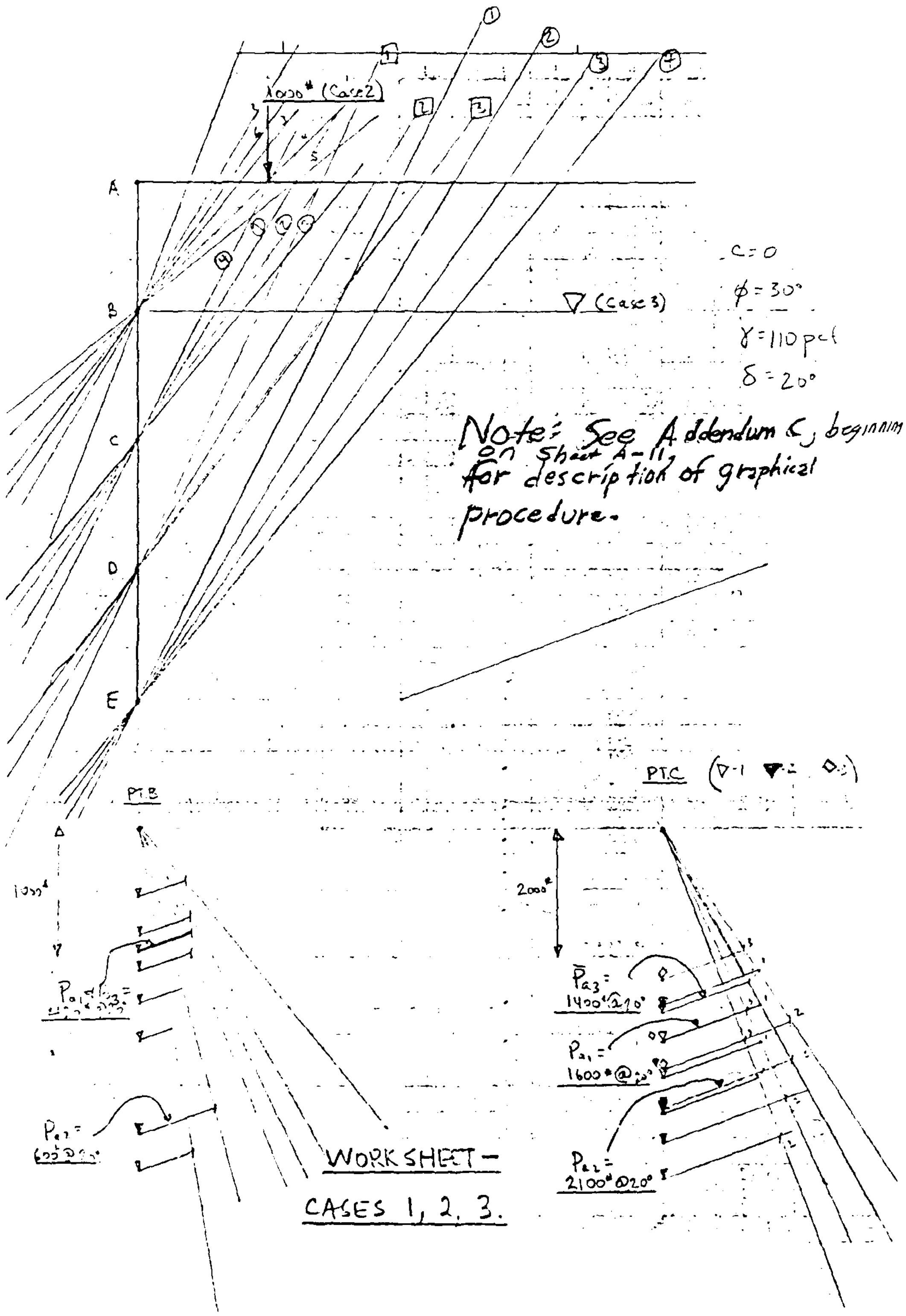
Cases 4, 5

Case 6

Case 7

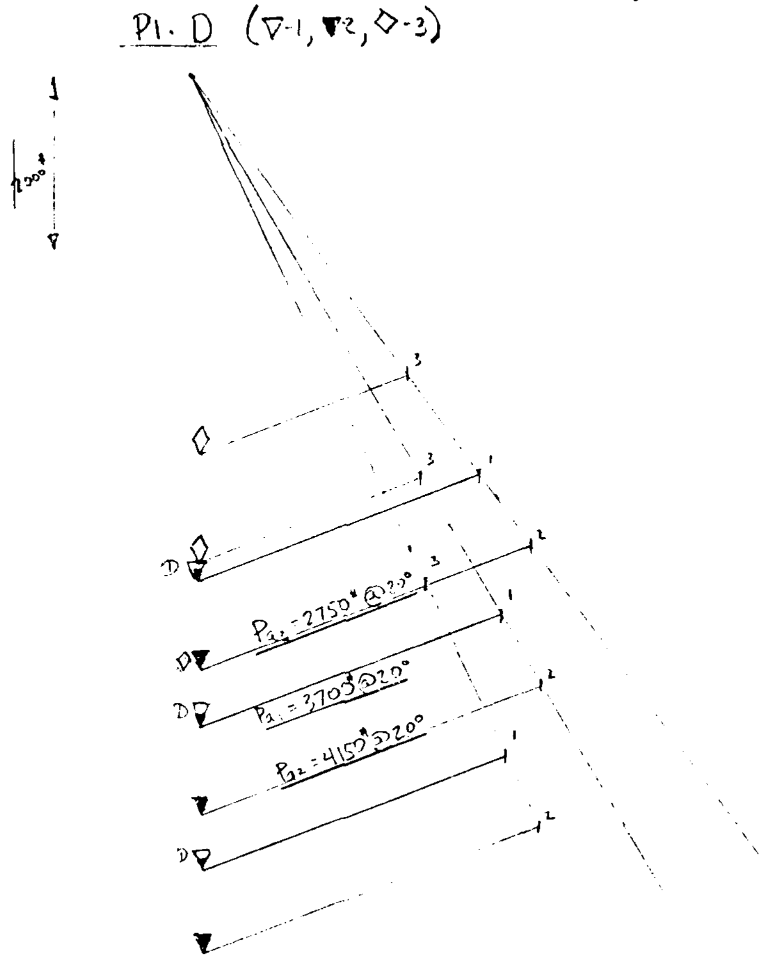
Case 8

Case 9



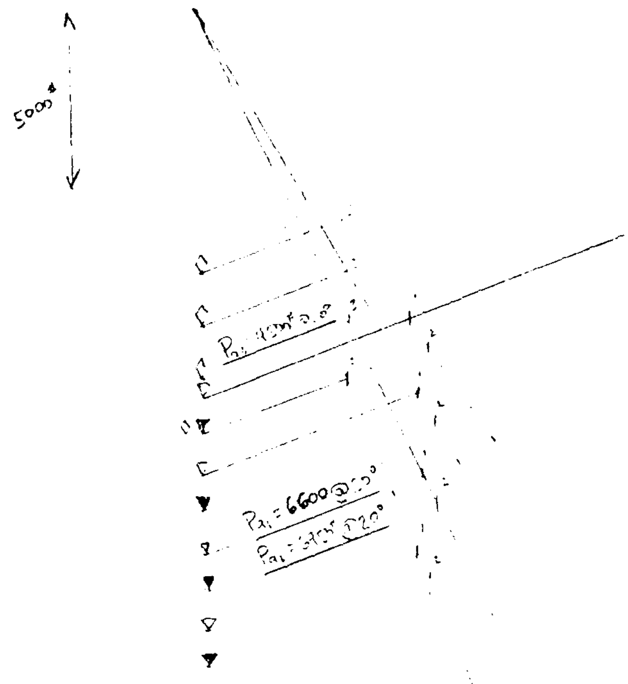
1000# (Case 2)
A
B
C
D
E
(Case 3)
c = 0
φ = 30°
γ = 110 pcf
δ = 20°
Note: See Addendum C, beginning on Sheet A-11, for description of graphical procedure.
PT. B
PT. C
1000#
2000#
Pa3 = 1400# @ 20°
Pa1 = 1600# @ 20°
Pa2 = 2100# @ 20°
WORKSHEET — CASES 1, 2, 3.

CASES 1, 2, 3
(CONT'D)



CASES 1, 2, 3

PTE (▽-1, ▼-2, ◇-3) (CONT'D)



TRIAL WEDGE
WEIGHTS FOR
CASE 1 & 2 & 3

			C1 + 3	C2	
PT B	Ray 1: W = 2 x 5 x 110 x .5 = 550		550	550	
	Ray 2: W = 3 x 5 x 110 x .5 = 825		825	825	
	Ray 3: W = 4 x 5 x 110 x .5 = 1100		1100	1100	
	Ray 4: W = 5 x 5 x 110 x .5 (+1000) = 2375		1375	2375	
	Ray 5: W = 6 x 5 x 110 x .5 (+1000) 2650		1650	2650	
	Ray 6: W = 3.5 x 5 x 110 x .5 (bel ; 3)		963	963	

		C1	C2	C3
PT C:	Ray 1: W = 10 (6) (110) (.5) (-62.4 (3) (5) (.5))[3] (+1000)[2]	3300	4300	2832
	Ray 2: W = 10 (7) (110) (.5) (-62.4 (3.5) (5) (.5))[3] (+1000)[2]	3850	4850	3304
	Ray 3: W = 10 (8) (110) (.5) (-62.4 (4) (5) (.5))[3] (+1000)[2]	4400	5400	3776
	Ray 4: W = 10 (5) (110) (.5) (-62.4) (2.5) (5) (.5))[3] (+1000)[2]	2750	3750	2310

CASES 1, 2, 3 (cont'd)

		C1	C2	C3
PT. D	Ray 1: W = 15 (7) (110) (.5)	5775	6775	4307
	[−62.4 (10) (4.7) (.5)][2]			
	[+1000][3]			
	Ray 2: W: 15 (9) (110) (.5)	7425	8425	5553
	[−62.4 (10) (6) (.5)][2]			
	[+1000][3]			
	Ray 3: W = 15 (11) (110) (.5)	9075	10,075	6797
	[−62.4 (7.3) (.5) (10)][2]			
	[+1000][3]			
PT. E	Ray 1: W = 20 (10) (110) (.5)	11,000	12,000	7420
	[−62.4 (15) (7.5) (.5)][2]			
	[+1000][3]			
	Ray 2: W = 20 (12) (110) (.5)	13,200	14,200	8958
	[−62.4 (15) (9) (.5)][2]			
	[+1000][3]			
	Ray 3: W = 20 (14) (110) (.5)	15,400	16,400	10,486
	[−62.4 (15) (10.5) (.5)][2]			
	[+1000][3]			
	Ray 4: W = 20 (16) (110) (.5)	17,600	18,600	11,948
	[−62.4 (15) (12) (.5)][2]			
	[+1000][3]			

CALCULATIONS OF HORIZONTAL PRESSURES
IN STEP INCREMENTS – TRIAL WEDGE; CASES 1, 2, 3

CENTER OF STEP	ΔP_{AT}			ΔP_{AHT} ($= \Delta P_{AT} \cos 20°$)			σ_h (psf) ($= \Delta P_{AHT} / 5'$)			
	C1	C2	C3*	C1	C2	C3*	C1	C2	C3*	C3**
2.5'	420	600	420	395	564	395	79	113	79	79
7.5'	1180	1500	980	1109	1410	921	222	282	184	340
12.5'	2100	2150	1350	1973	2020	1269	397	404	254	722
17.5'	2900	2800	1750	2725	2631	1644	545	526	329	1109

* DOES NOT CONTAIN HYDROSTATIC WATER EFFECT.

** INCLUDING " " "

WATER PRESSURES FOR CASE 3:

7.5' : $62.4\,(5)^2(0.5) = 780^{\#} \Rightarrow 156$ psf (step value for 5'-10')

12.5' : $62.4\,(10)^2(0.5) - 780 = 2340^{\#} \Rightarrow 468$ psf

17.5' : $62.4\,(15)^2(0.5) - 2340 - 780 = 3900^{\#} \Rightarrow 780$ psf

TOTAL TRIAL WEDGE FORCES FROM Σ of $\int$ OF PRESSURES
(MAY ALSO BE OBTAINED DIRECTLY FROM LAST (PTE) WEDGE FORCE)

HORIZ. COMPONENTS:

P_{AHT}

C1	C2	C3
79×5 + 222×5 + 397×5 + 545×5		
6215 lb	6625 lb	11,250 lb.

COULOMB PRESSURE DISTRIBUTIONS

CASE 1

at 20': $K_a = 0.297$

$p_N = 110(20)(0.297) = 653$ psf

$p_n = 653 \cos 20° = \underline{614 \text{ psf}}$ (20')

$\underline{P_{AHC} = 5140}$ #

CASE 2: same pressure distribution as Case 1 but add line load effect using Boussinesq chart (Fig 3.8b - Ret. Wall Manual)

Calculate Pressures at 2.5', 5', 7.5', 10', 12.5', 15', 18.125'

$\underline{m = 0.25}$ ← CONSTANT FOR THIS CASE

2.5': $n = 0.125$ $\sigma_h = \left(\frac{200}{20}\right) \frac{0.125}{(0.16+0.125^2)^2} = \frac{1.25}{(0.16+0.125^2)^2} = \underline{40.5 \text{ psf}}$

5': $n = 0.25$ $\sigma_h = \frac{2.5}{(0.16+.25^2)^2} = \underline{50.5 \text{ psf}}$

7.5': $n = 0.375$ $\sigma_h = \frac{3.75}{(0.16+0.375^2)^2} = \underline{41.5 \text{ psf}}$

10': $n = 0.5$ $\sigma_h = \frac{5}{(0.16+.5^2)^2} = \underline{29.7 \text{ psf}}$

12.5': $n = 0.625$ $\sigma_h = \frac{6.25}{(0.16+.625^2)^2} = \underline{20.6 \text{ psf}}$

15': $n = 0.75$ $\sigma_h = \frac{7.5}{(0.16+.75^2)^2} = \underline{14.4 \text{ psf}}$

18.125': $n = 0.906$ $\sigma_h = \frac{9.06}{(0.16+.906^2)^2} = \underline{9.4 \text{ psf}}$

$\underline{P_{AHC}} = 5140 + 40.5(3.75)(.5) + (50.5 + 41.5 + 29.7 + 20.6 + 14.4)12.5$
$+ 9.4(3.75) = \underline{5643 \text{ lb.}}$

COULOMB PRESSURE
DISTRIBUTIONS

CASE 3

at 5': $K_a = 0.297$

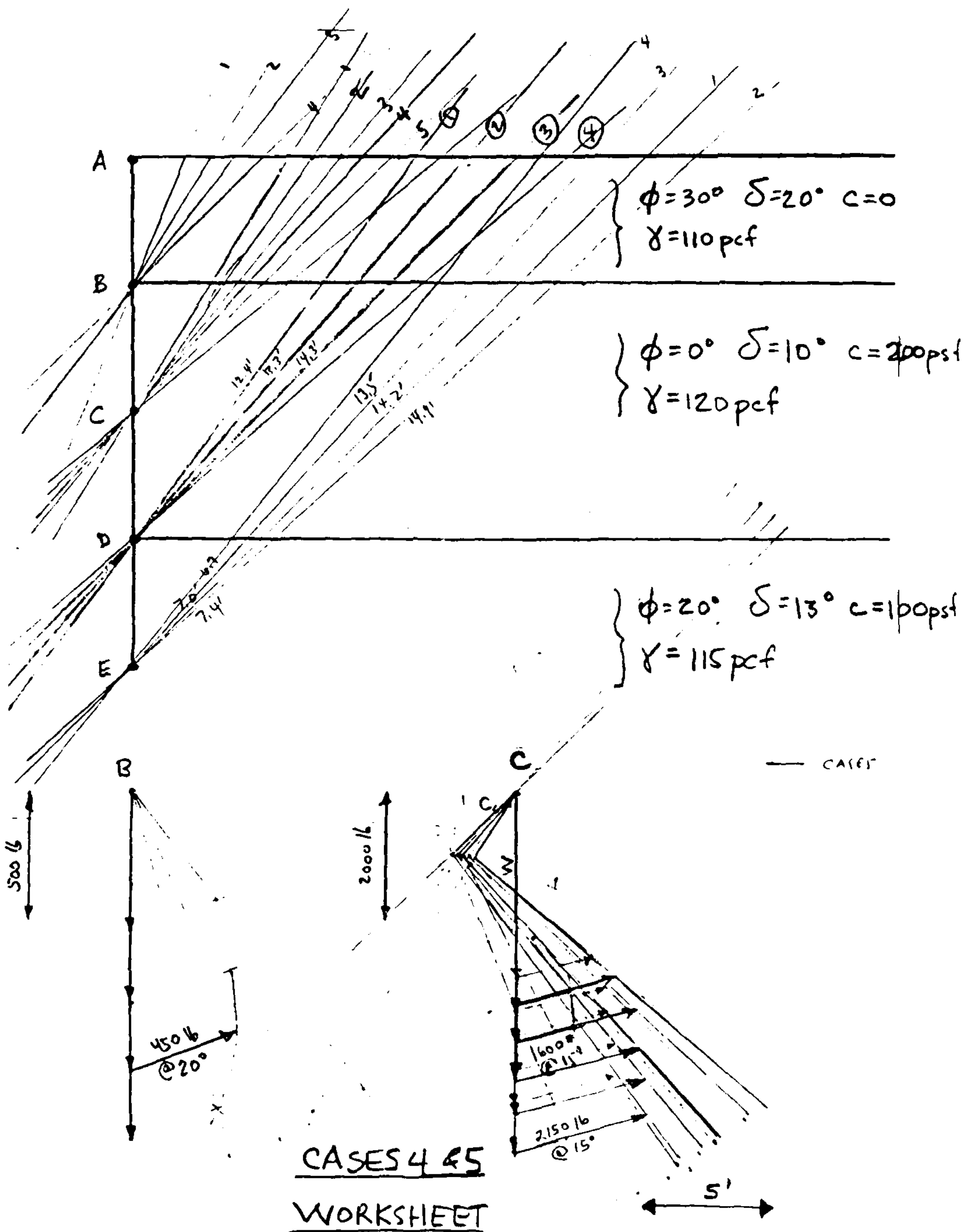
$\underline{p_h} = 110(5)(0.297)\cos 20° = \underline{154\ psf}$

at 20': $\bar{p}_h = [110(5) + 47.6(15)](0.297)\cos 20° = 353\ psf$

$u = 62.4(15) = 936$

$\underline{p_h = 1289\ psf}$

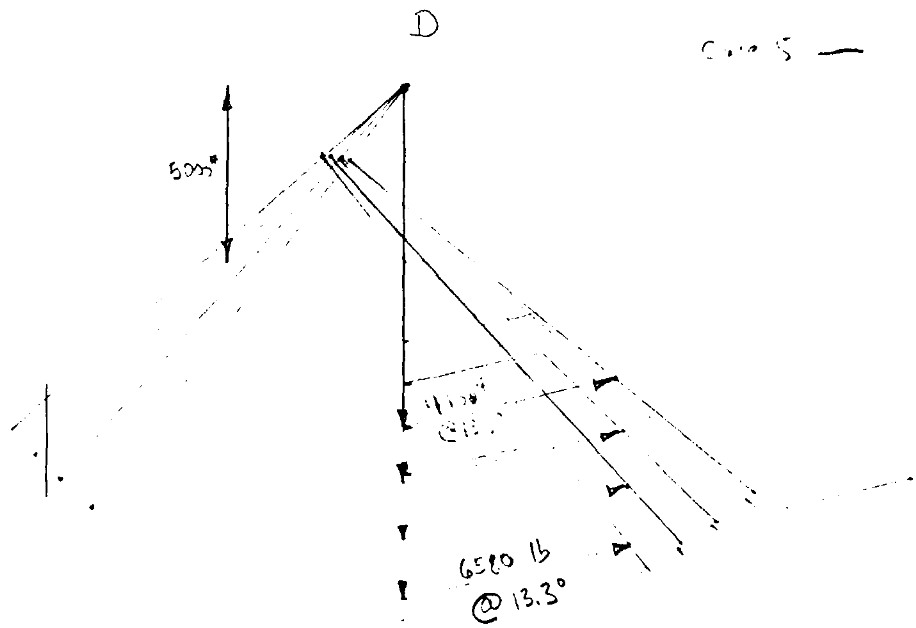
$\underline{\underline{P_{AHC}}} = 154(5)(.5) + (154 + 1289)(15)(.5) = \underline{\underline{11{,}208\ lb}}$

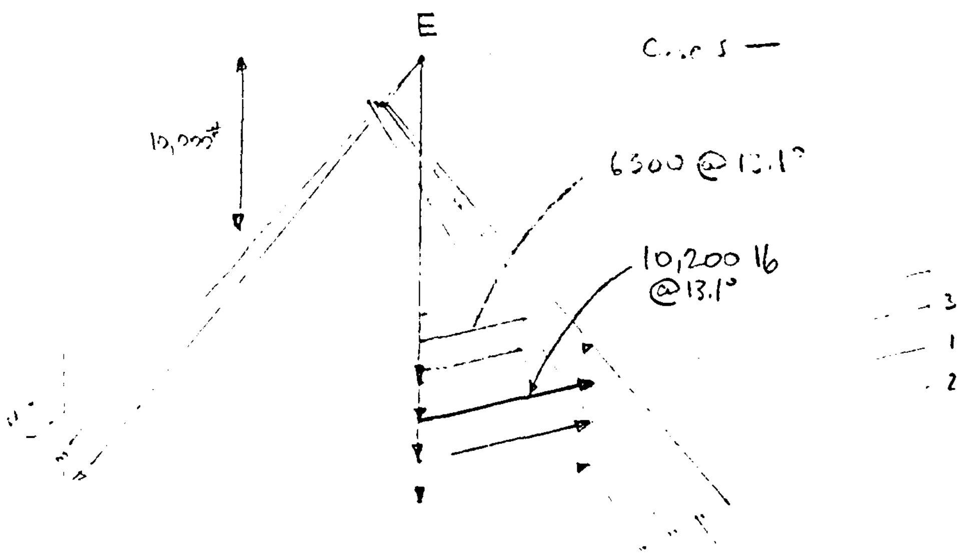


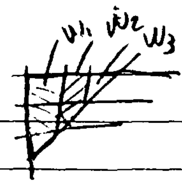
CASES 4 & 5

WORKSHEET

CASE 4







CASE 4

(no values for PTB)

PTC

USE WEIGHTING.

Ray 1

$W_1 = 2550$

$W_2 = 825$

$W = 3375$

$$\phi = \frac{2550(0^\circ) + 825(30)}{3375} = 7.33^\circ$$

$$\delta = \delta_{avg} = 15^\circ$$

Ray 2

$W_1 = 2975$ $\qquad \phi = 7.33^\circ$

$W_2 = 963$ $\qquad \delta = 15^\circ$

$W = 3938$

Ray 4

$W_1 = 3825$ $\qquad \phi = 7.33^\circ$

$W_2 = 1228$ $\qquad \delta = 15^\circ$

$W = 5062$

Ray 3

$W_1 = 3400$ $\qquad \phi = 7.33^\circ$

$W_2 = 1100$ $\qquad \delta = 15^\circ$

$W = 4500$

Ray 5

$W_1 = 4250$ $\qquad \phi = 7.33^\circ$

$W_2 = 1375$ $\qquad \delta = 15^\circ$

$W = 5625$

PT. D. Case 4

Ray 1

$W_1 = 7.5 \times 10 \times 120 \times 0.5 + 7.5 \times 5 \times 110 = 8625$

$W_2 = 3.5 \times 5 \times 110 \times 0.5 \qquad = 963$

$W = 9588$

$\phi = \frac{8625(0) + 963(30)}{9588} = 3.0°$

$\delta = 2(10) + 1(20) / 3 = 13.3°$

Ray 2

$W_1 = 8.7 \times 10 \times 120 \times 0.5 + 8.7 \times 5 \times 110 = 10,005$

$W_2 = 4.2 \times 5 \times 110 \times 0.5 \qquad = 1155$

$11,160$

$\phi = \frac{1155}{11160}(30) = 3.0°$

$\delta = 13.3°$

Ray 3

$W_1 = 10 \times 10 \times 120 \times 0.5 + 10 \times 5 \times 110 = 11,500$

$W_2 = 5 \times 5 \times 110 \times .5 \qquad = 1375$

$W = 12875$

$\phi = 3.2°$

$\delta = 13.3°$

Ray 4

$W_1 = 11.3 \times 10 \times 120 \times 0.5 + 11.3 \times 5 \times 110 = 12,995$

$W_2 = 5.7 \times 5 \times 110 \times 0.5 \qquad = 1568$

$14,563$

$\phi = 3.2°$

$\delta = 13.3°$

A-60

Point I Case 4

Bay 1

$W_1 = 5 \times 5 \times 0.5 \times 115 + 5 \times 10 \times 120 + 5 \times 5 \times 110 = 10,188$

$W_2 = 10 \times 10 \times .5 \times 120 + 10 \times 5 \times 110 = 11,500$

$W_3 = 5 \times 5 \times .5 \times 110 = 1375$

$W = 23,063$

$\phi = \frac{1375(30) + 10,188(20)}{23,063} = 10.6°$

$\delta = 1(20) + 2(10) + 1(13) = 13.1°$

$s \quad ... + 1+2(200) = 3540 lb$

2

$W_1 = 5.5 \times 5 \times 115 \times 0.5 + 5.5 \times 10 \times 120 + 5.5 \times 5 \times 110 = 11,206$

$W_2 = 11 \times 10 \times .5 \times 120 + 11 \times 5 \times 110 = 12,650$

$W_3 = 5.5 \times 5 \times 110(.5) = 1513$

$W = 25,370$

$\phi = \frac{11,206(20) + 1513(20)}{25,370} = 10.6°$

$\delta = 13.1°$

$s = 3770 lb$

Part E - Case U

Reg 3 (STEEPER THAN REG 1)

$W_1 = 4.5 \times 5 \times 115 \times 0.5 + 4.5(10)(120) + 4.5(5)(110) = 9169$

$W_2 = 9 \times 10 (120)(0.5) + 9(5)(110) = 10350$

$W_3 = 4.5(5)(110)(.5) = 1238$

$W = 20{,}757$

$$\phi = \frac{9169(20) + 1238(30)}{20{,}757} = 10.6^\circ$$

$\delta = 13.1^\circ$

$C_s = 3370$

Reg 4 (STEEPER THAN 1:43)

$W_1 = 4 \times 5 \times 115 \times 0.5 + 4(10)(120) + 4(5)(110) = 8150$

$W_2 = 8 \times 10 \times 120 \times .5 + 8(5)(110) = 9200$

$W_3 = 4 \times 5 \times 110 \times .5 = 1100$

$18{,}450$

$\phi = 10.6^\circ \quad \delta = 13.1^\circ \quad C_s = 3200$

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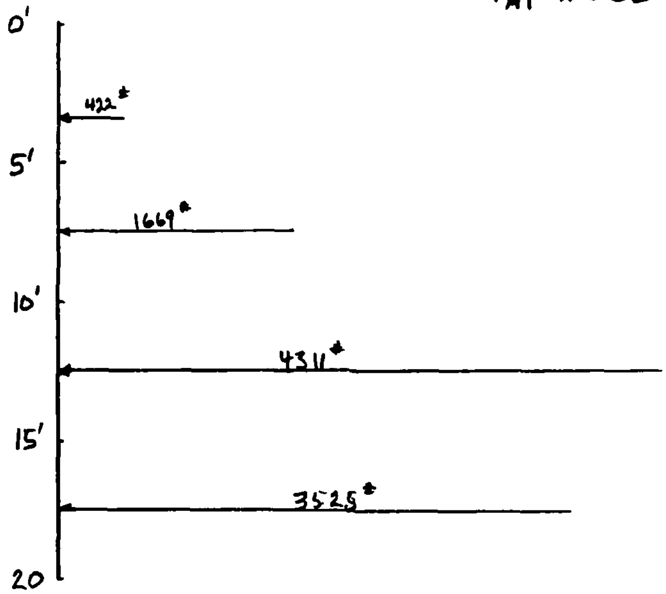
CASE 4 (HORIZONTAL FORCES - TW)

$P_{AT} \times \cos \delta$.

for point under consid



$P_{AHT} = 9930$ lb

CASE 4 —
HORIZONTAL
STRESSES

COULOMB PRESSURES
CASE 4

$\underline{0'} = 0$

$\underline{5'}$ (sand): $\delta = 20°$
$\phi = 30°$ $\quad K_a = 0.297$
$\gamma = 110$ pcf

$p_N = 5(110)(0.297) = 163$ psf $\overset{p_H = p_N \times}{\wedge} \cos 20° = \underline{154 \text{ psf}}$

$\underline{5'}$ (clay): $\delta = 10°$
$\phi = 0°$
$c = 200$ psf
$(\gamma = 120$ pcf$)$

$K_a = 1.015$
$p_N = \underset{110}{\gamma}(5) K_a - 2c\sqrt{K_a}$
$p_h = 357$ psf $\cos 10° = \underline{352 \text{ psf}}$

$\underline{15'}$ (clay): $K_a = 1.015$
$p_N = [5(110) + 10(120)](1.015) - 2(200)(\overset{1.007}{1.015})^{1/2}$
$= 1373$ psf
$p_H = 1373 \cos 10° = \underline{1352 \text{ psf}}$

$\underline{15'}$ (Clay-sand) $K_a = 0.44$
$p_N = (1750)(0.44) - 200(0.44)^{1/2} = 637$ psf
$p_H = p_N \cos 13° = \underline{621 \text{ psf}}$

$\underline{20'}$: $p_N = [1750 + 5(115)](.44) - 200(.44)^{1/2} = 890$ psf
$p_H = \underline{868 \text{ psf}}$ A-64

CASE 5
WORKSHEET

NOTE: ONLY THE WEIGHTS ARE DIFFERENT FROM CASE 4.
∴ USE CASE 4 DIAGRAMS W/ MODIFIED WEIGHTS. (RED LINES)

PTS A+B – same as for Case 4

PT C:

Ray 1: $W_1 = 2550 - 62.4(5)(3)(.5) = 2082$
$W_2 = 825$
$W = 2907$

$\phi = \frac{825(30)}{2907} = 8.5°$
$\delta = \delta_{avg} = 15°$
all rays, PTC.

Ray 2: $W_1 = 2975 - 62.4(5)(3.5)(.5) = 2429$
$W_2 = 963$
$W = 3392$

Ray 3: $W_1 = 3400 - 62.4(5)(4)(.5) = 2776$
$W_2 = 1100$
$W = 3876$

Ray 4: $W_1 = 3825 - 62.4(4.5)(5)(.5) = 3123$
$W_2 = 1228$
$W = 4351$

Ray 5: $W_1 = 4250 - 62.4(25)(.5) = 3470$
$W_2 = 1375$
$W = 4845$

PTD - Case 5

Ray 1

$W_1 = 8625 - 7.5 \times 10 \times .5 \times 62.4 = 6285$

$W_2 = \underline{963}$

$W = 7284$

$$\left.\begin{array}{l} \phi = \dfrac{963(30)}{7284} = 3.97^\circ \approx 4^\circ \\ \delta = 13.3^\circ \end{array}\right\} \text{all PTD}$$

Ray 2

$W_1 = -8.7 \times 10 \times 62.4 \times .5 + 10{,}005 = 7291$

$W_2 = \underline{1155}$

$W = 8446$

Ray 3

$W_1 = 11{,}500 - 10 \times 10 \times 62.4 \times .5 = 8380$

$W_2 = \underline{1375}$

$W = 9755$

Ray 4

$W_1 = 12{,}995 - 11.3 \times 10 \times 62.4 \times .5 = 9469$

$W_2 = \underline{1568}$

$W = 11{,}037$

PT E

Ray 1: $W_1 = 10{,}188 - 25(.5)(62.4) - 50(62.4) = 6288$

$W_2 = -100 \times .5 \times 62.4 + 11{,}500 = 8380$

$W_3 = 1375$

$W = 16043$

$\phi = \dfrac{1775(30) + 6288(20)}{16043} = 10.4°$

$\delta = 13.1°$

on all PTE

Ray 2: $W_1 = 11{,}206 - 5.5 \times 5 \times .5 \times 62.4 - 5.5 \times 10 \times 62.4 = 6916^{\#}$

$W_2 = 12{,}650 - 11 \times 10 \times .5 \times 62.4 = 9218$

$W_3 = \quad = 1513$

$W = 17647$

Ray 3: $W_1 = 9169 - 4.5 \times 5 \times .5 \times 62.4 - 4.5 \times 10 \times 62.4 = 5659$

$W_2 = 10350 - 9 \times 10 \times .5 \times 62.4 = 7542$

$W_3 = \quad = 1238$

$W = 14439$

Case 5 Horiz. Forces + Stresses

2.5' : 422# $\quad \sigma_x = 84\,psf$

7.5' : $-450(\cos 20°) + 1600(\cos 15°) = -422 + 1545 = \underline{1123}$

$+ 62.4 \times 5^2 \times .5 = \underline{780}$

$\underline{P_{AHT}} = \underline{1903\#} \quad \underline{\sigma_x = 381\,psf}$

12.5' : $-1600 \cos 15° + 4100 \cos 13° = -1545 + 3995 = 2450$

$+ 62.4 \times 10^2 \times .5 - 780 = \underline{2340\#}$

$\underline{P_{AHT}} = \underline{4790} \quad \underline{\sigma_x = 958\,psf}$

17.5' := $-4100 \cos 13 + 6300 \cos 13 = -3995 + 6139 = 2144$

$+ 62.4 \times 15^2 \times .5 - 2340 - 780$ $\quad \underline{3900}$

$\underline{P_{AHT}} = \underline{6044}$

$\underline{\sigma_x = 1209\,psf}$

COULOMB PRESSURES
CASE 5

$\underline{5'}$ (sand) : $p_N = 5(110)\overset{0.297}{(K_a)} = 163\,psf$

$\underline{p_h} = p_N \cos 20° = \underline{154\,psf}$

$\underline{5'}$ (clay) : $p_N = 5(110)\overset{1.015}{(K_a)} - 2\overset{20}{c}\overset{1.015}{\sqrt{K_a}} = 155\,psf$

$\underline{p_h} = 155 \cos 10° = \underline{153\,psf}$

$\underline{15'}$ (CLAY) : $\bar{p}_N = [5(110) + 10\ (57.6)](1.015) - 2(200)(1.015)^{1/2}$

$= 740\,psf$

$$\begin{array}{r} 120 \\ -62.4 \\ \hline 57.6 \end{array}$$

$\bar{p}_h = 729\,psf \quad (\bar{p}_N \cos 10°)$

$u = 10(62.4) = 624\,psf$

$\underline{p_h} = \bar{p}_h + u = \underline{1353\,psf}$

$\underline{15'}$ (SAND-CLAY) : $\bar{p}_N = [5(110) + 10(57.6)](0.44) - 200(0.44)^{1/2} = 363\,psf$

$\bar{p}_h = 362 \cos 13° = 353\,psf$

$u = 624\,psf$

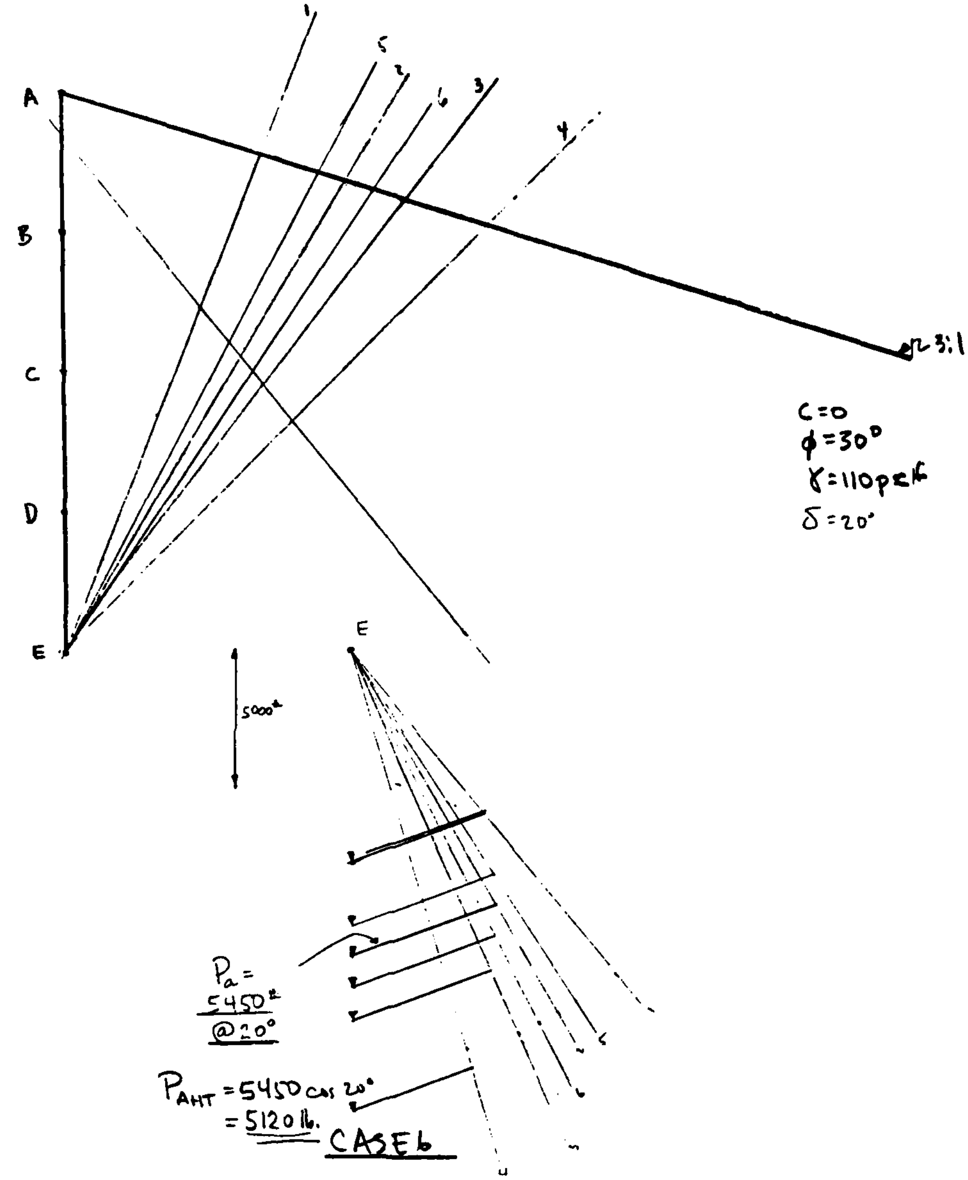
$\underline{p_h} = \bar{p}_h + u = \underline{977\,psf}$

$\underline{20'}$ (SAND-CLAY) : $\bar{p}_N = [5(110) + 10(57.6) + 5(52.6)](0.44) - 200(0.44)^{1/2} = 478\,psf$

$\bar{p}_h = 478 \cos 13° = 466\,psf$

$u = 15(62.4) = 936\,psf$

$\underline{p_h} = \bar{p}_h + u = \underline{1402\,psf}$



CASE 6

WORKSHEET

NOTE: COULOMB AND TRIAL WEDGE ARE MATHEMATICALLY IDENTICAL FOR THIS CASE. IT IS ONLY NECESSARY TO COMPUTE TOTAL TRIAL WEDGE FORCE FOR PURPOSES OF COMPARISON. PRESSURE DISTRIBUTIONS SHOWN AS STEP FUNCTIONS FOR CONSISTENCY W/ OTHER CASES.

