A Practical Review of Earth Pressure Theory and the Factors Affecting Earth-Pressure Retaining Structures

Stan Vitton, Ph.D., P.E.

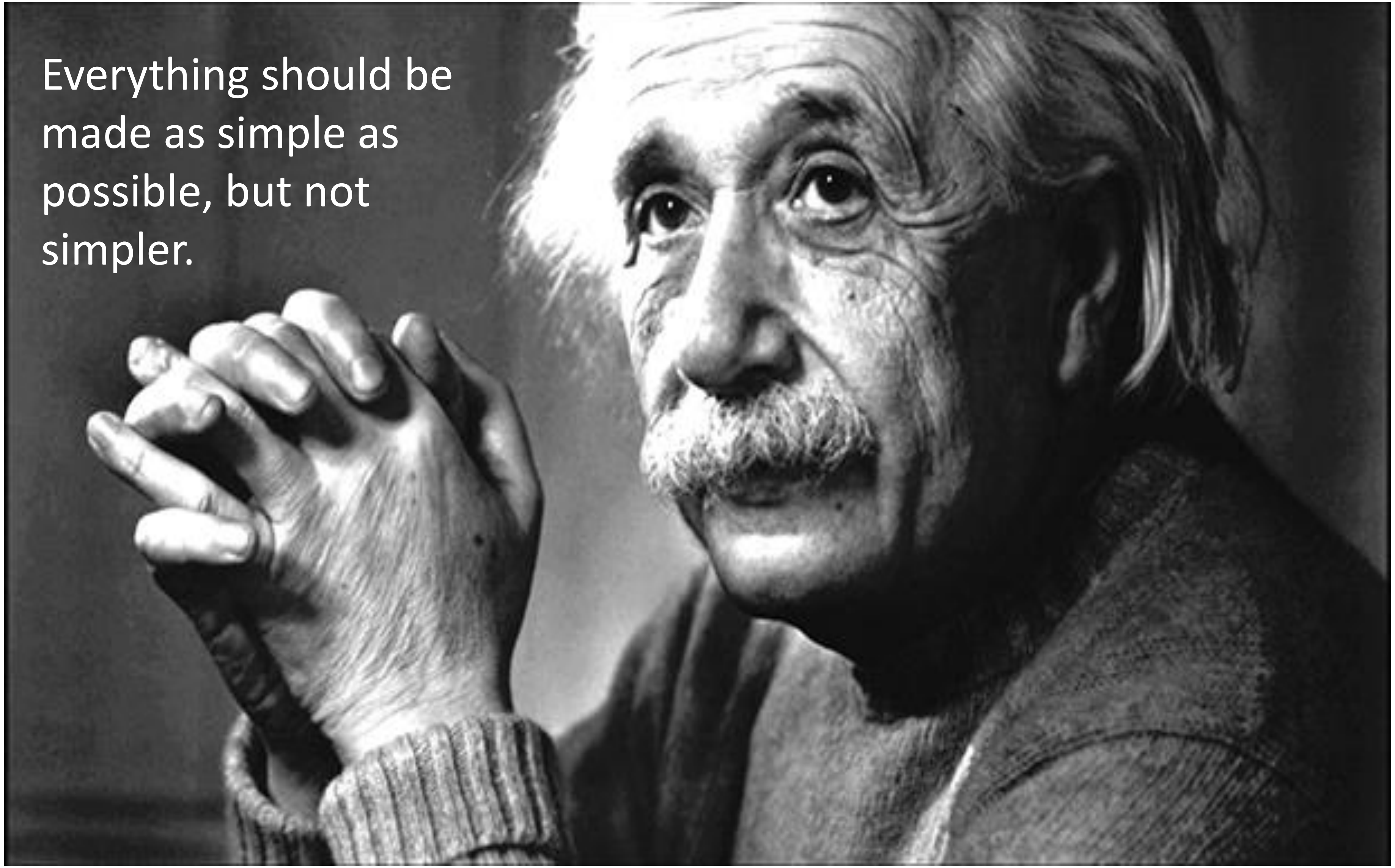
Civil & Environmental Engineering

Michigan Technological University

A Practical Review of Earth Pressure

Two primary topics:

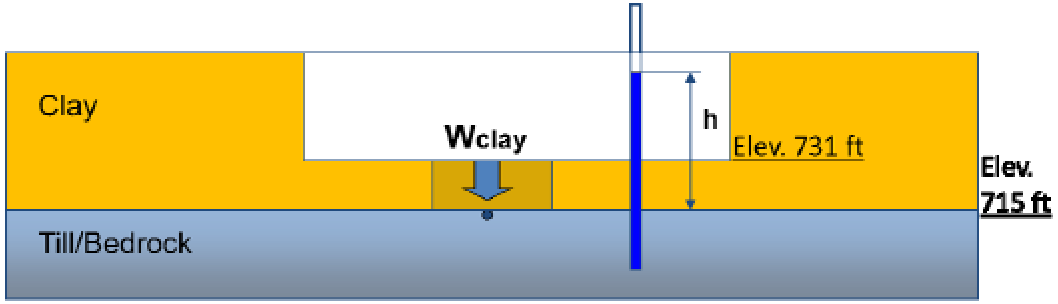
1. A practical review of earth pressure theories used in the design of earth retaining structures
2. Case study of a $8 million dollar slope failure comparing a simple conservative analysis to a Flac-3D analysis.

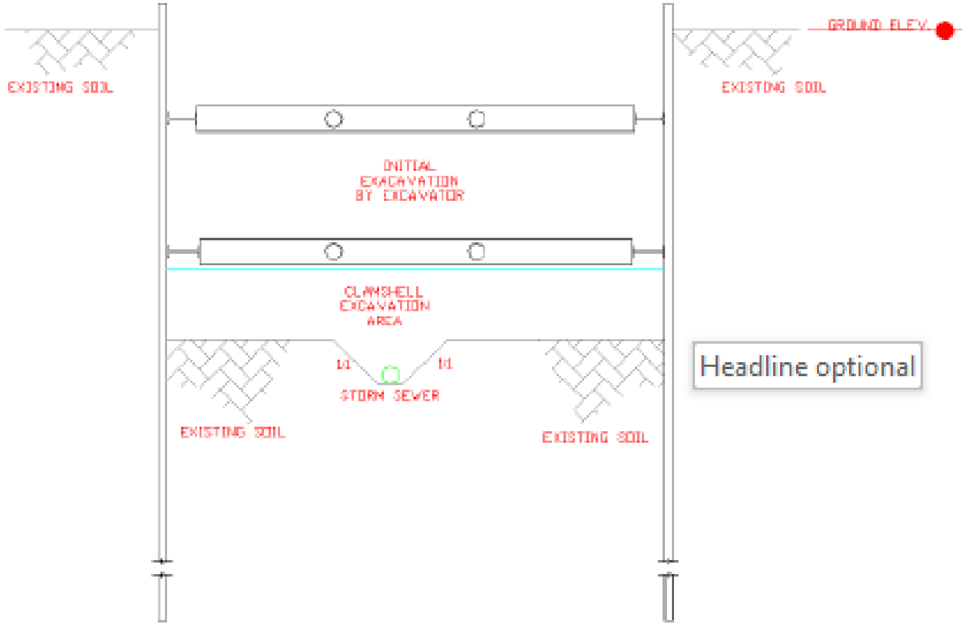

Everything should be made as simple as possible, but not simpler.

Introduction:

- How I got myself into this mess
- Michigan DOT Sheet Pile Manual
- ASCE 1st Conference on Earth Structures
  - Does anyone do hand calculations anymore?

Senior Design Project: Concrete Tunnel Under a Major Railway in Canada in Soft Saturated Clay



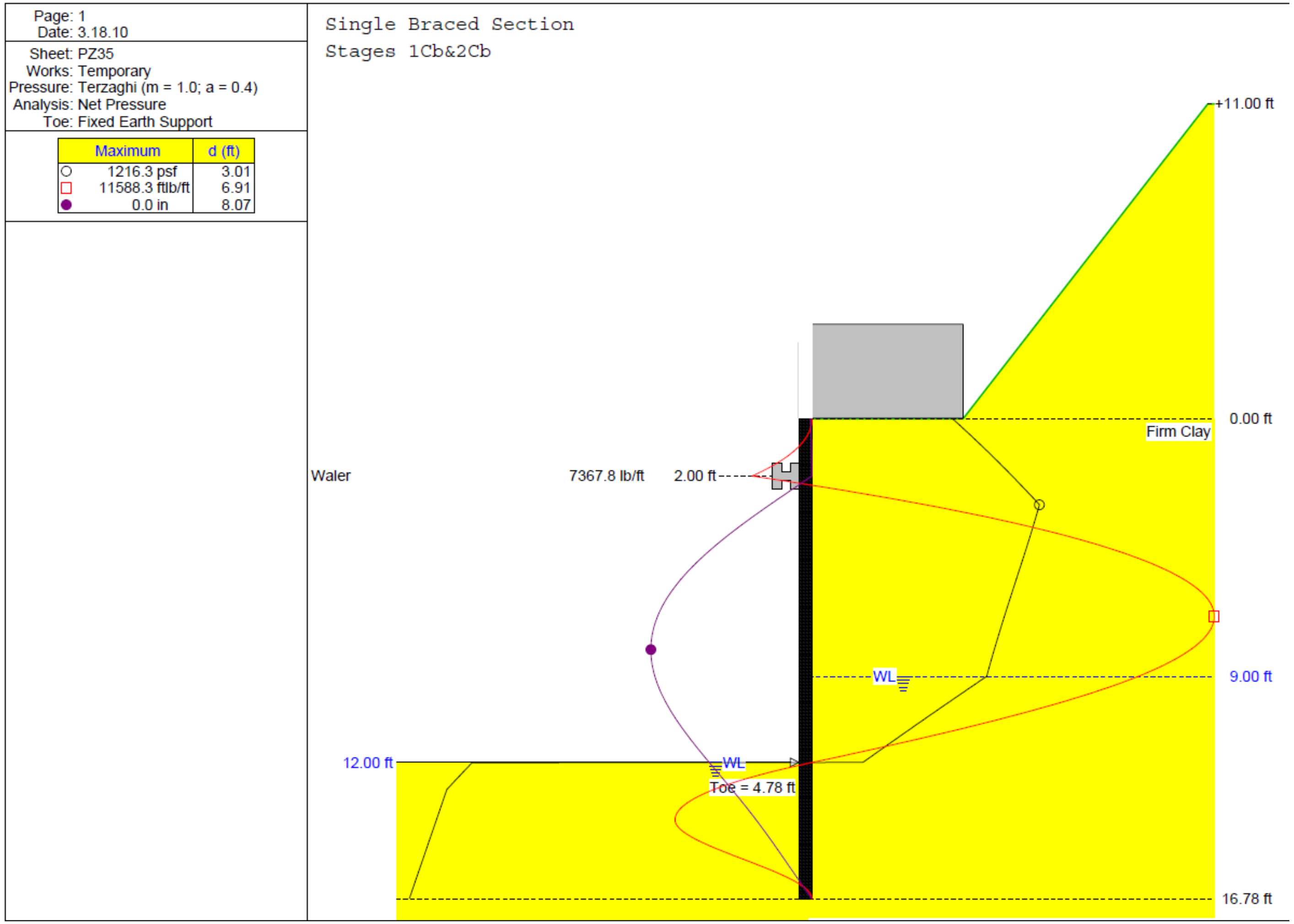


Headline optional

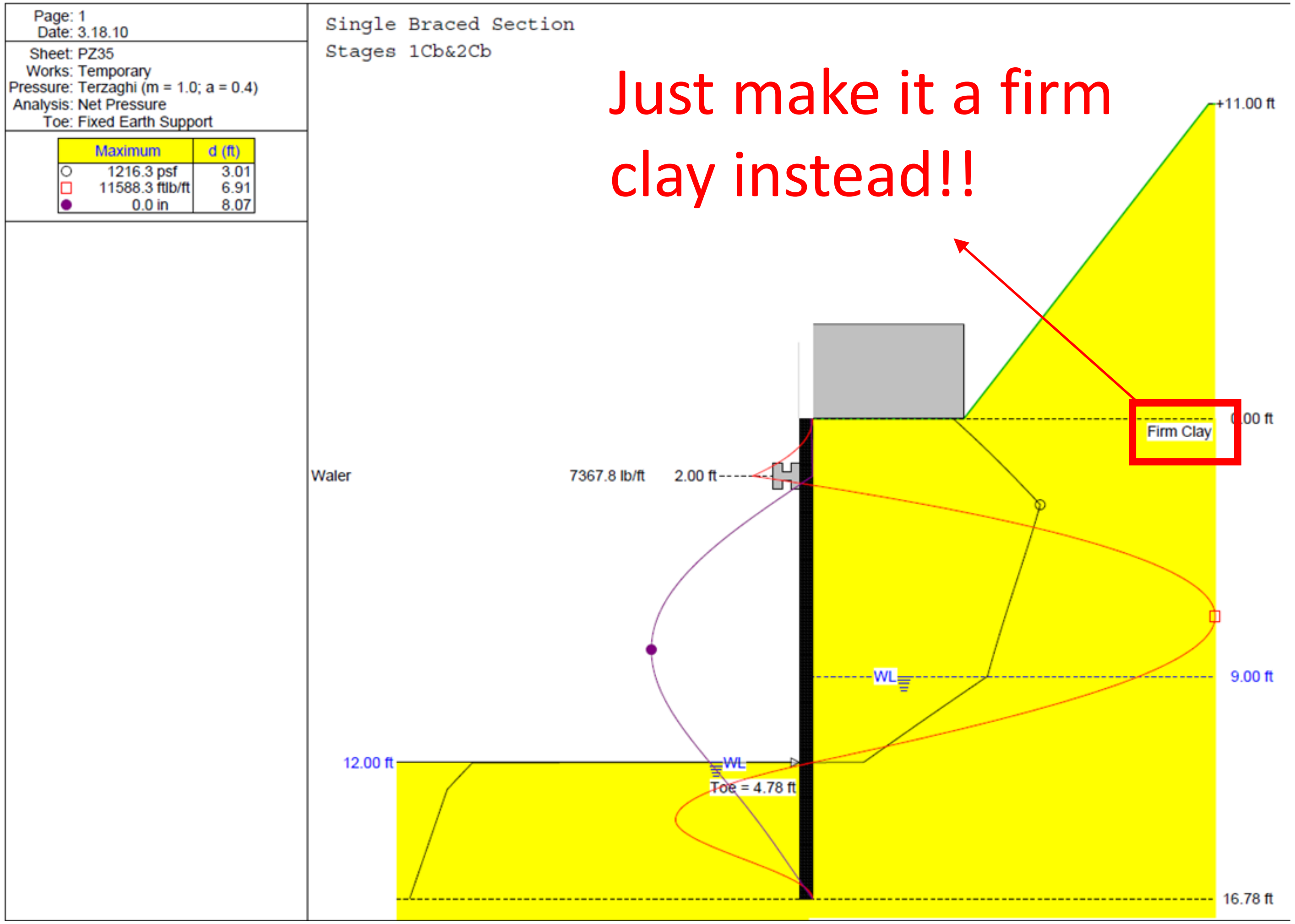
Figure 5: Illustration of Excavation Sequencing (with Depth) in Areas with Struts.

Senior Design Project: Concrete Tunnel Under a Major Railway in Canada in Soft Saturated Clay

SupportIT Software



So why does standard sheet pile programs calculate such deep embedment depths for soft clays?



- The problem is the assumptions made in the classical design methods:
  - $K_a = K_p$
  - Assuming that the strength of the "soft" clay doesn't change with depth, therefore
  - Passive Resistance:

    $\sigma_p = 2S_u$

Report No. RC-1633

Report Title:

A Manual for the Design of Temporary Earth Retention Systems (TERS) for the Michigan Dept. of Transportation

Center for Structural Durability at Michigan Tech – a MDOT Center of Excellence

FINAL REPORT

Part 1 of 2

Sponsoring Organization: Michigan Department of Transportation

Research Agency:

Michigan Technological University
1400 Townsend Drive
Houghton, MI 49931

Principal Investigator(s):

Theresa (Tess) M Ahlborn, PhD, P.E., FACI, FPCI
Professor
Civil & Environmental Engineering
Director, Center for Structural Durability

R. Andrew Swartz, PhD, P.E.
Associate Professor
Civil & Environmental Engineering

Stanley J. Vitton, PhD, P.E.
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Date Submitted: April 22, 2019

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Design of Temporary Earth Retention Systems (TERS)
for the Michigan Department of Transportation

Part 1 of 2

Michigan Technological University

Case 6, Step 1: Define the Dimensions and Soil Properties to be Analyzed for the Cantilever Wall

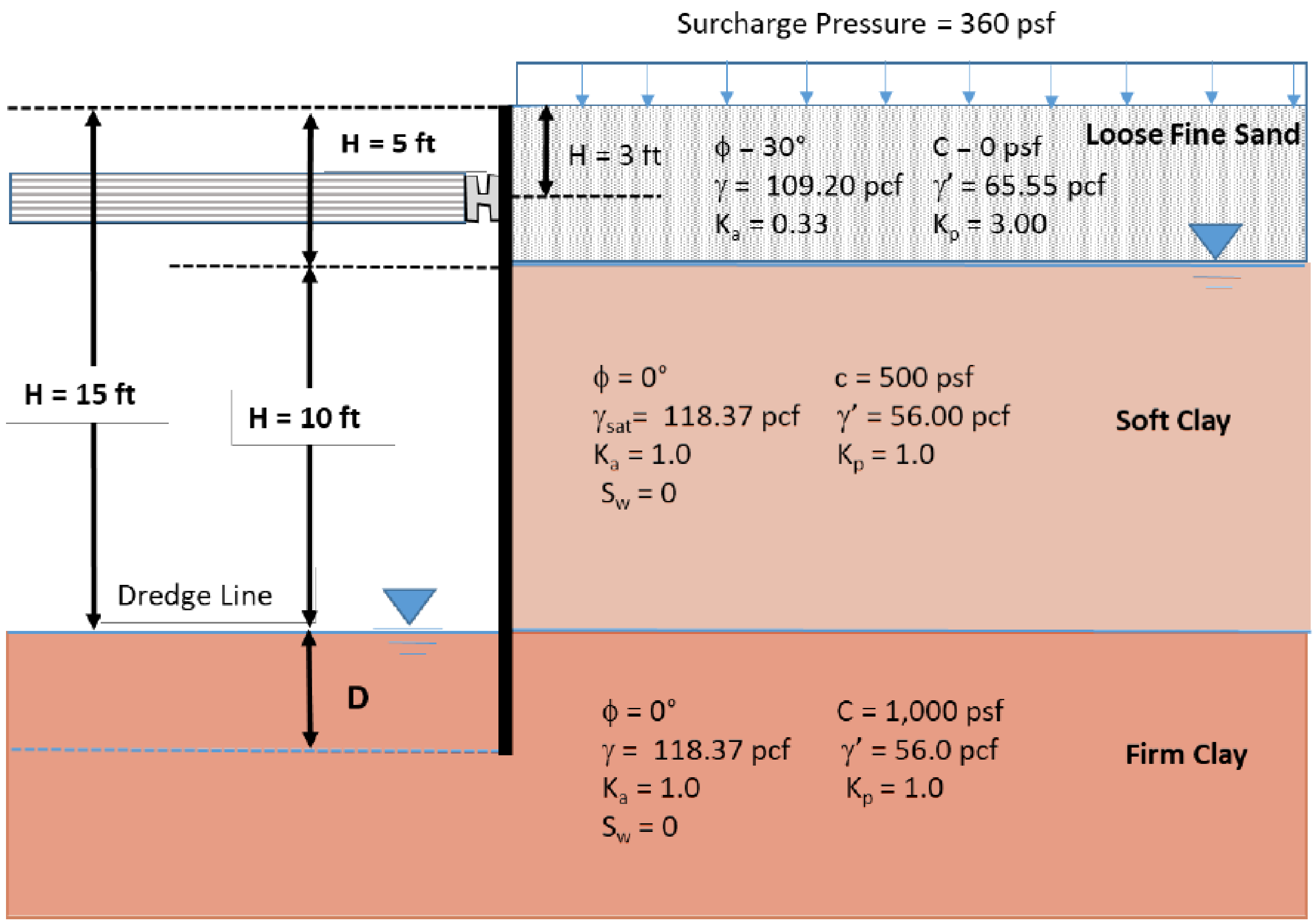


Figure 5-22 Case 6 Braced TERS in soft and firm clay.

Active Pressures:

Surcharge Pressure in sand backfill: $\sigma_{a1} = K_a\sigma_v$ = (0.33)(360 psf) = **118.8 psf**

Sand backfill: $\sigma_{a2} = K_a\sigma_v$ = (0.33)(5 ft)(109.2 pcf) = 180.2 psf

$\sigma_{a5} = \sigma_{a1} + \sigma_{a2}$ = 118.8 + 180.2 = **299 psf**

Minimum Fluid Pressure: $\sigma_{a5'} = K_a\gamma H$ = (1.0)(31.8)(5) = **159.0 psf**

$\sigma_{a7.92'} = K_a\gamma H$ = (1)(31.8)(7.92) = **251.8 psf**

Active Soil, Soft Clay Layer: *The active pressure starts at a depth of 5.79 ft,*

That is, $\sigma_{a5.79'}$ = 0 psf

$\sigma_{a7.92'} = K_a\gamma H$ = (1.0)(118.37)(7.92-5.79) = 252.1 psf

$\sigma_{a15'} = K_a\gamma H$ = (1.0)(118.37)(15.0 - 5.79) = **1,090.2 psf**

Active Soil, Firm Clay Layer: *Note: The critical height of the firm clay is -0.76 ft at a depth of 15 feet. That is, SupportIT will use the active pressure from a depth of 15-0.76 = 14.24 ft.*

$\sigma_{a15.08'} = K_a\gamma H$ = (1.0)(118.37)(15.08 – 14.24) = 99.4 psf

$\sigma_{a17.41'} = K_a\gamma H$ = (1.0)(118.37)(17.41 – 14.24) = 375.2 psf

$\sigma_{a20'} = K_a\gamma H$ = (1.0)(118.37)(20.0 – 14.24) = 681.8 psf

Active Water Pressure: $\sigma_{a15'} = K_a\gamma H$ = (10)(62.4) = **624.0 psf** > 99.4 psf (active)

$\sigma_{a20'} = K_a\gamma H$ = (15)(62.4) = **936 psf** > 681.8 psf (active)

Note: The water pressure exceeds the clay active pressure, therefore, the higher water pressure governs in the firm clay layer between 15-20 ft.

