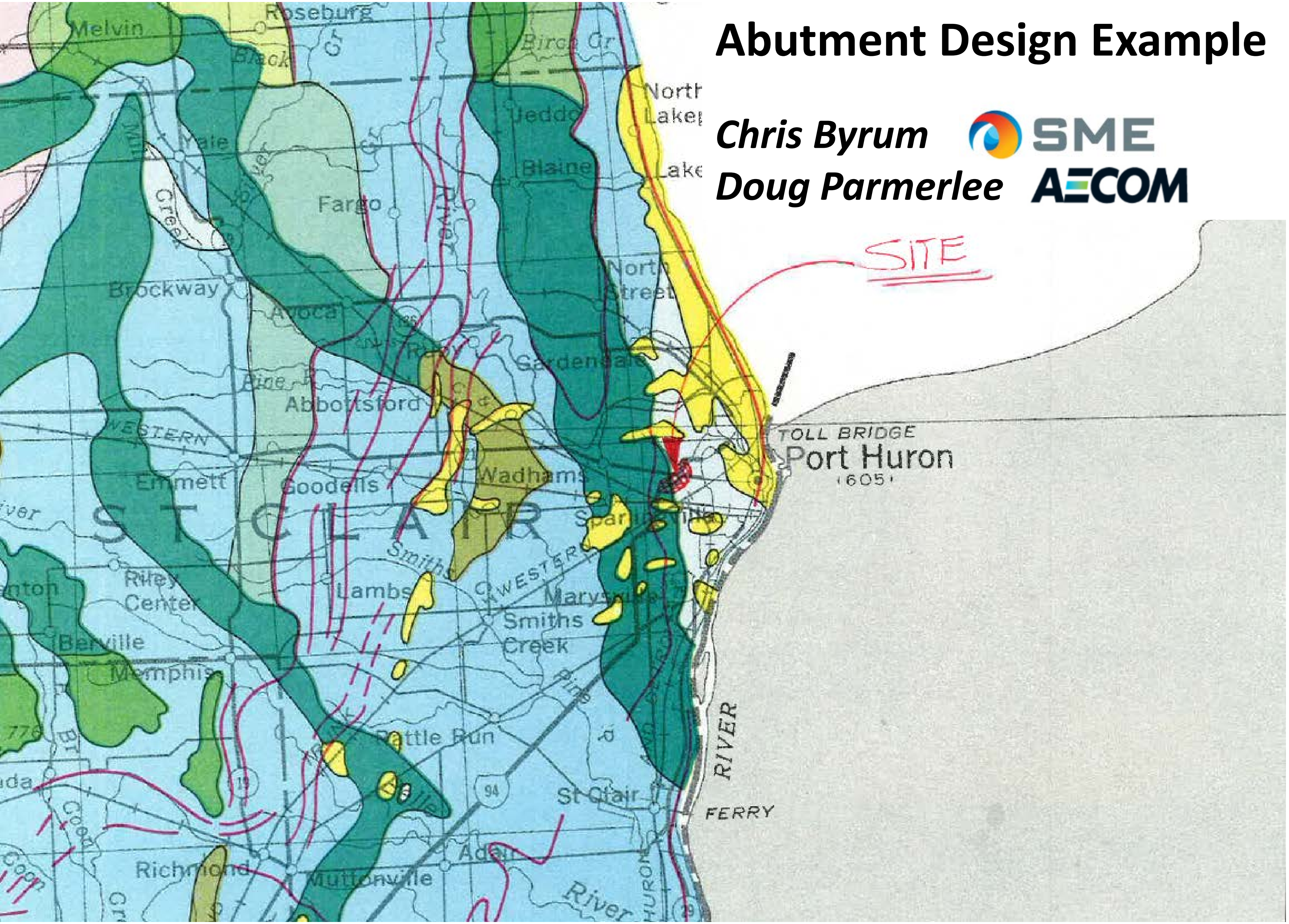
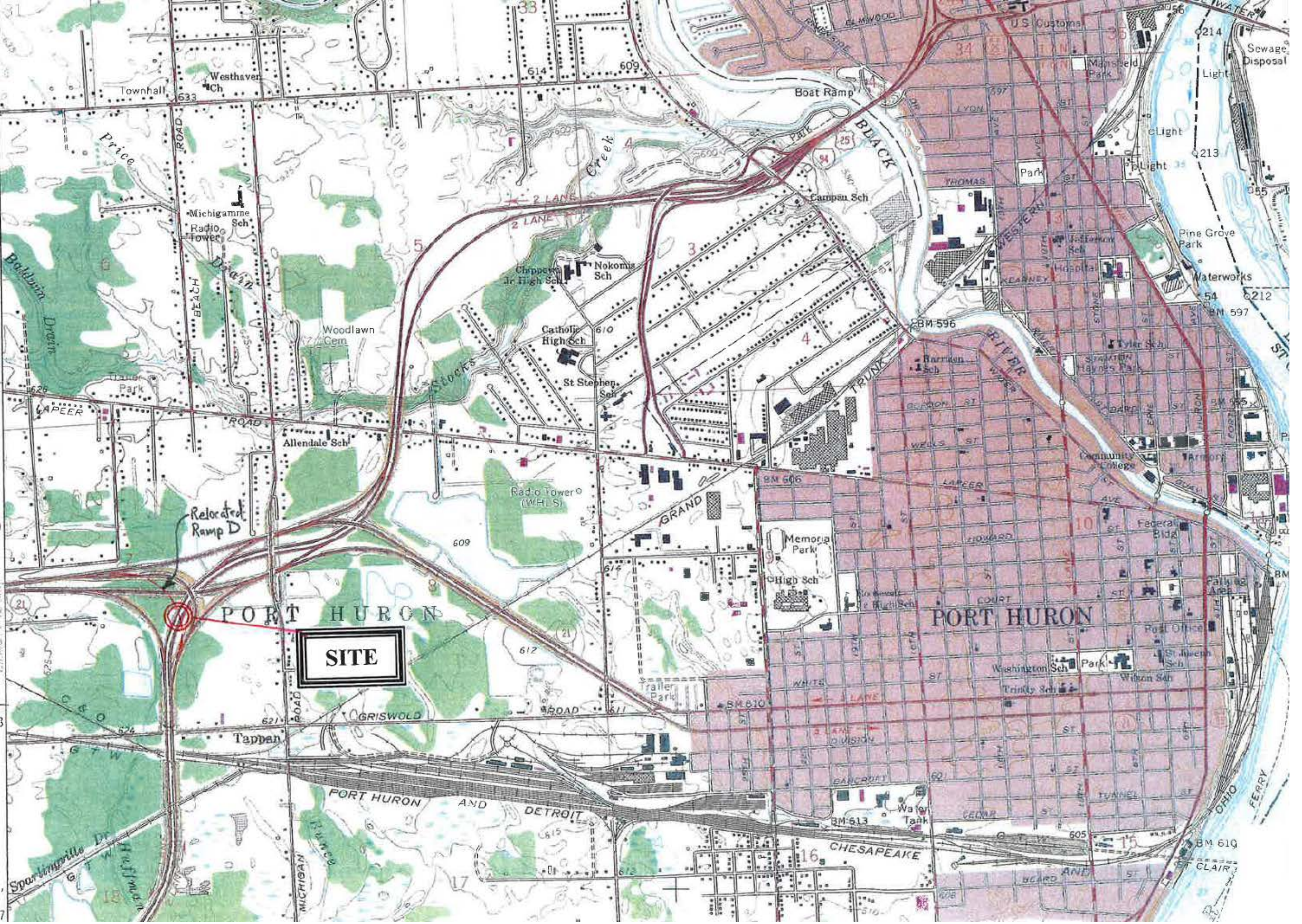
Abutment Design Example

Chris Byrum SME

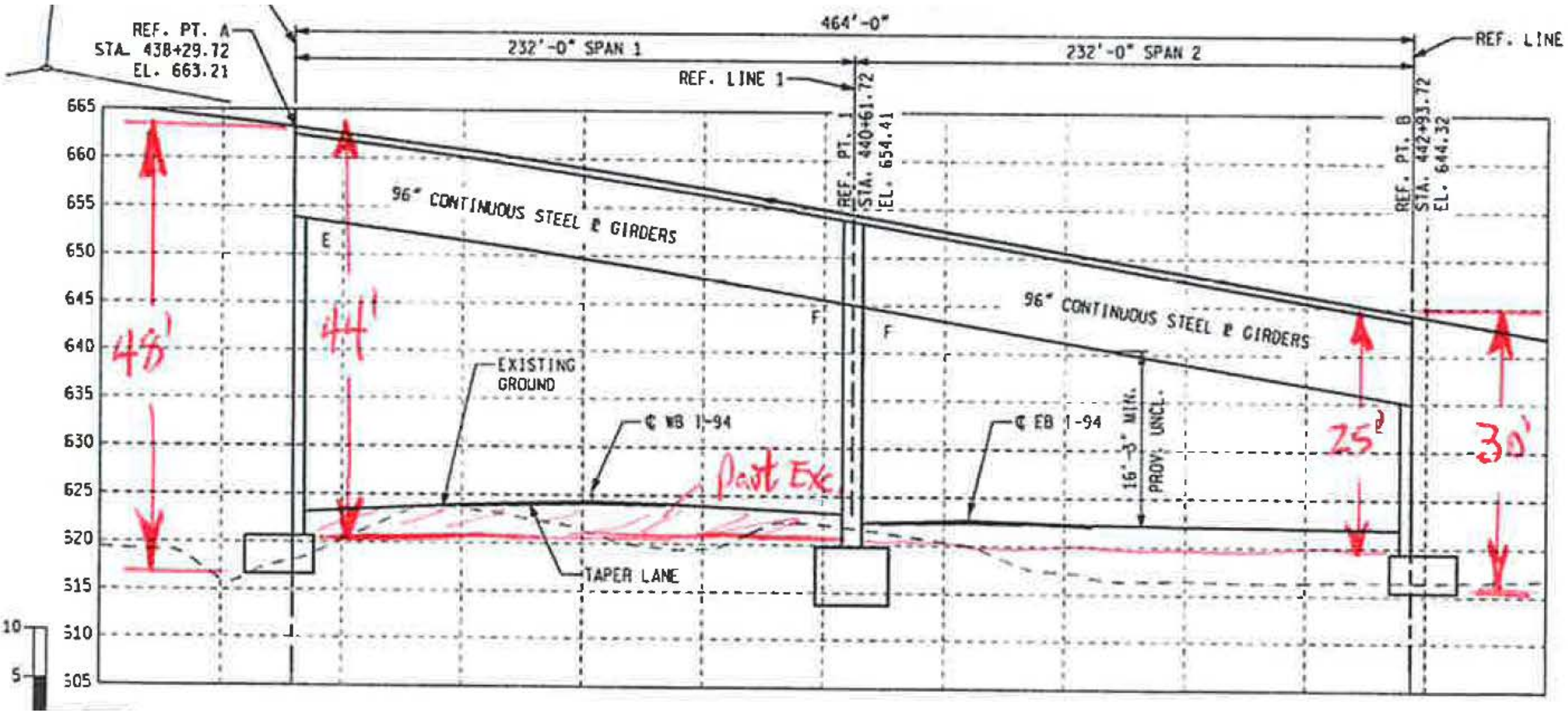
Doug Parmerlee AECOM




SITE
Relocated Ramp D
PORT HURON
PORT HURON
Westhaven Ch
Townhall
Price
Michigamme Sch
Radio Tower
Baldwin Drain
Drain
BEACH ROAD
Woodlawn Cem
Trailer Park
LAPEER ROAD
Allendale Sch
Stocks Creek
Chippewa Jr High Sch
Nokomis Sch
Catholic High Sch
St Stephen Sch
Radio Tower (WHLS)
2 LANE
Campau Sch
Boat Ramp
BLACK RIVER
GRAND TRUNK WESTERN
BM 596
BM 606
BM 610
BM 613
BM 610
U S Customs
Mansfield Park
Park
Jefferson Sch
Hospital
Tyler Sch
Haynes Park
Harrison Sch
Community College
Armory
Federal Bldg
Memorial Park
High Sch
Roosevelt Sch
Washington Sch
Park
Wilson Sch
Trinity Sch
St Joseph Sch
Post Office
Parking Area
Trailer Park
Water Tank
Pine Grove Park
Waterworks
Sewage Disposal
Light
Light
Light
GRISWOLD ROAD
Tappen
MICHIGAN ROAD
C & O
G T W
PORT HURON AND DETROIT
CHESAPEAKE AND OHIO
FERRY
ST CLAIR
Sparlingville Dr
Hufman
Bunce
ELMWOOD
LYON
THOMAS
KEARNEY
GLENWOOD
WELLS
LAPEER AVE
HOWARD
COURT
WHITE
DIVISION
BANCROFT
TUNNEL
CEDAR
BEARD
609
614
633
609
610
612
621
624
626
597
605
606
611
612
615
21
25
94
3
4
5
8
9
10
15
16
17
18
33
34
35

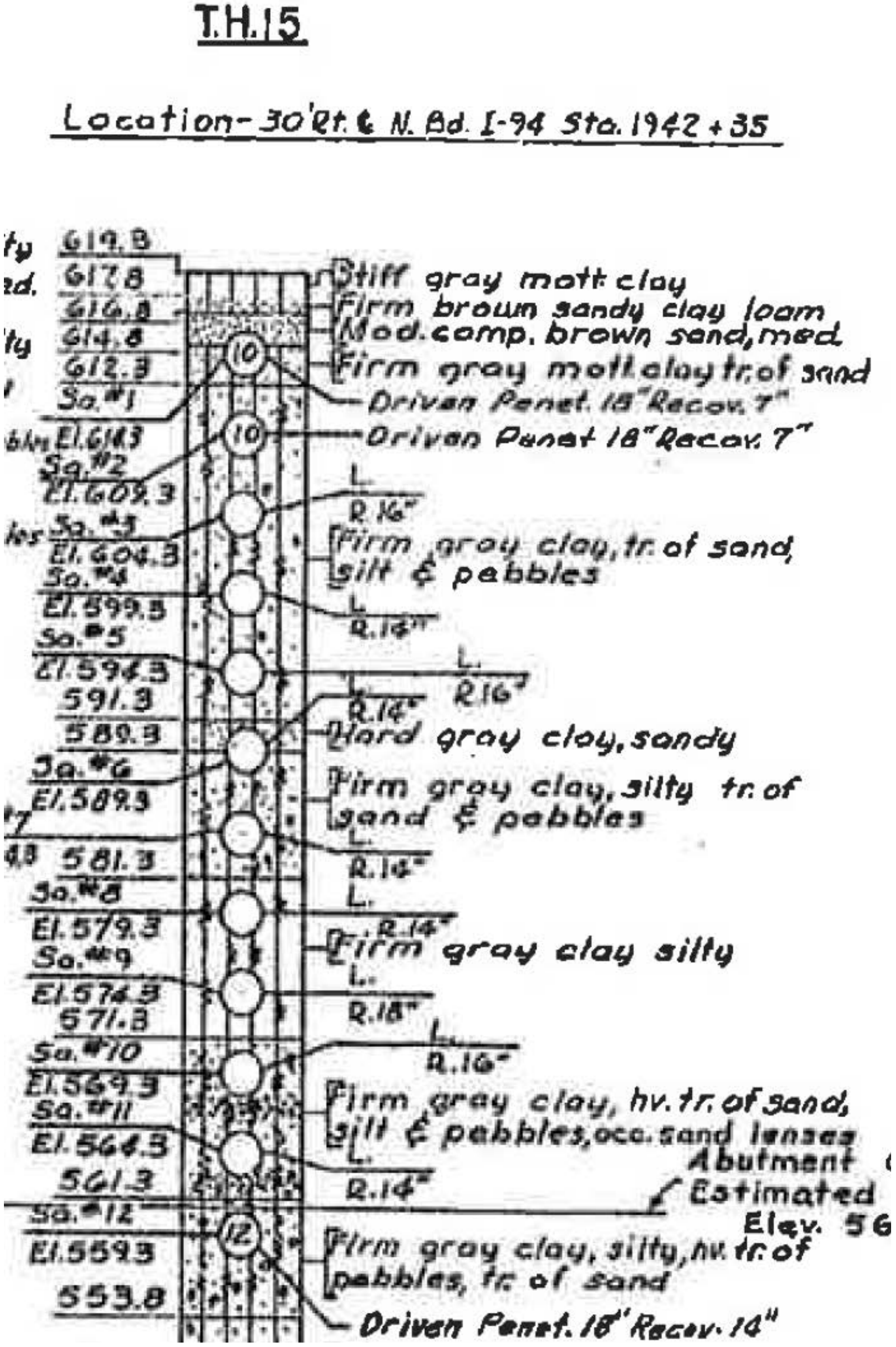

N
94
Michigan Rd
Old Michigan Rd
© 2015 Google
Google earth

Example Bridge



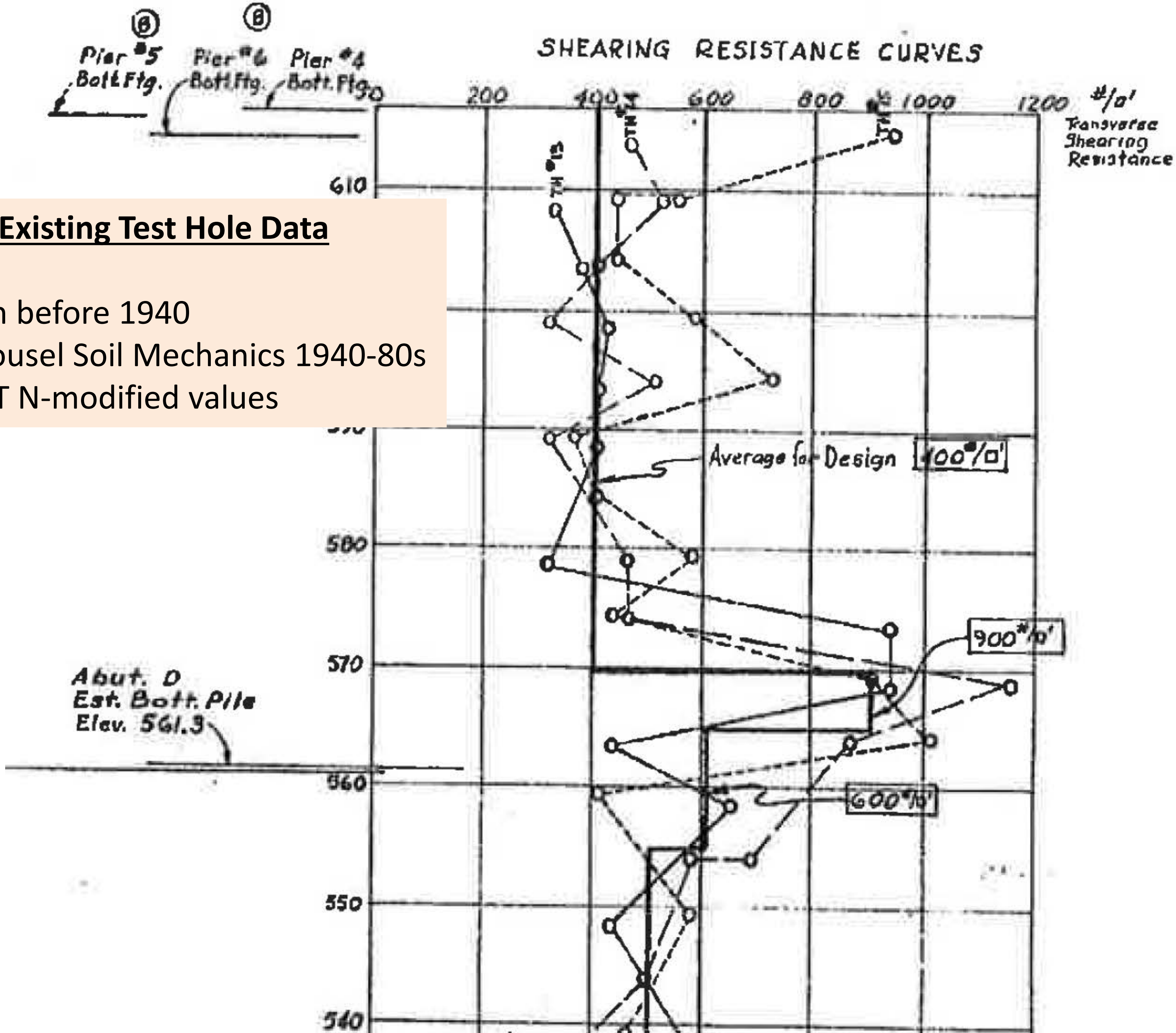
Evaluate Existing Test Hole Data

Not much before 1940
MDOT Housel Soil Mechanics 1940-80s
ASTM SPT N-modified values



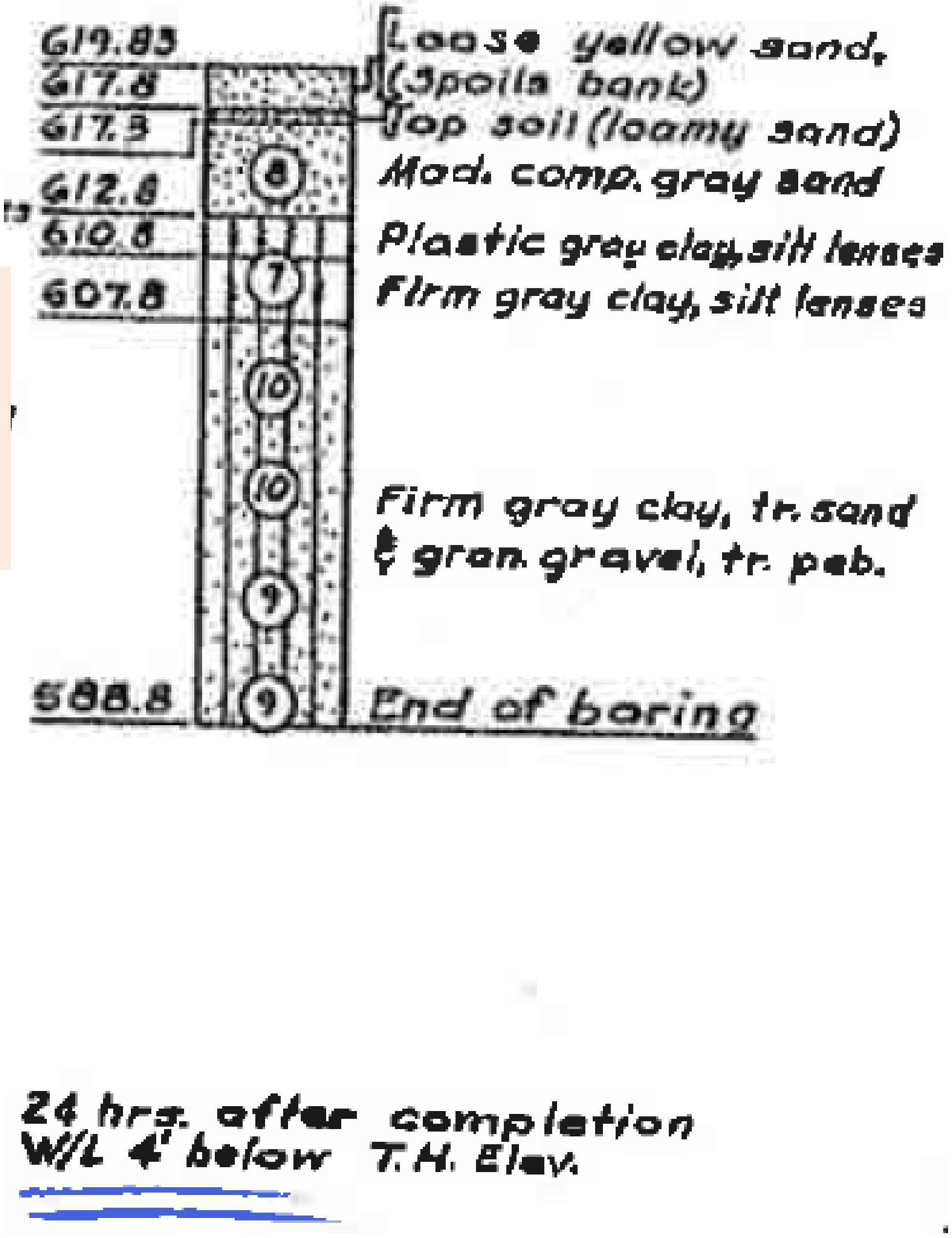
Evaluate Existing Test Hole Data

Not much before 1940
MDOT Housel Soil Mechanics 1940-80s
ASTM SPT N-modified values

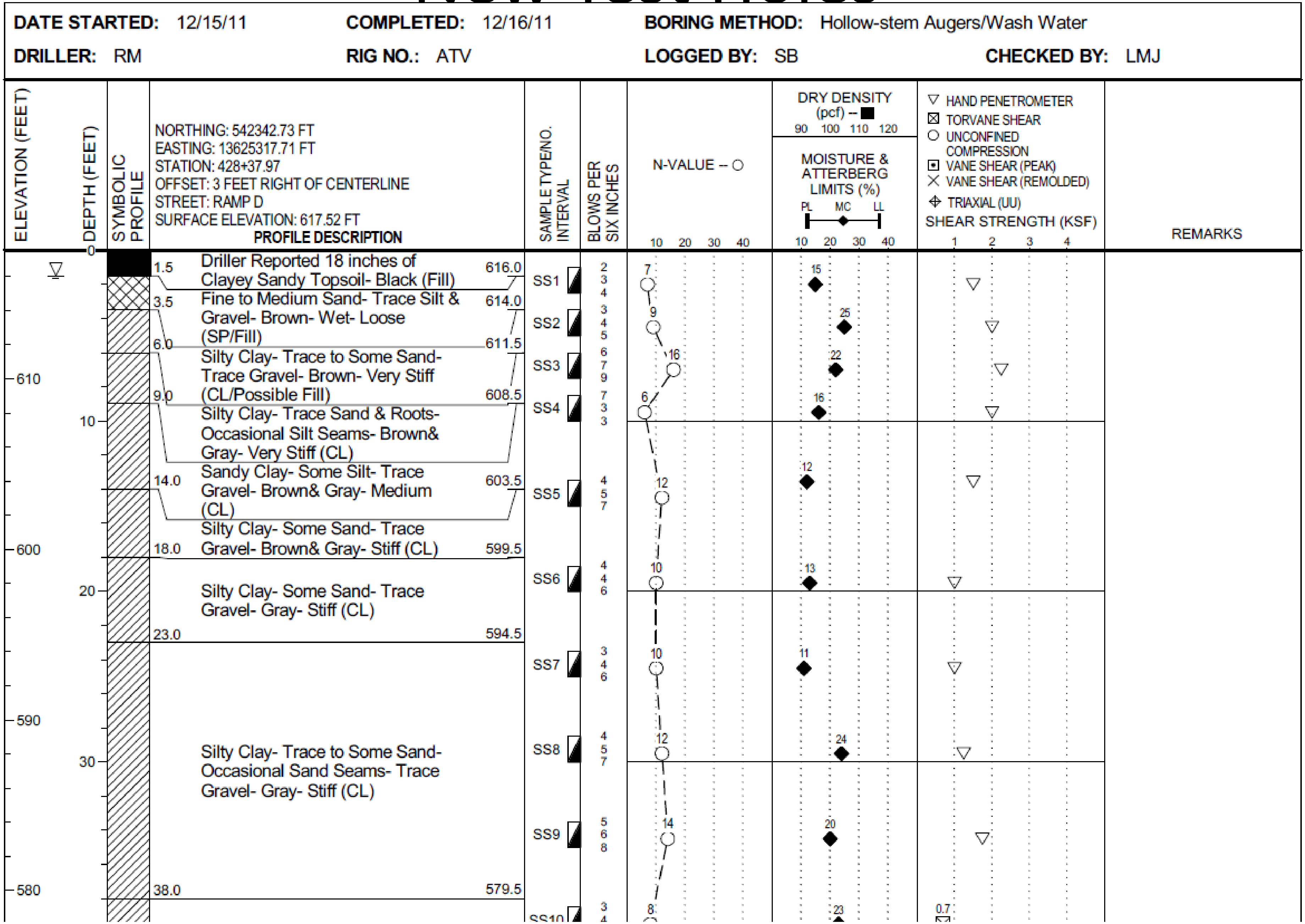


Evaluate Existing Test Hole Data

Not much before 1940
MDOT Housel Soil Mechanics 1940-80s
ASTM SPT N-modified values



New Test Holes



DATE STARTED: 12/15/11 **COMPLETED:** 12/16/11 **BORING METHOD:** Hollow-stem Augers/Wash Water

DRILLER: RM **RIG NO.:** ATV **LOGGED BY:** SB **CHECKED BY:** LMJ

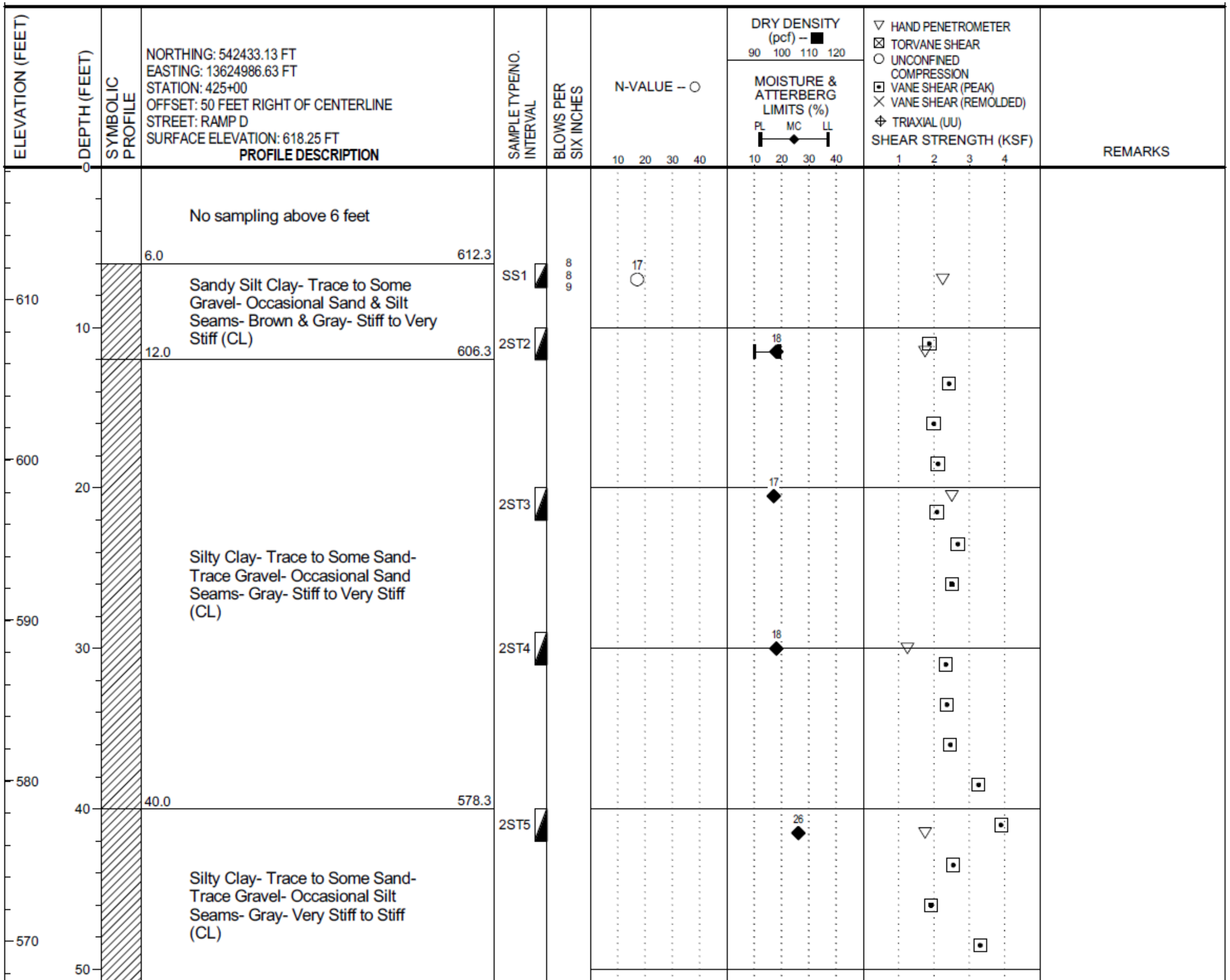
NORTHING: 542342.73 FT
EASTING: 13625317.71 FT
STATION: 428+37.97
OFFSET: 3 FEET RIGHT OF CENTERLINE
STREET: RAMP D
SURFACE ELEVATION: 617.52 FT

DEPTH (FEET)	PROFILE DESCRIPTION	ELEVATION (FEET)
1.5	Driller Reported 18 inches of Clayey Sandy Topsoil- Black (Fill)	616.0
3.5	Fine to Medium Sand- Trace Silt & Gravel- Brown- Wet- Loose (SP/Fill)	614.0
6.0	Silty Clay- Trace to Some Sand- Trace Gravel- Brown- Very Stiff (CL/Possible Fill)	611.5
9.0	Silty Clay- Trace Sand & Roots- Occasional Silt Seams- Brown& Gray- Very Stiff (CL)	608.5
14.0	Sandy Clay- Some Silt- Trace Gravel- Brown& Gray- Medium (CL)	603.5
18.0	Silty Clay- Some Sand- Trace Gravel- Brown& Gray- Stiff (CL)	599.5
	Silty Clay- Some Sand- Trace Gravel- Gray- Stiff (CL)	
23.0		594.5
	Silty Clay- Trace to Some Sand- Occasional Sand Seams- Trace Gravel- Gray- Stiff (CL)	
38.0		579.5

SAMPLE TYPE/NO. INTERVAL	BLOWS PER SIX INCHES
SS1	2 3 4
SS2	3 4 5
SS3	6 7 9
SS4	7 3 3
SS5	4 5 7
SS6	4 4 6
SS7	3 4 6
SS8	4 5 7
SS9	5 6 8
SS10	3 4

