Design of Deep Excavations

$2 Billion Hudson Yards, New York, NY

Circular wet soil mix shaft, Florida

Cofferdams for New Tapan Zee Bridge, NY

DEEP EXCAVATION

RELIABLE GEOEXPERTISE

Part 2: Mar/3/2015

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Webinar topics

- Philosophy of deep excavation design
- Identification of issues
- Understanding soil response
- Geotechnical investigations
- Wall systems
- Support systems
- Analysis methods
- Design approach/codes
- Design examples
- Case histories

Overview of Part 1

- Understanding soil response
  - Cohesive vs. frictional materials
- Apparent earth pressure diagrams are not soil properties.
- Analysis methods
  - Conventional (limit-equilibrium etc)
  - Elastoplastic with Winkler springs
  - Finite element methods
- Obtain wall moments, displacements, support forces

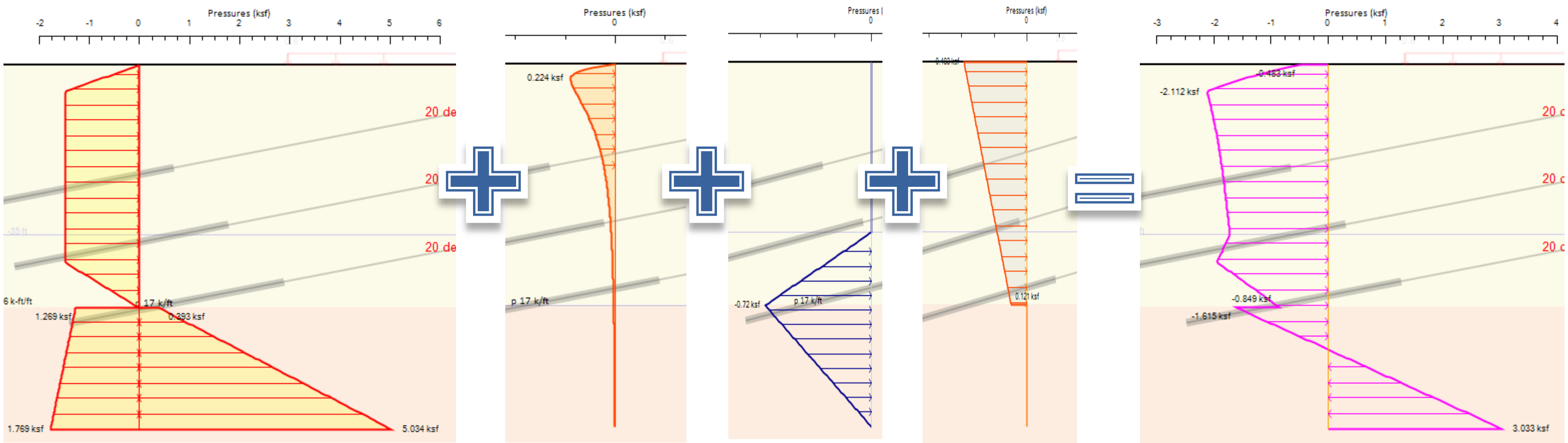
1.1 Conventional methods

General

- Assume lateral earth pressures.
- Determine fixity locations for forces at subgrade.
- Analyze wall beam with assumed loads.

- Advantages: Easy method to verify. Gives a back check for more rigorous methods.
- Disadvantages: Soil-structure interaction ignored.

1.2 Determine net loading diagram on wall



Soil pressures Surch. Water Seismic Net loading

Soldier pile walls (berlin type), 3D effects
Pile spacing above excavation,
Active and passive effective widths, Water width

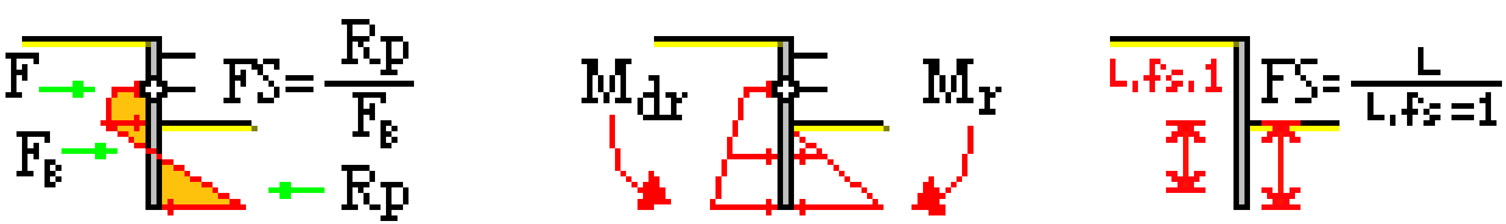
1.3 Wall embedment safety factors (limit-equilibrium)

- Horizontal force
- Moment
- Length

$$FSpas = \frac{Available\ Resistance\ beneath\ \text{virtual}\ fixity\ point}{Hor.reaction\ at\ virtual\ point + driving\ pressures\ beneath\ virtual\ fixity\ point}$$

$$FSrotation = \frac{Resisting\ moments\ about\ a\ point}{Driving\ moments\ about\ the\ same\ point} \quad (Eq.\ 9.2)$$

$$FSembed = \frac{Available\ wall\ embedment\ depth}{Max.Required\ embedment\ depth\ for\ FS = 1\ from\ Equations\ 1\&\ 2\ above} \quad (Eq.\ 9.3)$$

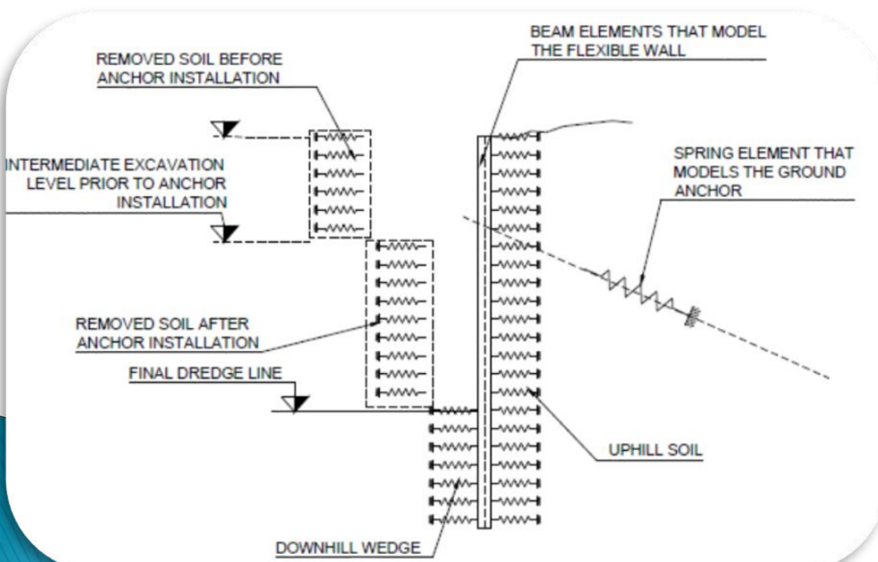


1.4 Beam on elastic foundations

Soil assumed as elastic (elastoplastic) springs.
Different methods available:

a) Driving pressures assumed, passive springs
b) Active and passive soil springs
c) Stage dependency?

- Subgrade reaction (depends on dimensions)
- From soil elasticity with active/passive wedges



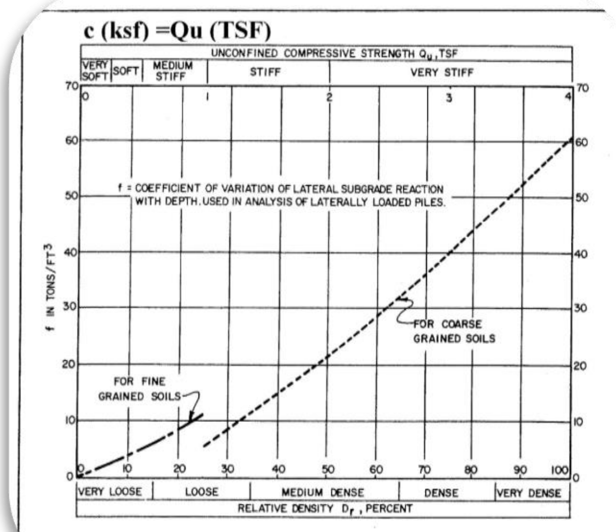
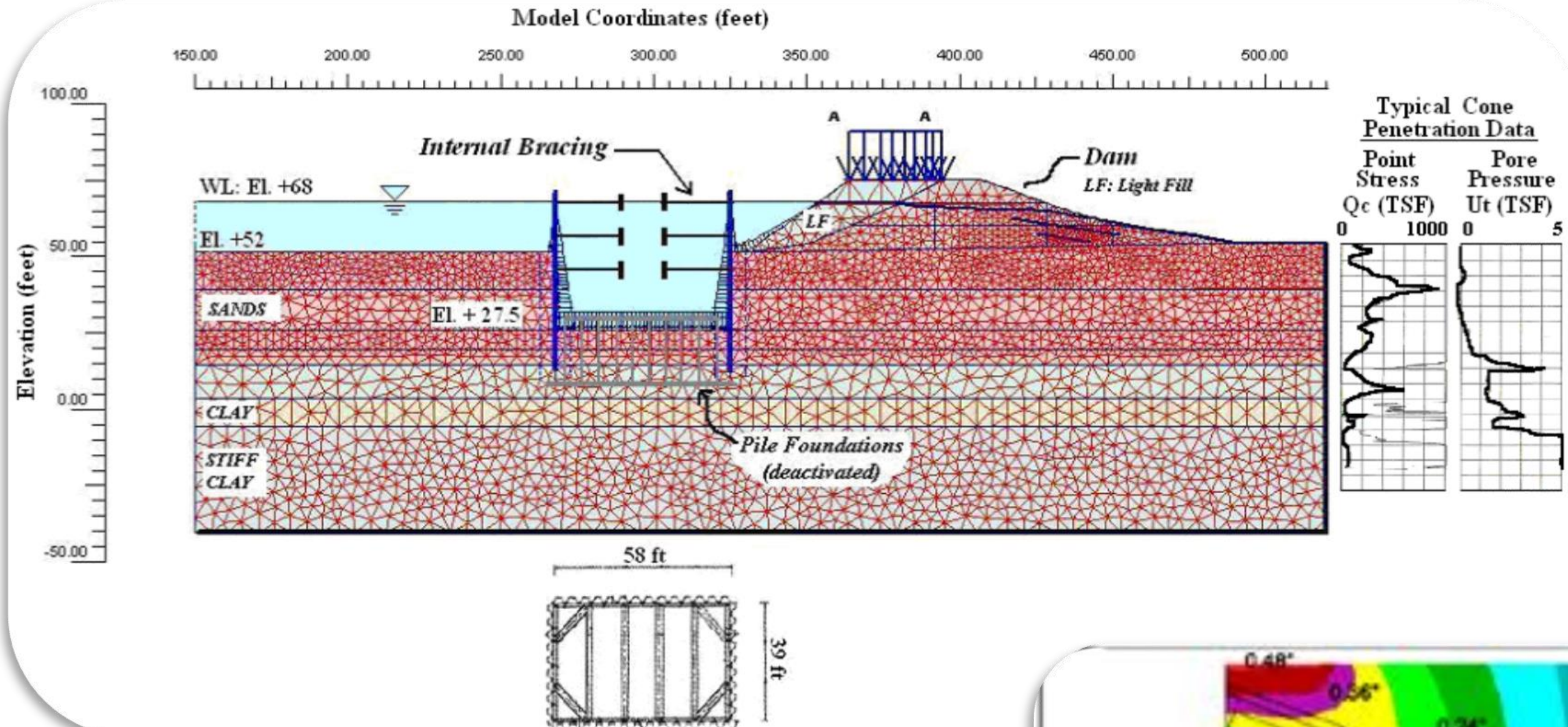
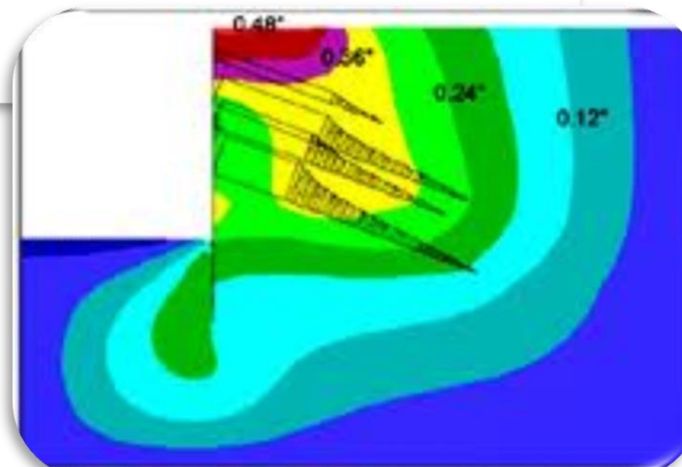