Passive Soil Pressures to a depth of 20 feet:

Lateral Pressure below Groundwater: σ_{p15} = 2c = 2(1,000) = **-2,000 psf** (constant with depth)

$\sigma_{p15.08}$ = 2000 + (15.08 – 15.00)(118.37) = -2,009.5 psf

$\sigma_{p17.41}$ = 2000 + (17.41 – 15.00)(118.37) = -2,009.5 psf

σ_{p20} = 2000 + (20 – 15.00)(118.37) = **2,591.9 psf**

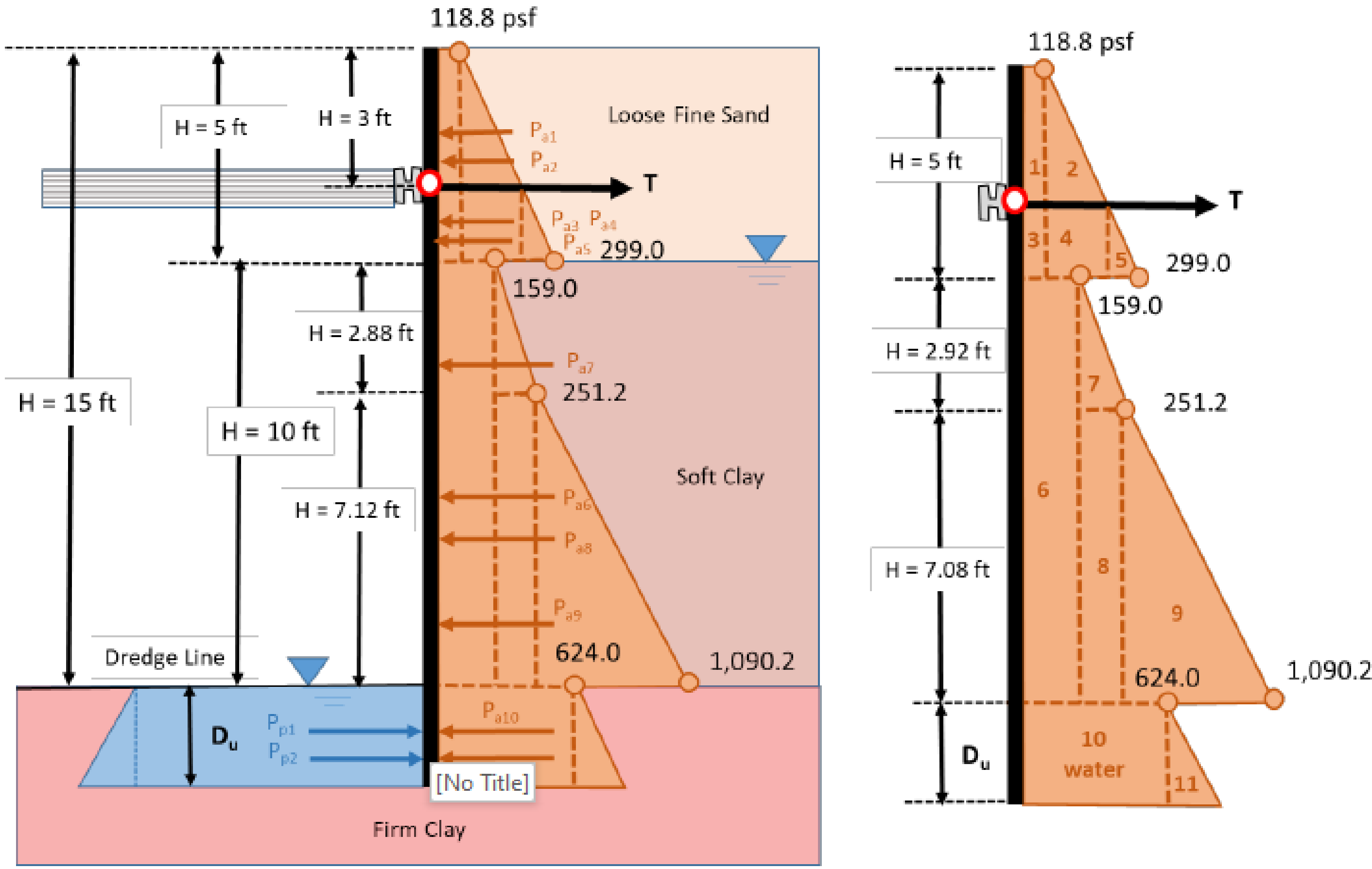


Figure 5-24 Case 6 Lateral and passive forces acting on the wall.

Table 5-17 Case 6 Embedment depth, D, for *FOS* = 1.0.

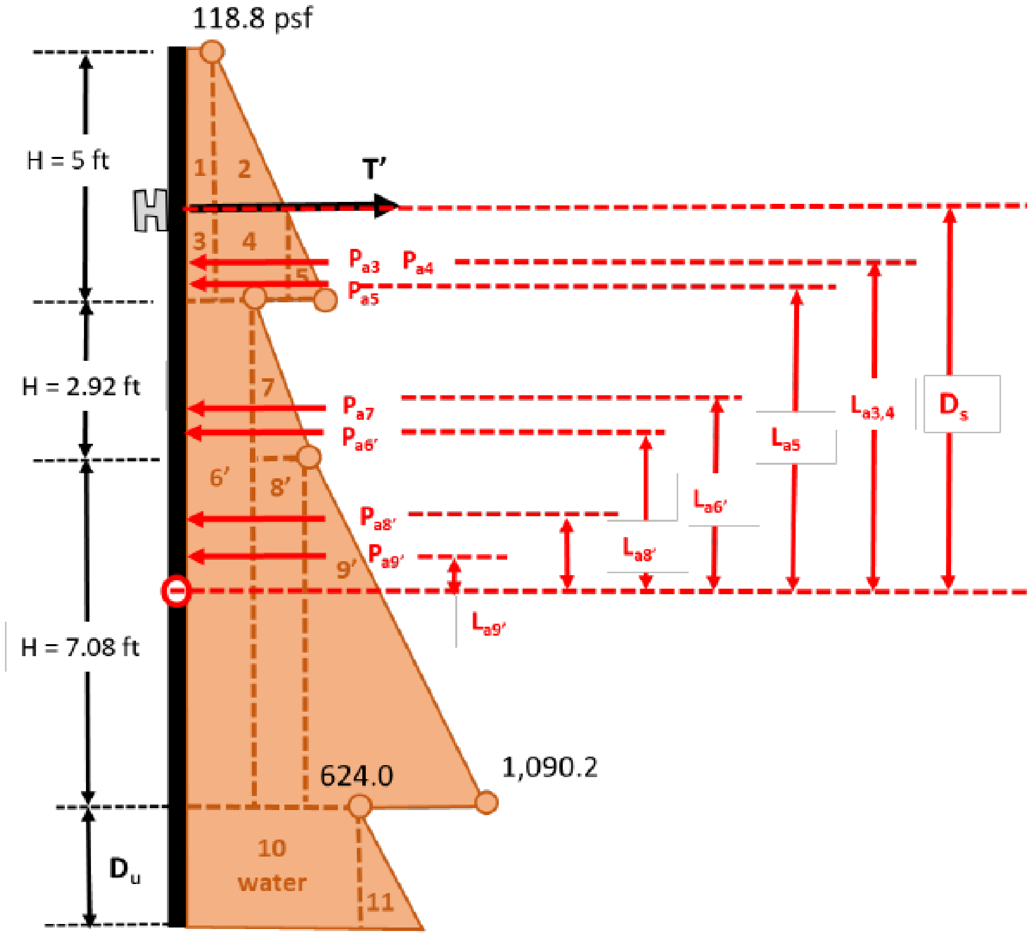


Figure 5-25 Determination of zero-shear location on sheet pile wall.

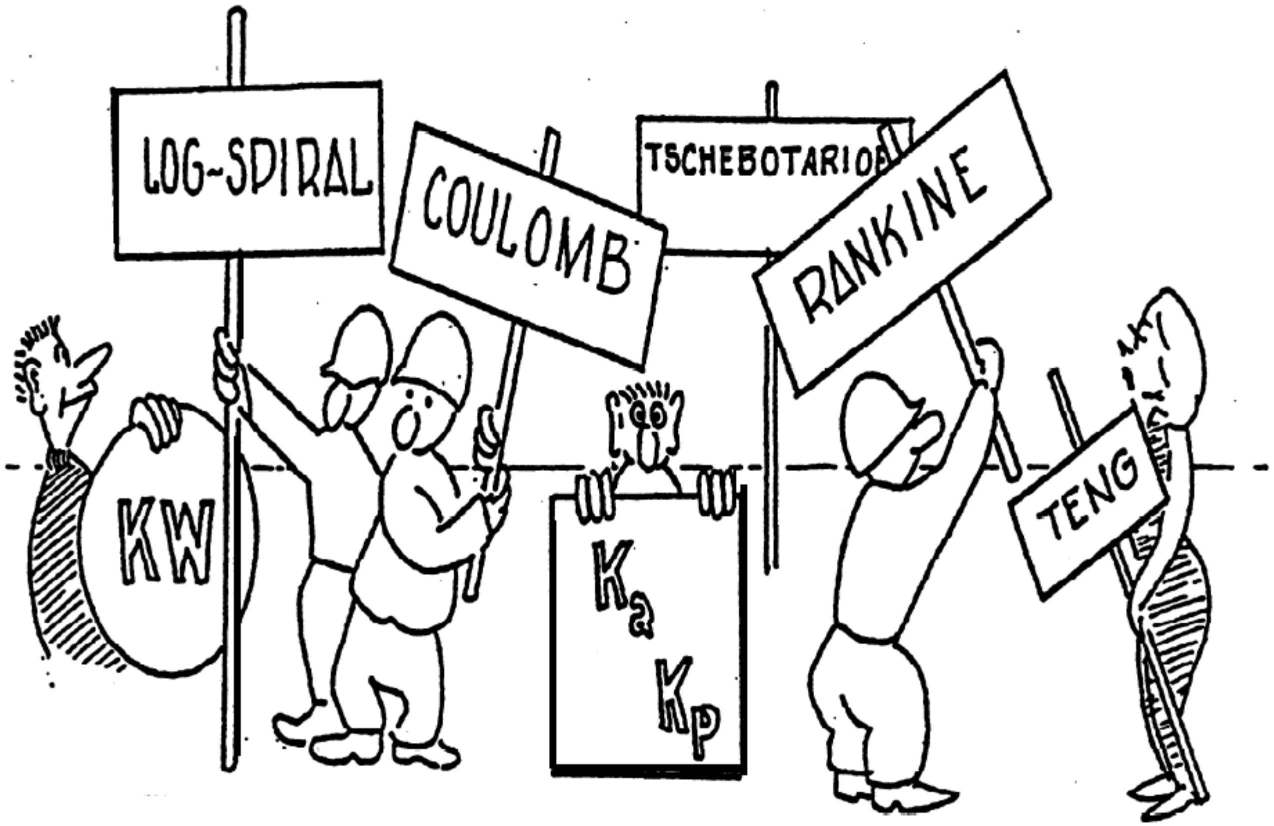
Calculation of Sheet Pile Depth, D_u (FOS = 1.0)			
Depth of Sheet Pile, D_u (ft)		2.42	
FOS		1.00	
Restoring Moment		Disturbing Moment:	
P_{a1}	356.4	P_{a3}	237.6
L_{a1}	1.5	L_{a3}	1.0
P_{a2}	162.2	P_{a4}	216.2
L_{a2}	1.0	L_{a4}	1.0
P_{P1}	4,838.8	P_{a5}	72.1
L_{P1}	13.2	L_{a5}	1.3
P_{P2}	345.9	P_{a6}	1590.0
L_{P2}	13.6	L_{a6}	7.0
M_r	68,794	P_{a7}	135.6
		L_{a7}	3.9
		P_{a8}	657.4
		L_{a8}	8.4
		P_{a9}	2966.7
		L_{a9}	9.6
		P_{a10}	1509.7
		L_{a10}	13.2
		P_{a11}	182.6
		L_{a11}	13.6
		M_d	68,759

	SupportIT (Total pile length, ft)	Hand Calculations (Total pile length, ft)
Maximum soil pressure at dredge line, (psf/ft)	1,089.7	1,090.2
Anchor Load, (lbs/ft)	2,948	2,901
Zero Shear location along sheet pile, (ft)	10.94	10.87
Maximum Moment, (ft-lbs/ft), FOS = 1.0	10,650	10,674
Sheet Pile Embedment, FOS = 1.00, D_u (ft)	2.42	2.42
USS 20% FOS Embedment Length, D_f (ft)	2.9 (18)	2.9 (18)
USS 40% FOS Embedment Length, D_f (ft)	3.3 (18.5)	3.4 (18.5)
CP2 FOS Embedment Length, D_f (ft) FOS = 1.5	4.38 (19)	4.37 (19)

Thoughts on earth pressure theories

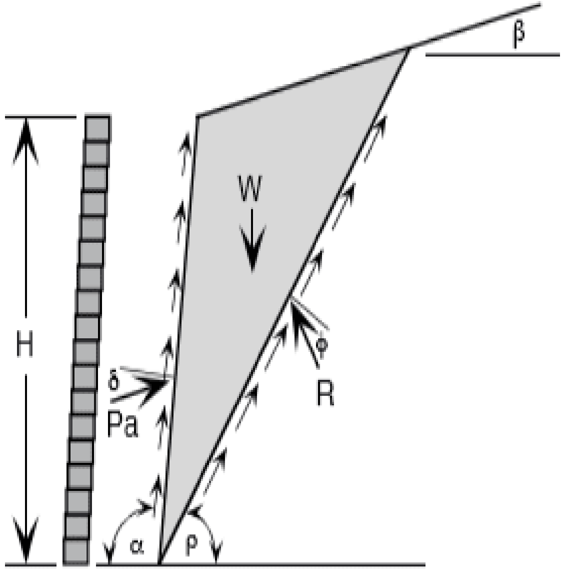
- What earth pressure theory should be used????
- What about sloped backfills?
- What critical height, h_c, should be used in clay?

What earth pressure theory should be used????



- <u>CalTran Revision 12, 1990</u>
  - Rankine?
  - Coulomb?
  - Log spiral?
  - Tschebotarioff?
  - Teng?

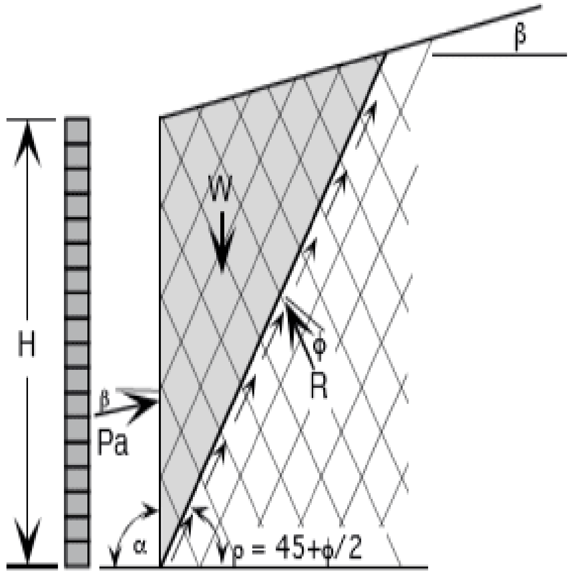
For level backfill, Coulomb and Rankine provide the same earth pressures but not so for sloped backfill



$$P_{ah} = 1/2\,\gamma\,H^2\,K_a\,\mathrm{Cos}(\delta-(90-\alpha))$$

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

Coulomb Wedge Analysis



$$P_{ah} = 1/2\,\gamma\,H^2\,K_a\,\mathrm{Cos}(\beta)$$

$$K_a = \cos\beta\left[\frac{\cos\beta-\sqrt{\cos^2\beta-\cos^2\phi}}{\cos\beta+\sqrt{\cos^2\beta-\cos^2\phi}}\right]$$

Rankine "state of stress" Analysis

Rankine Theory

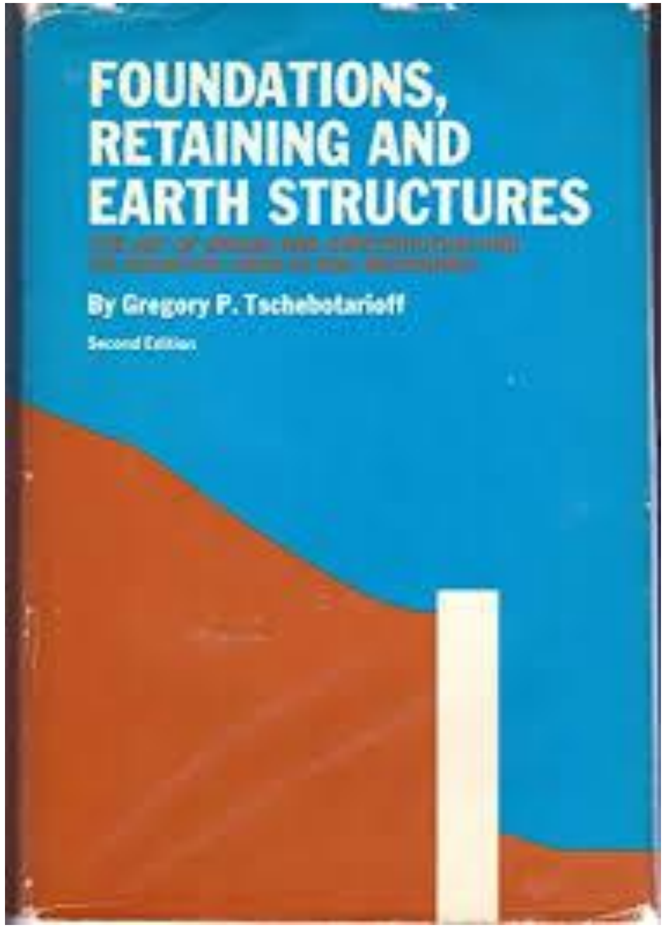
The Rankine formula for passive pressure can only be used correctly when the embankment slope angle β equals zero or is negative. If a large wall friction value can develop, the Rankine Theory is not correct and will give less conservative results. Rankine's theory is not intended to be used for determining earth pressures directly against a wall (friction angled does not appear in equations above). The theory is intended to be used for determining earth pressures on a vertical plane within a mass of soil.

Coulomb Theory

Since wall friction requires a curved surface of sliding to satisfy equilibrium, the Coulomb formula will give only approximate results as it assumes planar failure surfaces. The accuracy for Coulomb will diminish with increased depth. For passive pressures the Coulomb formula can also give inaccurate results when there is a large back slope or wall friction angle. These conditions should be investigated and an increased factor of safety considered.