REMARKS

In-Situ Vane-shear and 3” Shelby Tubes

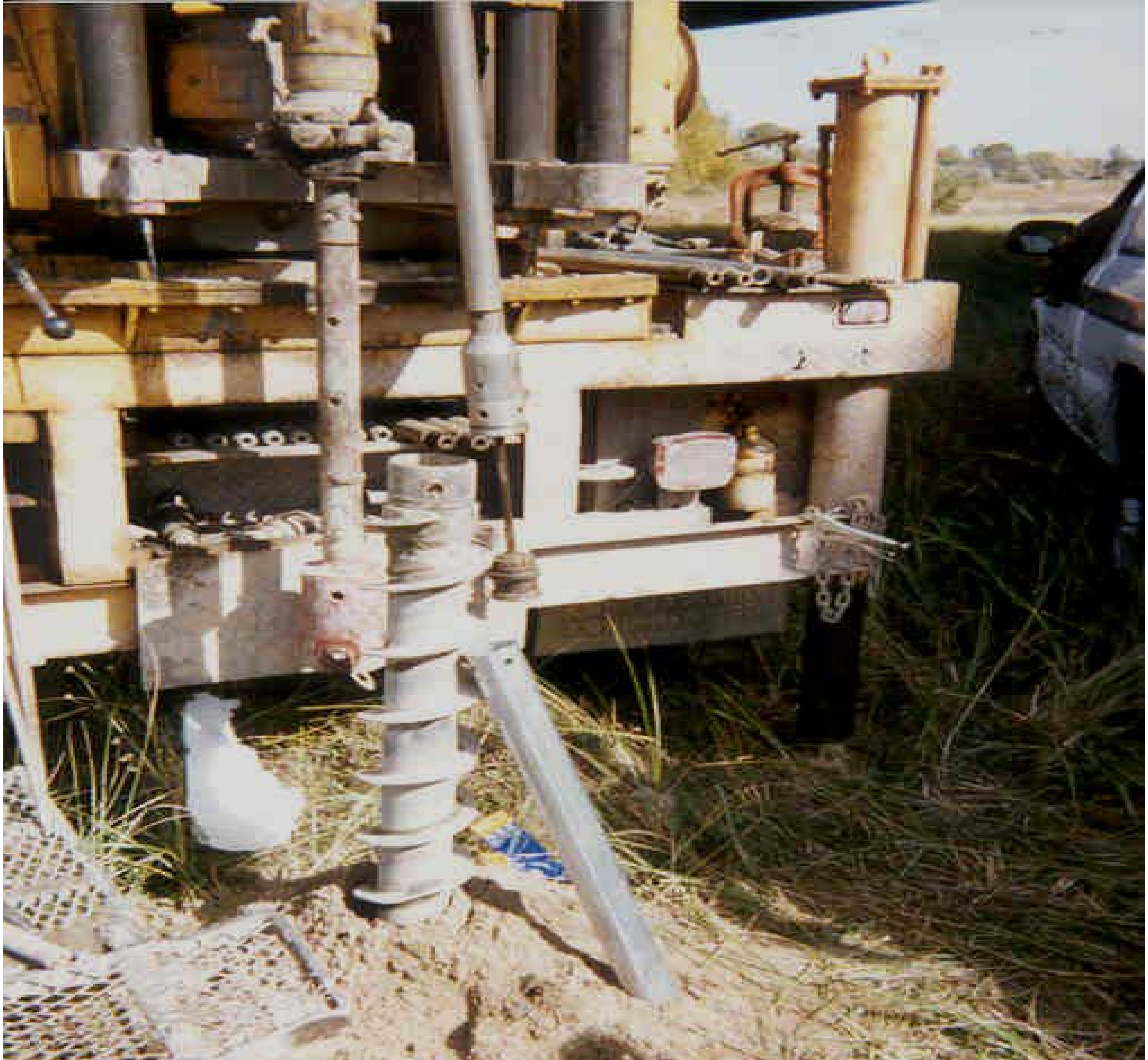


NORTHING: 542433.13 FT
EASTING: 13624986.63 FT
STATION: 425+00
OFFSET: 50 FEET RIGHT OF CENTERLINE
STREET: RAMP D
SURFACE ELEVATION: 618.25 FT

PROFILE DESCRIPTION

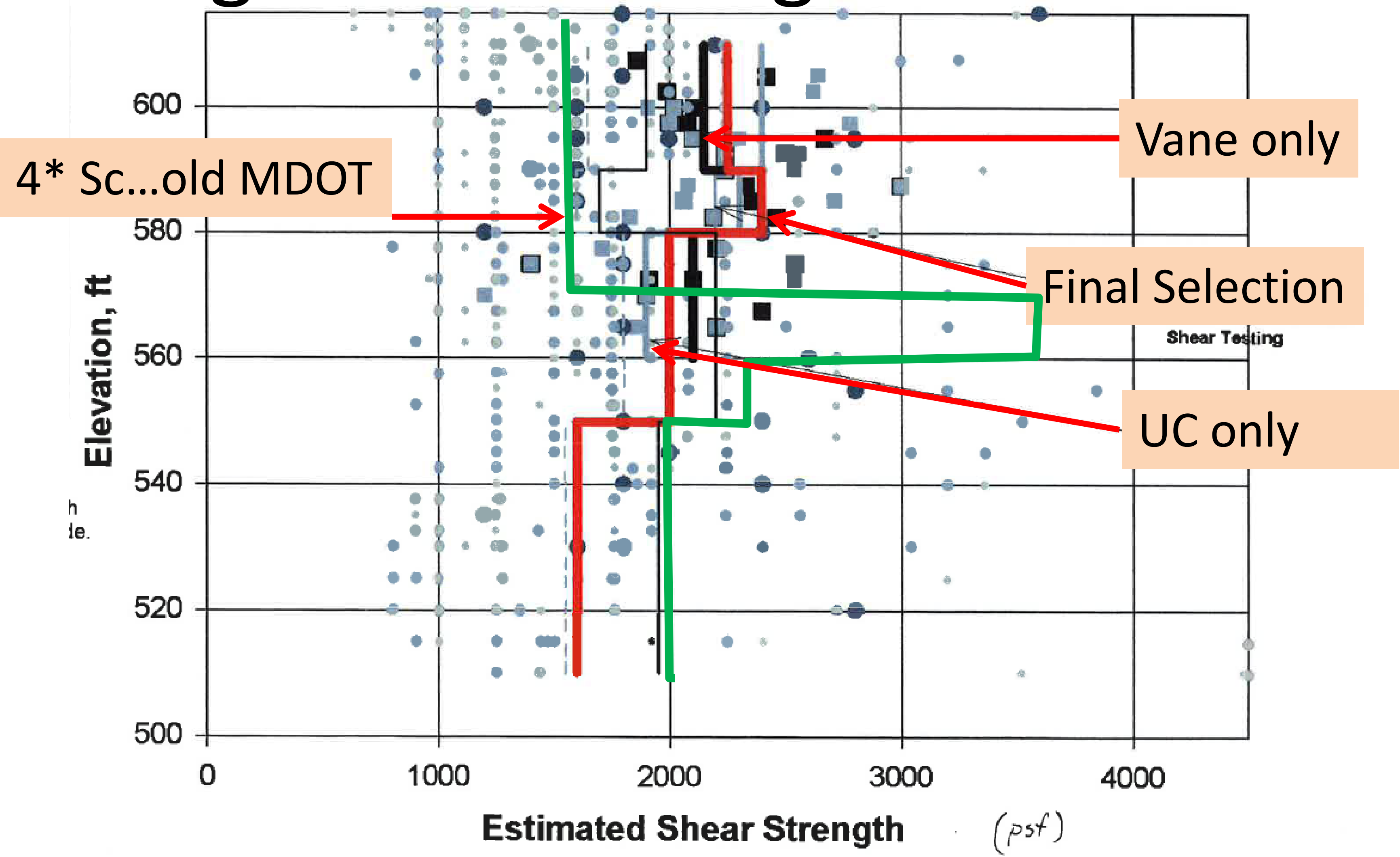
DEPTH (FEET)	PROFILE DESCRIPTION	ELEVATION (FEET)
0.0–6.0	No sampling above 6 feet	612.3
6.0–12.0	Sandy Silt Clay- Trace to Some Gravel- Occasional Sand & Silt Seams- Brown & Gray- Stiff to Very Stiff (CL)	606.3
12.0–40.0	Silty Clay- Trace to Some Sand- Trace Gravel- Occasional Sand Seams- Gray- Stiff to Very Stiff (CL)	578.3
40.0–	Silty Clay- Trace to Some Sand- Trace Gravel- Occasional Silt Seams- Gray- Very Stiff to Stiff (CL)	

SAMPLE TYPE/NO. INTERVAL	BLOWS PER SIX INCHES	N-VALUE	MOISTURE (%)
SS1	8 8 9	17	
2ST2			18
2ST3			17
2ST4			18
2ST5			26



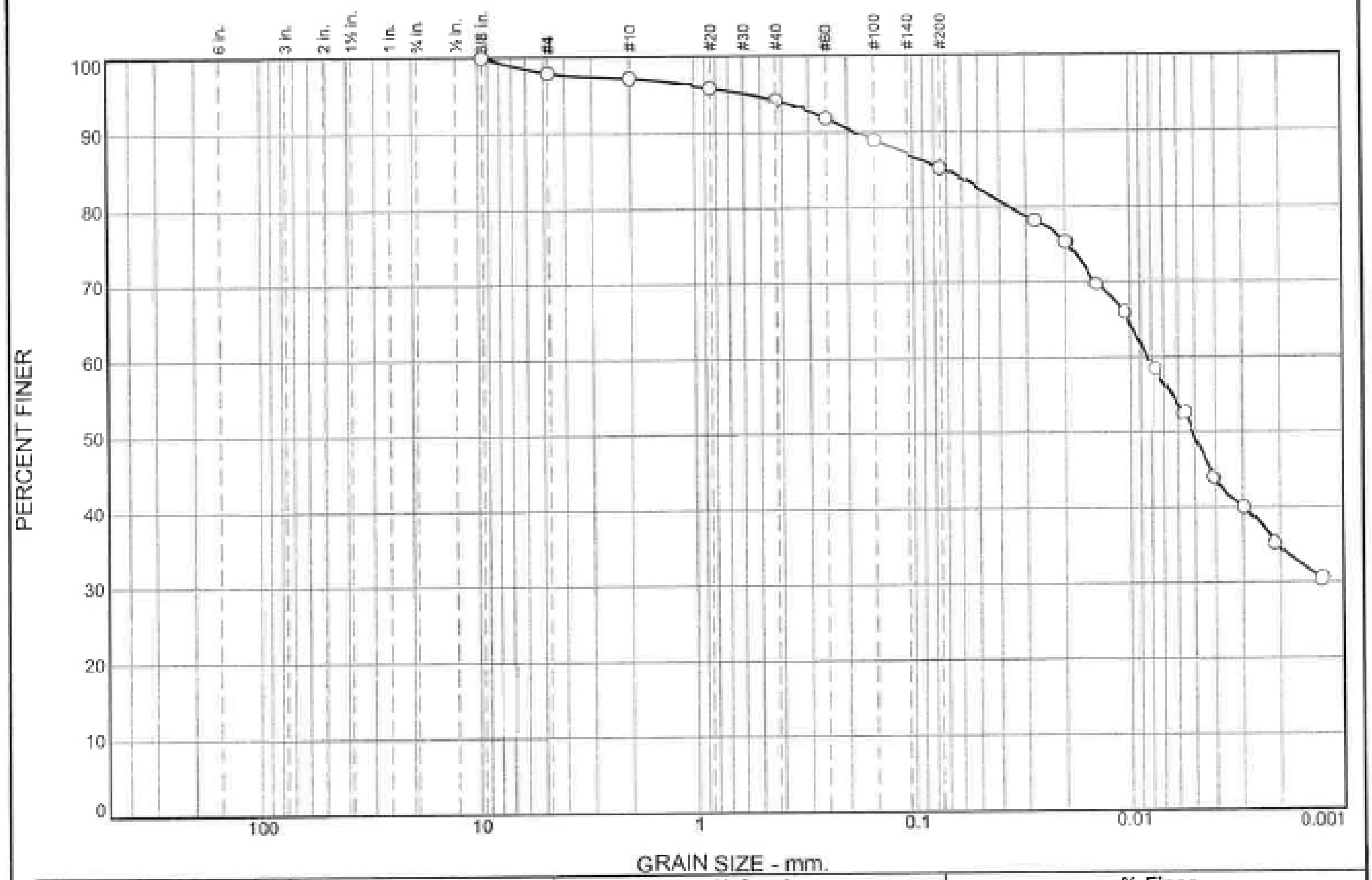

acker

Design Shear Strength Profiles



Used for: Global Stability, Bearing Capacity, Piling Side Resistance, Pile Lateral…..

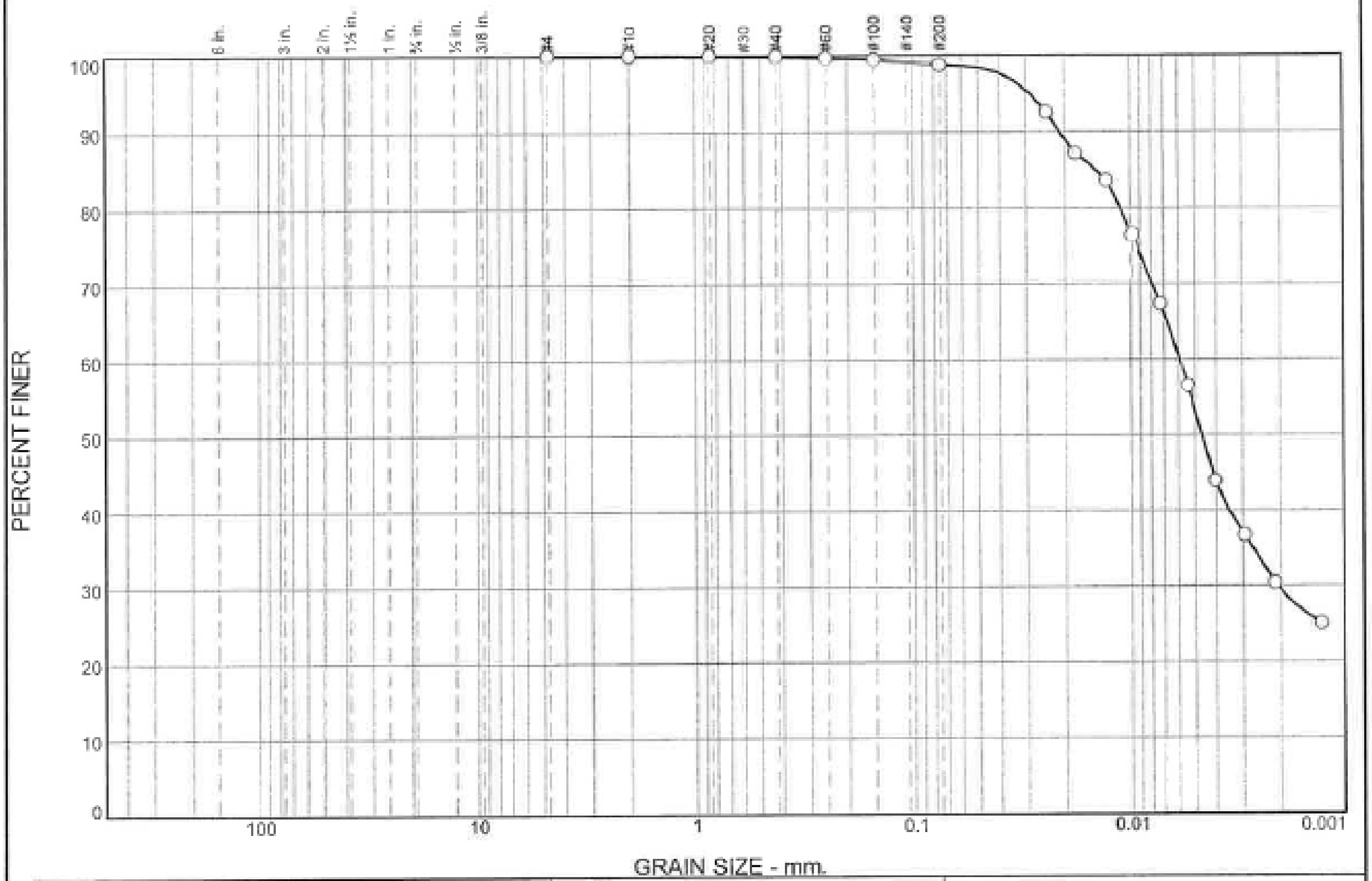
Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	2.0	0.8	3.0	9.0	36.1	49.1

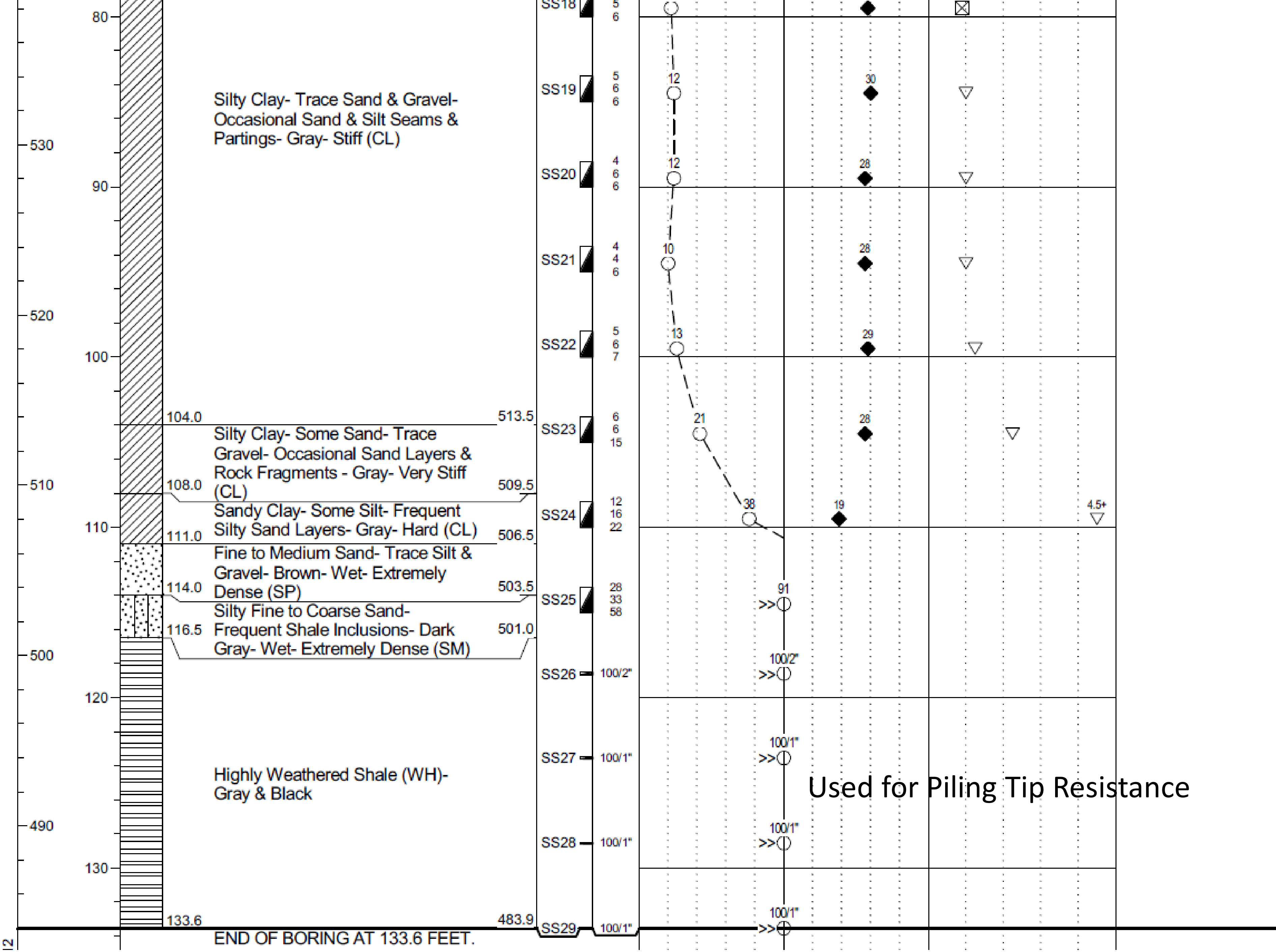
☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	27	13	0.0727	0.0084	0.0052					

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.0	0.0	0.1	1.1	45.9	52.9

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	31	16	0.0143	0.0059	0.0047	0.0021				



Used for Piling Tip Resistance

External Stability

- Task 1: Lateral Squeeze
- Task 2: Global Stability
- Task 3: Settlement Analyses
- Task 4:Bearing Capacity

LATERAL SQUEEZE ANALYSIS

$$\mathbf{FS =} \frac{2*Cu}{gama*Ds*tan(theta)} + \frac{(4.14*Cu)}{H*gama}$$

Also used in the past… H*gama < 3Cu, or 4Cu

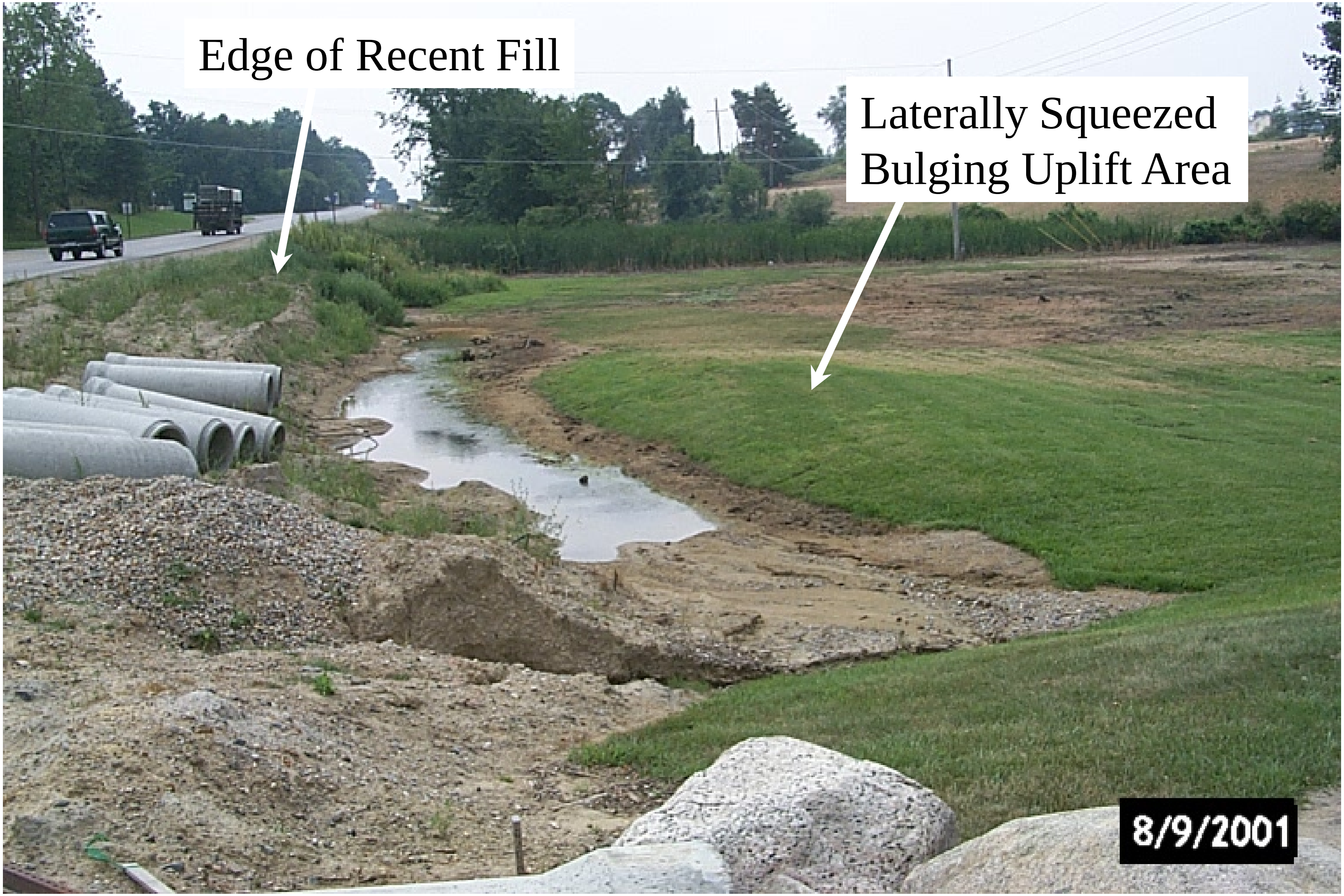
Where,

Cu = Undrained shear strength of soft layer, psf
Ds = Thickness of soft soil layer, ft
gama = Unit weight of fill soil, psf
theta = Angle of fill slope, degrees
H = Height of fill, ft

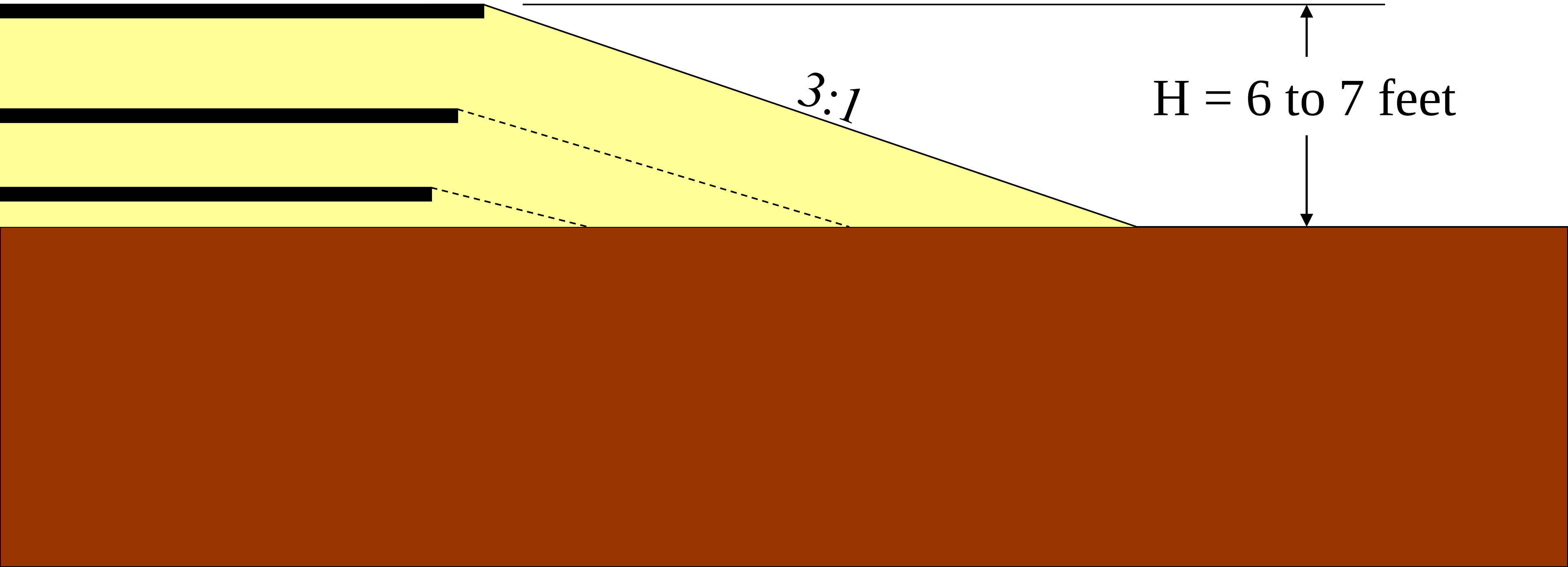

Primary Shear

8/9/2001

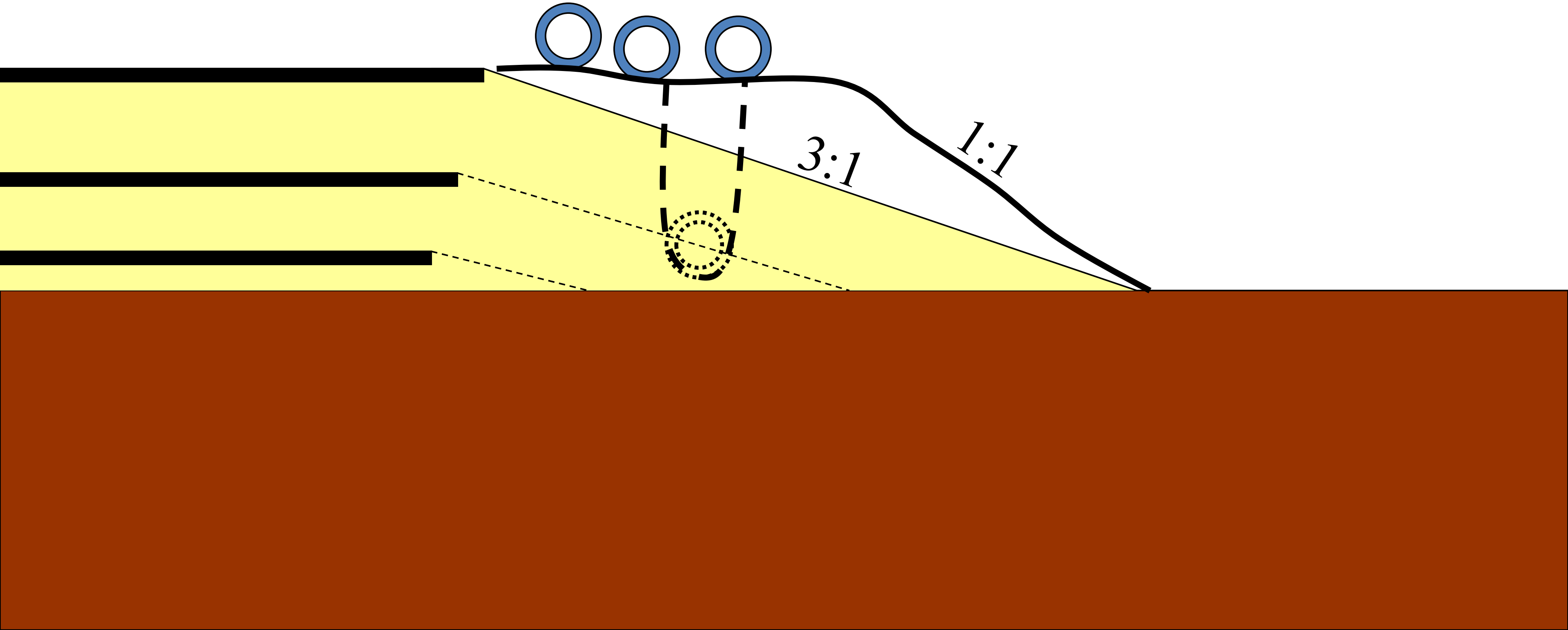
“Secondary/Sympathy” Shear


Edge of Recent Fill
Laterally Squeezed Bulging Uplift Area
8/9/2001

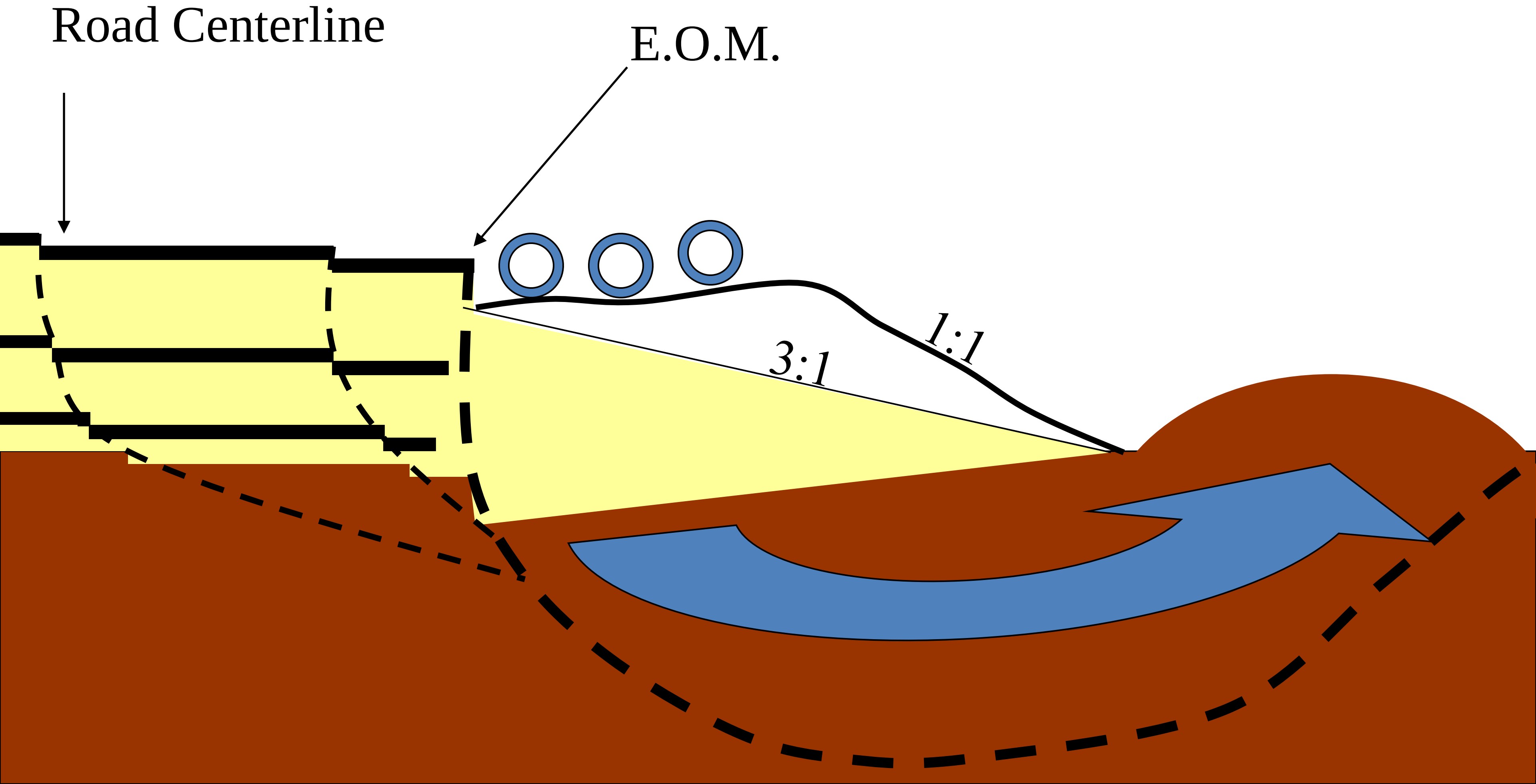
Previous Scenario



Utility Contractor Adds Weight



Reaches ➔ **F.S. = 1.0 Condition**



Failure Backcalculation

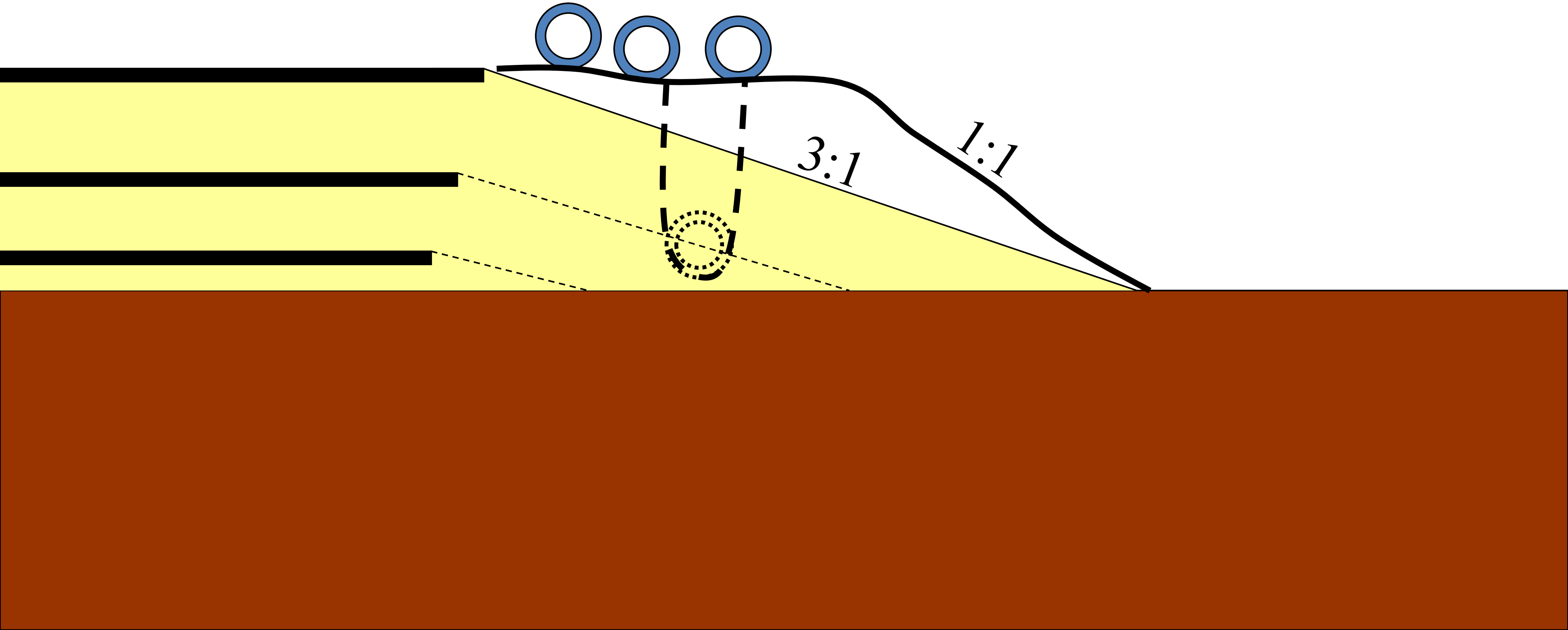
$$FS = \frac{2*Cu}{gama*Ds*\tan(theta)} + \frac{(4.14*Cu)}{H*gama}$$