FIGURE 9
Coefficient of Variation of Subgrade Reaction

1.5 Finite element analysis





$$
\begin{bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \varepsilon_{zz} \\ \gamma_{xx} \\ \gamma_{yy} \\ \gamma_{zz} \end{bmatrix} = \frac{1}{E} \begin{bmatrix} 1 & -v & -v & 0 & 0 & 0 \\ -v & 1 & -v & 0 & 0 & 0 \\ -v & -v & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 2(1+v) & 0 & 0 \\ 0 & 0 & 0 & 0 & 2(1+v) & 0 \\ 0 & 0 & 0 & 0 & 0 & 2(1+v) \end{bmatrix} \begin{bmatrix} \sigma'_{xx} \\ \sigma'_{yy} \\ \sigma'_{zz} \\ \sigma'_{xy} \\ \sigma'_{yz} \\ \sigma'_{zx} \end{bmatrix}
$$

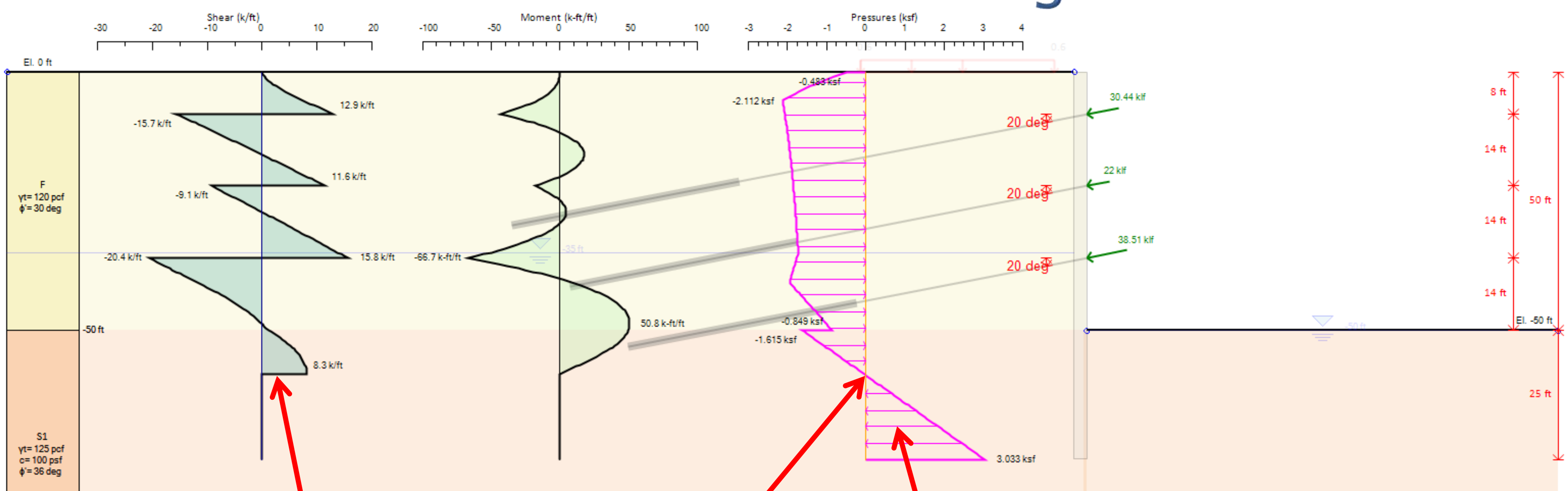
E= Young's modulus v= Poisson's ratio

σ'_{xx}= Effective stress

1.6 Blum’s method

- Pinned supports – continuous beam
- Point of zero net soil shear below subgrade.
- Use point of zero shear as a virtual support.