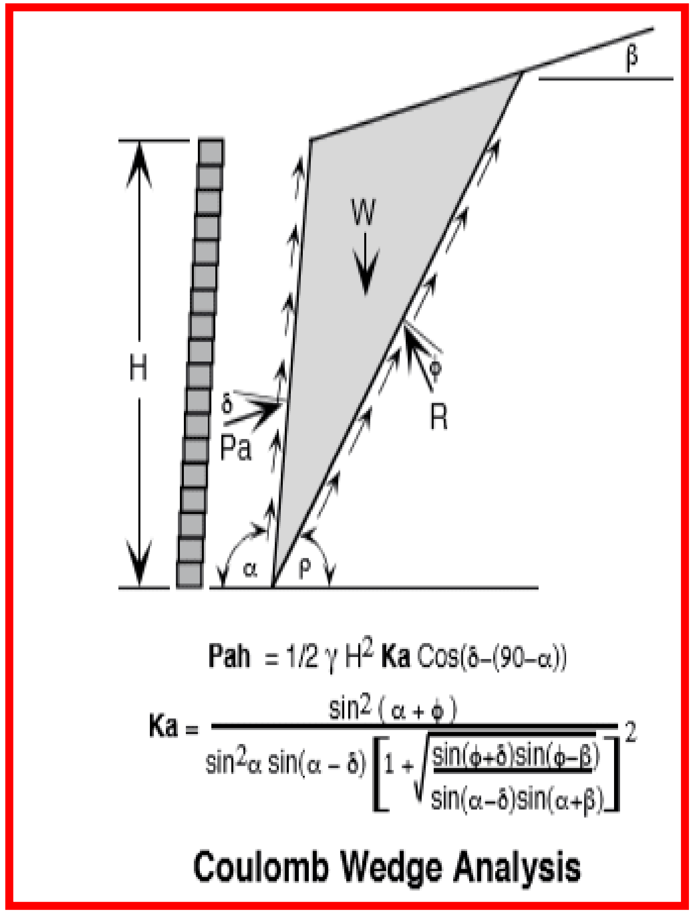
CalTran Revision 12, 1991

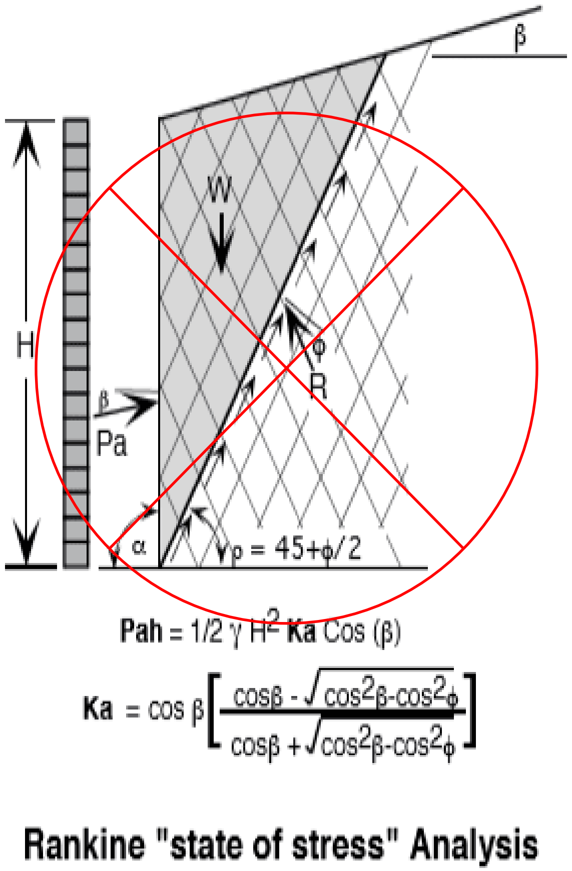
Gregory Tschebotarioff Lecture



"……. A further description of the Rankine Method of analysis and the deviation of his formula will therefore be omitted as unnecessary ballast for the general civil engineering practitioner" G. Tschebotarioff,

For level backfill, Coulomb and Rankine provide the same earth pressures but not so for sloped backfill





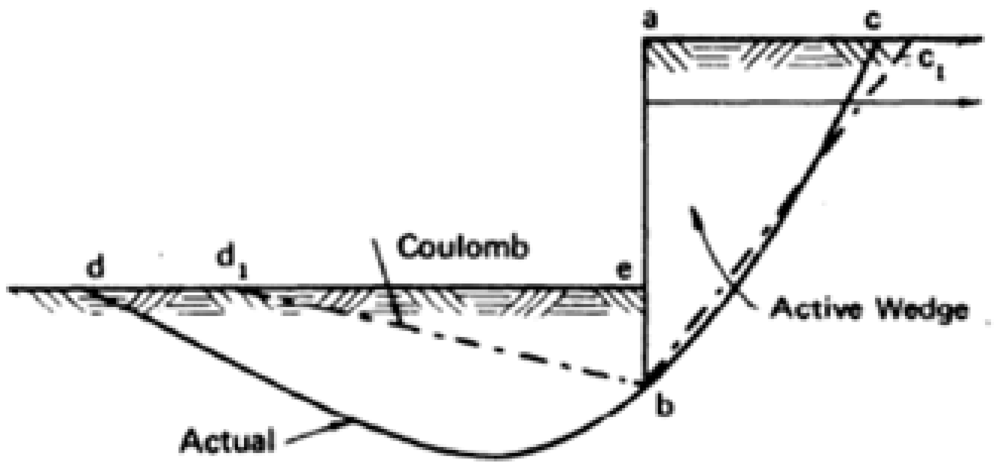
A couple of thoughts from writing the MDOT Sheet Pile Manual

- What earth pressure theory should be used????
- What about sloped backfills?
- In clay what critical height, h_c, should be used?

Log Spiral Caquot and Kérisel Method

CalTran Revision 12, 1991

A Log-spiral theory was developed because of the unrealistic values of earth pressures that are obtained by theories which assume a straight line failure plane. The difference between the Log-Spiral curved failure surface and the straight line failure plane can be large and on the unsafe side for Coulomb passive pressures (especially when wall friction exceeds $\phi/3$). The following figure shows a comparison of the Coulomb and Log-Spiral failure surfaces:



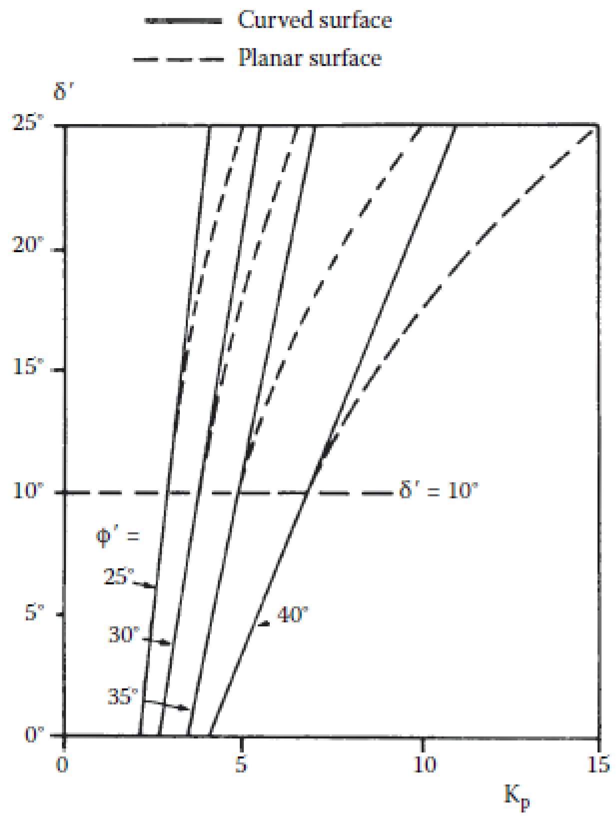
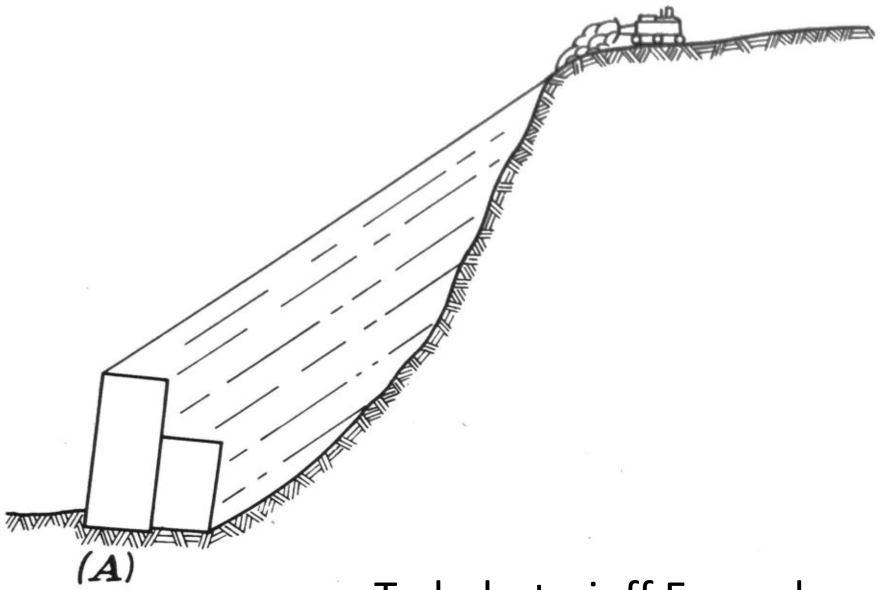


Figure A.18 Effect of wall friction on passive earth pressure coefficient ($c' = 0$, $c'_w = 0$, vertical back of wall and horizontal ground surface).



Tschebotarioff Example

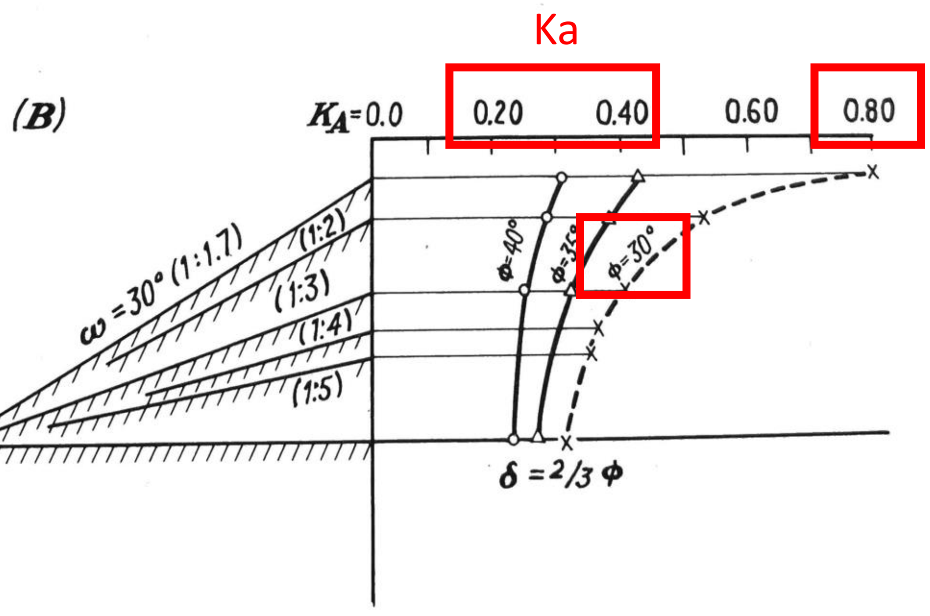


Fig. 10-10. (*A*) Damage to a 24-ft-high (7.3-m) crib wall backfilled by sand bulldozed over the edge of slope. (*B*) Explanation of the causes of trouble in (*A*).

USS Sheet Pile Manual:

Figure 5a (Caquot-Kersiel chart)

ϕ = 35° & β/ϕ = -0.5

K_p' = about 10

$K_p = K_p' \times R_d$

K_p = ~10 x 0.674 ≈ 6.7

Close to K_p = 6.56 OK!

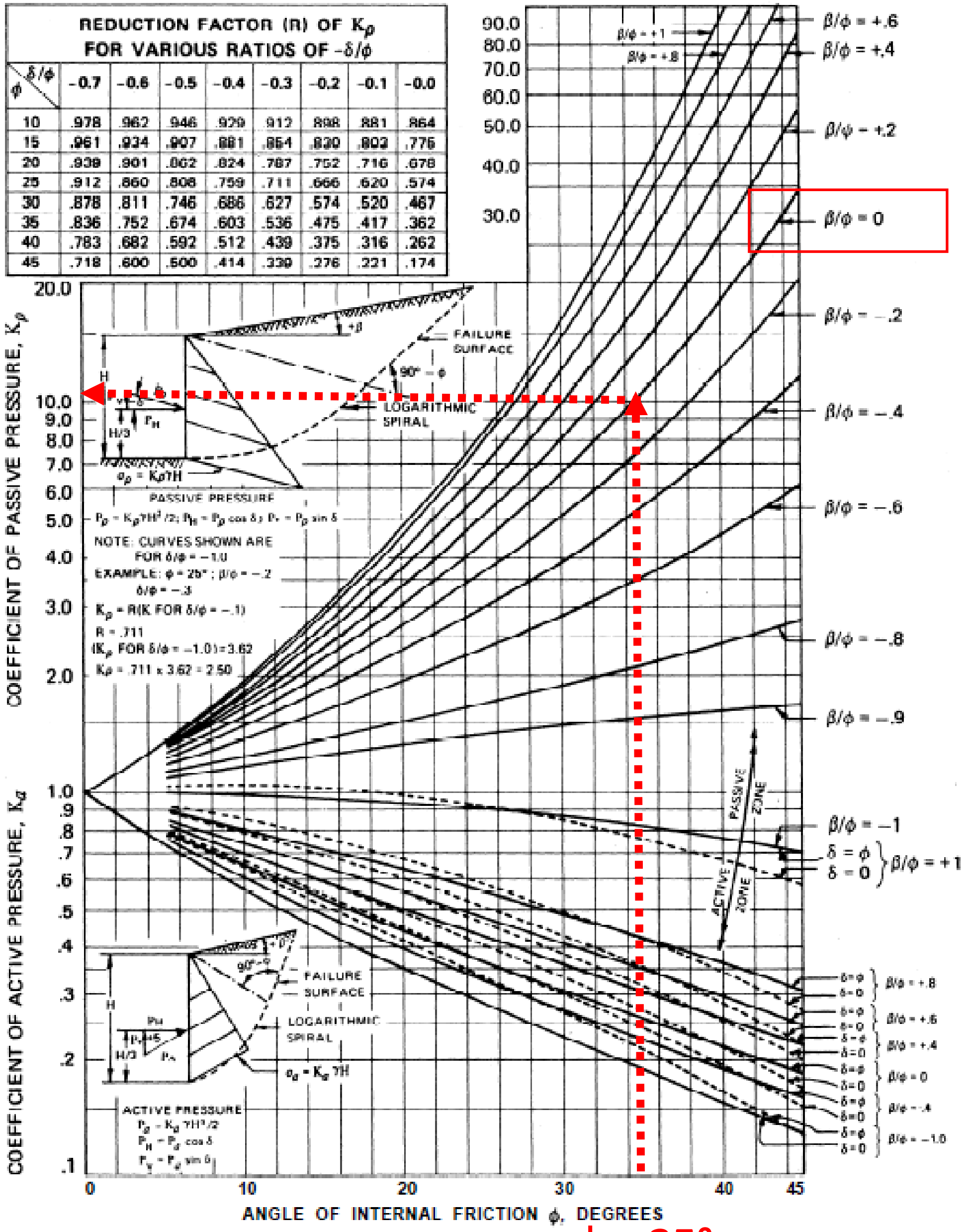
SupportIT© uses European Code 7 (1995):

$$K_h = cos^2\beta.\left[\frac{1+\sin\emptyset' \sin(2m_w + \emptyset')}{1-\sin\emptyset' \sin(2m_t + \emptyset')}\right].\exp(2v\tan\emptyset')$$

$$m_t = \frac{cos^{-1}\left(-\frac{\sin\beta}{\sin\emptyset'}\right) - \emptyset' - \beta}{2}$$

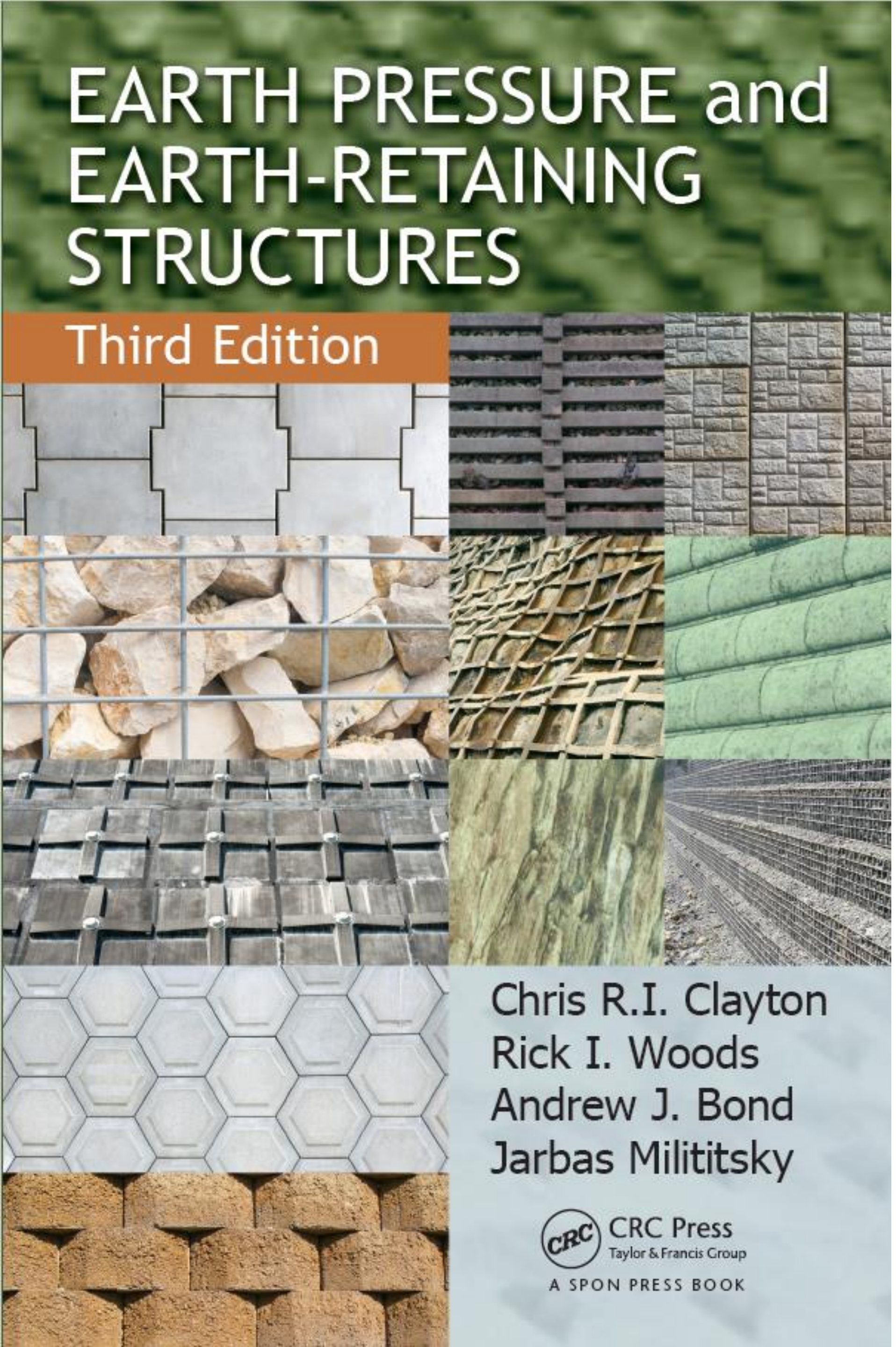
$$m_w = \frac{cos^{-1}\left(\frac{\sin\delta}{\sin\emptyset'}\right) - \emptyset' - \delta}{2}$$

$$v = m_t + \beta - m_w$$



ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.776
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

Fig. 5(a) - Active and passive coefficients with wall friction (sloping backfill) (after Caquot and Kerisel[21])



- One of the more comprehensive book available
  - Great Britain but based on EU Standards
  - Covers a range of walls
  - Cover a number of design methods
    - Limiting equilibrium
    - Discrete-spring models
    - Continuum models

Table 3.2 Development of analytical and graphical earth pressure coefficients

	Capability						
Source	α (°)	β (°)	c'	ϕ'	c'_w	δ'	A/P
Rankine (1857)	90	β		ϕ'		$= \beta$	A/P
Mayniel (1808)	90	0		ϕ'		δ'	A
Müller-Breslau (1906)	α	β		ϕ'		δ'	A
Bell (1915)	90	0	c'	ϕ'	0	0	A/P
Caquot and Kerisel (1948)	α	β		ϕ'		δ'	P
BS CP2 (1951) based on Packshaw (1946)	90	0	c'	ϕ'	c'_w	δ'	A/P
BS 8002 (1994) based on Kerisel and Absi (1990) and Bell (1915)	90	β	c'	ϕ'	c'_w, c'_w	δ'	A/P
BS EN 1997 (2004)	α	β	c'	ϕ'	c'_w	δ'	A/P

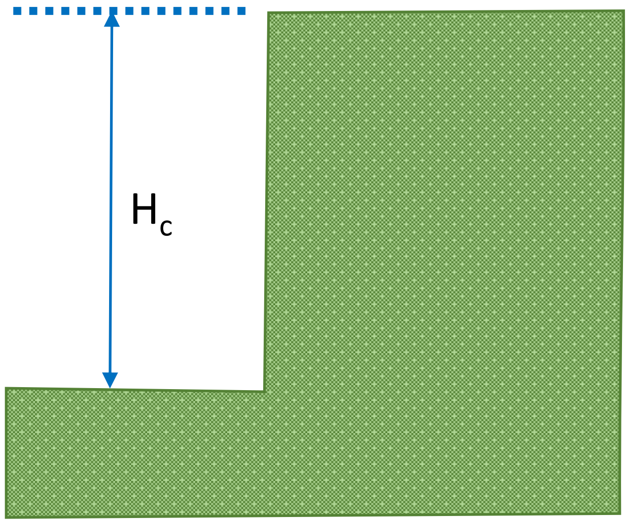
Note: A, active; P, passive.

A couple of thoughts from writing the MDOT Sheet Pile Manual

- What earth pressure theory should be used????
- What about sloped backfills?
- In clay what critical height, h_c, should be used?

Cohesive Soils: For stiff soils how high can the soil stand unsupported, h_c??