CASE 6

PT E

Ray 1: $W = 20 \times 7 \times .5 \times 110 = 7700^{\#}$

Ray 2: $W = 20 \times 10 \times .5 \times 110 = 11{,}000^{\#}$

Ray 3: $W = 20 \times 12 \times .5 \times 110 = 13{,}200^{\#}$

Ray 4: $W = 20 \times 15 \times .5 \times 110 = 16{,}500^{\#}$

Ray 5: $W = 20 \times 9 \times .5 \times 110 = 9900^{\#}$

Ray 6: $W = 20 \times 11 \times .5 \times 110 = 12{,}100^{\#}$

COULOMB PRESSURES

CASE 6

$\beta = 18°$ (measured)

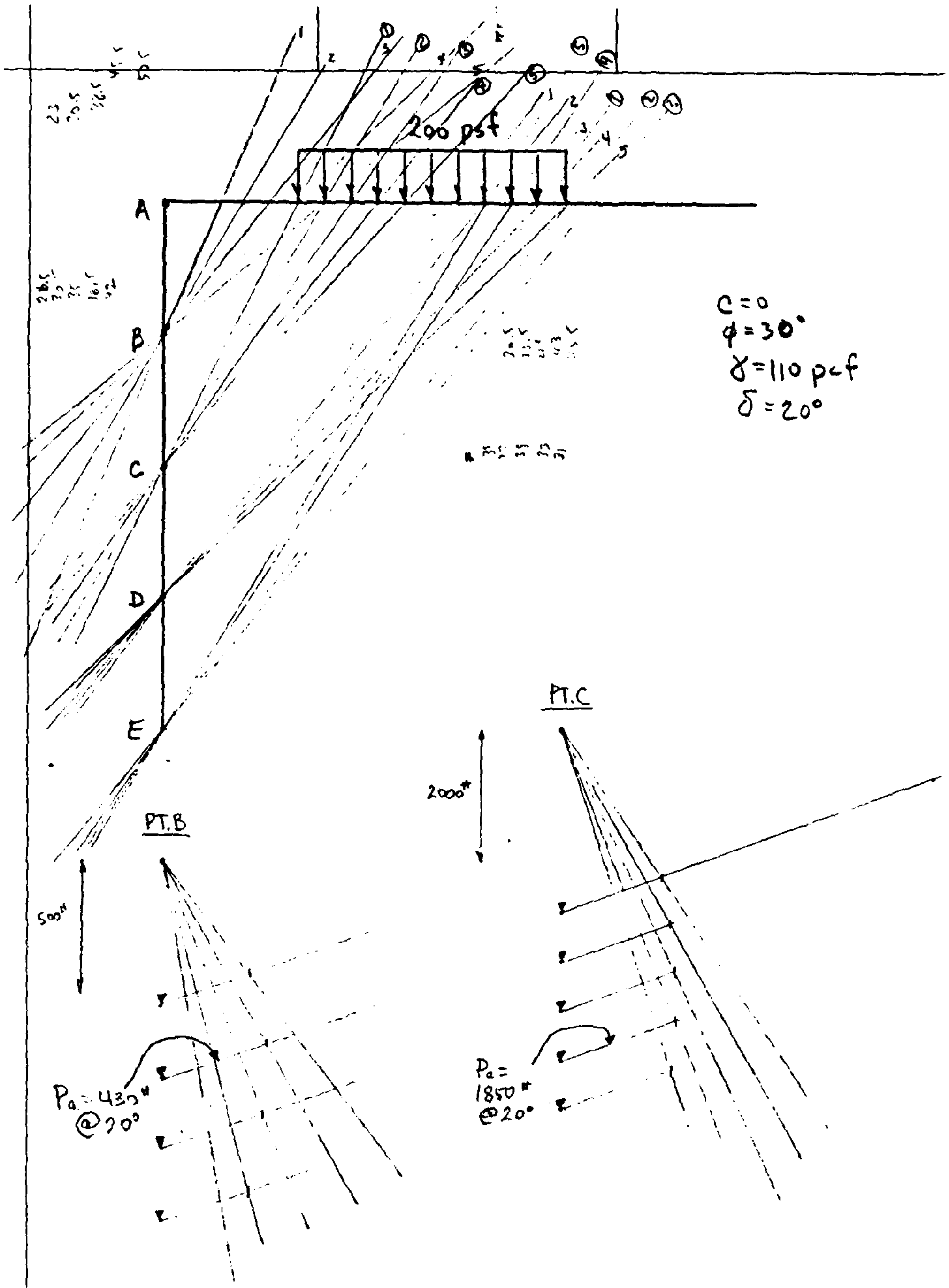
PT E:

$$K_a = \frac{\sin^2 120}{\sin^2(90)\sin(70)\left[1+\sqrt{\frac{\sin(50)\sin 48}{\sin 70 \sin 72°}}\right]^2} = 0.247$$

$p_N = 20(110)(0.247) = 543$ psf

$p_h = 543 \cos 20° = 510$ psf

$P_{AHC} = 510\ (20)(.5) = \underline{5100\ \text{lb.}} = P_{AHT}$

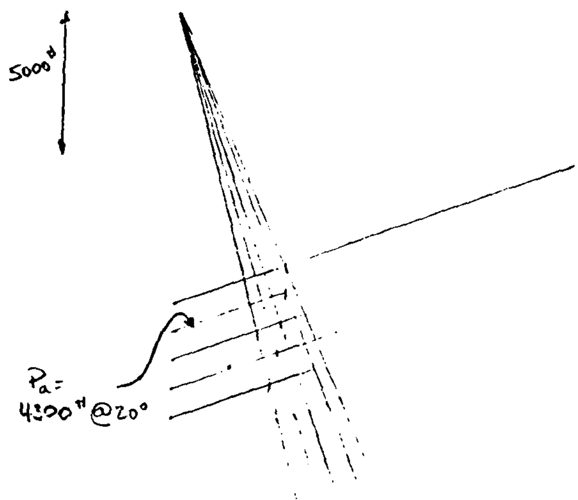


WORKSHEET

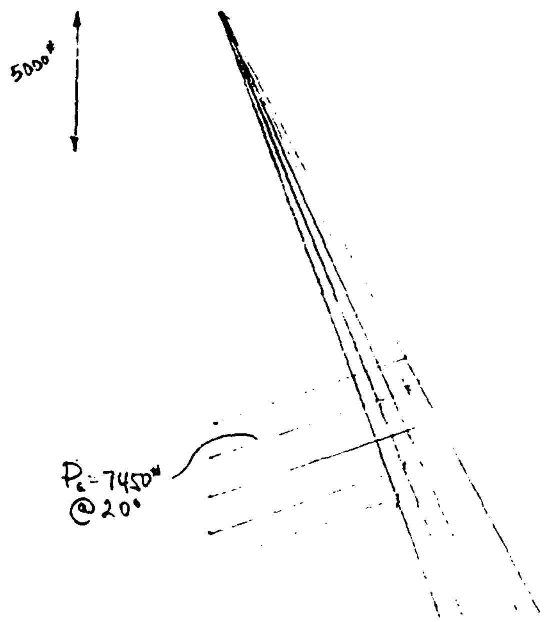
CASE 7

CASE 7 (CONT'D)

PT. D



PT. E



Weights
Case 7

PT B

Ray 1: W = 2 x 5 x 110 x .5 = 550

" 2: W = 3 x 5 x 110 x .5 = 825

" 3: W = 4 x 5 x 110 x .5 = 1100

" 4: W = 5 x 5 x 110 x .5 = 1375

" 5: W = 6 x 5 x 110 x .5 + 200 = 1850

PT C:

Ray 1: W = 10 x 5 x 110 x .5 = 2750

" 2: W = 10 x 6 x 110 x .5 + 200 = 3500

" 3: W = 10 x 7 x 110 x .5 + 400 = 4250

" 4: W = 10 x 8 x 110 x .5 + 600 = 5000

" 5: W = 10 x 9 x 110 x .5 + 800 = 5750

PT D:

Ray 1: W = 15 x 11 x 110 x .5 + 1200 = 10275

" 2: W = 15 x 12 x 110 x .5 + 1400 = 11300

" 3: W = 15 x 13 x 110 x .5 + 1600 = 12325

" 4: W = 15 x 14 x 110 x .5 + 1800 = 13350

" 5: W = 15 x 15 x 110 x .5 + 2000 = 14375

PT E:

Ray 1: W = 20 x 14 x 110 x .5 + 1600 = 17200

" 2: W = 20 x 15 x 110 x .5 + 2000 = 18500

" 3: W = 20 x 16 x 110 x .5 + 2000 = 19600

" 4: W = = 15900

CASE 7 - Trial Wedge horizontal forces and stresses

2.5': $P_{AH} = 430 \cos 20° =$ 404#

$p_h = 404/5 =$ 81 psf

7.5': $P_{AH} = 1850 \cos 20 - 404 =$ 1334#

$p_h = 1334/5$ 267 psf

12.5': $P_{AH} = 4300 \cos 20 - 404 - 1334 =$ 2303#

$p_h = 2303/5$ 461 psf

17.5': $P_{AH} = 7450 \cos 20 - 404 - 1334 - 2303$ 2960#

$p_h = 2960/5$ 592 psf

$P_{AHT} = 7001$#

COULOMB PRESSURES

$K_A = 0.297$

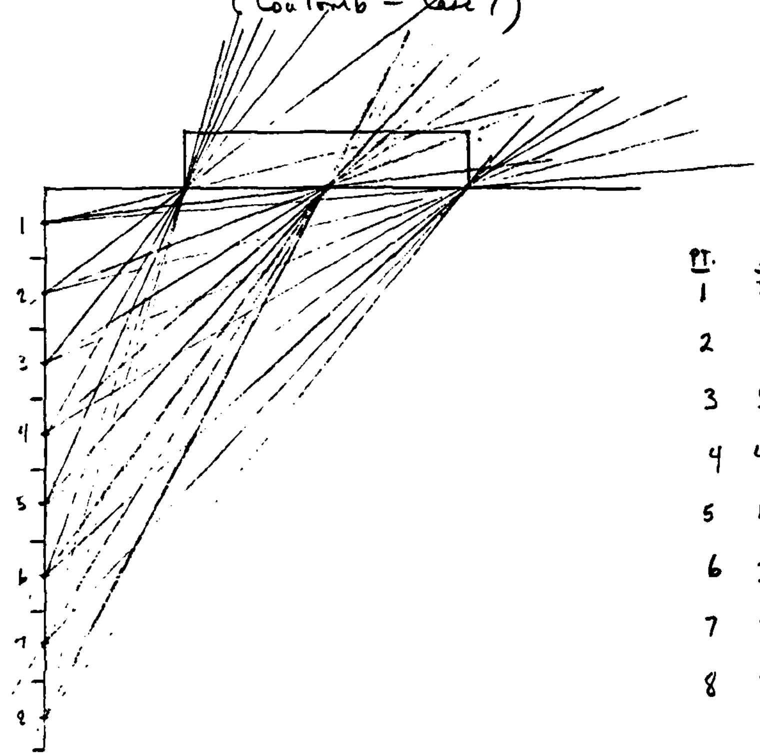
20': $p_h = 20(110)(0.297) \cos 20° = 614$ psf (linear)

added pressure due to surcharge - next page

$P_{AH} = 614(10) + 2.5(19.6 + 44.1 + 45.7 + 38.3 + 29.0 + 21.5 + 16.23 + 11.9)$

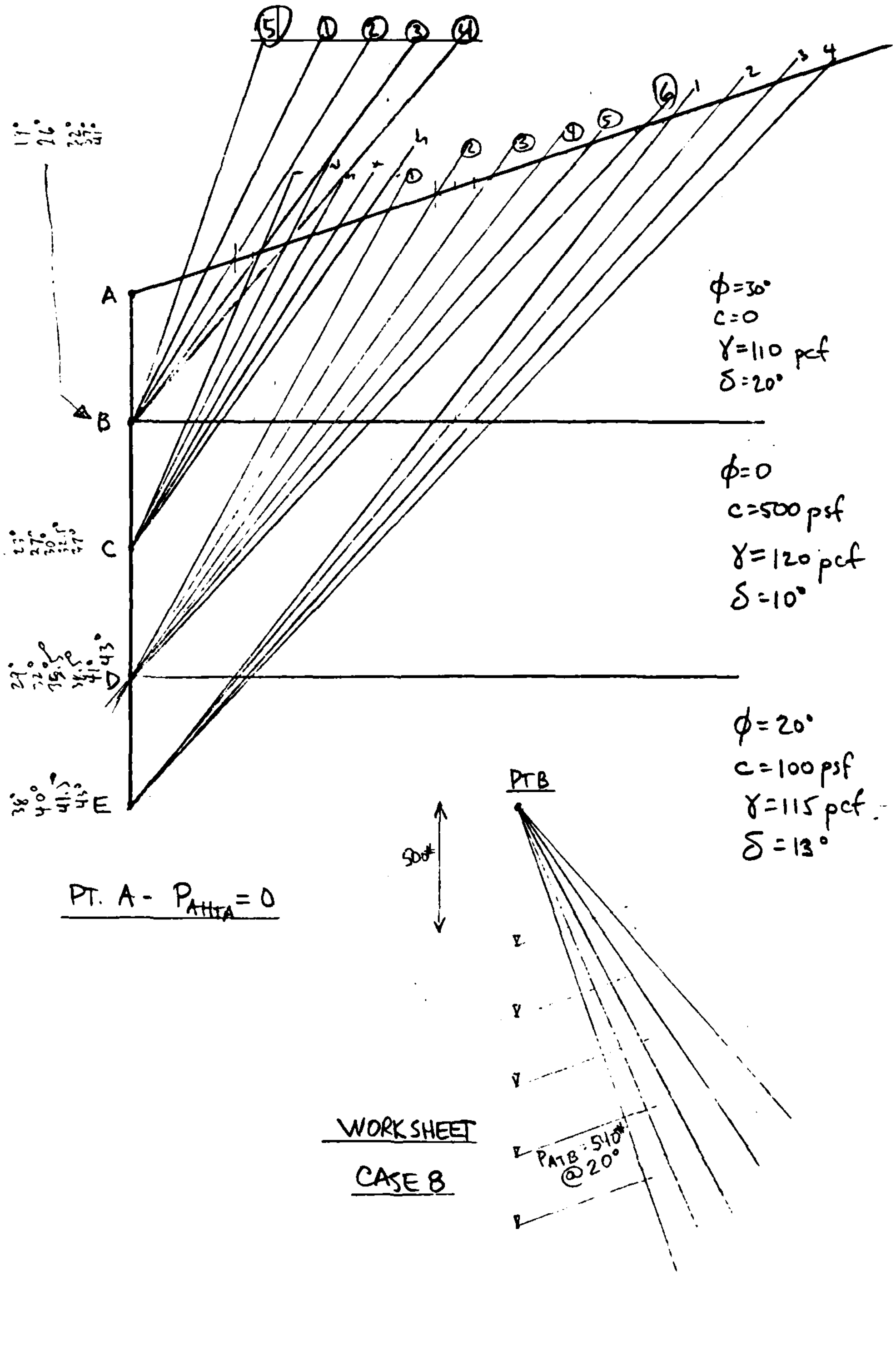
$= 6140 + 566 = 6706$#

Added Stresses due to Surcharge Loading
(Coulomb - Case 7)



PT.	α°	β°
1	83	9
2	69	23
3	58	29
4	49.5	30
5	42	29
6	36	27.5
7	31.5	26
8	28	23.5

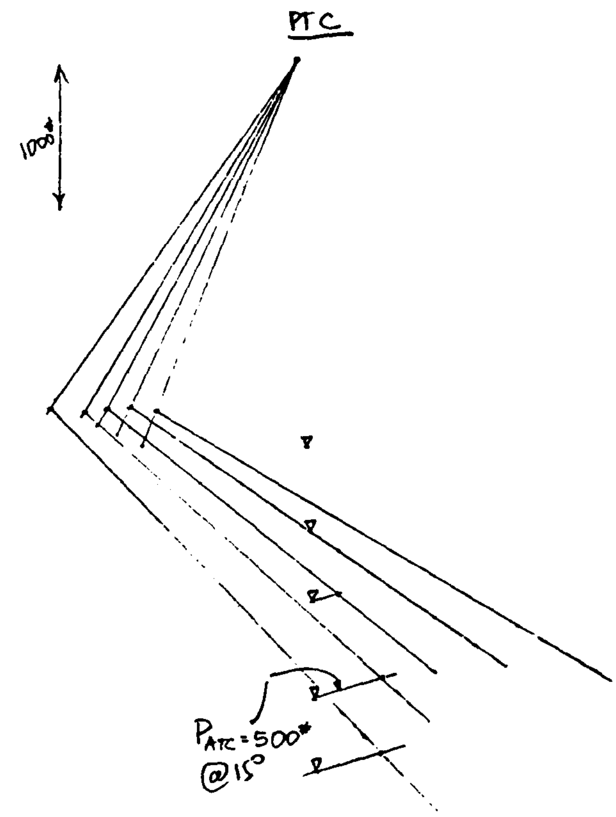
PT.	β (rad)	$\sin\beta$	$\sin^2\alpha$	$\cos^2\alpha$	$(\beta + \sin\beta \sin^2\alpha - \sin\beta \cos^2\alpha)$	$\Delta\sigma_h$ (psf)
1	0.157	0.156	0.985	0.0149	0.308	19.6
2	0.401	0.391	0.872	0.128	0.692	44.1
3	0.506	0.485	0.719	0.281	0.718	45.7
4	0.524	0.500	0.578	0.422	0.602	38.3
5	0.506	0.485	0.448	0.552	0.456	29.0
6	0.480	0.462	0.345	0.655	0.337	21.5
7	0.454	0.438	0.273	0.727	0.255	16.23
8	0.410	0.399	0.220	0.780	0.187	11.9



WORKSHEET

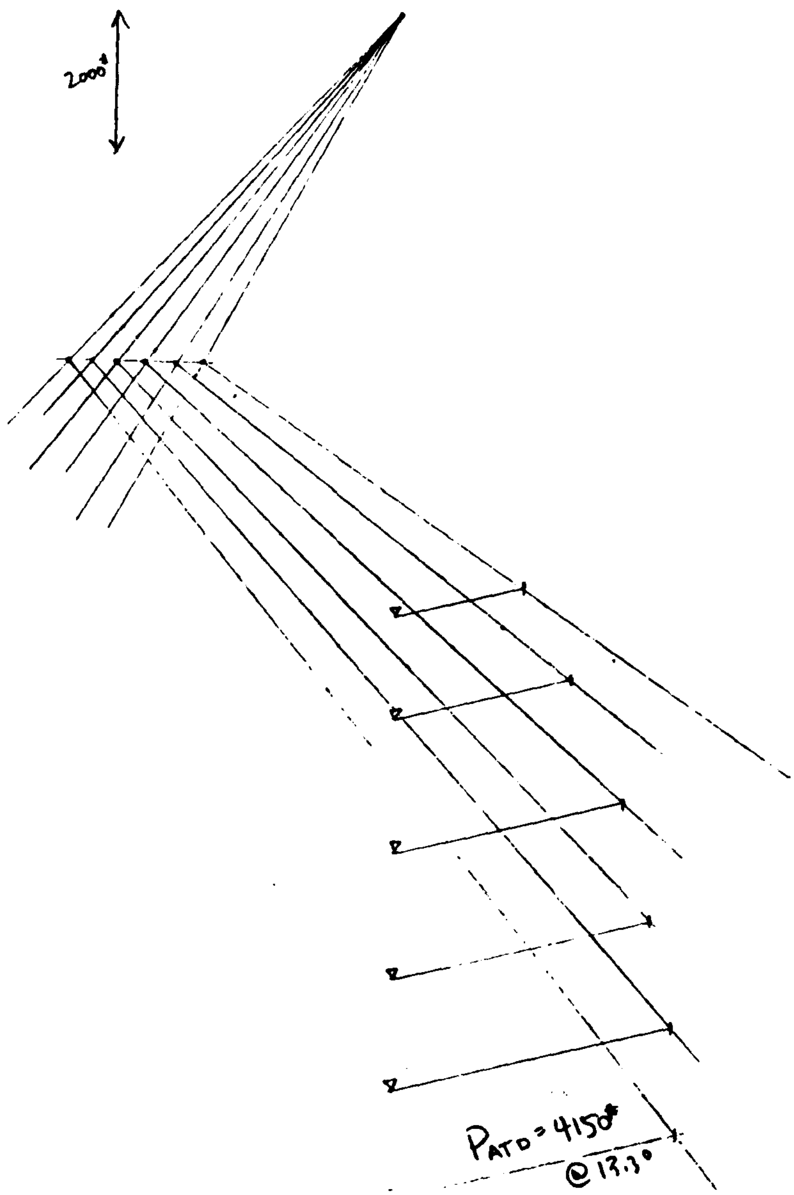
CASE 8

CASE 8 CONT'D



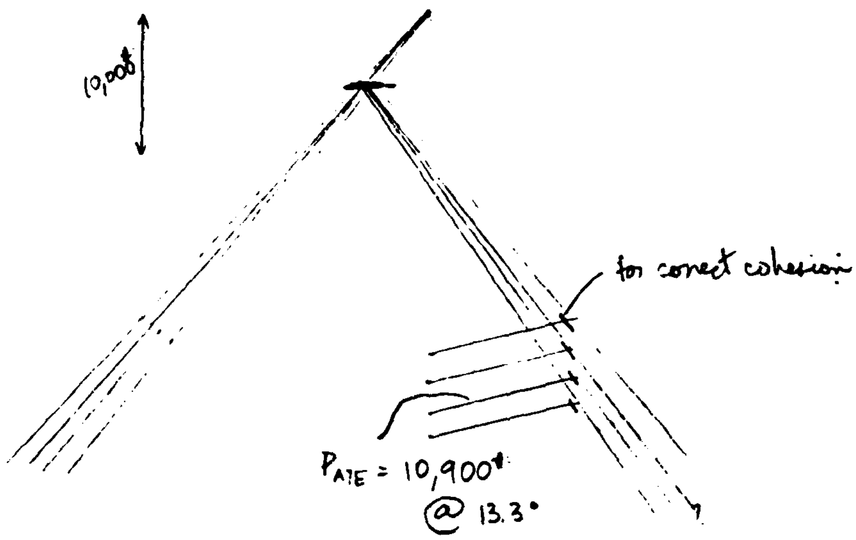
CASE 8 CON'T

PTD



CASE 8 (CONT'D)

PTE



HORIZ FORCES + STRESSES (TRIAL WEDGE).

0-5: $F_H = 507^{\#}$ $\quad p_h = 101$ psf

5-10: $F_H = 500 \cos 15 - 507 = \underline{0^{\#}}$ (483) $\quad p_h = 0$ psf

10-15: $F_H = 4150 \cos 13.3° - 483 = 4039 - 483 = \underline{3556^{\#}}$ $\quad p_h = 711$ psf

15-20: $F_H = 10,900 \cos 13.3 - 4039 = \underline{6569^{\#}}$ $\quad p_h = 1314$ psf

$$\underline{P_{AHT}} = 507 + 0 + 3556 + 6569 = \underline{\underline{10,632^{\#}}}$$

Case 8 - Weights

PT B:

Ray 1 : $(6 \times 3 - 1 \times 3)\,0.5 \times 110 = 5 \times 3 \times 0.5 \times 110 = 825^{\#}$

2 : $5 \times 4\ (.5) \times 110 = 1100$

3 : $5 \times 5\ (.5) \times 110 = 1375$

4 : $5 \times 6\ (.5) \times 110 = 1650$

5 : $5 \times 2 \times 110 \times .5 = 550^{\#}$

$\phi = 30°$

$\delta = 20°$

PTC. $\delta = 15°$

Ray 1: $W_1 = 5 \times 2.2 \times 0.5 \times 120$

$+ 5 \times 2.2 \times 110$

$+ 0.8 \times 2.2 \times 110 \times .5 = 1870$

$W_2 = 5.8 \times 2.8 \times 0.5 \times 110 = 893$

$W = 2763^{\#}$

$\phi = 9.7°$

Ray 2: $W_1 = 5 \times 2.5 \times 0.5 \times 120$

$+ 5 \times 2.5 \times 110$

$+ 0.85 \times 2.5 \times .5 \times 110 = 2279$

$W_2 = 5.85 \times 3.3 \times .5 \times 110 = 1062$

$W = 3341^{\#}$ $\phi = 9.5$

Ray 3: $W_1 = 5 \times 2.8 \times 0.5 \times 120$

$+ 5 \times 2.8 \times 110$

$+ 0.9 \times 2.8 \times .5 \times 110 = 2519$

$W_2 = 5.9 \times 4.1 \times .5 \times 110 = 1330$

$W = 3849$ $\phi = 10.$

Case 8 Weights (cont'd)

Pt C:

Ray 4: $W_1 = 5 \times 3.2 \times 0.5 \times 120$
$+ 5 \times 3.2 \times 110$
$+ 1.1 \times 3.2 \times .5 \times 110 = 2914$
$W_2 = 6.1 \times 4.9 \times .5 \times 110 = 1644$
$W = 4558$ $\phi = 11.0$

Ray 5: $W_1 = 5 \times 3.5 \times 0.5 \times 120$
$+ 5 \times 3.5 \times 110$
$+ 1.2 \times 3.5 \times 0.5 \times 110 = 3206$
$W_2 = 6.2 \times 5.5 \times 0.5 \times 110 = 1876$
$W = 5082$ $\phi = 11.1°$

$\phi = ?$ (check separately for each) (see above)

$\delta = 15°$

$C_1 = 5.4 \times 500 = 2700$#
$C_2 = 5.5 \times 500 = 2750$
$C_3 = 5.7 \times 500 = 2850$
$C_4 = 5.9 \times 500 = 2950$
$C_5 = 6.1 \times 500 = 3050$

PTD: $\boxed{\delta = 13.3^\circ}$ CASE 8 WGTS CONT.

Ray 1: $W_1 = 10 \times 5.5 \times 120 \times 0.5$
$+ 5 \times 5.5 \times 110$
$+ 1.8 \times 5.5 \times 110 \times 0.5 =$ 6870

C = 5700 $W_2 = 6.9 \times 4.5 \times 110 \times 0.5 =$ 1708
$W =$ 8578
$\phi = \frac{1708}{8578} 30 = \underline{6.0^\circ}$

Ray 2: $W_1 = 10 \times 6.2 \times 120 \times 0.5$
$+ 5 \times 6.2 \times 110$
$+ 2.1 \times 6.2 \times 110 \times 0.5 =$ 7846
$W_2 = 7.1 \times 5.6 \times 110 \times 0.5 =$ 2187
$W =$ 10,033

C = 5900 $\phi = \frac{2187}{10033} = \underline{6.5^\circ}$

Ray 3: $W_1 = 10 \times 7.2 \times 120 \times 0.5$
$+ 5 \times 7.2 \times 110$
$+ 2.4 \times 7.2 \times 110 \times 0.5 =$ 9230
$W_2 = 7.3 \times 6.8 \times 110 \times 0.5 =$ 2730
$W =$ 11,960

C = [illegible]150 $\phi = \frac{2730}{11960} 30 = \underline{6.8^\circ}$

Ray 4: $W_1 = 10 \times 8 \times 120 \times 0.5$
$+ 5 \times 8 \times 110$
$+ 2.7 \times 8 \times 110 \times 0.5 =$ 10388
$W_2 = 7.6 \times 8 \times 110 \times 0.5 =$ 3344
$W =$ 13,732

C = 6375 $\phi = \underline{7.3^\circ}$

Case 8 Wgts (Cont'd)

PT D (cont'd)

Ray 5: $W_1 = 8.6 \times 10 \times .5 \times 120$

$+ 8.6 \times 5 \times 110$

$+ 8.6 \times 2.9 \times 110 \times .5 = 11262$

$W_2 = 7.9 \times 9.3 \times 110 \times .5 = 4041$

C = 6600 $W = 15303$

$\phi = 7.9°$

Ray 6: $W_1 = 9.2 \times 10 \times .5 \times 120$

$+ 9.2 \times 5 \times 110$

$+ 9.2 \times 3 \times 110 \times .5 = 12098$

C = 6850 $W_2 = 8.1 \times .5 \times 10.7 \times 110 = 4767$

$W = 16865$

$\phi = 8.5°$

PT E $\boxed{\delta = 13.3°}$

Ray 1 $W_1 = 4 \times 5 \times 115 \times .5$

$+ 4 \times 10 \times 120$

$+ 4 \times 5 \times 110$

$+ 4 \times 1.3 \times 110 \times .5 = 8436$

C = ~~6700~~ 6950 $W_2 = 7.8 \times 10 \times 120 \times .5$

$+ 7.8 \times 6.3 \times 110$

all others no change in PT. E. $+ 7.8 \times 2.6 \times 110 \times .5 = 11201$

$W_3 = 8.9 \times 9.3 \times 110 \times .5 = 4552$

$W = 24189$

$\phi = \frac{8436(20) + 4552(30)}{24189} = 12.6°$

Ray 2 $W_1 = 4.2 \times 5 \times 115 \times .5$

$+ 4.2 \times 10 \times 120$

$+ 4.2 \times 5 \times 110$

$\phi = 12.8°$ $+ 4.2 \times 1.7 \times 110 \times .5 = 8858$

$C = 6550$ $W_2 = 8.3 \times 10 \times 120 \times .5 + 8.3 \times 6.4 \times 110$

$+ 8.3 \times 2.7 \times 110 \times 0.5 = 12056$

$W_3 = 9.1 \times 10.5 \times 110 \times .5 = 5255$

$W = 26169$

Ray 3 $W_1 = 4.5 \times 5 \times 115 \times .5$

$+ 4.5 \times 10 \times 120$

$+ 4.5 \times 5 \times 110$

$+ 4.5 \times 1.4 \times 110 \times .5 = 7515$

$\phi = 13.0°$ $W_2 = 8.8 \times 10 \times 120 \times .5$

$C = 6700$ $+ 8.8 \times 6.5 \times 110$

$+ 8.8 \times 2.8 \times 110 \times .5 = 12927$

$W_3 = 9.4 \times 11.7 \times 110 \times .5 = 6049$

$W = 28{,}491$

Ray 4 $W_1 = 4.8 \times 5 \times 115 \times .5$

$+ 4.8 \times 10 \times 120$

$+ 4.8 \times 5 \times 110$

$+ 4.8 \times 1.5 \times 110 \times .5 = 10{,}176$

$\phi = 13.3°$ $W_2 = 9.3 \times 10 \times 120 \times .5$

$+ 9.3 \times 6.6 \times 110$

$C = 6850$ $+ 9.3 \times 3 \times 110 \times .5 = 13{,}866$

$W_3 = 9.7 \times 13 \times 110 \times .5$ 6936

$W = 30{,}978$

Case 8 - Coulomb Stresses

5' sand: $K_A = 0.40$ $(\beta/\phi = 0.6)$

$p_h = 110(5)(0.40) = \underline{225.5 \text{ psf}} \cos 20° = \underline{\underline{211.9}}$

5' clay: use Rankine since Coulomb breaks down for $\beta > \phi$

$$K_A = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - 1}}{\cos\beta + \sqrt{\cos^2\beta - 1}}$$

$\beta = 18.435°$ $\cos 18.4 = .9487$ ✓

$$K_A = 0.9487 \frac{.9487 - \sqrt{0.1}}{.9487 + \sqrt{0.1}}$$

$$= 0.9487 \frac{.9487 - 0.1i}{.9487 + 0.1i}$$

$$= 0.9487 \frac{(.9487)^2 - 0.01}{(.9487)^2 + 0.01} + \sim i$$

$Re \approx 0.948$

$\therefore p_{h_{5'}} = 110(5)(.948) - 1000(.948)^{1/2} = \underline{0}$
$(521.4 - 973.6 = -452 \text{ psf})$

$p_{h_{15'}} = [110(5) + 120(10)](.948) - 1000(.948)^{1/2}$
$= \underline{\underline{685.4 \text{ psf}}}$

Case 8- Coulomb Stresses

15' sa cl $K_A = 0.81$

$$p_h = \left[0.81\,(1750) - .200\,(.81)^{1/2}\right] \cos 13.3°$$
$$\left[1417.5 - 180\right] \cos 13.3$$
$$= 1204 \text{ psf}$$

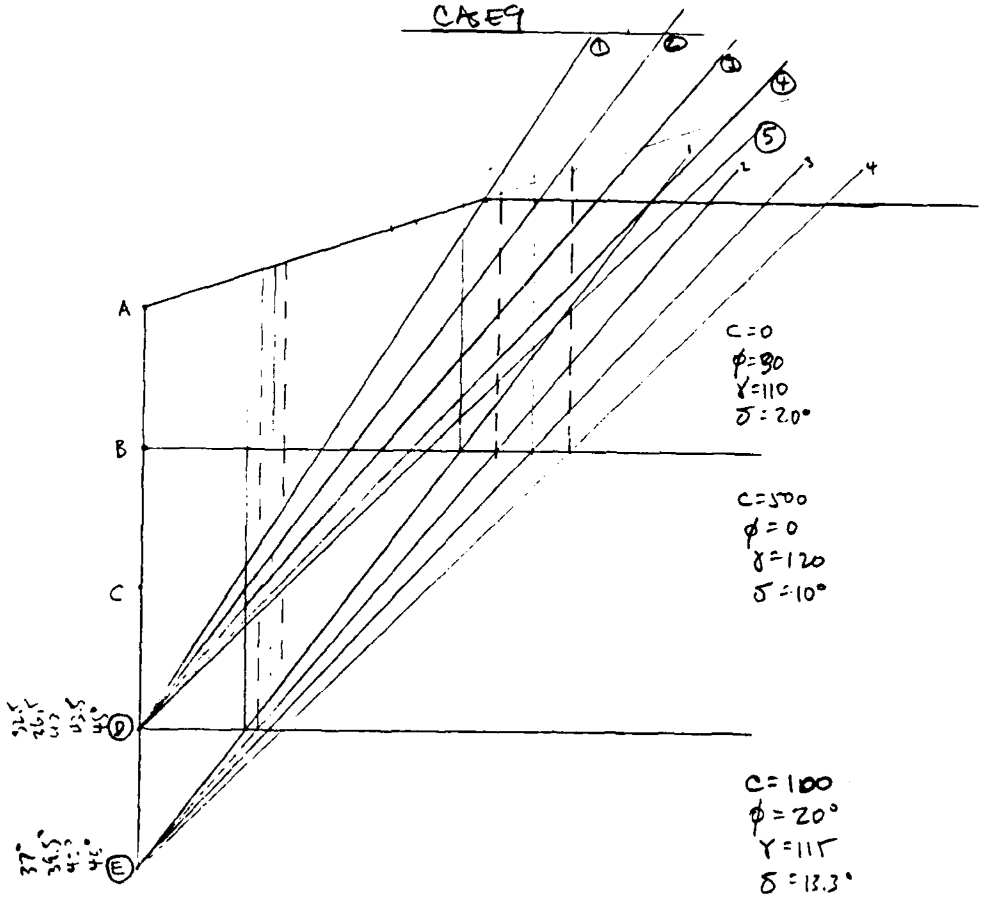
20' sa cl $K_A = 0.81$

$$p_h = \left[0.81\,(2325) - 180\right] \cos 13.3°$$
$$= 1657 \text{ psf}$$

$$P_{AH_c} = \frac{1204 + 1657}{2} \times 5 = 7152$$
$$+ \frac{-432 + 685.4}{2} \times 10 = 1165$$
$$+ \frac{211.9}{2} \times 5 = 530$$
$$8848$$