Shear moments Net loading



Reaction for embedment Fxb

Virtual support

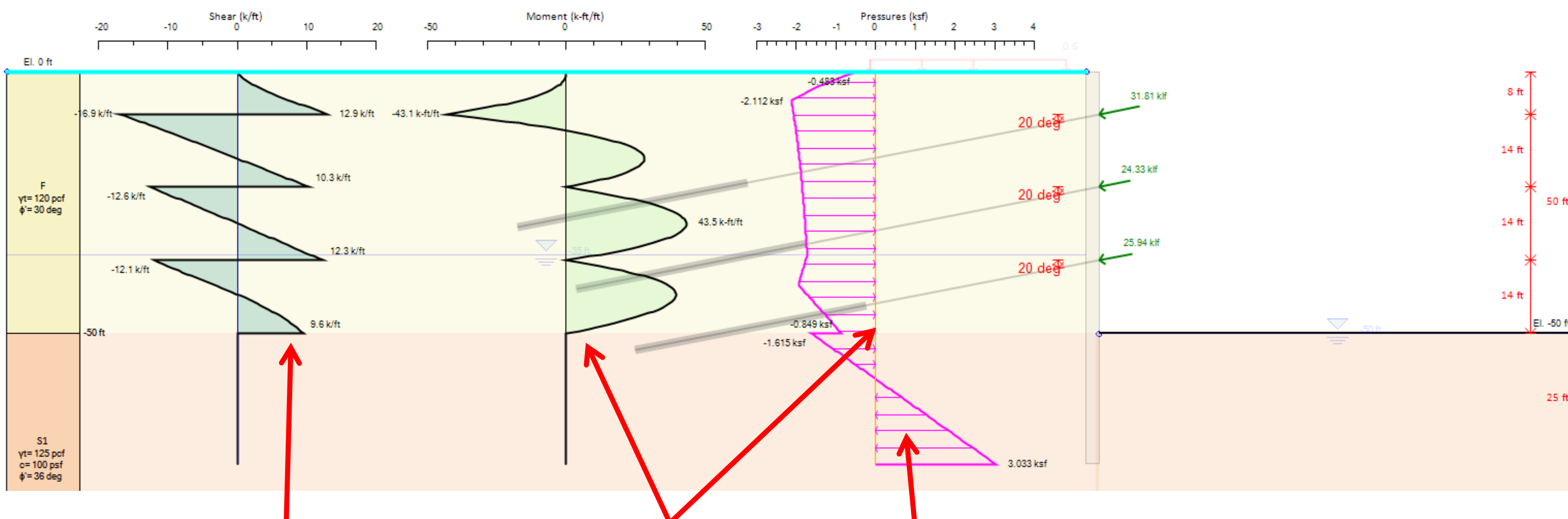
Available resistance Rx

$$FS.passive = \frac{Rx}{Fxb}$$

1.7 FHWA Simple Span Approach

- Pin support at excavation base, simple spans

Shear moments Net loading



Reaction for embedment Fxb

Virtual support

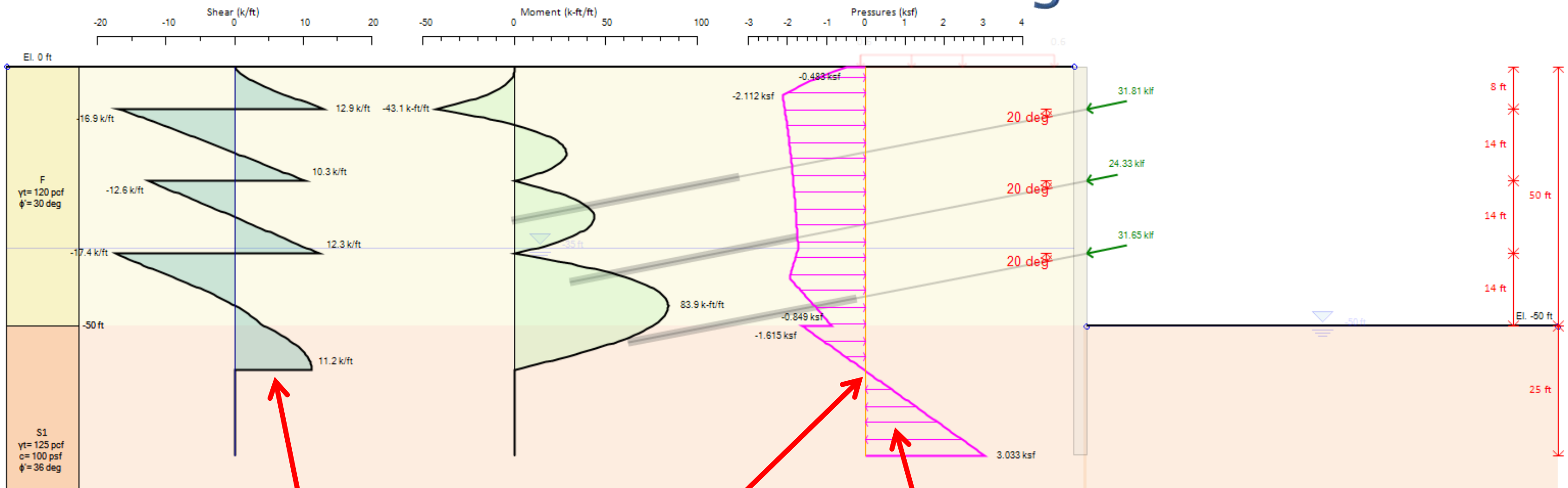
Available resistance Rx

$$FS.passive = \frac{Rx}{Fxb}$$

1.8 Modified FHWA-Blum

- Pinned supports – simple span
- Point of zero net soil shear below subgrade

Shear moments Net loading



Reaction for embedment Fxb

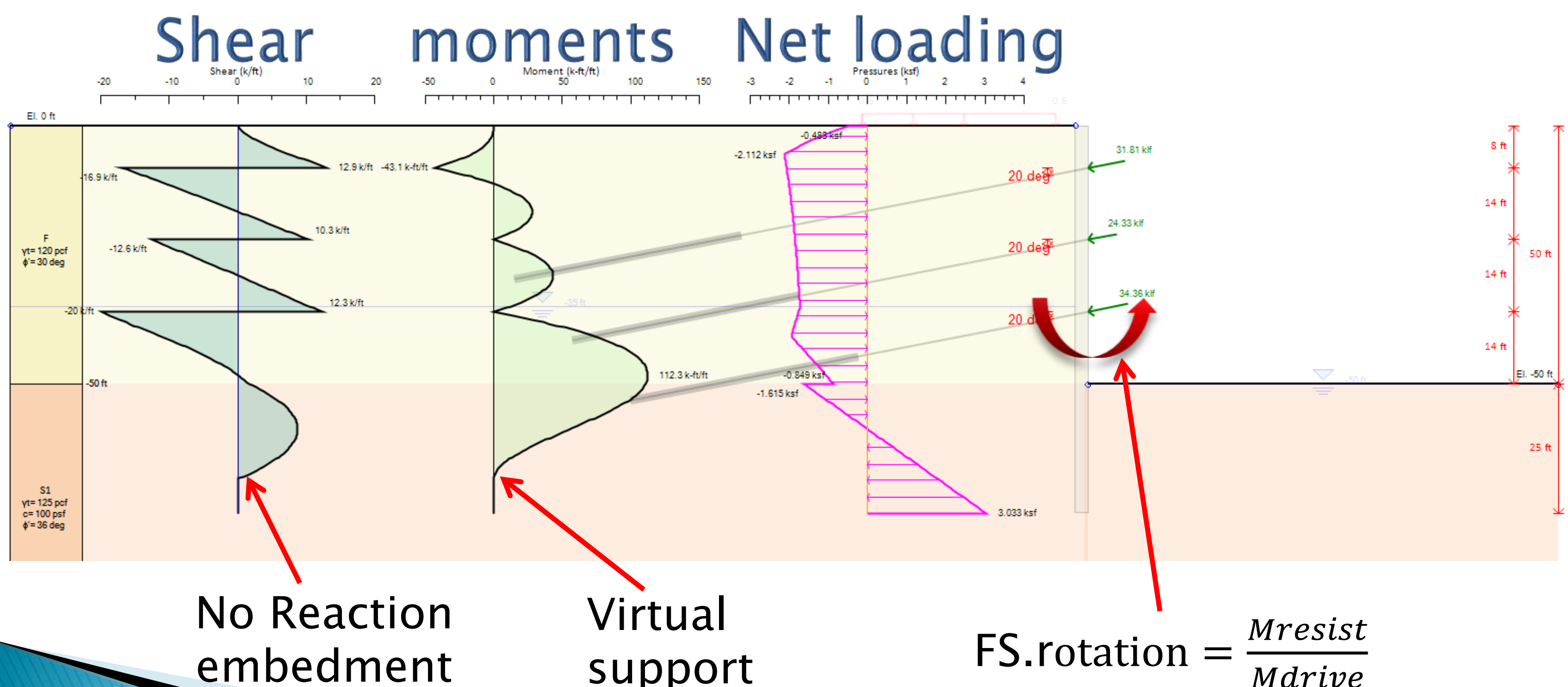
Virtual support

Available resistance Rx

$$\text{FS.passive} = \frac{Rx}{Fxb}$$

1.9 Caltrans method

- Pinned supports – simple span
- Base at point of zero moment below bottom support
- Shears and moments balance out



$$\text{FS.rotation} = \frac{Mresist}{Mdrive}$$

2.1 Design approach

- Allowable design – traditional way
- LRFD (Load and Resistance Factor Design)
  - AASHTO, US approach for Highway projects
- ULS (Ultimate Limit State Design)
  - Eurocode 7
  - Design approaches and cases

2.3 Allowable design

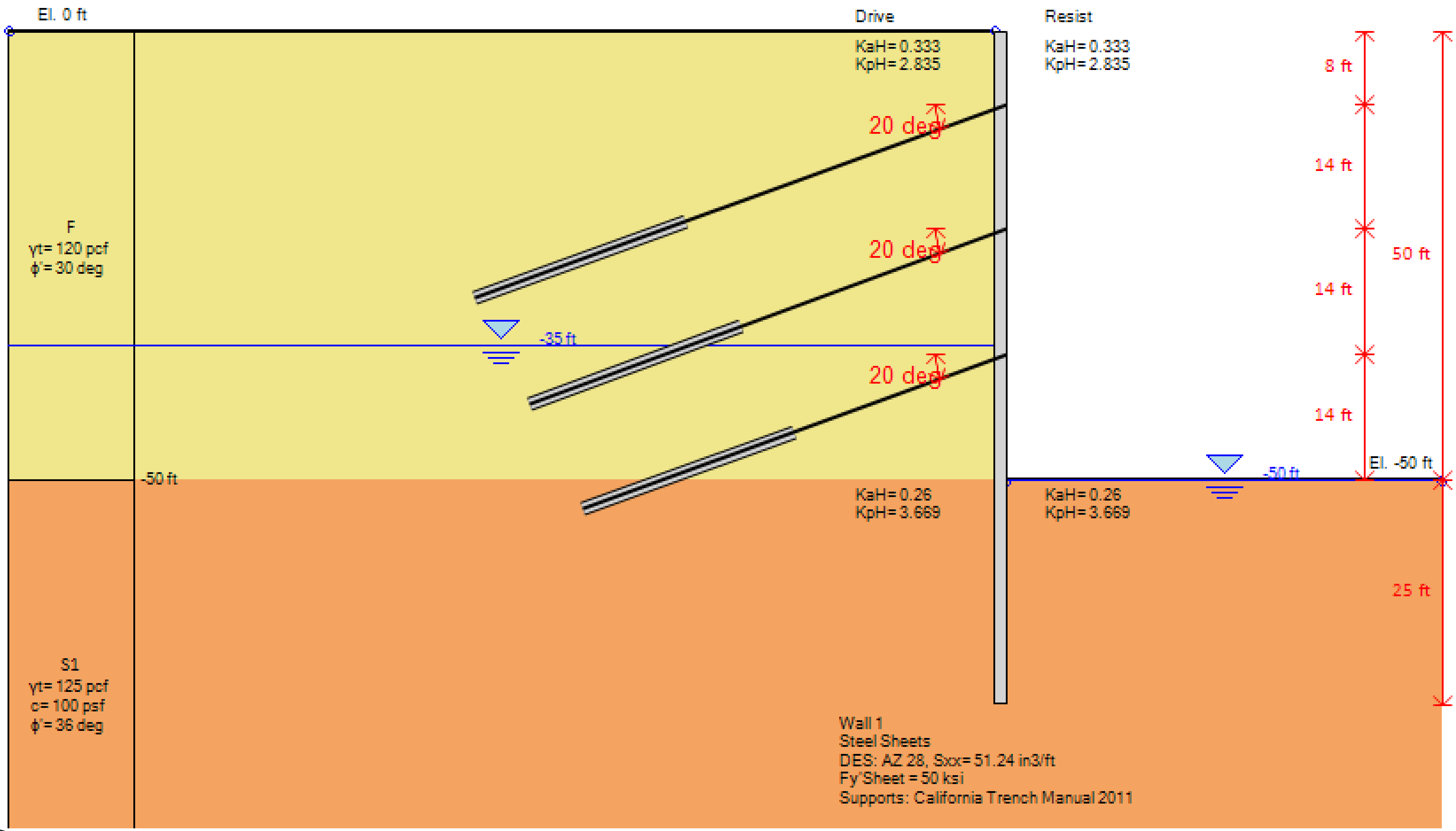
- Conservatively assume soil properties
- Determine lateral pressures on wall (soil, water, surcharges, etc.)
- Determine support reactions
- Determine wall embedment with an appropriate safety factor
- Determine bending moments on walls
- Determine shear forces on walls
- Design structural/geotechnical elements with a safety factor on their capacity

2.3.1 Allowable Design Example

- 50ft deep excavation (15m)
- 0–50ft F: sand, $\gamma = 120pcf$ $\phi = 30$ deg
- 50ft S1: sand, $\gamma = 125pcf$ $\phi = 36$ deg

  $c' = 100psf$
- Groundwater at -35ft $\gamma_w = 62.4pcf$
- Assume same permeability for both soils
- Ground is horizontal
- Tieback supports at 8ft, 22ft, and 36ft depth
- Tiebacks at 20 degrees from horizontal

2.3.2 Allowable Design Example



2.3.3 Allowable design example

- Assume FHWA apparent earth pressures
- Assume full passive pressures on resisting side
- No wall friction