For FS = 1

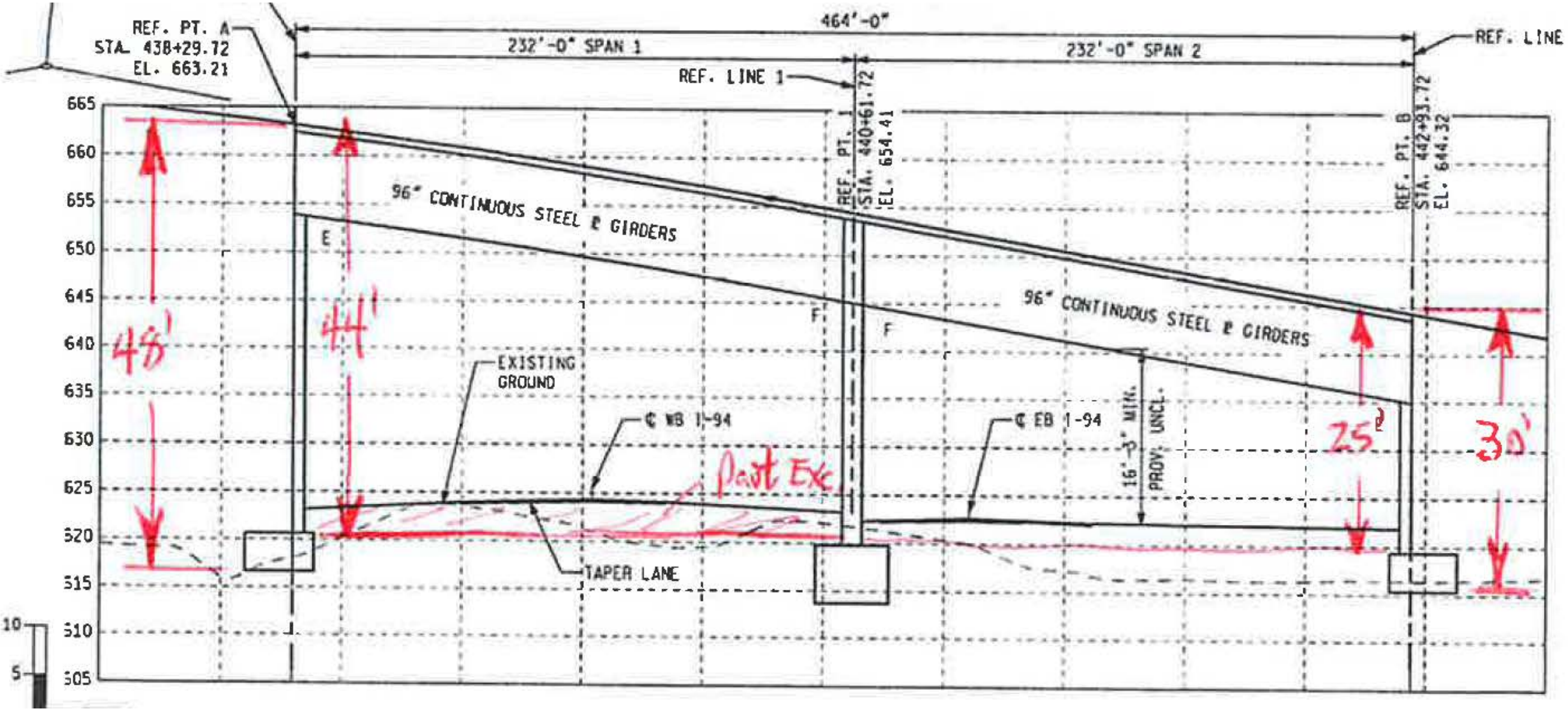
Cu =	180	psf
Ds =	30	ft
gama =	125	pcf
theta =	45	degrees
H =	6.5	ft

Component 1	**Component 2**	**FS**
0.10	*0.92*	***1.01***

3*Cu = 540
4*Cu = 720
gama*H = 812.5



Example Bridge



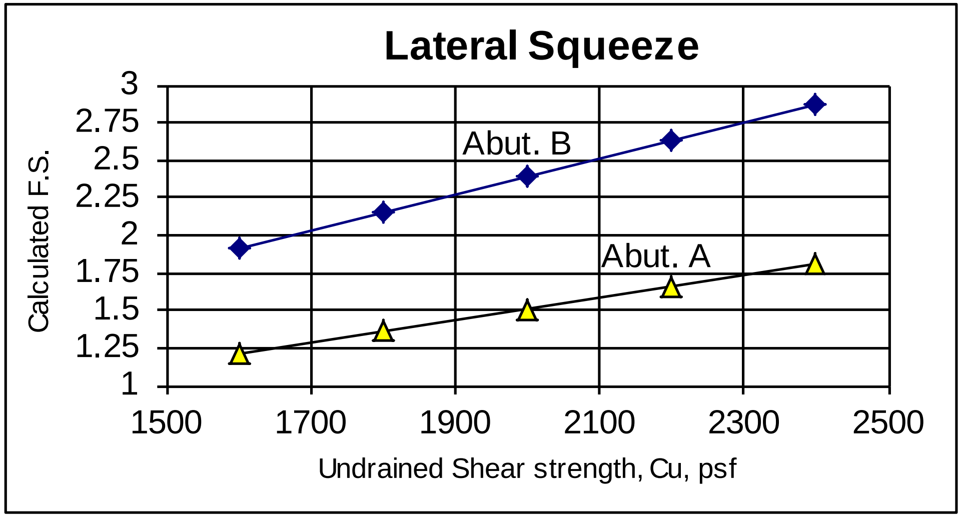
$$FS = \frac{2*Cu}{gama*Ds*tan(theta)} + \frac{(4.14*Cu)}{H*gama}$$

Cu =	2100	psf
Ds =	100	ft
gama =	120	pcf
theta =	88	degrees
H =	46	ft

Component 1	Component 2	FS
0.01	*1.58*	***1.59***

3*Cu = 6300
4*Cu = 8400
gama*H = 5520

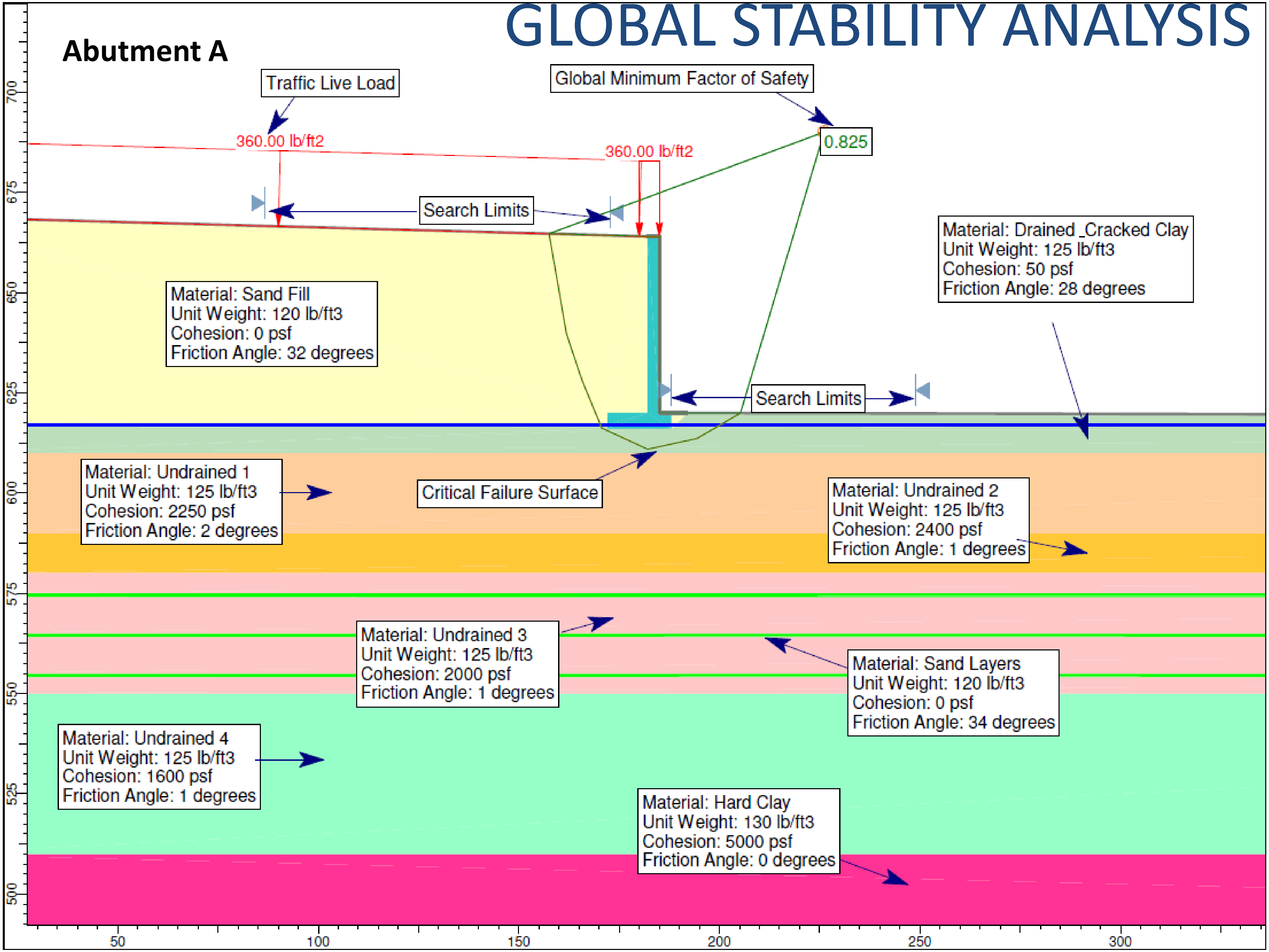
Cu	FS
1600	1.91
1800	2.15
2000	2.39
2200	2.63
2400	2.87



Where,
Cu = Undrained shear strength of soft layer, psf
Ds = Thickness of soft soil layer, ft
gama = Unit weight of fill soil, psf
theta = Angle of fill slope, degrees
H = Height of fill, ft

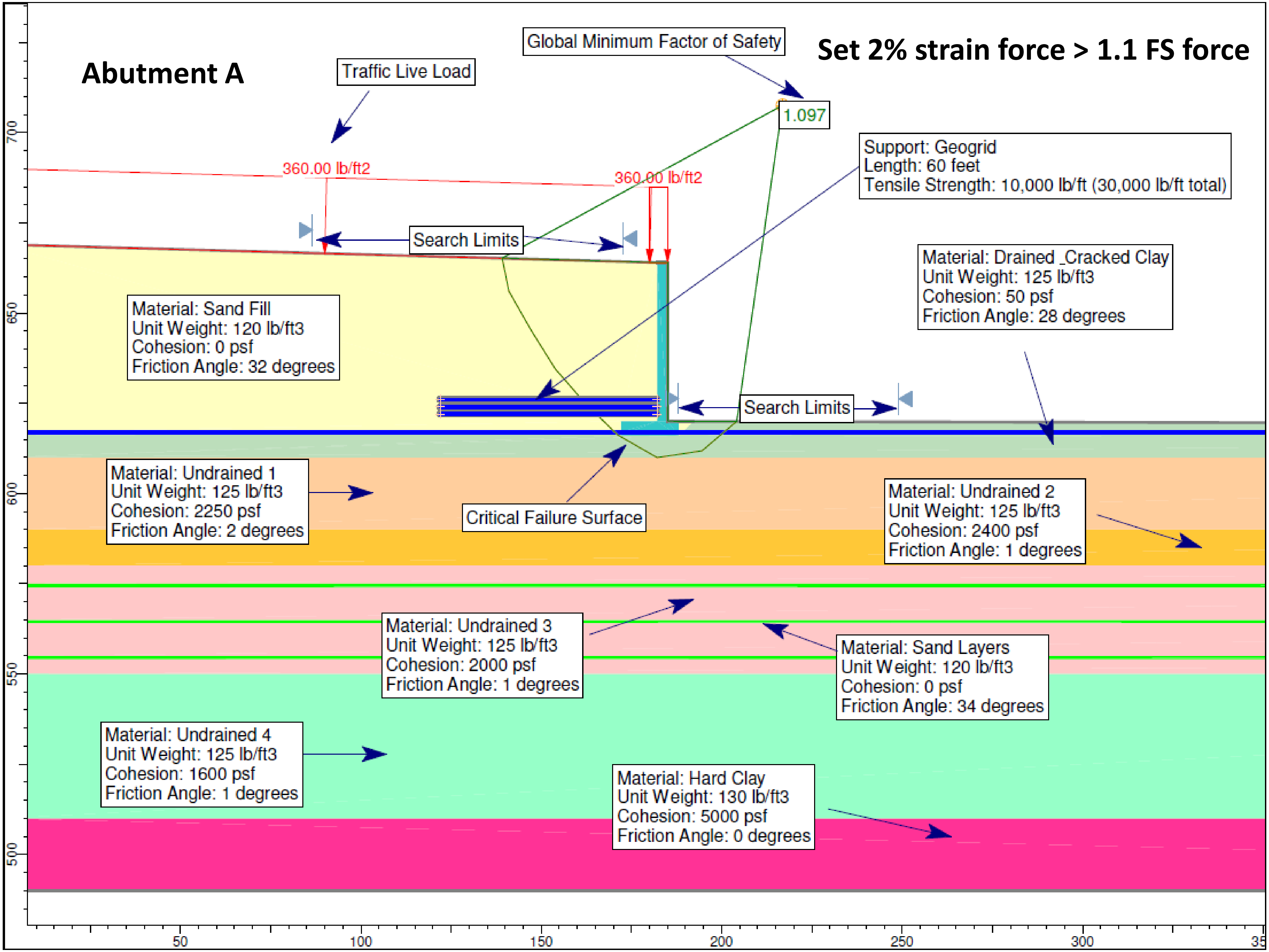
GLOBAL STABILITY ANALYSIS

Abutment A



Abutment A

Set 2% strain force > 1.1 FS force



Support: Geogrid
Length: 60 feet
Tensile Strength: 10,000 lb/ft (30,000 lb/ft total)

Material: Drained _Cracked Clay
Unit Weight: 125 lb/ft3
Cohesion: 50 psf
Friction Angle: 28 degrees

Material: Sand Fill
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 32 degrees

Material: Undrained 1
Unit Weight: 125 lb/ft3
Cohesion: 2250 psf
Friction Angle: 2 degrees

Material: Undrained 2
Unit Weight: 125 lb/ft3
Cohesion: 2400 psf
Friction Angle: 1 degrees

Material: Undrained 3
Unit Weight: 125 lb/ft3
Cohesion: 2000 psf
Friction Angle: 1 degrees

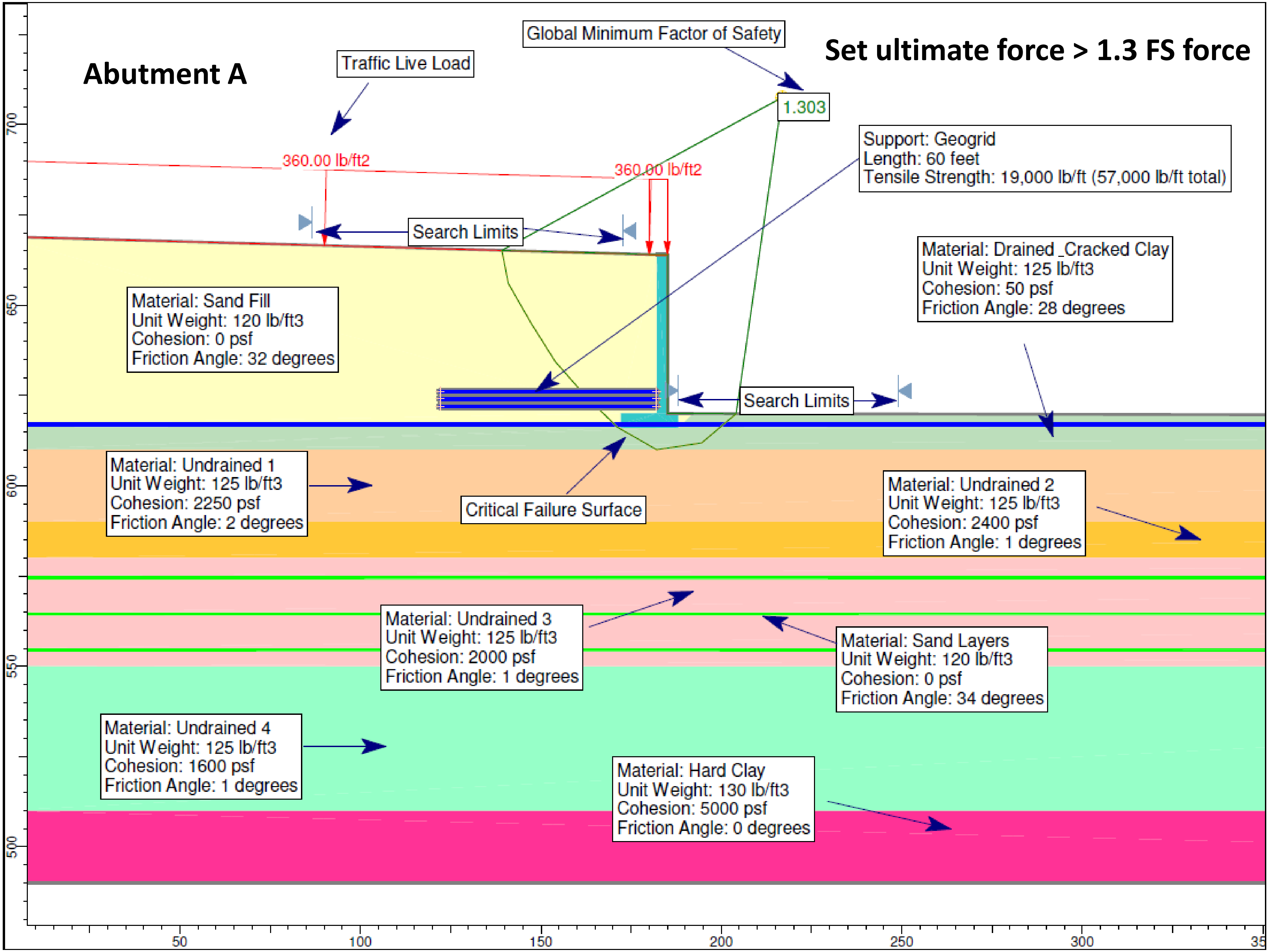
Material: Sand Layers
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 34 degrees

Material: Undrained 4
Unit Weight: 125 lb/ft3
Cohesion: 1600 psf
Friction Angle: 1 degrees

Material: Hard Clay
Unit Weight: 130 lb/ft3
Cohesion: 5000 psf
Friction Angle: 0 degrees

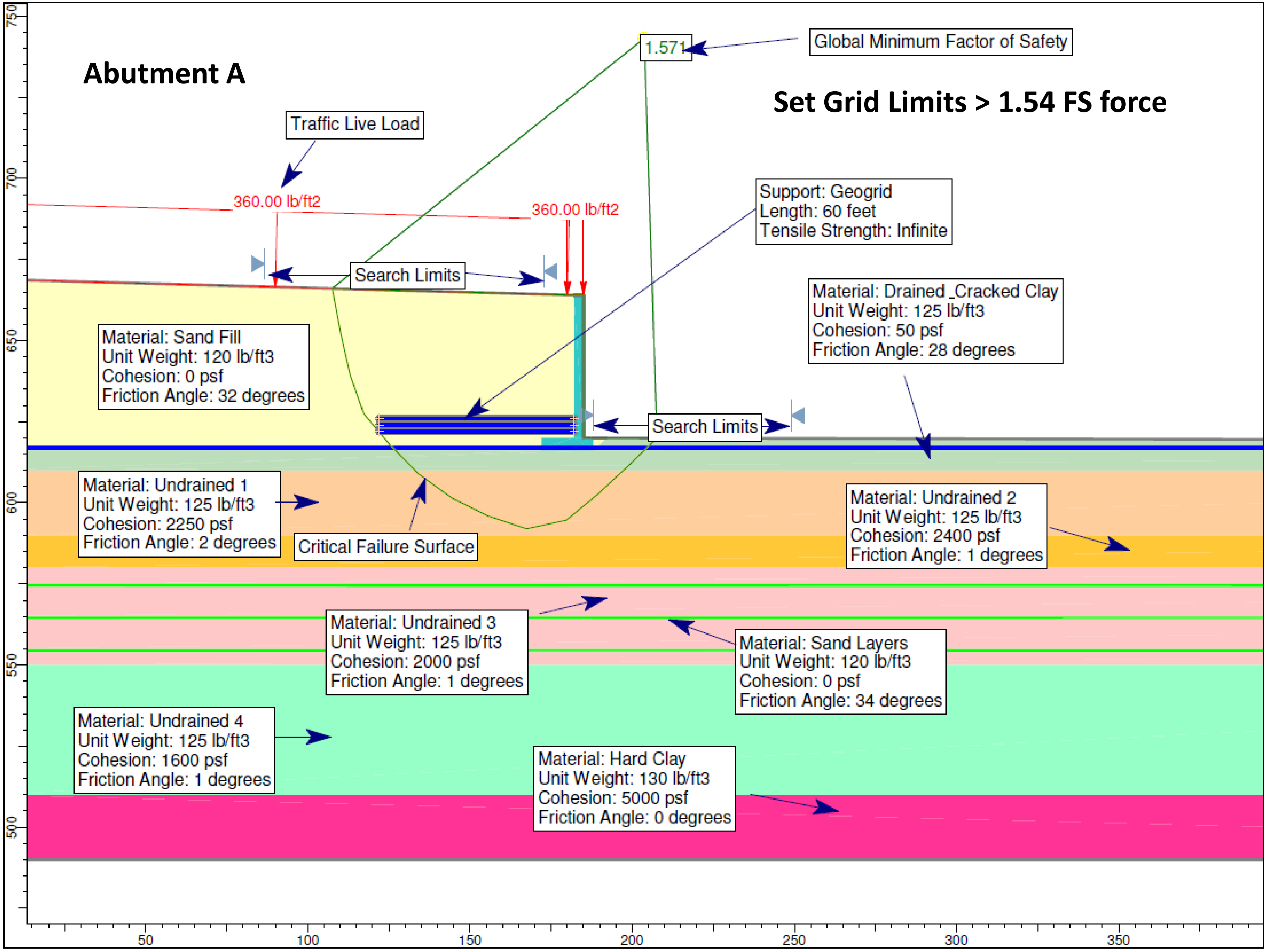
Abutment A

Set ultimate force > 1.3 FS force

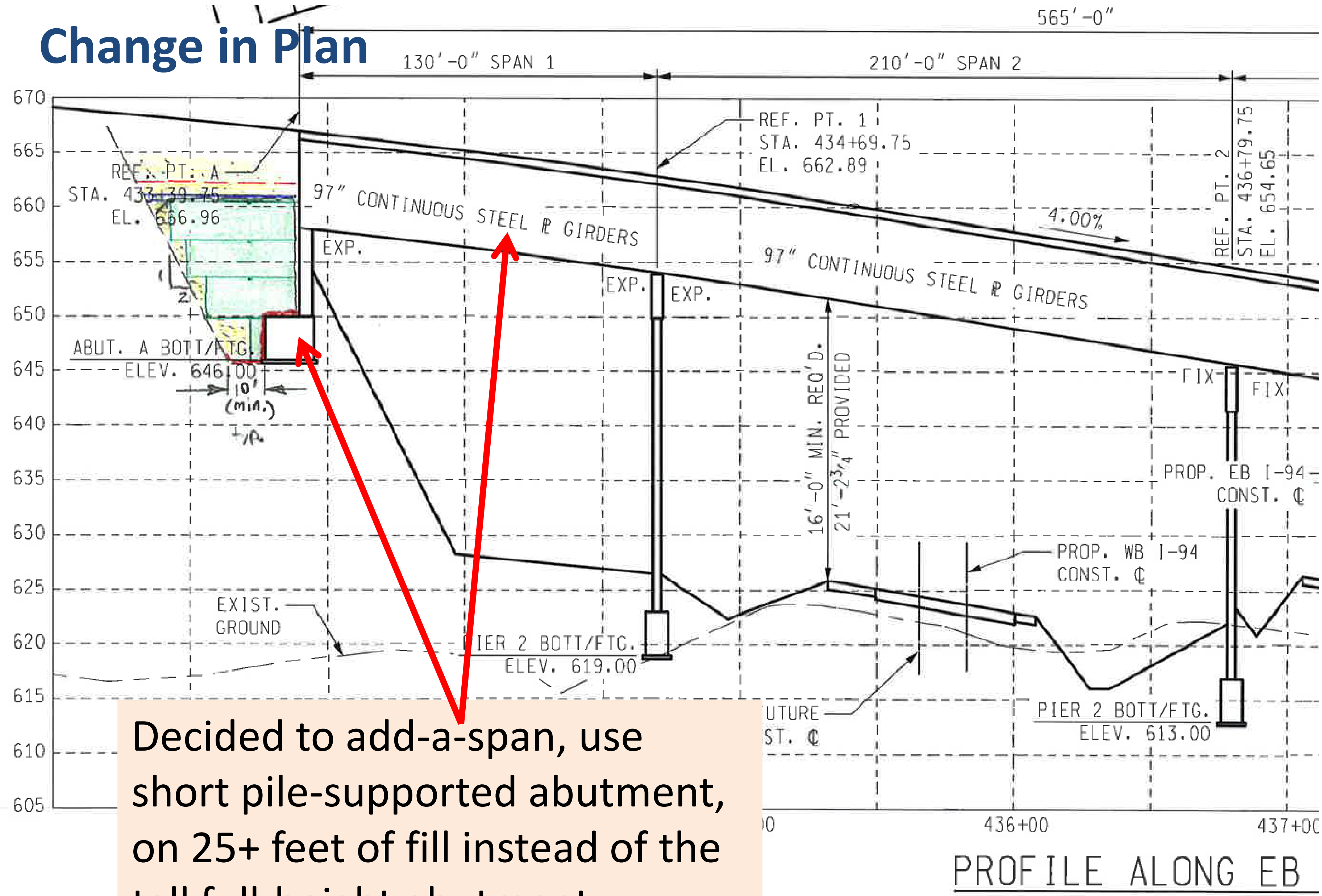


Abutment A

Set Grid Limits > 1.54 FS force

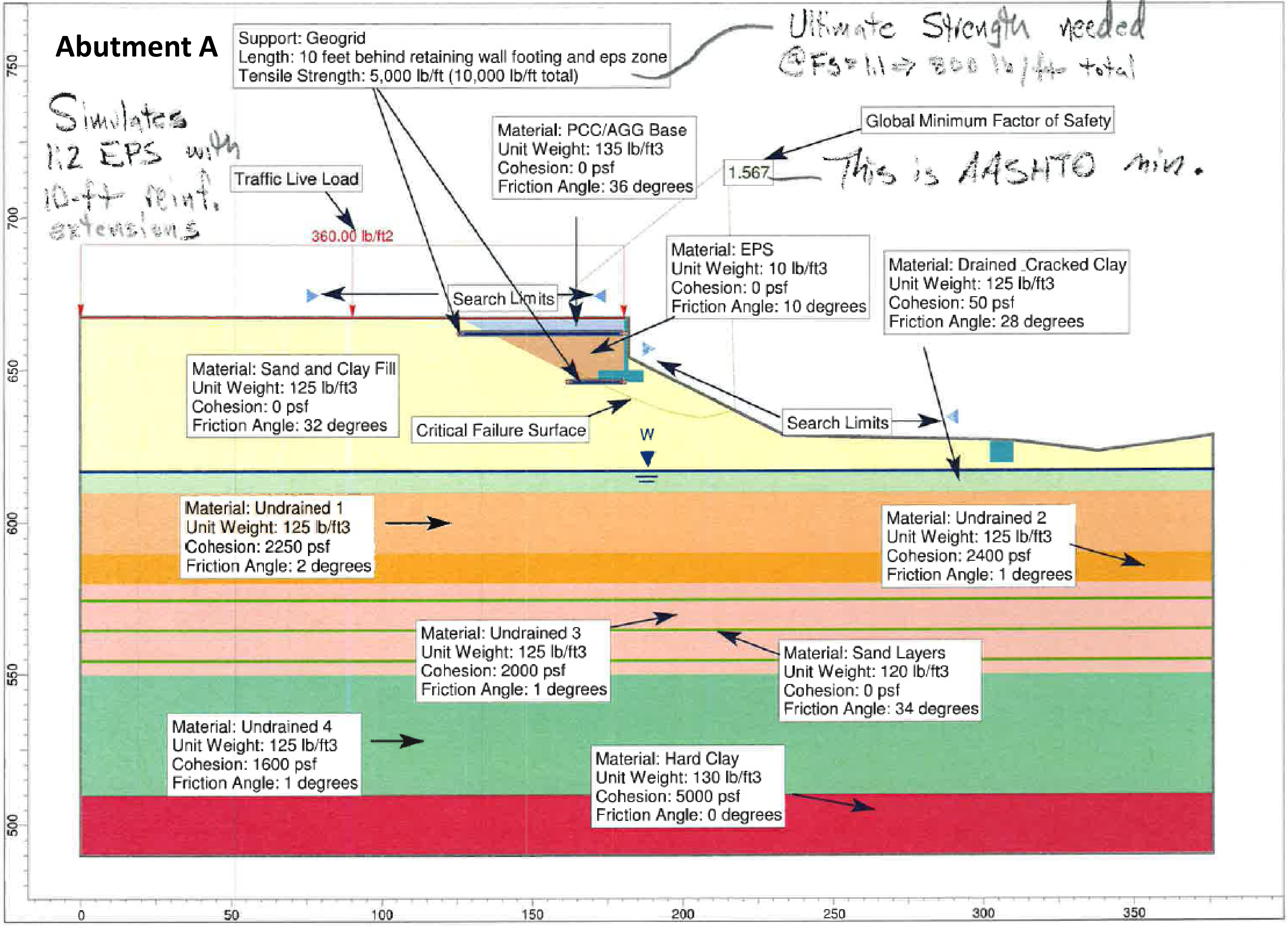


Change in Plan



Decided to add-a-span, use short pile-supported abutment, on 25+ feet of fill instead of the tall full-height abutment