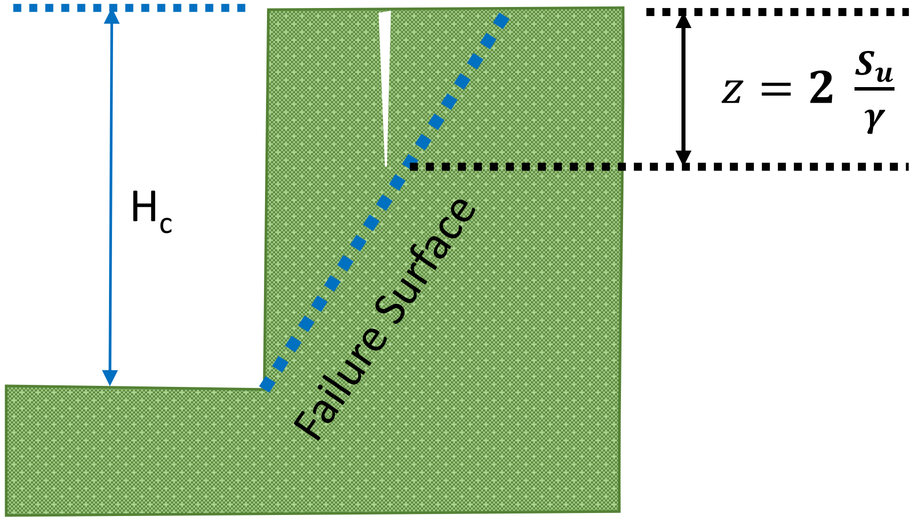
$$H_c = \frac{2c - q_{surcharge}}{\gamma}$$



$H_c = \mathbf{3.85}\ \frac{S_u}{\gamma}$ Fellenius (1927)

$H_c = \mathbf{4}\ \frac{S_u}{\gamma}$ Terzaghi (1943)

Tension Crack intersecting a potential failure surface



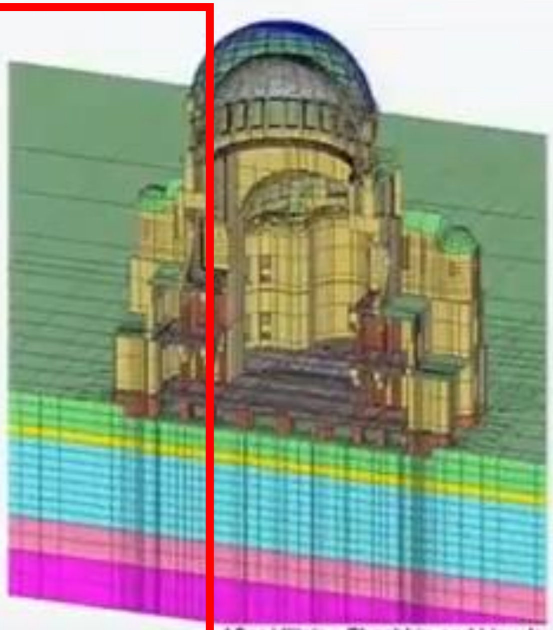
$H_c = \mathbf{2.67}\ \frac{S_u}{\gamma}$ Terzaghi (1943)

Part II
Case Study: $8 million dollar slope failure

Gregory Tschebotarioff Lecture

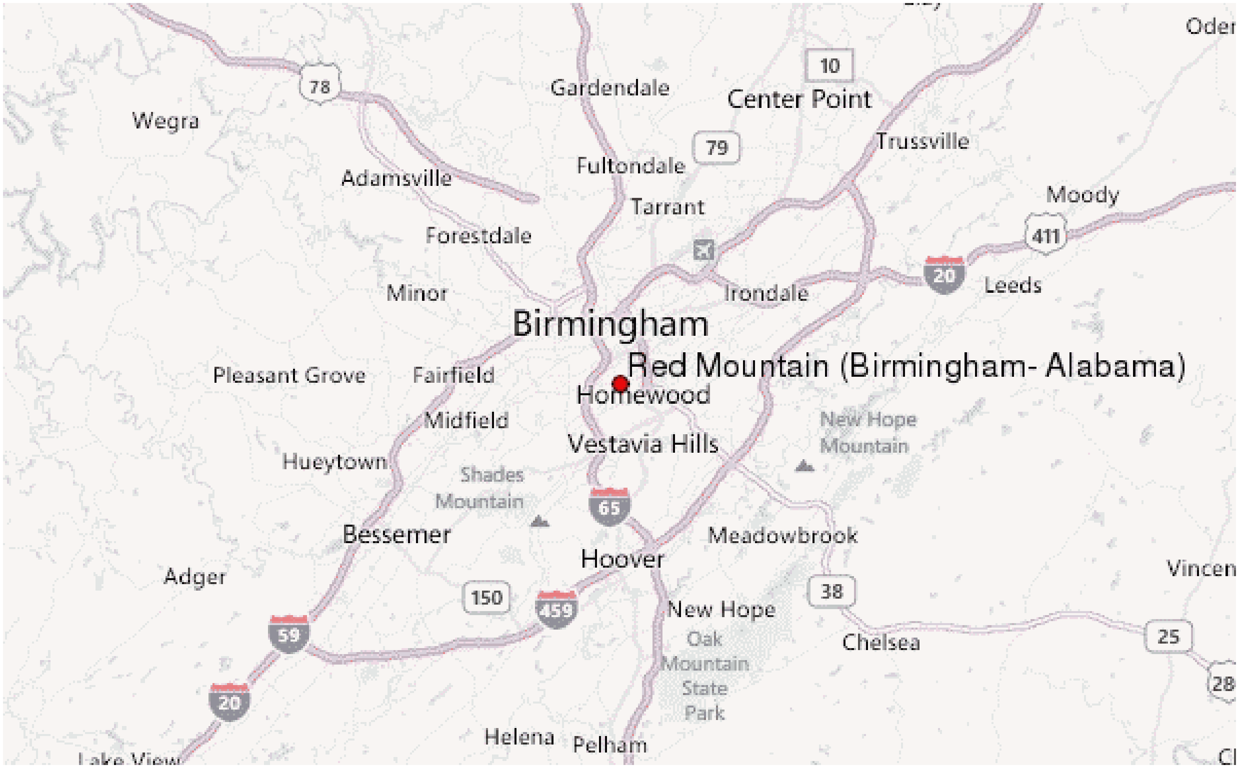
SSI Tools

- Complexity (i.e. time and cost) of tools ranges from
  - Simple equations, to
  - Three-dimensional FEA
- Tendency to use complex methods even when not required
- Value of simple methods:
  - Understanding of the system
  - Calibration and sensitivity checks
  - Essential for checking more sophisticated analysis
  - Fast and inexpensive


After Ulitsky, Shashkin and Lisyuk

Case Study – A simple $8 million dollar failure

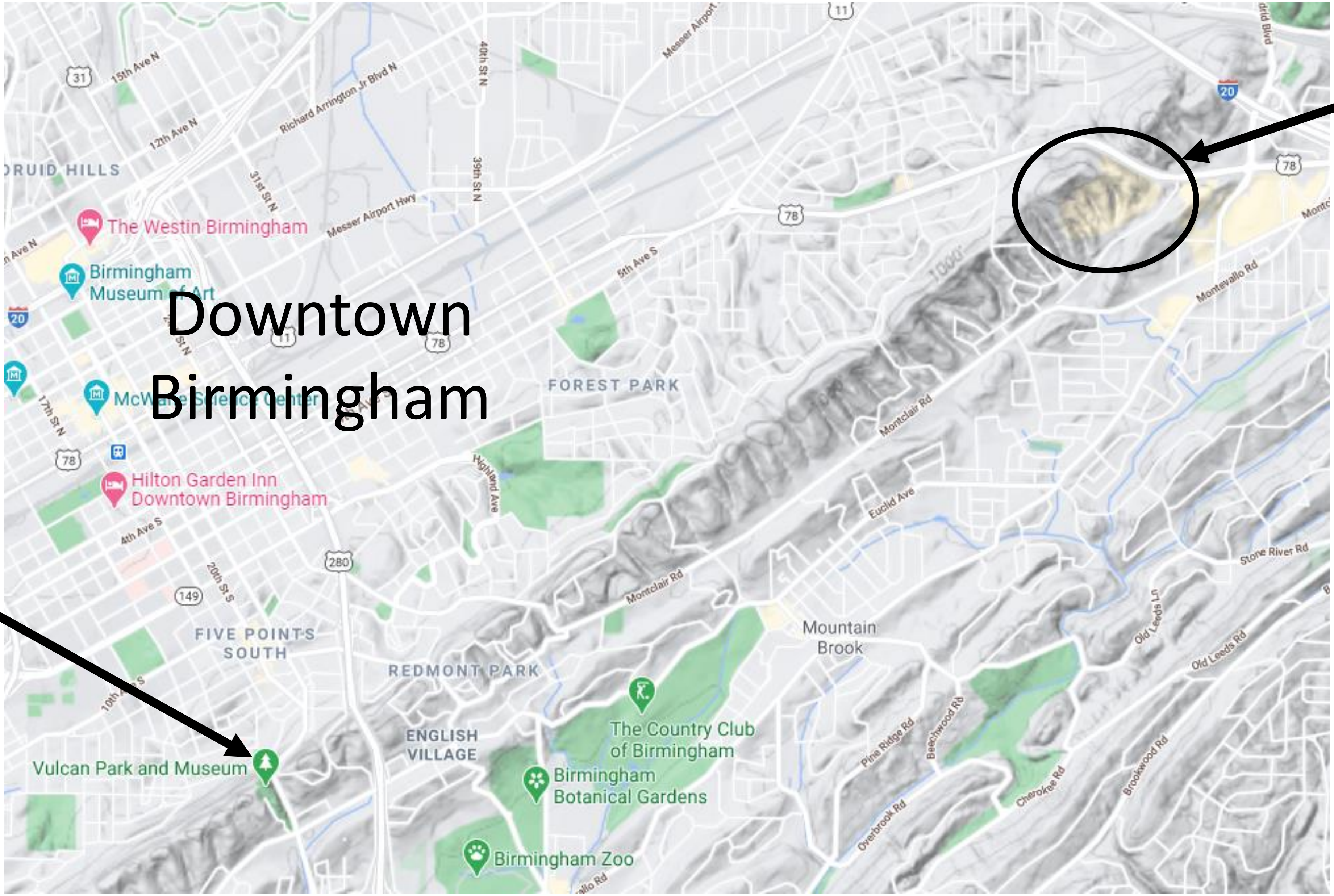
- 1988 Rock slope failure in Birmingham, AL

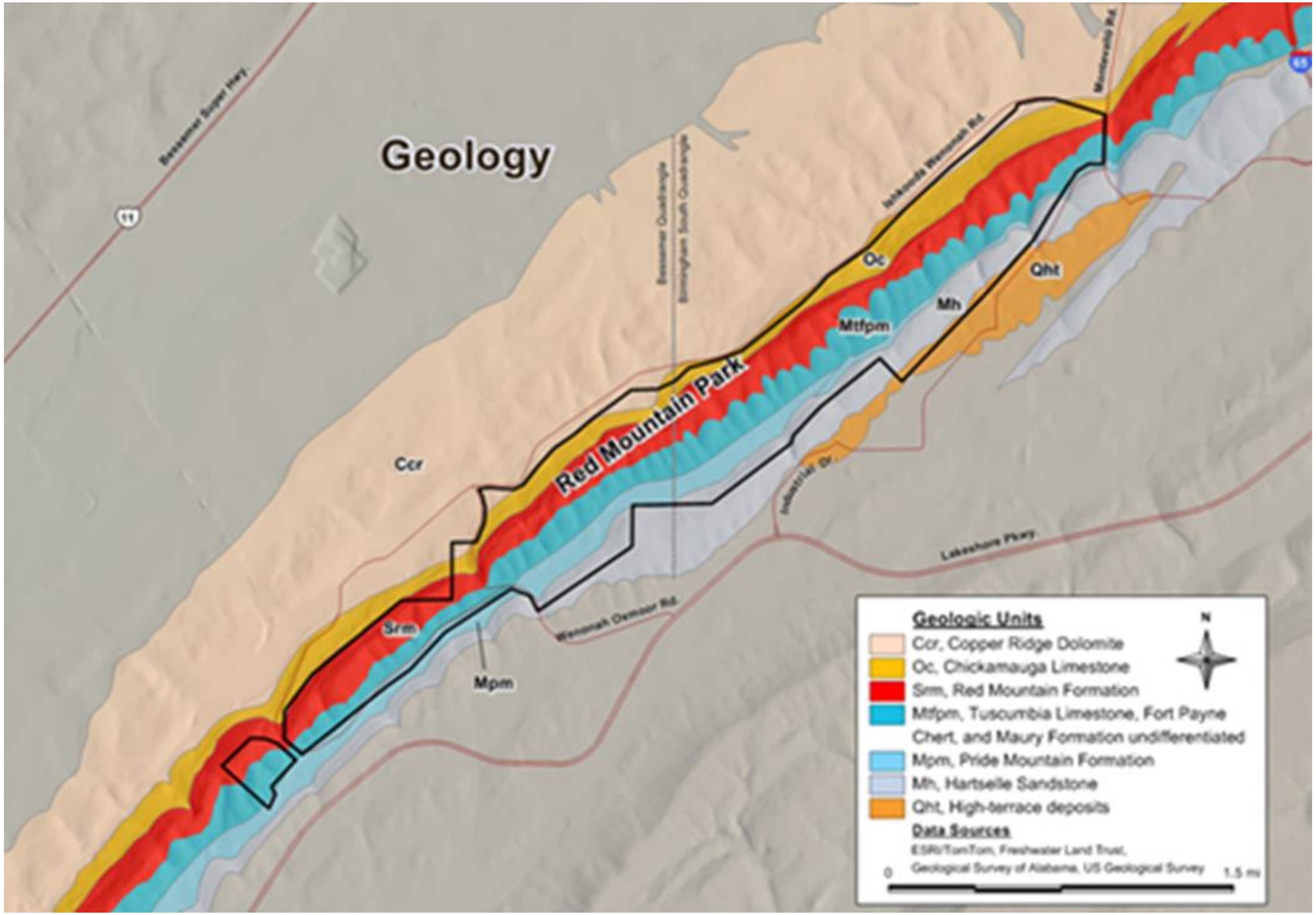


Red Mountain, Birmingham, AL

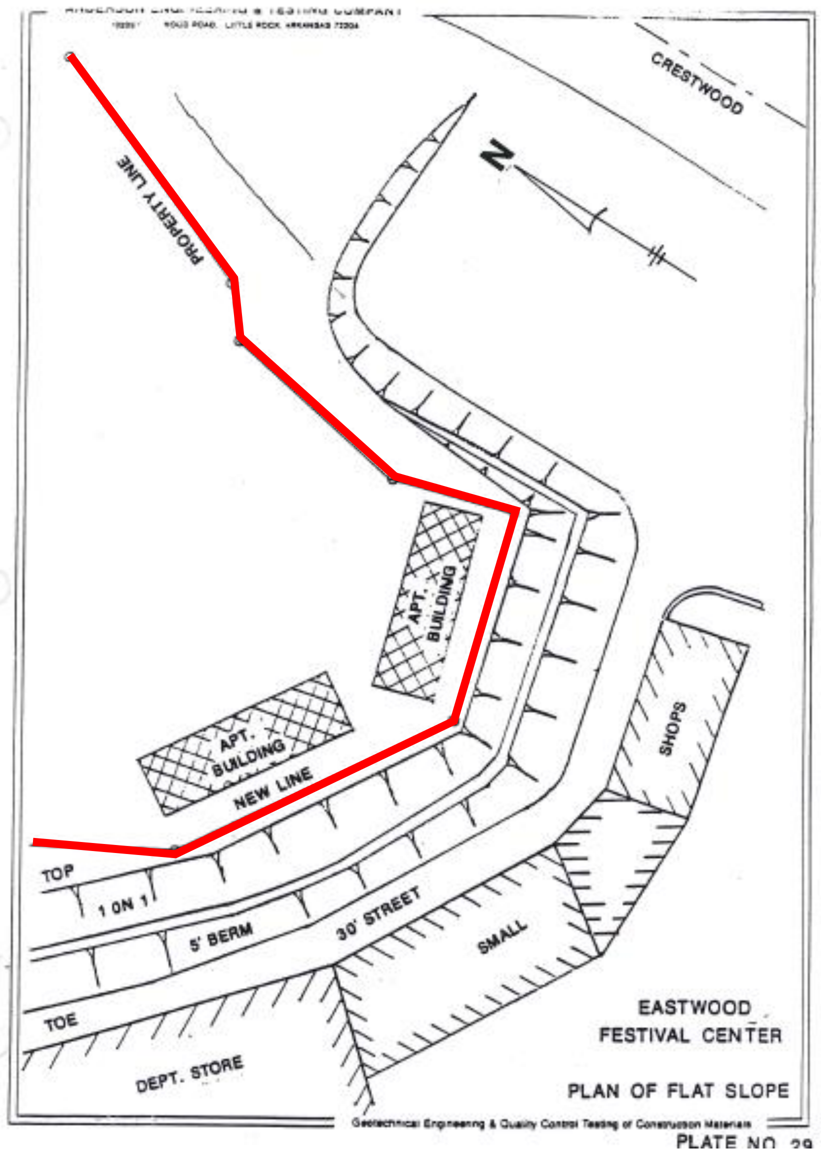
Rock Slope Failure

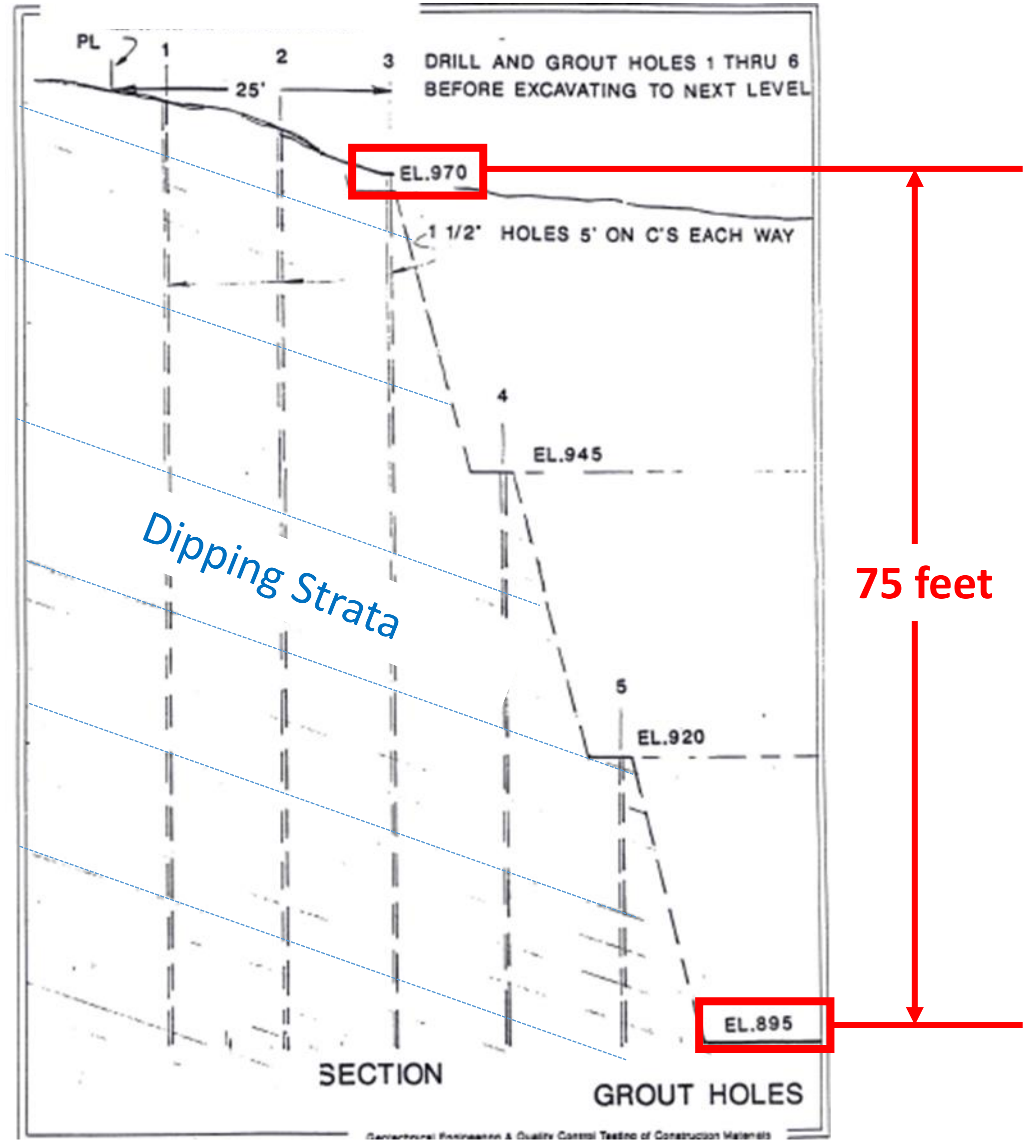
Vulcan Park



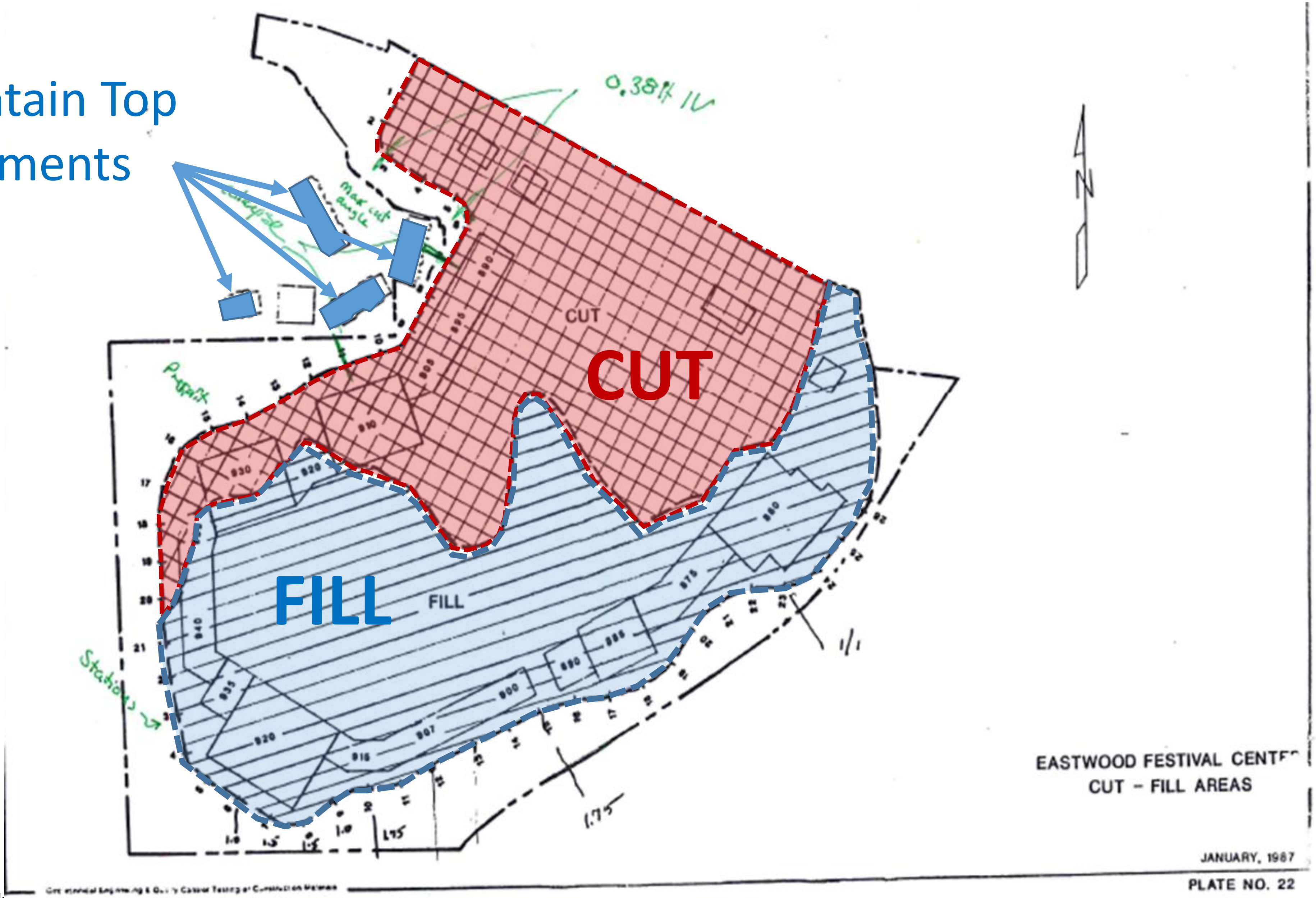

Geology
Bessemer Super Hwy.
11
Bessemer Quadrangle
Birmingham South Quadrangle
Ishkooda Wenonah Rd.
Montevallo Rd.
65
Red Mountain Park
Ccr
Oc
Srm
Mtfpm
Mh
Qht
Mpm
Industrial Dr.
Lakeshore Pkwy.
Wenonah Oxmoor Rd.
Geologic Units
Ccr, Copper Ridge Dolomite
Oc, Chickamauga Limestone
Srm, Red Mountain Formation
Mtfpm, Tuscumbia Limestone, Fort Payne Chert, and Maury Formation undifferentiated
Mpm, Pride Mountain Formation
Mh, Hartselle Sandstone
Qht, High-terrace deposits
Data Sources
ESRI/TomTom, Freshwater Land Trust, Geological Survey of Alabama, US Geological Survey
N
0
1.5 mi