Soil F

$$K_a = \frac{1 - \sin(\varphi)}{1 + \sin(\varphi)} = \frac{1 - \sin(30°)}{1 + \sin(30°)} = 0.333$$

$$K_p = \frac{1 + \sin(\varphi)}{1 - \sin(\varphi)} = \frac{1 + \sin(30°)}{1 - \sin(30°)} = 3.0$$

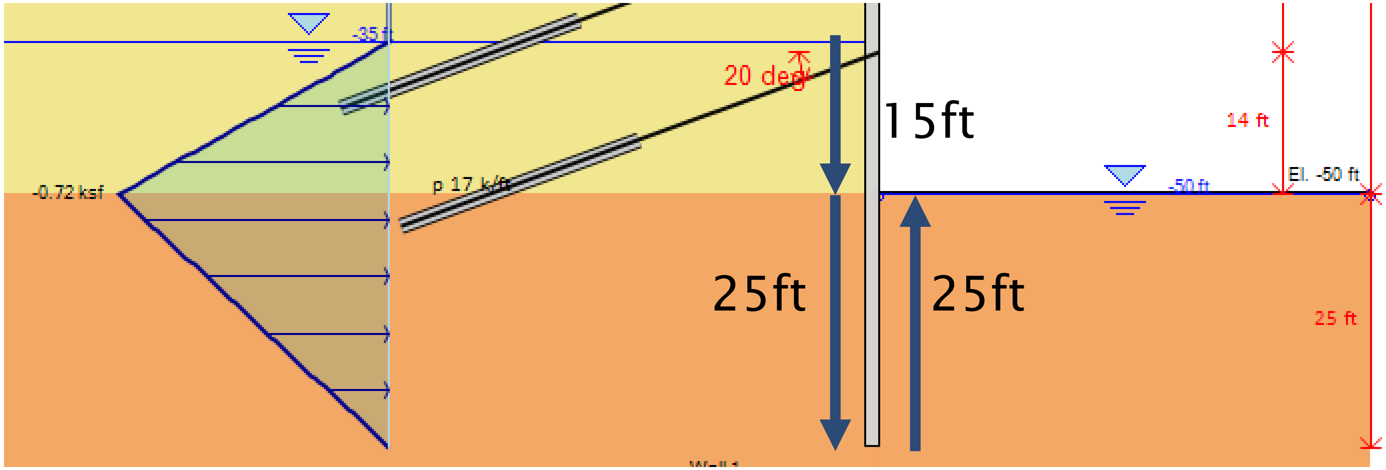
Soil S1

$$K_a = \frac{1 - \sin(\varphi)}{1 + \sin(\varphi)} = \frac{1 - \sin(36°)}{1 + \sin(36°)} = 0.26$$

$$K_p = \frac{1 + \sin(\varphi)}{1 - \sin(\varphi)} = \frac{1 + \sin(36°)}{1 - \sin(36°)} = 3.852$$

2.3.4 Establish water pressures

- 1D flow, because of same permeability
- Hydraulic head loss DH =(-35'-(-50')) =15ft
- Total travel path 15'+2 x 25'= 65'
- Hydraulic head gradient m= 15'/65' = 0.23077



2.3.5 Establish hydraulic heads

- Total hydraulic head at various locations

  $H_t = H_{initial}$ – Hyd. losses

  Hyd. losses = travel path x m

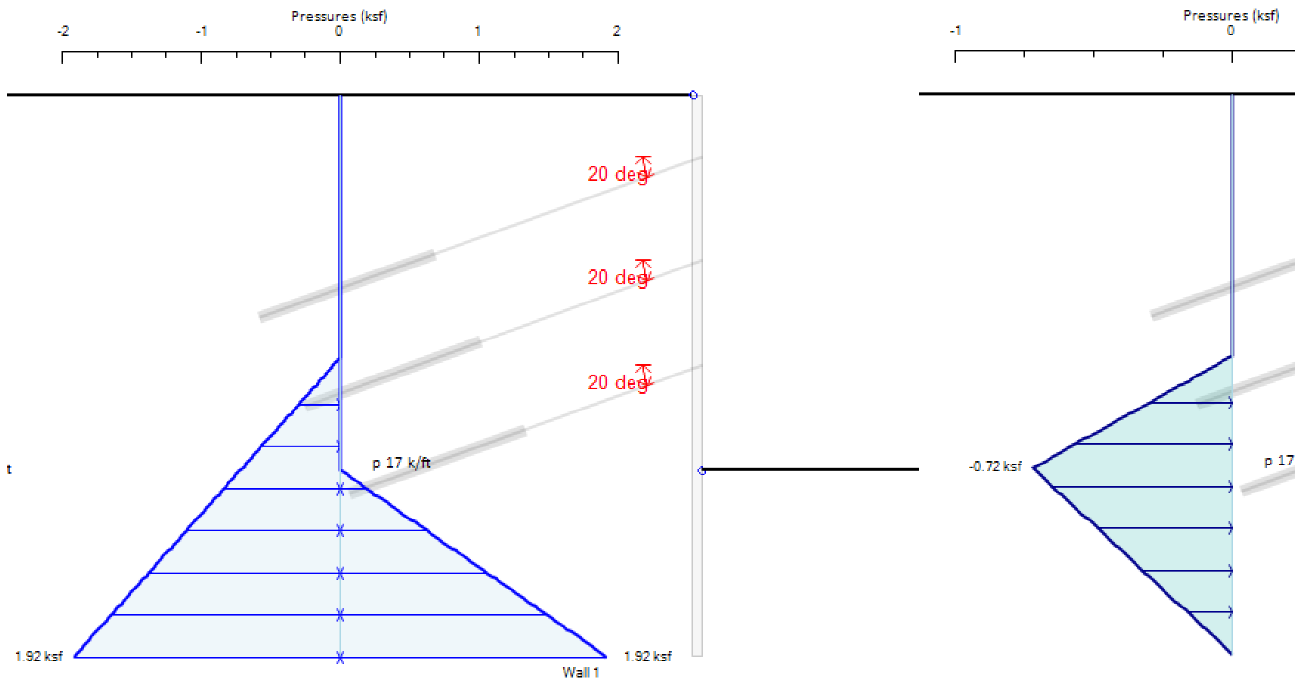
  Water pressure = $(H_t$ – El.) x γ_w

m= 0.23077

El. (ft)	Travel path L (ft)	Ht = L x m (ft)	Hu = Ht - El. (ft)	u = Hu x γw (ksf)
-35	0	-35.000	0.000	0
-50	15	-38.462	11.538	0.72
-75	40	-44.231	30.769	1.92
-50	65	-50.000	0.000	0

2.3.6 Establish net water pressures

- 0.72ksf at 15' below GWT

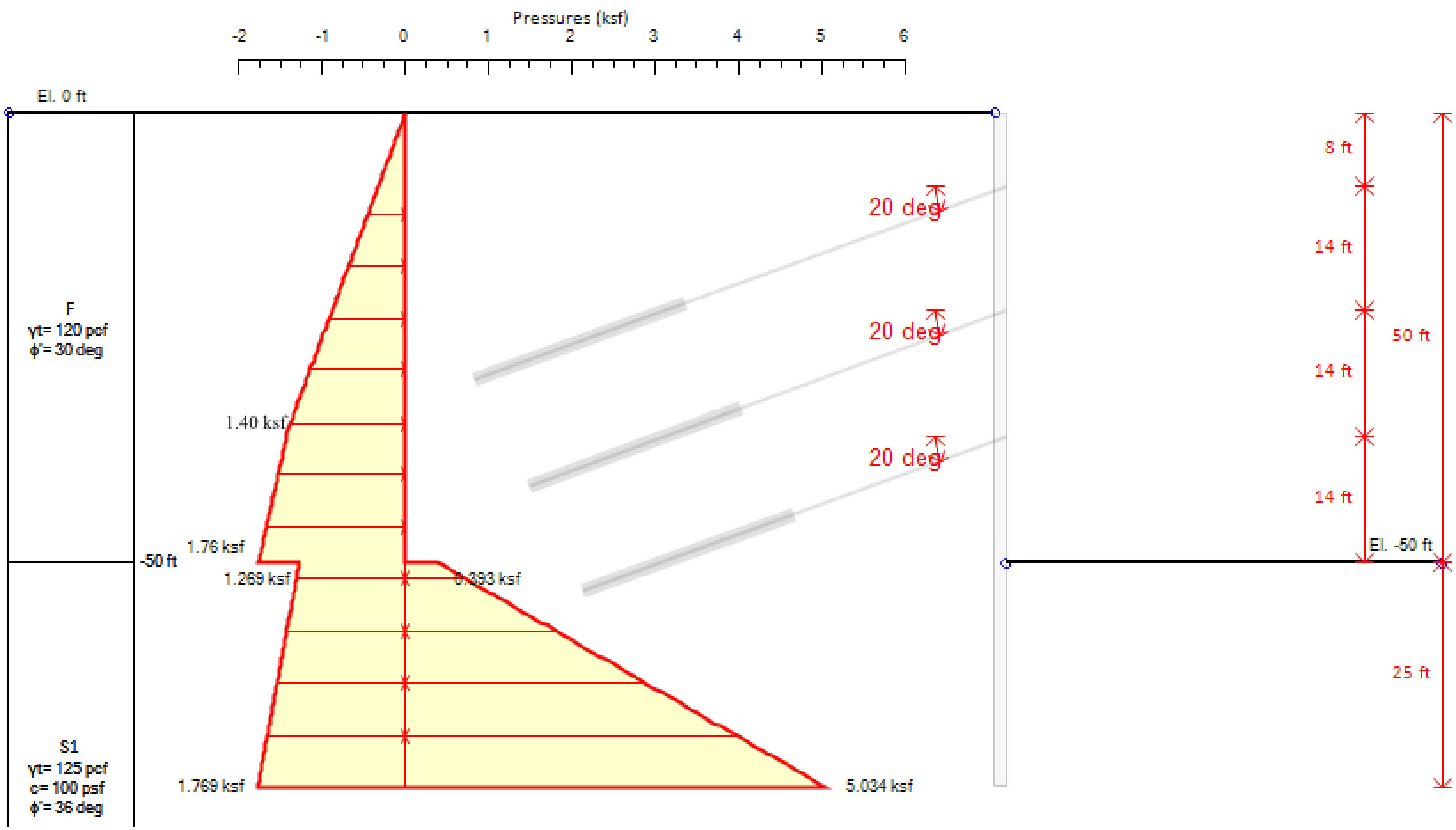


2.3.7 Active and Passive Pressures

	El. (ft)	γ (kcf)	$\sigma_{v.total}$ (ksf)	u (ksf)	$\sigma'_v = \sigma_{v.t} - u$ (ksf)	c' (ksf)	K_a -	$\sigma_{h.active} = K_a \sigma'_v - 2c'(K_a)^{0.5}$
F	0	0.12	0	0.000	0.000	0	0.333	0.000
	-35	0.12	4.2	0.000	4.200	0	0.333	1.400
	-40.667	0.12	4.88	0.272	4.608	0	0.333	1.536
	-50	0.12	6	0.720	5.280	0	0.333	1.760
S1	-50	0.125	6	0.720	5.280	0.1	0.2596	1.269
	-75	0.125	9.125	1.920	7.205	0.1	0.2596	1.769