Abutment A

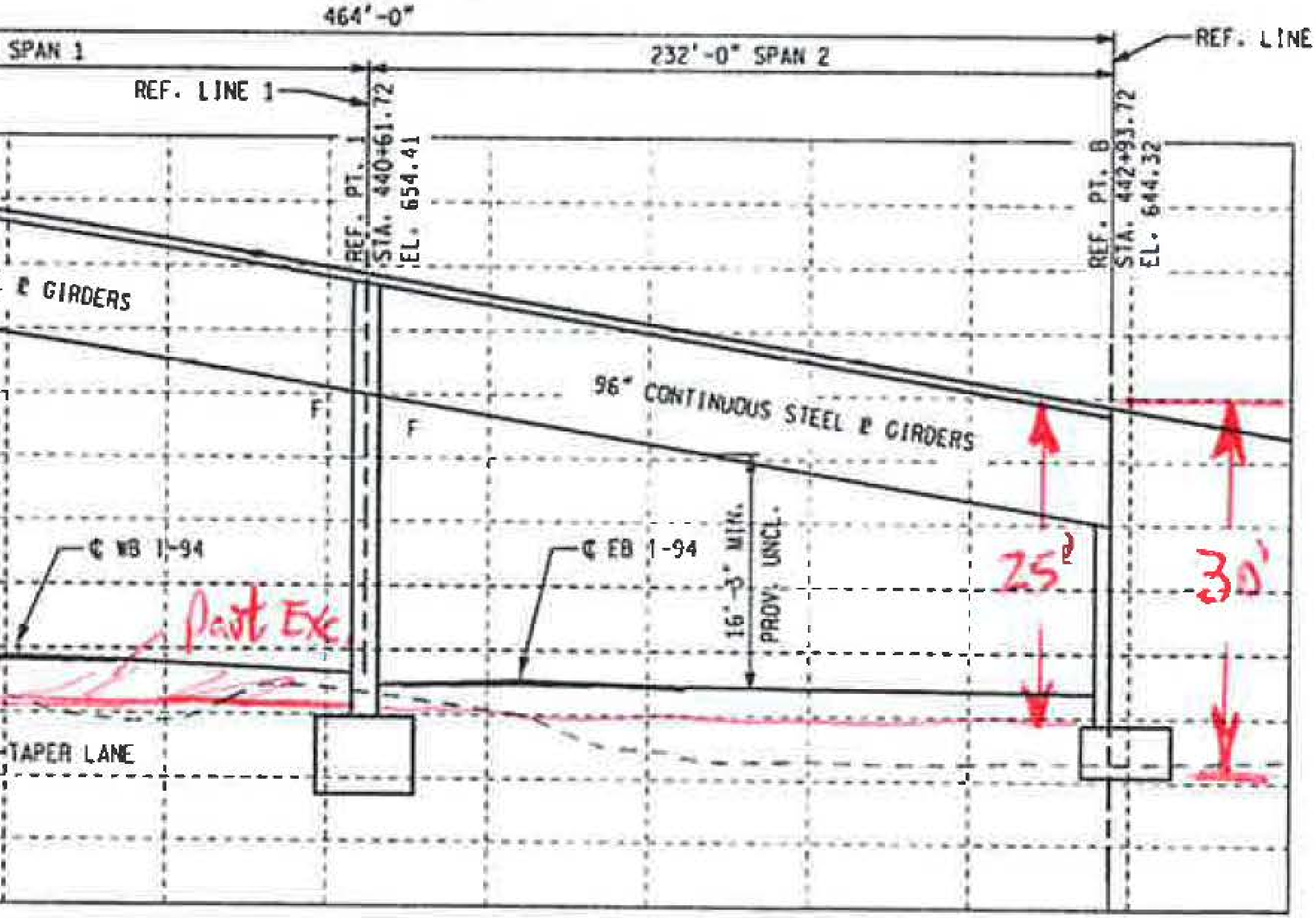


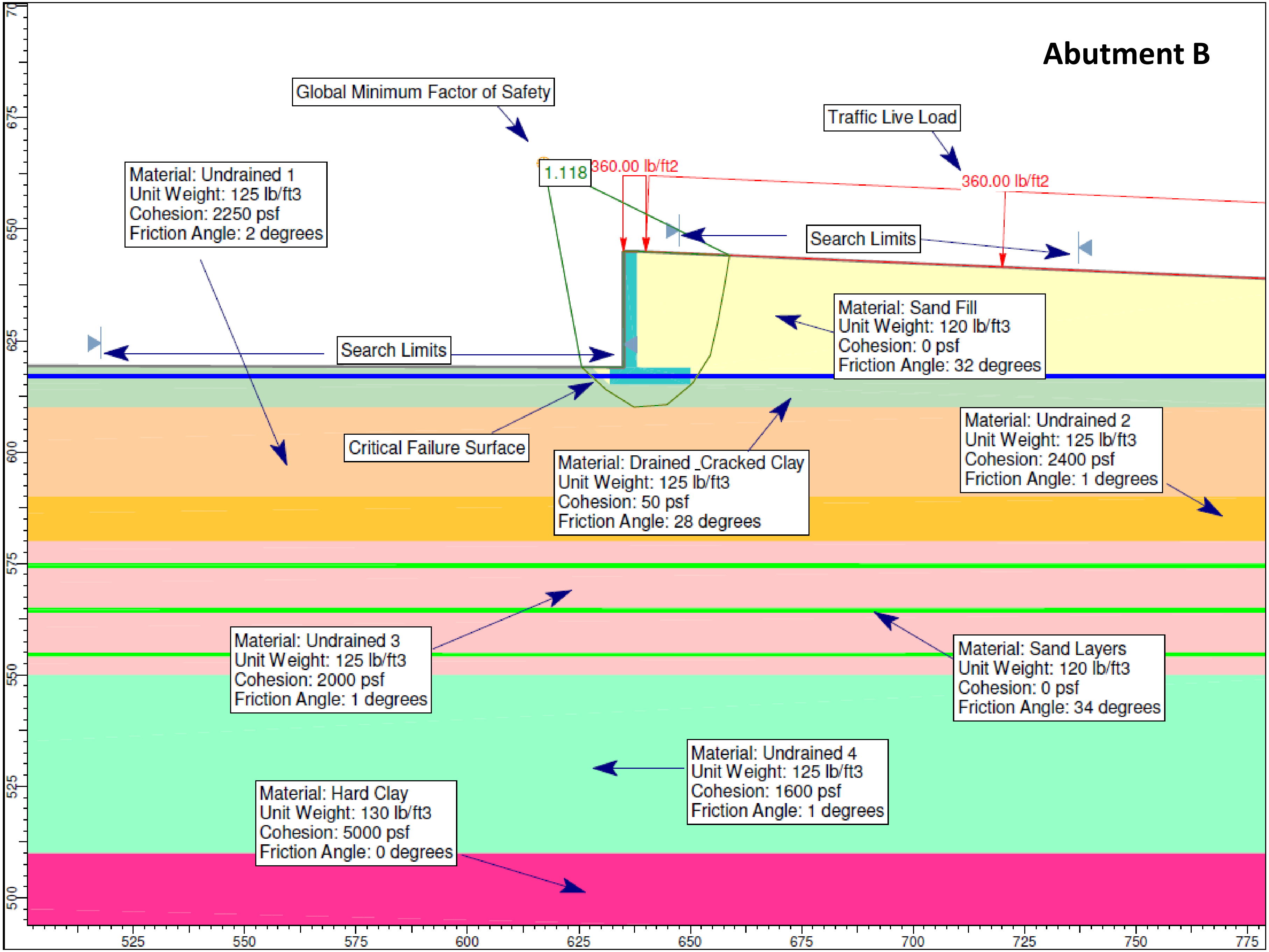
Ultimate Strength needed @FS=1.1 => 800 lb/ft total

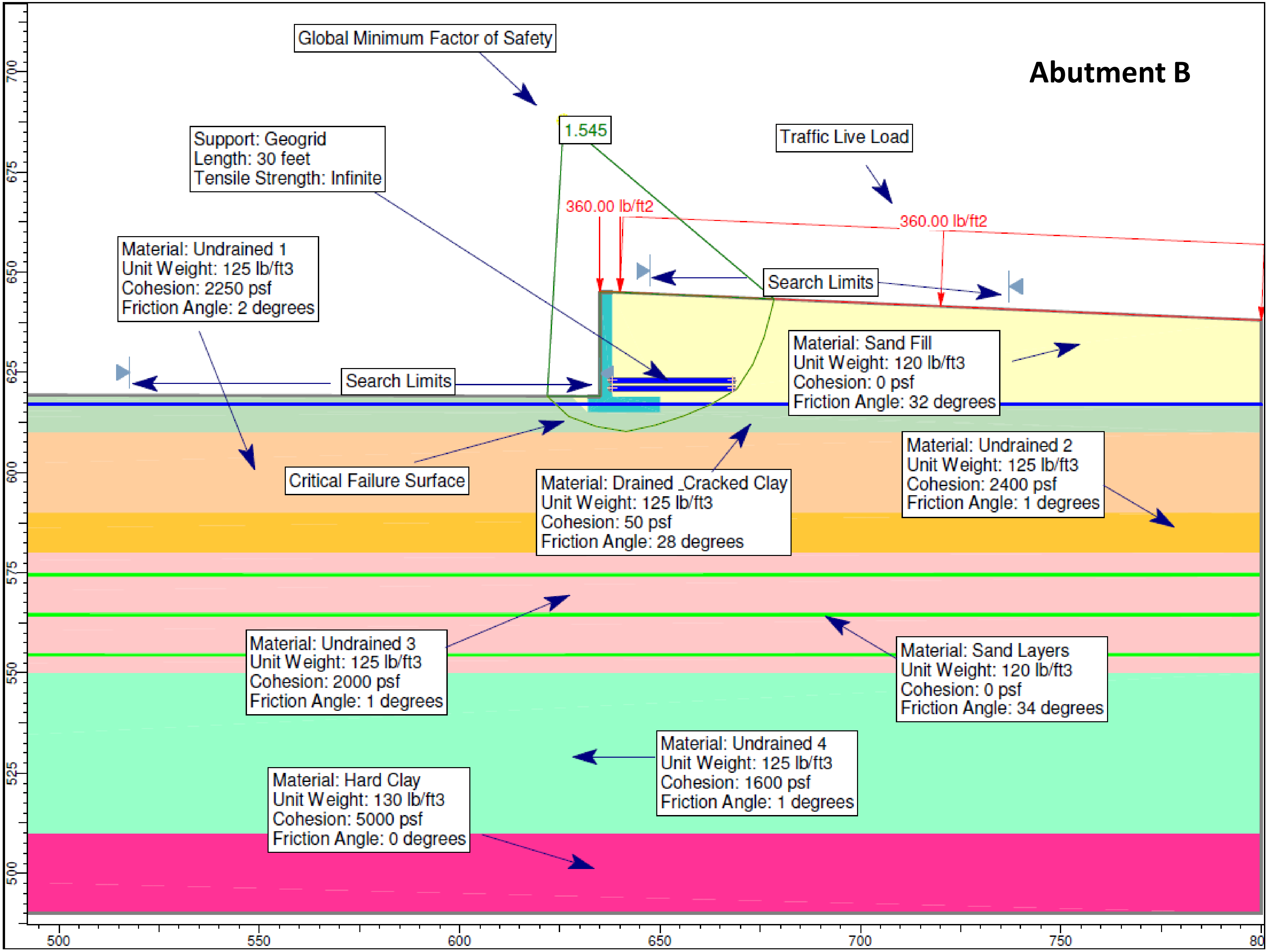
Simulates 1:2 EPS with 10-ft reinf. extensions

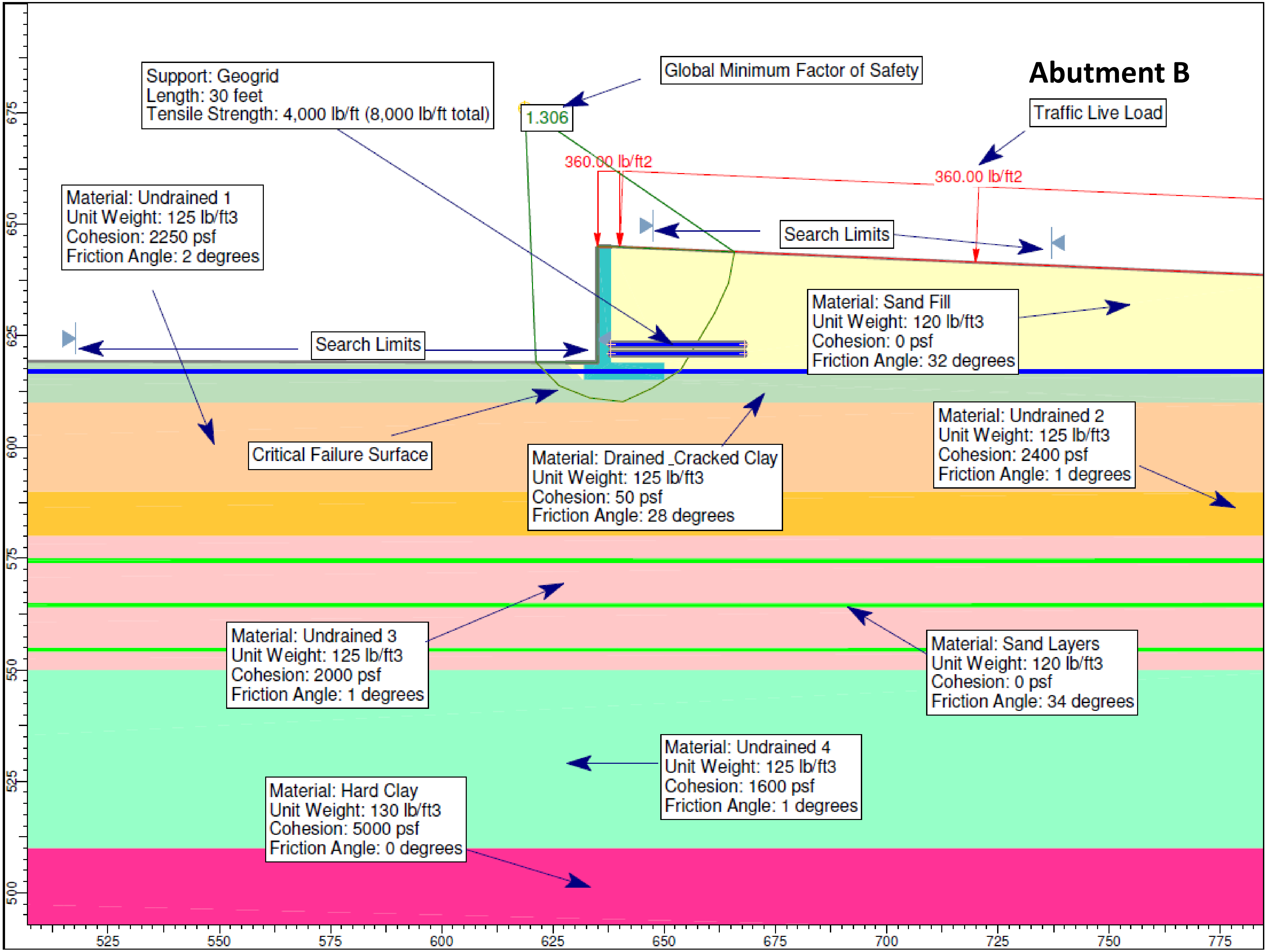
This is AASHTO min.

Abutment B


464'-0"
SPAN 1
232'-0" SPAN 2
REF. LINE
REF. LINE 1
REF. PT. 1
STA. 440+61.72
EL. 654.41
REF. PT. B
STA. 442+93.72
EL. 644.32
℄ GIRDERS
F
F
96" CONTINUOUS STEEL ℄ GIRDERS
℄ WB I-94
℄ EB I-94
16'-3" MIN.
PROV. UNCL.
Part Exc
25'
30'
TAPER LANE


Abutment B
Global Minimum Factor of Safety
Traffic Live Load
1.118
360.00 lb/ft2
360.00 lb/ft2
Material: Undrained 1
Unit Weight: 125 lb/ft3
Cohesion: 2250 psf
Friction Angle: 2 degrees
Search Limits
Search Limits
Material: Sand Fill
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 32 degrees
Critical Failure Surface
Material: Drained _Cracked Clay
Unit Weight: 125 lb/ft3
Cohesion: 50 psf
Friction Angle: 28 degrees
Material: Undrained 2
Unit Weight: 125 lb/ft3
Cohesion: 2400 psf
Friction Angle: 1 degrees
Material: Undrained 3
Unit Weight: 125 lb/ft3
Cohesion: 2000 psf
Friction Angle: 1 degrees
Material: Sand Layers
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 34 degrees
Material: Undrained 4
Unit Weight: 125 lb/ft3
Cohesion: 1600 psf
Friction Angle: 1 degrees
Material: Hard Clay
Unit Weight: 130 lb/ft3
Cohesion: 5000 psf
Friction Angle: 0 degrees
675
650
625
600
575
550
525
500
525
550
575
600
625
650
675
700
725
750
775


Abutment B
Global Minimum Factor of Safety
1.545
Support: Geogrid
Length: 30 feet
Tensile Strength: Infinite
Traffic Live Load
360.00 lb/ft2
360.00 lb/ft2
Material: Undrained 1
Unit Weight: 125 lb/ft3
Cohesion: 2250 psf
Friction Angle: 2 degrees
Search Limits
Search Limits
Material: Sand Fill
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 32 degrees
Critical Failure Surface
Material: Drained _Cracked Clay
Unit Weight: 125 lb/ft3
Cohesion: 50 psf
Friction Angle: 28 degrees
Material: Undrained 2
Unit Weight: 125 lb/ft3
Cohesion: 2400 psf
Friction Angle: 1 degrees
Material: Undrained 3
Unit Weight: 125 lb/ft3
Cohesion: 2000 psf
Friction Angle: 1 degrees
Material: Sand Layers
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 34 degrees
Material: Undrained 4
Unit Weight: 125 lb/ft3
Cohesion: 1600 psf
Friction Angle: 1 degrees
Material: Hard Clay
Unit Weight: 130 lb/ft3
Cohesion: 5000 psf
Friction Angle: 0 degrees
700
675
650
625
600
575
550
525
500
500
550
600
650
700
750
80


Abutment B
Global Minimum Factor of Safety
1.306
Traffic Live Load
360.00 lb/ft2
360.00 lb/ft2
Support: Geogrid
Length: 30 feet
Tensile Strength: 4,000 lb/ft (8,000 lb/ft total)
Material: Undrained 1
Unit Weight: 125 lb/ft3
Cohesion: 2250 psf
Friction Angle: 2 degrees
Search Limits
Search Limits
Material: Sand Fill
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 32 degrees
Critical Failure Surface
Material: Drained _Cracked Clay
Unit Weight: 125 lb/ft3
Cohesion: 50 psf
Friction Angle: 28 degrees
Material: Undrained 2
Unit Weight: 125 lb/ft3
Cohesion: 2400 psf
Friction Angle: 1 degrees
Material: Undrained 3
Unit Weight: 125 lb/ft3
Cohesion: 2000 psf
Friction Angle: 1 degrees
Material: Sand Layers
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 34 degrees
Material: Undrained 4
Unit Weight: 125 lb/ft3
Cohesion: 1600 psf
Friction Angle: 1 degrees
Material: Hard Clay
Unit Weight: 130 lb/ft3
Cohesion: 5000 psf
Friction Angle: 0 degrees
675
650
625
600
575
550
525
500
525
550
575
600
625
650
675
700
725
750
775

ARE SPREAD FOOTINGS OK?

- Bearing Capacity

$$q_{ult} = cN_c s_c b_c i_c + 0.5\gamma B N_\gamma s_\gamma b_\gamma i_\gamma + qN_q s_q b_q i_q$$

ARE SPREAD FOOTINGS OK?

- Bearing Capacity

$$q_{ult} = cN_c s_c b_c i_c + 0.5\gamma BN_\gamma s_\gamma b_\gamma i_\gamma + qN_q s_q b_q i_q$$

……Factored B.C. = approx. 5000 - 6000 psf

Approach Embankment Weight next to Abutment:

Abut. A = 48*125 = 6000 psf ➔ not likely!

Abut. B = 30 * 125 = 3750 psf ➔ maybe

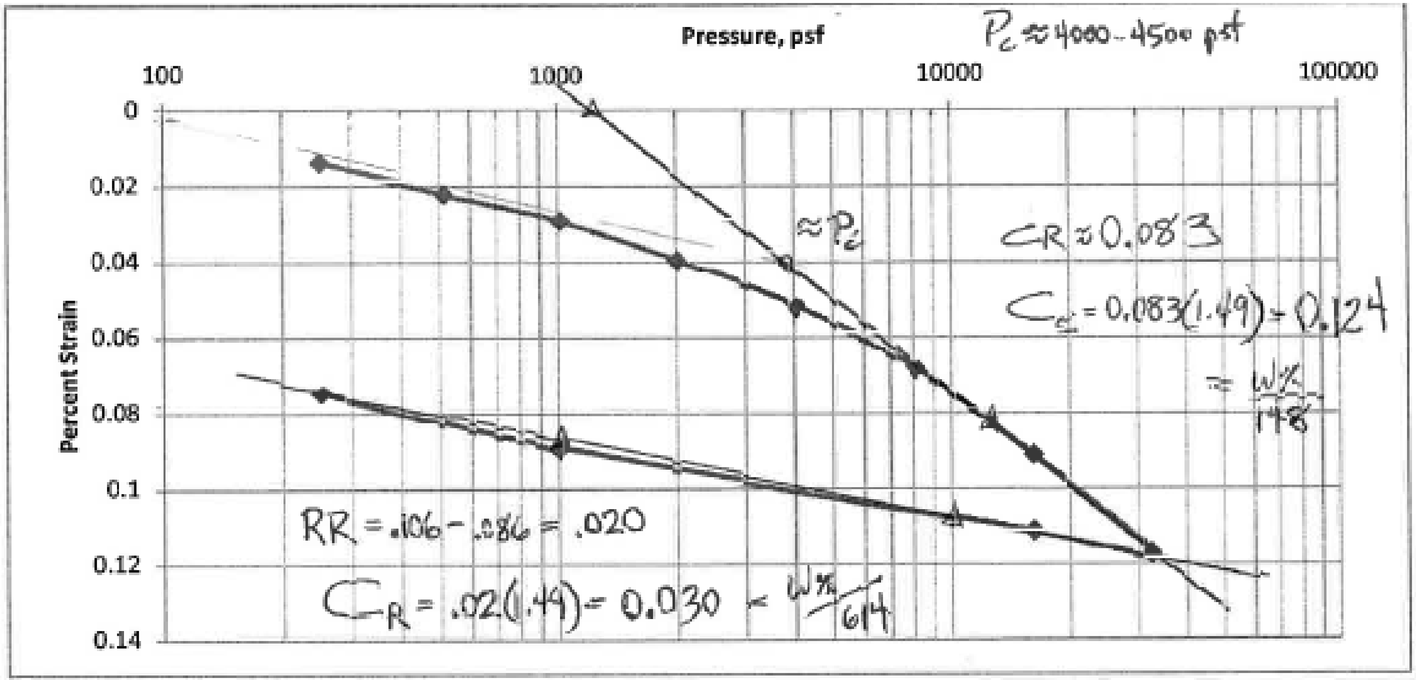
ARE SPREAD FOOTINGS OK?

- Settlement Management

$$S = \frac{C_S H}{1+e_o} \log\left(\frac{p_c}{p_o}\right) + \frac{C_c H}{1+e_o} \log\left(\frac{p_o + \Delta p}{p_c}\right)$$

- Need to estimate settlement of footings caused by approach embankments
- And Footing pressures causing settlement serviceability-limit (1-inch and 1.5-inch limits)
- Pre-loads? Lightweight Fills? Pile Downdrag?

SETTLEMENT MANAGEMENT



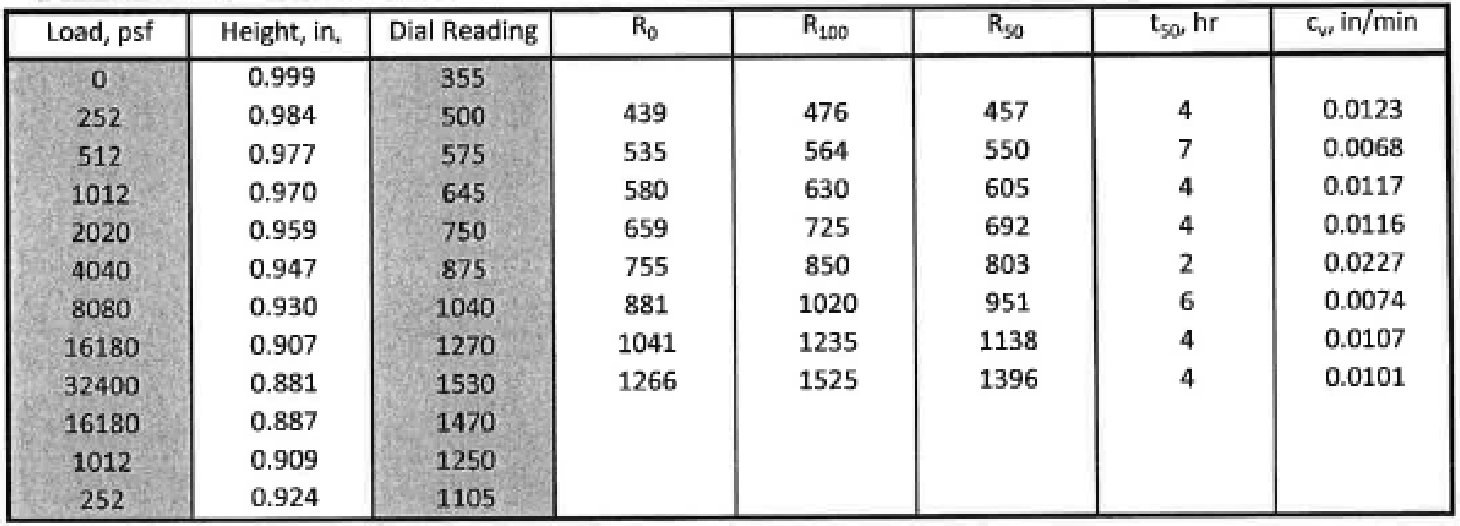
$P_c \approx 4000 - 4500$ psf

$CR \approx 0.083$

$C_c = 0.083(1.49) = 0.124 \approx \frac{W\%}{148}$

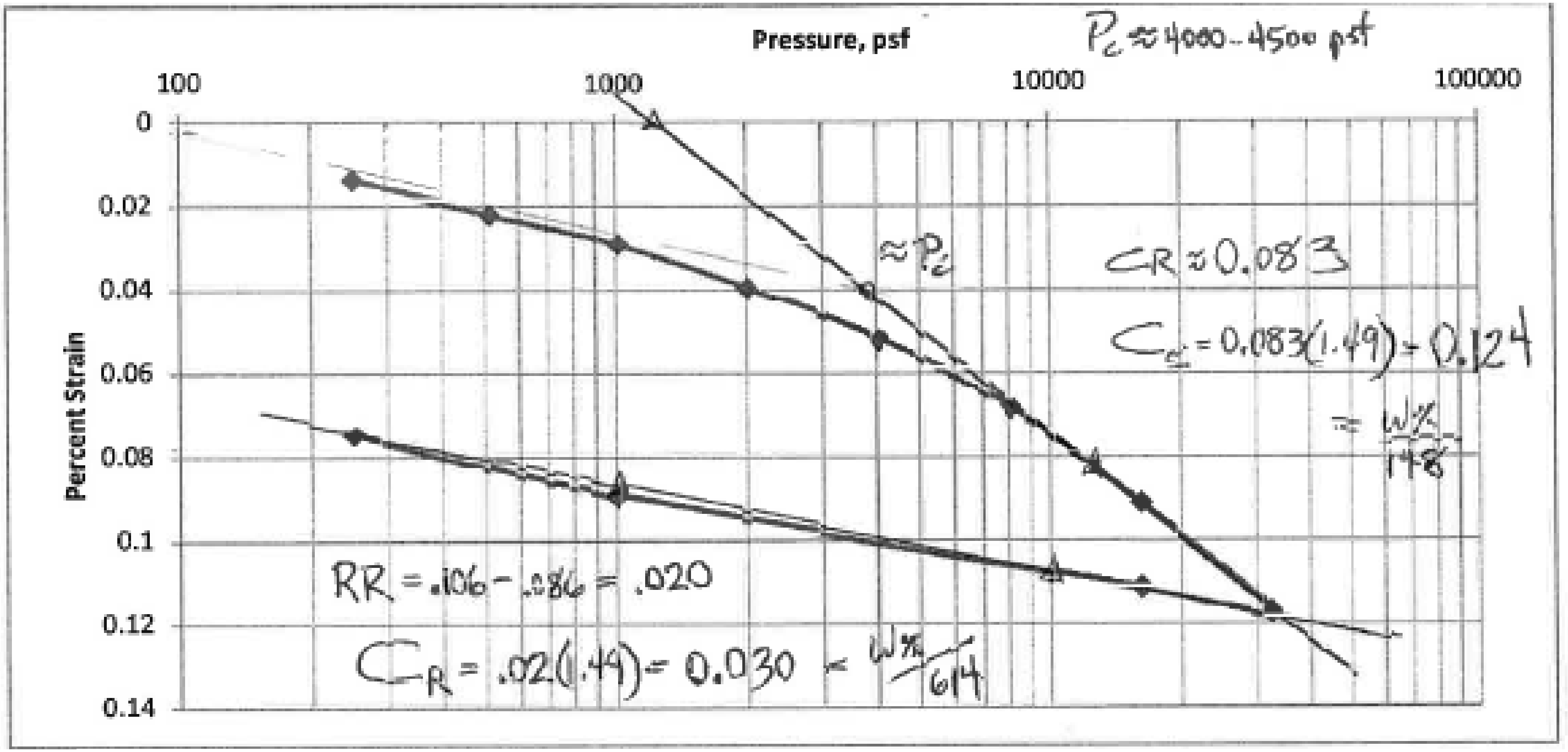
$RR = .106 - .086 = .020$

$C_R = .02(1.49) = 0.030 \approx \frac{W\%}{614}$



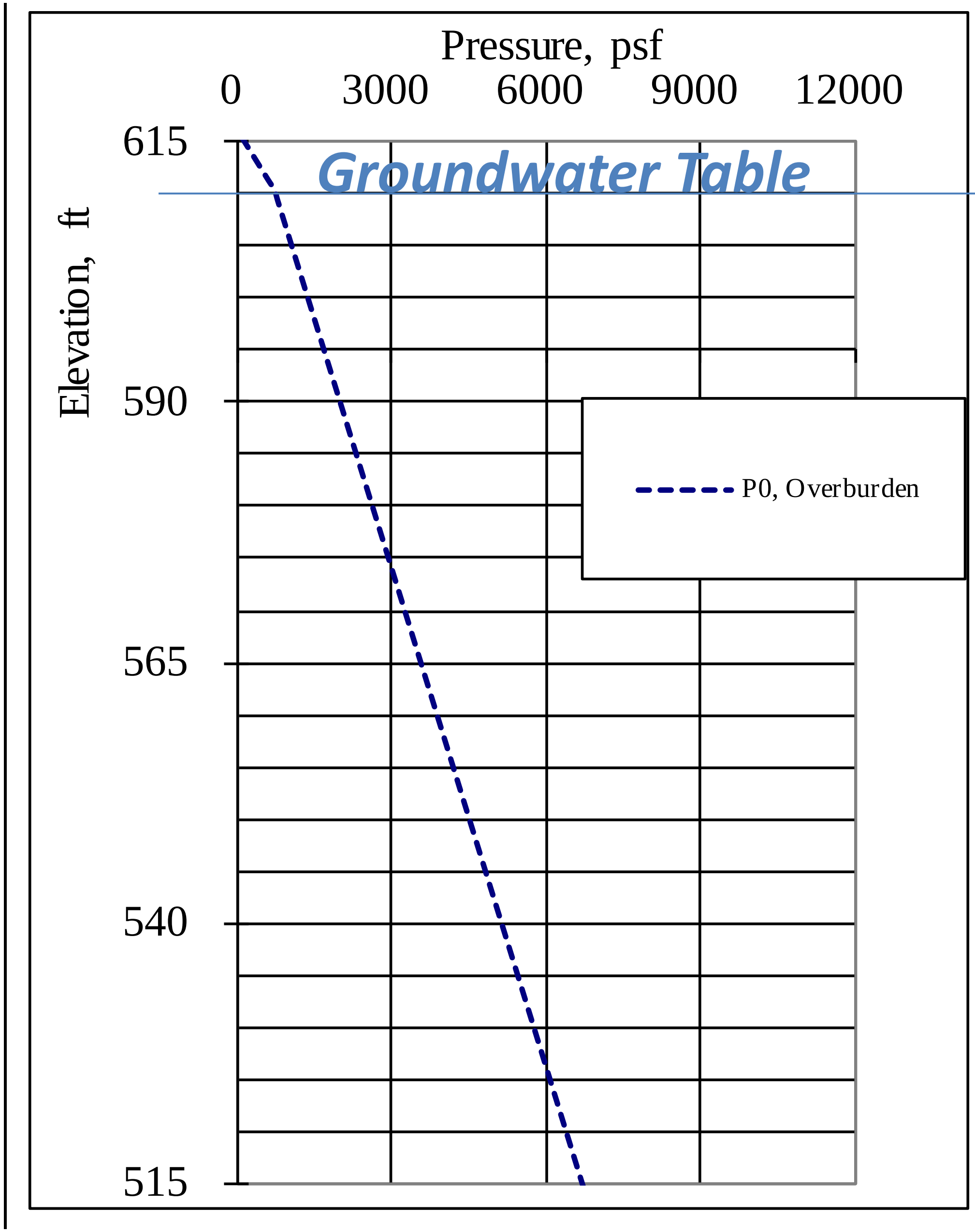
Load, psf	Height, in.	Dial Reading	R_0	R_{100}	R_{50}	t_{50}, hr	c_v, in/min
0	0.999	355					
252	0.984	500	439	476	457	4	0.0123
512	0.977	575	535	564	550	7	0.0068
1012	0.970	645	580	630	605	4	0.0117
2020	0.959	750	659	725	692	4	0.0116
4040	0.947	875	755	850	803	2	0.0227
8080	0.930	1040	881	1020	951	6	0.0074
16180	0.907	1270	1041	1235	1138	4	0.0107
32400	0.881	1530	1266	1525	1396	4	0.0101
16180	0.887	1470					
1012	0.909	1250					
252	0.924	1105					