Mountain Top Apartments
Slope Direction
Crestwood Festival Center (Formerly called Eastwood)
Crestwood Festival Center

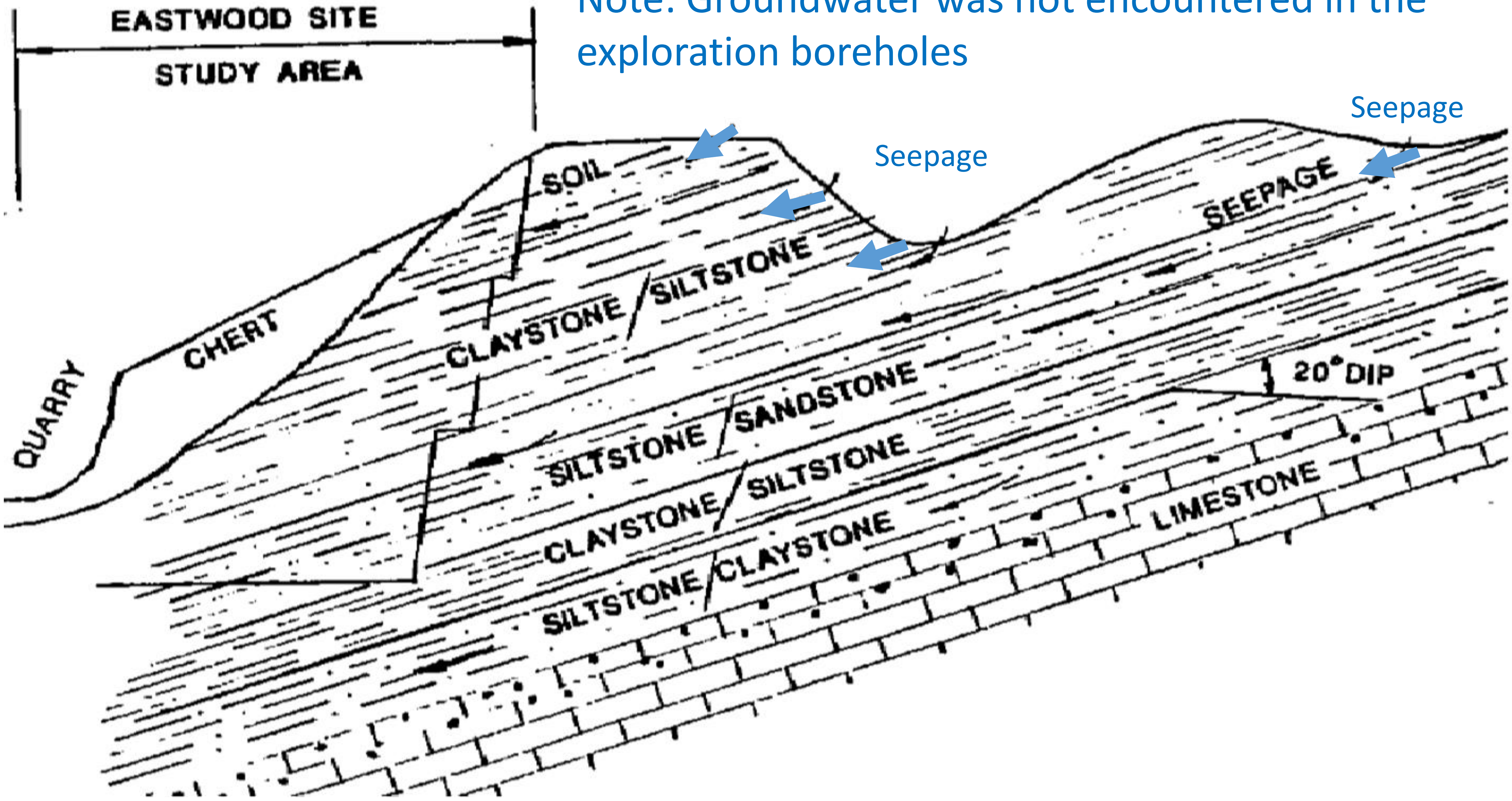

PROPERTY LINE
N
CRESTWOOD
APT. BUILDING
APT. BUILDING
NEW LINE
TOP
1 ON 1
5' BERM
30' STREET
SMALL
SHOPS
TOE
DEPT. STORE
EASTWOOD
FESTIVAL CENTER
PLAN OF FLAT SLOPE
PLATE NO. 29

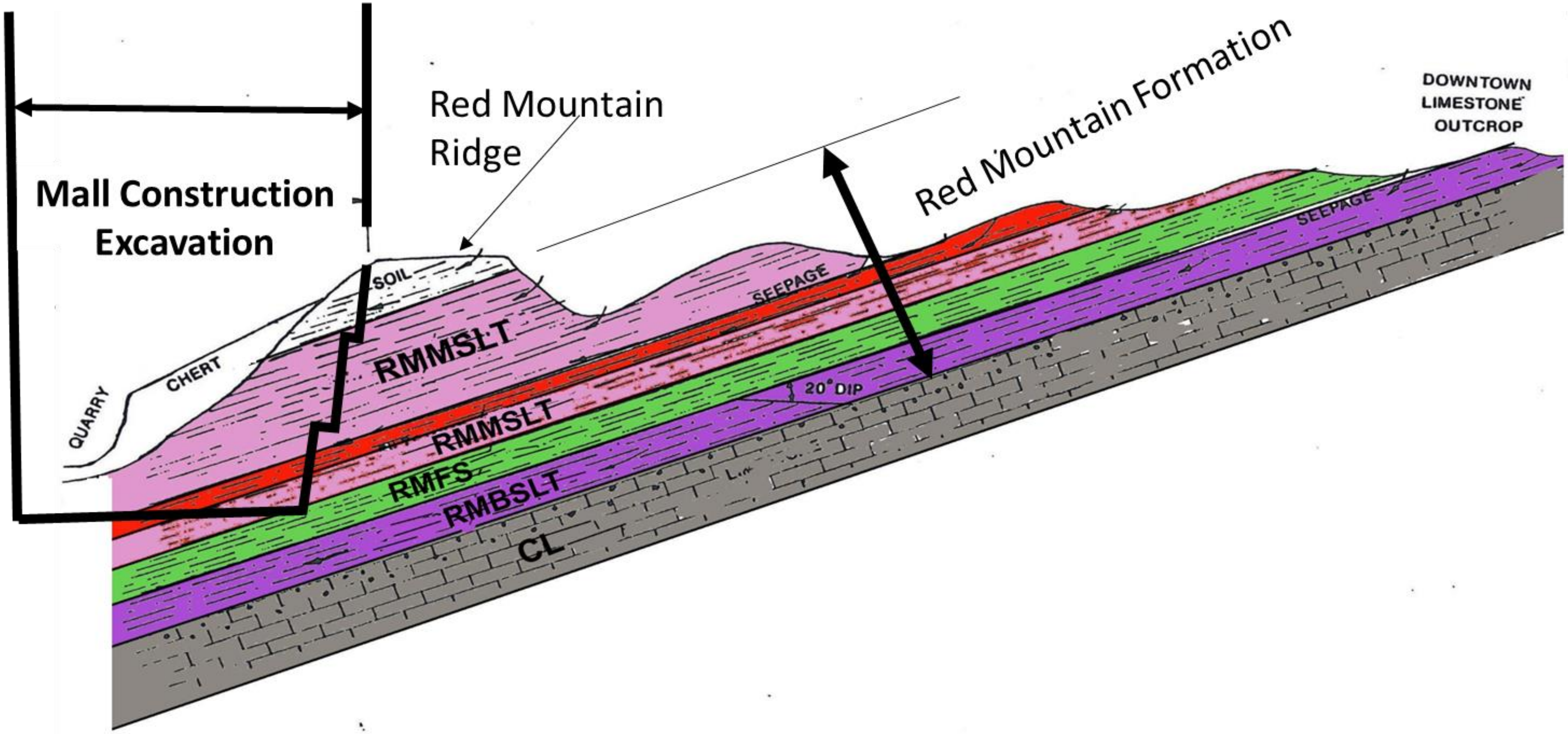

PL
1
2
3
25'
DRILL AND GROUT HOLES 1 THRU 6
BEFORE EXCAVATING TO NEXT LEVEL
EL.970
1 1/2" HOLES 5' ON C'S EACH WAY
4
EL.945
5
EL.920
EL.895
Dipping Strata
75 feet
SECTION
GROUT HOLES

Mountain Top Apartments



Note: Groundwater was not encountered in the exploration boreholes

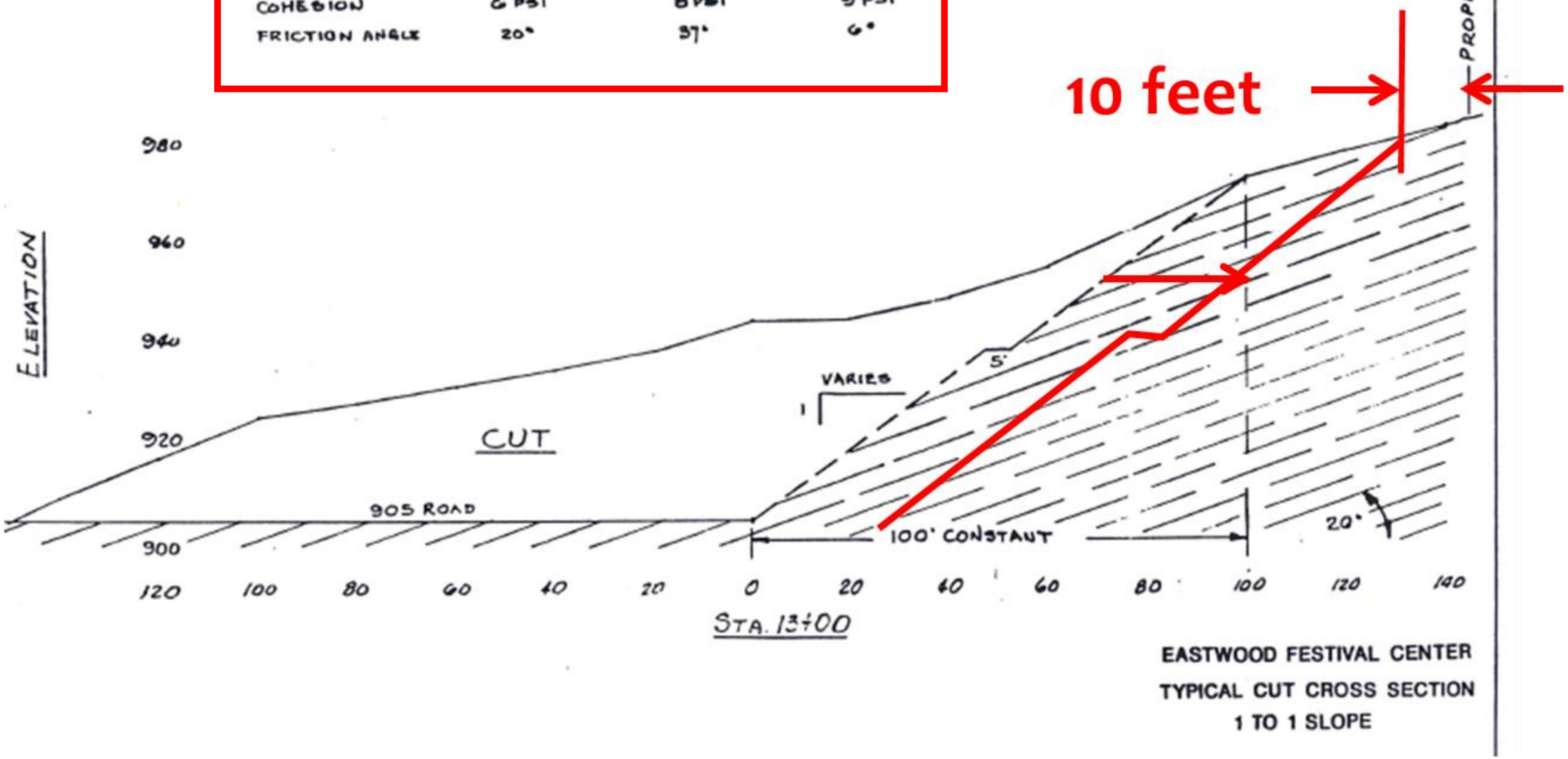



Mall Construction Excavation
Red Mountain Ridge
Red Mountain Formation
DOWNTOWN LIMESTONE OUTCROP
SOIL
CHERT
QUARRY
RMMSLT
SEEPAGE
RMMSLT
RMFS
RMBSLT
20° DIP
SEEPAGE
CL

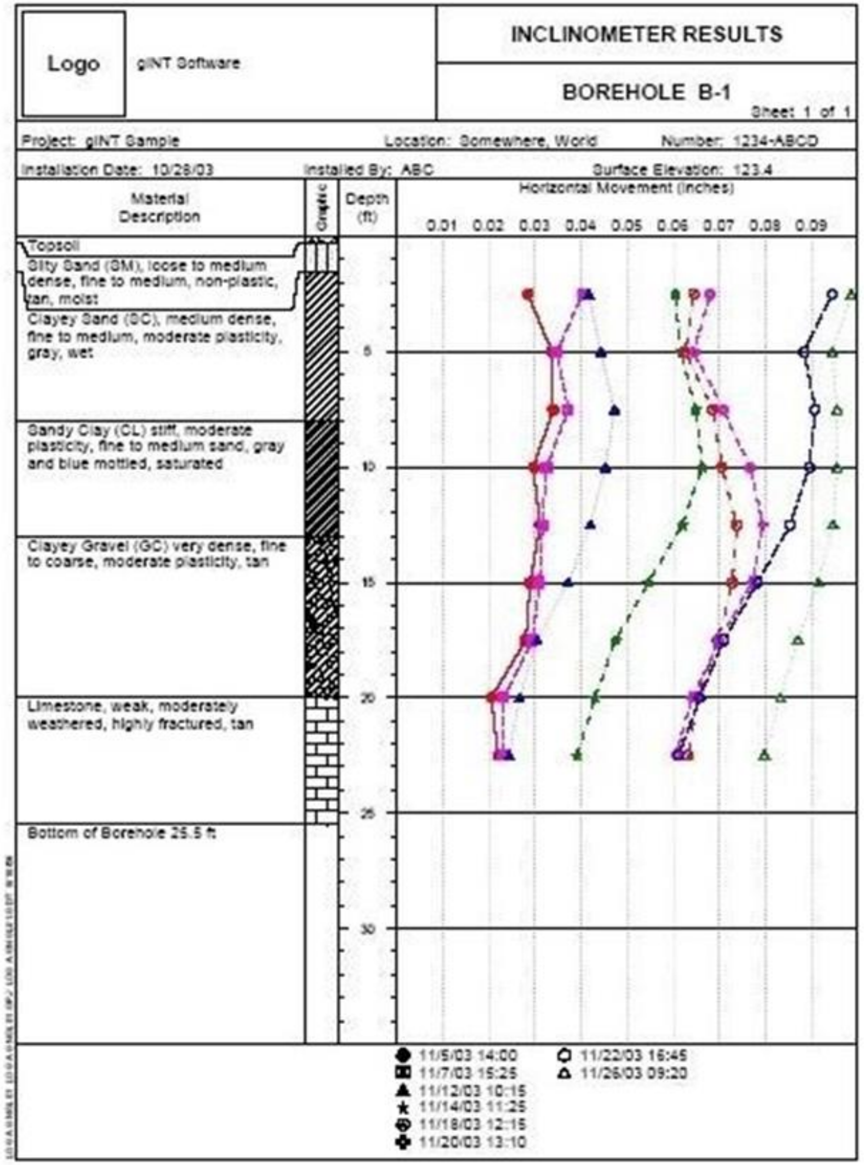
Rock Required Blasting

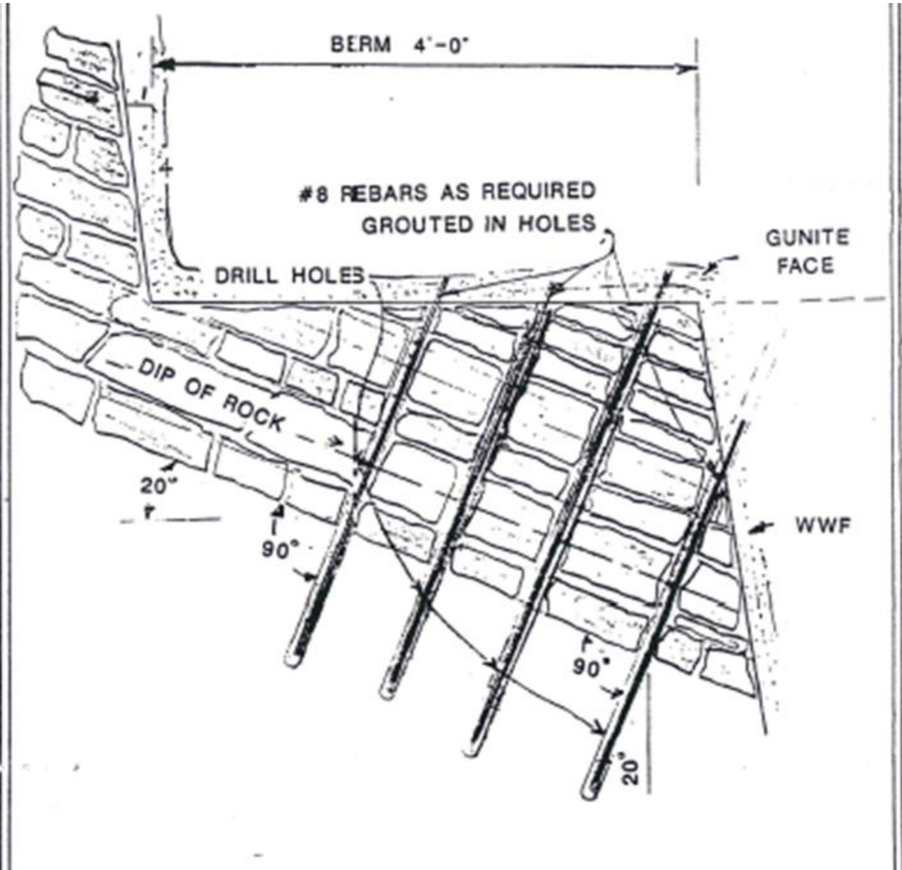
APPROXIMATE INSITU PROPERTIES

PROPERTY	CLAY/SILTSTONE	SILT/SANDSTONE	LIMESTONE
RQD	40%	85%	95%
UNIT WEIGHT	133 PCF	144 PCF	150 PCF
STRENGTH, QU	2300 PSI	3500 PSI	8000 PSI
COHESION	6 PSI	8 PSI	9 PSI
FRICTION ANGLE	20°	37°	6°