	El. (ft)	γ (kcf)	$\sigma_{v.total}$ (ksf)	u (ksf)	$\sigma'_v = \sigma_{v.t} - u$ (ksf)	c' (ksf)	K_p -	$\sigma_{h.passive} = K_p \sigma'_v + 2c'(K_p)^{0.5}$
F								
S1	-50	0.125	0	0.000	0.000	0.1	3.852	0.000
	-75	0.125	3.125	1.920	1.205	0.1	3.852	4.249

2.3.8 Active & Passive Pressures



2.3.9 Integrate active pressures

- FHWA method -> Determine active thrust above excavation subgrade
- Total thrust = active x 1.3

From El. 0 to -35ft: $F_{H1} = \frac{1.4\,ksf\ x\ 35'}{2} = 24.5\,klf$

From El. -35 to -50ft: $F_{H2} = \frac{(1.4\,ksf + 1.76kf)x\ 15'}{2} = 23.7\,klf$

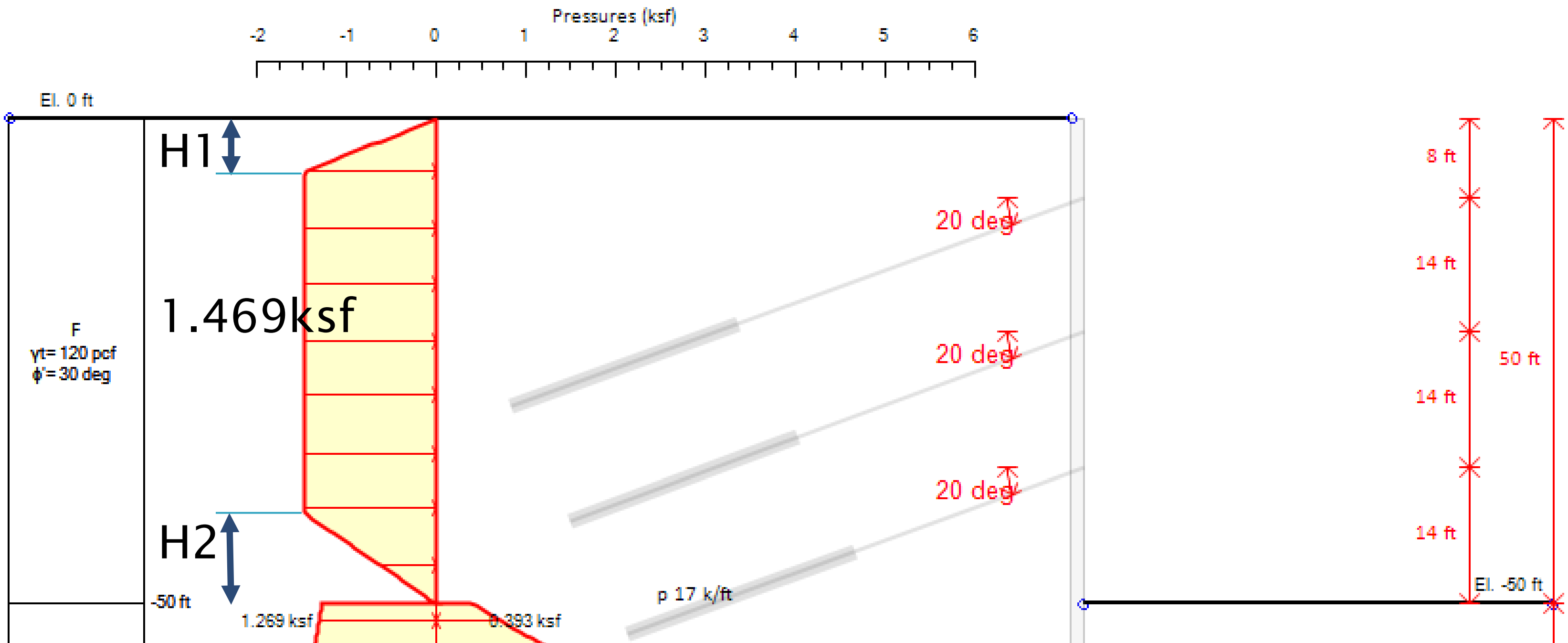
$$F_H = F_{H1} + F_{H2} = 48.2\,klf$$

$$F_{H.APP} = 1.3F_H = 62.66\,klf$$

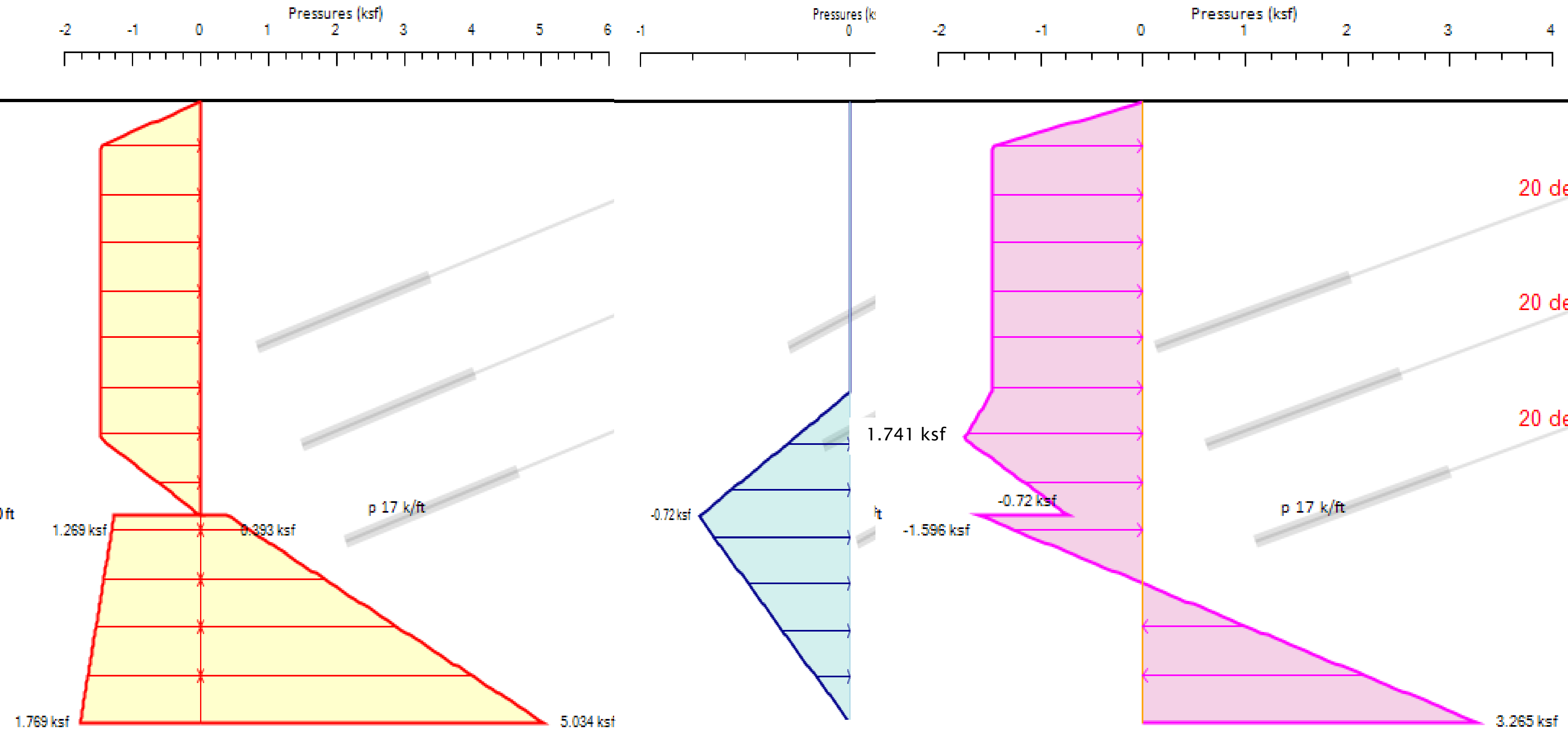
2.3.10 Establish Apparent FHWA

- $H_1 = 2/3 \times 8' = 5.333'$
- $H_4 = 2/3 \times 14' = 9.333'$
- $H = 50'$

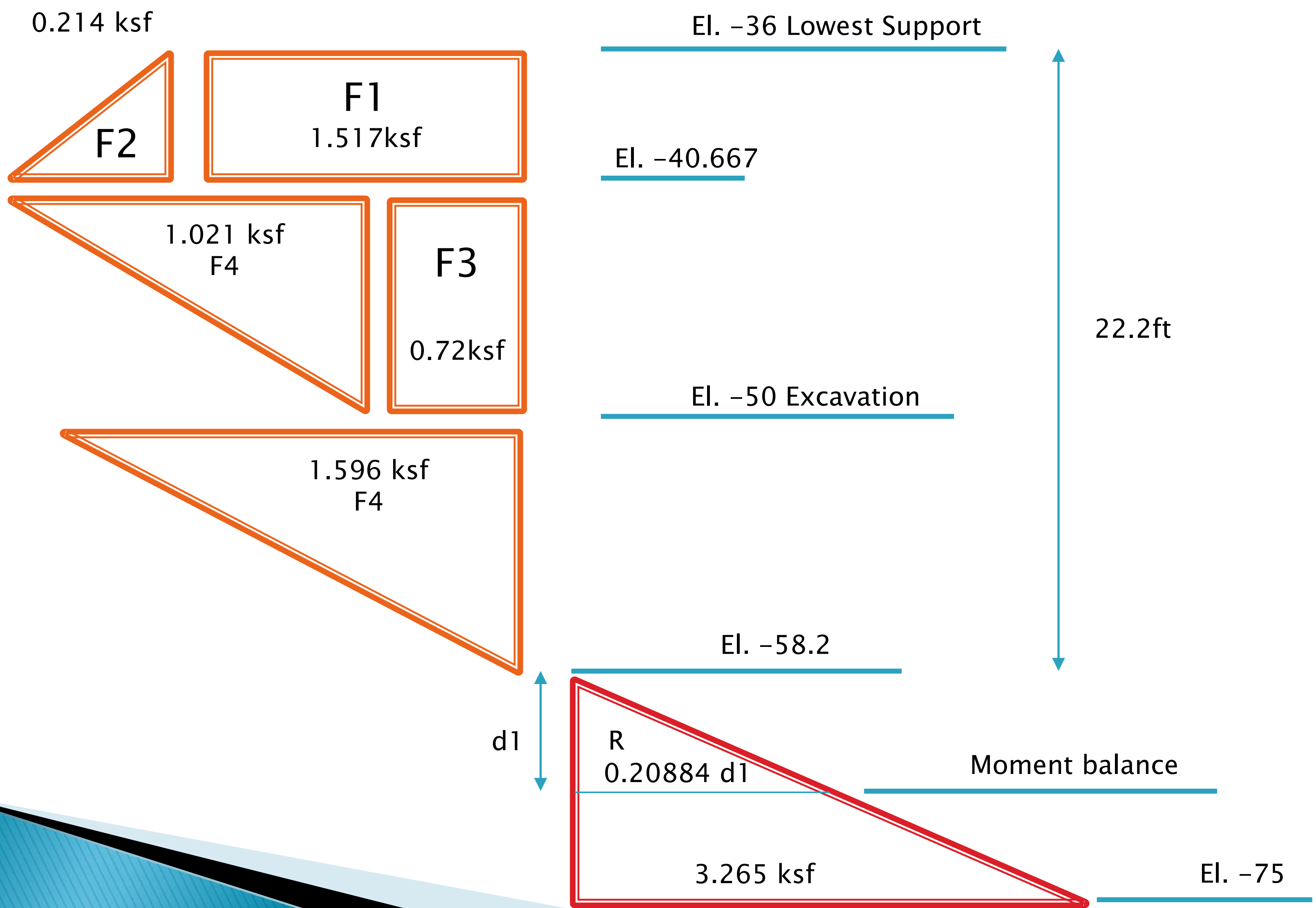
$$\sigma_{H.App} = \frac{2\,F_{H.APP}}{H + (H - H_1 - H_4)} = 1.469\ ksf$$



2.3.11 Establish net loading



2.3.12 Moment Equilibrium

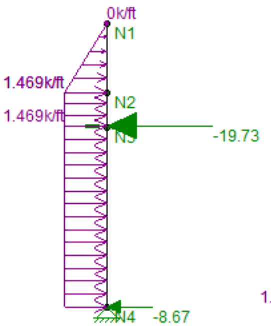


2.3.13 Establish Base Point

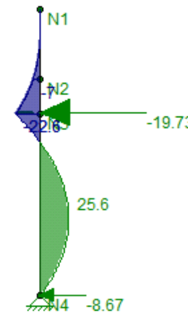
- Trial and error -> El. -67.007

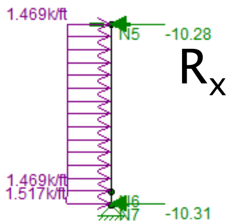
	Force	El1 (ft)	El2 (ft)	DZ (ft)	Force (k)	Moment arm (ft)	Moment (k-ft)
	F1	-36.000	-40.667	4.667	7.079	2.333	16.518
	F2	-36.000	-40.667	4.667	0.499	3.111	1.553
DRIVE	**F3**	-40.667	-50	9.333	6.720	9.333	62.720
	F4	-40.667	-50	9.333	4.765	7.778	37.059
	F5	-50	-58.2	8.200	6.544	16.733	109.496
							227.347
RESIST	**R**	-58.2	-67.007	8.807	8.099	28.071	227.347
						Moment Balance	0.000