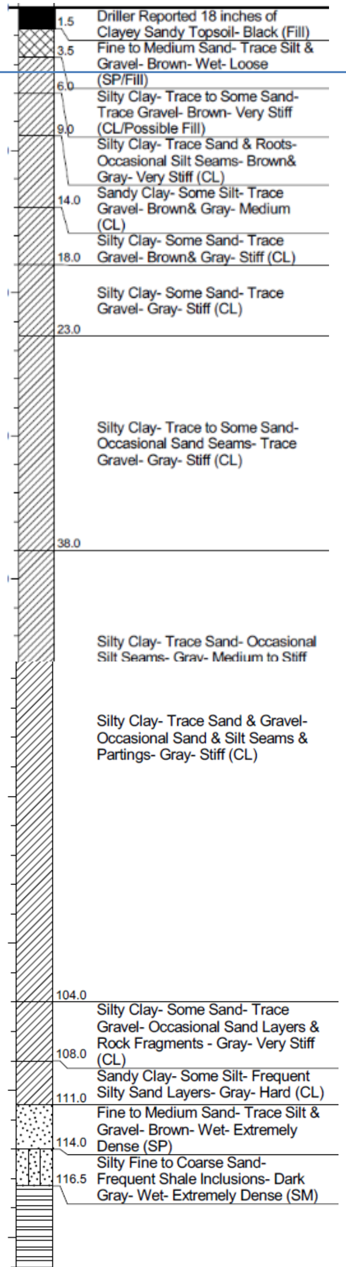
SETTLEMENT MANAGEMENT



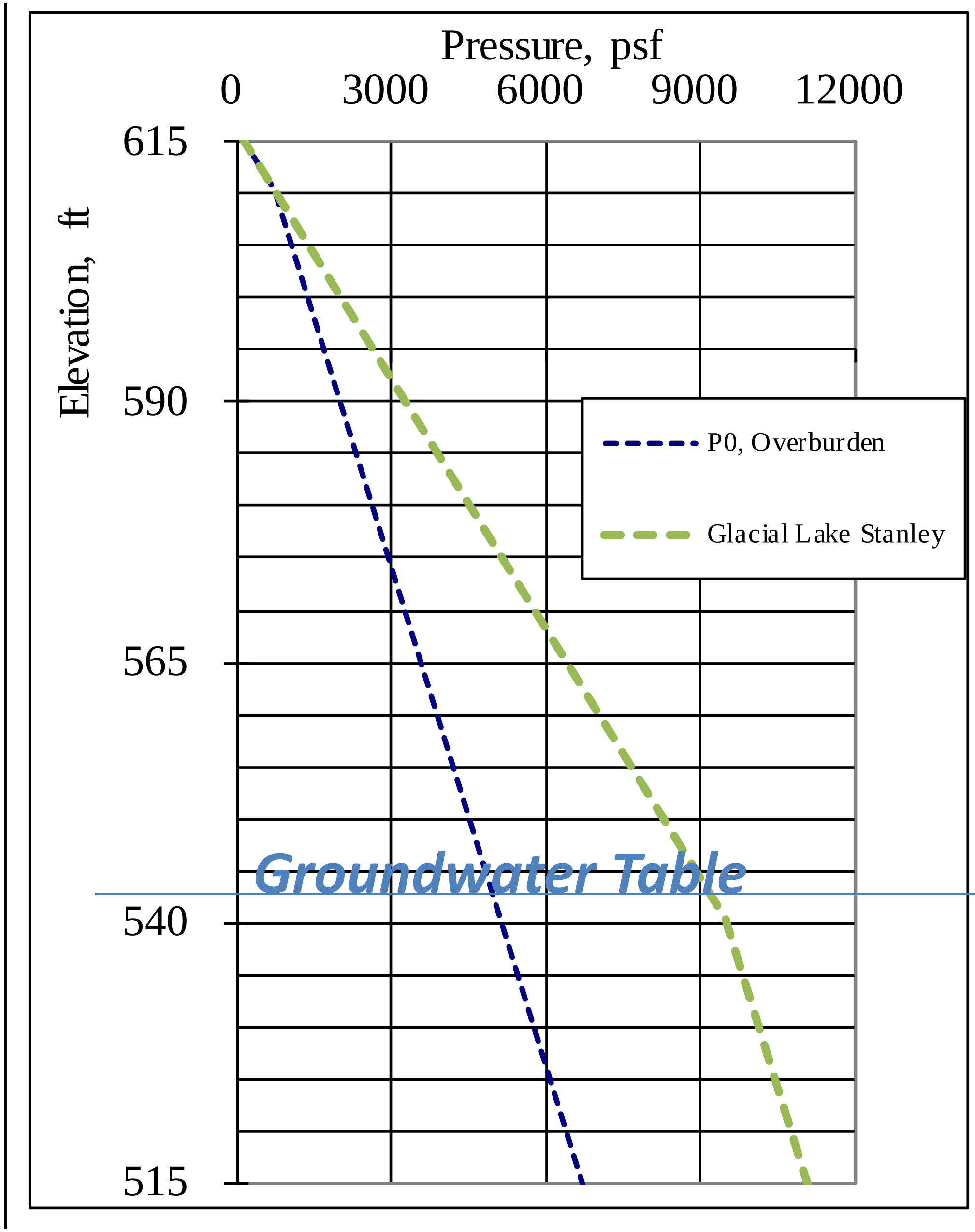
$$\delta_c = \frac{C_r}{1+e_0} H \log\left(\frac{p_c}{p_o}\right) + \frac{C_c}{1+e_0} H \log\left(\frac{p_o + \Delta p}{p_c}\right)$$

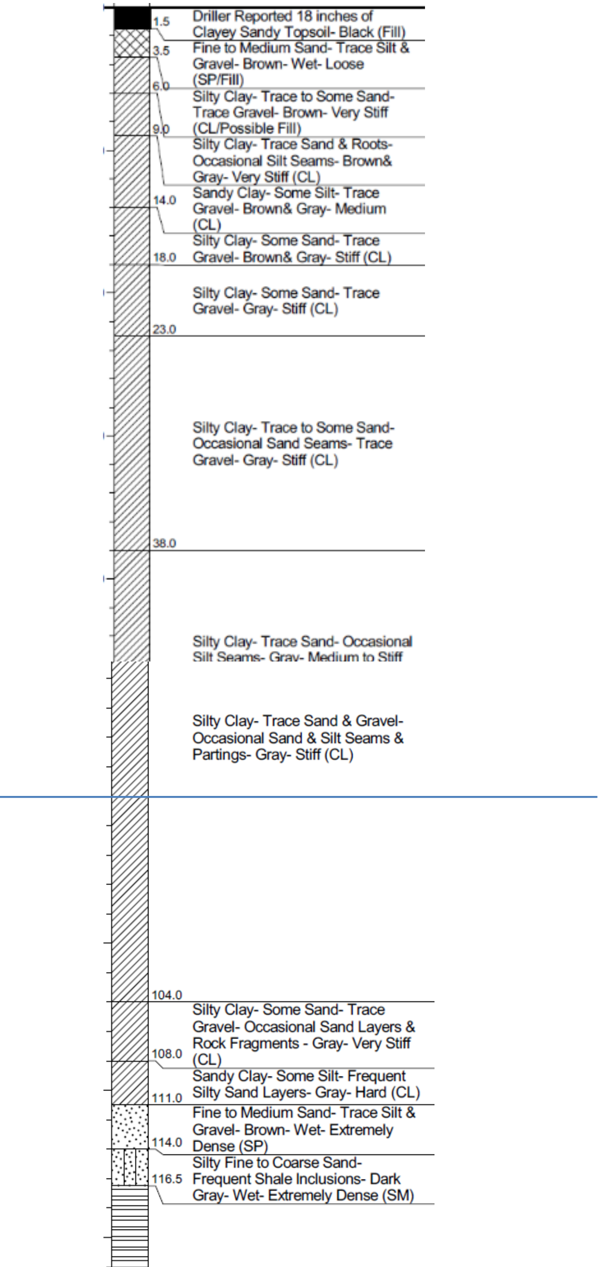
Today's Condition



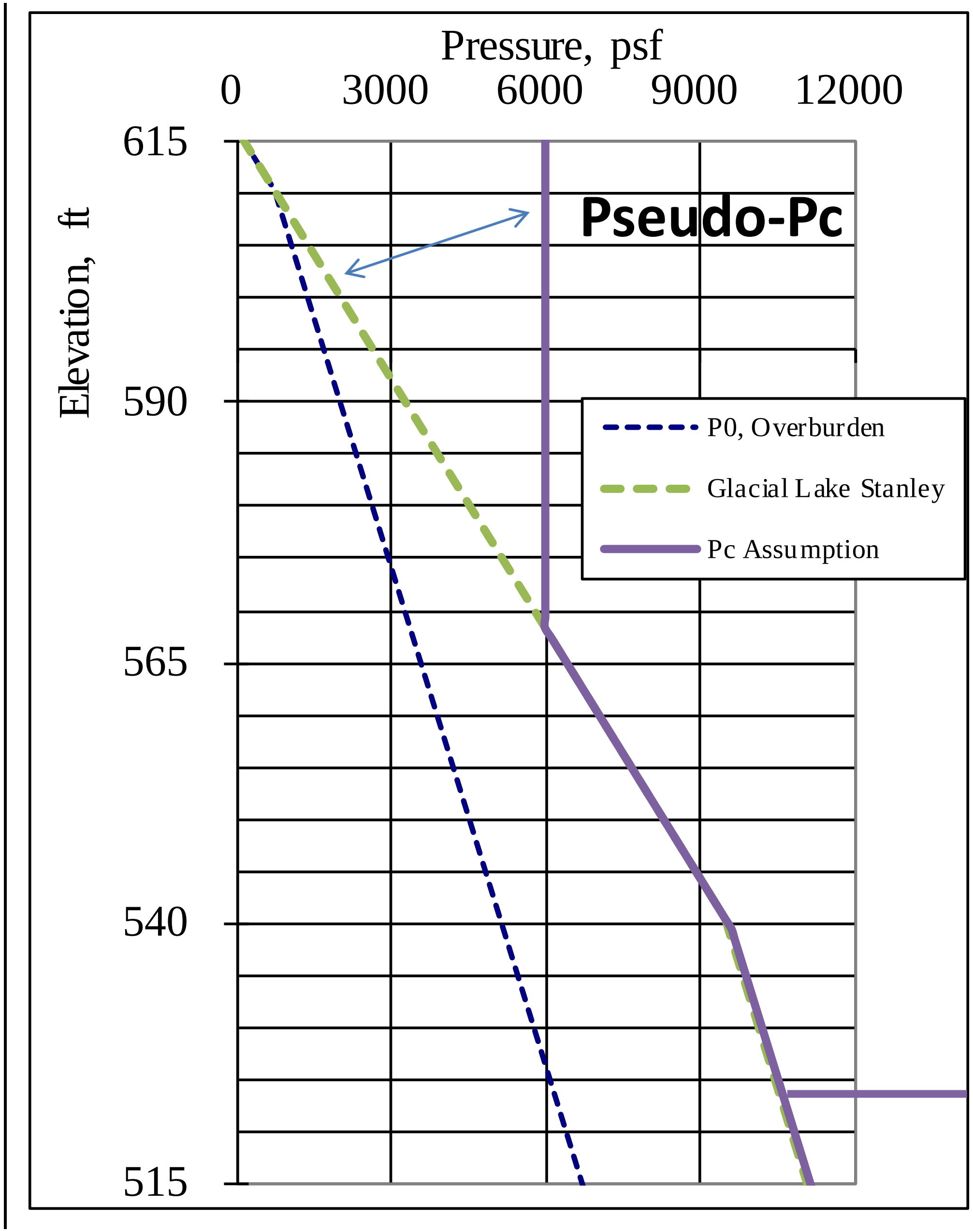


Lake Stanley Dry Period

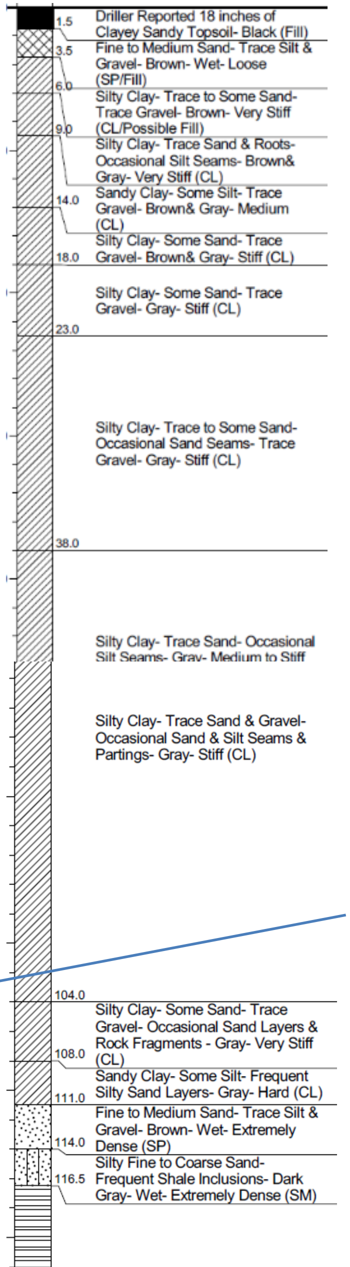




Preconsolidation

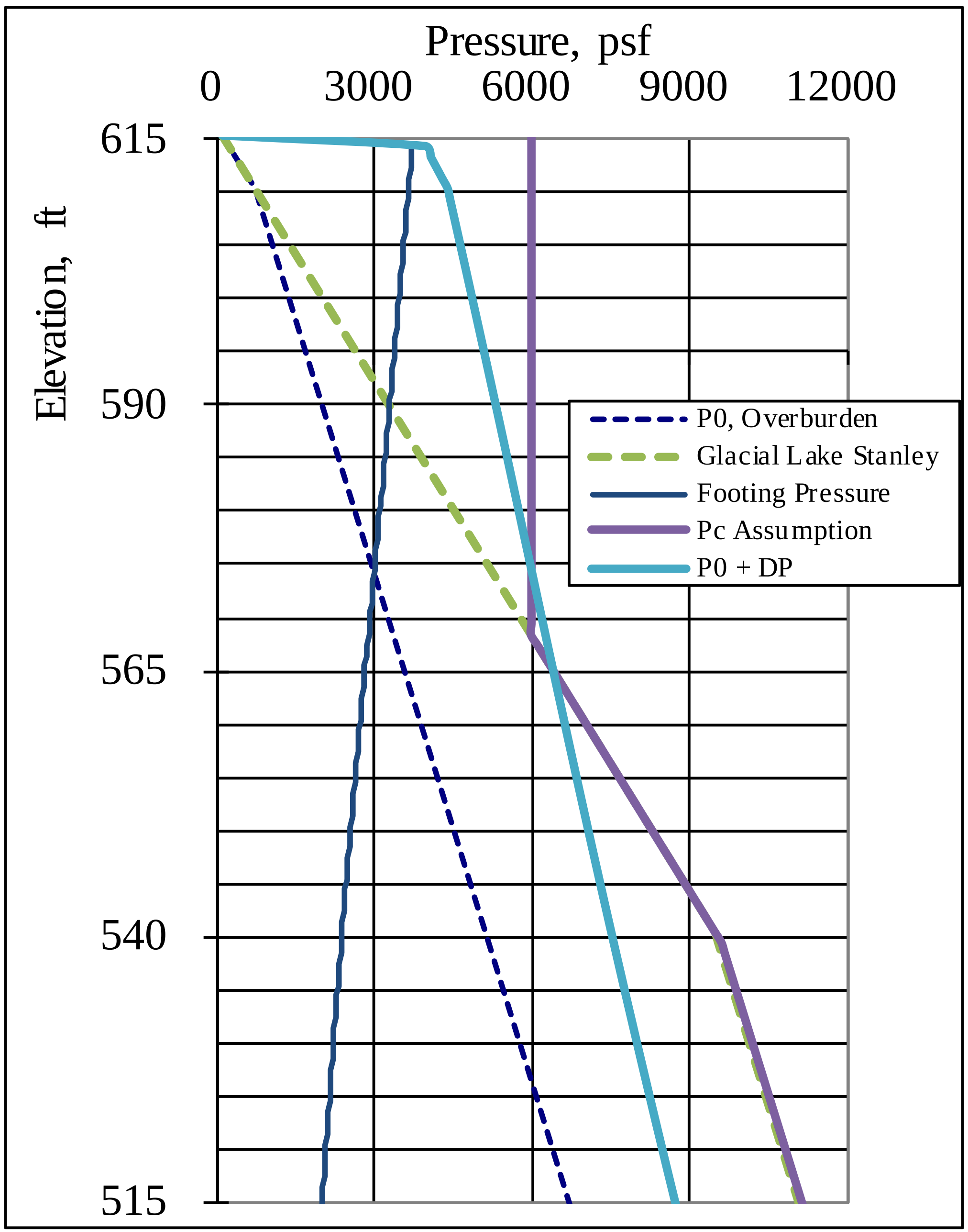


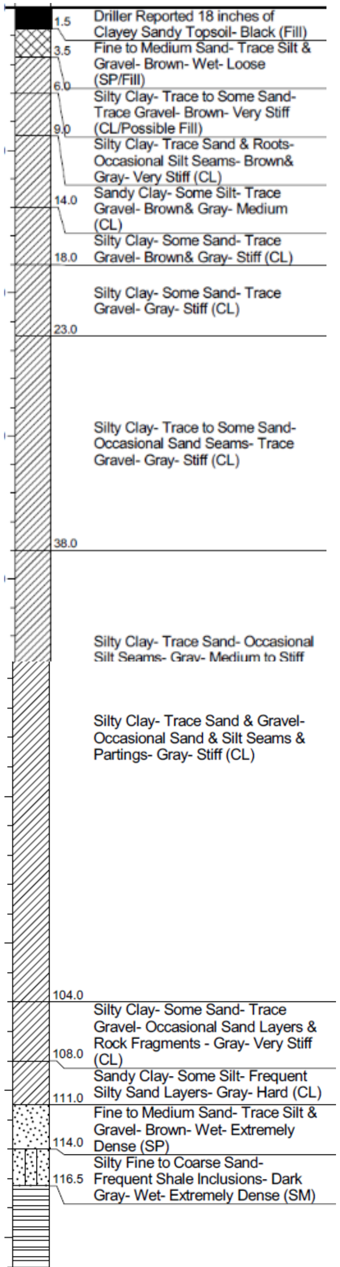
Pseudo-Pc

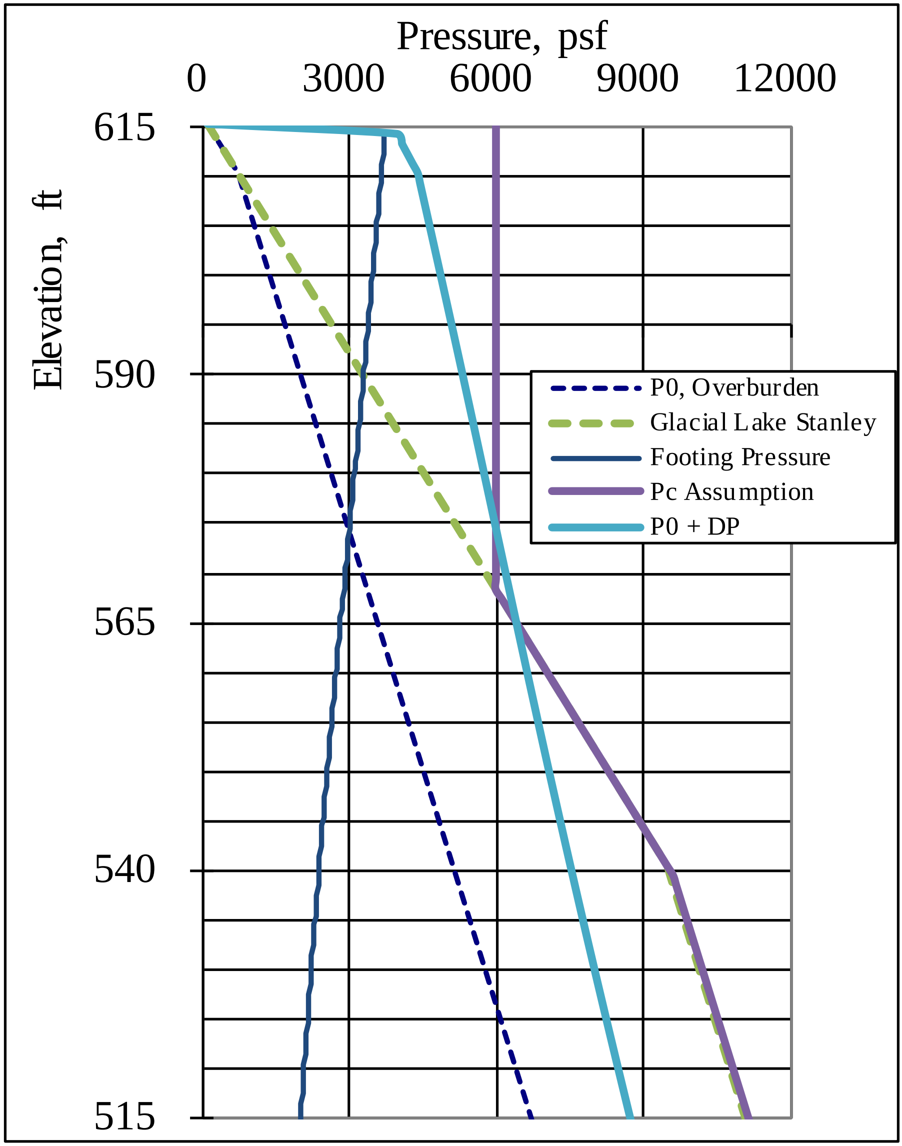


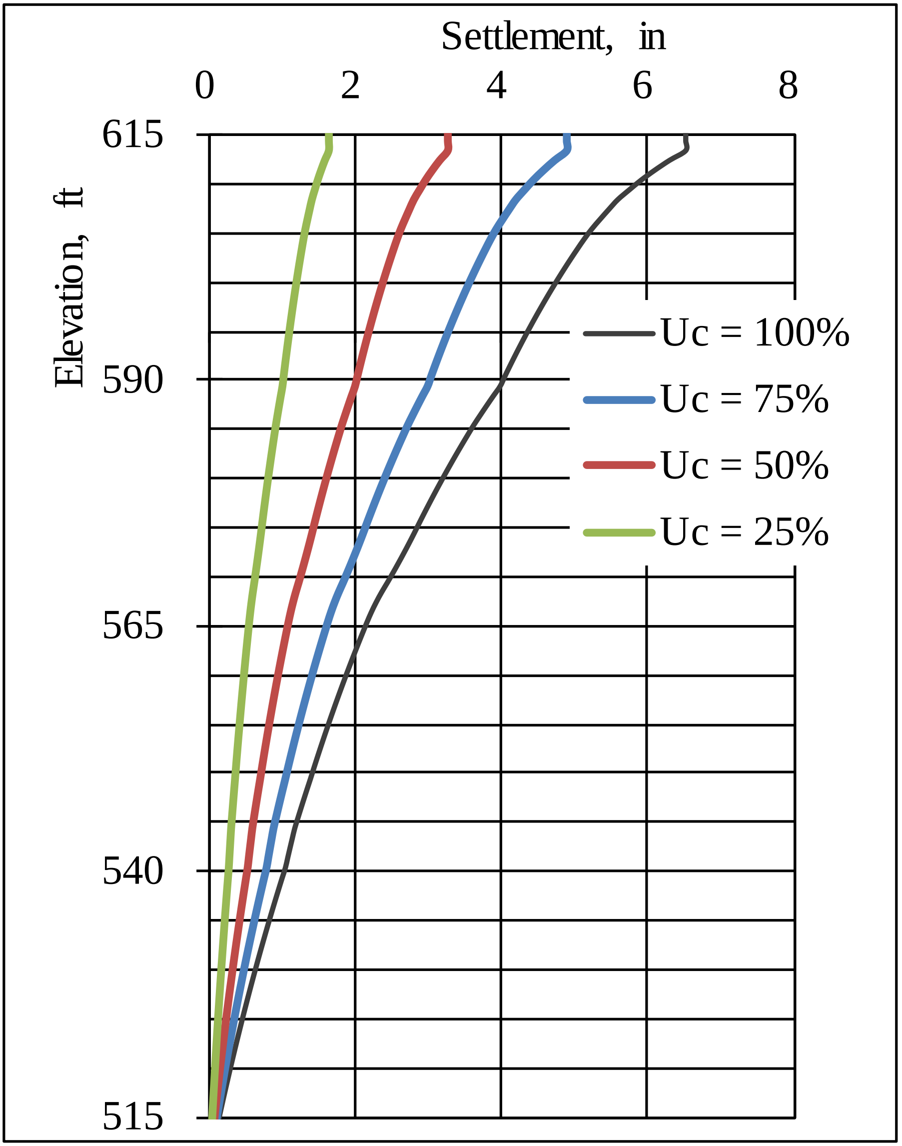
Ice Weight on Hard Till

Load Effects

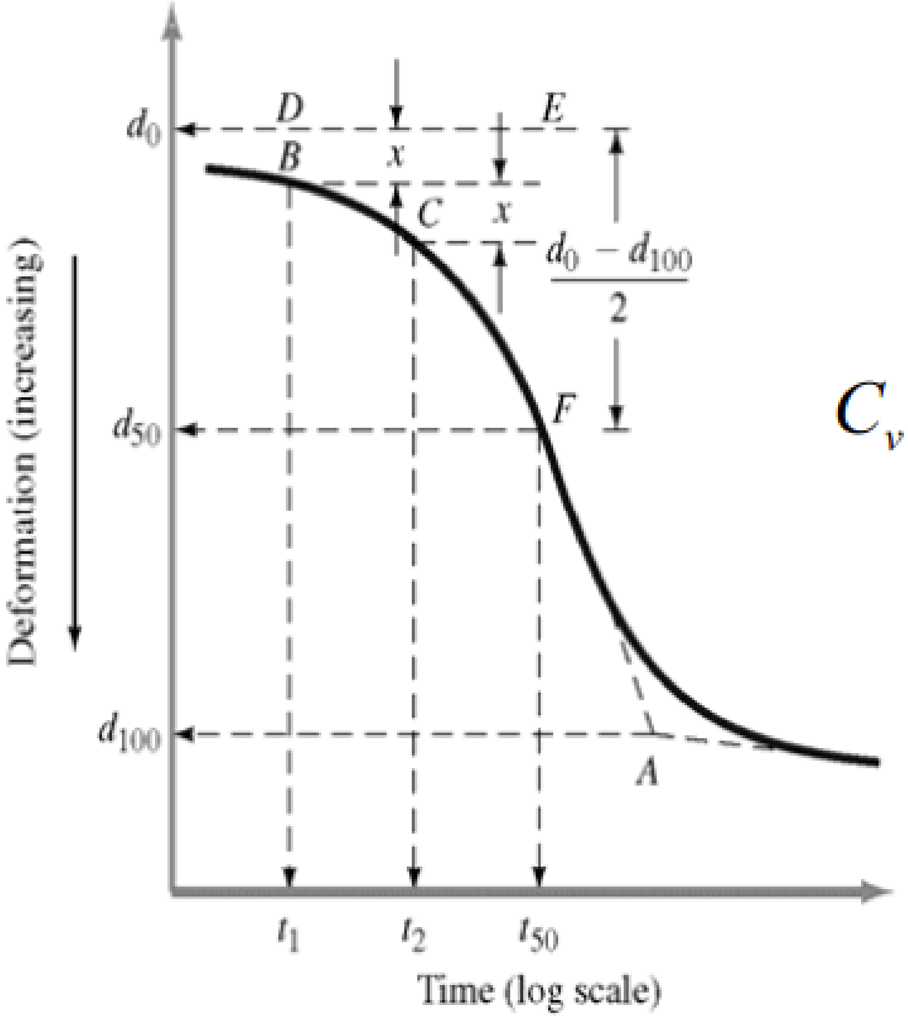





Pressure, psf
0
3000
6000
9000
12000
Elevation, ft
615
590
565
540
515
P0, Overburden
Glacial Lake Stanley
Footing Pressure
Pc Assumption
P0 + DP


Settlement, in
0
2
4
6
8
Elevation, ft
615
590
565
540
515
Uc = 100%
Uc = 75%
Uc = 50%
Uc = 25%

Determining C_v using Log Time Method

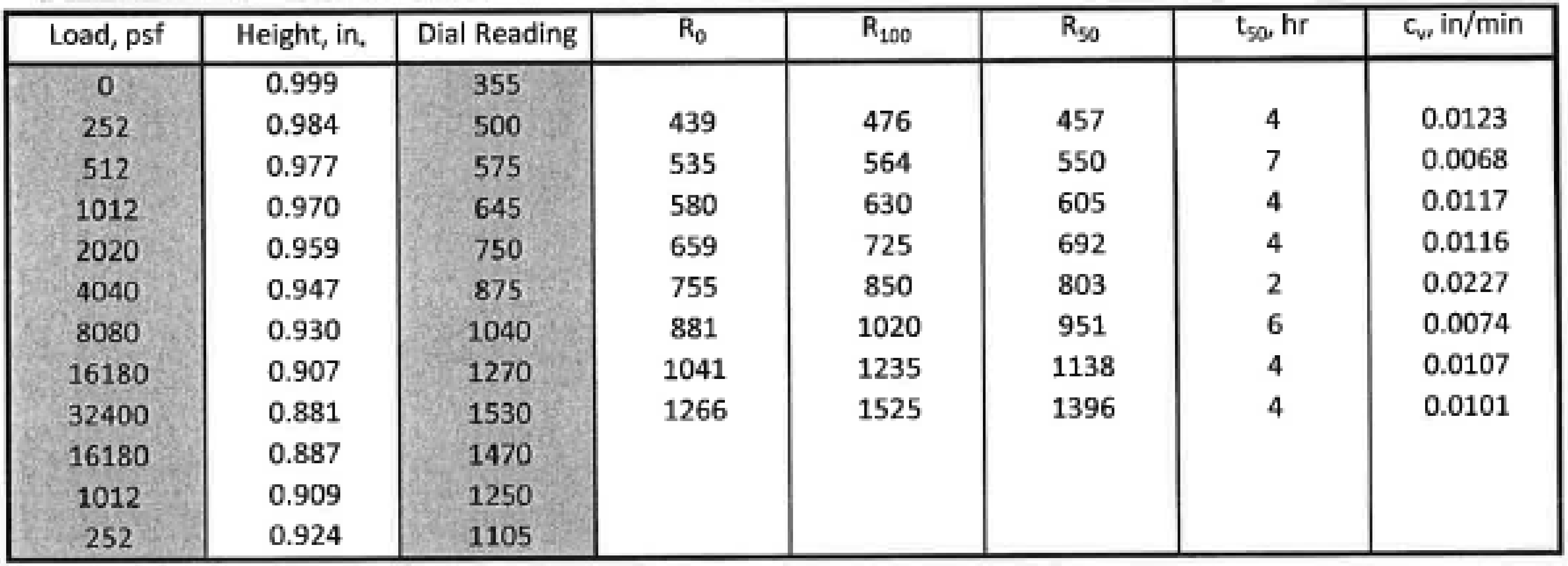


$$T_v = \frac{C_v t}{H^2_{dr}}$$

$$C_v = \frac{T_{V\,50} H^2_{dr}}{t_{50}} = \frac{0.196\left[\frac{H}{2}\right]^2}{t_{50}}$$

Figure 7.19
Logarithm-of-time method for determining coefficient of consolidation

SETTLEMENT MANAGEMENT



Load, psf	Height, in,	Dial Reading	R_0	R_{100}	R_{50}	t_{50}, hr	c_v, in/min
0	0.999	355					
252	0.984	500	439	476	457	4	0.0123
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1012	0.970	645	580	630	605	4	0.0117
2020	0.959	750	659	725	692	4	0.0116
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8080	0.930	1040	881	1020	951	6	0.0074
16180	0.907	1270	1041	1235	1138	4	0.0107
32400	0.881	1530	1266	1525	1396	4	0.0101
16180	0.887	1470					
1012	0.909	1250					
252	0.924	1105					

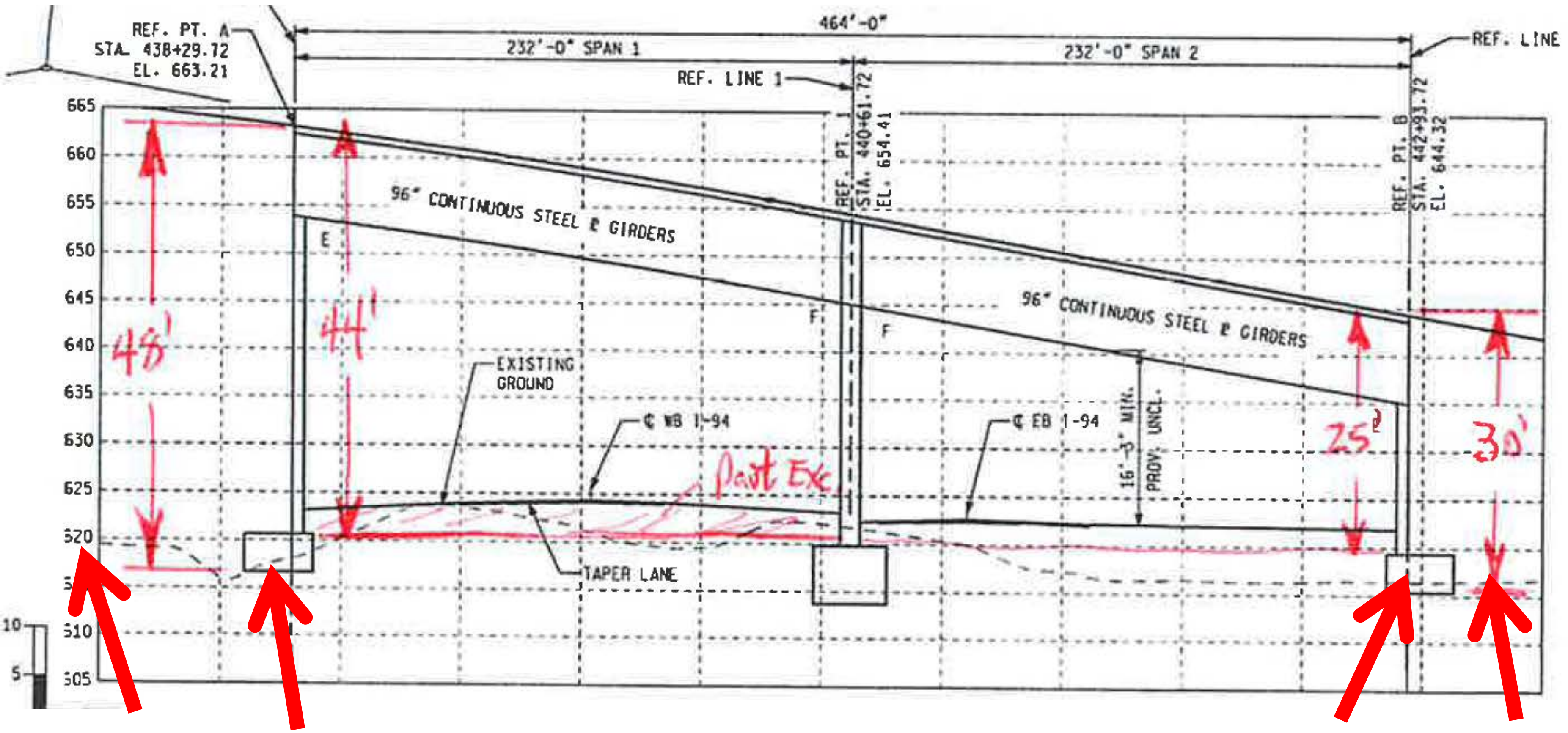
Wick Drains

Wick Drains


22'
21'