WORKSHEET

CASE 9

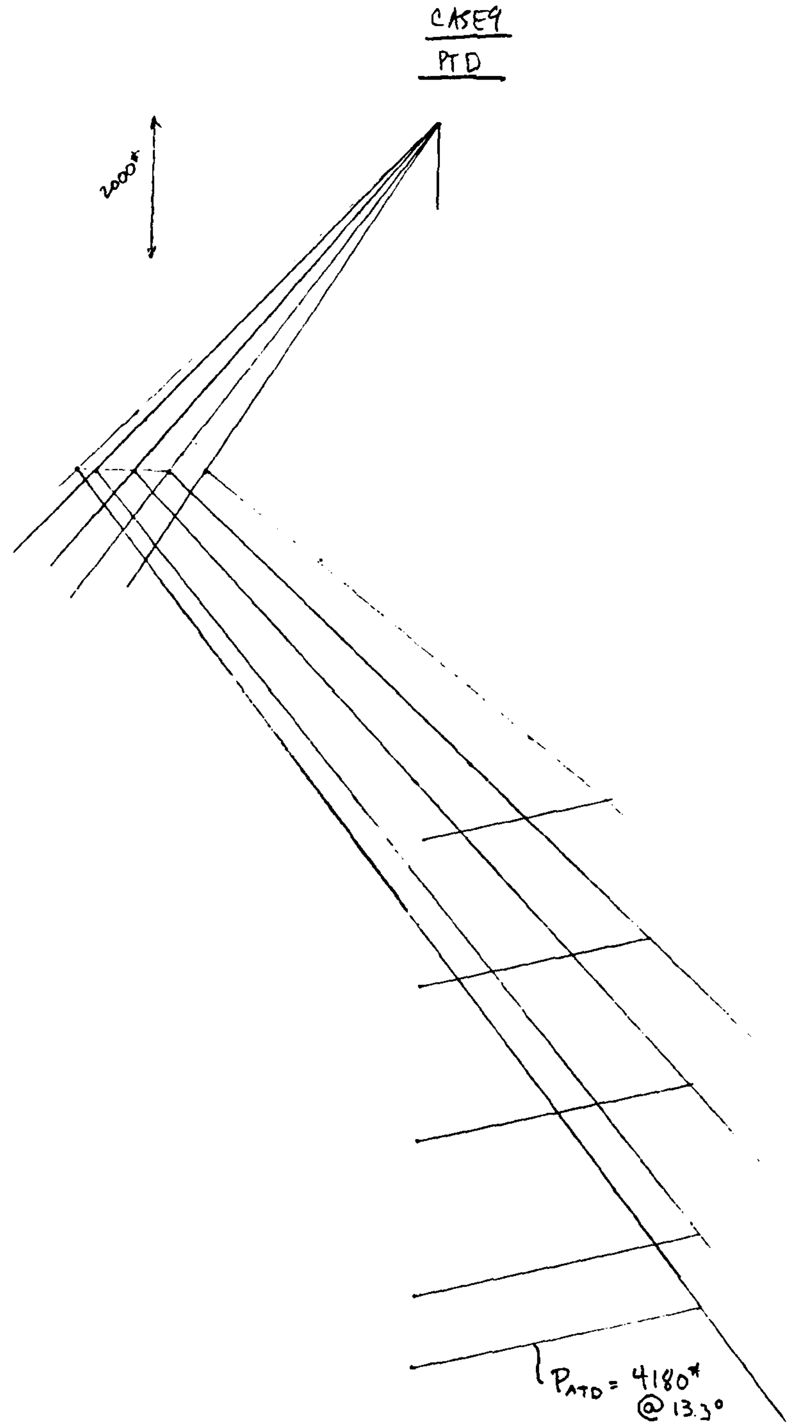


PT A - $P_{AH1A} = 0$

PT B - P_{ATB} = 540# @ 20° (same as Case 8)

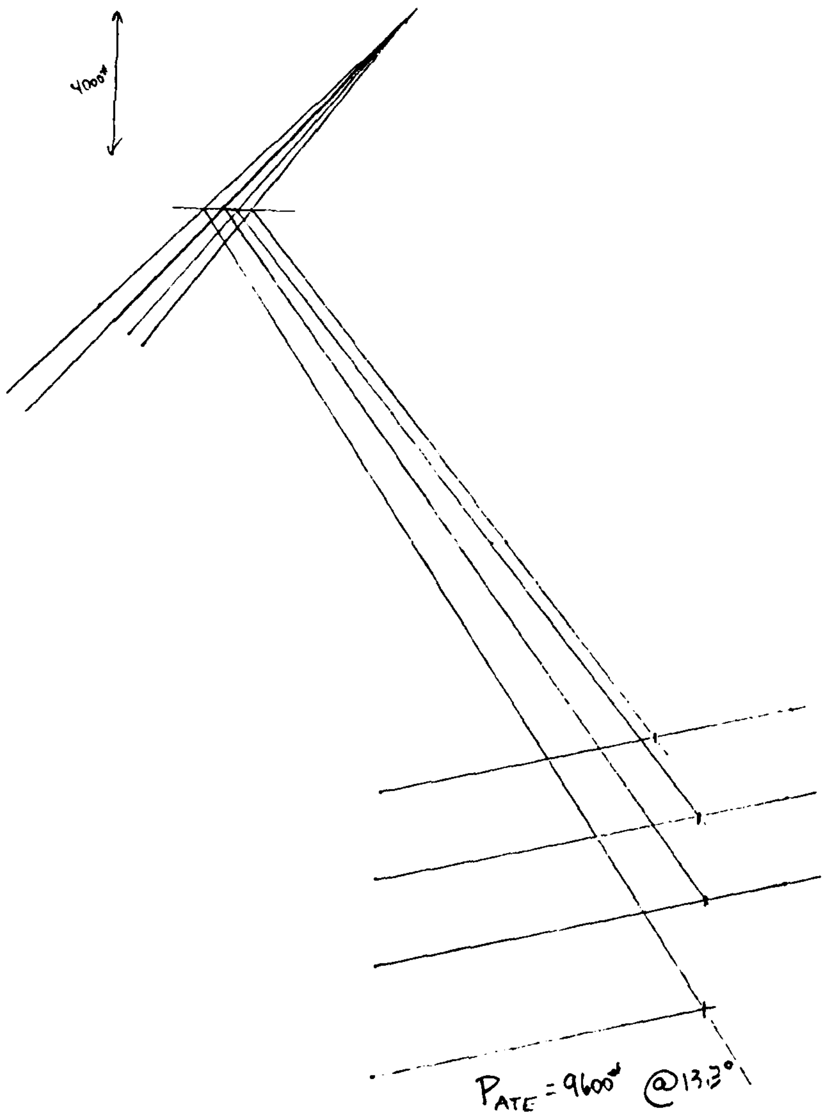
PT C - P_{ATC} = 500# @ 15° (same as Case 8)

} critical wedge intersects before break.


CASE 9
PTD
2000#
P_{ATD} = 4180#
@ 13.3°

CASE 9

PT E



WEIGHTS - CASE 9

PID - ① $W_1 = 6.3(10)(.5)(120)$ $\delta = 13.3°$

$+ 6.3\left(\frac{5+7.1}{2}\right)(110) = 7973$ $\phi = \frac{2226}{10,199} 50 = 6.5°$

$W_2 = 5.7(7.1)(110)(.5) = \underline{2226}$ $C = 5950$

$W = 10,199$

② $W_1 = 7.3(10)(.5)(120)$

$+ 7.3\left(\frac{5+7.6}{2}\right)(110) = 9439$ $\delta = 13.3°$

$W_2 = (7.5)(7.2)(110)(.5)$ $\phi = 6.9°$

$- 2(0.9)(110)(.5) = \underline{2871}$ $C = 6240$

$W = 12,310$

③ $W_1 = 8.4(10)(.5)(120)$

$+ 8.4\left(\frac{5+7.9}{2}\right)(110) = 11,000$ $\delta = 13.3°$

$W_2 = (7.9)(8.9)(110)(.5)$ $\phi = 7.2°$

$- 4(1.9)(110)(.5) = \underline{3449}$ $C = 6575$

$W = 14,449$

④ $W_1 = 9.5(10)(.5)(120)$

$+ 9.5\left(\frac{8.2+5}{2}\right)(110) = 12,597$ $\delta = 13.3°$

$W_2 = (8.2)(11.3)(110)(.5)$ $\phi = 7.4°$

$- 6(3)(110)(.5) = \underline{4106}$ $C = 6925$

$W = 16,703$

⑤ $W_1 = 10(10)(.5)(120)$

$+ 10\left(\frac{3.3+5}{2}\right)(110) = 13,315$ $\delta = 13.3°$

$W_2 = (8.3)(12.8)(110)(.5)$ $\phi = 7.5°$

A-92 $- 7(3.7)(110)(.5) = 4419$ $C = 7125$

$W = 17,734$

WEIGHTS - CASE 9 (CONT'D)

PT E - (1) $W_1 = 3.7\,(5)(115)(.5)$
$+3.7\,(10)(120)$
$+\left(\frac{5+6.2}{2}\right)(3.7)(110) = 7{,}783$

$W_2 = 7.4\,(10)(120)(.5)$
$+\left(\frac{6.2+8.7}{2}\right)(7.4)(110) = 10{,}504$

$\phi = 14°$

$\delta = 13.3°$

$W_3 = 0.8\,(8.8)(110)$
$+9(6)(110)(.5) = 3744$

c = 6950 ft.

$W = 22{,}031$

(2) $W_1 = 4.2\,(5)(115)(.5)$
$+4.2\,(10)(120)$
$+\left(\frac{5+6.4}{2}\right)(4.2)(110) = 8835$

$W_2 = 8.3\left(\frac{6.4+9.2}{2}\right)(110) - (.6)(.3)(.5)(110)$
$+(8.3)(10)(120)(.5) = 12{,}092$

$\phi = 11.6°$

$W_3 = 9(7.3)(110)(.5) = 3614$

$\delta = 12.3°$

$W = 24{,}541$

Weights Case 9 (Cont'd)

Pt. E

③ - $W_1 = 5(4.5)(115)(.5)$
$+10(4.5)(120)$
$+5.75(4.5)(110) = 9540$

$W_2 = 9.1(10)(120)(.5)$
$+9.1\left(\frac{6.5+9.7}{2}\right)(110)$
$-1.8(0.6)(110)(.5) = 13509$ $\phi = 11.5°$

$W_3 = 8.15(9)(110)(.5) = 4034$

$W = 27083$

④ - $W_1 = 5(5)(115)(.5)$
$+5(10)(120)$
$+5.85(5)(110) = 10,655$

$W_2 = 10(10)(120)(.5)$
$+10\left(\frac{6.7+10.1}{2}\right)(110)$ 15,070
$-(1)(3.1)(110)(.5) =$

$W_3 = 9(9)(110)(.5) = 4455$

$W = 30180$ $\phi = 11.5°$

TRIAL WEDGE STRESSES:

0-5 $\sigma_h = \underset{(507^{\#})}{540} \cos 20° \left(\frac{1}{5}\right) = 101$ psf

5-10 $\sigma_h = 0$ psf

10-15 $\sigma_h = (\underset{4068}{4180 \cos 13.3°} - 483)\frac{1}{5} = 717$ psf

15-20 $\sigma_h = (\underset{}{9600 \cos 13.3°} - 4068)\frac{1}{5} = 1055$ psf

$P_{A,T} = 507 + 3585 + 5275 = \underline{\underline{9370 \text{ lb}}}$

COULOMB STRESSES (USE EQUIVALENT BACKSLOPE)

5' (SAND): K_A: $\beta = 10°$ $\beta/\phi = 0.33$

K_A (Corps Manual) = 0.34

$\sigma_h = 0.34(110)(5)\cos 20° = 176$ psf

5' & 15' (CLAY) - SAME AS CASE 8.

15' (SAND-CLAY) - $K_A = 0.50$ (Corps Manual)

$\sigma_h = \{0.5[110(5) + 120(10)] \cancel{\cos 13.3°} - 200\sqrt{.5}$ $\cos 13.3$

$= \underline{\underline{714 \text{ psf}}}$

20' (Sand-Clay) $\sigma_h = \{0.5[110(5) + 120(10) + 115(5)] - 200\sqrt{.5}\}\cos 13.3$

$\underline{\underline{994}}$ psf.

CALCULATION OF P_{A11C}: (Case 9)

$$176(5)\left(\tfrac{1}{2}\right)$$

$$+\ 685\,(6.1)\left(\tfrac{1}{2}\right)$$

$$+\ (714 + 994)(5)(.5) = \underline{\underline{6800\ \text{lb}}}$$

Addendum F

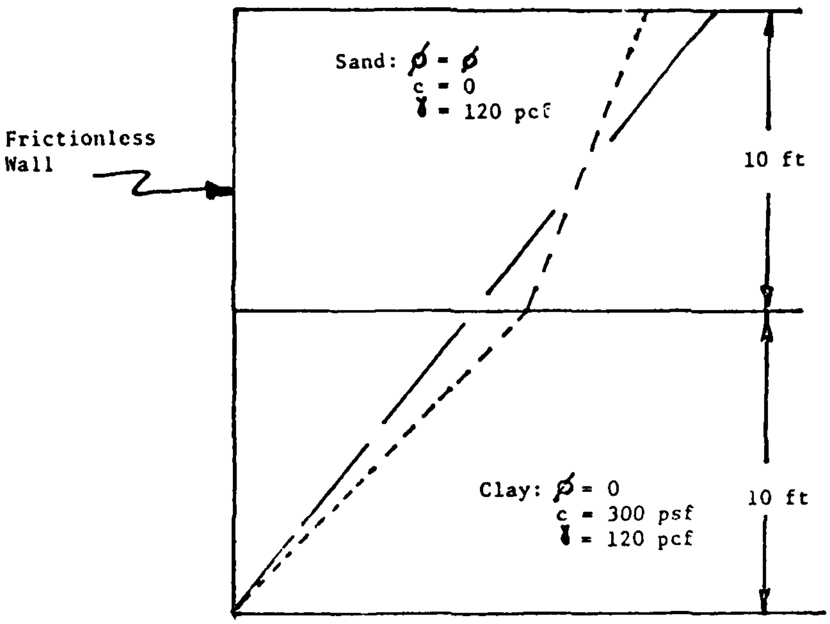
Errors inherent in trial wedge method

Planar sliding is a basic premise of the trial wedge method. In a homogenous soil system planar sliding is consistent with the failure mode assumed in other methods of analysis, such as the Rankine and Coulomb methods, and is essentially correct. In a stratified system planar sliding can be assumed to occur within each stratum, but the angle of failure will vary depending on the angle of internal friction of the various soils. Hence, the overall failure surface will be piecewise planar and not fully planar in the sense assumed in the trial wedge method. See Fig. AF-1. In such a case the maximum active force obtained from a trial wedge analysis will be slightly less than true maximum active force.

In order to evaluate the approximate upper limit of the error, an extreme case was studied, as depicted in Fig. AF-1. A wall that is completely frictionless is backfilled with strata of clay ($\phi = 0$) and sand (variable ϕ) of equal thickness. The total active force acting on the wall was evaluated from the Rankine earth pressure equation (correct solution) and from the trial wedge method (approximate solution). The apparent maximum error observed in the study was 15 percent.

Since the problem studied represents an extreme "mismatch" between backfill materials, the error for most practical problems encountered should be less than that reported above. Nonetheless, the user is cautioned that such an inherent error does exist in using the trial wedge method in stratified soils.

A second type of inherent error occurs whenever stiff cohesive soil overlies cohesionless soil, the trial wedge method will yield unconservative results. An error occurs because the entire wedge is assumed to fail as a block in the trial wedge method, so the underlying cohesionless soil cannot fail and produce active forces until the weight of the wedge and overlying surcharge is sufficient to cause shearing failure in the clay. In a physical sense, active forces can be produced in the cohesionless soil before failure of the overlying clay occurs. Information relative to this condition is given in the discussion of Example No. 4, earlier in Exhibit A.



——— ——— Trial wedge failure surface (planar)

- - - - - - - - Possible true failure surface

c (Clay)	ϕ (Sand)	γ	P_{AHT}	$P_{RANKINE}$	% ERROR
300 psf	30 deg.	120 pcf	13120 lb.	14000 lb.	6
300 psf	35 deg.	120 pcf	12337 lb.	13625 lb.	9
300 psf	40 deg.	120 pcf	11715 lb.	13305 lb.	12
300 psf	45 deg.	120 pcf	11080 lb.	13030 lb.	15

Fig. AF-1. Inherent Errors Arising from Assumption of Planar Sliding

ERROR COMPUTATIONS

Case 1: $c = 300$ psf (clay)
$\phi = 30°$ (sand)
$\gamma = 120$ pcf (both)

P_{AHT} = 13120 lb. (obtained by computer)

$P^{*}_{RANKINE}$:

a. Sand Layer: top: $\sigma_h = 0$
bottom: $\sigma_h = 120(10) K_A$
$= 1200 \tan^2(45° - 15°)$
$= 400$ psf

$P_{A,SAND} = \frac{10 \times 400}{2} = 2000$ lb

b. Clay Layer: top: $\sigma_h = \gamma h - 2c$
$= 120(10) - 600 = 600$ psf
bottom: $\sigma_h = 120(20) - 600 = 1800$ psf

$P_{A,CLAY} = \frac{600 + 1800}{2} \times 10$
$= 12,000$ lb

$P_{RANKINE} = P_{A,SAND} + P_{A,CLAY} = 14,000$ lb

% ERROR $= \frac{14,000 - 13120}{14000} \times 100 = 6.286\%$ SAY 6%

Case 2: $c = 300$ psf (clay)
$\phi = 35°$ (sand)
$\gamma = 120$ pcf (both)

P_{AHT} = 12337 lb (obtained by computer)

---

* Rankine and Coulomb Pressures are Identical when There Is No Wall Friction, As Is the Case for This Problem

$P_{RANKINE}$:

a. Sand Layer: top: $\sigma_h = 0$

bottom: $\sigma_h = 120(10)K_A = 1200\tan^2(45° - 17.5°) = 325$ psf

$P_{A,SAND} = \frac{10(325)}{2} = \underline{1625 \text{ lb.}}$

b. Clay Layer: no change: $P_{A,CLAY} = \underline{12{,}000 \text{ lb}}$

$P_{RANKINE} = P_{A,SAND} + P_{A,CLAY} = \underline{13625 \text{ lb}}$

% ERROR $= \frac{13625 - 12337}{13625} \times 100 = 9.453\%$ SAY $\underline{9\%}$

Case 3: $c = 300$ psf (clay)
$\phi = 40°$ (sand)
$\gamma = 120$ pcf (both)

$P_{AHT} = 11715$ lb. (obtained by computer)

$P_{RANKINE}$: a. Sand Layer: top: $\sigma_h = 0$

bottom: $\sigma_h = 120(10)K_A = 1200\tan^2(45° - 20°) = 261$ psf

$P_{A,SAND} = \frac{10 \times 261}{2} = \underline{1305 \text{ lb}}$

b. Clay Layer: no change: $P_{A,CLAY} = \underline{12{,}000 \text{ lb}}$

$P_{RANKINE} = P_{A,SAND} + P_{A,CLAY} = \underline{13305 \text{ lb}}$

% ERROR $= \frac{13305 - 11715}{13305} \times 100 = 11.95\%$ SAY $\underline{12\%}$

Case 4: $c = 300$ psf (clay)
$\phi = 45°$ (sand)
$\gamma = 120$ pcf

$P_{AHT} = 11\,080$ lb (obtained by computer)

$P_{RANKINE}$: a Sand Layer: top: $\sigma_h = 0$

bottom: $\sigma_h = 120(10)K_A = 1200\tan^2(45° - 22.5°) = 206$ lb

$P_{A,SAND} = \frac{10 \times 206}{2} = \underline{1030 \text{ lb}}$

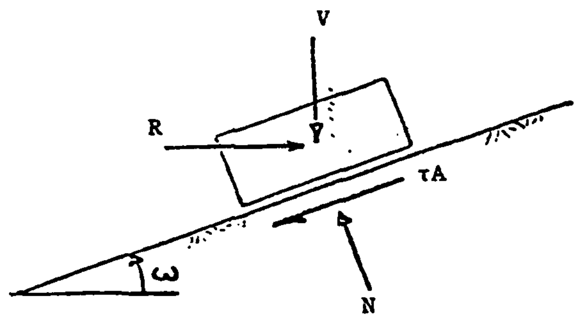
b. Clay Layer: no change: $P_{A,CLAY} = \underline{12{,}000 \text{ lb}}$

$P_{RANKINE} = P_{A,SAND} + P_{A,CLAY} = \underline{13030 \text{ lb}}$

% ERROR $= \frac{13030 - 11080}{13030} = 14.97\%$ SAY $\underline{15\%}$

EXHIBIT B: DERIVATION OF EQUATIONS FOR SLIDING

Derivation of equations for uphill sliding as given in ETL 1110-2-184. Note that the horizontal component of the applied loads is not included in the derivation.



Vertical Section Through Sliding Mass on Sloping Surface

The safety factor $S_{s-f} = \frac{R}{H}$

Where R = resisting force (It is applied as a driving force in the derivation to determine its value when it is just equal to the horizontal components of the resisting forces)

H = driving force

<u>Sum Vertical Forces</u>:

$$o = V - N \cos\omega + \tau A \sin\omega$$

Where $\tau = C + \frac{N}{A} \tan \phi$

then

$$o = V - N \cos\omega + cA \sin\omega + N \sin\omega \tan \phi$$

$$o = V - N(\cos\omega - \sin\omega \tan \phi) + cA \sin\omega$$

$$N = \frac{V + sA \sin\omega}{\cos\omega - \sin\omega \tan \phi} \qquad \text{(B-1)}$$

<u>Sum Horizontal Forces</u>:

$$o = R - N \sin\omega - \tau A \cos\omega$$

$$o = R - N \sin\omega - CA \cos\omega - N \cos\omega \tan \phi$$

$$= R - N(\sin\omega + \cos\omega \tan \phi) - CA \cos\omega$$

$$N = \frac{R - CA \cos\omega}{\sin\omega + \cos\omega \tan \phi} \qquad \text{(B-2)}$$

Equate N in equations B-1 and B-2:

$$\frac{V + cA\sin\omega}{\cos\omega - \sin\omega\tan\phi} = \frac{R - cA\cos\omega}{\sin\omega + \cos\omega\tan\phi}$$

$$R - CA\cos\omega = \left(V + cA\sin\omega\right)\left(\frac{\sin\omega + \cos\omega\tan\phi}{\cos\omega - \sin\omega\tan\phi}\right) \qquad \text{(B-3)}$$

but $$\frac{\sin\omega + \cos\omega\tan\phi}{\cos\omega - \sin\omega\tan\phi} = \frac{\frac{\sin\omega}{\cos\omega} + \frac{\cos\omega}{\cos\omega}\tan\phi}{\frac{\cos\omega}{\cos\omega} - \frac{\sin\omega}{\cos\omega}\tan\phi}$$

$$= \frac{\tan\omega + \tan\phi}{(1 - \tan\omega\tan\phi)},$$

by identity

$$\frac{\tan\omega + \tan\phi}{1 - \tan\omega\tan\phi} = \tan(\phi + \omega)$$

Substituting into equation B-3 we have

$$\begin{aligned} R &= cA\cos\omega + V\tan(\phi + \omega) + cA\sin\omega\tan(\phi + \omega) \\ &= V(\tan(\phi + \omega)) + cA(\sin\omega\tan(\phi + \omega)) + cA\cos\omega \\ &= V(\tan(\phi + \omega)) + cA\left[\sin\omega\tan(\phi + \omega) + \cos\omega\right] \qquad \text{(B-4)} \end{aligned}$$

by identities

$$\tan(\phi + \omega) = \frac{\tan\phi + \tan\omega}{1 - \tan\phi\tan\omega}$$

since $\tan\phi = \dfrac{\sin\phi}{\cos\phi}$ and similar for ω:

$$\therefore \tan(\phi + \omega) = \frac{\frac{\sin\phi}{\cos\phi} + \frac{\sin\omega}{\cos\omega}}{1 - \frac{\sin\phi}{\cos\phi}\frac{\sin\omega}{\cos\omega}}$$

Get common denominator:

$$= \frac{\dfrac{\cos\omega \sin\phi + \sin\omega \cos\phi}{\cos\phi \cos\omega}}{\dfrac{\cos\phi \cos\omega - \sin\phi \sin\omega}{\cos\phi \cos\omega}}$$

$$= \frac{\cos\omega \sin\phi + \sin\omega \cos\phi}{\cos\phi \cos\omega - \sin\phi \sin\omega},$$

divide by $\cos \phi$ we have :

$$= \frac{\dfrac{\cos\omega \sin\phi}{\cos\phi} + \sin\omega \dfrac{\cos\phi}{\cos\phi}}{\dfrac{\cos\phi \cos\omega}{\cos\phi} - \dfrac{\sin\phi \sin\omega}{\cos\phi}}$$

$$= \frac{\sin\omega + \cos\omega \tan\phi}{\cos\omega - \sin\omega \tan\phi}$$

multiply by $\sin\omega$:

$$\sin\omega \tan(\phi + \omega) = \frac{\sin^2\omega + \cos\omega \sin\omega \tan\phi}{\cos\omega - \sin\omega \tan\phi},$$

add $\cos\omega$:

$$\cos\omega + \sin\omega \tan(\phi + \omega) = \frac{\sin^2\omega + \cos\omega \sin\omega \tan\phi}{\cos\omega - \sin\omega \tan\phi} + \cos\omega$$

$$= \frac{\sin^2\omega + \cos\omega \sin\omega \tan\phi + \cos^2\omega - \sin\omega \cos\omega \tan\phi}{\cos\omega - \sin\omega \tan\phi}$$

$$= \frac{\sin^2\omega + \cos^2\omega}{\cos\omega - \sin\omega \tan\phi}$$

$$= \frac{1}{\cos\omega - \sin\omega \tan\phi},$$

substitute into equation B-4 and multiply the denominator of the last term by $\frac{\cos\omega}{\cos\omega}$ we have:

$$R = V(\tan(\phi + \omega)) + \frac{CA}{\frac{\cos\omega}{\cos\omega}(\cos\omega - \sin\omega \tan\phi)} = V(\tan(\phi+\omega)) + \frac{CA}{\cos\omega\left[1 - \tan\omega \tan\phi\right]}$$

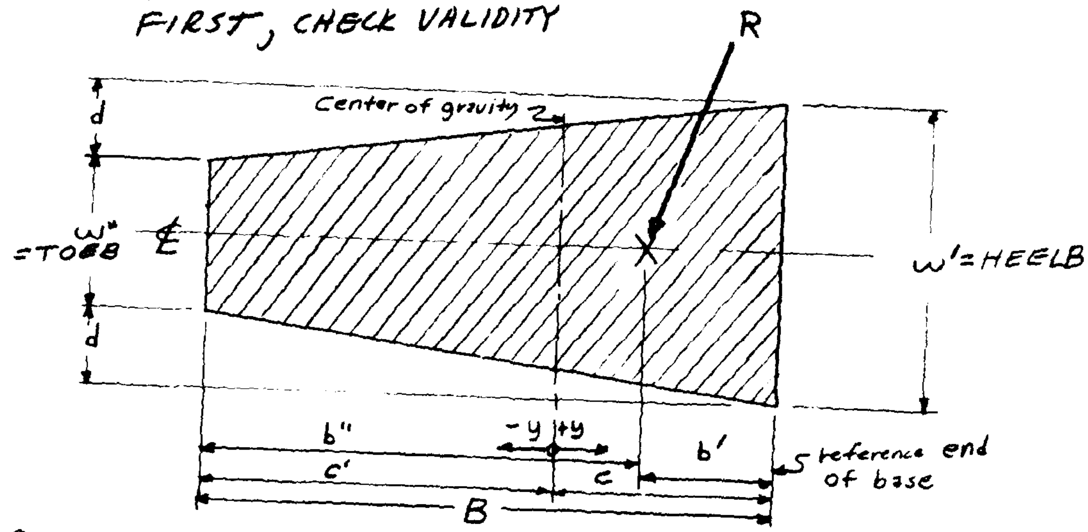
The formula for downhill sliding is derived in a similar manner.

EXHIBIT C: EQUATIONS FOR EARTH PRESSURE UNDER A TRAPEZOIDAL BASE

CASE 1: Resultant Within Kern

CASE 2: Resultant Outside Kern

C-1. CALCULATE CASE I (RESULTANT WITHIN KERN) FIRST, CHECK VALIDITY



C-1.1 Geometric properties of base contact area:

$$A_g = \frac{w' + w''}{2} B$$

$$I_{cg} = \frac{B^3 (w'^2 + 4 w' w'' + w''^2)}{36 (w' + w'')}$$

$$c = \frac{B (2 w'' + w')}{3 (w'' + w')}$$

ref: AISC "Manual of Steel Construction", Seventh Edition p. 6-27.

C-1.2 Stress equation: (y + as shown above, general expression)

$$f = \frac{R}{A_g} + \frac{R(c - b')(\pm y)}{I_{cg}}$$

C-1.2.1 Stress at reference end of base: (y = c-0)

$$f = \frac{R}{A_g} + \frac{R(c - b')c}{I_{cg}}$$

Note: if $b' > c$, then the second term goes negative and the My/I stress subtracts from the P/A stress.

C-1.2.2 Stress at far end of base: $(y = c-B)$

$$f = \frac{R}{A_g} + \frac{R(c-b')(c-B)}{I_{cg}}$$ The second term is actually negative since $B > c$.

Note: if $b' > c$, then the second term changes sign and the two terms will wind up adding numerically.

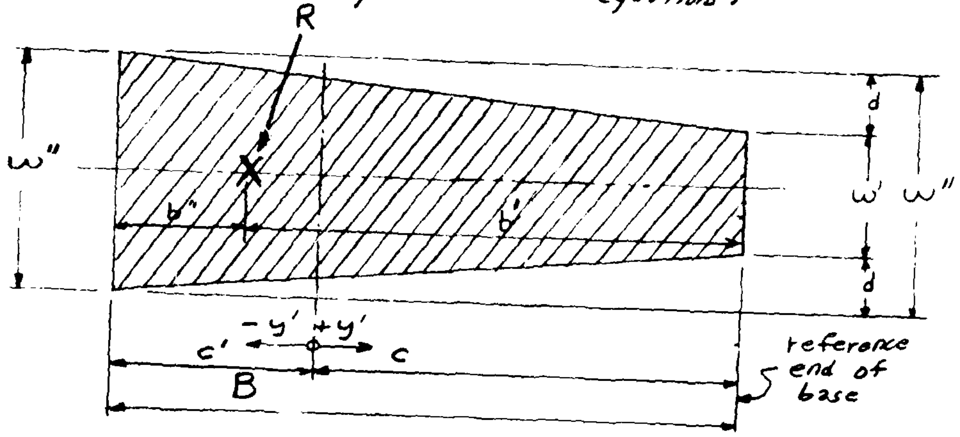
C-1.3 Kern Check:

a. If the stress at the narrow end is negative, then the resultant is outside the kern, toward the wide end (Case 2) and it must be calculated accordingly.

b. If the stress at the wide end is negative, then the resultant is outside the kern, toward the narrow end (Case 3) and it must be calculated accordingly.

c. If the stresses at both ends are positive, then the resultant is inside the kern and the Case 1 expression is valid.

C-1.4 Check to see if making the reference end be the narrow end will require different equations:



A_g = no change if w' and w'' are interchanged,

I_{cg} = no change if w' and w'' are interchanged,

$$c = \frac{B(2w' + w'')}{3(w' + w'')}$$ as shown above.

$$f = \frac{R}{A_g} + \frac{R(c-b')(\pm y')}{I_{cg}}$$

$$f_{\text{reference end}} = \frac{R}{A_g} + \frac{R(c-b')(c)}{I_{cg}}$$

In the expression for $f_{\text{reference end}}$ only the sub-expression $+(c-b')(c)$ is different between the two reference locations:

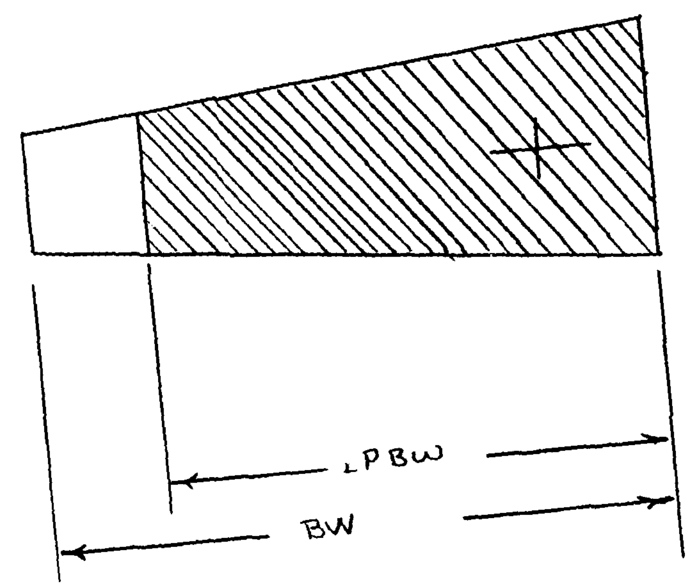
$(c-b')c$ on page C-4 is checked against $(c'-b'')c'$ on page C-1: From page C-4, c and b' are seen to be equal to c' and b'', respectively. Therefore the two equations are equivalent.

CASE 2

RESULTANT OUTSIDE THE KERN

C-2.0

When the resultant is outside the kern, a portion of the base is no longer in compression. The remainer of the base which is in compression and applying a stress to the sabsarface is called the "EFFECTIVE BASE AREA", shown ▨ below:



BW = ACTUAL BASE WIDTH

EPBW = EFFECTIVE BASE WIDTH

C-2.1 DETERMINING THE EFFECTIVE BASE WIDTH

To find the width of effective base area, the performs an incremental search across the actual base width to locate the zero-pressure point. This is done by holding the location of Resultant on the base constrant and varying the base deminsion so that $f = 0$ in the equation:

$$f = P/A \pm \frac{Mc}{I}$$

FOR A TRAPEZOIDAL BASE

$$f = \frac{R}{A_g} + \frac{R\,(c - b')(C - B)}{I_{cg}}$$

$$A_g = \frac{w' + w''}{2} B$$

$$I_{cg} = \frac{B^3\,(w'^2 + 4w'w'' + w''^2)}{36\,(w' + w'')}$$

$$c = \frac{B\,(2w'' + w')}{3(w'' + w')}$$

$B = BW -$ DELTA B

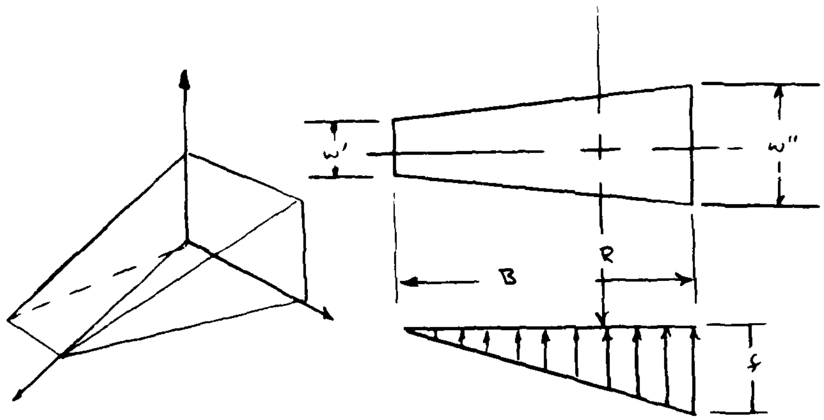
DELTA B = Incremental Distance from end of zero compression

w' & w'' are dependent on B

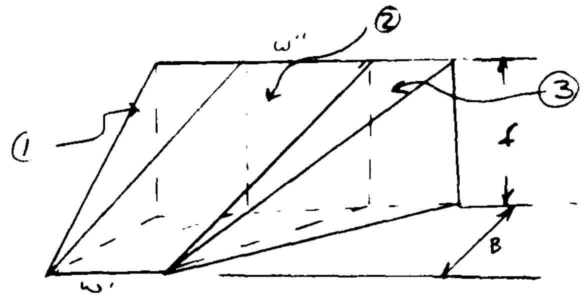
DELTA B is varied in a search routine until $f = 0$, yielding a corresponding "Effective Base Width"

C-22 DETERMINING THE MAXIMUM STRESS

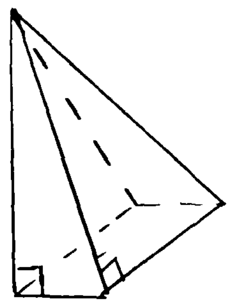
The maximum stress occurs opposite to the end of zero compression, which yields the pressure distribution diagram below



Applying the law of simple mechanics, Resultant force is equal to the volume of the pressure distribution diagram. This diagram can be divided in to three geometrical shape; One Prism and Two Pyramids:



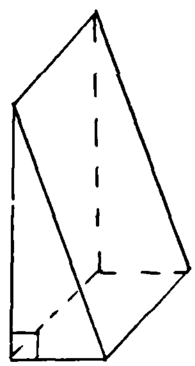
1, 3



$$V_{pyramid} = \frac{1}{3}\,\frac{\omega'' - \omega'}{2}\, f B$$

$$V_{prism} = \frac{1}{2} f B \omega'$$

2



$$V_T = V_{prism} + 2V_{pyramid}$$

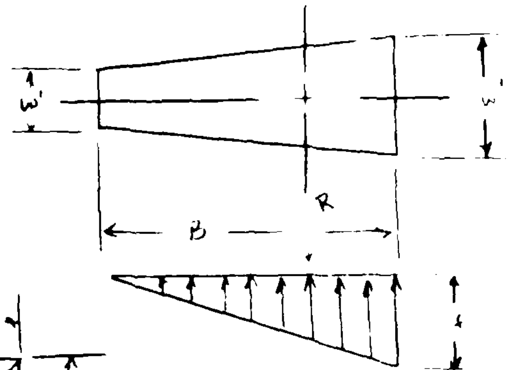
$$V = \frac{1}{2} fB\omega' + \frac{2}{3}\left(\frac{\omega'' - \omega}{2}\right) fB$$

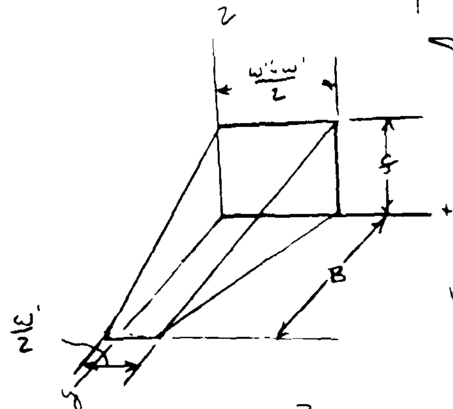
$$= fB\left[\frac{1}{2}\omega' + \frac{1}{3}(\omega'' - \omega')\right]$$

$$= fB\left[\frac{1}{6}\omega' + \frac{1}{3}\omega''\right]$$

$$= \frac{fB}{6}\left[\omega' + 2\omega''\right]$$

Another method to find the volume of the diagram would be to integrate the distribution over the area as show below:





$$z = \frac{f}{B} x$$

$$V_{volume} = \iint z \, dy \, dy$$

$$= \int_0^B dx \, \frac{f}{B} \cdot \int_{-\left[\frac{1}{2} w' + \frac{w''-w'}{2} x\right]}^{\left[\frac{1}{2} w' + \frac{w''-w'}{2} x\right]} dy$$

$$= \int_0^B \left(\frac{f}{B} x\right) \left[w' + \frac{w''-w'}{2} x \right] dx$$

$$= \frac{f}{B} \left[\frac{w'}{2} B^2 + \left(\frac{w''-w'}{B} \right) \frac{1}{3} B^3 \right]$$

$$= fB \left[\frac{1}{2} w' + \frac{1}{3} (w''-w') \right]$$

$$= \frac{fB}{6} (w' + 2w')$$

The two method yield the same equation.