2.3.14 Divide Wall into Beams

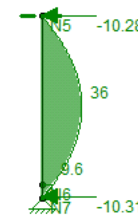


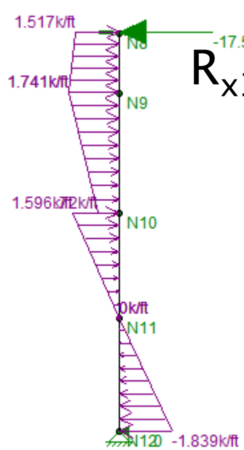
$R_{x1} = 19.73$ klf



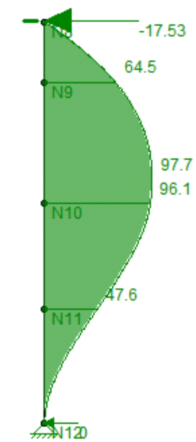


$R_{x2} = 8.67$ klf $+ 10.28$ klf $= 18.95$ klf

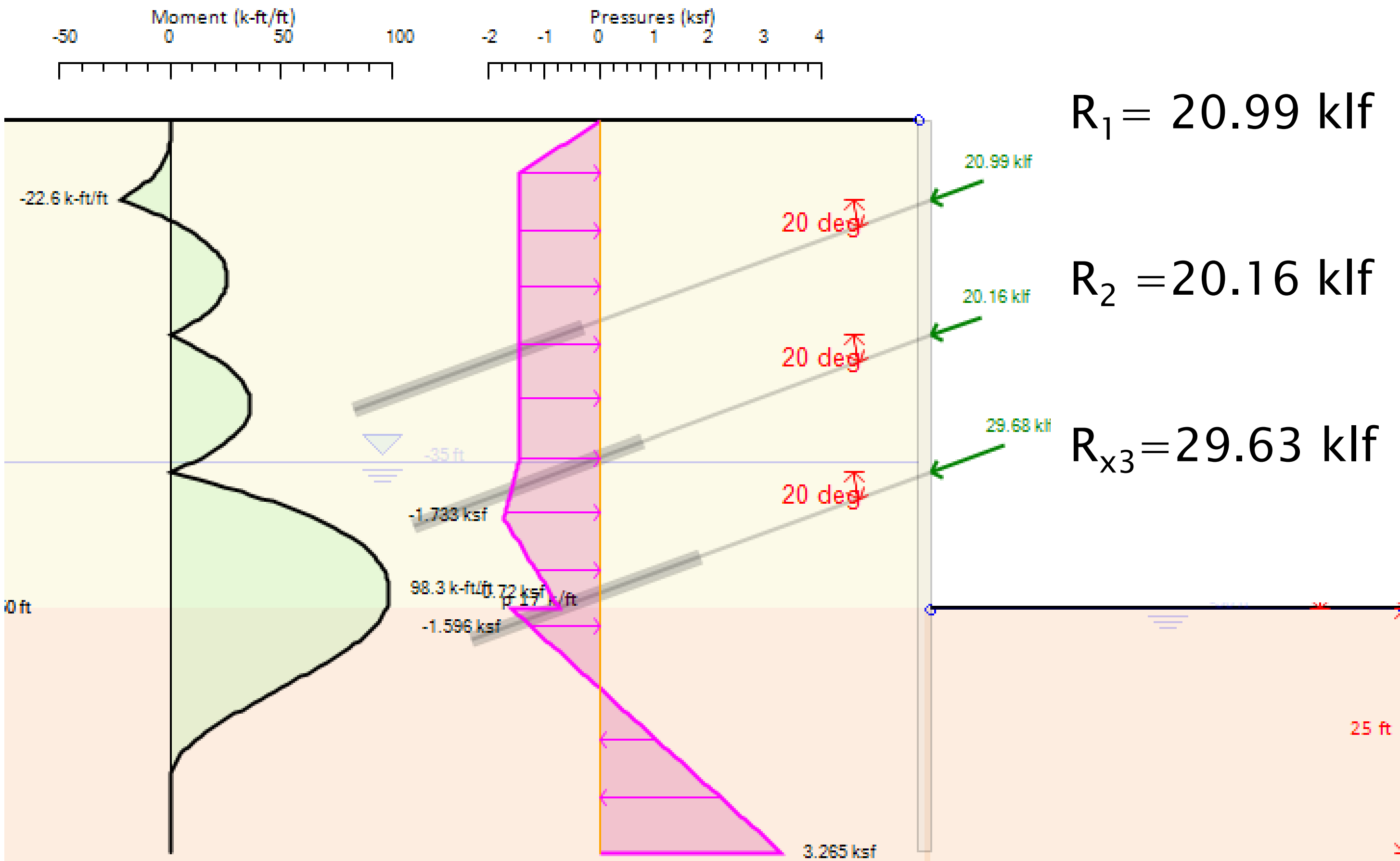




$R_{x3} = 10.31$ klf $+ 17.53$ klf $= 27.84$ klf



2.3.15 Model models & Reactions

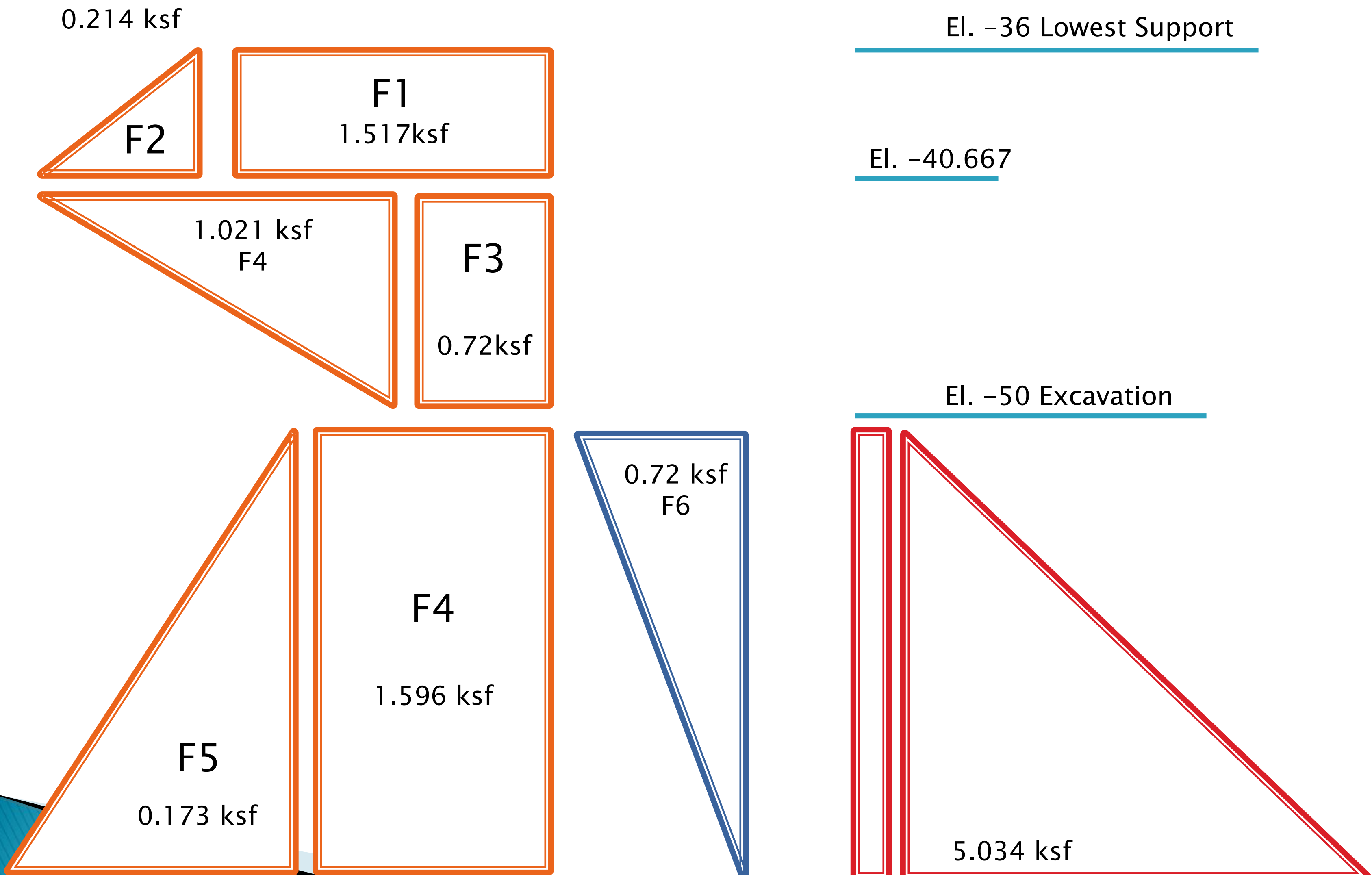


$R_1 = 20.99$ klf

$R_2 = 20.16$ klf

$R_{x3} = 29.63$ klf

2.3.16 Wall embedment FS



2.3.17 Wall Embedment

	Force	El1 (ft)	El2 (ft)	DZ (ft)	Force (k)	Moment arm (ft)	Moment (k-ft)
	F1	-36.000	-40.667	4.667	7.079	2.333	16.52
	F2	-36.000	-40.667	4.667	0.499	3.111	1.55
	F3	-40.667	-50	9.333	6.720	9.333	62.72
DRIVE	**F4**	-40.667	-50	9.333	4.765	7.778	37.06
	F5	-50	-75	25.000	2.163	30.667	66.32
	F6	-50.000	-75	25.000	39.900	26.500	1057.35
	Fw	-50	-75	25.000	9.000	17.667	159.00
						M_{drive}=	1400.52
	R1	-50	-75.000	25.000	9.825	26.500	260.36
RESIST	**R2**	-50	-75.000	25.000	58.013	30.667	1779.05
						M_{resist}=	2039.41
						FS.rotation=	1.456

- For FS=1.3, Required wall L=73ft d=23ft

2.3.18 All. Wall bending design

- Bending moment M_{max}= 98.3 k–ft
- f_y = 50 ksi
- f_{allow}= 0.6 x 50 ksi = 30 ksi
- If axial load is ignored!
- Required Elastic Section modulus =>

  $Sxx = M_{max} / f_{allow} = 39.32 \text{ in}^3/\text{ft}$

2.3.19 All. ground anchor design

- Assume presumptive ULT bond resistane
- Safety factor of $FS_{geo} = 2.0$
- Assume fixed bond diameter (6inch)
- Determine length
- Allowable Geotechnical Capacity:

  $F_{cap.GEO} = L \times q \times d \times \pi / FS_{geo}$
- Allowable Structural Capacity:

  $F_{cap.STR} = \alpha A_s f_y$

  $\alpha = 0.6$, $f_y = 270$ ksi (1862MPa)

3.1.1 LRFD

- Adopted in USA, AASHTO
- Load combinations
- Multiply loads by a load factor
- Divide resistances by resistance factor
- Many DOT's find it very difficult to use
- Engineers find it too cumbersome
- Inconsistent results can be produced due to lack of guidance and proper calibration

3.1.2 AASHTO LRFD Overview

- Safety factor on earth loads (ES)

  γp = 1.35 for Apparent Earth Pressures (AEP)