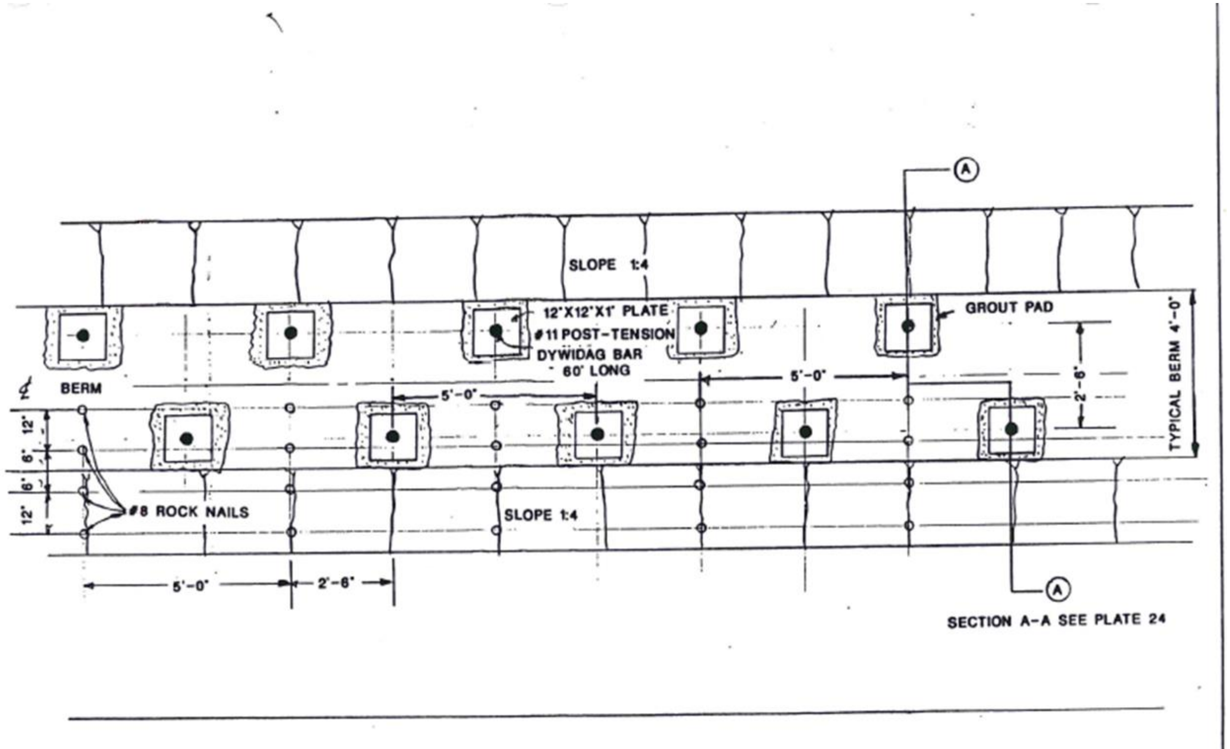
Inclinometers Installed





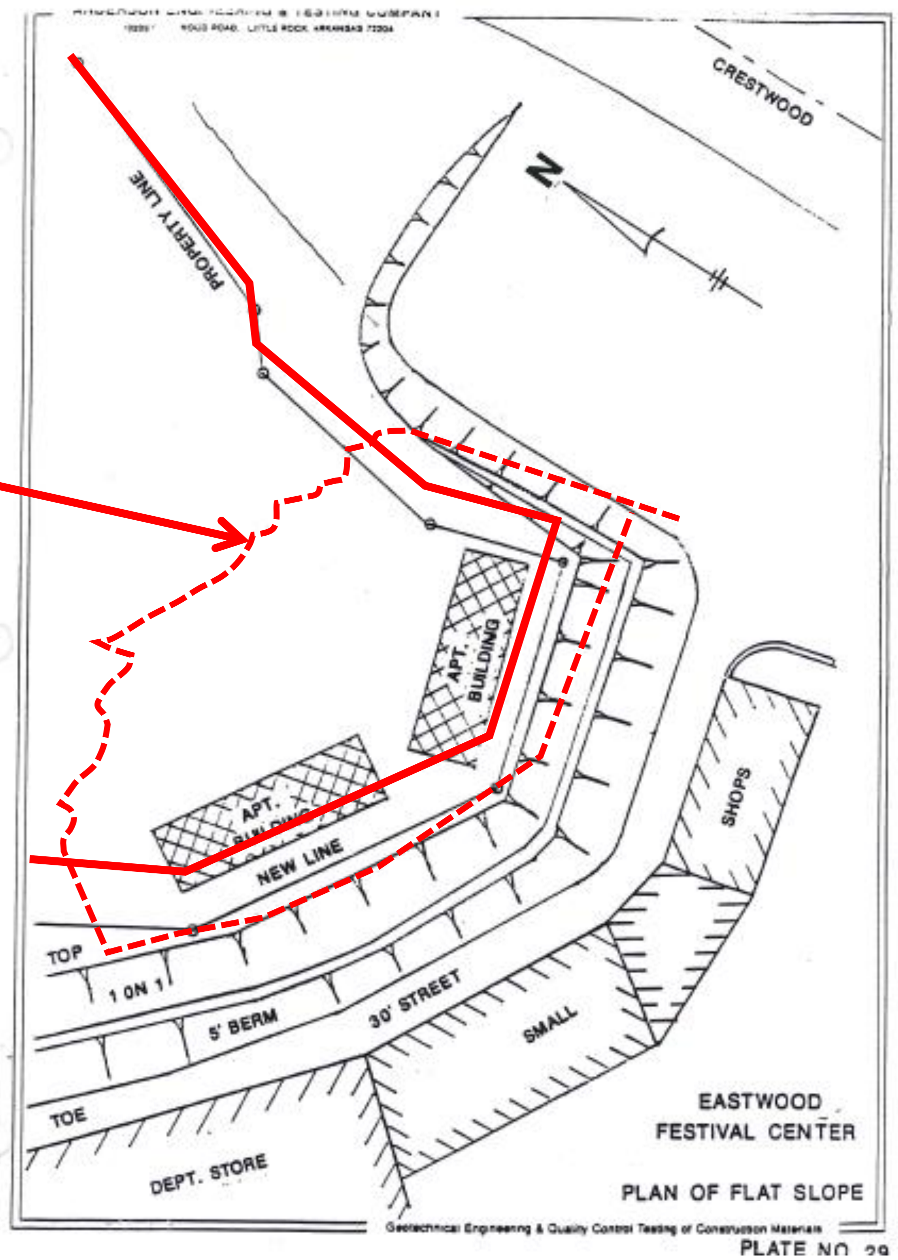
ROCK NAILS

NAILS ARE REQUIRED TO PREVENT LOSS OF CORNERS

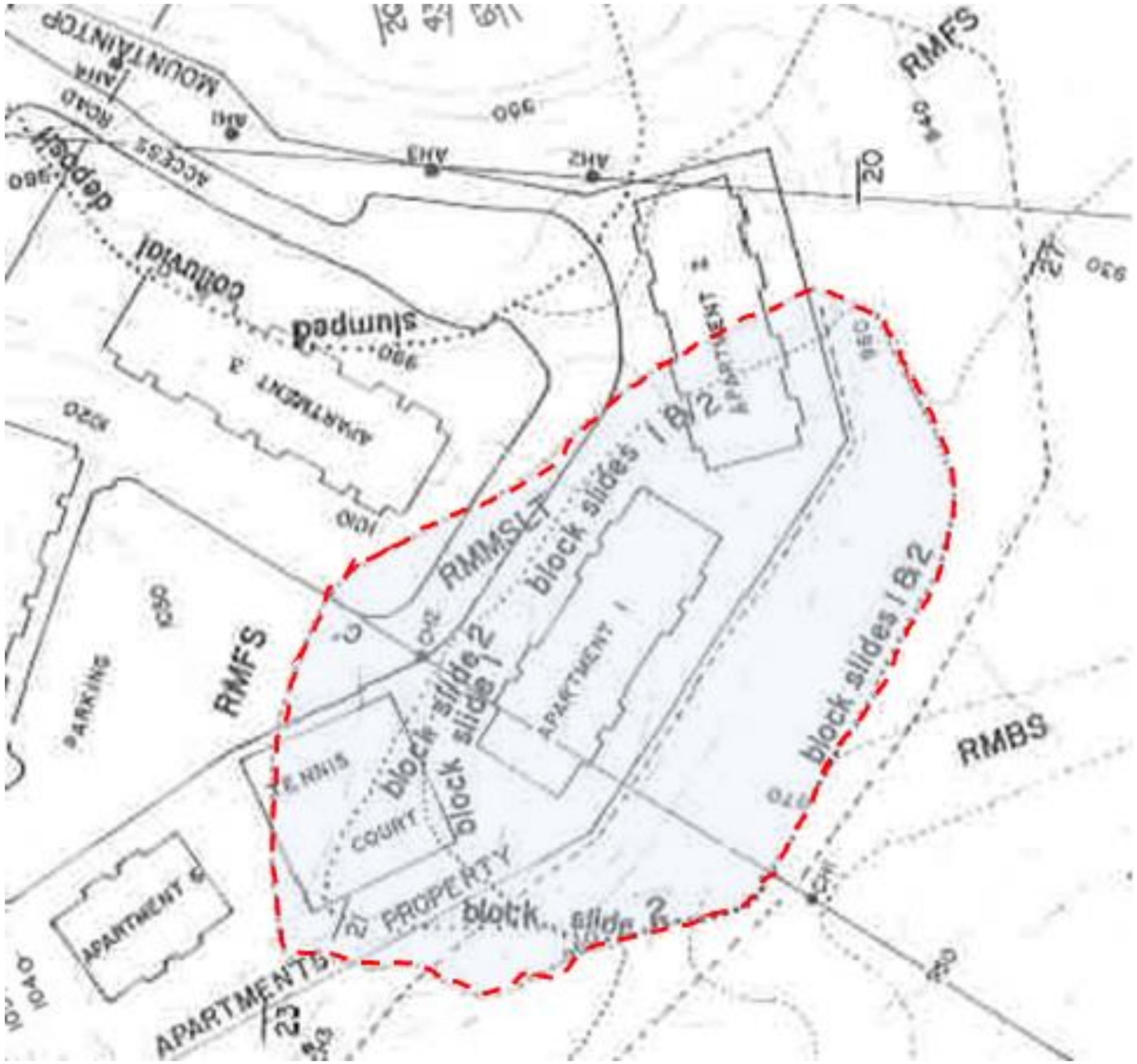


PLAN ROCK ANCHORS

Landslide #1



CRESTWOOD
N
PROPERTY LINE
APT. BUILDING
APT. BUILDING
NEW LINE
TOP
1 ON 1
5' BERM
30' STREET
SHOPS
SMALL
TOE
DEPT. STORE
EASTWOOD FESTIVAL CENTER
PLAN OF FLAT SLOPE
Geotechnical Engineering & Quality Control Testing of Construction Materials
PLATE NO. 29



RMFS
RMMSLT
block slides I & 2
block slide 2
RMBS
APARTMENT 1
APARTMENT 2
APARTMENT 3
APARTMENT 6
TENNIS COURT
PARKING
PROPERTY
APARTMENTS
ACCESS ROAD
colluvial
slumped
deposit

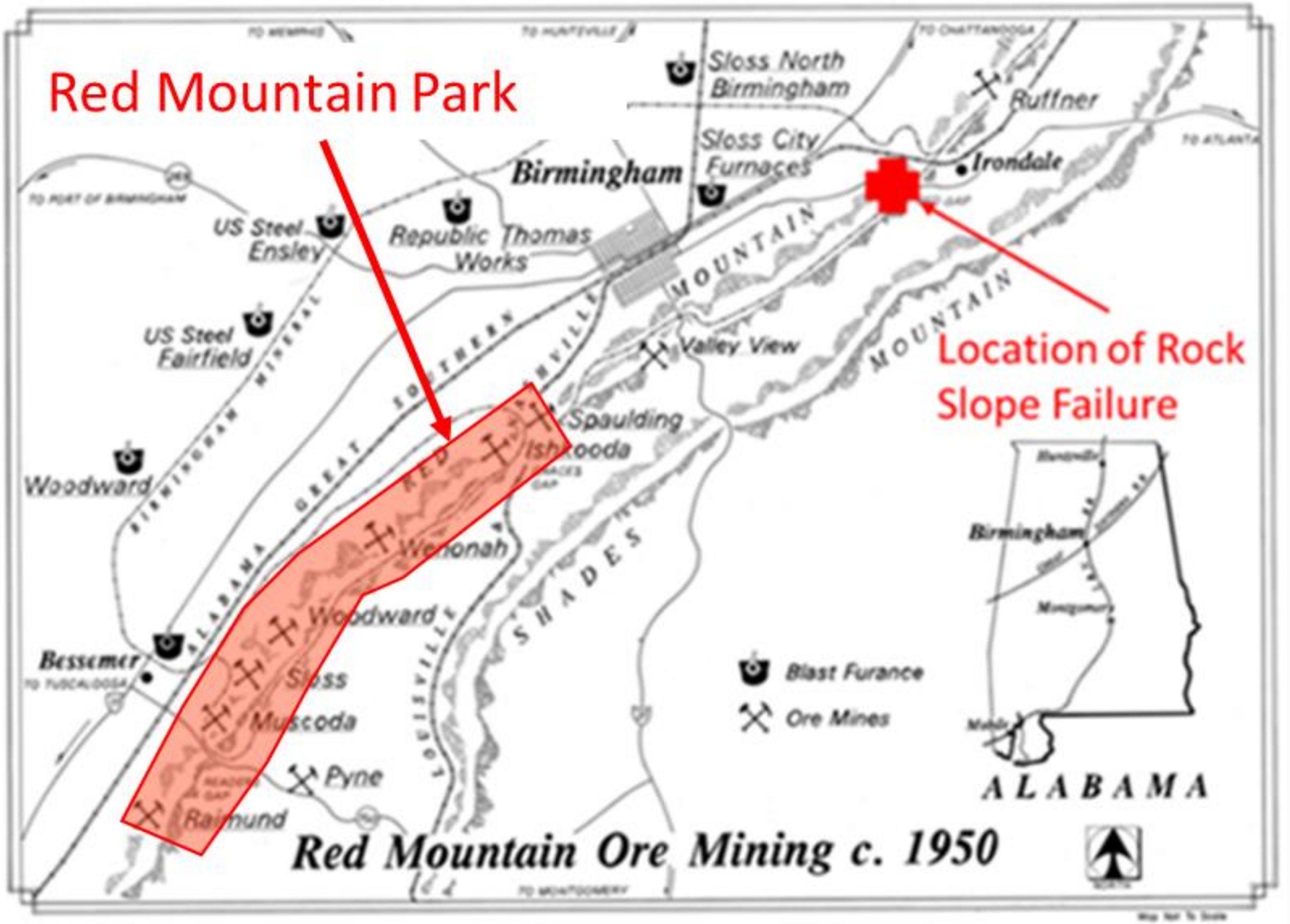

a


b

Potential Causes for the failure:

- Dipping bedrock at 17° towards the cut
- Interbedded weathered clay layers
- Blasting
- Excavation at greater than a 45°
- Potential underground mining activity

Historic Underground Iron Mining on Red Mountain



1924

Red Iron Ore Mining Methods in the Birmingham District*

By W. R. Crane, † Birmingham, Ala.

(Birmingham Meeting, October, 1924)

Mining of the red iron ores of the Birmingham district has been carried on energetically during the past 50 years, and their development has created a large iron and steel manufacturing center, the only important one in the South. The district produces approximately 10 per cent. of all the iron ore of the United States (80 per cent. of the Alabama ore mined is red ore); also, 40 of the 400 blast furnaces of the country are in the district tributary to Birmingham. The rapid growth of the district has been made possible through investigations that resulted in radical changes in furnace practice, and a still greater impetus will come from the study of the low-grade, high-silica ores, as a result oi which they will be made amenable to treatment by concentration.

Mining practice in this district has been comparatively simple because of the occurrence of the ore; but with the rapid extension of the workings and the disturbed condition of the ore bed at some distance from the outcrop, more difficult conditions are encountered and the tendency is toward worse rather than better conditions.

Should the high-silica ores of the lower bench of the Big Seam become available through beneficiation, mining practice will have to be modified to meet the new conditions, which will be rendered more difficult by the increased weight of cover that will exist at considerable distances from the outcrop. Support of workings will require greater attention and the efficient and economical operation of the mines will depend largely on the successful solution of the problems of working and handling the ore.

History and Early Development

The first explorers of the coal and mineral lands of Alabama were blacksmiths and mechanics mustered out of the army after the war of 1812. These men recognized the red rock of the Birmingham district as iron ore, and utilized it in making cooking utensils and farm implements. The first blast furnace was built and operated at Russellville in 1818, where also was a foundry and rolling mill. Soft ore was used at that time, but, in 1864, the first red hematite ore of Red Mountain was smelted, near Irondale. Birmingham was founded in 1871, and the successful use of coke in making pig iron in 1876 was the beginning of

* Published with approval of the Director of the Bureau of Mines.
† Superintendent, Southern Experiment Station, Birmingham-Tuscaloosa, Ala.

1924

Roof Support in the Red Ore Mines of the Birmingham District*

By W. R. Crane,† Birmingham, Ala.

(Birmingham Meeting, October, 1924)

The support of roof in mines is dependent largely on the character of the top rock and its occurrence. The formations overlying the orebed in the Birmingham district are sandstone and slate. The sandstone occurs in beds of sufficient thickness to constitute important elements in the system of support; the slate is relatively strong but is thinly stratified. The alternating beds of sandstone and slate furnish an excellent combination in that the former are strong and the latter are impervious; however, their continuity is broken by jointing or slip planes.

The thickness, and consequent weight of the overlying formations, is important in so far as the size and arrangement of pillars are concerned but may be relatively unimportant, temporarily at least, as a factor in the support of the roof. The position of the respective beds of sandstone and slate, their relative thickness, and their inherent weakness because of the occurrence of faults and folds, may exert a predominating influence on their support in place, which is affected only in part by the weight of the cover.

The only mine in this district that operates under a cover of approximately 2000 ft. (1900 ft.) is the Shannon so-called "twin slope," situated 14,000 ft. southeast of the portal of the No. 7 mine of the Tennessee Coal, Iron, & Railroad Co. The conditions in this mine indicate what may be expected in other mines when the same depth is attained; in fact, the effect of pressure is shown in several mines that have not reached two-thirds of that depth. It is evident, then, that conditions affecting the support of roof will not improve but rather will become more difficult with the extension of the mines into the valley under a constantly increasing weight of cover. Further, the difficulties will probably be augmented by the occurrence of faults and other disturbed ground, the presence and position of which at present are largely unknown.

* Published with approval of the Director of the Bureau of Mines.
† Superintendent, Southern Mining Experiment Station.

1958

Mining Hydrology Problems in the Birmingham Red Iron Ore District

by Thomas A. Simpson

The Birmingham red iron ore district in Jefferson County, north central Alabama, Fig. 1, is bounded on the northwest by the Warrior and Plateau coal fields and on the southeast by the Cahaba and Coosa coal fields. The area of study includes the ridge and valley between Red Mountain and Shades Mountain, Fig. 2, approximately 70 square miles extending from Homewood in the northeast to Greenwood in the southwest.

The district is one of the most important producers of hematite in the United States, with an annual production of about 7 million tons.[1] The amount of hematite mined between 1870 and 1950 ranged from 6 to 16 pct of the nation's total annual production,[2] and the iron and steel products from blast furnaces in the Birmingham district supply the entire southeastern section of the country.

Most of the mining in the Birmingham district is from slope mines along the outcrop of the ore on Red Mountain, such as Red Ore, Muscoda, Spaulding, and Sloss. The Pyne and the Shannon are the shaft mines of the area. The Shannon mine of Republic Steel Corp., in the central part of the area under study, is in Shades Valley at the foot of Shades Mountain. The Pyne mine of Woodward Iron Co. is 2 miles east of Readers Gap in Shades Valley.

The hydrology problem has become more prevalent as mining in the area has progressed downdip. An extensive exploratory diamond drilling program is necessary to determine areas where water pressures and abnormal flows are excessive. This operation, added to higher pumpage rates, has greatly increased the costs of ore extraction.

In November 1952 the U. S. Geological Survey began a detailed study of occurrence and movement of ground water in the iron mining areas of the Birmingham district. The study is a part of a small-scale but nationwide program to develop data that will be of help to the mining industry in solving mine water problems.

The area of the study, in the Ridge and Valley province of the Appalachian system, Fig. 1, is a series of alternate ridges and valleys trending northeast. The principal ridges are Red Mountain, altitude about 1000 ft, and Shades Mountain, about 1100 ft. Shades Valley is between these two ridges, and its lowest point is about 480 ft. Shades Moun-

T. A. SIMPSON is a Geologist, Ground Water Branch, U.S. Geological Survey, University, Ala.

Discussion on this paper, TP 39561, may be sent (2 copies) to AIME before March 31, 1955. Manuscript, June 14, 1954. New York Meeting, February 1954.

Publication of this paper is authorized by the Director, U.S. Geological Survey.

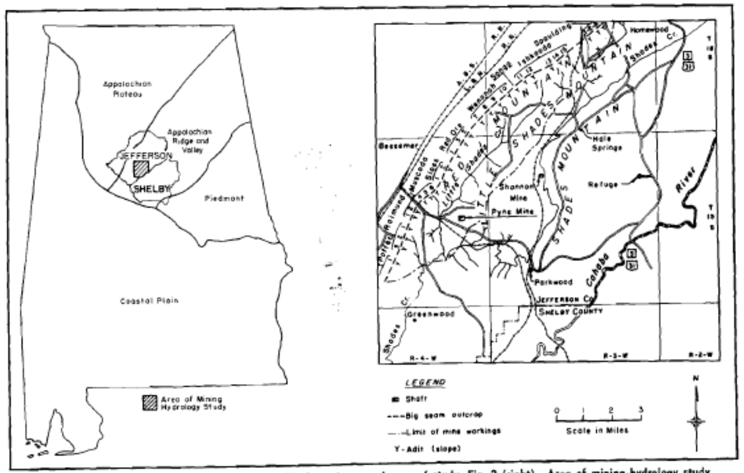


Fig. 1 (left)—Map of Alabama shows physiographical provinces and area of study. Fig. 2 (right)—Area of mining-hydrology study.

1925

Subsidence and Its Relation to Drainage in the Red Iron Mines of the Birmingham District, Alabama*

By W. R. Crane,† Birmingham, Ala.

(New York Meetings, February, 1925, and February, 1927)

The effect of mining in the red-ore mines of the Birmingham district has been observed for some time, but, except in a few localities, little difficulty has been experienced from disturbance of cover. Cave-ins near the outcrop and fracturing of the surface at greater distances are the most pronounced manifestations of disturbance, while limited areas at points distant from the outcrop and under greater depth of cover have actually subsided.