Where for the maximum stress is:

$$f = \frac{6R}{B(w' + 2w'')}$$

EXHIBIT D: COMPUTATION OF AT REST EARTH PRESSURE

D-1 PRESSURE CALCULATION

D-1.1 Direct Determination of Expected Pressure - Two of the more common methods available for the direct determination of horizontal at-rest earth pressure are described below:

a. The finite element method can be used to predict at-rest earth pressures. It accounts for multiple layer soil systems and for irregular backfill and surcharges. However, it is beyond the scope of this program.

b. Boussinesq-type elasticity equations can be used to calculate pressures. Manipulation is required to account for sloping backfill, and is beyond the scope of this program.

D-1.2 Use of Horizontal Earth Pressure Coefficients - There are several common methods available for determining coefficients (K_r) to be multiplied by the earth weight to obtain horizontal pressure due to a level backfill. Several of them are discussed below:

a. Field or laboratory tests by soils engineers can yield a suitable value for K_r. WES Technical Report S-75-16 discusses various methods and their accuracy.

b. Approximate coefficients for typical soil types are shown in paragraph 3.d on page 3 of EM 1110-2-2502. These coefficients are valid for only level backfill. The procedure described in the referenced EM paragraph can be used to get a coefficient for a sloping backfill, but the method becomes invalid when the backfill slope angle exceeds the equivalent ϕ angle (ϕ') calculated from equation 6 on page 3 of the EM.

   (1) For an average sand ($K_r = 0.5$), the maximum slope is 1V:2.82H (19.47122°) for this method.

   (2) For an average clay ($K_r = 0.85$), the maximum slope is 1V:12.29H (4.6507°) for this method.

c. Jaky's equation

$$K_r = 1-\sin\phi$$

can be used for cohesionless soils with level backfill. See WES TR S-75-16 for a discussion and comparison with measured values. It has been suggested that Jaky's equation can be modified to approximately account for sloping backfills, based on the similarity between the numerators of the Rankine active earth pressure coefficient for level and sloping backfill conditions. This can be demonstrated as shown below:

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} \quad \text{Rankine's equation for level backfill}$$

$$K_r = \frac{1-\sin\phi}{1} \quad \text{Jaky's equation for level backfill}$$

$$K_a = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}\right] \quad \text{Rankine for sloping fill *}$$

$$K_r = \cos\beta \left[\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}\right] \quad \text{Modified Jaky for sloping fill*}$$

Where ϕ = angle of internal friction

β = angle of backfill slope

D-2 GRAPHIC COMPARISON OF METHODS - Figure D-1 shows a graphic comparison of several methods for calculating K_r, with ϕ varying from 0° to 90° and β varying from 0° to 80°.

* This is the total earth pressure coefficient, not its horizontal component.

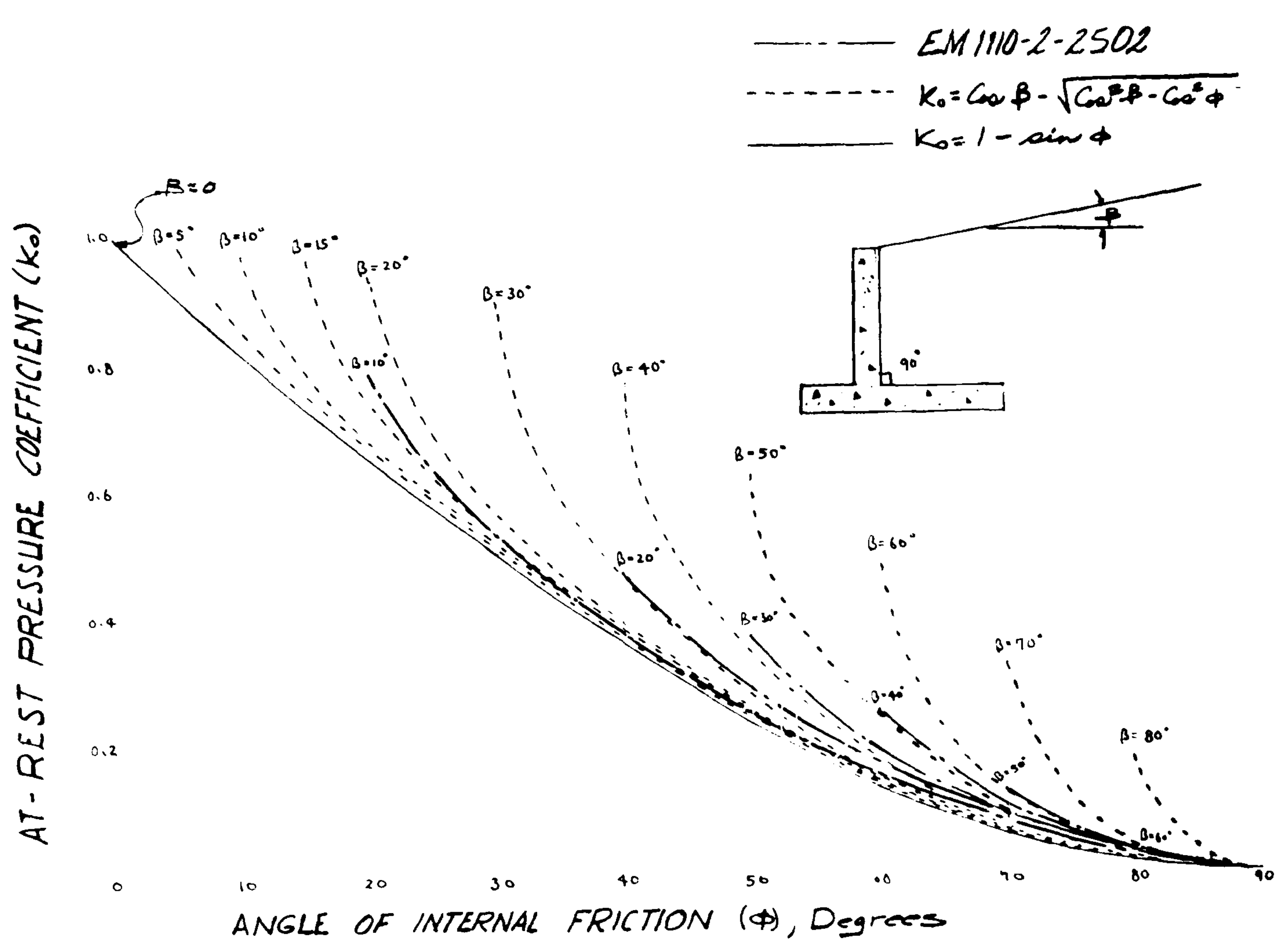


Figure D-1. Graphical comparisons of methods

EXHIBIT E: FORMULAS FOR WORKING STRESS DESIGN OF CANTILEVER SLABS WITH AXIAL FORCE (ACCORDING TO APPENDIX B OF ACI 318-77)

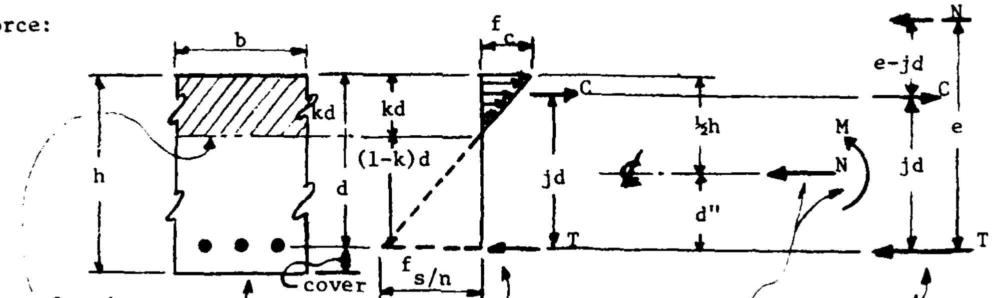
E-1 NOTATION is as shown in ACI 318-71.

E-2. DEFAULT VALUES

item	for hydraulic structures	for other structures
f_c/f'_c	0.35	0.45
f_s max.	20000 psi	20000 psi for grades 40 or 50 24000 psi for grade 60 or higher
$n = E_s/E_c$	$\frac{29000000}{57000\sqrt{f'_c}} = \frac{508.77}{\sqrt{f'_c}}$	$\frac{508.77}{\sqrt{f'_c}}$

E-3. FLEXURE

E-3.1 BASIC RELATIONSHIPS (Reference ACI Publication SP-3, "Reinforced Concrete Design Handbook for Working Stress Method") for moment and axial force:



neutral axis = centroid of shaded area and transformed steel area nA_s

Cross-Section

Stress Diagram

Applied force and moment (force at center of gravity of slab, compression is positive)

Equivalent Equilibrium forces with e > jd

a. For the concrete stress f_c to be at its maximum allowable value,

$$k = \frac{1}{1 + \frac{f_s}{nf_c}} \qquad \text{and } j = 1 - \frac{k}{3}$$

b. The equivalent external (applied) moment is then Ne, which is found from

$$Ne = N(\frac{M}{N} + d'') = M + Nd'',$$

where N is plus for axial compression. Manipulating this relationship, and the ones in the previous paragraph, to get N out of the denominator, yields

$$Ne = Nd'' + M = N(d-\frac{h}{2}) + M = N(d - \frac{d+cover}{2}) + M = \frac{N}{2}\ (d-cover) + M$$

E-3.2 DESIGN

E-3.2.1 Determining minimum slab thickness

Summing moments about T yields

$$Ne = Cjd = f_c bkjd^2\frac{1}{2}$$

or

$$f_c bkjd^2\frac{1}{2} = \frac{N}{2}d + M - \frac{N(cover)}{2}$$

or

$$(f_c bkj)d^2 - Nd - (2M-N(cover)) = 0.0 \qquad \text{(E-1)}$$

Solving the quadratic equation E-1 for the smaller real, positive root will yield the minimum allowable effective depth d. Adding the cover to the center of the reinforcing steel yields the minimum slab thickness for moment at that location. Note that the effect of axial tension is to reduce the compressive stress and so yield a smaller value for minimum slab thickness. Such reduction must be applied with caution.

E-3.2.2 Determining reinforcement for balanced design in Sections with minimum effective depth, as in Equation E-1:

Taking moments about the concrete force C, using the relationships in paragraph E-3.1a and E-3.1b, derive an expression for A_s to have the steel stressed to its allowable stress simultaneously with the extreme fiber concrete stress reaching its allowable:

$$N\ (e-jd) = Tjd$$

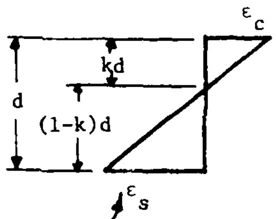
$$Ne - Njd = Tjd$$

$$\frac{N}{2}(d-cover) + M - Njd = Tjd$$

$$M + N\left[d\left(\frac{1}{2} - j\right) - \frac{\text{cover}}{2}\right] = Tjd = A_s f_s jd$$

$$A_s = \frac{M}{f_s jd} + \frac{N}{f_s jd}\left[d\left(\frac{1}{2} - j\right) - \frac{\text{cover}}{2}\right] \qquad \text{(E-2)}$$

E-3.2.3 <u>Determining reinforcement for Sections with more than the minimum effective depth (under-reinforced) from Equation E-1:</u>



Given: M, N, d, f_s, h, cover, d"

Find = A_s

a. Basic relationships:

$f_s = E_s \varepsilon_s$, by definition

$\varepsilon_c = \varepsilon_s \dfrac{k}{1 - k}$, from similar triangles

$$f_c = E_c \varepsilon_c = E_c \varepsilon_s \frac{k}{1 - k} = E_s \varepsilon_s \frac{E_c}{E_s} \frac{k}{1 - k} = \frac{f_s}{n} \frac{k}{1 - k}$$

b. Summation of axial forces = 0:

$$C - T = N$$

$$C = \frac{f_c bkd}{2} = \frac{f_s bd}{2n} \frac{k^2}{1 - k}, \quad T = A_s f_s$$

c. Summation of moments about t = 0:

$$Cjd - Nd'' - M = 0$$

$$j \left(1 - \frac{k}{3}\right) = \frac{3-k}{3}$$

$$\frac{f_s bd}{2n} \frac{k}{1-k} \frac{(3-k)d}{3} - Nd'' - M = 0$$

$$\frac{f_s bd^2}{6n} \frac{(3k^2 - k^3)}{1-k} - Nd'' - M = 0$$

$$\frac{f_s bd^2}{6n} k^3 - \frac{f_s bd^2}{2n} k^2 - (M + Nd'')k + (M + Nd'') = 0$$

Solving this cubic equation for the smallest real, positive root will yield the actual value of k. Then, from the force summation C-T = N, get

$$\frac{f_s bd}{2n} \frac{k^2}{1-k} - f_s A_s = N$$

$$A_s = \frac{bd}{2n} \frac{k^2}{1-k} - \frac{N}{f_s} \qquad \text{(E-3)}$$

E-3.3 ANALYSIS (by rearrangement of Equation E-1)

E-3.3.1 Concrete stress:

$$f_c bkjd^2 - Nd - (2M - N(\text{cover})) = 0$$

$$f_c bkjd^2 = N(d\text{-cover}) + 2M$$

$$f_c = N \frac{d\text{-cover}}{bkjd^2} + M \frac{2}{bkjd^2} \qquad \text{(E-4)}$$

E-3.3.2 Steel Stress:

Substituting the basic relationships of T for those of C in equation E-1:

$$A_s f_s jd = M + N \left(d \left(\frac{1}{2} - j\right) - \frac{\text{cover}}{2}\right)$$

$$f_s = \frac{M}{A_s jd} + \frac{N}{A_s jd} \left[d \left(\frac{1}{2} - j\right) - \frac{\text{cover}}{2}\right] \qquad \text{(E-5)}$$

E-4. SHEAR (See ACI 318-71, paragraph 8.10.3)

Allowable stresses will be 55% of the code allowable for ultimate strength design given in ACI 318-71, Chapter 11. In section 11.4.4 of ACI 318-71, the term N_u will be used as 2.0 times the axial force N from the key.

a. Shear stress is calculated using equation 11-3 from ACI 318-71:

$$v_u = \frac{V_u}{\phi\, b_w d}$$

which for working stress design rules, becomes

$$v = \frac{V}{bd}$$

b. The allowable stress is calculated according to equation 11-8 in ACI 318-71 for sections with axial load (tension is used as negative):

$$v_c = 2\left(1 + \frac{N_u}{500\, A_g}\right)\sqrt{f'_c}$$

which is ACI 318-71 paragraph 11.4.1 when $N_u = 0$.
Equation 11-4 in ACI 318-71 is used for sections without axial load:

$$v_c = 1.9\sqrt{f'_c} + 2500\, \frac{A_s}{b_w d}\, \frac{V_u d}{M_u},$$ but not over 3.5 f'_c, using V for V_u and M for M_u since they would have the same load factor. (In equation 11-4, $(V_u d)$ shall not exceed M_u).
Both values of v_c are multiplied by 0.55 before comparison with v.

E-5. DEVELOPMENT OF REINFORCEMENT

Development of reinforcement (bond strength) shall be as required in ACI 318-71 Chapter 12, except that computed shear V shall be multiplied by 2.0 and substituted for V_u. In computing M_t, the quantity (d-a/2) may be taken as 0.85d. Where the A_s provided is more than twice that required, the stress may be considered as always less than $0.5f_y$ for the purpose of satisfying provisions relating to splices. Splices will not be calculated in this program, but the information is presented in this appendix for completeness. This presentation assumes no hooked ends except in keys.

Reinforcement in T-Wall components is assumed to be negative moment reinforcement as considered in ACI 318-71 paragraph 12-3 on page 43 of the code. Stirrups will not be used in this program.

Embedment length into the span shall be as required by ACI 318-71 paragraphs 12.1.1 (hooks shall be used if necessary) and 12.1.4 (reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member (d) or 12 bar diameters, except at ... the free end of cantilevers.

Flexural reinforcement shall not be terminated in a tension zone unless one of the following conditions is satisfied:

1. The shear at the cutoff point does not exceed two-thirds that permitted ...
2. For #11 and smaller bars, the continuing bars provide double the area required for flexure at the cutoff point and the shear does not exceed three-fourths that permitted.

This may be applicable to the heel slab when there is a key under the heel: At least one-third of the total reinforcement provided for negative moment at the support shall have an embedment length beyond the point of inflection not less than the effective depth d, 12 times the bar diameter, or one-sixteenth the clear span (one-eighth of the cantilever span), whichever is greater.

Development length in inches shall be the product of the basic length for #11 or smaller bars,

$$\frac{0.04A_b f_y}{\sqrt{f'_c}}$$

times one or more of the appropriate factors listed below:

1. Top bars (more than 12" or concrete below) = 1.4 .
2. F_y over 60000 psi = not permitted.
3. Bars modified by 1. or 2. above, and spaced laterally at least 6 inches on center = 0.8 (1.4 X 0.8 = 1.12).
4. Bars modified by 1. or 2. above and in excess of that required $= \dfrac{A_s \text{ (req'd)}}{A_s \text{ (used)}}$.

The development length shall not be less than twelve inches, but not less than $0.0004\ d_b\ f_y$.

EXHIBIT F: FORMULAS FOR ULTIMATE STRENGTH DESIGN OF NON-HYDRAULIC STRUCTURES WITH CANTILEVER SLABS WITH AXIAL FORCE (ACCORDING TO ACI STANDARD NO. 318-77)

F-1. NOTATION is as shown in Appendix B to ACI 318-71.

F-2. DEFAULT VALUES

	Item	Default Value
a.	E_s	29000000
b.	Maximum actual f_s from normal long-term loading	not applicable
c.	f_y	40000 psi
d.	Capacity reduction factors: flexure shear, bond	 0.9 0.85
e.	Steel area limiting reinforcement ratio: $\frac{A_s \text{ allowable}}{A_s \text{ balanced}} = R$	(0.5 if load case includes earthquake)
f.	Maximum ratio $\rho = \frac{f_y A_s}{f'_c bd}$ *	not applicable when $h \geq \frac{span}{10}$ (ACI 318-71 Table 9.5(a))
g.	Minimum ratio $\frac{f_y A_s}{bd}$ **	200.0
h.	Maximum strain ϵ	0.003
i.	Stress rectangle magnitude	$0.85f'_c$
j.	β_1 in ACI 318-71 par. 10.2.7	0.85
k.	Load factors (1) flexure shear on concrete bond on top bars bond on other bars	 not applicable " " "
	(2) Using D=concrete weight + water weight V = applied forces vertical + earth weight W = wind E = earthquake, horizontal or vertical H = earth horizontal + surcharge horizontal F = seepage horizontal + uplift	(1) 1.4(D+F)+1.7(V+H) except: a. If sign of D is opposite to H or F, use 0.9 for D. b. If sign of V is opposite to H or F, omit V. (2) With wind or earthquake: a. Add(1.7W+1.87E) and use additional overall factor of 0.75. b. If D+W+E only: 0.9D+1.3W+1.43E. c. Use bigger of a or b above.

Note that V and H are the orthogonal components of the ACI Code load factor group called L.

*for deflection control, based on ACI 318-63, par. 1507

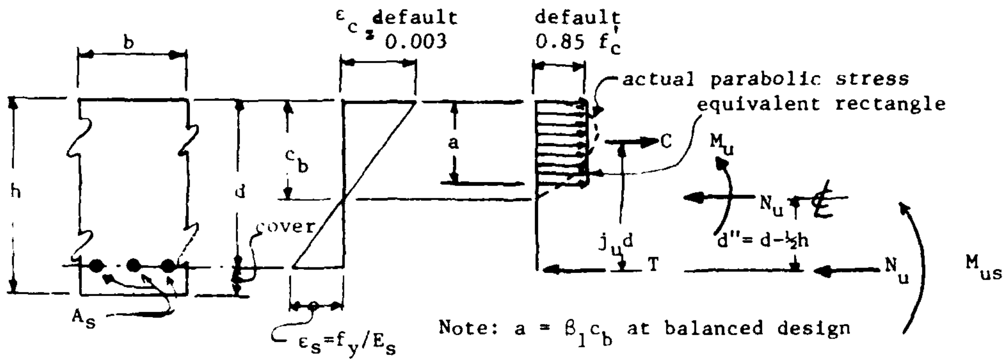
**ACI 318-71, par. 10.5.1

rev Sep 78

F-3. **FLEXURE**

References:
1. Commentary on ACI Code 318-71, Chapter 10.
2. PCA Notes on ACI 318-71, Lecture 4.
3. ACI Publication SP-17(73), Commentary on Flexure.

F-3.1 BASIC RELATIONSHIPS for Moment and Axial Force.



CROSS SECTION	STRAIN DIAGRAM	STRESS DIAGRAM	APPLIED FORCE AND MOMENT (Force at center of gravity of slab, axial compression is positive.)	EQUIVALENT EQUILIBRIUM FORCES

$C = (0.85\ f'_c)\ a\ b$

$T = A_s f_y$

With N as shown (for tension use -N):
The external equivalent moment is

$$M_{us} = M_u + N_u\ (d-\frac{h}{2}) \qquad \text{(F-1)}$$

From the strain diagram:

$$c_b = d\left[\frac{\varepsilon_c}{\varepsilon_c + \frac{f_y}{E_s}}\right] = 0.685d \text{ for } f_y = 40{,}000,\ \varepsilon_c = 0.003,\ E_s = 29000000 \qquad \text{(F-2)}$$

From the stress diagram:

$$j_u d = d - \frac{\beta_1 c_b}{2}. \qquad \text{(F-3)}$$

The internal moment is

$$\frac{M_{us}}{\phi} = (j_u d) \left[b \quad a \ (0.85 f'_c) \right] \text{ for concrete} \qquad \text{(F-4)}$$

Where $a = \beta_1 c_b$ for balanced design, and

$$\frac{M_u}{\phi} = j_u d \left[A_{s_{balanced}} \ f_y \right] \text{ for steel (excluding N)} \qquad \text{(F-5)}$$

F-3.2 DESIGN

F-3.2.1 Concrete thickness for strength. - The expression for minimum effective depth "d" is derived by substituting equations F-1 through F-3 into equation F-4 and solving for d, as shown in Addendum A to Exhibit F.

$$d = f(\phi, b, \beta_1, f'_c, \varepsilon_c, E_s, M_u, N_u, d'') \qquad \text{(F-6)}$$

To account for the R factor defined in paragraph F-2e, an adjustment must be made in the application of Equation F-6. This adjustment is described in the following discussion. The effective depth "d" and the reinforcing steel area "A_s" must be sufficient to resist the applied moment and axial force. However, the ACI code limits the allowable steel area to a fraction "R" of the steel area for a balanced design. These two conditions can be satisfied either by determining the effective depth for an increased moment equal to M_{us}/R or by going into equation F-6 with an "effective width" equal to b times R.

F-3.2.2 Steel reinforcement

a. The area of steel required for balanced design, with the R factor added to get the maximum reinforcement allowed is

$$A_{s_{max}} = \frac{M_{us} R}{\left[d - \frac{\beta_1 c_b}{2} \right] f_y \phi} - \frac{N_u}{\phi f_y}, \qquad \text{(F-7)}$$

which will be strong enough because R was included when equation F-6 was used.

b. Since the actual value of d will, in general, be greater than the theoretical value, the full $A_{s_{max}}$ is excessive. The actual need is based on (1) calculating a from the concrete strength based on actual values of M_{us}, N_u, and d, and then (2) using this actual value of a instead of $\beta_1 c_b$ in equation F-7:

(1) Get a:

$$M_u + N_u \left(d - \frac{h}{2}\right) = M_{us} = C(j_u d)\phi \ ;$$

substituting $C = (0.85\ f'_c)\ a\ b$

and $(j_u d) = d - \frac{a}{2}$

to get

$$\left[\frac{.85 f'_c\ b\phi}{2} \right] a^2 - \left[.85 f'_c\ b\ \phi\ d \right] a + \left[M_u + N_u \left(d - \frac{h}{2}\right) \right] = 0,$$

using the absolute value of M_u and N_u as + for compression. Then solve for a as the smaller real, positive root of the quadratic equation.

(2) Get A_s:

$$A_s = \frac{M_u + N_u\,(d - \frac{h}{2})}{\phi f_y \left[d - \frac{a}{2}\right]} - \frac{N_u}{\phi f_y} \qquad \text{(F-8)}$$

This must not be greater than the value from equation F-7 or the value of the expression in paragraph F-2f. It must not be less than the value of the expression in paragraph F-2g.

F-3.3 ANALYSIS

F-3.3.1 The concrete resisting moment in flexure can be calculated using equation F-4 with equations F-2 and F-3, using $\left[\beta_1 \quad c_b\right]$ for a:

$$M_{us}\ \text{(available)} = (d - \frac{\beta_1 c_b}{2}) \left[0.85\ f'_c\ b\ \beta_1 c_b\right], \qquad \text{(F-9)}$$

with c_b calculated using equation F-2

F-3.3.2 The steel resisting moment, which must be *less than the concrete* resisting moment by at least the ratio R from paragraph F-2e, can be calculated by substituting equation F-1 into equation F-8, using the relationship

$$M_{us}\ \text{(available)} = (j_u d)\ (\phi f_y A_s + N_u) \qquad \text{(F-10)}$$

F-4 SHEAR

a. The actual shear stress can be calculated with equation 11-3 from ACI 318-71:

$$\upsilon_u = \frac{V_u}{\phi b_w d}$$

b. The allowable shear stress is calculated from equation 11-8 in ACI 318-71 for sections with axial load (tension is entered with a negative sign):

$$\upsilon_c = 2\ (1+\frac{N_u}{500A_g})\ \sqrt{f'_c}$$

which is ACI 318-71 paragraph 11.4.1 when N_u=0.

Equation 11-4 in ACI 318-71 is used for sections without axial load:

$$\upsilon_c = 1.9\sqrt{f'_c} + 2500\ \frac{A_s}{b_w d}\ \frac{V_u d}{M_u}\text{, but not over } 3.5\sqrt{f'_c}.$$

(In this equation, ($V_u d$) shall not exceed M_u.)

F-5 DEVELOPMENT OF REINFORCEMENT

Development of reinforcement is the same as for Working Stress Design (Exhibit E, par. E-5), except that the computed shear is V_u and is not multiplied by 2.0.

Addendum A: Derivation of Equation for d

Given $M_{us} = (j_u d)\left[(b\beta_1 c_b)(0.85\ f'_c)\right]\phi$ (F-4)

with $C_b = \dfrac{d\epsilon_c}{\epsilon_c + \dfrac{f_y}{E_s}} = \dfrac{d\epsilon_c}{\dfrac{\epsilon_c E_s + f_y}{E_s}} = \dfrac{d\epsilon_c E_s}{\epsilon_c E_s + f_y}$ (F-2)

and $j_u d = d - \dfrac{\beta_1 c_b}{2}$, (F-3)

derive equation F-6:

1. Substitute equation F-3 into equation F-4 and simplify:

$$M_{us} = \left(d - \frac{\beta_1 c_b}{2}\right) b\beta_1 c_b\ 0.85f'_c\phi$$

$$= \frac{d\phi b\beta_1 c_b\ 0.85f'_c}{1} - \frac{\beta_1^2\ c_b^2 b\ 0.85f'_c\phi}{2}$$

$$2M_{us} = \phi 2db\beta_1 c_b\ 0.85f'_c - \beta_1^2 c_b^2\ b\ 0.85f'_c\phi$$

2. Substitute equation F-2 into the equation above:

$$2M_{us} = \phi 2db\beta_1\ 0.85\ f'_c\left[\frac{d\epsilon_c E_s}{\epsilon_c E_s + f_y}\right] - \phi b\beta_1^2\ 0.85f'_c\left[\frac{d\epsilon_c E_s}{\epsilon_c E_s + f_y}\right]^2$$

$$2M_{us} = \frac{2\phi b\beta_1\ 0.85f'_c\ \epsilon_c E_s}{\epsilon_c E_s + f_y}\ d^2 - \frac{\phi b\beta_1^2\ 0.85f'_c\ \epsilon_c^2 E_s^2}{(\epsilon_c E_s + f_y)^2}\ d^2$$

$$2M_{us} = \frac{\phi b\beta_1\ 0.85f'_c\ \epsilon_c E_s\left[2(\epsilon_c E_s + f_y) - \beta_1\epsilon_c E_s\right]}{(\epsilon_c E_s + f_y)^2}\ d^2$$

$$d^2 = \frac{2M_{us}\ (\epsilon_c E_s + f_y)^2}{\phi b\beta_1\ 0.85f'_c\ \epsilon_c E_s\left[2(\epsilon_c E_s + f_y) - \beta_1\epsilon_c E_s\right]}$$

3. Substitute equation F-1 for M_{us} with h = d+d":

$$d^2 = \frac{2\left[M_u + N_u\left(d - \frac{d+d''}{2}\right)\right](\epsilon_c E_s + f_y)^2}{\phi b\beta_1\ 0.85f'_c\ \epsilon_c E_s\left[2(\epsilon_c E_s + f_y) - \beta_1\epsilon_c E_s\right]}$$

$$1 = \frac{\left[M_u + \tfrac{1}{2}N_u d - \tfrac{1}{2}N_u d''\right] (\varepsilon_c E_s + f_y)^2 \; 2}{\phi b \beta_1 0.85 f'_c \varepsilon_c E_s \left[2(\varepsilon_c E_s + f_y) - \beta_1 \varepsilon_c E_s\right] d^2}$$

$$1 = \frac{\left[M_u - \tfrac{1}{2}N_u d''\right] (\varepsilon_c E_s + f_y)^2 \; 2}{\phi b \beta_1 \; 0.85 f'_c \; \varepsilon_c E_s \left[2(\varepsilon_c E_s + f_y) - \beta_1 \varepsilon_c E_s\right] d^2} + \frac{N_u (\varepsilon_c E_s + f_y)^2}{\phi b \beta_1 (0.85) f'_c \; \varepsilon_c E_s \left[2(\varepsilon_c E_s + f_y) - \beta_1 \varepsilon_c E_s\right] d}$$

4. Substituting $\text{DDEN} = \dfrac{\phi b \beta_1 (0.85) f'_c \varepsilon_c E_s \left[2(\varepsilon_c E_s + f_y) - \beta_1 \varepsilon_c E_s\right]}{(\varepsilon_c E_s + f_y)^2}$:

$$0 = \frac{\left[M_u - \tfrac{1}{2}N_u d''\right] 2}{\cancel{\text{DDEN}} \; d^2} + \frac{N_u}{\cancel{\text{DDEN}} \; d} - 1 \; * \; \text{DDEN}$$

$$0 = \frac{\left[M_u - \tfrac{1}{2}N_u d''\right] \; 2}{1} + \frac{N_u d}{1} - \text{DDEN } d^2$$

5. Change signs and rearrange:

$$\text{DDEN } d^2 - N_u d - \left[M_u - \tfrac{1}{2} N_u d''\right] = 0$$

6. Solve quadratic for roots of d.

7. Keep the smallest real, positive root as the value of d.

EXHIBIT G: COMPARISON OF BOUSSINESQ AND BOWLES SOLUTIONS FOR PRESSURES ON RETAINING WALLS DUE TO SURCHARGE LOADS

G-1 SOURCE - This comparison was made by Dr. W. P. Dawkins on 20 June 1977.

G-2 USE OF BOUSSINESQ THEORY-VERTICAL P TIMES K - The comparisons are based on the premise that horizontal pressures doe to surcharges could be obtained by multiplying the vertical pressure predicted by the Boussinesq equation by the coefficient of active earth pressure. The Boussinesq solutions are given by Eqns 67, page 87, for a line load, and Eqns (c), page 363, for a point load, in <u>Theory of Elasticity</u> by Timoshenko and Goodier.

G-3 USE OF BOWLES' EQUATIONS - The quations for the Bowles solution are given in Fig. 11-20(a), page 356, for a point load, and in Fig. 11-21 page 357, for a line load, in <u>Foundation Analysis and Design</u>, second edition, by Bowles.

G-4 PRESSURES - Pressure distributions for the two solutions are shown in Figs. G-1 and G-2. The Boussinesq solutions are plotted for $K_a = 1.0$. Pressures for the point load cases are those which occur at points on the wall nearest to the point load.

G-5 RESULTANT FORCES and their points of action on the wall are shown in Fig. G-3 for each of the pressure distributions. For the point loads, pressures are assumed to be constant over a unit length of the wall.

G-6 COMPARISON - For both line and point load cases, the Bowles equations give larger resultant forces, with the point of action of the resultant being higher on the wall than the Boussinesq solutions for loads at a distance greater than two tenths of the wall height.