  γp = 1.50 for Active earth pressures
- Resistance factors for passive resistance Φ= 0.75
- Calibrated on limit-equilibrium analysis methods
- No guidance on non-linear analysis….

3.1.3 AASHTO Load Factors

Table 1. Load combinations and load factors (from AASHTO table 3.1.4.1)

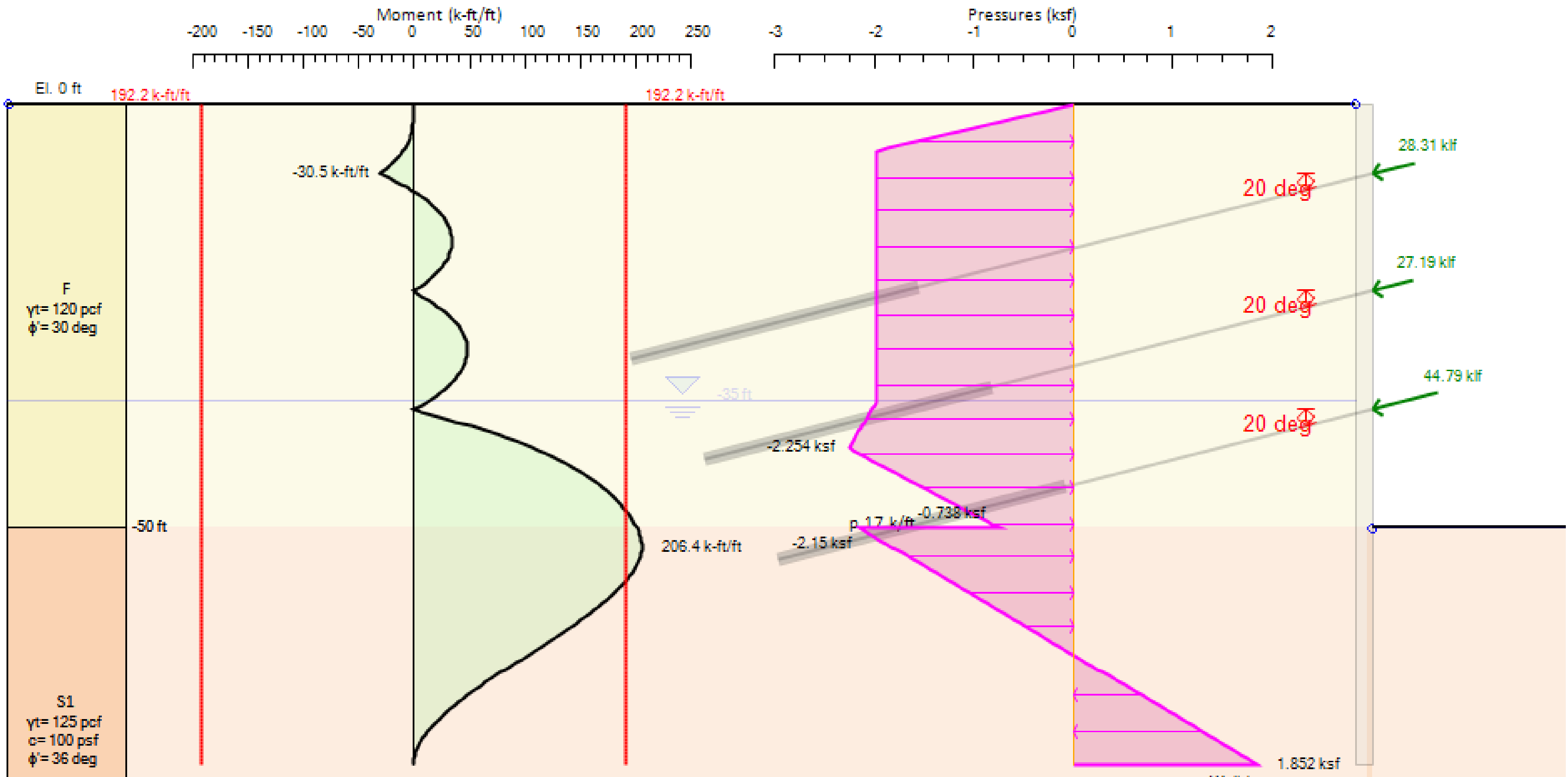
Load Combination Limit State	DC DD DW EH EV ES EL PS CR SH	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	EQ
Strength I	γ_P	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-
Strength II	γ_P	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-
Strength III	γ_P	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-
Strength IV	γ_P	-	1.00	-	-	1.00	0.50/1.20	-	-	-
Strength V	γ_P	1.35	1.00	0.40	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-
Extreme Event I	γ_P	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00
Extreme Event II	γ_P	0.50	1.00	-	-	1.00	-	-	-	-

Table 2. Factors from permanent loads γ_p (from AASHTO Table 3.1.4.2)

	Load factor	
EH: Horizontal earth pressure	**Maximum**	**Minimum**
• Active	**1.50**	**0.90**
• At-rest	**1.35**	**0.90**
• AEP for anchored walls	**1.35**	**N/A**

3.2.0 Example with AASHTO LRFD

- Strength 1a Load Combination



3.2.1 AASHTO LRFD – Wall

- Required wall length L= 78ft, d= 28ft
  Increase 22% over ASD
- Wall moment M= 206.4 k-ft/ft
- $F_{y.des}$= 0.9 x 50 ksi = 45 ksi
- If axial load is ignored!
- Required Elastic Section modulus =>
  $S_{xx} = M_{max} / f_{y.des} = 55.04\ in^3/ft$

Increase 40% over allowable stress design

3.2.2 AASHTO LRFD – Anchors

- Force increases to 44.8 klf
- $\phi=0.8$
- Results in 13% increase vs. allowable stress

4.0 Eurocode 7 ULS approach

- **EQU**: Loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance.
- **STR**: Internal failure or excessive deformation of the structure or structural elements, ... in which the strength of structural materials is significant in providing resistance.
- **GEO**: Failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance.
- **UPL**: Loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions.
- **HYD**: Hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients.