Wick Drains

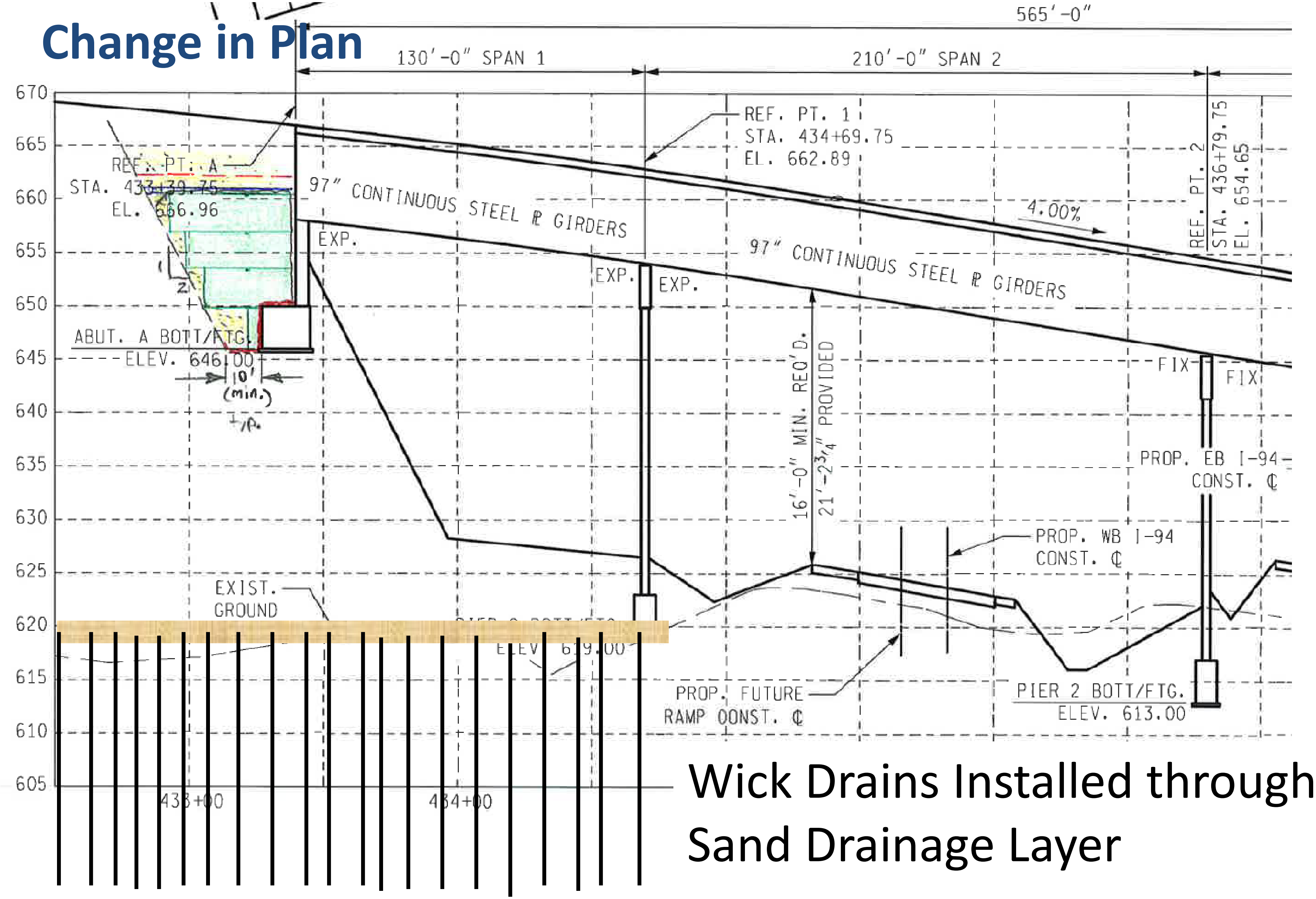
Example Bridge



14”   10”   (5-20 years w/o wicks)   6”   8”

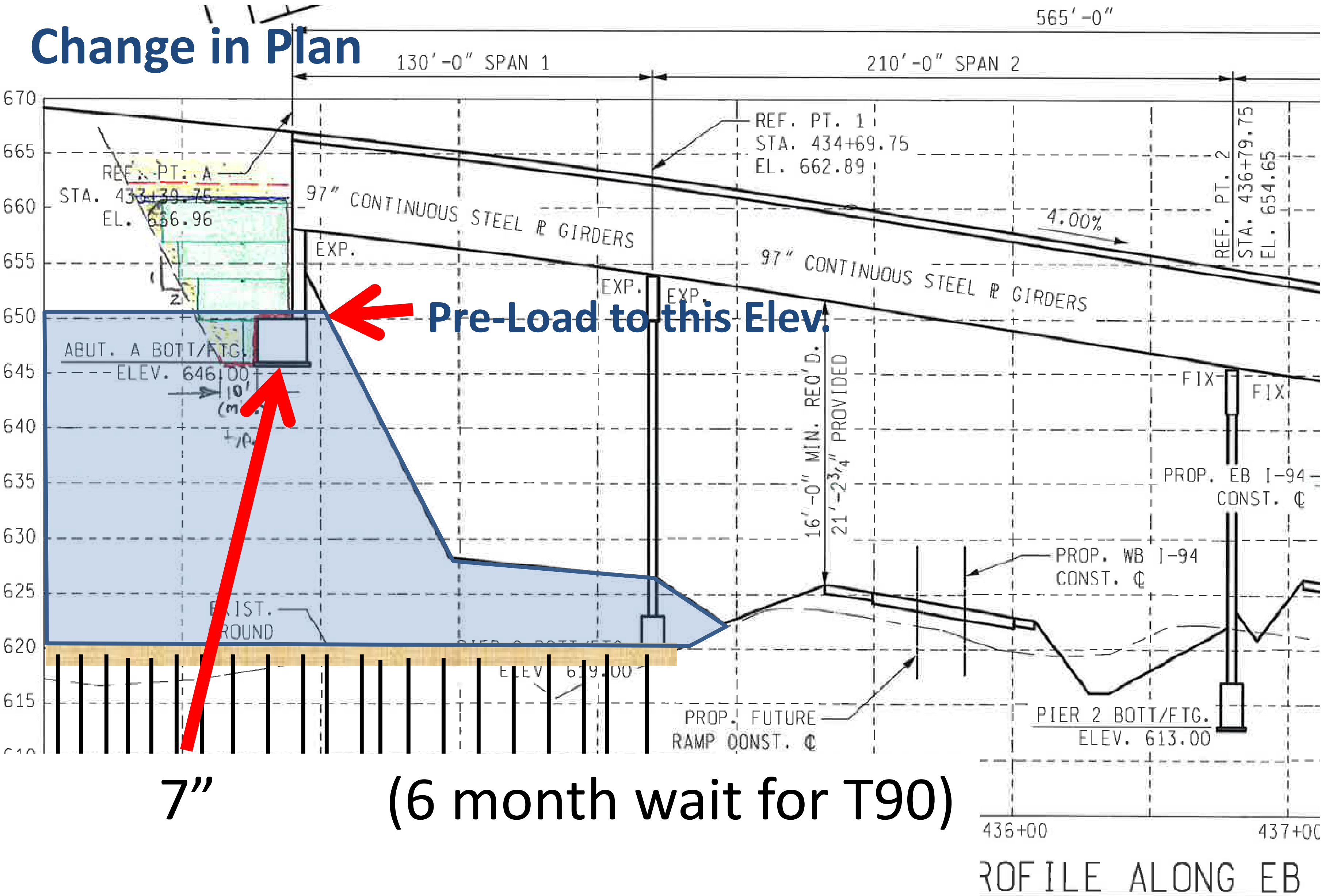
Settlement Estimates – Soil Only, no footing pressures

Change in Plan



Wick Drains Installed through Sand Drainage Layer

Change in Plan



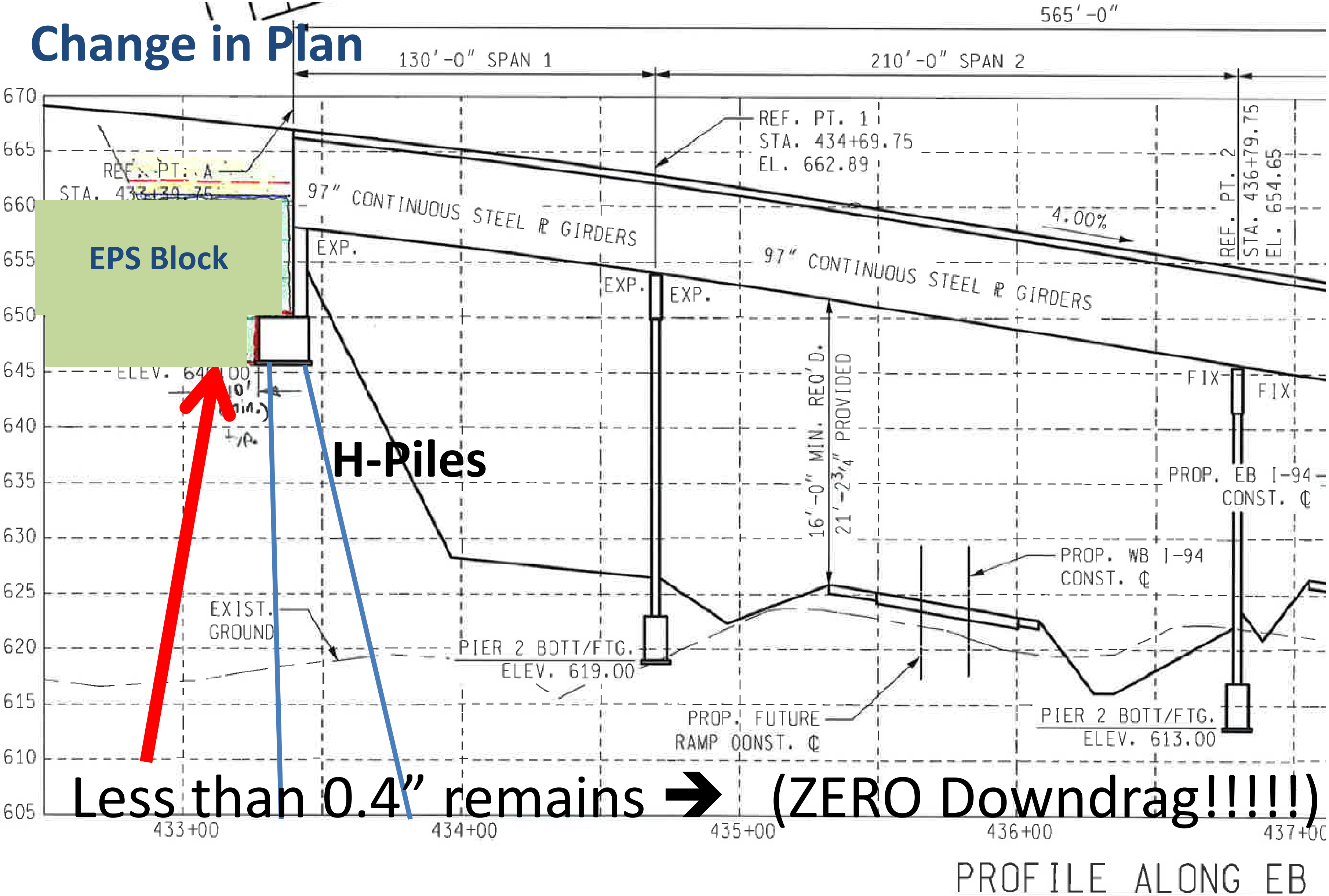
Pre-Load to this Elev.

7"

(6 month wait for T90)

Settlement Estimates

Change in Plan



EPS Block

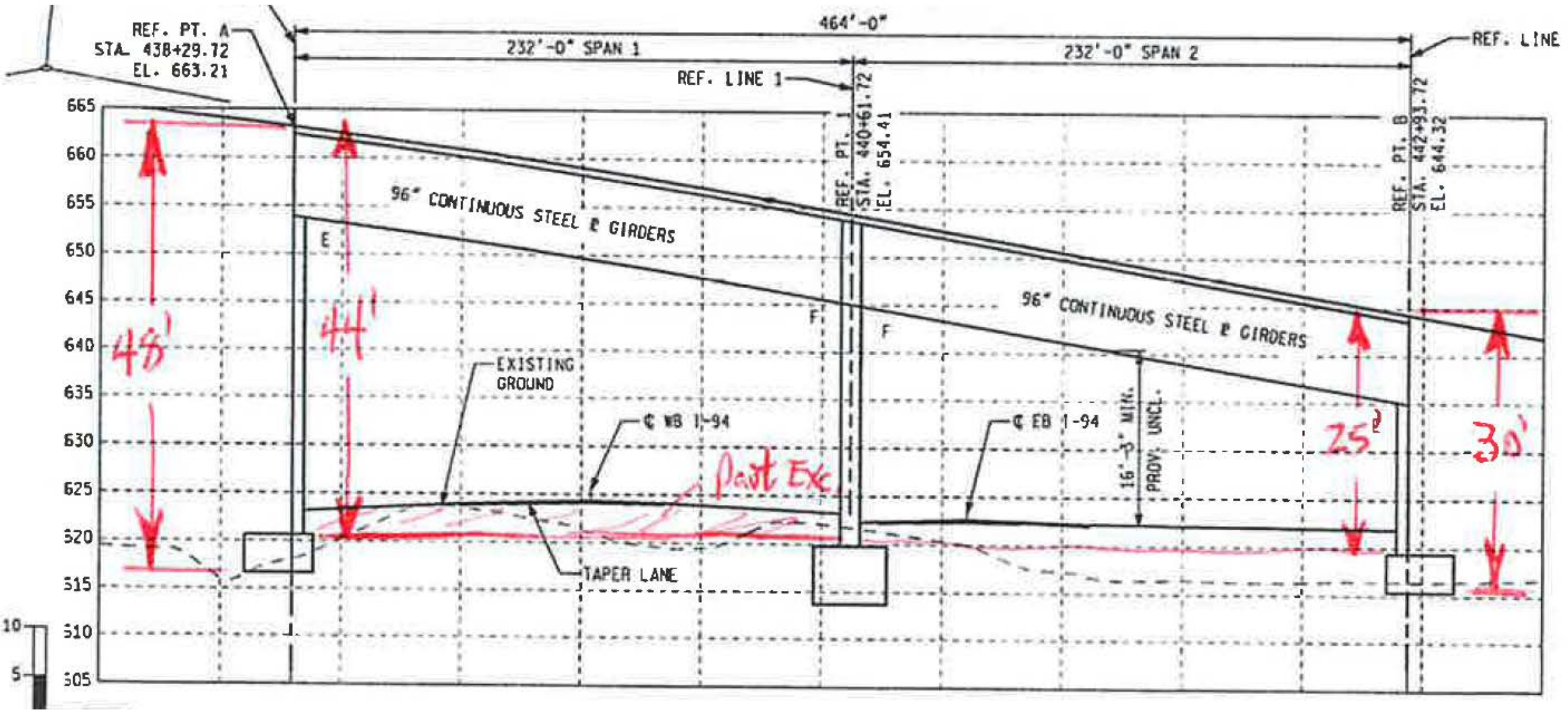
H-Piles

Less than 0.4" remains → (ZERO Downdrag!!!!!)

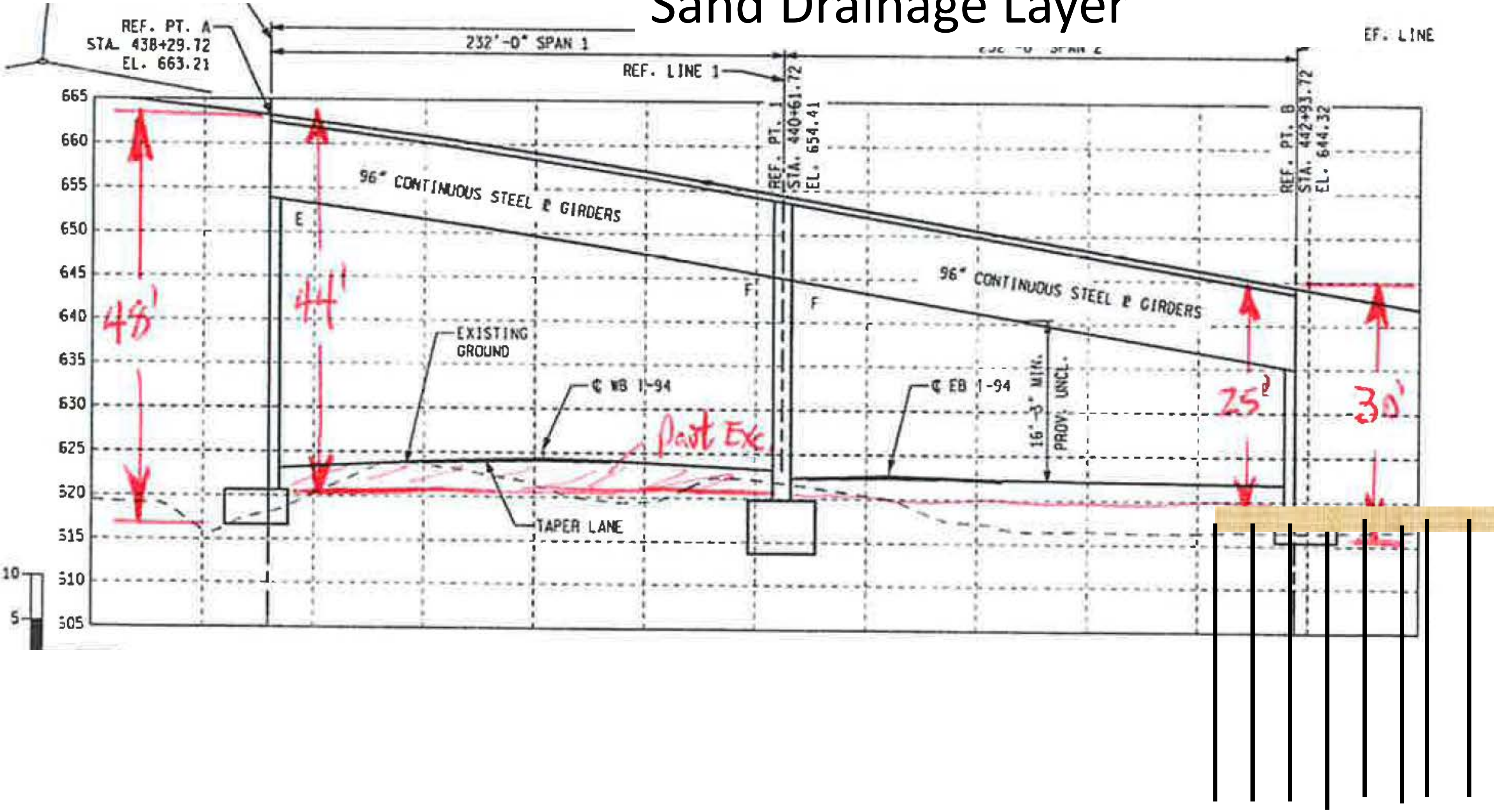
Settlement Estimates

Placement of EPS and Geogrid behind sheeting.

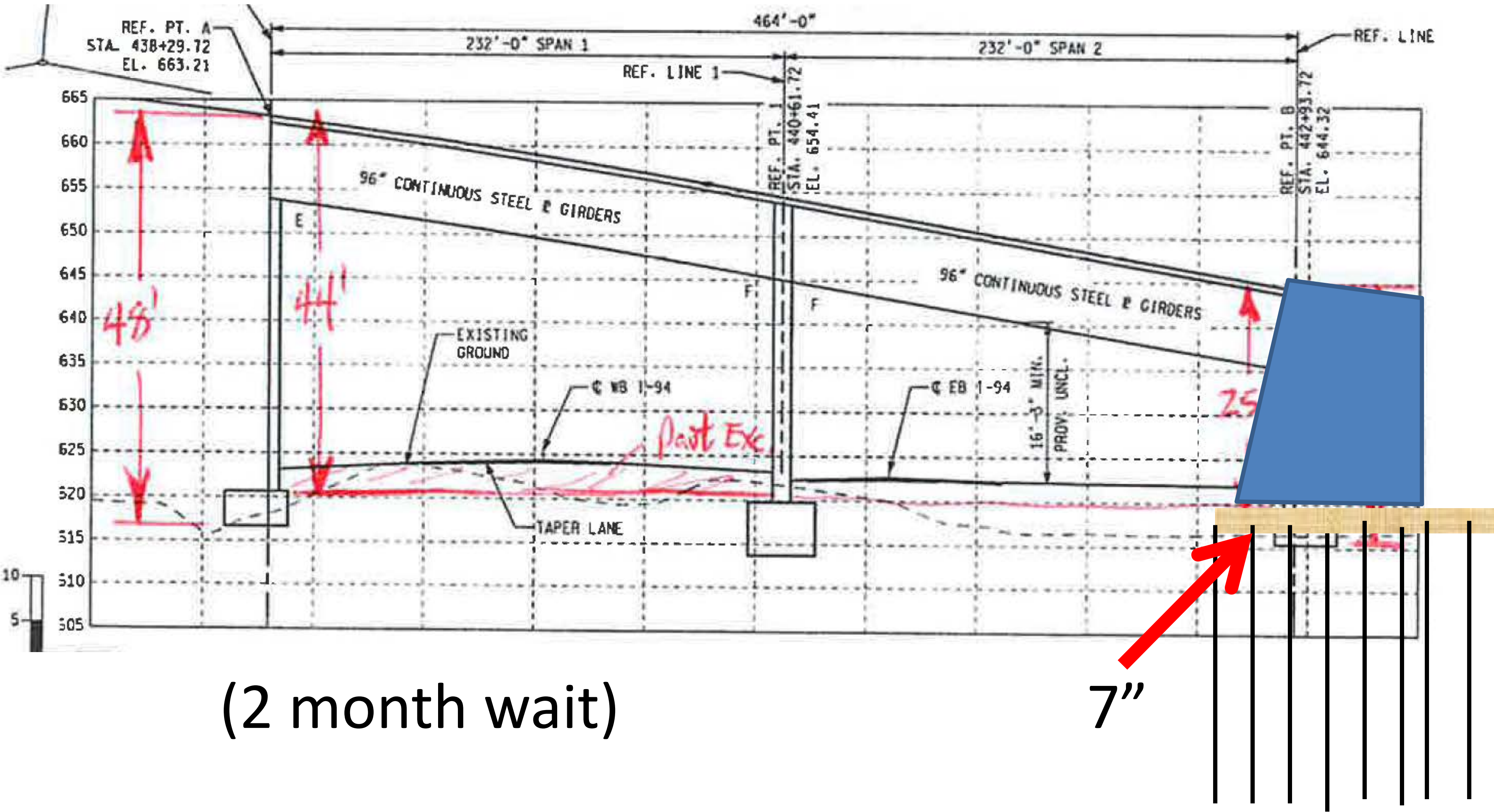
Example Bridge



Wick Drains Installed through Sand Drainage Layer



Pre-Load to Full Height

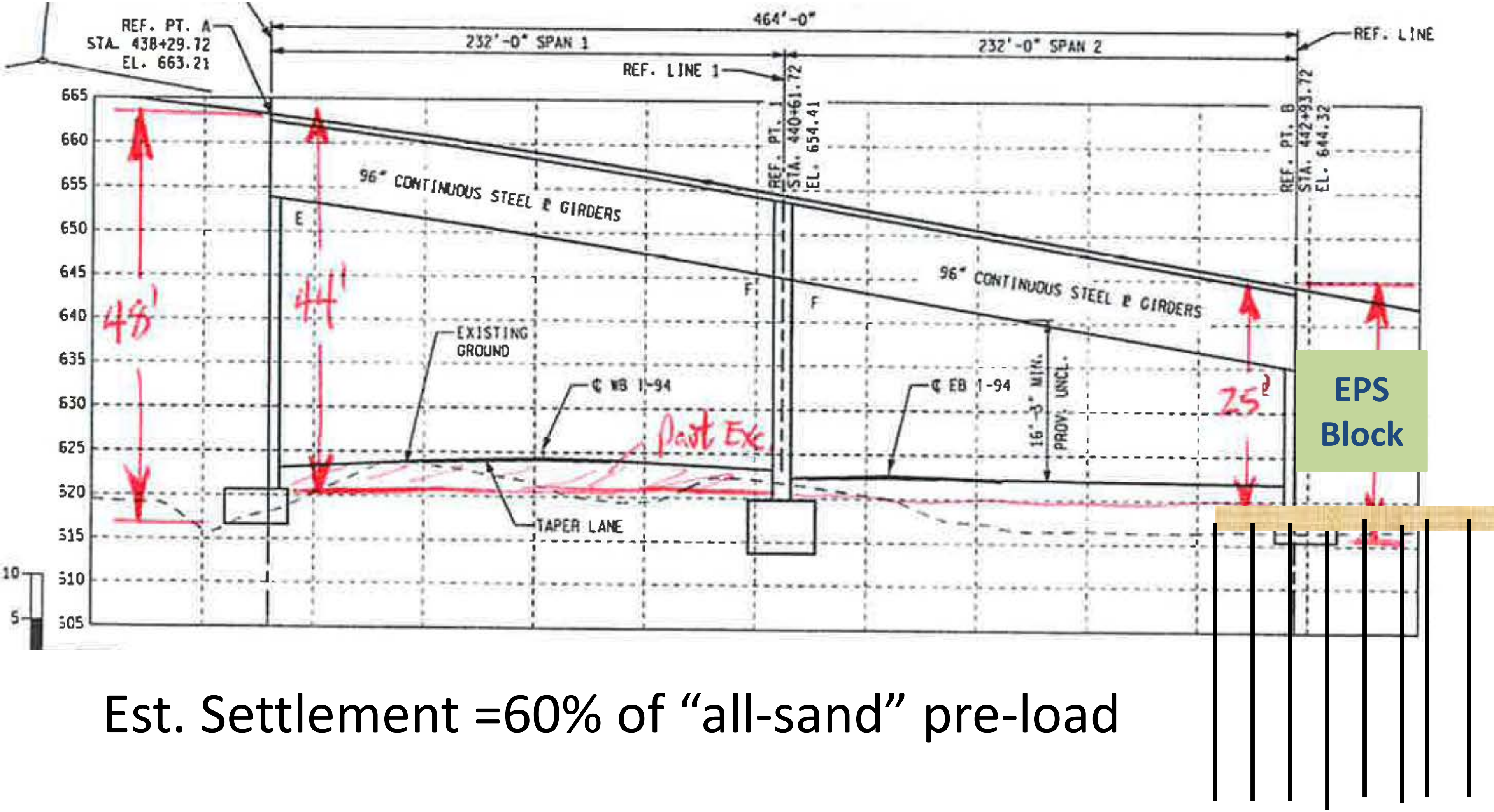


(2 month wait)

7”

Settlement Estimates

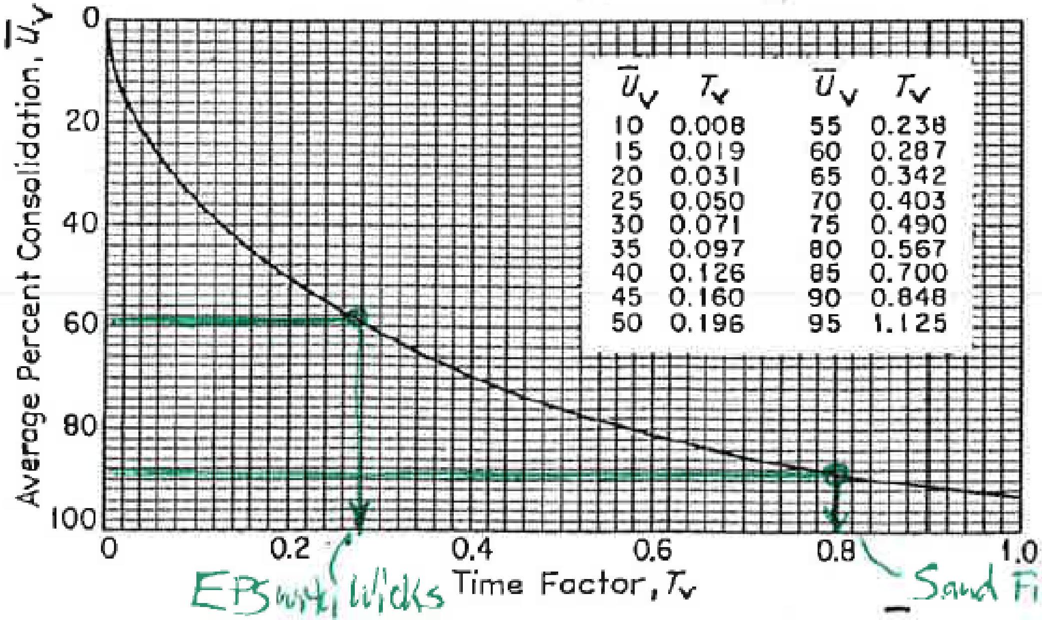
Remove Pre-Load, Piles, and Partial EPS



Est. Settlement =60% of “all-sand” pre-load

$$t = T_v(H_d)^2/c_v \Longrightarrow \approx .848(36)^2/.004 \text{ for } T_{90} \Rightarrow 6.3 \text{ months} \quad (4)$$

The mathematical relationship between $\overline{U}_v$ and T_v is rather complex. Figure 3 presents a plot of $\overline{U}_v$ versus T_v based on

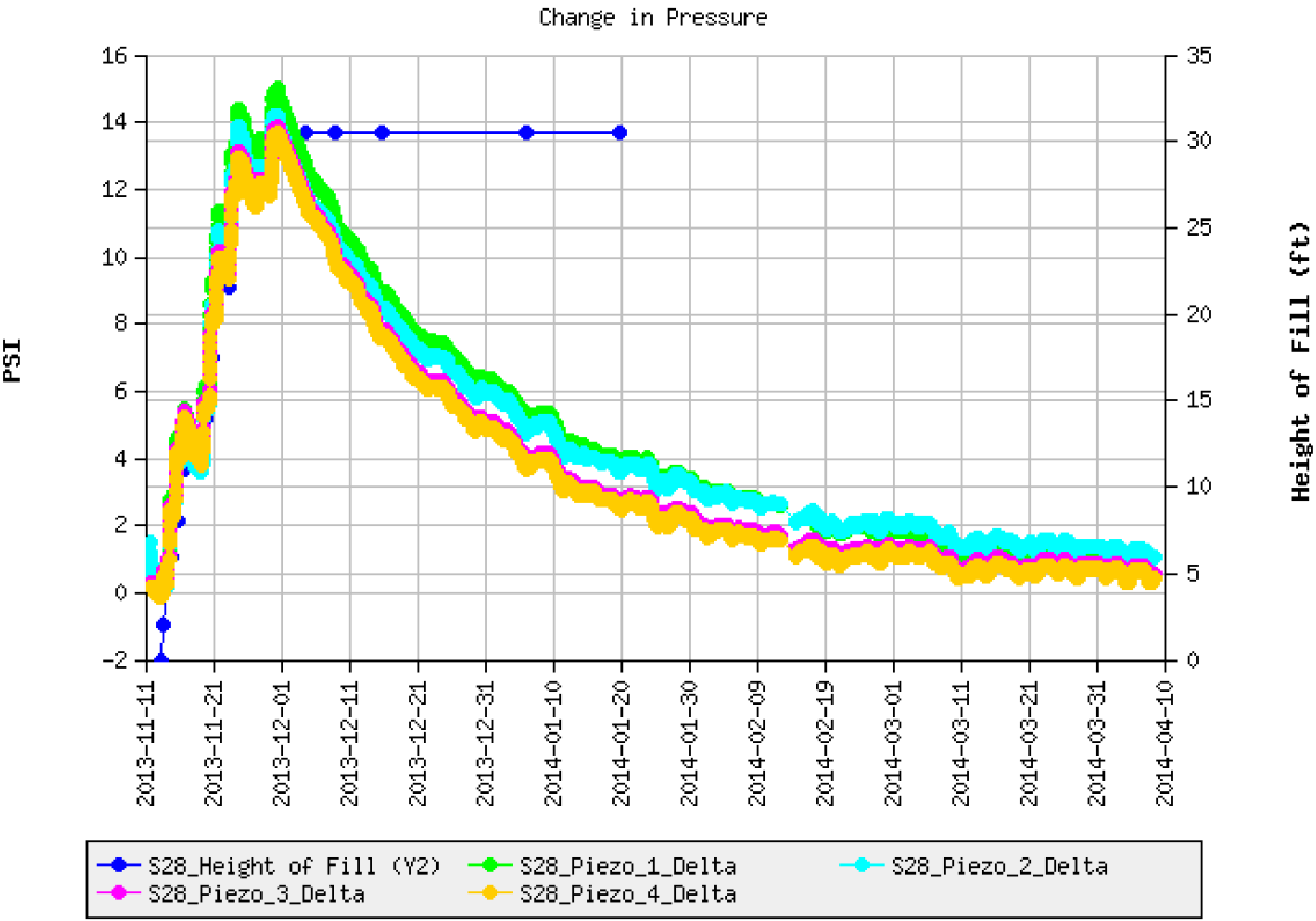


$\overline{U}_v$	T_v	$\overline{U}_v$	T_v
10	0.008	55	0.238
15	0.019	60	0.287
20	0.031	65	0.342
25	0.050	70	0.403
30	0.071	75	0.490
35	0.097	80	0.567
40	0.126	85	0.700
45	0.160	90	0.848
50	0.196	95	1.125

Figure 3. Variation of T_v with $\overline{U}_v$.