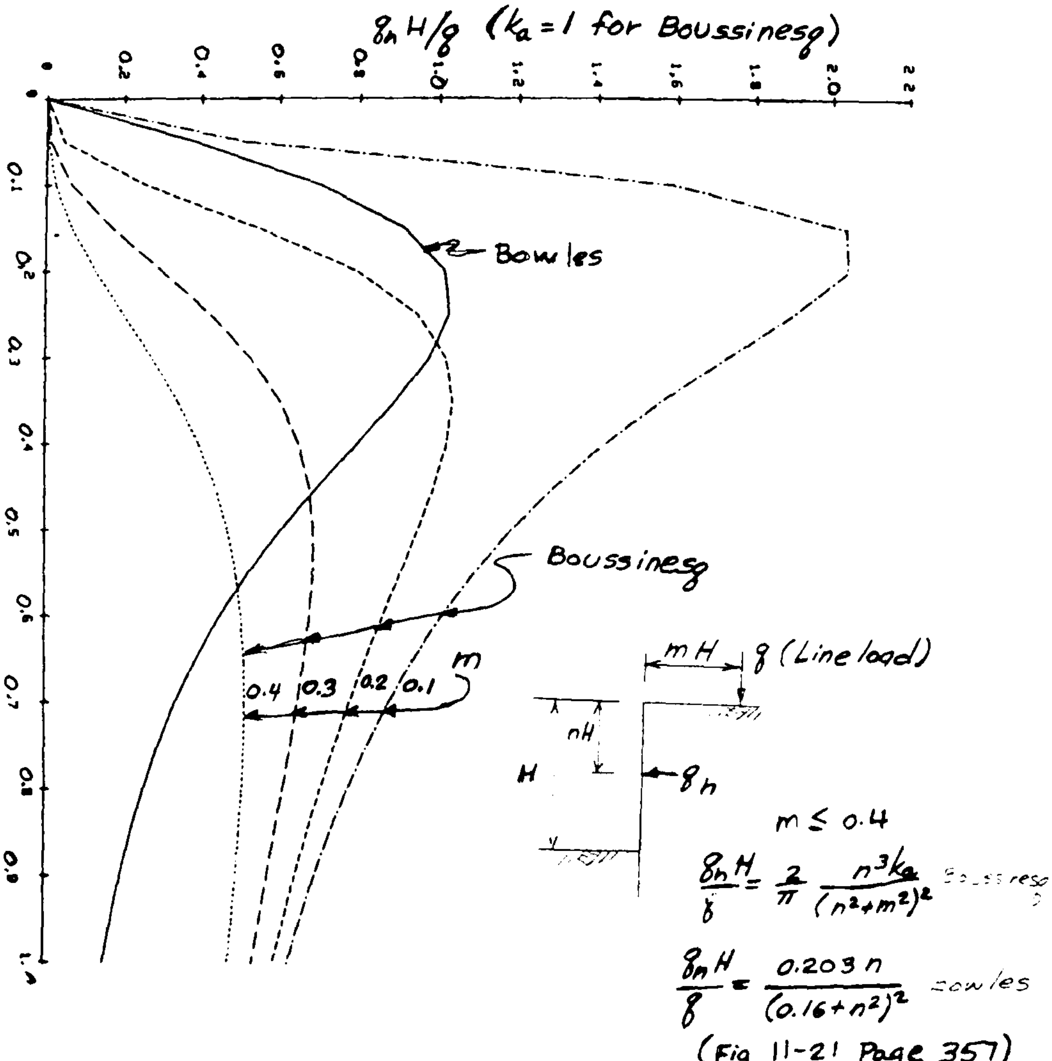
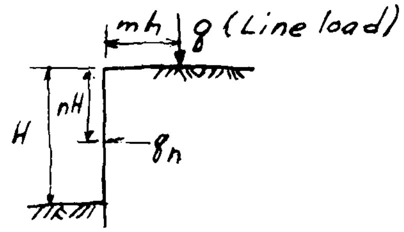


Fig. G-1a Line Load



$m > 0.4$

$$\frac{q_n H}{q} = \frac{2}{\pi} \frac{n^3 k_a}{(n^2+m^2)^2} \quad \text{Boussinesq}$$

$$\frac{q_n H}{q} = \frac{4}{\pi} \frac{m^2 n}{(n^2+m^2)^2} \quad \text{Bowles}$$

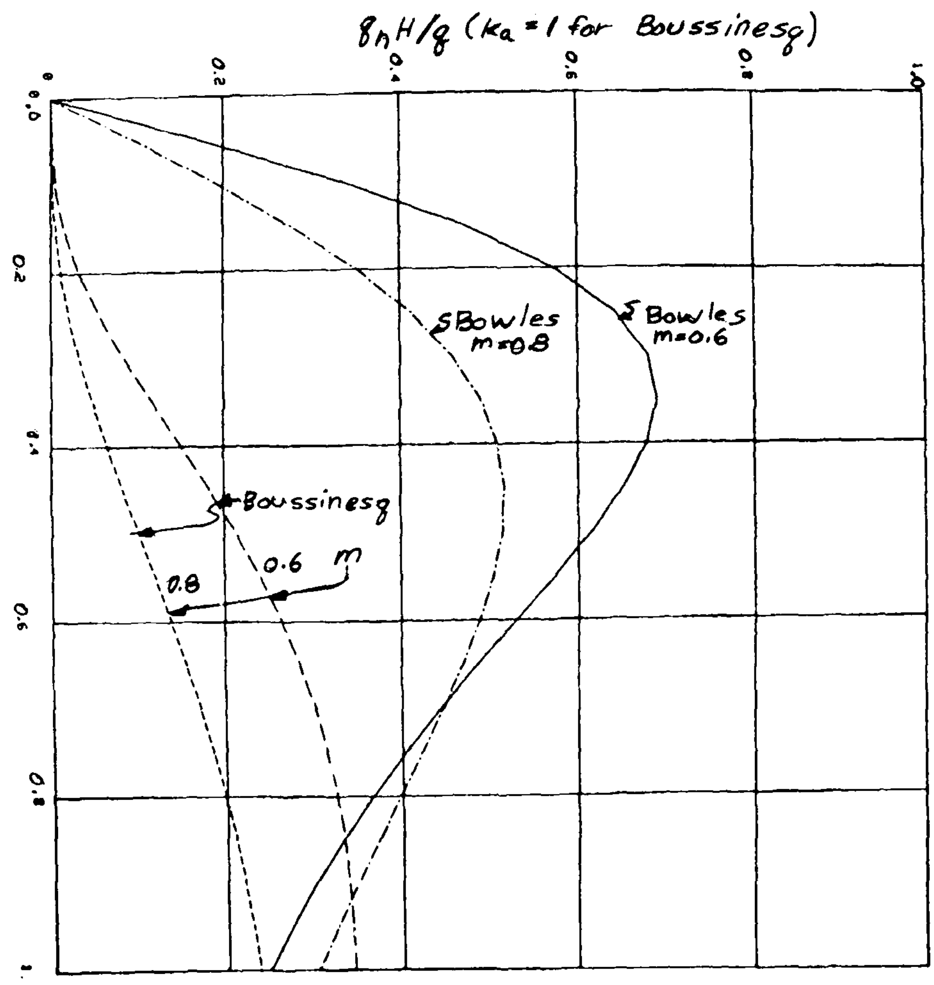
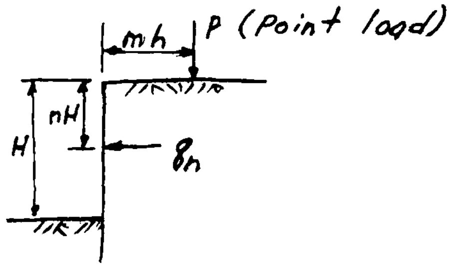


Fig. G-1b Line Load



$m \leq 0.4$

$$\frac{\sigma_n H^2}{P} = \frac{3}{2\pi} \frac{n^3 K_a}{(n^2+m^2)^{5/2}} \quad \text{Boussinesq}$$

$$\frac{\sigma_n H^2}{P} = 0.28 \frac{n^2}{(0.16+n^2)^3} \quad \text{Bowles}$$

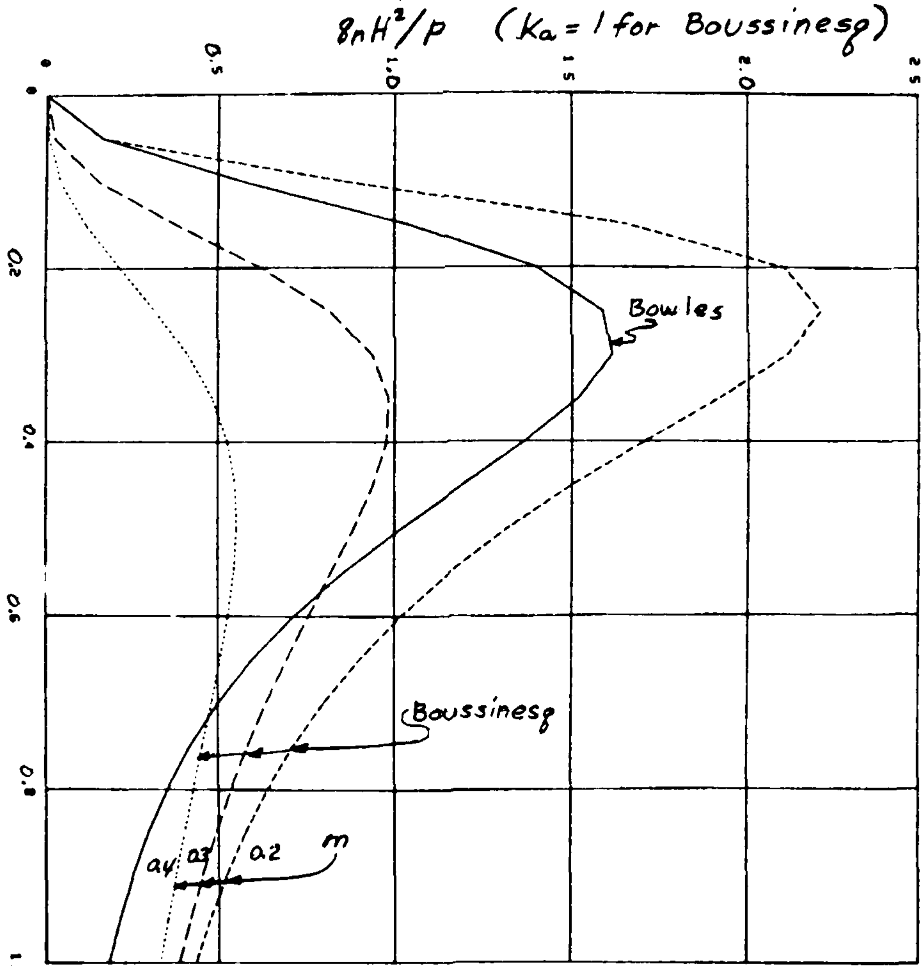
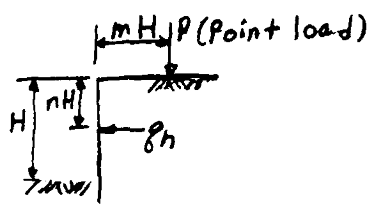


Fig. G-2a Point Load



$m > 0.4$

$$\frac{q_n H^2}{P} = \frac{3}{2\pi}\,\frac{n^3 k_a}{(n^2+m^2)^{5/2}} \quad \text{Boussinesq}$$

$$\frac{q_n H^2}{P} = 1.77\,\frac{m^2 n^2}{(n^2+m^2)^3} \quad \text{Bowles}$$

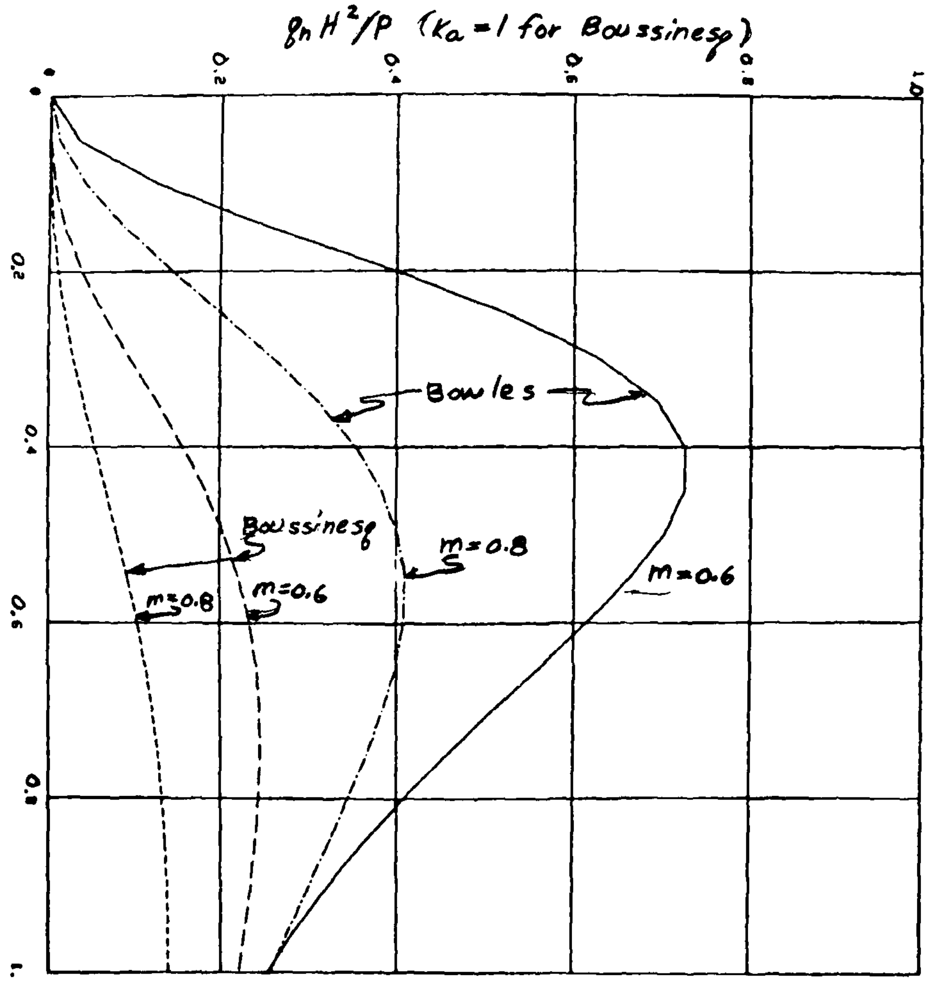
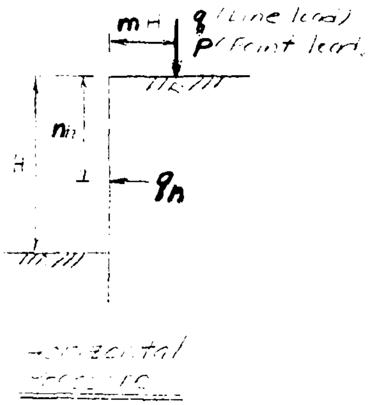
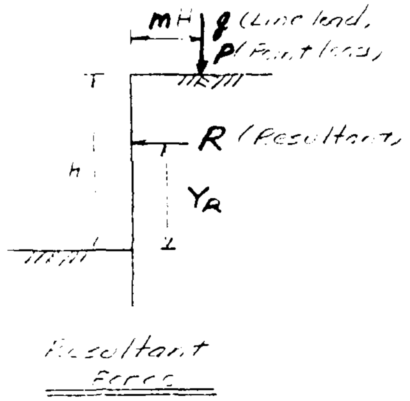


Fig. G-2b - Point Load





$$R = \int_H q_h \, dH$$

Line load: $\bar{R} = q_h / q$

Point load: $\bar{R} = q_h H / P$

$\bar{Y} = Y_R / H$

	Line Load				Point Load			
	Boussinesq‡		Bowles		Boussinesq‡		Bowles	
m	$\bar{R}$	$\bar{Y}$	$\bar{R}$	$\bar{Y}$	$\bar{R}$	$\bar{Y}$	$\bar{R}$	$\bar{Y}$
0.1	1.156	0.568			—	—		
0.2	0.731	0.471	*	*	1.128	0.570	*	*
0.3	0.502	0.409			0.616	0.482		
0.4	0.356	0.366	0.547	0.608	0.372	0.421	0.788	0.587
0.6	0.189	0.310	0.468	0.519	0.157	0.344	0.457	0.476
0.8	0.105	0.276	0.388	0.464	0.073	0.299	0.277	0.407

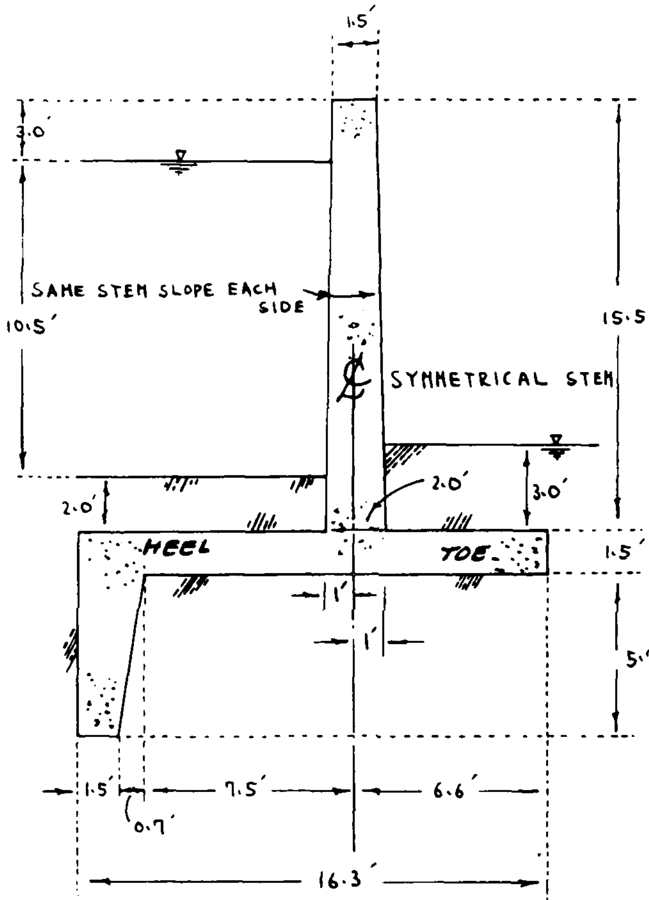
‡ For $k_a = 1.0$

* Bowles solution constant for $m \le 0.4$

Fig. G-3 Resultants

SUBJECT EXHIBIT H SAMPLE RETAINING WALL SHEAR FRICTION SLIDING ANALYSIS	COMPUTED BY CHECKED BY	DATE DATE	FILE NO. SHEET NO. 1

Exhibit H illustrates the shear friction method for SLIDING ANALYSIS AS GIVEN IN ETL 1110-2-184, the default method for retaining walls



$\gamma_c = 0.150$ kcf

$\gamma_w = 0.0625$ kcf

$\gamma_{sat} = 0.125$ kcf

$\phi = 20°$

$c = 0.35$ tsf $= 0.7$ ksf

$k_A = \tan^2(45 - \phi/2)$ *

$k_P = \tan^2(45 + \phi/2)$ *

* COHESION EFFECT NOT SHOWN HERE

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EXHIBIT H

SAMPLE RETAINING WALL
SHEAR FRICTION SLIDING ANALYSIS

$H_1 = H_{1a} + H_{1b} + H_{1c} = 11.316$ k/ft

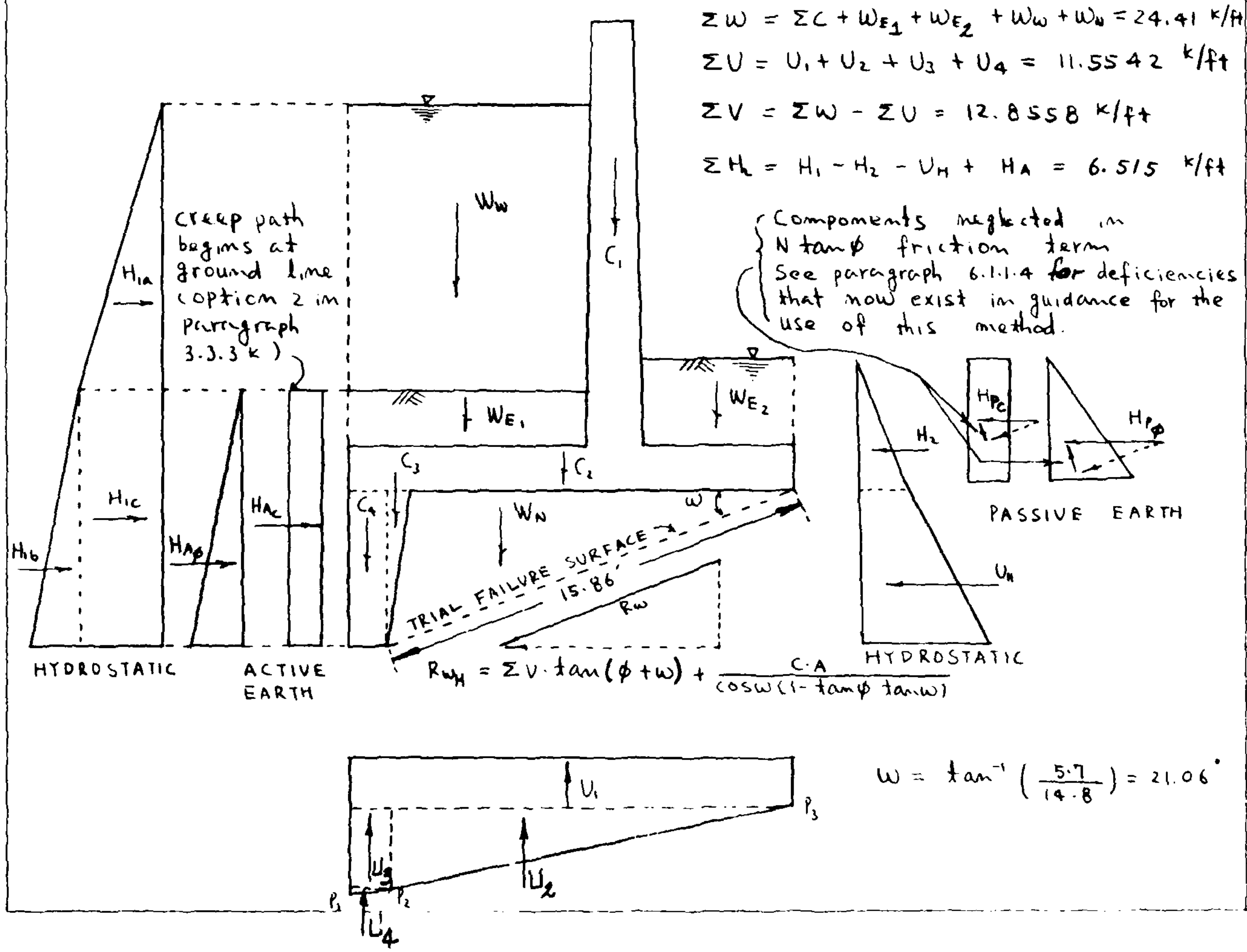
$\Sigma W = \Sigma C + W_{E_1} + W_{E_2} + W_W + W_N = 24.41$ k/ft

$\Sigma U = U_1 + U_2 + U_3 + U_4 = 11.5542$ k/ft

$\Sigma V = \Sigma W - \Sigma U = 12.8558$ k/ft

$\Sigma H_L = H_1 - H_2 - U_H + H_A = 6.515$ k/ft

Components neglected in N tan ϕ friction term
See paragraph 6.1.1.4 for deficiencies that now exist in guidance for the use of this method.



$$R_{WH} = \Sigma V \cdot \tan(\phi + \omega) + \frac{C \cdot A}{\cos\omega(1 - \tan\phi \tan\omega)}$$

$$\omega = \tan^{-1}\left(\frac{5.7}{14.8}\right) = 21.06^\circ$$

SHEET NO. 2

WES FORM NO. 1253
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SUBJECT EXHIBIT H SAMPLE RETAINING WALL SHEAR FRICTION SLIDING ANALYSIS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 3

CALCULATE HYDROSTATIC UPLIFT :

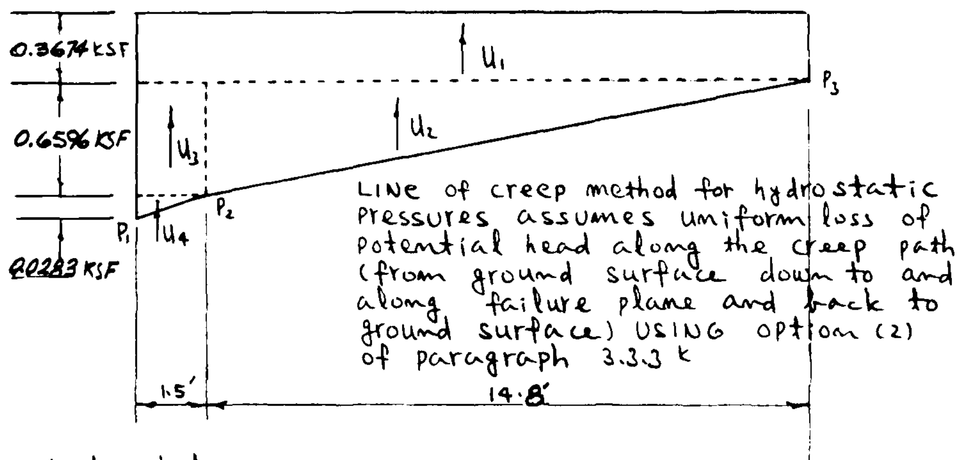
Net Head $(\Delta H) = 15.5 - 3 - 3 = 9.5$ FT

Creep path $(L_c) = 9.2 + 1.5 + \sqrt{14.8^2 + 5.7^2} + 1.5 + 3.0$

$= 9.2 + 1.5 + 15.86 + 1.5 + 3.0$

$= 31.06$ FT

$$\frac{\Delta H}{L_c} = \frac{9.5}{31.06} = 0.306 = \text{hydraulic gradient}$$



Hydrostatic pressures:

P @ base of H1a $= 0.0625(10.5) = 0.65625$ ksf

$P_1 = 0.65625 + 0.0625(9.2) - 0.306(9.2)(0.0625) = 1.0553$ ksf

$P_2 = 1.0553 - 1.5(0.306)(0.0625) = 1.027$ ksf

$P_3 = 1.027 - (0.0625)(5.7) - (0.306)(15.86)(0.0625) = 0.3674$ ksf

P @ ground over toe $= 0.3674 - 4.5(0.0625) - 0.306(4.5)(0.0625)$

≈ 0

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT H SAMPLE RETAINING WALL SHEAR FRICTION SLIDING ANALYSIS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 4

UPLIFT FORCES:

$U_1 = 0.3674\ (16.3) = 5.9886\ ^k/ft$

$U_2 = (1.027 - 0.3674)(14.8)(\frac{1}{2})(\cos 21.06) = 4.555\ ^k/ft$

$U_3 = (1.027 - 0.3674)(1.5) = 0.9894\ ^k/ft$

$U_4 = (1.0553 - 1.027)(1.5)(1/2) = 0.0212\ ^k/ft$

$\Sigma U = 11.5542\ ^k/ft$

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SUBJECT	COMPUTED BY	DATE	FILE NO.
EXHIBIT H SAMPLE RETAINING WALL SHEAR FRICTION SLIDING ANALYSIS	CHECKED BY	DATE	SHEET NO. 5

SYMBOL	FACTORS	FORCE →	FORCE ←	FORCE ↓	FORCE ↑
C_1	$\frac{1}{2}(1.5 + 2.0)(15.5)(0.15)$			4.07	
C_2	$1.5(16.3)(0.15)$			3.67	
C_3	$\frac{1}{2}(0.7)(5.7)(0.15)$			0.30	
C_4	$(1.5)(5.7)(0.15)$			1.28 9.32	
WE_1	$(2)(8.7)(0.125)$ $\frac{1}{2}(2)\left[(2)\left(\frac{0.25}{15.5}\right)\right](0.125)$			2.17 0.00 2.17	
WE_2	$(3)(0.125)(5.6)$ $\frac{1}{2}(3)\left[(3)\left(\frac{0.25}{15.5}\right)\right](0.125)$			2.10 0.01 2.11	
W_w above E_1	$(10.5)(0.0625)(8.73)$ $\frac{1}{2}(10.5)\left[(10.5)\left(\frac{0.25}{15.5}\right)\right](0.0625)$			5.73 0.06 5.79	
W_N	$\frac{1}{2}(14.8)(5.7)(0.125)$ $-\frac{1}{2}(0.7)(5.7)(0.125)$			5.27 -0.25 5.02	
$C + W_E + W_w + W_N$				24.41	
H_{1a}	$0.656(10.5)(\frac{1}{2})$	3.444			
H_{1b}	$(1.0553 - 0.656)(9.2)(\frac{1}{2})$	1.837			
H_{1c}	$0.656(9.2)$	6.035			
H_2	$0.3674(4.5)(\frac{1}{2})$		0.827		
U_H	$\frac{1.027 + 0.3674}{2}(5.7)$		3.974		
H_A	$\frac{0.0625(9.2)^2}{2}\left[\tan^2\left(45° - \frac{20°}{2}\right)\right]$ $-2(0.7)(9.2)\left[\tan\left(45° - \frac{20°}{2}\right)\right]$	1.297 -9.019	} 0 use because neglecting active pressure cannot exist		
$H_{1a} + H_{1b} + H_{1c} - H_2 - U_H + H_A$		6.515			

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REV. OCT 1968

SUBJECT EXIBIT H
SAMPLE RETAINING WALL
SHEAR FRICTION SLIDING ANALYSIS

COMPUTED BY | DATE | FILE NO.
CHECKED BY | DATE | SHEET NO. 6

$$H_{P_c} = 2ch\sqrt{K_p} = 2ch\sqrt{\tan^2(45^\circ + \phi/2)} = 2(0.7)(4.5)\sqrt{\tan^2(45^\circ + \phi/2)}$$

$H_{P_c} = 8.997$ KIPS

$$H_{P\phi} = \frac{1}{2}\gamma h^2 K_p = \frac{1}{2}\gamma h^2 \tan^2(45^\circ + \phi/2) = \frac{1}{2}(0.0625)(4.5)^2 \tan^2(45^\circ + 20/2)$$

$H_{P\phi} = 1.291$ KIPS

$\Sigma H_p = \Sigma H_{P_c} + \Sigma H_{P\phi} = 8.997 + 1.291 = 10.288$ KIPS

THE SHEAR-friction safety factor against sliding as given in ETL 1110-2-184, is obtained by dividing the horizontal sliding resistance (ΣR) obtained from resisting forces including those along the critical path (plane or combination of planes which offers the least resistance to sliding) by the summation of horizontal service loads (ΣH) to be applied to the structure.

$\Sigma V = 24.41 - 11.5542 = 12.8558^K$

$$\Sigma R = \Sigma V \cdot \tan(\phi + w) + \frac{C \cdot A}{\cos w[1 - \tan\phi \cdot \tan w]} + H_p$$

$$= 12.8558 \cdot \tan(20^\circ + 21.06^\circ) + \frac{(0.7)(15.86)}{\cos 21.06^\circ[1 - \tan 20^\circ \tan 21.06^\circ]} + 10.288$$

$$= 11.199 + 13.836 + 10.288$$

$$= 35.323 \text{ KIPS}$$

ΣH from sheet H-2

shear-friction safety factor against sliding along the total surface on sheet H-2:

$$F_s = \frac{\Sigma R}{\Sigma H} = \frac{35.323}{6.515} = 5.42$$

SUBJECT EXHIBIT H

SAMPLE RETAINING WALL
SHEAR FRICTION SLIDING ANALYSIS

COMPUTED BY | DATE | FILE NO.
CHECKED BY | DATE | SHEET NO. 9

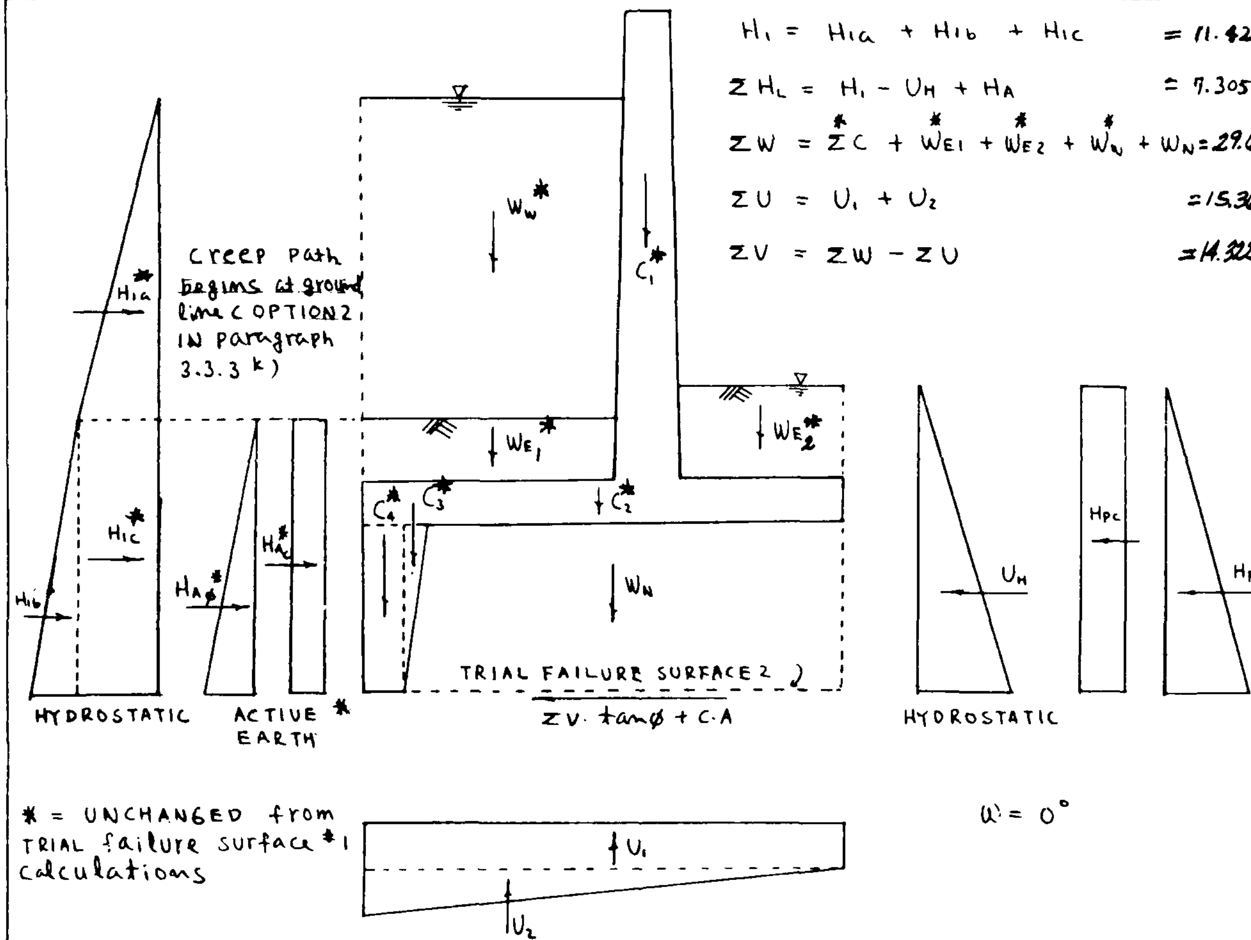
$H_1 = H_{1a} + H_{1b} + H_{1c} = 11.422 \text{ k/ft}$

$\Sigma H_L = H_1 - U_H + H_A = 7.305 \text{ k/ft}$

$\Sigma W = \overset{*}{\Sigma C} + \overset{*}{W_{E1}} + \overset{*}{W_{E2}} + \overset{*}{W_w} + W_N = 29.69 \text{ k/ft}$

$\Sigma U = U_1 + U_2 = 15.3677 \text{ k/ft}$

$\Sigma V = \Sigma W - \Sigma U = 14.3223 \text{ k/ft}$



$\omega = 0°$

* = UNCHANGED from TRIAL failure surface #1 calculations

WES FORM NO. 1253
REV OCT 1968

EXHIBIT H
SAMPLE RETAINING WALL
SHEAR FRICTION SLIDING ANALYSIS

SHEET 8

SYMBOL	FACTORS	FORCE →	FORCE ←	FORCE ↑	FORCE ↓
C_1	$\frac{1}{2}(1.5+2.0)(15.5)(0.15)$			4.07	
C_2	$1.5(16.3)(0.15)$			3.67	
C_3	$\frac{1}{2}(0.7)(5.7)(0.15)$			0.30	
C_4	$(1.5)(5.7)(0.15)$			1.28	
				9.32	
W'_{E_1}	$(2)(8.7)(0.125)$			2.17	
	$\frac{1}{2}(2)\left[(2)\left(\frac{0.25}{15.5}\right)\right](0.125)$			0.00	
				2.17	
W_{E_2}	$(3)(0.125)(5.6)$			2.10	
	$\frac{1}{2}(3)\left[(3)\left(\frac{0.25}{15.5}\right)\right](0.125)$			0.01	
				2.11	
W_w above E_1	$(10.5)(0.0625)(8.73)$			5.73	
	$\frac{1}{2}(10.5)\left[(10.5)\left(\frac{0.25}{15.5}\right)\right](0.0625)$			0.06	
				5.79	
W_N	$(0.125)(14.8)(5.7)$			10.55	
	$-(0.125)(0.7)(5.7)(0.5)$			-0.25	
				10.30	
	$C + W_E + W_w + W_N$			29.69	
H_{1a}	$0.656(10.5)(\frac{1}{2})$	3.444			
H_{1b}	$(1.0783-0.656)(9.2)(\frac{1}{2})$	1.943			
H_{1c}	$0.656(9.2)$	6.035			
U_H	$(0.5)(10.2)(0.8073)$		4.117		
H_A	$\frac{0.0625(9.2)^2}{2}\left[\tan^2\left(45^\circ-\frac{20^\circ}{2}\right)\right]$	1.299 } 0			
	$-2.0(0.7)(9.2)\left[\tan\left(45^\circ-\frac{20^\circ}{2}\right)\right]$	-9.019			
$H_{1a}+H_{1b}+H_{1c}-U_H+H_a =$		7.305			

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT H SAMPLE RETAINING WALL SHEAR FRICTION SLIDING ANALYSIS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 9

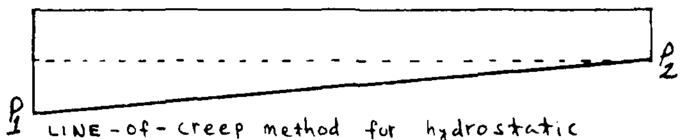
CALCULATE HYDROSTATIC UPLIFT :

Net Head (ΔH) = 15.5 − 3 − 3 = 9.5′

Creep path (L_c) = 9.2 + 16.3 + 5.7 + 1.5 + 3.0

= 35.7 FT.

$$\frac{\Delta H}{L_c} = \frac{9.5}{35.7} = 0.266 = \text{hydraulic gradient}$$



LINE-of-creep method for hydrostatic pressures assumes uniform loss of potential head along the creep path (from ground surface down to and along failure plane and back to ground surface) USING option (2) of paragraph 3.3.3 k

16.3′

Hydrostatic Pressures :

P @ base of H_{1a} = 0.0625 (10.5) = 0.65625 ksf

P_1 = 0.65625 + 0.0625 (9.2) − 0.266 (9.2)(0.0625) = 1.0783

P_2 = 1.0783 − 0.266 (16.3)(0.0625) = 0.8073

UPLIFT FORCES :

U_1 = (0.8073)(16.3) = 13.1590 k/′

U_2 = (0.5)(16.3)(0.271) = 2.2087 k/′

ΣU = 15.3677 k/′

WES FORM NO. 1253 REV. OCT 1968

EXHIBIT H SAMPLE RETAINING WALL SHEAR FRICTION SLIDING ANALYSIS	COMPUTED BY CHECKED BY	DATE DATE	SHEET NO. 10

the forces which change due to checking trial failure surface 2 is W_N, the resistance forces, driving forces affected by changes in uplift forces over new creep path, and uplift forces (verticals).

$$W_N = (0.125)(14.8)(5.7) = 10.55$$
$$- (0.125)(0.7)(5.7)(0.5) = -\ 0.25$$
$$= 10.30$$

$$\Sigma V = \Sigma C + W_{E1} + W_{E2} + W_w + W_N - U$$
$$= 9.32 + 2.17 + 2.11 + 5.79 + 10.30 - 15.3677$$
$$= 14.32 \text{ KIPS}$$

$$H_{p_\phi} = \tfrac{1}{2}(10.2)^2(0.0625)(\tan^2(45° + 20/2))$$
$$= 3.25\ (2.04) = 6.63 \text{ KIPS}$$

$$H_{p_c} = 2\ (0.7)(10.2)\sqrt{\tan^2(45° + 20°/2)}$$
$$= 20.39 \text{ KIPS}$$

$$H_p = H_{p\phi} + H_{pc} = 6.63 + 20.39 = 27.02 \text{ KIPS}$$

$$\Sigma R = \Sigma V \cdot \tan\phi + C \cdot A + H_p$$
$$= 14.32 \tan 20° + 0.7\ (14.8) + 27.02$$
$$= 5.212 + 10.36 + 27.02$$
$$= 42.592$$

shear friction safety factor against sliding (Trial failure surface 2) $= S.F = \dfrac{\Sigma R}{\Sigma H} = \dfrac{42.592}{7.305}$

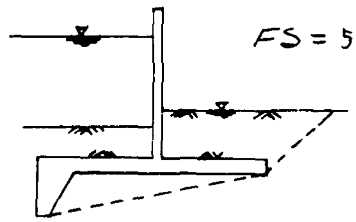
$$= 5.83$$

WES FORM NO. 1253
REV ... 1968

SUBJECT: EXHIBIT H SAMPLE RETAINING WALL SHEAR FRICTION SLIDING ANALYSIS	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO. 11

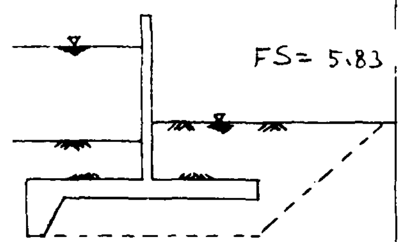
SUMMARY

Failure Surface #1



FS = 5.42

Failure Surface #2



FS = 5.83

So, for these two limiting failure planes, FS = 5.42

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT H SAMPLE RETAINING WALL SHEAR FRICTION SLIDING ANALYSIS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO.