Were it not that the orebed is overlain by one or more water-bearing formations, the fracturing or settlement of the surface would not be serious; but because of these formations, fracturing of the top rock may in itself be of sufficient importance to warrant the adoption of protective measures. Further, the collapse of pillars over a wide area may develop squeezes that, if not controlled, may jeopardize the integrity of the mines and necessitate a change in development, from slopes to vertical shafts, at a much earlier period than is now contemplated.

Drainage in these mines has been discussed in papers on mining practice,[1] but the fact that water, varying from 0.16 to 3.46 and averaging 1.43 times the amount of ore mined, has to be pumped from the mines daily, indicates the importance of the problem and the advisability of making a thorough investigation of the source of mine water, its mode of entry into the mines, its effect upon mining conditions, and consideration of means of improving the conditions.

* Published by permission of the Director of the Bureau of Mines. This paper is a consolidation of two papers by the same author, "Mine Subsidence in the Red Iron Ore Mines of the Birmingham District, Alabama," presented at the New York Meeting, February, 1925, and issued as A. I. M. E. *Pamphlet* No. 1475-A, and "Drainage in the Red Iron Ore Mines of the Birmingham District, Alabama," presented at the New York Meeting, February, 1927, and issued as A. I. M. E. *Pamphlet* No. 1639-I.

† Mining engineer, U. S. Bureau of Mines.

[1] W. R. Crane: Iron-Ore (Hematite) Mining Practice in the Birmingham District, Alabama. *Bull.* 239, Bur. Mines (1926).

W. R. Crane: Red Iron Ore Mining Methods in the Birmingham District. *Trans.* (1925) **72**, 157.

W. R. Crane: Roof Support in the Red Iron Ore Mines of the Birmingham District. *Trans.* (1925) **72**, 187.

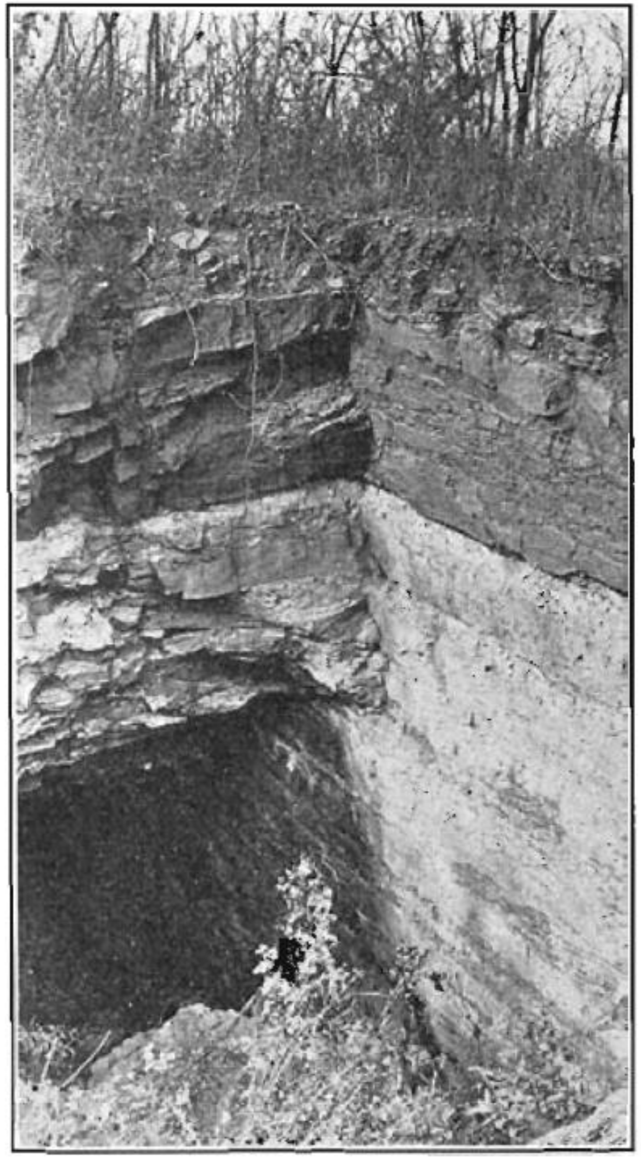


Fig. 2.—Break of top rock along slip plane.

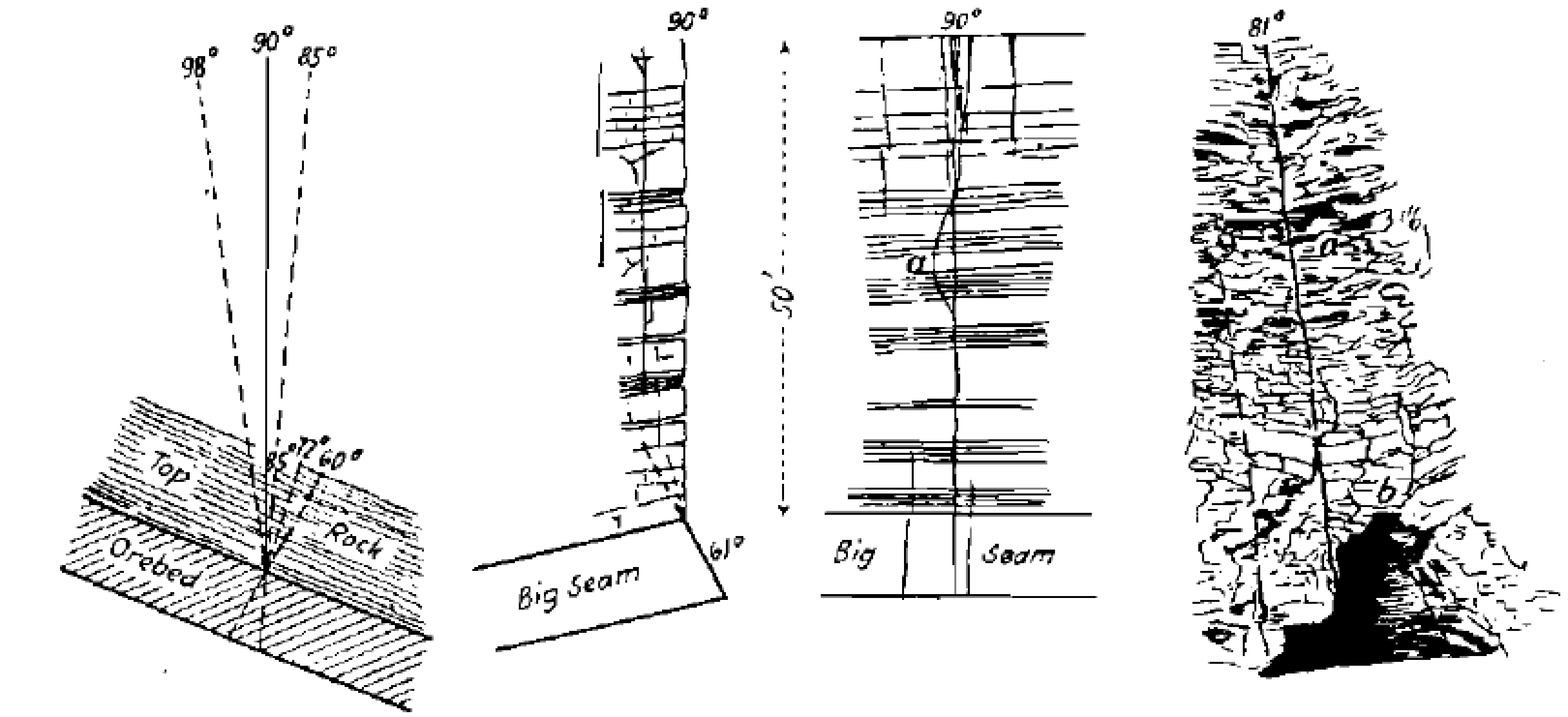
Fig. 3.—Caving wall of open cut following slip planes.

Subsidence and Its Relationship to Drainage of the Red Iron Mines in Birmingham District, Alabama

W.R. Crane

1925, 1927

Drainage in these mines has been discussed in papers on mining practice,[1] but the fact that water, varying from 0.16 to 3.46 and averaging 1.43 times the amount of ore mined, has to be pumped from the mines daily, indicates the importance of the problem and the advisability of making a thorough investigation of the source of mine water, its mode of entry into the mines, its effect upon mining conditions, and consideration of means of improving the conditions.



a. Average angle in ore and a few feet in top rock 72°, average angle of slips reaching surface 90°, all angles parallel with strike

b. Change in angle of slip plane in passing from orebed to top rock

c. Continuation of vertical slip plane from orebed to top rock parallel with dip

d. Slip plane parallel with dip extending from top of cave-in into top rock, showing breaking and slipping along plane

FIG. 1.—DIRECTION TAKEN BY SLIP PLANES IN TOP ROCK.

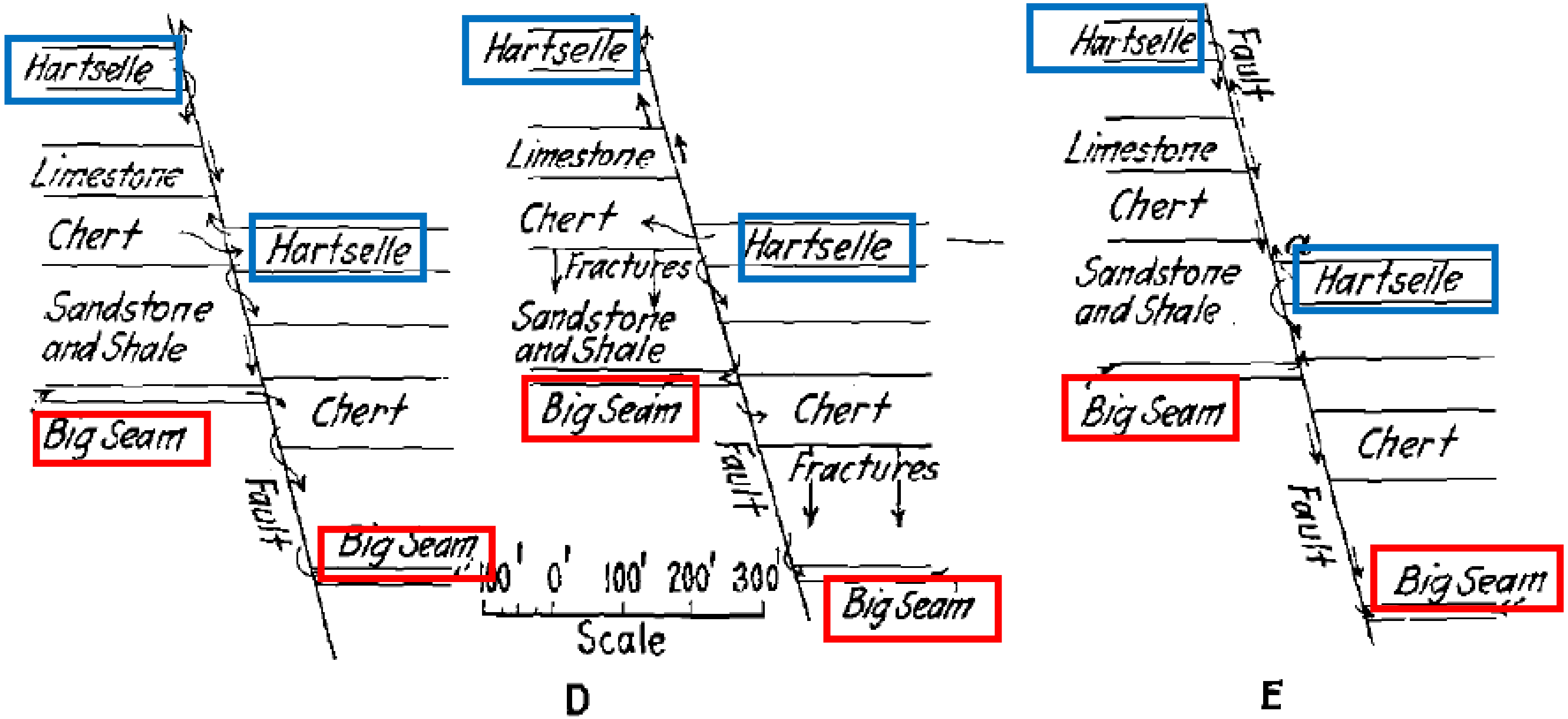


FIG. 20.—SKETCHES SHOWING TRANSFER OF WATER BETWEEN PERVIOUS AND IMPERVIOUS FORMATIONS AS AFFECTED BY FAULTING AND FRACTURING. A, B and C show effect of faulting on movement of water down a slope. D and E show effect of faulting on vertical movement of water in overlying formations.

Mined out layer

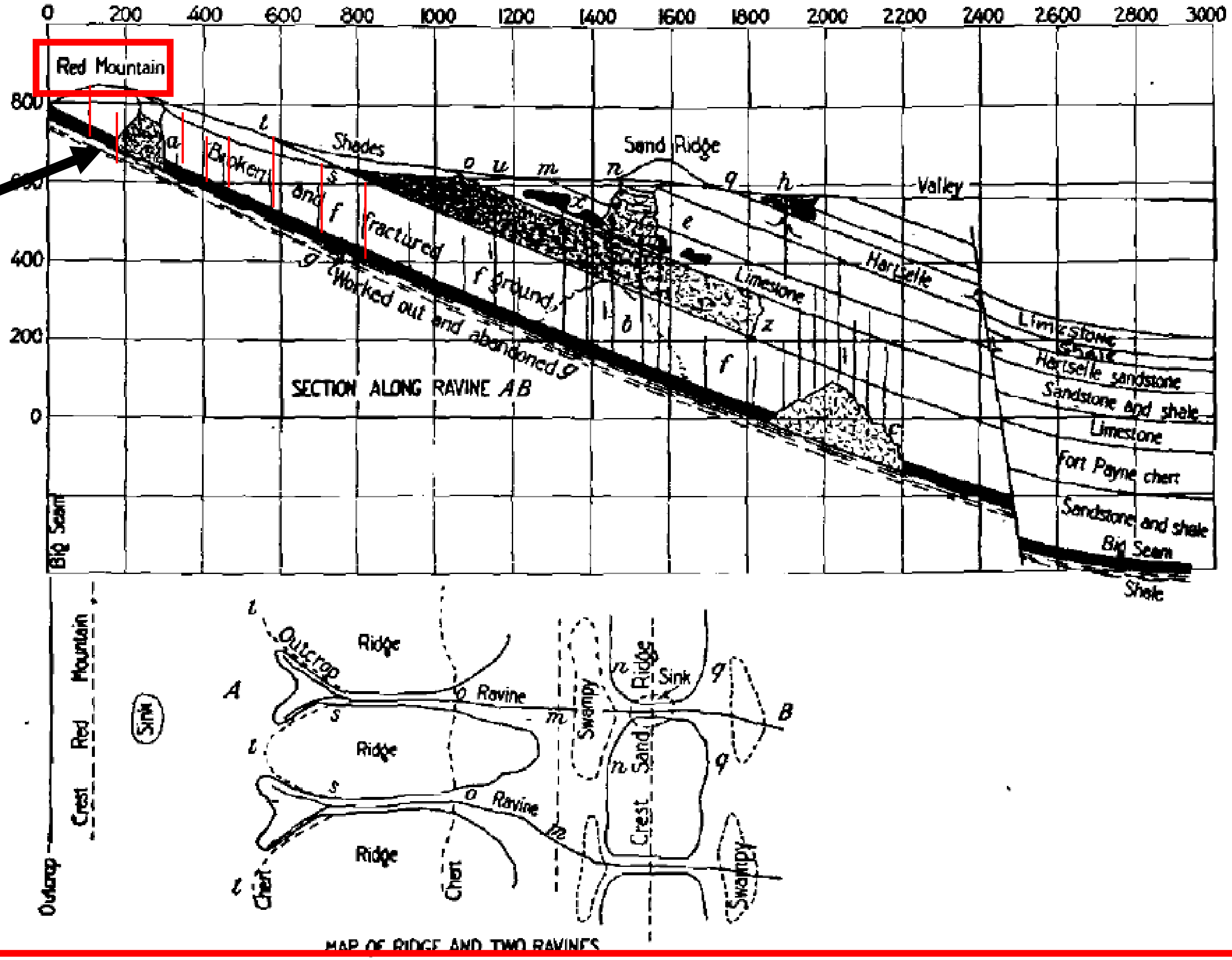
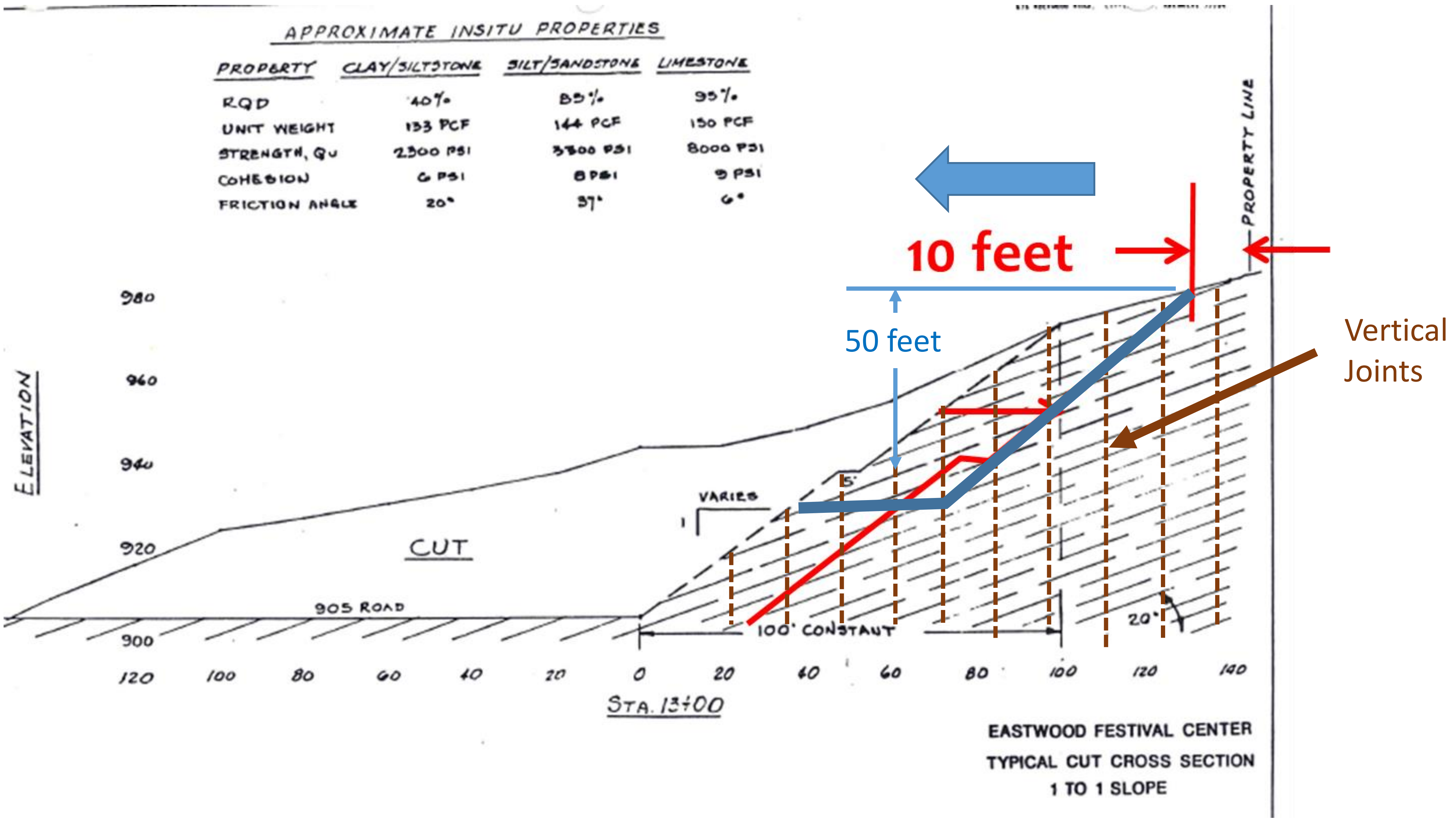


FIG. 21.—IDEAL SECTION SHOWING SOURCE AND MOVEMENT OF MINE WATER.


APPROXIMATE INSITU PROPERTIES
PROPERTY CLAY/SILTSTONE SILT/SANDSTONE LIMESTONE
RQD 40% 85% 95%
UNIT WEIGHT 133 PCF 144 PCF 150 PCF
STRENGTH, QU 2300 PSI 3300 PSI 8000 PSI
COHESION 6 PSI 8 PSI 9 PSI
FRICTION ANGLE 20° 37° 6°
10 feet
50 feet
PROPERTY LINE
Vertical Joints
ELEVATION
980
960
940
920
900
CUT
905 ROAD
VARIES
5'
100' CONSTANT
20°
120 100 80 60 40 20 0 20 40 60 80 100 120 140
STA. 13+00
EASTWOOD FESTIVAL CENTER
TYPICAL CUT CROSS SECTION
1 TO 1 SLOPE

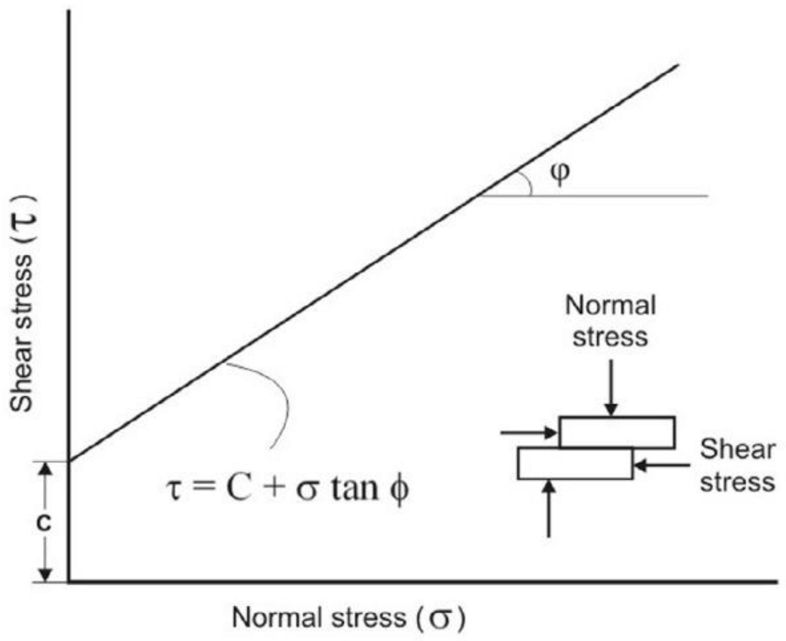


Fig. 3. Relation between normal and shear stress along potential failure plane.