≈ 30% of Wait Time for EPS versus Sand Fill (2 month versus 6 months)

*Mandate a 2-month hold using 5 to 6 ft wick spacing and EPS Backfill


Change in Pressure
PSI
Height of Fill (ft)
16
14
12
10
8
6
4
2
0
-2
35
30
25
20
15
10
5
0
2013-11-11
2013-11-21
2013-12-01
2013-12-11
2013-12-21
2013-12-31
2014-01-10
2014-01-20
2014-01-30
2014-02-09
2014-02-19
2014-03-01
2014-03-11
2014-03-21
2014-03-31
2014-04-10
S28_Height of Fill (Y2)
S28_Piezo_1_Delta
S28_Piezo_2_Delta
S28_Piezo_3_Delta
S28_Piezo_4_Delta

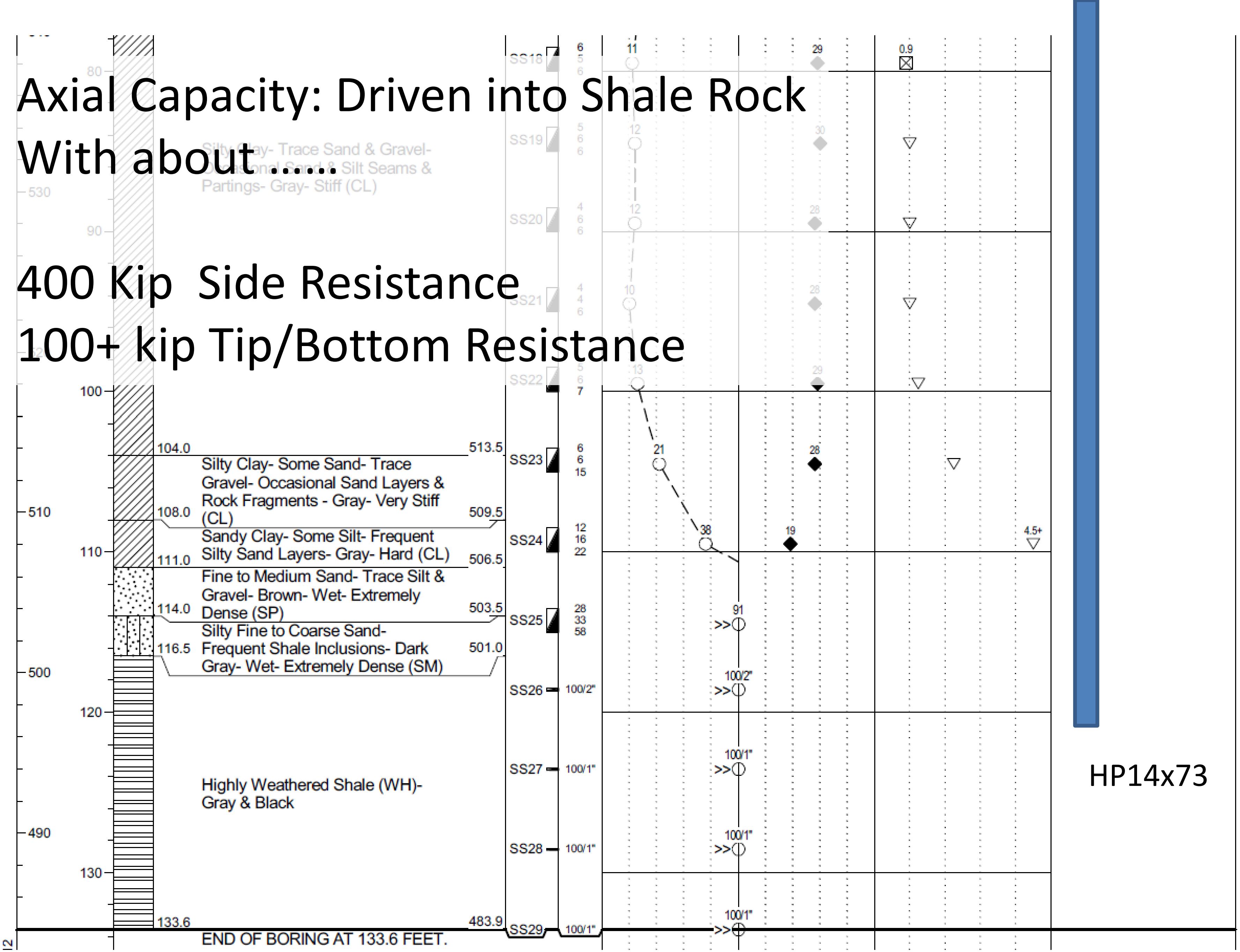
Piling Analyses

- Axial Resistance
- Lateral Resistance: batter vs COM624P
- Bridge Approach Fill Settlement
- Downdrag Negative Skin Friction

Axial Capacity: Driven into Shale Rock With about……

400 Kip Side Resistance

100+ kip Tip/Bottom Resistance



10.7.1.6.2—Downdrag

The provisions of Article 3.11.8 shall apply for determination of load due to negative side resistance.

Where piles are driven to end bearing on a dense stratum or rock and the design of the pile is structurally controlled, downdrag shall be considered at the strength and extreme limit states.

For friction piles that can experience settlement at the pile tip, downdrag shall be considered at the service, strength and extreme limit states. Estimate pile and pile group settlement according to Article 10.7.2.

The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive side and tip resistance below the lowest layer contributing to downdrag computed as specified in Article 3.11.8.

In the case of friction piles with limited tip resistance, the downdrag load can exceed the geotechnical resistance of the pile, causing the pile to move downward enough to allow service limit state criteria for the structure to be exceeded. Where pile settlement is not limited by nominal bearing resistance below the downdrag zone, service limit state tolerances will govern the geotechnical design.

This design situation is not desirable and the preferred practice is to mitigate the downdrag induced foundation settlement through a properly designed surcharge and/or preloading program, or by extending the piles deeper for higher resistance.

If compressible soils are located beneath the embankment, piles should be driven after embankment settlement is complete, if possible, to minimize or eliminate downdrag forces.

Force effects due to downdrag on piles or drilled shafts should be determined as follows:

Step 1—Establish soil profile and soil properties for computing settlement using the procedures in Article 10.4.

Step 2—Perform settlement computations for the soil layers along the length of the pile or shaft using the procedures in Article 10.6.2.4.3.

Step 3—Determine the length of pile or shaft that will be subject to downdrag. If the settlement in the soil layer is 0.4 in. or greater relative to the pile or shaft, downdrag can be assumed to fully develop.

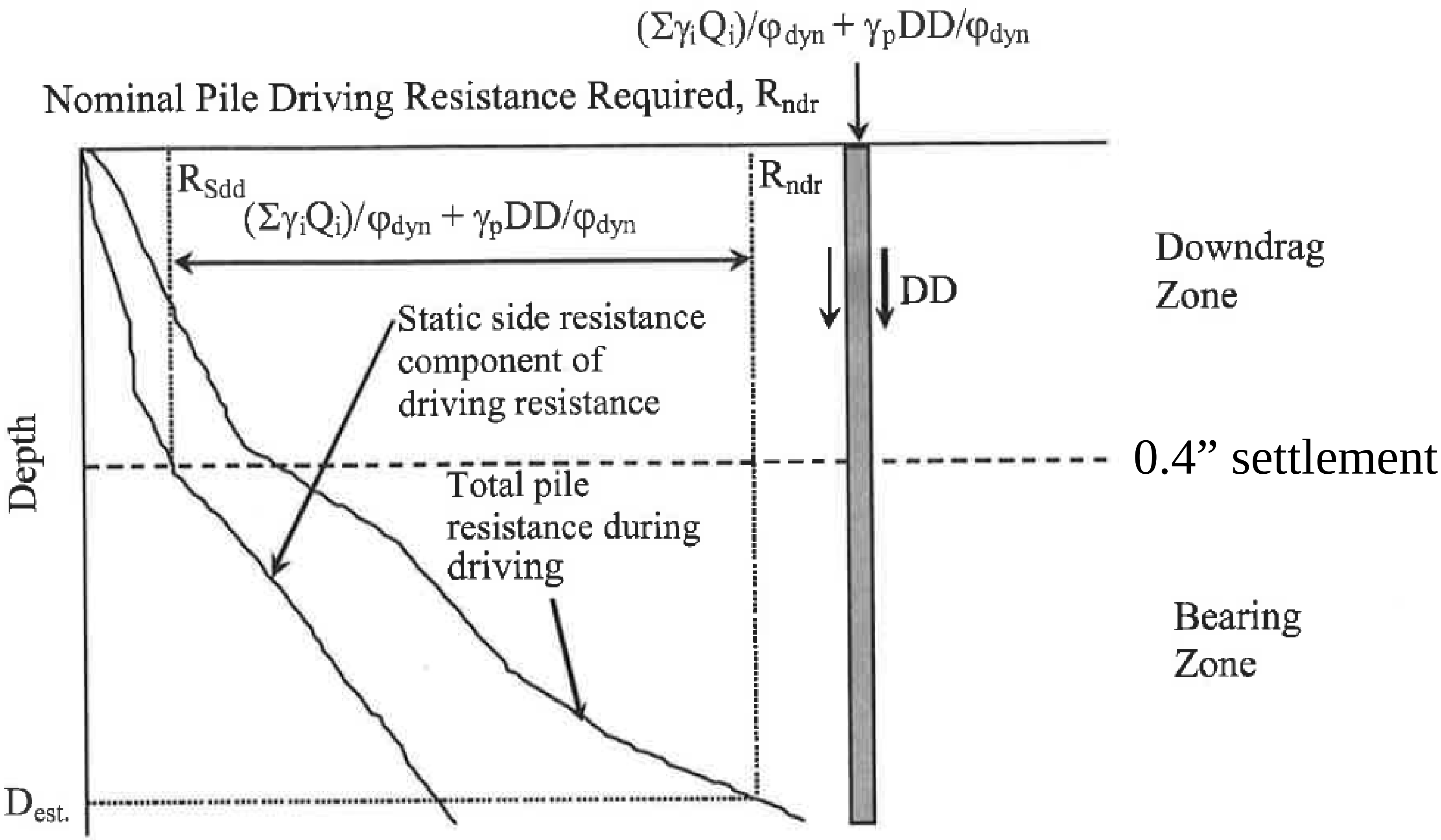
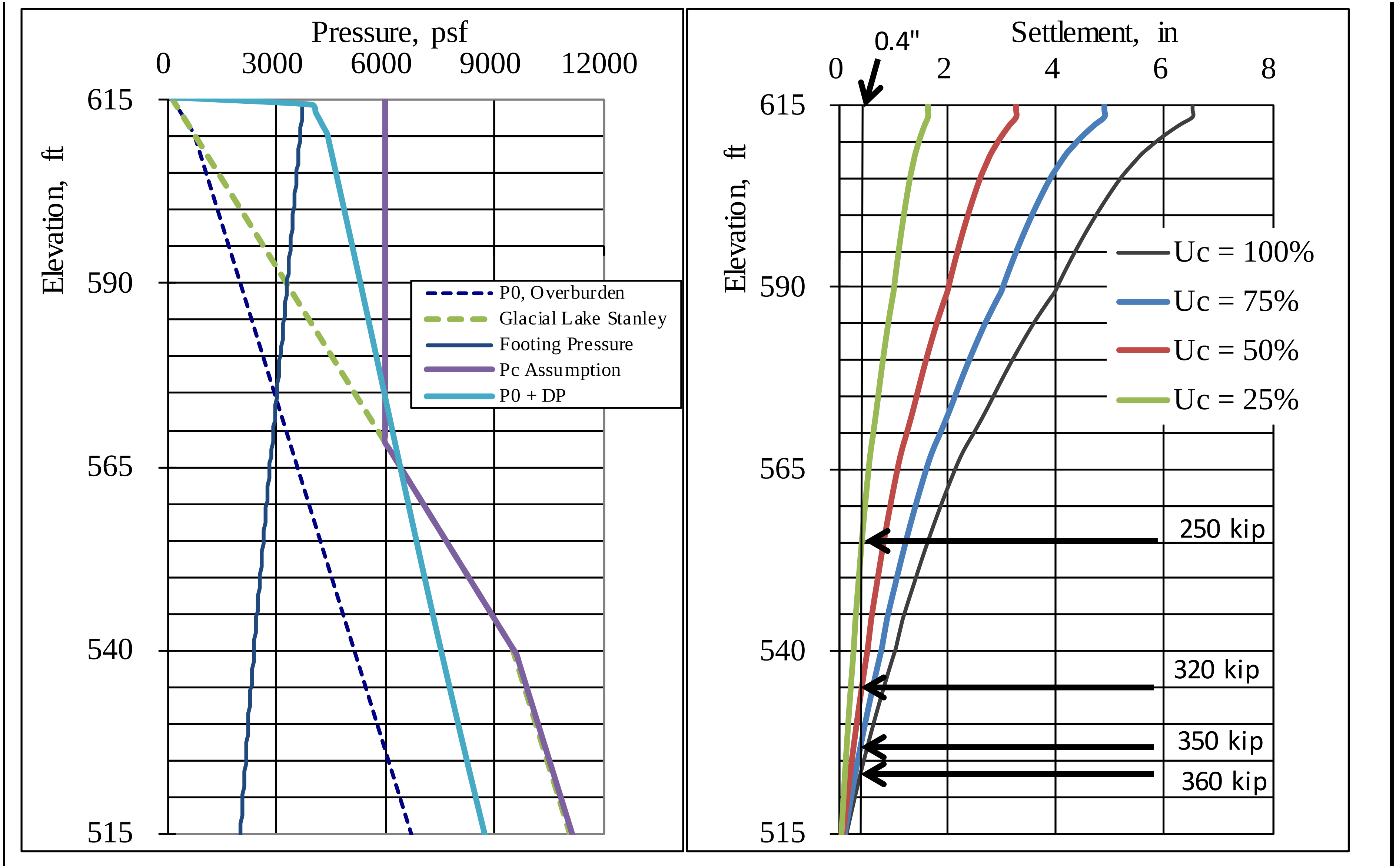
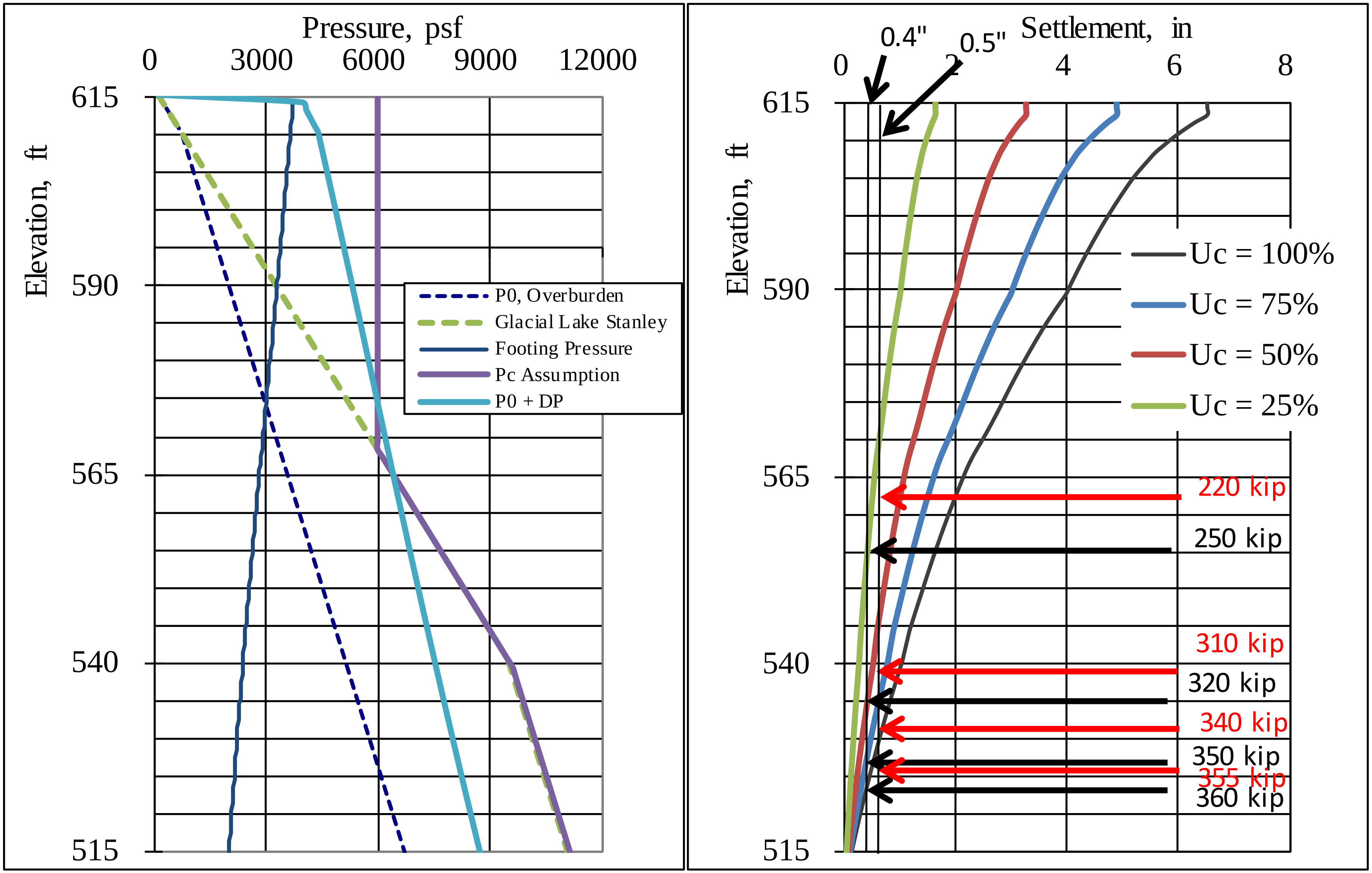


Figure C10.7.3.7-1—Design of Pile Foundations for Downdrag


Pressure, psf
0
3000
6000
9000
12000
615
590
565
540
515
Elevation, ft
P0, Overburden
Glacial Lake Stanley
Footing Pressure
Pc Assumption
P0 + DP
Settlement, in
0.4"
0
2
4
6
8
615
590
565
540
515
Elevation, ft
Uc = 100%
Uc = 75%
Uc = 50%
Uc = 25%
250 kip
320 kip
350 kip
360 kip



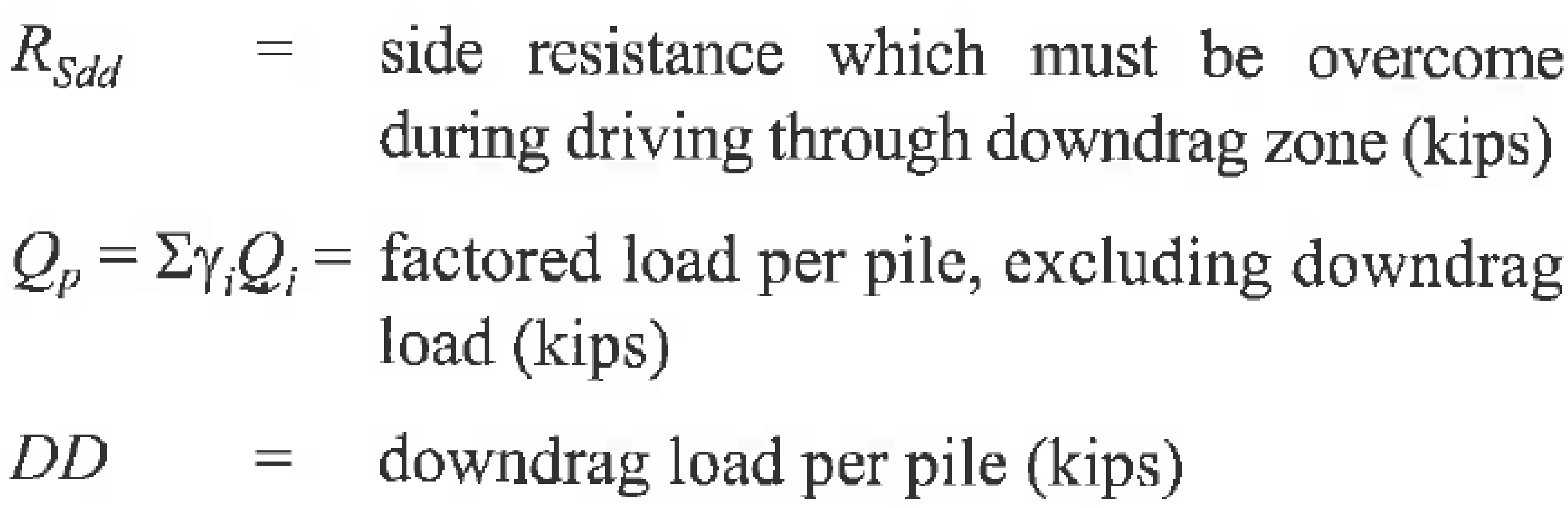
An extra 0.1” allowance for elastic pile shortening….

The summation of the factored loads ($\Sigma\gamma_i Q_i$) should be less than or equal to the factored resistance ($\varphi_{dyn} R_n$). Therefore, the nominal resistance R_n should be greater than or equal to the sum of the factored loads divided by the resistance factor φ_{dyn}. The nominal bearing resistance (kips) of the pile needed to resist the factored loads, including downdrag, is therefore taken as:

$$R_n = \frac{(\Sigma\gamma_i Q_i)}{\varphi_{dyn}} + \frac{\gamma_p DD}{\varphi_{dyn}} \qquad \text{(C10.7.3.7-1)}$$

The total nominal driving resistance, R_{ndr} (kips), needed to obtain R_n, accounting for the side resistance that must be overcome during pile driving that does not contribute to the nominal resistance of the pile, is taken as:

$$R_{ndr} = R_{Sdd} + R_n \qquad \text{(C10.7.3.7-2)}$$



R_{Sdd} = side resistance which must be overcome during driving through downdrag zone (kips)

$Q_p = \Sigma\gamma_i Q_i$ = factored load per pile, excluding downdrag load (kips)

DD = downdrag load per pile (kips)

For $\boldsymbol{R_{ndr}}$ = 500 kip HP14x73, 25% settlement remaining:

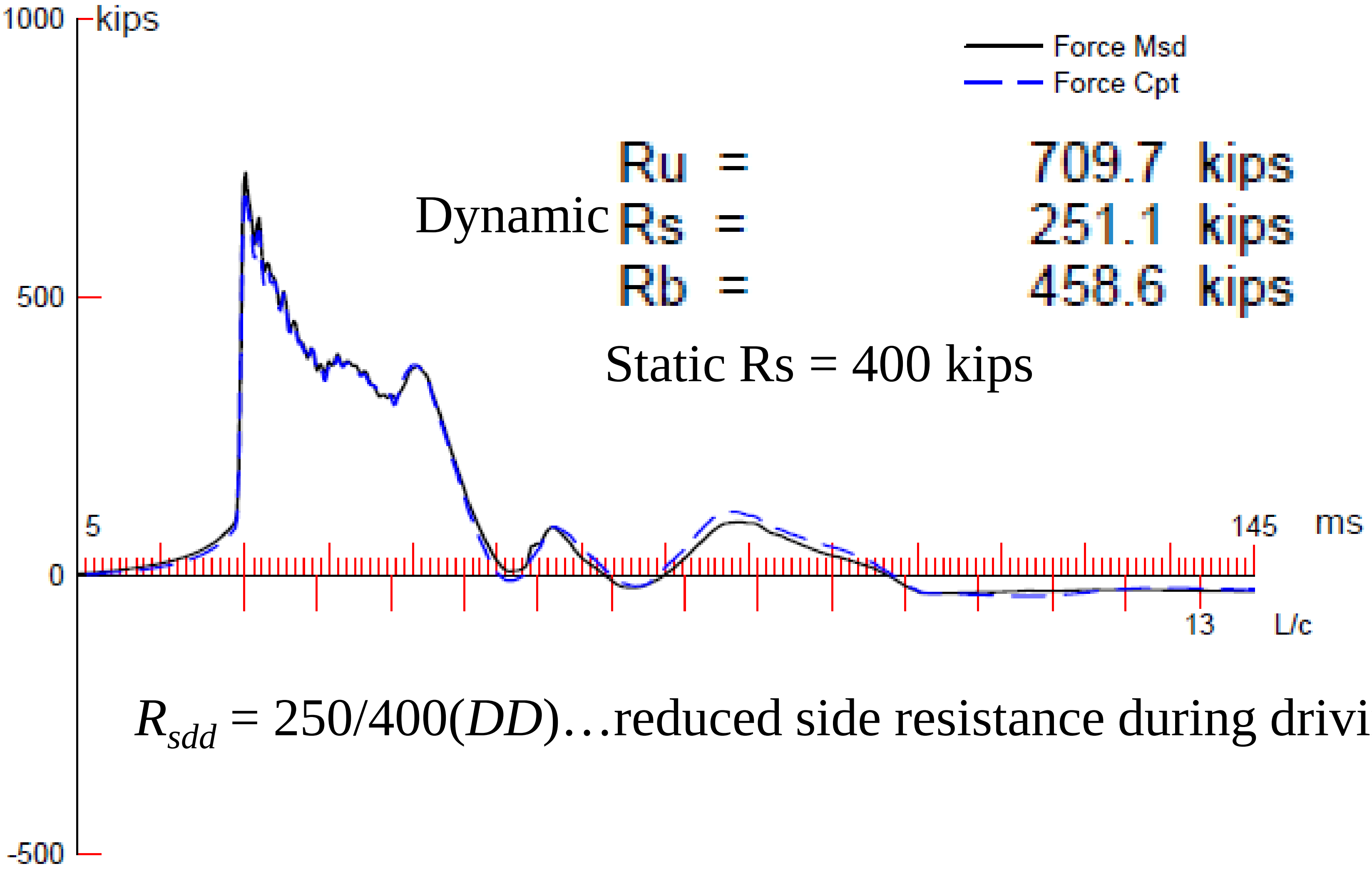
$\boldsymbol{R_n}$ = 500 – 220(250/400) = 362.5 kips

$\boldsymbol{Q_p}$ = 0.75(362.5) – 220 = 52 kips/pile OUCH!!!!

Drive 500 kip pile, only 52 kip available for bridge weight!!!

NO GO!!

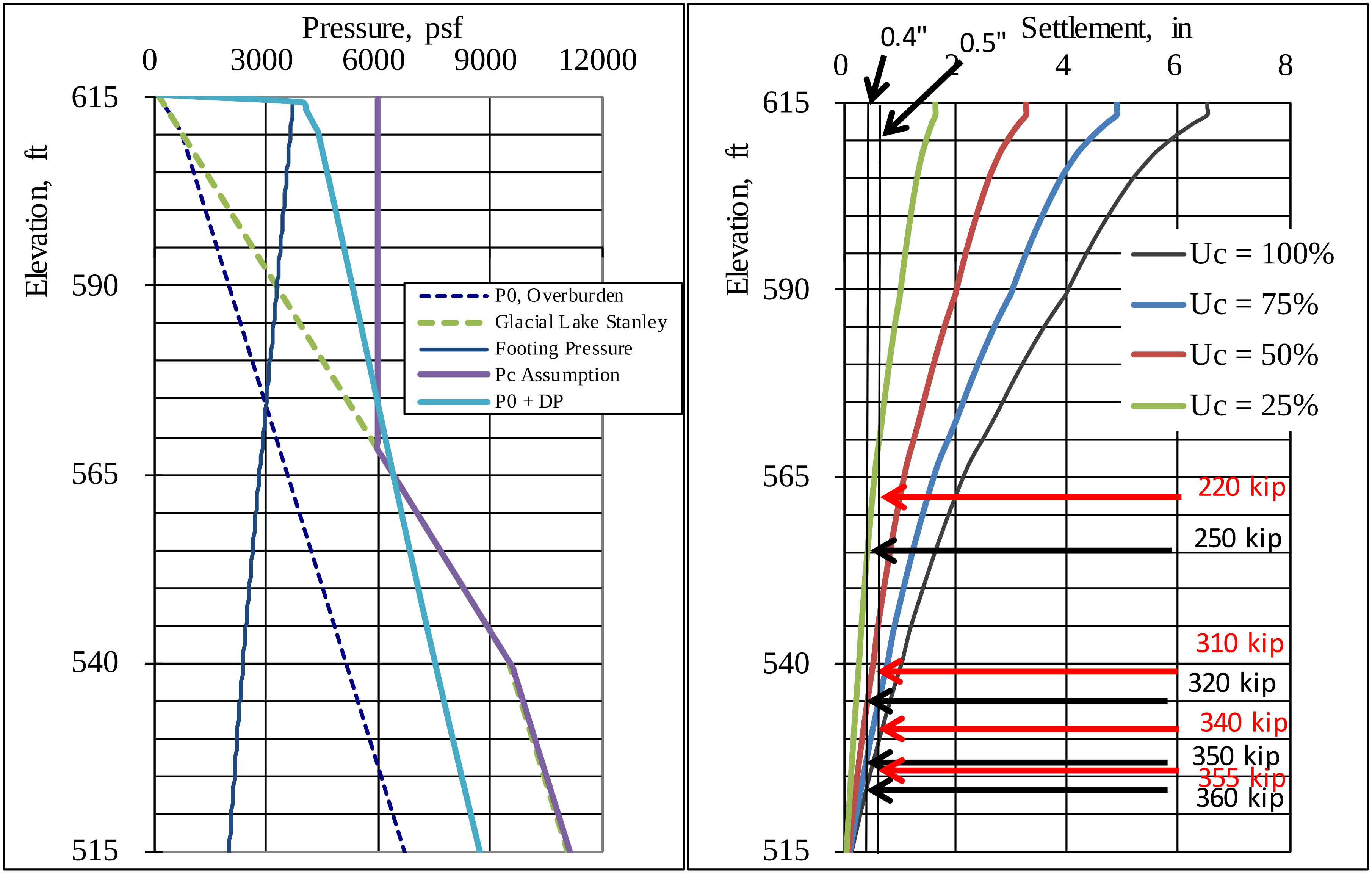
PDA with Dynamic Signal Matching



Dynamic

Static Rs = 400 kips

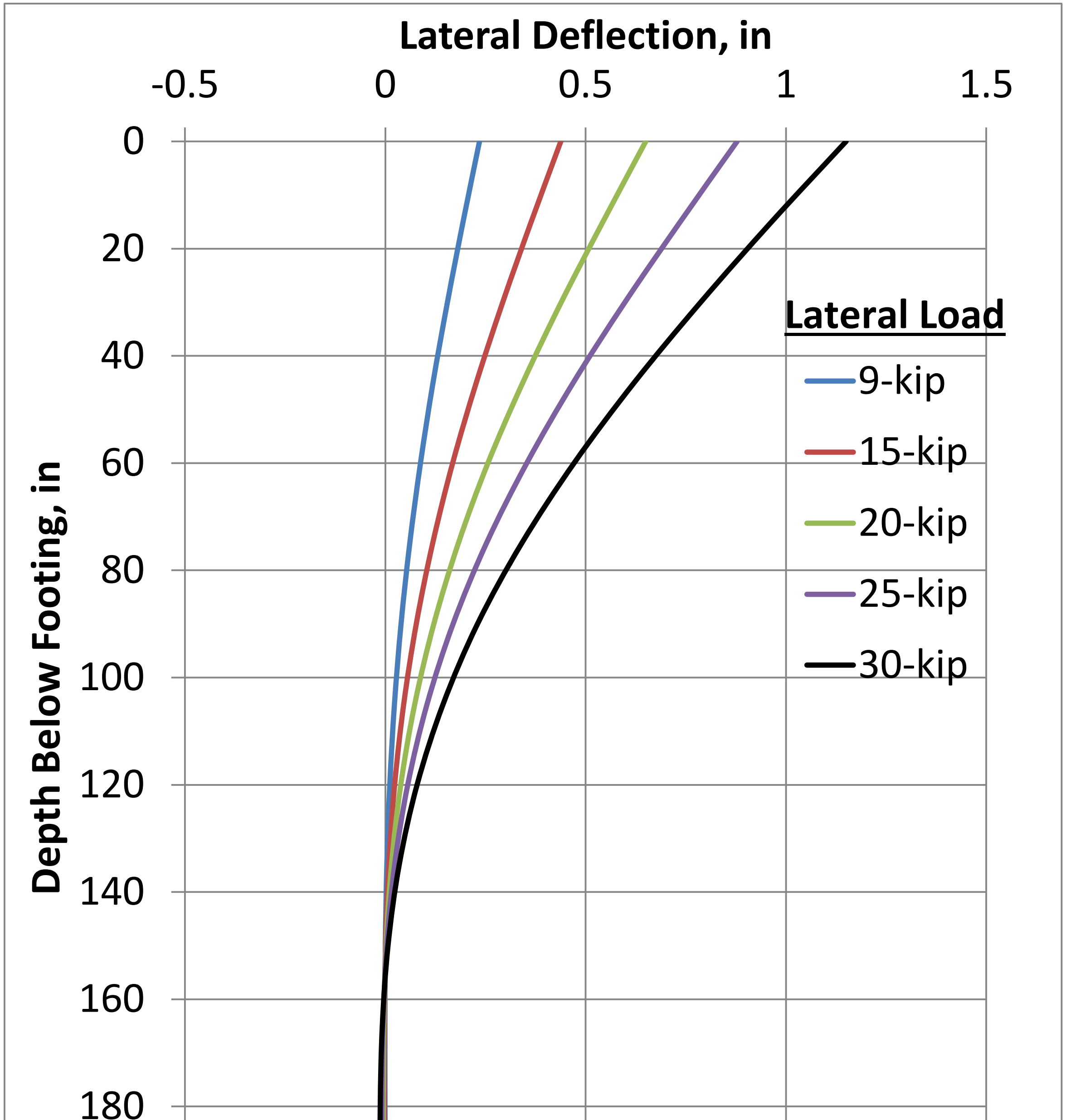
R_{sdd} = 250/400(*DD*)…reduced side resistance during driving



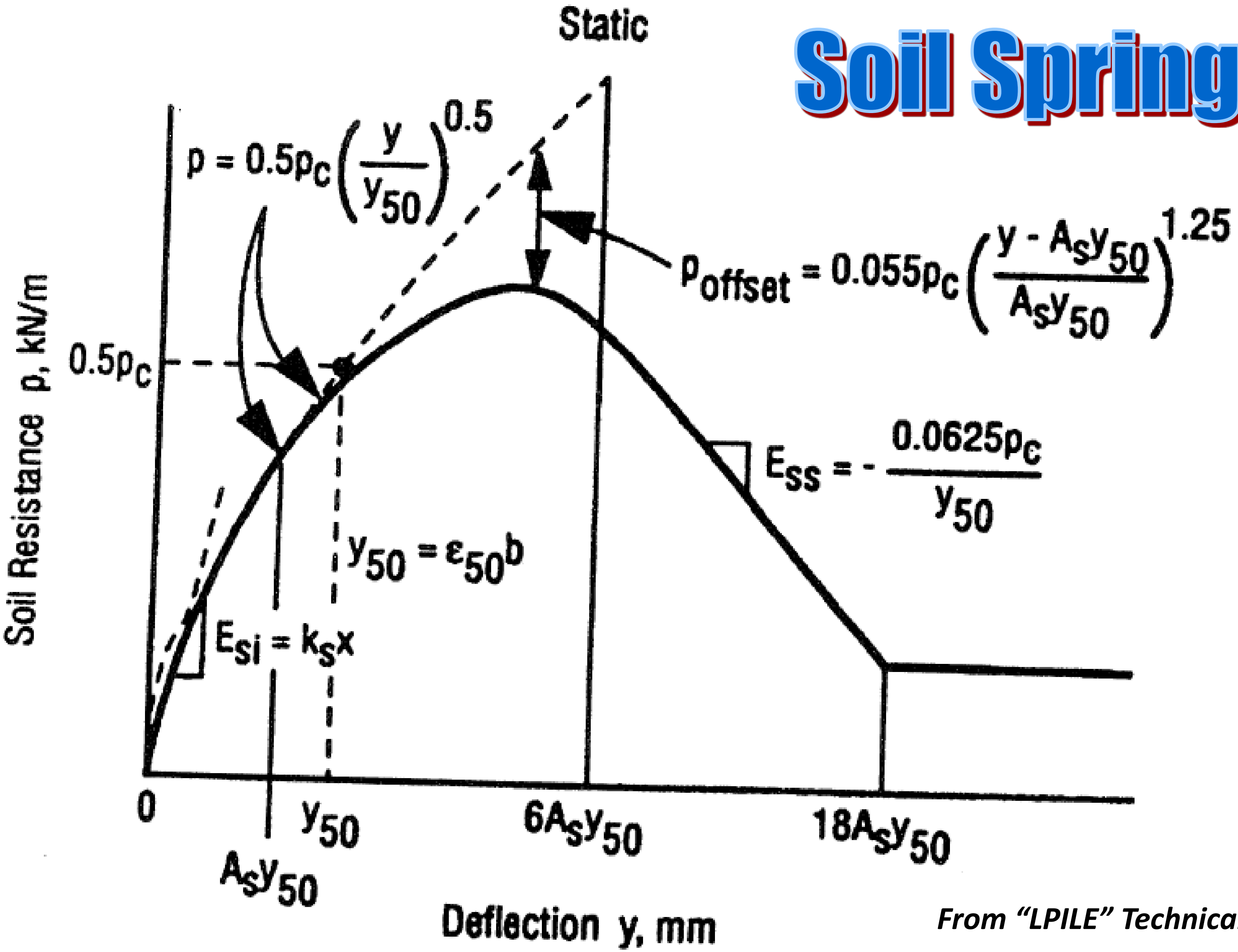
An extra 0.1” allowance for elastic pile shortening….