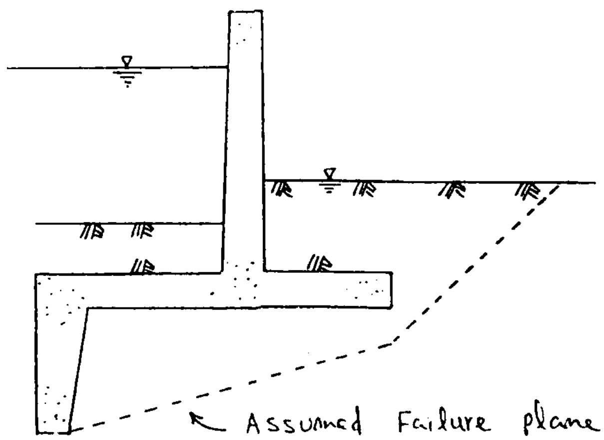
The total base-foundation interface of the T-wall may not be in compression with the foundation. If any part of the surface under consideration is along the base-foundation interface and is not in contact with the foundation, this portion should be neglected when obtaining the effective base area to resist sliding. However, if the assumed failure surface is not along the base-foundation interface but through the soil, no reduction in the area to resist sliding is made.

A portion of the base of a T-wall will not be in compression when the resultant falls outside the kern thus creating a crack which can result in an increase in uplift pressures. This condition will affect the sliding stability analysis when the assumed sliding plane acts along the soil-structure interface below the base of the wall (for this condition the program will have to recycle back through the line of creep calculations until the creep path assumptions match the final part of the base that is in contact

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT H SAMPLE RETAINING WALL SHEAR FRICTION SLIDING ANALYSIS	COMPUTED BY: CHECKED BY:	DATE: DATE:	FILE NO. SHEET NO.

with the foundation). For example, consider a wall without a key and with a horizontal base, when the resultant falls outside the kern and the assumed sliding plane is along the interface between the base of the structure and the soil foundation, uplift pressures will be computed assuming no creep loss for the portion of the foundation not in compression. For the condition where the resultant falls outside the kern but the assumed sliding plane is through the soil, for example a wall with a key positioned at the extreme end of the heel (see figure shown below), no increase in uplift pressure will be considered because the soil does not lift and form a crack as is the case at the soil-structure interface.



WES FORM NO. 1253
REV OCT 1968

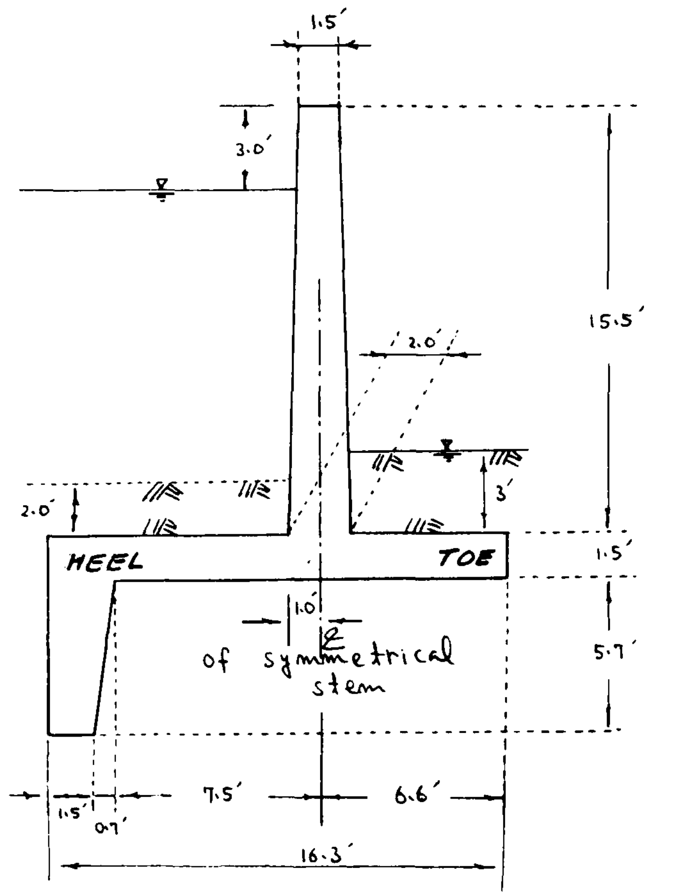
SUBJECT EXHIBIT H SAMPLE RETAINING WALL SHEAR FRICTION SLIDING ANALYSIS	COMPUTED BY: CHECKED BY:	DATE DATE	FILE NO. SHEET NO.

Another reason the uplift forces are not affected for this condition is because they are forces inside the soil-structure free body and therefore, do not affect the overall sliding stability.

The ϕ and C values should be consistent with the material being sheared. A plane of failure through the soil should use the ϕ and C of the soil. For any of the failure planes along the soil-structure interface, use the ϕ and C for sliding friction at the interface.

WES FORM NO. 1253
REV OCT 1968

EXHIBIT I SAMPLE FLOODWALL SLIDING ANALYSIS $\tan\phi' = \frac{\tan\phi}{FS}$ AND $c' = \frac{c}{FS + 2c'}$	DATE DATE	FILE NO. SHEET NO. 1



$\gamma_c = 150$ pcf $\phi = 20°$ $c = 0.35$ tsf $= 0.7$ ksf

$\gamma_w = 62.5$ pcf

$\gamma_{sat} = 125$ pcf $K_a = \tan^2(45° - \phi/2)$ $K_p = \tan^2(45° + \phi/2)$

While the flood wall manual, EM 1110-2-2501, does not provide a clear-cut procedure for computing the factor of safety against sliding, a method is presented here that complies with the provisions of this manual. This method is the program default procedure for flood walls.

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT I SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 2

$H_1 = H_{1a} + H_{1b} + H_{1c} = 11.316\ k/ft$

$\Sigma W = \Sigma C + W_{E1} + W_{E2} + W_W + W_N = 24.41\ k/ft$

$\Sigma U = U_1 + U_2 + U_3 + U_4 = 11.5542\ k/ft$

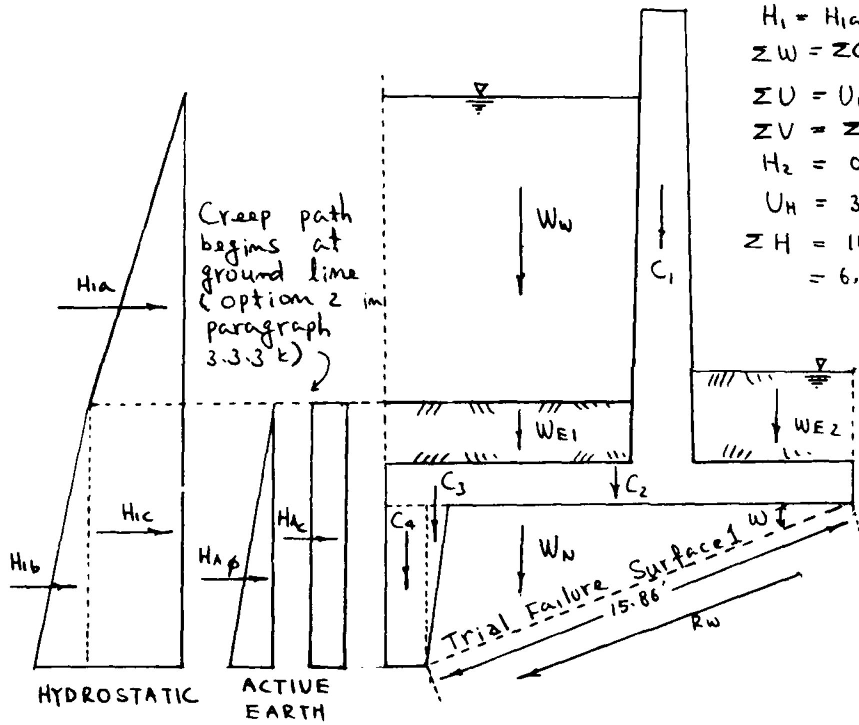
$\Sigma V = \Sigma W - \Sigma U = 12.8558\ k/ft$

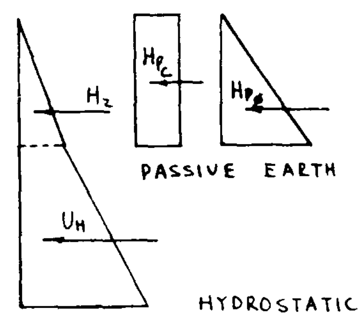
$H_2 = 0.827\ k/ft$

$U_H = 3.974\ k/ft$

$\Sigma H = 11.316 - 0.827 - 3.974 + H_A - H_P$

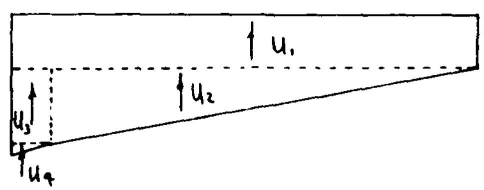
$= 6.515 + H_A - H_P$





$\omega = \tan^{-1}\left(\frac{5.7}{14.8}\right) = 21.06°$

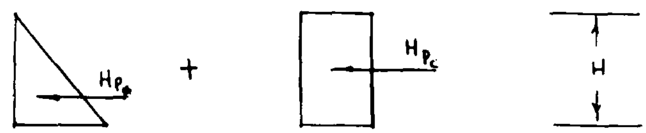
All values shown on this sheet except H_A and H_P are identical to values on sheet H-2



WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT I SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 3

PASSIVE EARTH RESISTANCE AT TOE OF WALL



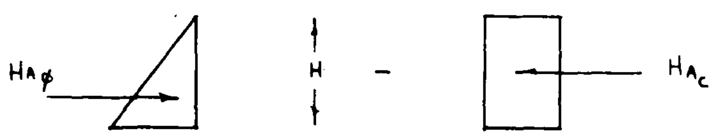
$$H_{p_w} = (H_{p_\phi} + H_{p_c}) \cos 21.06^\circ$$

$$= \left[\tfrac{1}{2}\gamma_b k_p \cdot H^2 + 2c' H\sqrt{k_p}\right] \cos 21.06^\circ$$

$$= \left[\tfrac{1}{2}(0.125 - 0.0625)\, k_p \cdot (4.5)^2 + 2c'(4.5)\sqrt{k_p}\right] \cos 21.06^\circ$$

$$= 0.5905\, k_p + 8.399\, c' \sqrt{k_p}$$

ACTIVE EARTH DRIVING FORCE AT HEEL OF WALL



$$H_{A_w} = (H_{A\phi} - H_{A_c}) \cos 21.06^\circ$$

$$= \tfrac{1}{2}\gamma_b k_A H^2 \cdot \cos 21.06^\circ - 2c' H\sqrt{k_A} \cos 21.06^\circ$$

$$= \tfrac{1}{2}(0.125 - 0.0625)\, k_A \cdot (9.2)^2 \cdot \cos 21.06^\circ$$

$$- 2 \cdot c' \cdot (9.2)\sqrt{k_A} \cdot \cos 21.06^\circ$$

$$= 2.4683\, k_A - 17.1710\, c' \sqrt{k_A}$$

WES FORM NO. 1252
REV OCT 1968

EXHIBIT I

SAMPLE FLOODWALL SLIDING ANALYSIS

4

NET APPLIED FORCE TENDING TO INDUCE SLIDING

$$\Sigma D_w = (H_1 - H_2 - V_H)\cos\omega + H_{A_w} \quad \text{(See sheet H-5 for } H_1, H_2 \text{ \& } V_H \text{ values)}$$

$$= (11.316 - 0.827 - 3.974)\cos 21.06 + 2.4683\,k_A - 17.1710\,c'\sqrt{k_A}$$

$$= 6.08 + 2.4683\,k_A - 17.1710\,c'\sqrt{k_A}$$

REACTION FORCES TENDING TO RESIST SLIDING

$$\Sigma R_w = H_{R_w} + H_{P_w} = [\Sigma V\cdot\cos\omega + \Sigma H\cdot\sin\omega]\cdot\tan\phi' + c'\cdot A + H_{P_w}$$

$$= \Big[12.8558\cdot\cos 21.06 + \{11.316 - 0.827 - 3.974\}\sin 21.06^\circ + \{\tfrac{1}{2}(0.125 - 0.0625)\cdot k_A\cdot(9.2)^2 - 2\cdot c'(9.2)\sqrt{k_A}\}\sin 21.06^\circ - \{\tfrac{1}{2}(0.125 - 0.0625)\cdot k_p\cdot(4.5)^2 + 2\cdot c'\cdot(4.5)\sqrt{k_p}\}\cdot\sin 21.06^\circ\Big]\cdot\tan\phi' + 15.86c' + 0.5905\,k_p + 8.399\cdot c'\sqrt{k_p}$$

$$= [14.338 + 0.9505\,k_A - 6.612\,c'\sqrt{k_A} - 0.2274\,k_p - 3.23411\,c'\sqrt{k_p}]\cdot\tan\phi' + 15.86c' + 0.5905\,k_p + 8.399\cdot c'\sqrt{k_p}$$

EQUILIBRIUM OF APPLIED AND REACTION FORCES CONSIDERING TRIAL FAILURE SURFACE 1

With $[\Sigma D_w = f_1(FS, FS + 2c')$ and $\Sigma R_w = f_2(FS, FS + 2c')]$, the equilibrium relationship $[\Sigma D_w = \Sigma R_w]$ becomes $[f_1(FS, FS + 2c') = f_2(FS, FS + 2c')]$ which can be transposed to read $[f_1(FS, FS + 2c') - f_2(FS, FS + 2c') = 0]$

The computation of FS involves solving the expression

$$\Sigma D_w - \Sigma R_w = 0$$

$$f_1(FS, FS + 2c') - f_2(FS, FS + 2c') = 0$$

by an iterative procedure, either graphically (by hand) or analytically (on the computer). The procedure includes (1) assuming a trial value of FS, (2) calculating allowable values of ϕ' and c', (3) calculating k_A and k_p from ϕ', (4) substituting c', k_A and k_p into ΣD_w and ΣR_w, and (5) either plotting $\Sigma D_w = f_1(FS, FS+2c')$ and $\Sigma R_w = f_2(FS, FS+2c')$ to get the the point of intersection or solving for FS to make $\Sigma D_w - \Sigma R_w = 0$

SUBJECT EXHIBIT I SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY:	DATE	FILE NO.
	CHECKED BY	DATE:	SHEET NO. 5

An example of the above procedure is as follows:

$$\phi' = \tan^{-1}\left(\frac{\tan\phi}{FS}\right) \qquad \text{where } \phi = 20^\circ$$

$$c' = \frac{c}{FS + 2q} \qquad \text{where } q = c', \quad c = 0.35 \text{ tsf}$$

the positive value of c' is

$$c' = \frac{\sqrt{FS^2 + 8c} - FS}{4} \qquad \text{where } c \text{ and } c' \text{ is in tsf}$$

For $FS = 1$

$$\phi' = \tan^{-1}\left(\frac{\tan 20^\circ}{1}\right)$$

$$= 20^\circ$$

$$c' = \frac{\sqrt{(1)^2 + 8(0.35)} - 1}{4} = 0.23734 \text{ tsf}$$

or 0.47468 ksf

$$\tan\phi' = \tan 20^\circ = 0.36397$$

$$k_A = \tan^2(45^\circ - \phi'/2) = \tan^2(45^\circ - 20^\circ/2) = 0.49029$$

$$k_P = \tan^2(45^\circ + \phi'/2) = \tan^2(45^\circ + 20^\circ/2) = 2.03961$$

$$\sqrt{k_P} = \sqrt{2.03961} = 1.42815$$

$$H_{A_w} = 2.4683\, k_A - 17.1710\, c'\sqrt{k_A}$$

$$= 1.21018 - 5.70720 = -4.50 \text{ (neglect if negative)}$$

$$ZD_w = 6.08 + 2.4683\, k_A - 17.1710\, c'\sqrt{k_A} = 6.08 + H_{A_w}$$

$$= 6.08 - \overset{\text{neglect}}{\cancel{4.50}} = 6.08$$

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT I SAMPLE FLOODWALL SLICING ANALYSIS	COMPUTED BY / CHECKED BY	DATE / DATE	FILE NO. / SHEET NO. 6

$$H_{R_\phi} = \left[14.338 + 0.9505\, k_A - 6.612\, c'\sqrt{k_A} - 0.2294\, k_P - 3.23411\, c'\sqrt{k_P}\right] \cdot \tan\phi' = 3.62$$

$$H_{R_c} = 15.86\, c' = 7.53$$

$$H_{P_\phi} = 0.5905\, k_P = 1.20$$

$$H_{P_c} = 8.399\, c'\sqrt{k_P} = 5.69$$

$$\Sigma R_w = H_{R_\phi} + H_{R_c} + H_{P_\phi} + H_{P_c} = 18.04$$

$$\Sigma R_w - \Sigma D_w = 18.04 - 6.08 = 11.96$$

Continue assuming safety factors and calculating $\Sigma R_w - \Sigma D_w$ until their difference is negligible. The system is then in equilibrium with the driving forces equal to the resisting forces and the safety factor which produces this condition is the safety factor against sliding.

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT I

SAMPLE FLOODWALL SLIDING ANALYSIS

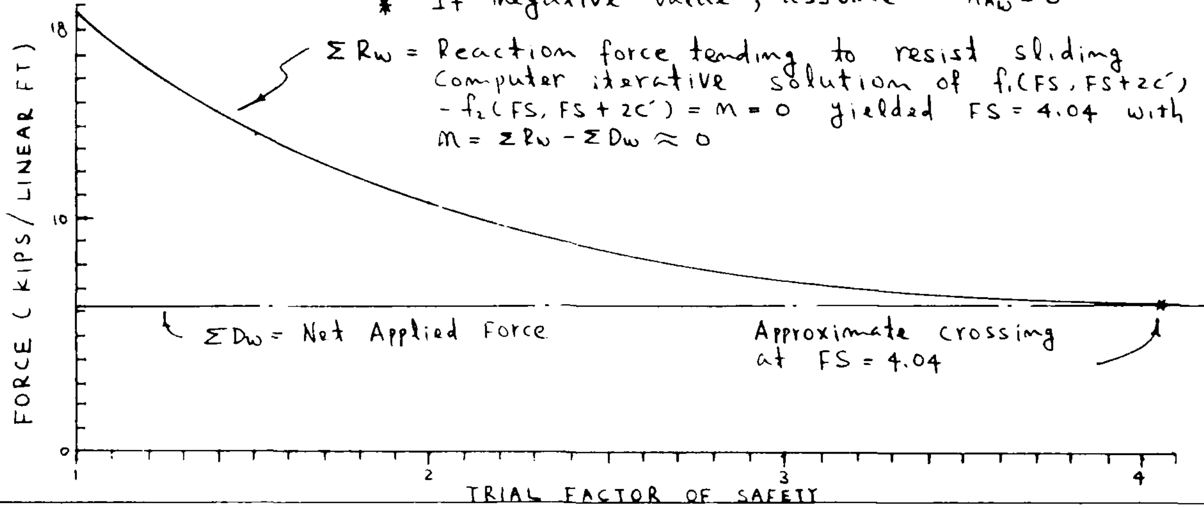
COMPUTED BY | DATE | FILE NO.

CHECKED BY: | DATE | SHEET NO. 7

GRAPHICAL SOLUTION

FS	ϕ'	$\tan\phi'$	k_A	k_P	$\sqrt{k_P}$	c'	H_{A_W}*	ΣD_W	H_{R_ϕ}	H_{R_c}	H_{P_ϕ}	H_{P_c}	ΣR_W	$\Sigma R_W - \Sigma D_W$
1.0	20.0	0.364	0.490	2.04	1.43	0.475	-4.50	6.08	3.62	7.53	1.20	5.69	18.04	11.96
1.5	13.64	0.243	0.618	1.62	1.27	0.374	-3.52	6.08	2.69	5.93	0.95	3.99	13.56	7.48
2.0	10.31	0.182	0.696	1.44	1.20	0.304	-2.63	6.08	2.15	4.82	0.85	3.06	10.88	4.80
3.0	6.92	0.121	0.785	1.27	1.13	0.218	-1.37	6.08	1.54	3.45	0.75	2.06	7.80	1.72
4.0	5.20	0.091	0.834	1.20	1.10	0.168	-0.58	6.08	1.21	2.66	0.71	1.54	6.12	0.04
4.01	5.19	0.091	0.834	1.20	1.09	0.168	-0.57	6.08	1.20	2.66	0.71	1.54	6.11	0.03
4.02	5.17	0.091	0.835	1.20	1.09	0.167	-0.56	6.08	1.20	2.65	0.71	1.54	6.10	0.02
4.03	5.16	0.090	0.835	1.20	1.09	0.167	-0.56	6.08	1.20	2.65	0.71	1.53	6.09	0.01
4.04	5.15	0.090	0.835	1.20	1.09	0.166	-0.55	6.08	1.20	2.64	0.71	1.53	6.08	0.0
4.10	5.07	0.089	0.838	1.19	1.09	0.164	-0.51	6.08	1.18	2.60	0.71	1.51	6.00	-0.08

* If negative value ; assume $H_{A_W} = 0$



WES FORM NO. 1253
REV OCT 1968

SUBJECT **EXHIBIT I**
SAMPLE FLOODWALL SLIDING ANALYSIS

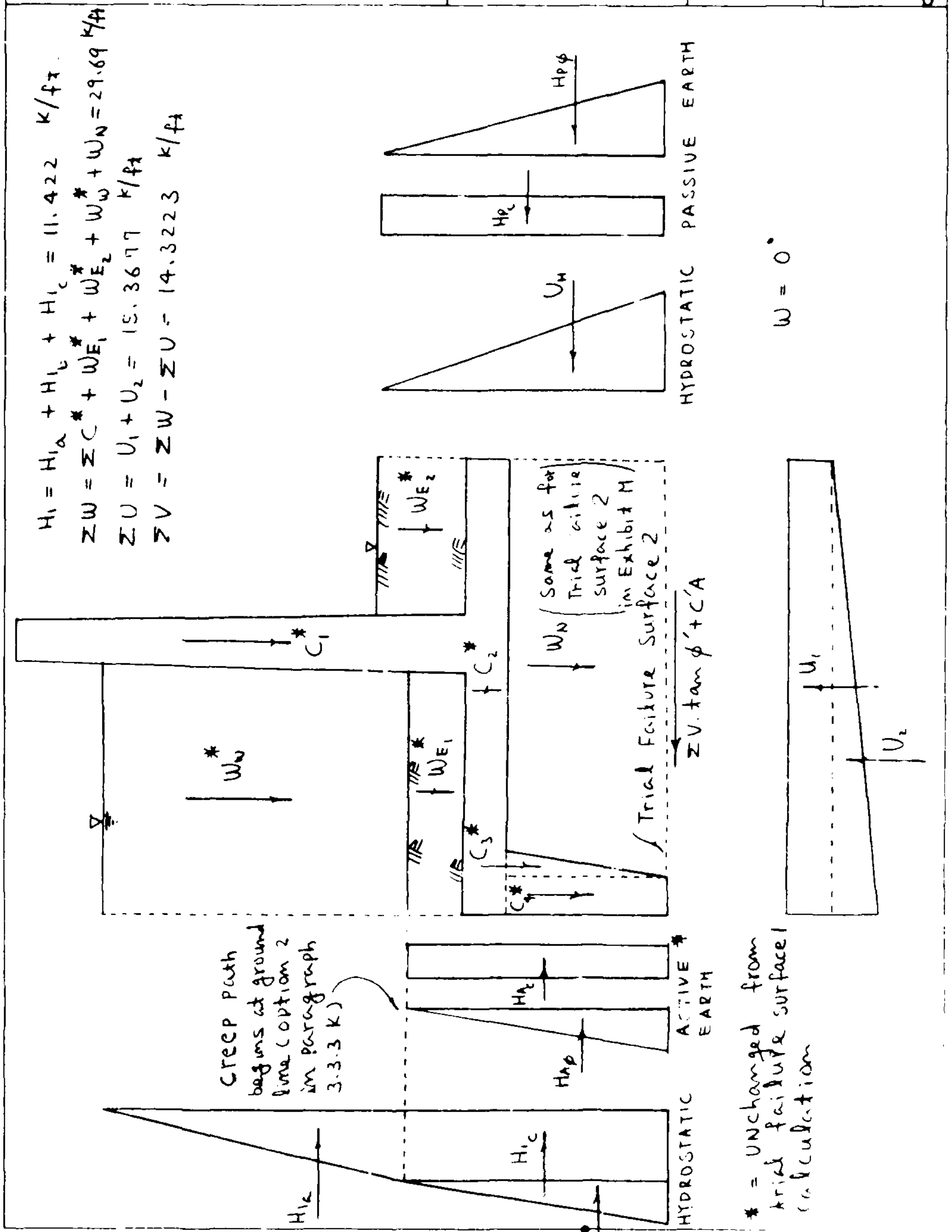
SUBJECT	COMPUTED BY	DATE	FILE NO.
EXHIBIT I SAMPLE FLOODWALL SLIDING ANALYSIS	CHECKED BY	DATE	SHEET NO. 8

$$H_1 = H_{1a} + H_{1b} + H_{1c} = 11.422 \ k/ft$$

$$\Sigma W = \Sigma C^* + W_{E_1}^* + W_{E_2}^* + W_w^* + W_N = 29.69 \ k/ft$$

$$\Sigma U = U_1 + U_2 = 15.3677 \ k/ft$$

$$\Sigma V = \Sigma W - \Sigma U = 14.3223 \ k/ft$$



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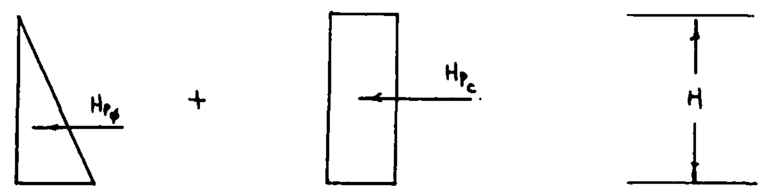
SUBJECT EXHIBIT I SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO. 9

PASSIVE EARTH RESISTANCE AT TOE OF WALL



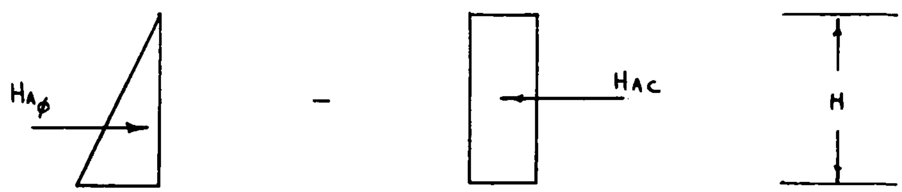
$$H_{p_w} = H_{p_\phi} + H_{p_c}$$

$$= \frac{1}{2} \gamma_b k_p H^2 + 2c' H \sqrt{k_p}$$

$$= \frac{1}{2} (0.125 - 0.0625) k_p (10.2)^2 + 2c' (10.2) \sqrt{k_p}$$

$$= 3.2513 \, k_p + 20.4 \, c' \sqrt{k_p}$$

ACTIVE EARTH DRIVING FORCE AT HEEL OF WALL



$$H_{A_w} = H_{A_\phi} - H_{A_c}$$

$$= \frac{1}{2} \gamma_b k_A H^2 - 2c' H \sqrt{k_A}$$

$$= \frac{1}{2} (0.125 - 0.0625) k_A (9.2)^2 - 2 \cdot c' (9.2) \sqrt{k_A}$$

$$= 2.645 \, k_A - 18.4 \, c' \sqrt{k_A}$$

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT I SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 10

NET APPLIED FORCE TENDING TO INDUCE SLIDING

$\Sigma D_H = H_1 - U_H + H_A$ (See Sheet H-5 for H_1 and U_H values)

$= 11.422 - 4.117 + 2.645\, k_A - 18.4\, c'\sqrt{k_A}$

$= 7.305 + 2.645\, k_A - 18.4\, c'\sqrt{k_A}$

REACTION FORCES TENDING TO RESIST SLIDING

$\Sigma R_H = H_{R_W} + H_{P_W} = V_1 \tan\phi' + c' \cdot A + 3.2513\, k_p + 20.4\, c'\sqrt{k_p}$

$= 14.3223 \tan\phi' + 14.80\, c' + 3.2513\, k_p + 20.4\, c'\sqrt{k_p}$

EQUILIBRIUM OF APPLIED AND REACTION FORCES CONSIDERING TRIAL FAILURE SURFACE 2

WITH [$\Sigma D_H = f_1(FS, FS + 2c')$ AND $\Sigma R_H = f_2(FS, FS + 2c')$], the equilibrium relationship [$\Sigma D_H = \Sigma R_H$] becomes [$f_1(FS, FS + 2c') = f_2(FS, FS + 2c')$] which can be transposed to read [$f_1(FS, FS + 2c') - f_2(FS, FS + 2c') = 0$]

The computation of FS involves solving the expression

$$\Sigma D_H - \Sigma R_H = 0$$

$$f_1(FS, FS + 2c') - f_2(FS, FS + 2c') = 0$$

by an iterative procedure, either graphically (by hand) or analytically (on the computer). The procedure includes (1) assuming a trial value of FS, (2) calculating allowable values of ϕ' and c', (3) calculating k_A and k_p from ϕ', (4) substituting c', k_A, and k_p into ΣD_H and ΣR_H, and (5) either plotting $\Sigma D_H = f_1(FS, FS + 2c')$ and $\Sigma R_H = f_2(FS, FS + 2c')$ to get the point of intersection or solving for FS to make $\Sigma D_H - \Sigma R_H = 0$

WES FORM NO. 1253
REV OCT 1968

SUBJECT: EXHIBIT I

SAMPLE FLOODWALL SLIDING ANALYSIS

COMPUTED BY: | DATE: | FILE NO.

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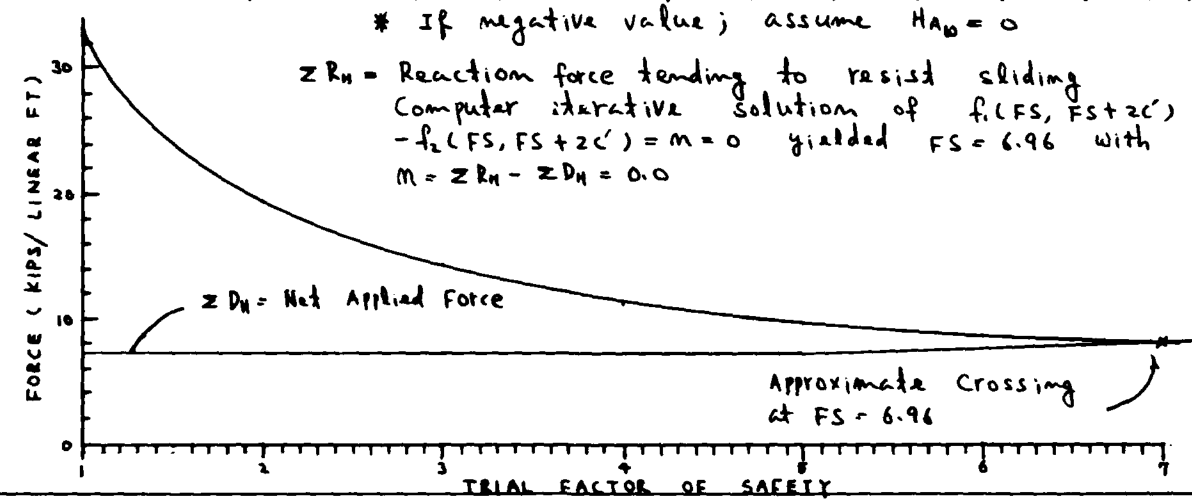
GRAPHICAL SOLUTION

FS	ϕ'	$\tan\phi'$	K_A	K_P	$\sqrt{K_P}$	C'	H_{A_W}*	ΣD_H	H_{R_ϕ}	H_{R_c}	H_{P_ϕ}	H_{P_c}	ΣR_H	$\Sigma R_H - \Sigma D_H$
1.0	20.0	0.364	0.490	2.04	1.43	0.495	-4.82	7.31	5.21	7.03	6.63	13.83	32.70	25.39
1.5	13.64	0.243	0.618	1.62	1.27	0.374	-3.77	7.31	3.48	5.53	5.26	9.69	23.96	16.65
2.0	10.31	0.182	0.696	1.44	1.20	0.304	-2.82	7.31	2.61	4.50	4.67	7.43	19.21	11.90
3.0	6.92	0.121	0.785	1.27	1.13	0.218	-1.47	7.31	1.74	3.22	4.14	5.01	14.11	6.80
4.0	5.20	0.091	0.834	1.20	1.10	0.168	-0.62	7.31	1.30	2.49	3.90	3.75	11.44	4.13
5.0	4.16	0.073	0.865	1.16	1.08	0.136	-0.04	7.31	1.04	2.02	3.76	2.99	9.81	2.50
6.0	3.47	0.061	0.886	1.13	1.06	0.114	0.36	7.67	0.87	1.69	3.67	2.48	8.71	1.04
6.9	3.02	0.053	0.900	1.11	1.05	0.100	0.63	7.94	0.76	1.48	3.61	2.15	8.00	0.06
6.92	3.01	0.053	0.900	1.11	1.05	0.100	0.64	7.95	0.75	1.48	3.61	2.14	7.98	0.03
6.94	3.00	0.052	0.900	1.11	1.05	0.099	0.65	7.95	0.75	1.47	3.61	2.14	7.97	0.02
6.96	2.99	0.052	0.901	1.11	1.05	0.095	0.65	7.96	0.75	1.47	3.61	2.13	7.96	0.0
6.99	2.99	0.052	0.901	1.11	1.05	0.099	0.65	7.96	0.75	1.47	3.61	2.13	7.93	-0.01
7.00	2.98	0.052	0.901	1.11	1.05	0.099	0.66	7.97	0.74	1.46	3.61	2.12	7.93	-0.04

* If negative value; assume $H_{A_W} = 0$

ΣR_H = Reaction force tending to resist sliding

Computer iterative solution of $f_1(FS, FS+2c') - f_2(FS, FS+2c') = m = 0$ yielded $FS = 6.96$ with $m = \Sigma R_H - \Sigma D_H = 0.0$



WES FORM NO. 1253
REV OCT 1960

SUBJECT: EXHIBIT I SAMPLF FLOODWALL SLIDI:.G ANALYSIS	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO.