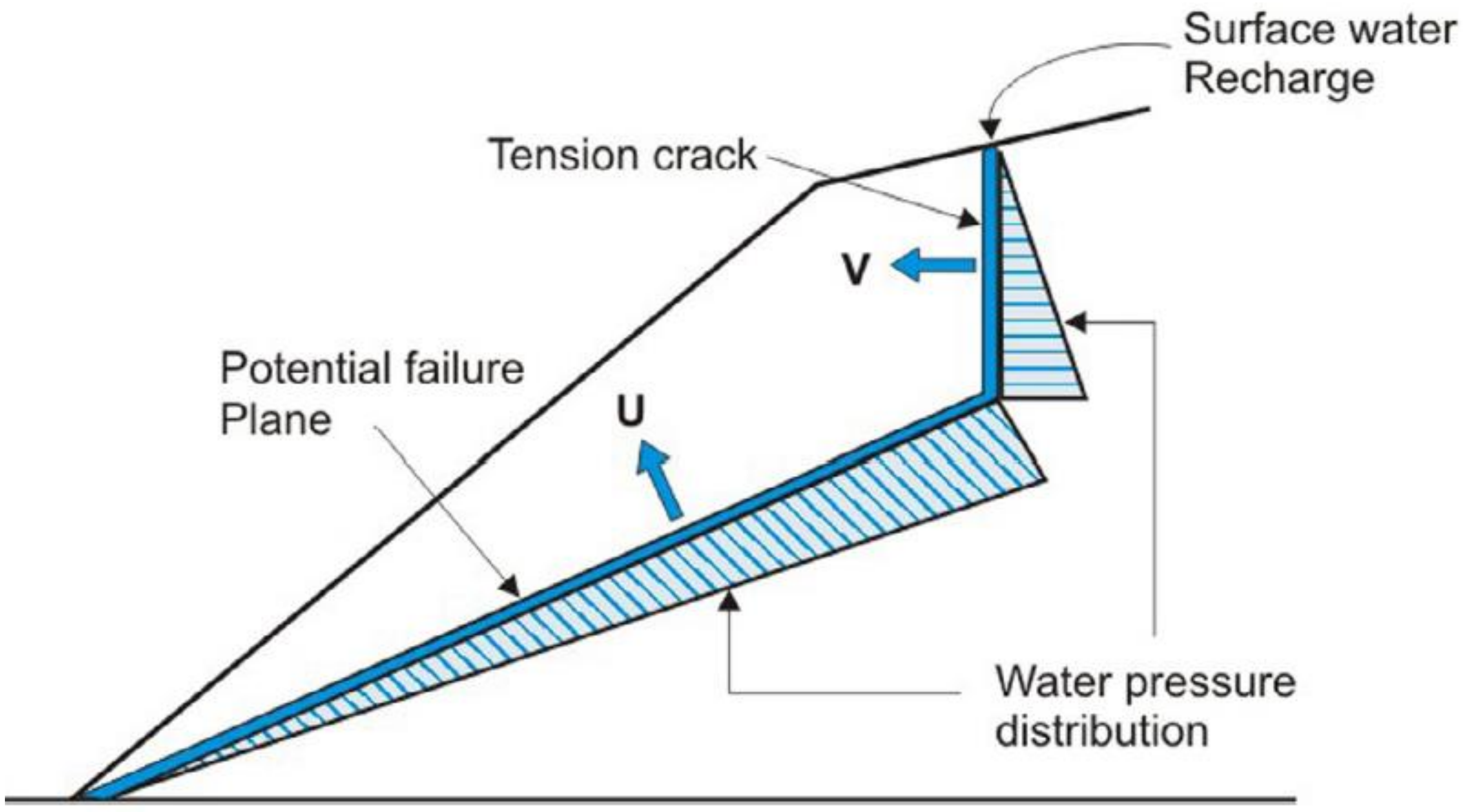
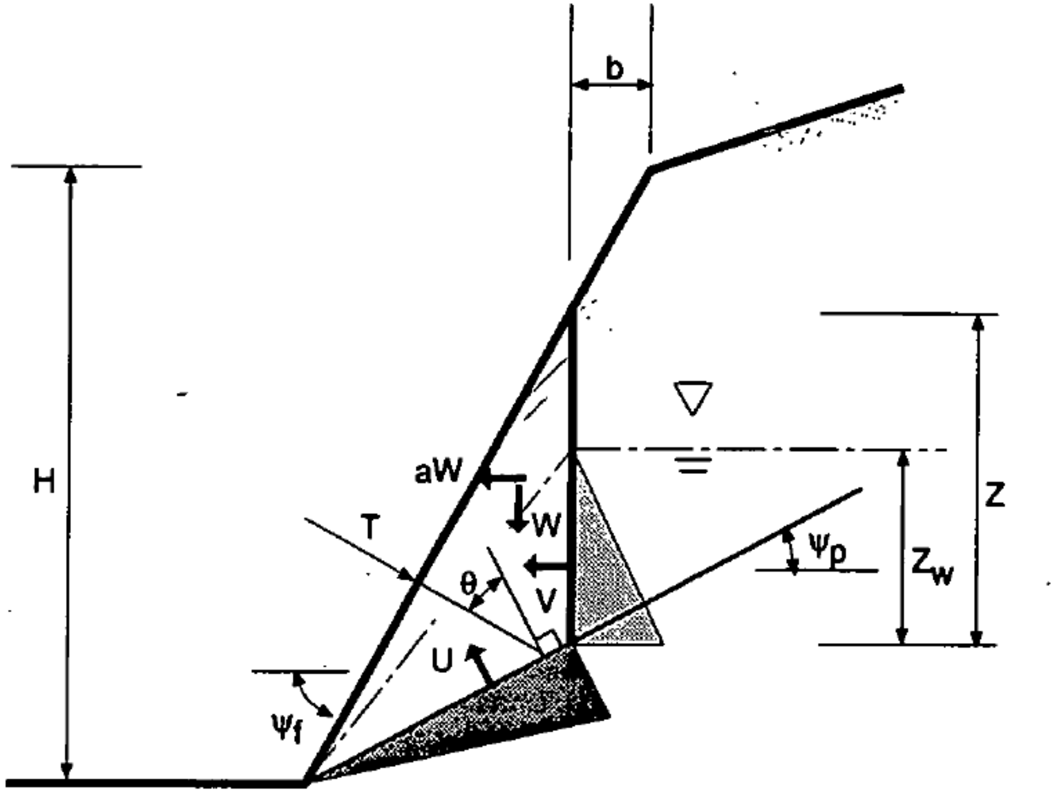


Fig. 4. Water force and pressure distribution in slope having potential plane mode of failure.

Kinematic analysis for planar failure.



(a) Tension Crack in Slope Face

Factor of safety:

$$FS = \frac{\{cA + [W(\cos\Psi_p - a\sin\Psi_p) - U - V\sin\Psi_p + T\cos\theta]\tan\phi\}}{[W(\sin\Psi_p + a\cos\Psi_p) + V\cos\Psi_p - T\sin\theta]}$$

where

H = height of slope face;
Ψ_f = inclination of slope face;
Ψ_s = inclination of upper slope face;
Ψ_p = inclination of failure plane;
b = distance of tension crack from slope crest;
a = horizontal acceleration, blast or earthquake loading;
T = tension in bolts or cables;
θ = inclination of bolt or cable to normal to failure plane;
c = cohesive strength of failure surface;
ϕ = friction angle of failure surface;
γ_r = density of rock;
γ_w = density of water;
Z_w = height of water in tension crack;
Z = depth of tension crack;
U = uplift water force;
V = driving water force;
W = weight of sliding block; and
A = area of failure surface.

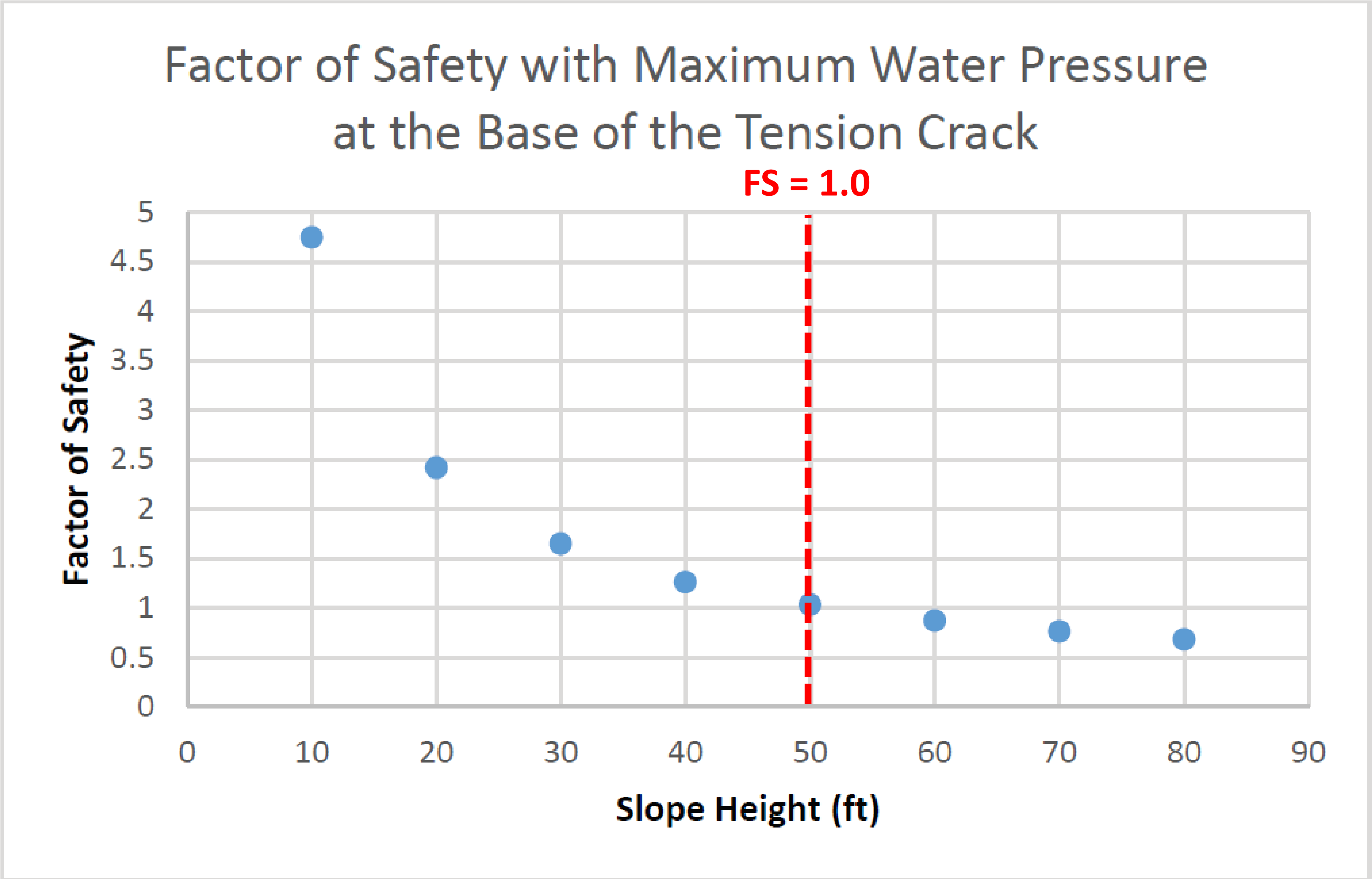


FIGURE 12: GRAPH OF THE FACTOR OF SAFETY VERSUS SLOPE HEIGHT WITH FRACTURES AND WATER PRESSURE AT THE BASE OF THE FRACTURE.

Clinometer Data

Ground Movement
due to excavation

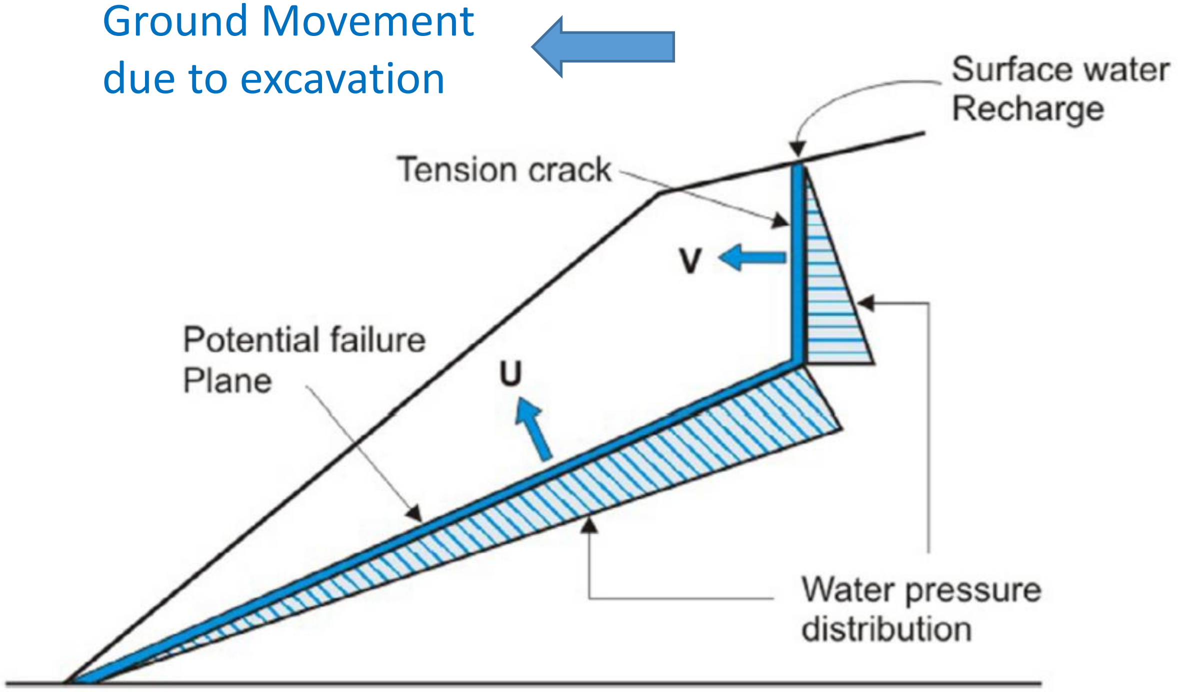
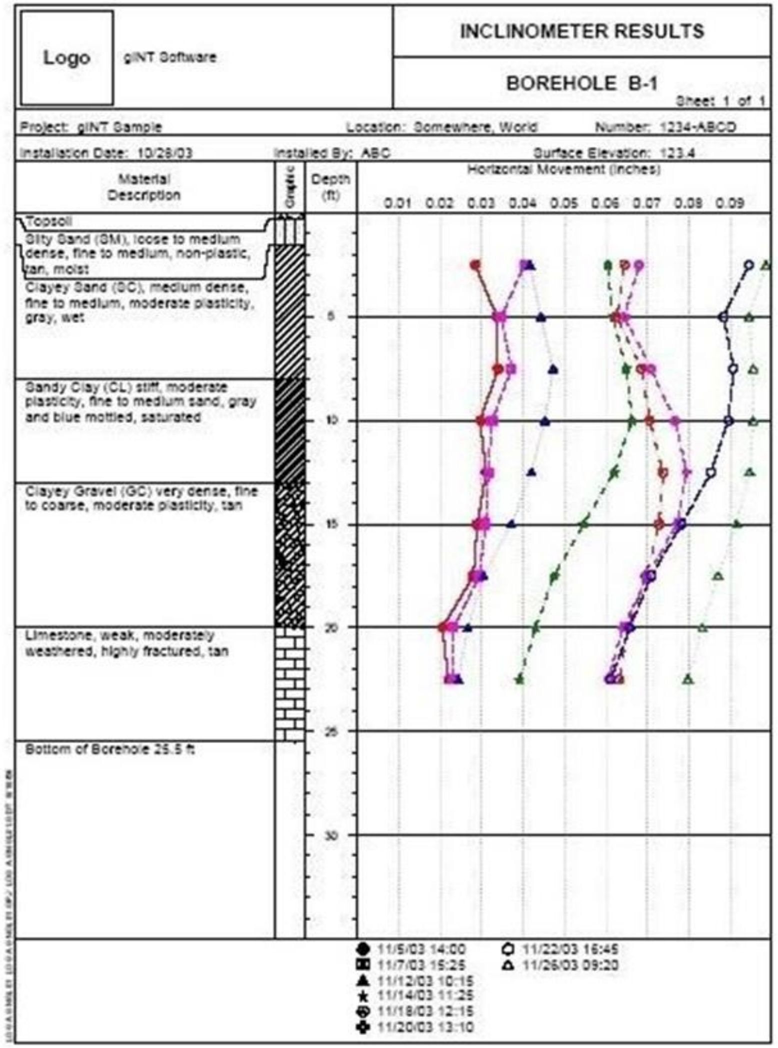


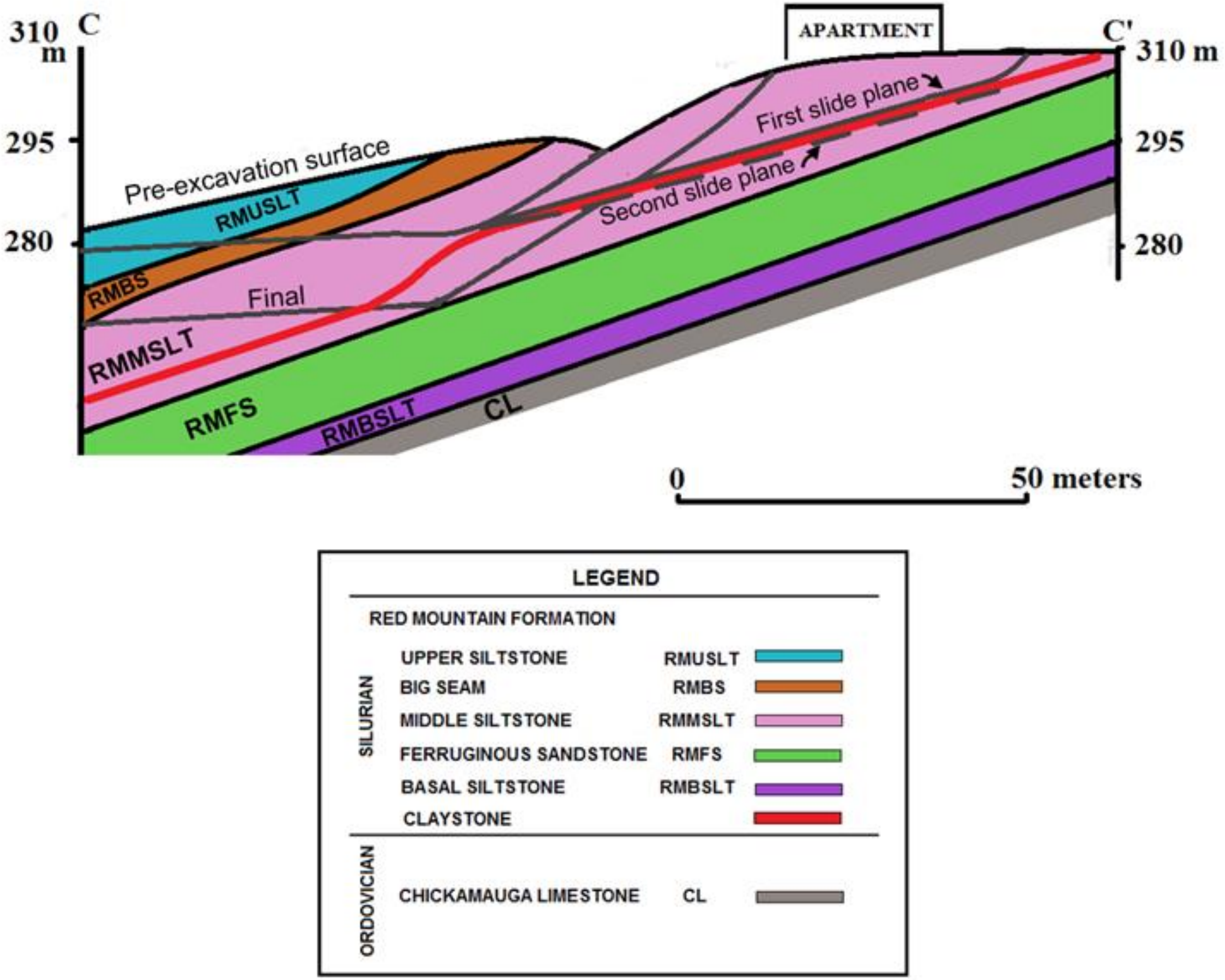
Fig. 4. Water force and pressure distribution in slope having potential plane mode of failure.



Flac 3D Model

PhD Student:

- Spent a month – part time setting up the model
- Was not able to accurately model the joint system's deformation movement
- Results were inconclusive



Lessons Learned

- On many projects, close enough is good enough when using classical methods of analysis, at least to get an initial understand of the project.
- Coulomb Earth Pressure Theory is adequate for most analysis
- Earth pressures for slopes should use a log-spiral method
- The most important use of classical methods is to investigate the big issues quickly and inexpensively.
- Numerical methods, however, should be use don more complex projects.