Pile Lateral Resistance

- COM624P
- LPILE



Soil Springs



From "LPILE" Technical Manual

Stiff Clay Below the Water Line

$$EI\frac{d^4y}{dx^4}+Q\frac{d^2y}{dx^2}-p+W=0$$

P = soil pressure

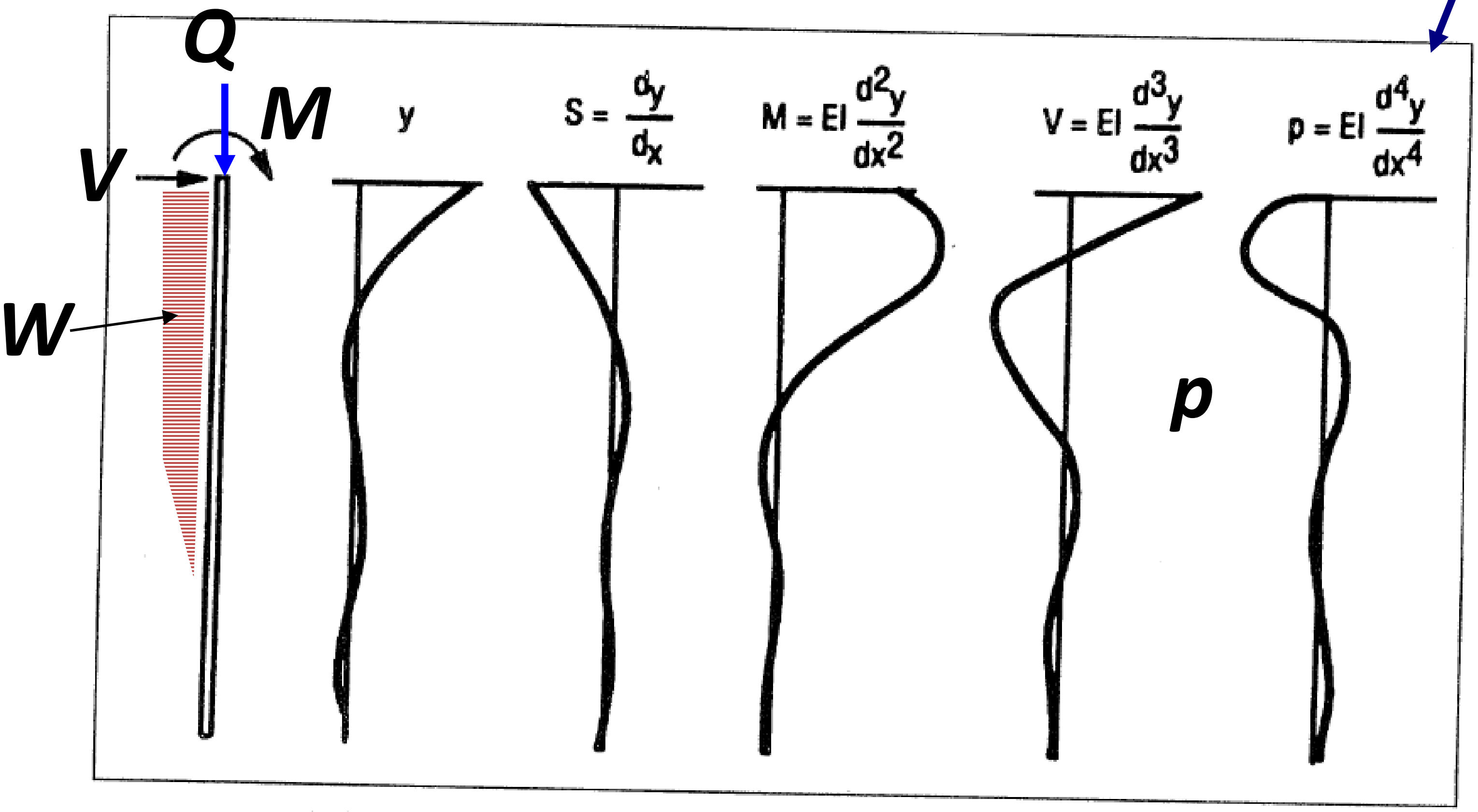


Fig. 2.15 Form of the results obtained from a complete solution

From "LPILE" Technical Manual

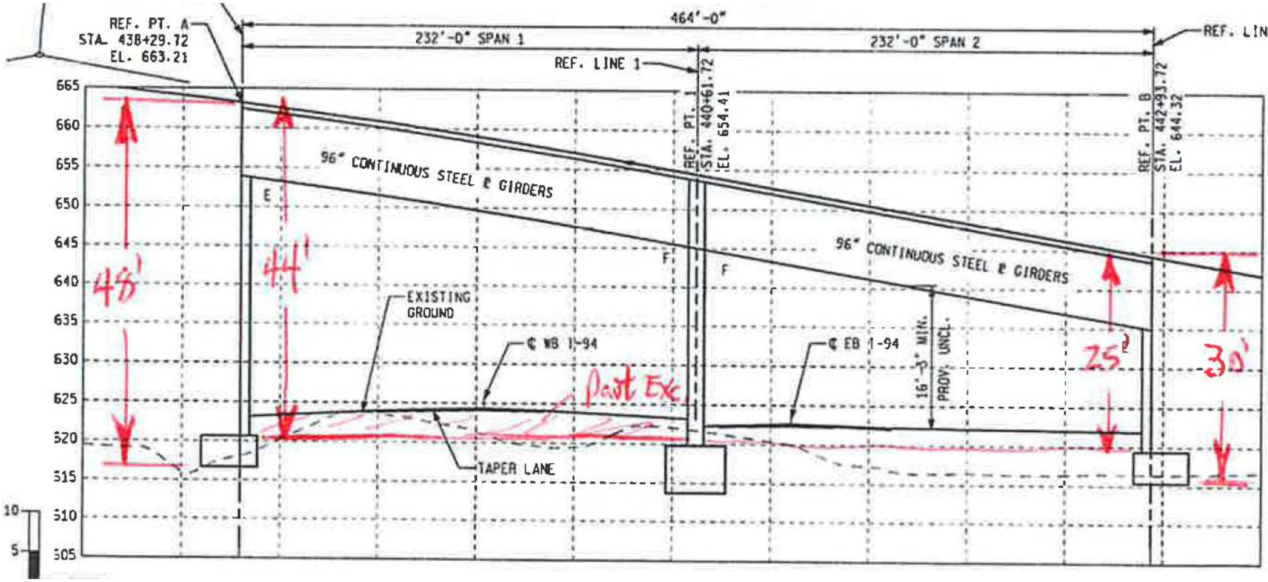
List of Recs Given To Bridge Engineer

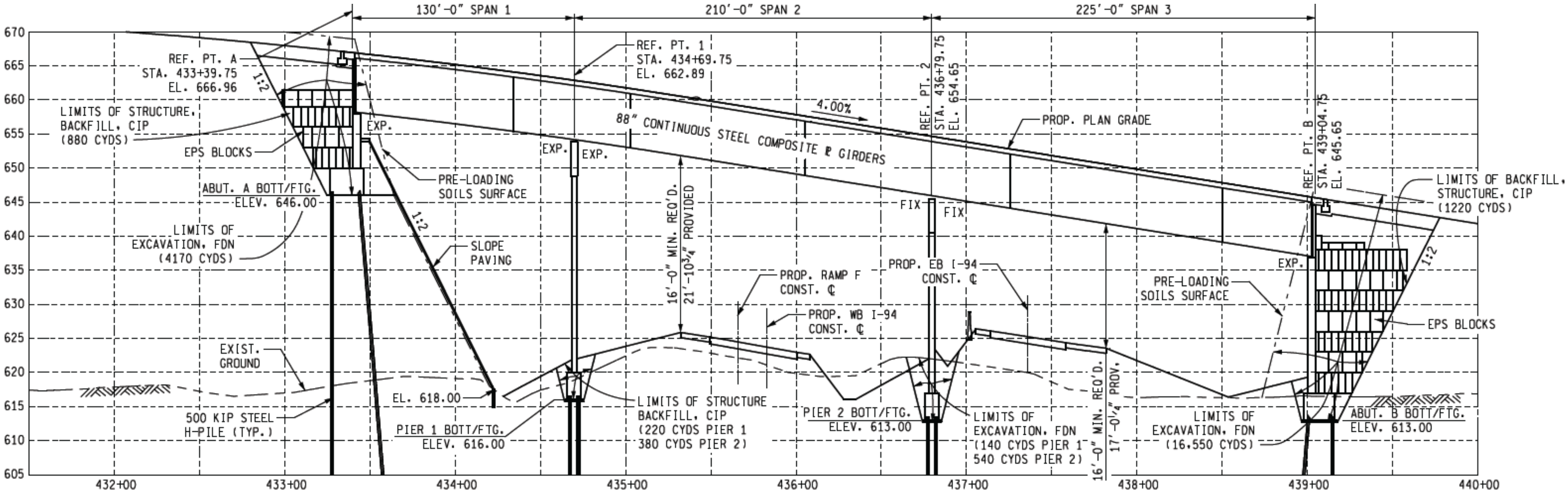
- Global Stability
- Settlement Amounts and Rates
- Spreads versus Deep Foundations
- Lateral Resistances
- Special Provisions/Materials Specifications
- Construction Considerations
  - Water control
  - Surface preparation
  - Temporary Walls
  - Vibrations
  - Geotechnical Instrumentation needed?

Doug Parmerlee

- Overview of Abutment Design Concepts

Geotechnical Engineering During Construction

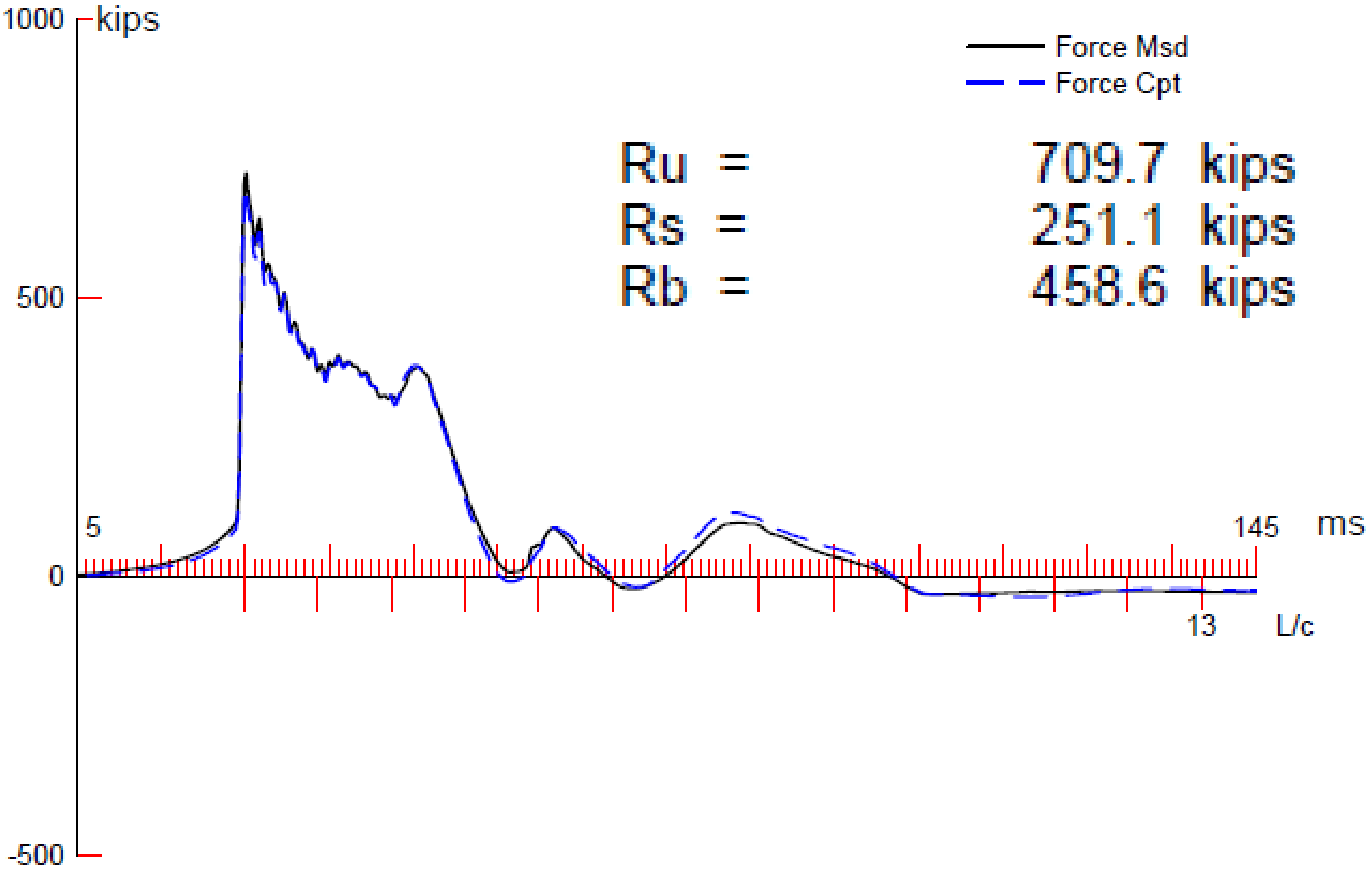




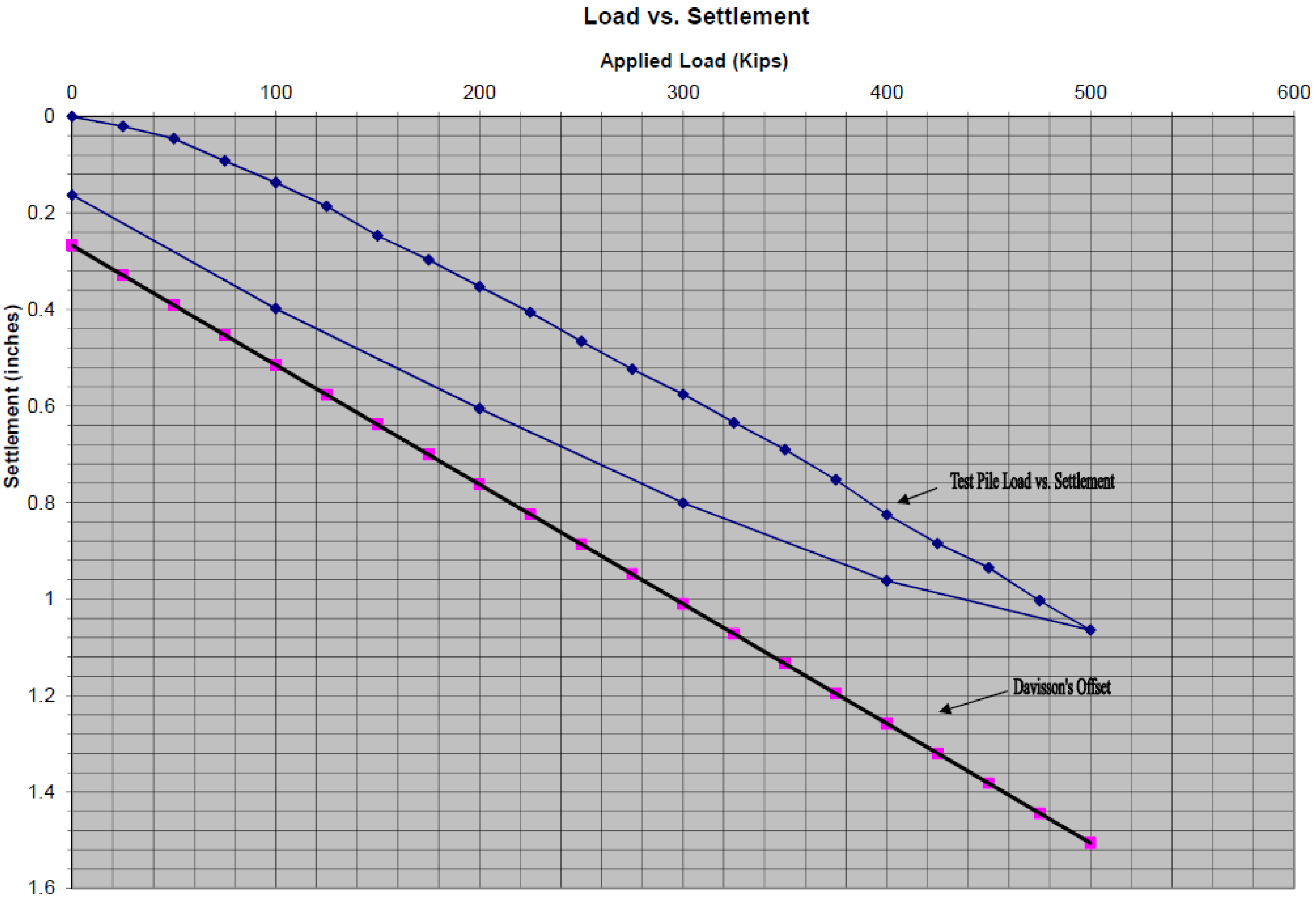
Geotechnical Field Monitoring

- Pile Axial Capacity
- Settlement Rates and Amounts
- Geosynthetics: Limits/Continuity/Splicing
- Lightweight Fills: Limits/Materials

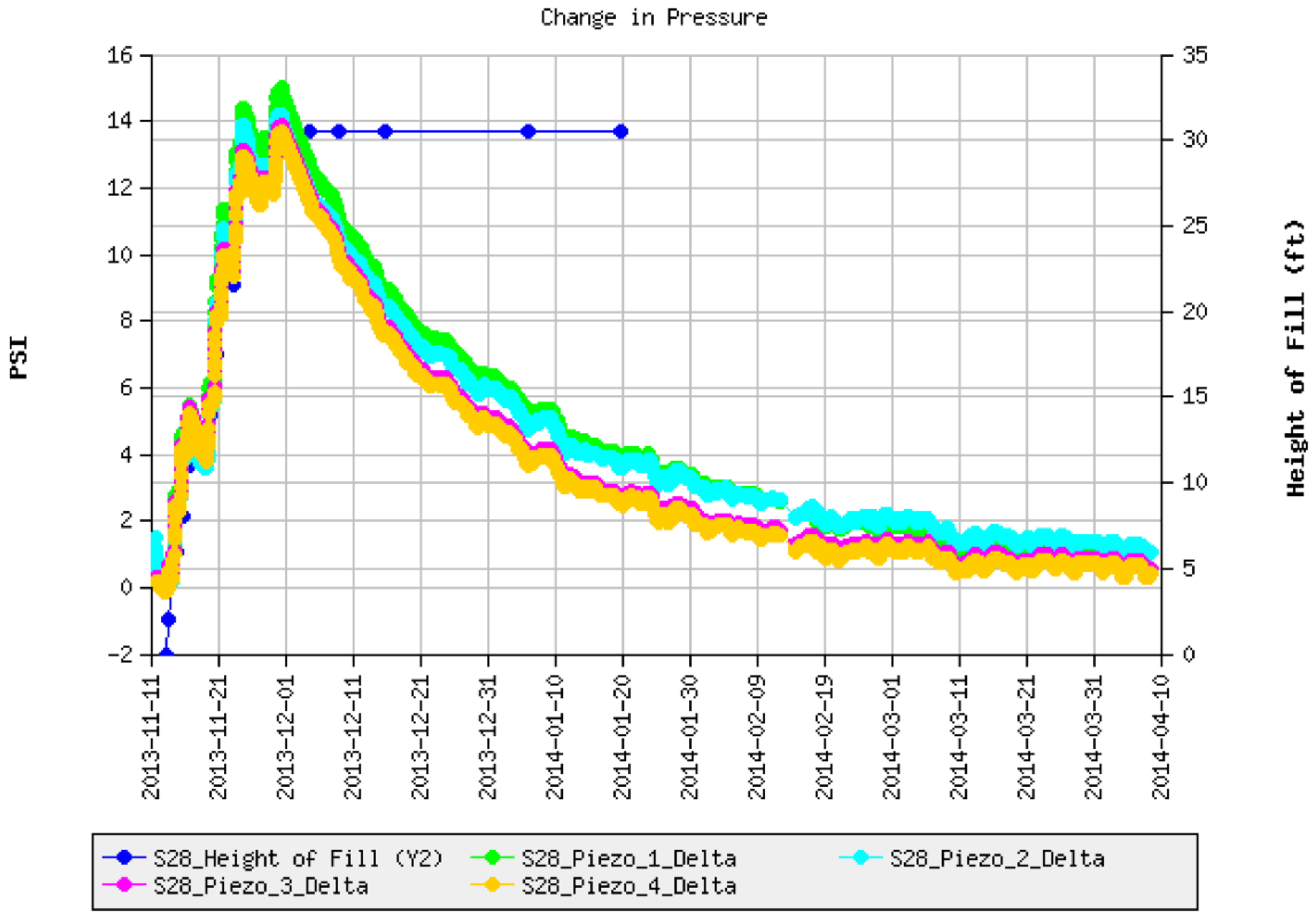
PDA with Dynamic Signal Matching



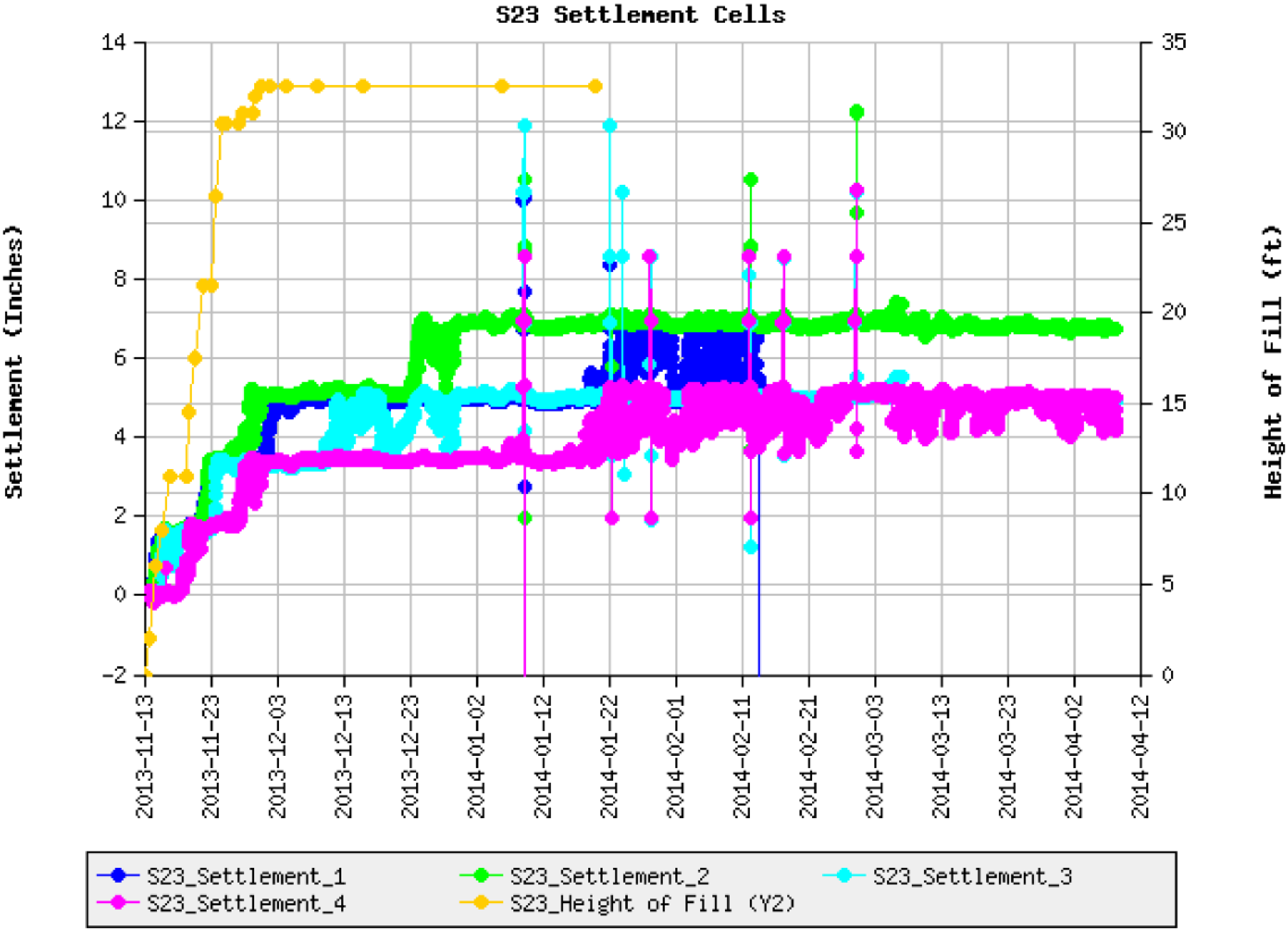
Ru = 709.7 kips
Rs = 251.1 kips
Rb = 458.6 kips

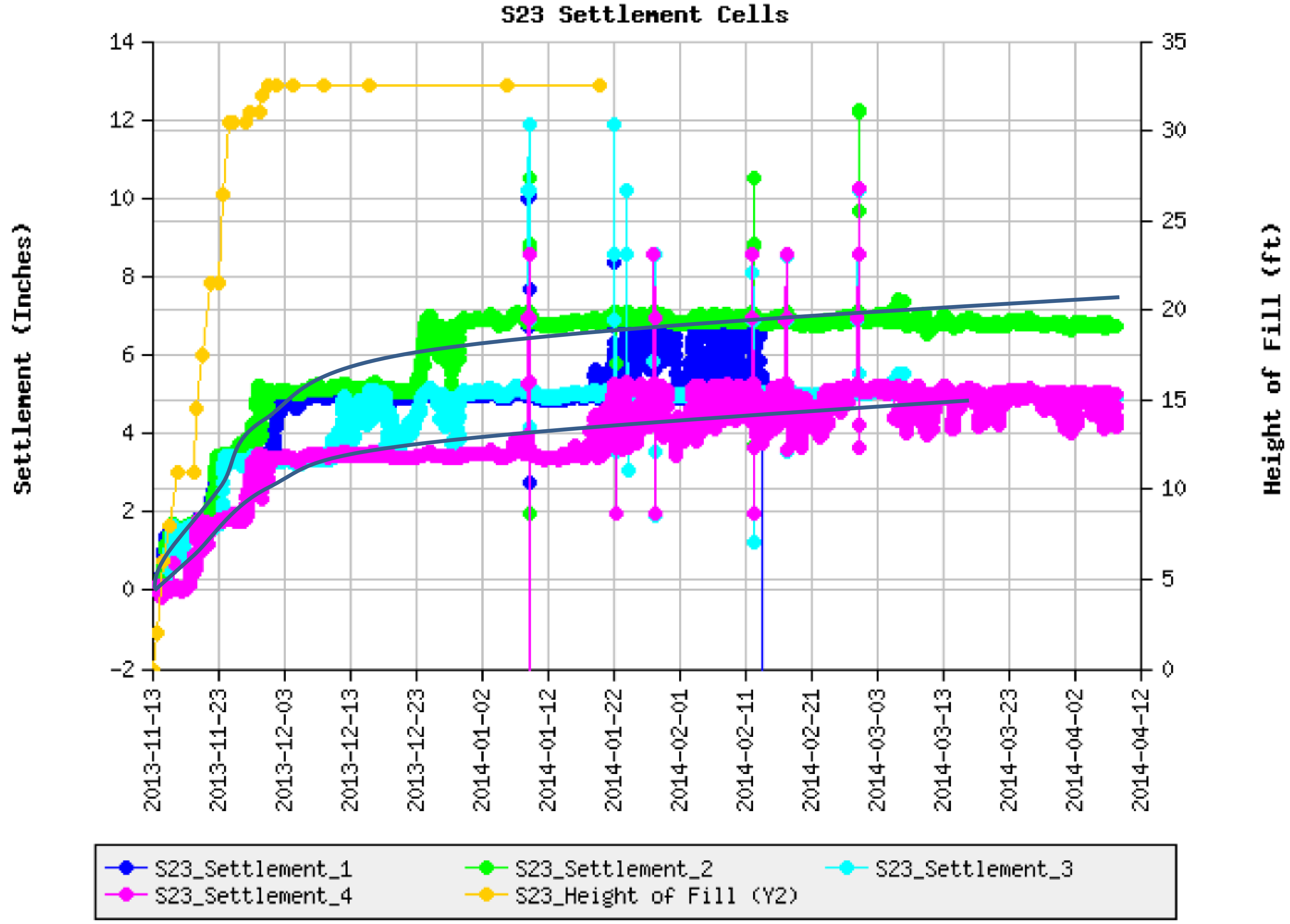


Static Pile Load Tests



Soil Pore Pressure Dissipation


S23 Settlement Cells
Settlement (Inches)
Height of Fill (ft)
14
12
10
8
6
4
2
0
-2
35
30
25
20
15
10
5
0
2013-11-13
2013-11-23
2013-12-03
2013-12-13
2013-12-23
2014-01-02
2014-01-12
2014-01-22
2014-02-01
2014-02-11
2014-02-21
2014-03-03
2014-03-13
2014-03-23
2014-04-02
2014-04-12
S23_Settlement_1
S23_Settlement_2
S23_Settlement_3
S23_Settlement_4
S23_Height of Fill (Y2)

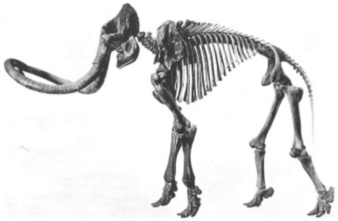

S23 Settlement Cells
Settlement (Inches)
Height of Fill (ft)
14
12
10
8
6
4
2
0
-2
35
30
25
20
15
10
5
0
2013-11-13
2013-11-23
2013-12-03
2013-12-13
2013-12-23
2014-01-02
2014-01-12
2014-01-22
2014-02-01
2014-02-11
2014-02-21
2014-03-03
2014-03-13
2014-03-23
2014-04-02
2014-04-12
S23_Settlement_1
S23_Settlement_2
S23_Settlement_3
S23_Settlement_4
S23_Height of Fill (Y2)

Ancient Glacial Lake "Beach" on top of Grand Portal Point, Pictured Rocks

Comments or Questions?

Michigan's State Fossil:

Mastodon



$$M = EI \frac{d^2 z}{dx^2}$$