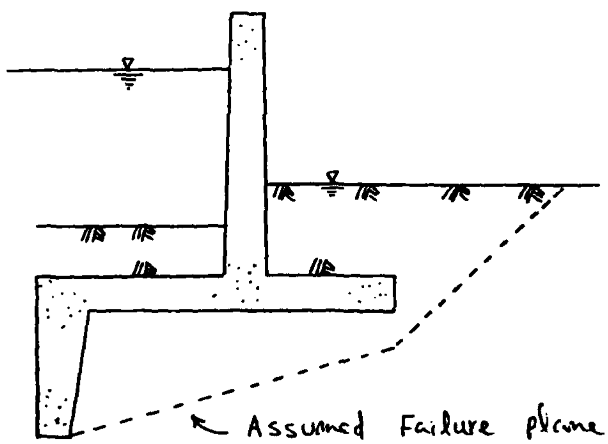
The total base-foundation interface of the T-wall may not be in compression with the foundation. If any part of the surface under consideration is along the base-foundation interface and is not in contact with the foundation, this portion should be neglected when obtaining the effective base area to resist sliding. However, if the assumed failure surface is not along the base-foundation interface but through the soil, no reduction in the area to resist sliding is made.

A portion of the base of a T-wall will not be in compression when the resultant falls outside the kern thus creating a crack which can result in an increase in uplift pressures. This condition will affect the sliding stability analysis when the assumed sliding plane acts along the soil-structure interface below the base of the wall (for this condition the program will have to recycle back through the line of creep calculations until the creep path assumptions match the final part of the base that is in contact

WES FORM NO. 1253
REV OCT 1966

SUBJECT: EXHIBIT I SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO.

with the foundation). For example, consider a wall without a key and with a horizontal base, when the resultant falls outside the kern and the assumed sliding plane is along the interface between the base of the structure and the soil foundation, uplift pressures will be computed assuming no creep loss for the portion of the foundation not in compression. For the condition where the resultant falls outside the kern but the assumed sliding plane is through the soil, for example a wall with a key positioned at the extreme end of the heel (see figure shown below), no increase in uplift pressure will be considered because the soil does not lift and form a crack as is the case at the soil-structure interface.



WES FORM NO. 1253
REV OCT 1960

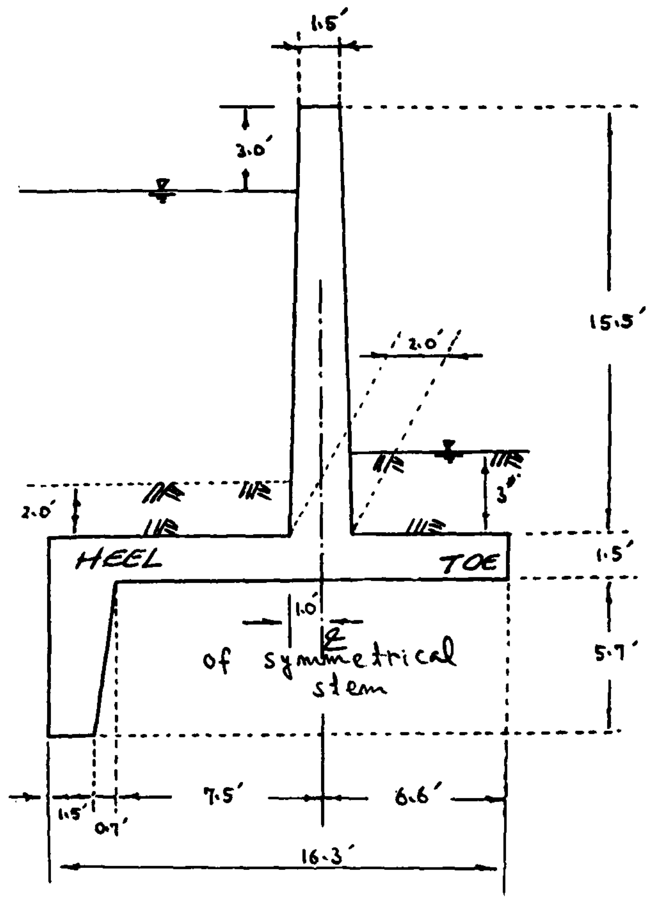
SUBJECT: EXHIBIT I SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO.

Another reason the uplift forces are not affected for this condition is because they are forces inside the soil-structure free body and therefore, do not affect the overall sliding stability.

The ϕ and C values should be consistent with the material being sheared. A plane of failure through the soil should use the ϕ and C of the soil. For any of the failure planes along the soil-structure interface, use the ϕ and C for sliding friction at the interface.

WES FORM NO. 1253
REV OCT 1968

EXHIBIT J SAMPLE FLOODWALL SLIDING ANALYSIS $\tan\phi' = \frac{\tan\phi}{FS}$ AND $c' = \frac{c}{FS}$	DATE: DATE:	FILE NO.. SHEET NO. 1



γ_c = 150 pcf $\qquad \phi = 20°$ $\qquad c = 0.35$ tsf $= 0.7$ ksf

γ_w = 62.5 pcf

γ_{sd} = 125 pcf $\qquad k_a = \tan^2(45° - \phi/2) \qquad k_p = \tan^2(45° + \phi/2)$

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT J
SAMPLE FLOODWALL
SLIDING ANALYSIS

COMPUTED BY: | DATE: | FILE NO.
CHECKED BY: | DATE: | SHEET NO. 2

$H_1 = H_{1a} + H_{1b} + H_{1c} = 11.316 \text{ k/ft}$

$\Sigma W = \Sigma C + W_{E1} + W_{E2} + W_W + W_N = 24.41 \text{ k/ft}$

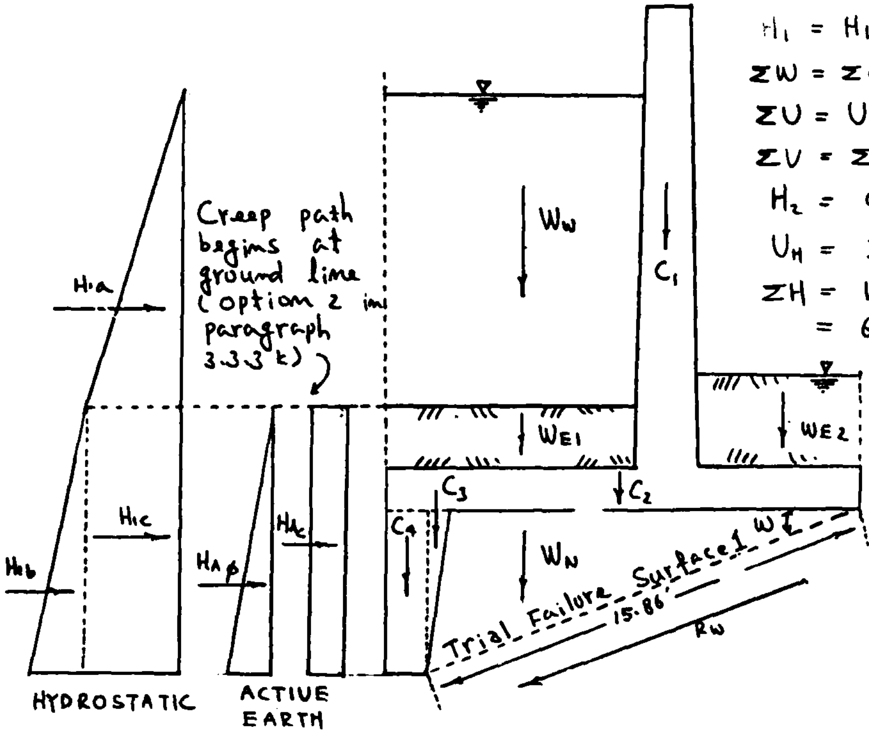
$\Sigma U = U_1 + U_2 + U_3 + U_4 = 11.5542 \text{ k/ft}$

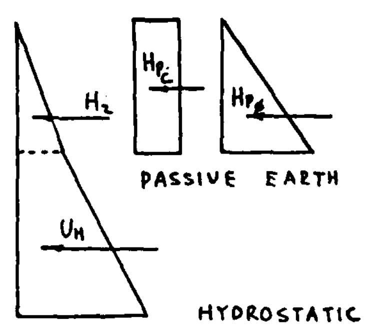
$\Sigma V = \Sigma W - \Sigma U = 12.8558 \text{ k/ft}$

$H_2 = 0.827 \text{ k/ft}$

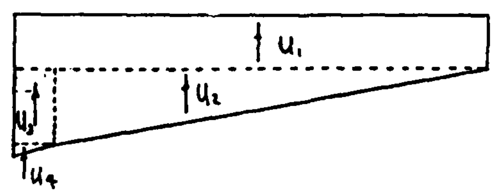
$U_H = 3.974 \text{ k/ft}$

$\Sigma H = 11.316 - 0.827 - 3.974 + H_A - H_P$
$= 6.515 + H_A - H_P$





$\omega = \tan^{-1}\left(\frac{5.7}{14.8}\right) = 21.06°$

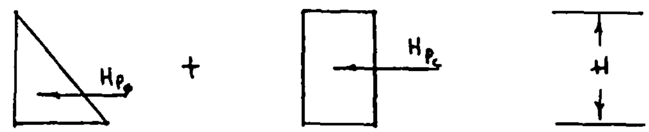


All values shown on this sheet except H_A and H_P are identical to values on sheet H-2

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT J SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY	DATE:	SHEET NO. 3

PASSIVE EARTH RESISTANCE AT TOE OF WALL



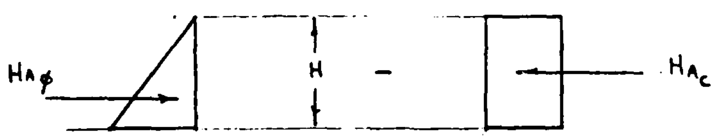
$$H_{P_W} = (H_{P_\phi} + H_{P_c}) \cos 21.06^\circ$$

$$= \left[\tfrac{1}{2} \gamma_b \, k_p \cdot H^2 + 2c' H \sqrt{k_p} \right] \cos 21.06^\circ$$

$$= \left[\tfrac{1}{2} (0.125 - 0.0625)\, k_p \cdot (4.5)^2 + 2c' (4.5) \sqrt{k_p} \right] \cos 21.06^\circ$$

$$= 0.5905\, k_p + 8.399\, c' \sqrt{k_p}$$

ACTIVE EARTH DRIVING FORCE AT HEEL OF WALL



$$H_{A_W} = (H_{A\phi} - H_{A_c}) \cos 21.06^\circ$$

$$= \tfrac{1}{2} \gamma_b \, k_A H^2 \cos 21.06^\circ - 2c' H \sqrt{k_A} \cdot \cos 21.06^\circ$$

$$= \tfrac{1}{2} (0.125 - 0.0625)\, k_A \cdot (9.2)^2 \cdot \cos 21.06^\circ$$

$$- 2 \cdot c' \cdot (9.2) \sqrt{k_A} \cdot \cos 21.06^\circ$$

$$= 2.4683\, k_A - 17.1710\, c' \sqrt{k_A}$$

WES FORM N 1253
REV OCT 1968

SUBJECT: EXHIBIT J SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY CHECKED BY	DATE DATE	FILE NO. SHEET NO. 4

NET APPLIED FORCE TENDING TO INDUCE SLIDING

$$\Sigma D_\omega = (H_1 - H_2 - V_H)\cos\omega + H_{A_\omega} \quad \text{(See sheet H-5 for } H_1, H_2 \text{ \& } V_H \text{ values)}$$

$$= (11.316 - 0.827 - 3.974)\cos 21.06 + 2.4683\,k_A - 19.1910\,c'\sqrt{k_A}$$

$$= 6.08 + 2.4683\,k_A - 17.1710\,c'\sqrt{k_A}$$

REACTION FORCES TENDING TO RESIST SLIDING

$$\Sigma R_\omega = H_{R_\omega} + H_{P_\omega} = [\Sigma V \cdot \cos\omega + \Sigma H \cdot \sin\omega] \cdot \tan\phi' + c' \cdot A + H_{P_\omega}$$

$$= [12.8558 \cdot \cos 21.06 + \{11.316 - 0.827 - 3.974\} \sin 21.06^\circ +$$
$$\{\tfrac{1}{2}(0.125 - 0.0625) \cdot k_A \cdot (9.2)^2 - 2 \cdot c'(9.2)\sqrt{k_A}\} \sin 21.06^\circ -$$
$$\{\tfrac{1}{2}(0.125 - 0.0625) \cdot k_p \cdot (4.5)^2 + 2 \cdot c' \cdot (4.5)\sqrt{k_p}\} \cdot \sin 21.06^\circ] \cdot \tan\phi' +$$
$$15.86c' + 0.5905\,k_p + 8.399 \cdot c'\sqrt{k_p}$$

$$= [14.338 + 0.9505\,k_A - 6.612\,c'\sqrt{k_A} - 0.2274\,k_p - 3.23411\,c'\sqrt{k_p}] \cdot \tan\phi' +$$
$$15.86c' + 0.5905\,k_p + 8.399 \cdot c'\sqrt{k_p}$$

EQUILIBRIUM OF APPLIED AND REACTION FORCES CONSIDERING TRIAL FAILURE SURFACE 1

WITH [$\Sigma D_\omega = f_1(FS)$ AND $\Sigma R_\omega = f_2(FS)$], the equilibrium relationship [$\Sigma D_\omega = \Sigma R_\omega$] becomes [$f_1(FS) = f_2(FS)$] which can be transposed to read [$f_1(FS) - f_2(FS) = 0$]

The computation of FS involves solving the expression

$$\Sigma D_\omega - \Sigma R_\omega = 0$$
$$f_1(FS) - f_2(FS) = 0$$

by an iterative procedure, either graphically (by hand) or analytically (on the computer). The procedure includes (1) assuming a trial value of FS, (2) calculating allowable values of ϕ' and c', (3) calculating k_A and k_p from ϕ', (4) substituting c', k_A, and k_p into ΣD_ω and ΣR_ω, and (5) either plotting $\Sigma D_\omega = f_1(FS)$ and $\Sigma R_\omega = f_2(FS)$ to get the point of intersection or solving for FS to make $\Sigma D_\omega - \Sigma R_\omega = 0$

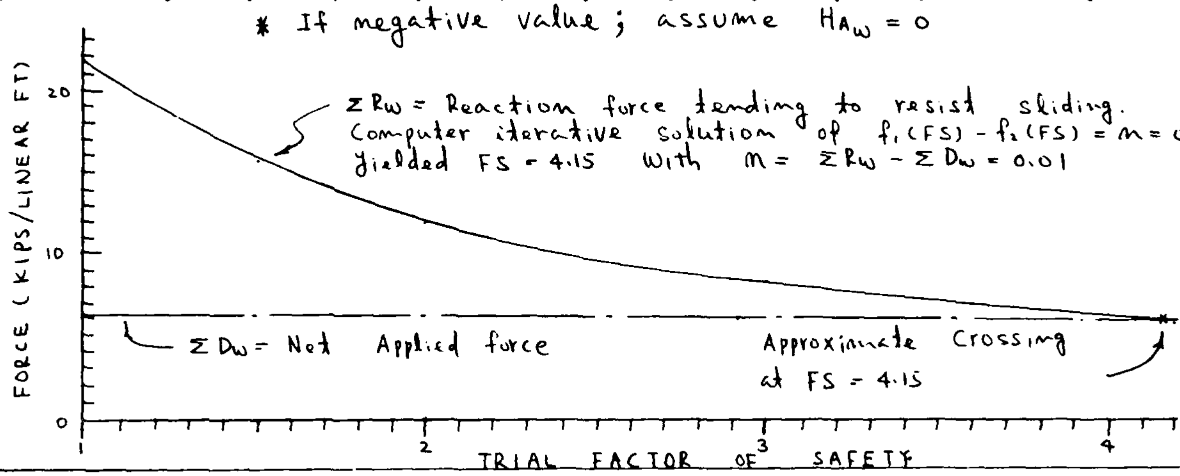
WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT J
SAMPLE FLOODWALL SLIDING ANALYSIS
GRAPHICAL SOLUTION

COMPUTED BY: | DATE | FILE NO.
CHECKED BY: | DATE: | SHEET NO. 5

FS	φ'	tan φ'	K_A	K_P	$\sqrt{K_P}$	C'	H_{A_W}*	ΣD_W	H_{R_ϕ}	H_{R_c}	H_{P_ϕ}	H_{P_c}	ΣR_W	$\Sigma R_W - \Sigma D_W$
1.0	20.0	0.364	0.490	2.04	1.43	0.70	-9.21	6.08	2.86	11.10	1.20	8.40	23.57	17.49
1.5	13.64	0.243	0.618	1.62	1.27	0.467	-4.77	6.08	2.48	9.40	0.95	4.98	15.82	9.74
2.0	10.31	0.182	0.696	1.44	1.20	0.350	-3.30	6.08	2.07	5.55	0.85	3.52	11.99	5.91
3.0	6.92	0.121	0.785	1.27	1.13	0.233	-1.61	6.08	1.53	3.70	0.75	2.21	8.19	2.11
4.0	5.20	0.091	0.834	1.20	1.10	0.175	-0.69	6.08	1.20	2.78	0.71	1.61	6.29	0.21
4.1	5.07	0.089	0.838	1.19	1.09	0.171	-0.62	6.08	1.17	2.71	0.71	1.57	6.15	0.07
4.11	5.06	0.089	0.838	1.19	1.09	0.170	-0.61	6.08	1.17	2.70	0.70	1.56	6.14	0.06
4.12	5.05	0.088	0.838	1.19	1.09	0.170	-0.60	6.08	1.17	2.69	0.70	1.56	6.13	0.05
4.13	5.04	0.088	0.839	1.19	1.09	0.169	-0.60	6.08	1.17	2.69	0.70	1.55	6.11	0.03
4.14	5.02	0.088	0.839	1.19	1.09	0.169	-0.59	6.08	1.16	2.68	0.70	1.55	6.10	0.02
4.15	5.01	0.088	0.839	1.19	1.09	0.169	-0.58	6.08	1.16	2.68	0.70	1.55	6.09	0.01
4.16	5.00	0.087	0.840	1.19	1.09	0.168	-0.58	6.08	1.16	2.67	0.70	1.54	6.07	-0.01
4.20	4.95	0.087	0.841	1.19	1.09	0.167	-0.55	6.08	1.15	2.64	0.70	1.53	6.02	-0.06

* If negative value; assume $H_{A_W} = 0$



WES FORM NO. 1253
REV OCT 1968

EXHIBIT J

SAMPLE FLOODWALL SLIDING ANALYSIS

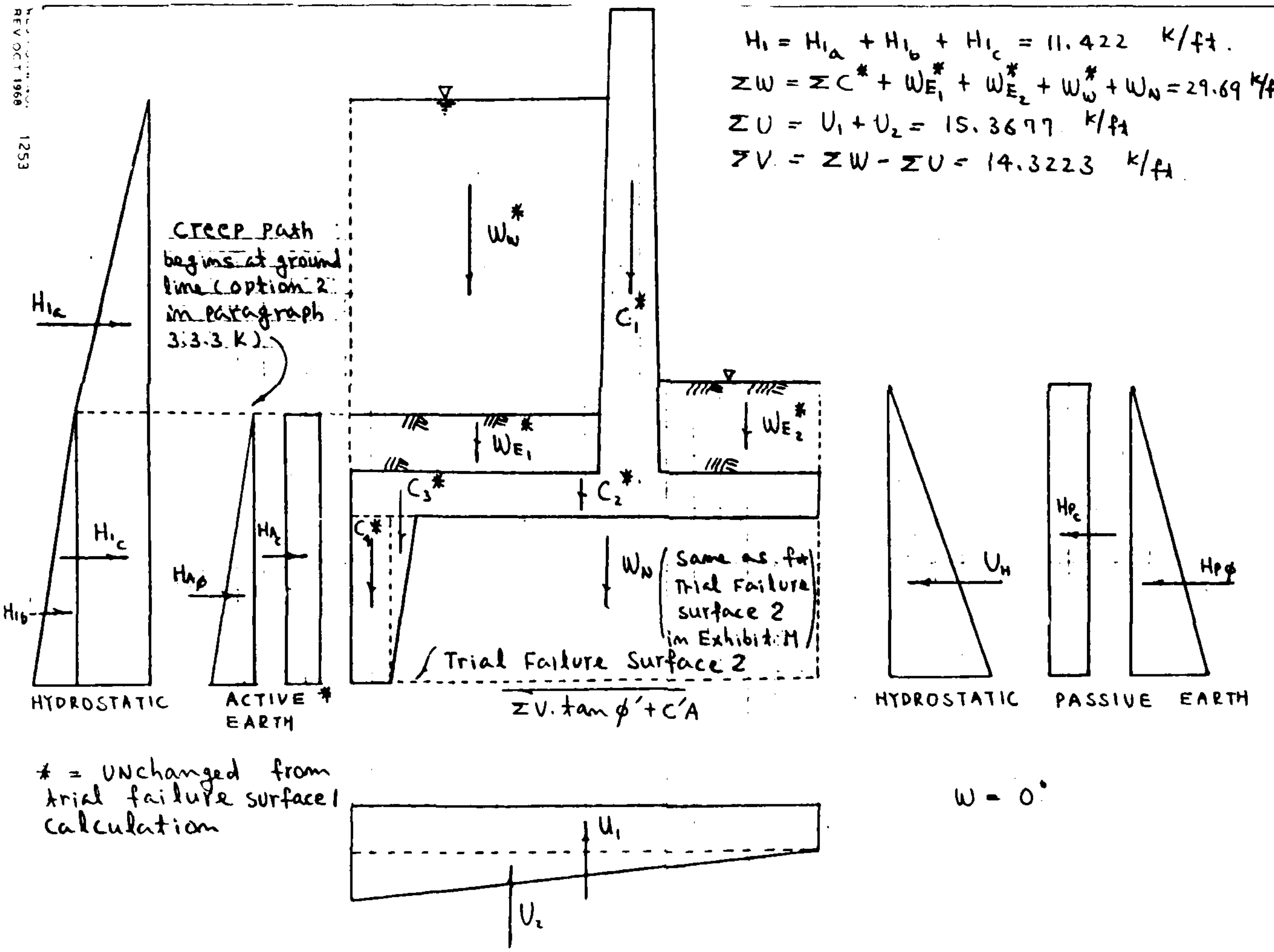
SHEET NO. 6

$H_1 = H_{1a} + H_{1b} + H_{1c} = 11.422$ K/ft.

$\Sigma W = \Sigma C^* + W_{E_1}^* + W_{E_2}^* + W_W^* + W_N = 29.69$ K/ft

$\Sigma U = U_1 + U_2 = 15.3677$ K/ft

$\Sigma V = \Sigma W - \Sigma U = 14.3223$ K/ft



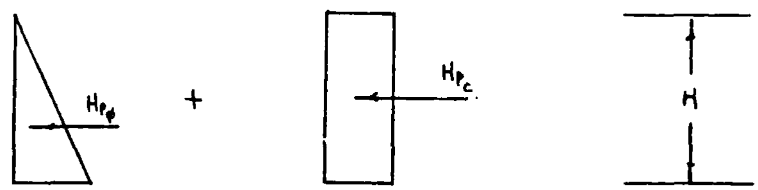
* = UNchanged from trial failure surface 1 calculation

$\omega = 0°$

REV OCT 1968 1253

SUBJECT – EXHIBIT J SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE:	SHEET NO. 7

PASSIVE EARTH RESISTANCE AT TOE OF WALL



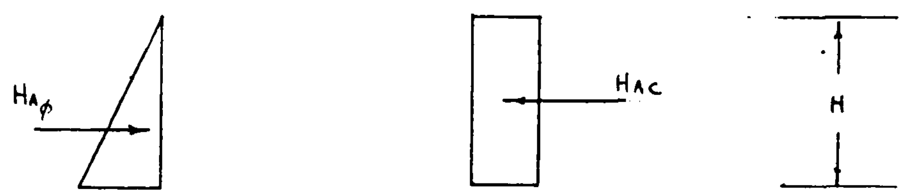
$$H_{P_W} = H_{P_\phi} + H_{P_c}$$

$$= \frac{1}{2}\gamma_b k_P H^2 + 2c' H\sqrt{k_P}$$

$$= \frac{1}{2}(0.125 - 0.0625)\, k_P\, (10.2)^2 + 2c'(10.2)\sqrt{k_P}$$

$$= 3.2513\, k_P + 20.4\, c'\sqrt{k_P}$$

ACTIVE EARTH DRIVING FORCE AT HEEL OF WALL



$$H_{A_W} = H_{A_\phi} - H_{A_c}$$

$$= \frac{1}{2}\gamma_b k_A H^2 - 2c' H\sqrt{k_A}$$

$$= \frac{1}{2}(0.125 - 0.0625)\, k_A\, (9.2)^2 - 2 \cdot c'(9.2)\sqrt{k_A}$$

$$= 2.645\, k_A - 18.4\, c'\sqrt{k_A}$$

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT J SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 8

NET APPLIED FORCE TENDING TO INDUCE SLIDING

$\Sigma D_H = H_1 - U_H + H_A$ (See Sheet H-5 for H_1 and U_H values)

$= 11.422 - 4.117 + 2.645\, k_A - 18.4\, c'\sqrt{k_A}$

$= 7.305 + 2.645\, k_A - 18.4\, c'\sqrt{k_A}$

REACTION FORCES TENDING TO RESIST SLIDING

$\Sigma R_H = H_{R_W} + H_{P_W} = V \cdot \tan\phi' + c' \cdot A + 3.2513\, k_p + 20.4\, c'\sqrt{k_p}$

$= 14.3223 \tan\phi' + 14.80\, c' + 3.2513\, k_p + 20.4 \cdot c'\sqrt{k_p}$

EQUILIBRIUM OF APPLIED AND REACTION FORCES CONSIDERING TRIAL FAILURE SURFACE 2

With [$\Sigma D_H = f_1(FS)$ AND $\Sigma R_H = f_2(FS)$], the equilibrium relationship [$\Sigma D_H = \Sigma R_H$] becomes [$f_1(FS) = f_2(FS)$] which can be transposed to read [$f_1(FS) - f_2(FS) = 0$] The computation of FS involves solving the expression

$$\Sigma D_H - \Sigma R_H = 0$$

$$f_1(FS) - f_2(FS) = 0$$

by an iterative procedure, either graphically (by hand) or analytically (on the computer). The procedure includes (1) assuming a trial value of FS, (2) calculating allowable values of ϕ' and c', (3) calculating k_A and k_p from ϕ', (4) substituting c', k_A and k_p into ΣD_H and ΣR_H, and (5) either plotting $\Sigma D_H = f_1(FS)$ and $\Sigma R_H = f_2(FS)$ the point of intersection or solving for FS to make $\Sigma D_H - \Sigma R_H = 0$

WES FORM NO. 1253
REV OCT 1968

SUBJECT: EXHIBIT J
SAMPLE FLOODWALL
SLIDING ANALYSIS

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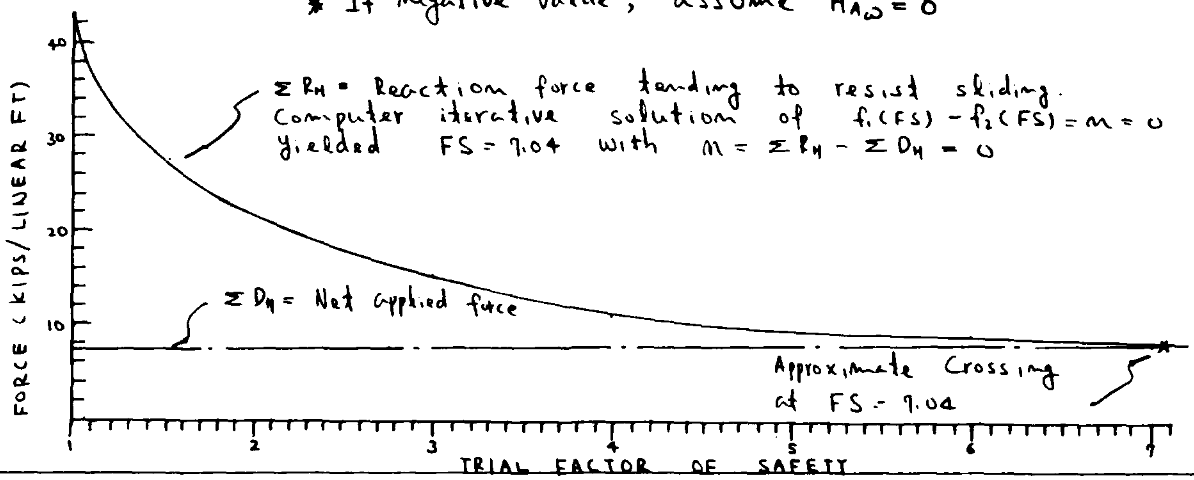
FS	ϕ'	$\tan\phi'$	K_A	K_P	$\sqrt{K_P}$	C'	$H_{A\omega}$*	ΣD_H	$H_{R\phi}$	H_{Rc}	$H_{P\phi}$	H_{Pc}	ΣR_H	$\Sigma R_H - \Sigma D_H$
1.0	20.0	0.364	0.490	2.04	1.43	0.700	-7.72	7.31	5.21	10.36	6.63	20.39	42.60	35.29
1.5	13.64	0.243	0.618	1.62	1.27	0.467	-5.12	7.31	3.48	6.91	5.26	12.11	27.75	20.44
2.0	10.31	0.182	0.696	1.44	1.20	0.350	-3.53	7.31	2.61	5.18	4.67	8.56	21.01	13.71
3.0	6.92	0.121	0.785	1.27	1.13	0.233	-1.73	7.31	1.74	3.45	4.14	5.37	14.71	7.40
4.0	5.20	0.091	0.834	1.20	1.10	0.175	-0.73	7.31	1.30	2.59	3.90	3.91	11.70	4.40
5.0	4.16	0.073	0.865	1.16	1.08	0.140	-0.11	7.31	1.04	2.07	3.76	3.07	9.95	2.64
6.0	3.47	0.061	0.886	1.13	1.06	0.117	0.32	7.63	0.87	1.73	3.67	2.53	8.79	1.16
7.0	2.98	0.052	0.901	1.11	1.05	0.100	0.64	7.94	0.74	1.48	3.61	2.15	7.98	0.04
7.01	2.97	0.052	0.901	1.11	1.05	0.100	0.64	7.94	0.74	1.48	3.61	2.15	7.97	0.03
7.02	2.97	0.052	0.902	1.11	1.05	0.100	0.64	7.95	0.74	1.48	3.61	2.14	7.97	0.02
7.03	2.96	0.052	0.902	1.11	1.05	0.100	0.65	7.95	0.74	1.47	3.61	2.14	7.96	0.01
7.04	2.96	0.052	0.902	1.11	1.05	0.099	0.65	7.95	0.74	1.47	3.61	2.14	7.95	0.0
7.05	2.96	0.052	0.902	1.11	1.05	0.099	0.65	7.96	0.74	1.47	3.60	2.13	7.93	- 0.01
7.10	2.93	0.051	0.903	1.11	1.05	0.099	0.66	7.97	0.73	1.46	3.60	2.12	7.91	- 0.06

* If negative value; assume $H_{A\omega} = 0$



WES FORM NO. 1253
REV OCT 1968

SUBJECT: EXHIBIT J SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY:	DATE:	FILE NO.
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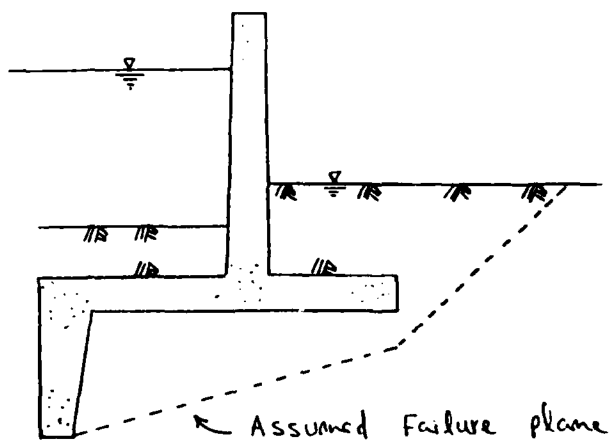
The total base-foundation interface of the T-wall may not be in compression with the foundation. If any part of the surface under consideration is along the base-foundation interface and is not in contact with the foundation, this portion should be neglected when obtaining the effective base area to resist sliding. However, if the assumed failure surface is not along the base-foundation interface but through the soil, no reduction in the area to resist sliding is made.

A portion of the base of a T-wall will not be in compression when the resultant falls outside the kern thus creating a crack which can result in an increase in uplift pressures. This condition will affect the sliding stability analysis when the assumed sliding plane acts along the soil-structure interface below the base of the wall (for this condition the program will have to recycle back through the line of creep calculations until the creep path assumptions match the final part of the base that is in contact

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT J SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY: CHECKED BY:	DATE DATE	FILE NO. SHEET NO.

with the foundation). For example, consider a wall without a key and with a horizontal base, when the resultant falls outside the kern and the assumed sliding plane is along the interface between the base of the structure and the soil foundation, uplift pressures will be computed assuming no creep loss for the portion of the foundation not in compression. For the condition where the resultant falls outside the kern but the assumed sliding plane is through the soil, for example a wall with a key positioned at the extreme end of the heel (see figure shown below), no increase in uplift pressure will be considered because the soil does not lift and form a crack as is the case at the soil-structure interface.



WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT J SAMPLE FLOODWALL SLIDING ANALYSIS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE:	SHEET NO.

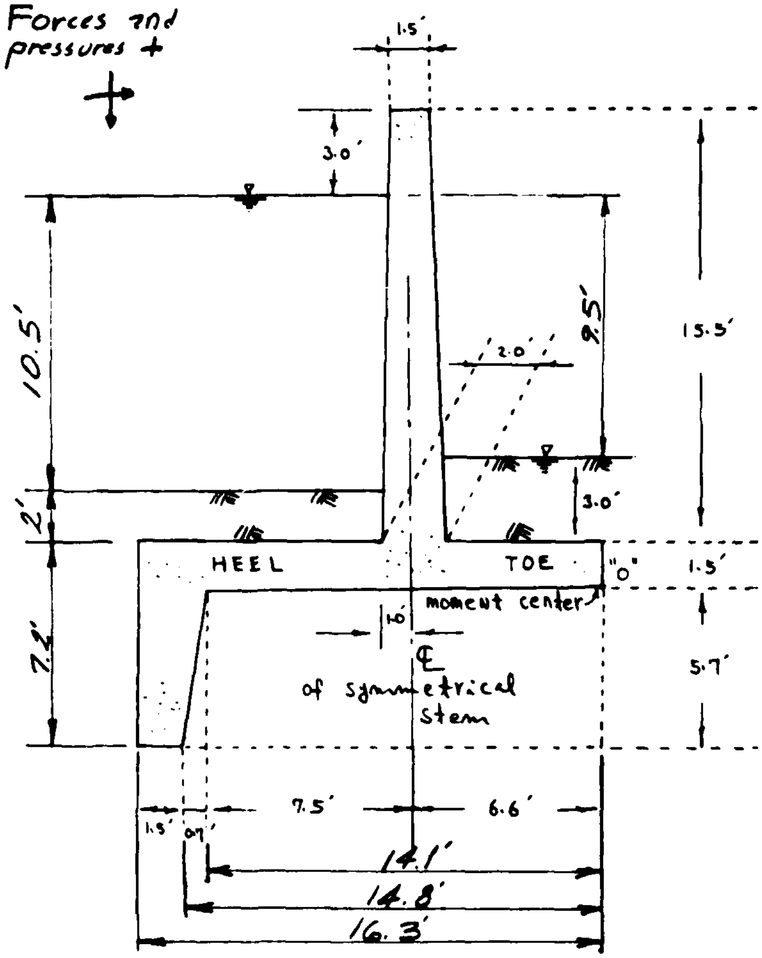
Another reason the uplift forces are not affected for this condition is because they are forces inside the soil-structure free body and therefore, do not affect the overall sliding stability.

The ϕ and C values should be consistent with the material being sheared. A plane of failure through the soil should use the ϕ and C of the soil. For any of the failure planes along the soil-structure interface, use the ϕ and C for sliding friction at the interface.

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT K SAMPLE OVERTURNING ANALYSIS FOR FLOOD WALLS & RETAINING WALLS	COMPUTED BY	DATE	FILE NO
	CHECKED BY	DATE	SHEET NO 1

Sign convention:
Forces and pressures +



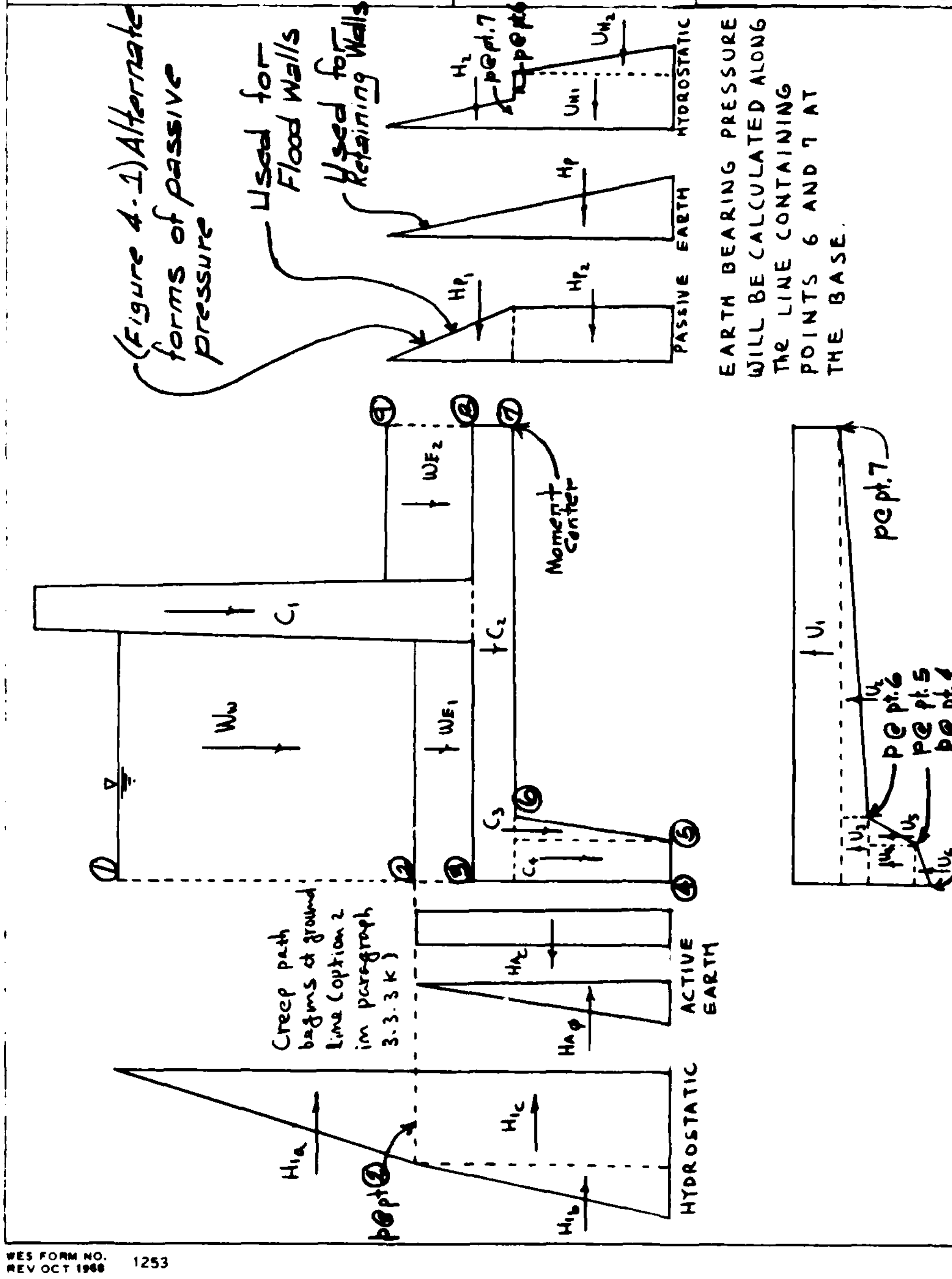
γ_c = 150 pcf ϕ = 20° C = 0.35 tsf = 0.7 ksf

γ_w = 62.5 pcf

γ_{sat} = 125 pcf $k_a = \tan^2(45° - \phi/2)$ $k_p = \tan^2(45° + \phi/2)$

WES FORM NO. 1253
REV OCT 1968

SUBJECT: EXHIBIT K SAMPLE OVERTURNING ANALYSIS FOR FLOOD WALLS & RETAINING WALLS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 2



WES FORM NO. 1253
REV OCT 1968

EXHIBIT K
SAMPLE OVERTURNING ANALYSIS FOR FLOOD WALLS & RETAINING WALLS

SHEET NO. 3

CALCULATE LINE OF CREEP HYDROSTATIC PRESSURES

Note that these pressure will be different from the pressure shown in Exhibits H, I, and J because the assumed line of creep is in a different location (along the soil structure interface).

1. Lost head = (headwater - tailwater) $\frac{\text{total distance}}{\text{max. total distance}} = \frac{9.5 \text{ * col. 2}}{35.04}$
2. Potential head = (headwater - tailwater) - lost head
   = 9.5 - lost head
   = remaining seepage head
3. Position head = Vertical distance to tail water, positive if below tailwater
4. Uplift head = potential head + position head
5. Uplift pressure = uplift head x Wt. of water

COLUMN	1	2	3	4	5	6	7
	Creep path						
POINT	increment	total distance	lost head	Potential head	Position head	Uplift head	Uplift Pressure psf
1	0	0	0	9.50	-9.50	0	0
2	0	0	0	9.50	1.00	10.50	656
4	9.2	9.2	2.49	7.01	10.20	17.21	1075.4
5	1.5	10.7	2.90	6.60	10.20	16.80	1050
6	5.74	16.44	4.46	5.04	4.50	9.54	596.5
7	14.1	30.54	8.28	1.22	4.50	5.72	357.5
9	4.5	35.04	9.50	0	0	0	0

CALCULATE ACTIVE EARTH PRESSURE

$$K_{A_\phi} = \tan^2(45° - \phi/2) = \tan^2(45° - 20°/2) = 0.490$$

$$P_A = \gamma H K_A - 2C\sqrt{K_A} = (125 - 62.5)(9.2)(0.49) - 2(700)(\sqrt{0.49})$$

$$= 281.75 - 980 = -698.25$$

The soil will not pull on the wall in the active state; therefore, let $P_A = 0$.

SUBJECT EXHIBIT K

SAMPLE OVERTURNING ANALYSIS FOR FLOOD WALLS & RETAINING WALLS

COMPUTED BY | DATE | FILE NO.

CHECKED BY | DATE | SHEET NO. 5

FORCE AND MOMENT SUMMARY

Item	FACTORS (ft, lb)	FORCE →	FORCE N ↓	MOMENT ARM, FT	MOMENT +"O"
C_1	$\frac{1}{2}$ (1.5 + 2.0) (15.5) (150)		4069	6.6	26854
C_2	1.5 (16.3) (150)		3668	8.15	29890
C_3	$\frac{1}{2}$ (0.7) (5.7) (150)		299	14.57	4360
C_4	1.5 (5.7) (150)		1283	15.55	19943
	CONCRETE SUBTOTALS		9319	8.70	81047
W_{E_1}	$2(125)\ \frac{8.7 + 8.732}{2}$		2179	11.942	26022
W_{E_2}	$3(125)\ \frac{5.6 + 5.648}{2}$		2109	2.812	5931
W_w	10.5 (8.732) (62.5)		5730	11.93	68382
above E_1	10.5 (0.169) ($\frac{1}{2}$) (62.5)		55	7.512	413
H_A	281.75 (9.2) ($\frac{1}{2}$)	1296 } 0		2.633	3412 } 0
	-980.0 (9.2)	-9016		1.100	-9917.6
H_{1A}	656 (10.5) ($\frac{1}{2}$)	3444		-7.0	-24108
H_{1B}	(1075.4 - 656) (9.2) ($\frac{1}{2}$)	1929		2.633	5080
H_{1C}	656 (9.2)	6035		1.1	6639
	SUBTOTAL H_1	11408		-1.086	-12389
H_2	357.5 (4.5) ($\frac{1}{2}$)	-804		-1.5	1207
U_{H_1}	596.5 (5.7)	-3400		2.85	-9690
U_{H_2}	(1050 - 596.5) (5.7) ($\frac{1}{2}$)	-1293		3.80	-4911
	SUBTOTAL U_H	-4693		3.11	-14601
U_1	357.5 (16.3)		-5827	8.15	-47492
U_2	(596.5 - 357.5) (14.1) ($\frac{1}{2}$)		-1685	9.4	-15839
U_3	239 (2.2)		-526	15.2	-7992
U_4	(1050 - 596.5) (1.5)		-680	15.55	-10578
U_5	453.5 (0.7) ($\frac{1}{2}$)		-159	14.567	-2312
U_6	(1075.4 - 1050) (1.5) ($\frac{1}{2}$)		-19	15.8	-301
	SUBTOTAL $U_1 + \cdots + U_6$		-8896	9.50	-84514
SUBTOTAL F → w/o passive Resistance		5911	10496		71,498

SUBJECT: EXHIBIT K SAMPLE OVERTURNING ANALYSIS FOR FLOOD WALLS & RETAINING WALLS	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO. 6

DISCUSSION OF OVERTURNING ASSUMPTIONS (GENERAL)

a. For overturning of a wall with a key, founded on either a soil or rock foundation, the horizontal equilibriant force to resist the ΣH forces will not be subjected to any limits. This assumption assumes the wall is safe against sliding; the assumption is checked later. If the wall is not safe against sliding the design is inadequate and a new trial design is started. The equilibriant force may have a distribution corresponding to any of the four options as discussed in para 4.4.2. However, the default equilibrant pressure distribution for flood walls corresponds to option "a" as shown in Figure K-18. The default equilibriant pressure distribution for retaining walls correspond to option "c" as shown in Figure K-18. For the floodwall example, as presented on pages K-12 thru K-13 of this Exhibit, the unbalanced sum of horizontal forces (ΣH = 5911 pounds) will be resisted by an equilibriant force in

WES FORM NO. 1253
REV OCT 1968

accordance with distribution option "a" of Figure K-1B. For the retaining wall example option "c" Figure K-1B, is used.

b. For overturning of a wall with horizontal base and without a key (see Figure K-2A), founded on either a soil or rock foundation, the horizontal equilibrant force to resist the ΣH forces will not be subjected to any limits. This assumption assumes the wall is safe against sliding; the assumption is checked later. If the wall is not safe against sliding the design is inadequate and a new trial design is started. The equilibrant force will be broken into two parts:

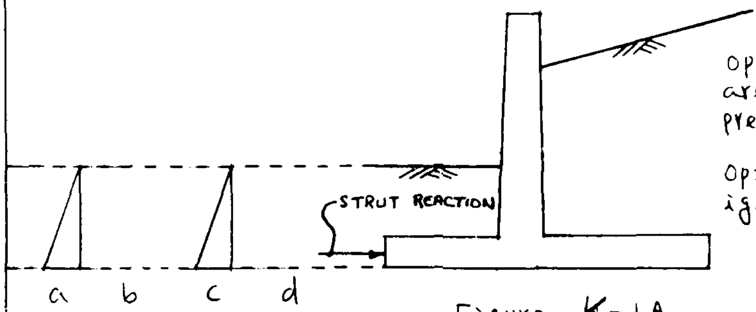
(1) A force (friction and/or cohesion) along the base of the wall, and

(2) A force due to passive pressure distribution according to options shown in Figure K-1A.

For a T-wall with a horizontal base, the force along the base of the wall is to be assumed to be mobilized first but will be limited to a maximum value of

$\Sigma V \tan \phi + CL$ (see Figure K-2A of this Exhibit)

SUBJECT EXHIBIT K SAMPLE OVERTURNING ANALYSIS FOR FLOOD WALLS & RETAINING WALLS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 8



option "a" and "c" are a triangular pressure distribution

option "b" and "d" ignores passive pressure

Figure K-1A

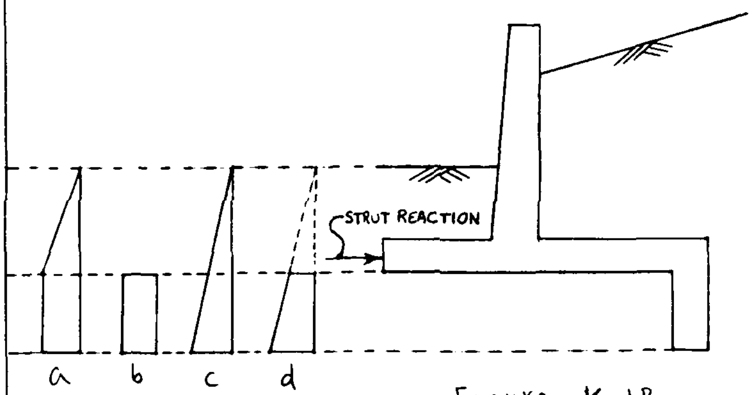
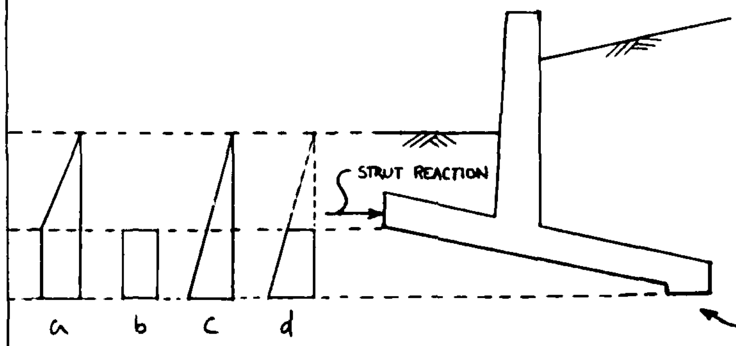


Figure K-1B



Any key at least 0.01 foot long.

Figure K-1C

WES FORM NO. 1253
REV OCT 1968

SUBJECT	COMPUTED BY	DATE	FILE NO.
EXHIBIT K SAMPLE OVERTURNING ANALYSIS FOR FLOOD WALLS & RETAINING WALLS	CHECKED BY	DATE	SHEET NO. 9

HORIZONTAL BASE WALLS

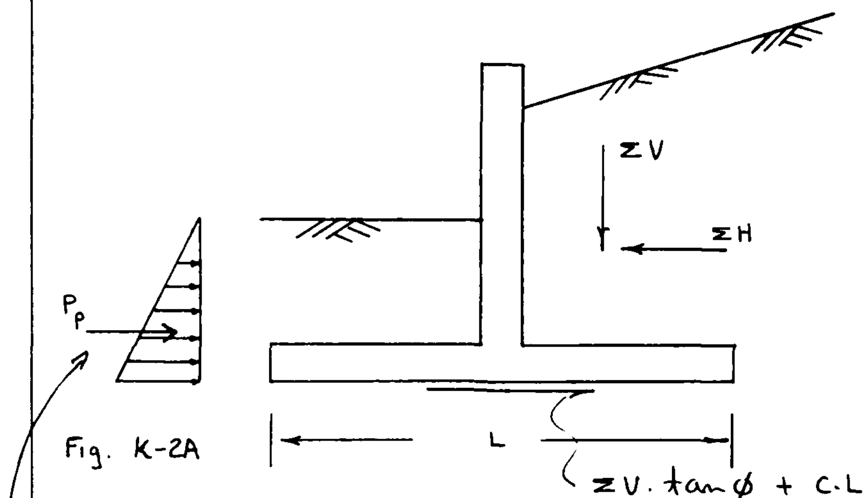


Fig. K-2A

INCLINED BASE WALLS

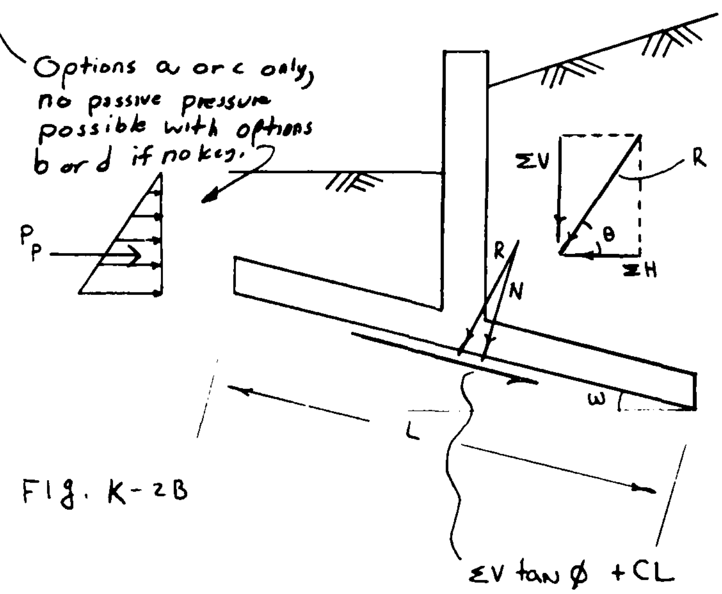


FIG. K-2B

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT K SAMPLE OVERTURNING ANALYSIS FOR FLOOD WALLS & RETAINING WALLS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 10

If any additional force is required to satisfy equilibrium, it will come from passive pressure (Figure K-1A) distributed with no limit. It is recognized that a wall without a key with an inclined base

falls into a "gray area" of design with respect to overturning, since this wall could behave in similar fashion as a wall with a key (Fig. K-1c). Therefore, for overturning stability, an option will be provided in the program which will permit the user to design this type of wall in similar fashion as a wall with a key as described in paragraph "a" of this Exhibit or as a wall without a key as described in "b" of this Exhibit. For the wall without a key (inclined base wall) to be analyzed for overturning as if the wall has a key the user would

WES FORM NO. 1253
REV. OCT 1968

SUBJECT EXHIBIT K SAMPLE OVERTURNING ANALYSIS FOR FLOOD WALLS & RETAINING WALLS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 11

(see Figure K-1c)

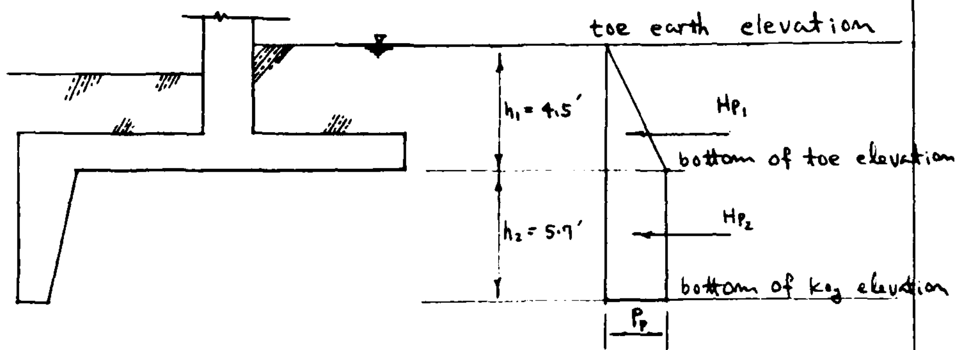
Simply input a key length of 0.01 foot to obtain the desired result. If the user chooses to let the wall friction and cohesive forces (inclined base walls, see Figure K-2B) be developed first, the maximum value of this force will be

$$N \tan \phi + CL$$

If any additional force is required to satisfy equilibrium, it will come from passive pressure (see figure K-2E) distributed with no limit. The rest of this Exhibit is divided into two parallel sections. Sheets K-12 through K-13 illustrate the procedure for flood walls and sheets K-14 through K-16 illustrate the proceedure for retaining walls.

SUBJECT EXHIBIT K SAMPLE OVERTURNING ANALYSIS FOR FLOOD WALLS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 12

FLOOD WALL CALCULATION OF HORIZONTAL REACTION FORCE



$$H_P = H_{P_1} + H_{P_2} = \tfrac{1}{2} h_1 P_p + h_2 P_p$$

$$5911 = (\tfrac{1}{2})(4.5) P_p + 5.7\ P_p$$

$$P_p = \frac{5911}{(\frac{1}{2})(4.5) + 5.7} = 743.52 \quad \#/ft.$$

$$H_{P_1} = \tfrac{1}{2}(4.5)(743.52) = 1673^{\#}$$

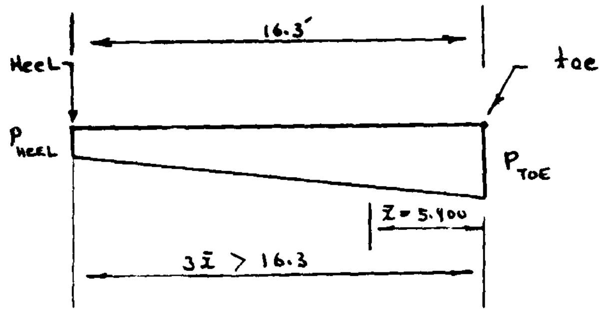
$$H_{P_2} = (5.7)(743.52) = 4238^{\#}$$

Get N:

Factors	Force →	Force lb/ft ↓	Arm	Moment ↻+ about point "O"
subtotals from pg Q-5	5,911	10,496		71,498
Horizontal reaction force	-1,673		-1.5	2,509.5
	-4,238		2.85	-12,078.3
Totals	0	10,496		61,929.2

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT K SAMPLE OVERTURNING ANALYSIS FOR FLOOD WALLS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO. 13



Resultant vertical force is located $\dfrac{61929.2}{10496} = 5.900'$ from the toe

Resultant ratio $= \dfrac{5.900}{16.3} = 0.3620 > 0.333$ INSIDE the kern

BASE PRESSURE

$$P_{\substack{TOE\\ OR\\ HEEL}} = \frac{P}{A} \pm \frac{MC}{I} = \frac{10,496}{16.3} \pm \frac{(10,496)\left[\frac{16.3}{2} - 5.9\right]\left[\frac{16.3}{2}\right]}{\frac{(1)(16.3)^3}{12}}$$

$= 643.93 \pm 533.31$

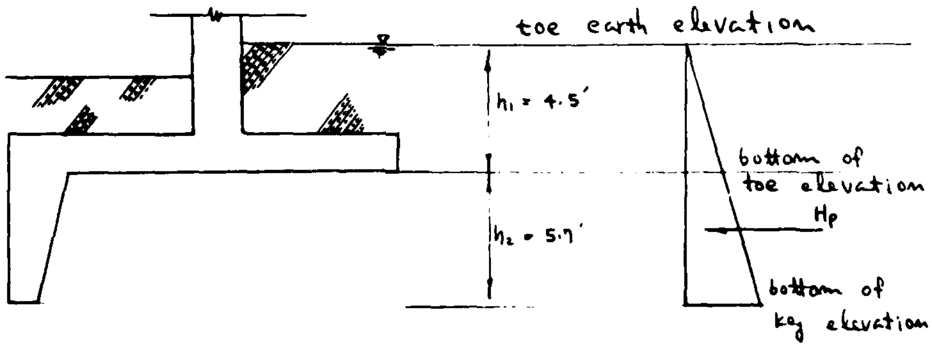
$= 1177.24$ PSF TOE

110.62 PSF HEEL

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT K SAMPLE OVERTURNING ANALYSIS FOR RETAINING WALL	COMPUTED BY / CHECKED BY	DATE / DATE	FILE NO. / SHEET NO. 14

RETAINING WALL CALCULATION OF HORIZONTAL REACTION FORCE



$H_p = \frac{1}{2}(h_1 + h_2)\, P_p$

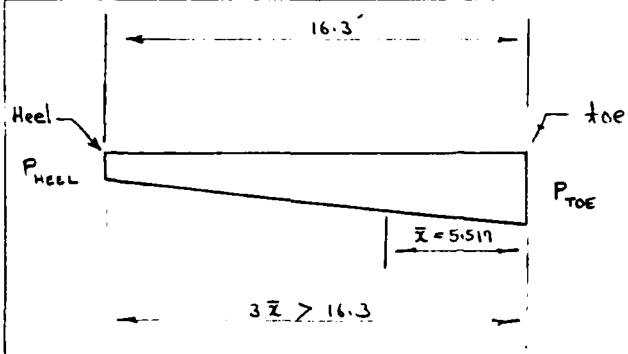
$5911 = \frac{1}{2}(10.2)\, P_p$ $\qquad P_p = 1159.02$ #/ft.

Get N:

Factors	Force →	Force ↓ lb/ft	Arm	MOMENT ↶+ about point "o"
Subtotals from page K5	5,911	10,496		71,498
Horizontal reaction force	-5,911		2.3	-13,595.3
Totals	0	10,496		57902.7

WES FORM NO. 1253
REV OCT 1968

SUBJECT EXHIBIT K SAMPLE OVERTURNING ANALYSIS FOR RETAINING WALLS	COMPUTED BY:	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO 15



Resultant vertical force is located $\frac{57902.7}{10496} = 5.517'$ from the toe

Resultant ratio $= \frac{5.517}{16.3} = 0.3385 > 0.333$ inside the kern

Base Pressure

$$P_{\substack{TOE\\OR\\HEEL}} = \frac{P}{A} \pm \frac{Mc}{I} = \frac{10,496}{16.3} \pm \frac{(10,496)\left[\frac{16.3}{2} - 5.517\right]\left[\frac{16.3}{2}\right]}{\frac{(1)(16.3)^3}{12}}$$

$= 643.93 \pm 624.10$

$= 1268.03$ PSF TOE

19.83 PSF HEEL

WES FORM NO. 1253
REV OCT 1966

SUBJECT EXHIBIT K SAMPLE OVERTURNING ANALYSIS FOR FLOOD WALLS & RETAINING WALLS	COMPUTED BY	DATE	FILE NO.
	CHECKED BY	DATE	SHEET NO 16

The seepage line-of-creep calculations for overturning should neglect any part of the T-wall base which is not in contact with the foundation. If part of the concrete base is not in contact with the foundation the program will have to recycle back through the line line-of-creep calculations until the creep path assumptions match the final part of the base that is in contact with the foundation. If the toe-side face of the key falls within the projection of the base which is not in contact with the foundation two options exist as follows:

a) If passive pressure is not used the toe-side face of the key should be omitted from the creep path calculations.

b) If passive pressure is used the toe-side face of the key should not be omitted from the creep path calculations.

The base pressure to be used for structural design/analysis will be that developed by the loading used in the overturning analysis and as described in detail in Section 9, para 9.1.1.

WES FORM NO. 1253
REV OCT 1968

Exhibit L:

Passive Resistance For T-Walls	P. 2 of 2	Dressler	12 Dec 77
Equivalent Relation For Coulomb Theory & ETL 1110-2-184		CHECKED BY	DATE

But: $\sin\alpha \cos(\phi+\alpha) = \sin(45-\phi/2)\cos(45+\phi/2) = \cos^2(45+\phi/2)$

$$= \cos^2 \tfrac{1}{2}(90+\phi) = \tfrac{1}{2}\left[1+\cos(90+\phi)\right] = \frac{1-\sin\phi}{2}$$

And: $\dfrac{\tan(\phi+\alpha)}{\tan\alpha} = \dfrac{\tan(45+\phi/2)}{\tan(45-\phi/2)} = \dfrac{\tan(45+\phi/2)}{\cot(45+\phi/2)} = \tan^2(45+\phi/2)$

$$P_p = \frac{\gamma' D_p^2}{2}\tan^2(45+\phi/2) + \frac{s\,D_p \cos\phi}{1/2(1-\sin\phi)}$$

But: $\dfrac{\cos\phi}{1-\sin\phi} = \cot\dfrac{90-\phi}{2} = \tan\left(90-\dfrac{90-\phi}{2}\right) = \tan(45+\phi/2)$

$$P_p = \frac{\gamma' D_p^2}{2}\tan^2(45+\phi/2) + 2s\,D_p\tan(45+\phi/2)$$

$$K_p = \tan^2(45+\phi/2)$$

$$P_p = \tfrac{1}{2}\gamma' D_p^2 K_p + 2s\,D_p\sqrt{K_p}$$

Note: The symbol "c" is often used instead of "s" to represent the cohesion of the passive wedge.

LMV FORM 1073 COMPUTATION SHEET

Exhibit L:

Project: Passive Resistance For T-Walls	Page 1 of 2	Computed by: Dressler	Date: 12 Dec 77
Subject: Equivalent Relation For Coulomb Theory & ETL 1110-2-184		Checked by:	Date:

The theoretical resistance of a passive wedge is defined in ETL 1110-2-184 dated 25 Feb 74 as:

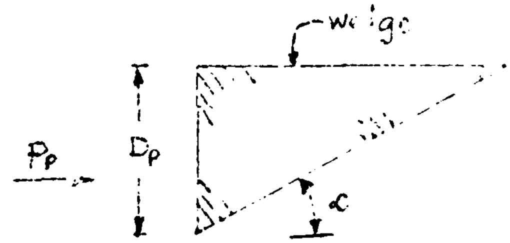
$$P_p = W\tan(\phi+\alpha) + \frac{sA}{\cos\alpha\,(1-\tan\phi\tan\alpha)}$$

This general formula is applicable to sliding along a plane surface in rock or earth. This formula is directly related to the Coulomb equation for passive earth force as shown below.

$A = D_p/\sin\alpha$

$W = \frac{1}{2}\gamma' D_p (D_p/\tan\alpha)$

$\alpha = 45 - \phi/2$



$$P_p = \frac{D_p^2\gamma'}{2\tan\alpha}\tan(\phi+\alpha) + \frac{sD_p}{\sin\alpha\cos\alpha\,(1-\tan\phi\tan\alpha)}$$

But: $\cos\alpha\,(1-\tan\phi\tan\alpha) = \cos\alpha - \sin\alpha\frac{\sin\phi}{\cos\phi} = \frac{\cos(\phi+\alpha)}{\cos\phi}$

$$P_p = \frac{\gamma' D_p^2}{2}\,\frac{\tan(\alpha+\phi)}{\tan\alpha} + \frac{sD_p}{\sin\alpha}\,\frac{\cos\phi}{\cos(\phi+\alpha)}$$

COMPUTATION SHEET

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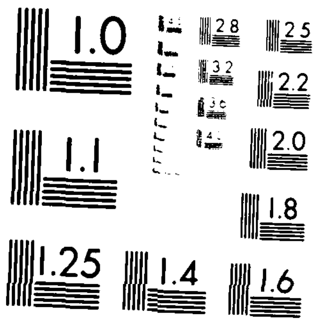

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

SUPPLEMENTARY

INFORMATION

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
P. O. BOX 631
VICKSBURG, MISSISSIPPI 39180

REPLY TO
ATTENTION OF:

AD A0?776

WESKD 24 January 1983

SUBJECT: Replacement Sheets for WES Report, Program Criteria Specifications Document for Computer Program TWDA

TO: All Recipients of Subject Report

1. Those sheets containing pages 8-8, 8-9, and 8-10 should be replaced with the attached sheets containing identically numbered revised pages.

2. We hope that a set of these changes will be replaced into every copy of the original documents in your office. Additional copies of these changes or of the basic documents can be obtained through informal request to Mrs. Rosemary Peck, Engineering Computer Programs Library, FTS: 542-2581.

a fully passive earth pressure due to an earthquake is computed as follows:

$$\Delta P_{pe} = \frac{\gamma H^2}{2} (\Delta K_{pe})$$

where

$$\Delta K_{pe} = K_{pe} - K_p$$

where

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

and

K_p = static passive pressure coefficient for the fully passive case (see 4.1.4b)

For the case where passive earth pressure is used as a stabilizing force, a reduction in the passive earth pressure due to an earthquake acceleration is assumed at the same instant the fill pressure behind the structure is increased. If a reduction in K_p has been used for computing an effective K_p for the static case, this same reduction in ΔK_{pe} is used. ΔP_{pe} is applied at 2/3H above the base. The pressure distribution of ΔP_{pe} is the same as assumed for the active earth pressure condition.

8.5.3 At Rest Earth Pressure Conditions - The increase in an at rest earth pressure due to an earthquake is approximated by the Mononobe-Okabe method. The change in the active earth pressure coefficient ΔK_{ae} is first computed as described in paragraph 8.5.1, and then multiplied by the ratio K_r/K_a to obtain the change in the at rest earth pressure coefficient ΔK_{re} . The change in at rest earth pressure is then computed as follows:

$$\Delta P_{re} = \frac{\gamma H^2}{2} (\Delta K_{re})$$

Only the horizontal component of the earthquake acceleration is considered. ΔP_{re} is applied at two thirds the height of the fill above the base. The pressure distribution of ΔP_{re} is the same as assumed for the active earth pressure condition.

8.6 LATERAL COHESIVE EARTH PRESSURES DUE TO EARTHQUAKES

The computation of the dynamic earth pressure for cohesive soils is beyond the scope of this computer program. A nonlinear finite element analysis to account for inelastic strains in the soil could possibly be used for critical cases.

8.7 WATER PRESSURE DUE TO EARTHQUAKES

8.7.1 Method - If the backfill over the heel is saturated to some level, the increase of soil force over the heel is determined by the Mononobe-Okabe theory described in paragraphs 8.5 and 8.5.1, using the saturated unit weight for submerged earth. When water exists above the soil top surface, the increase of force of the water above the soil top surface is computed by the Westergaard theory described in paragraph 8.7.2. Earthquake on water above the soil top surface on the heel side causes an increase in the total force when the acceleration is positive, a decrease when the acceleration is negative. Earthquake on water above the soil top surface on the toe side causes a decrease in the total force when the acceleration is positive and an increase when the acceleration is negative. The Westergaard theory yields a parabolic shape to the added-pressure diagram. All forces due to earth on the toe side of the stem are computed as a passive reaction to the summation of all other forces. A replacement figure 8-3 for page 8-10 is attached.

The heel-side increase in soil forces due to earthquake is placed in array EH for use in stability calculations and in array EHS for use in stem stress analysis, after the inversion shown in diagram (4) in Figure 8-3. The net dynamic pressure diagram of the heel-side Westergaard water pressures (a parabola) summed algebraically with the toe-side dynamic pressures (another parabola) is placed in array EFH. See Chapter 11 in the User's Reference Manual for more detail on these arrays and how the user can modify the computed values or substitute his own.

8.7.2 Westergaard Theory - By the Westergaard theory, the dynamic water pressure down to depth y below the surface for a total water depth h [D]
is expressed by Equation 3 on page 5 of EM 1110-2-2200 as

$$P_{e_2} = \frac{2}{3} C_e \alpha y \sqrt{hy}$$

The additional moment at depth y due to P_{e_2} is given by

$$M_e = \frac{4}{15} C_e \alpha y^2 \sqrt{hy}$$

with

$$C_e = \frac{51}{\sqrt{1 - 0.72 \left(\frac{h}{1000 \, t_e} \right)^2}}$$

where g is acceleration of gravity (32.2 ft/sec^2).

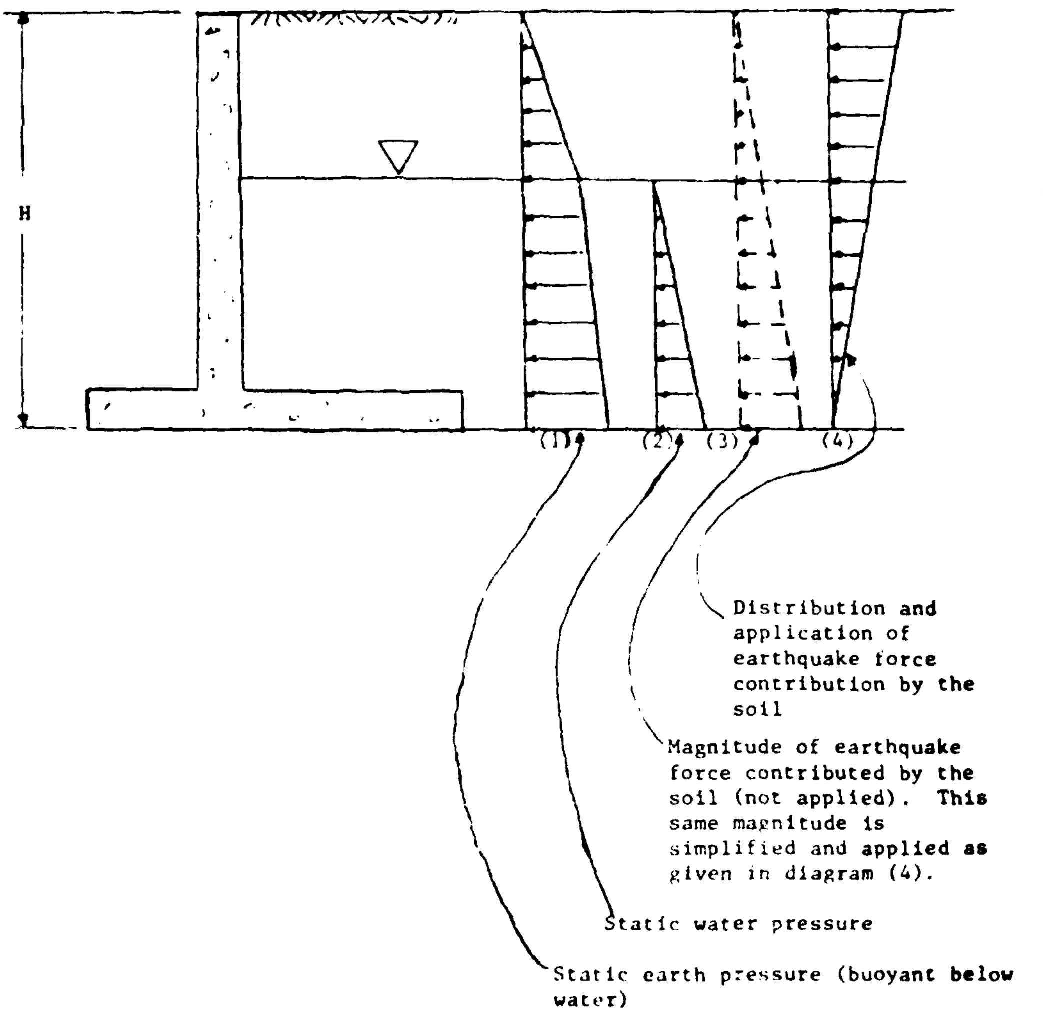

Figure 8-3. DYNAMIC PRESSURE VARIATIONS

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