4.0.1EC7 Design Approaches

- **Design Approach 1**:
  Combination 1: *A1* "+" *M1* "+" *R1* (DA-1/1)
  Combination 2: *A2* "+" *M2* "+" *R1* (DA-1/2)
- **Design Approach 2**:
  Combination: *A1* "+" *M1* "+" *R2* (DA-2)
- **Design Approach 3**:
  Combination: (*A1** or *A2*†) "+" *M2* "+" *R3* (DA-3)

*on structural actions, †on geotechnical actions

Where "+" implies: "to be combined with".

4.0.2 EC7 Comments

- More comprehensive approach over LRFD
- Different design approach methods may result in very different results
- Impractical to perform calculations by hand
- Numerous partial factors, most practitioners confused.

4.0.3 EC7 Factors

Table 1. Partial factors on actions (γ_F) or the effects of actions (γ_E)

Action		**Symbol**	**Set**	
			A1	A2
Permanent	Unfavorable	γ_G	1.35	1.0
	Favorable		1.0	1.0
Variable	Unfavorable	γ_Q	1.5	1.3
	Favorable		0	0

Table 2. Partial factors for soil parameters (γ_M)

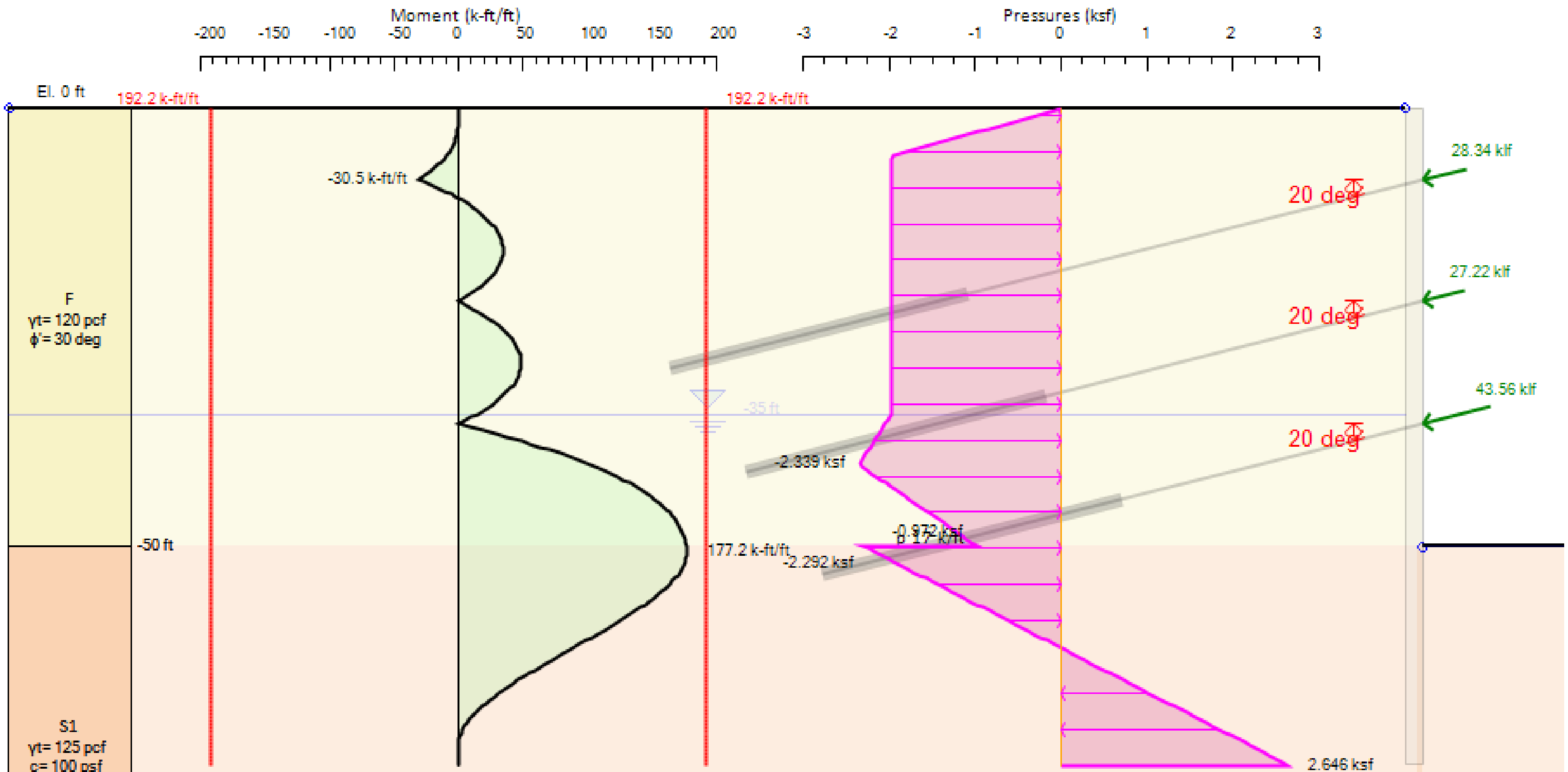
Soil parameter	**Symbol**	**Set**	
		M1	M2
Angle of shearing resistance (applied to tan φ)	γ_{ϕ}'	1.0	1.25
Effective cohesion	γ_c'	1.0	1.25
Undrained shear strength	γ_{cu}	1.0	1.4
Unconfined strength	γ_{qu}	1.0	1.4
Weight density	γ_{γ}	1.0	1.0

Table 3. Partial resistance factors for earth resistance, pre-stressed anchors (γ_R)

Resistance	**Symbol**	**Set**			
		R1	R2	R3	R4
Earth resistance	$\gamma_{R;e}$	1.0	1.4	1.0	-
Ground anchors (temporary)	$\gamma_{a;t}$	1.1	1.1	1.0	1.1
Ground anchors (permanent)	$\gamma_{a;p}$	1.1	1.1	1.0	1.1

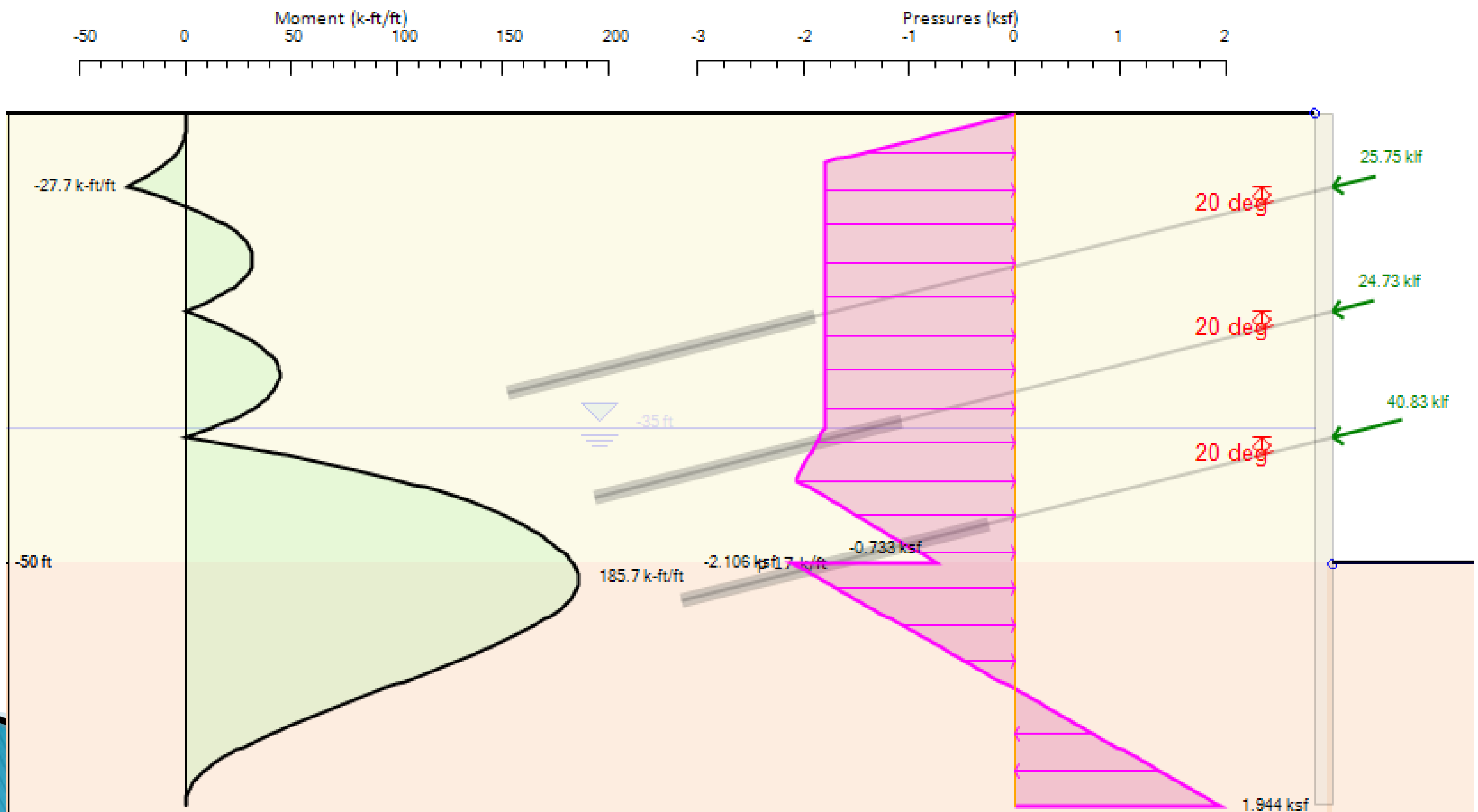
4.1.1 EC7-DA1/Comb. 1

- 1.35 factor on apparent pressures
- Moment 177.2k-ft/ft -> S_{xx}= 47.25 in^3 – 20% greater
- Anchor: 43.56 klf -> 10% more steel



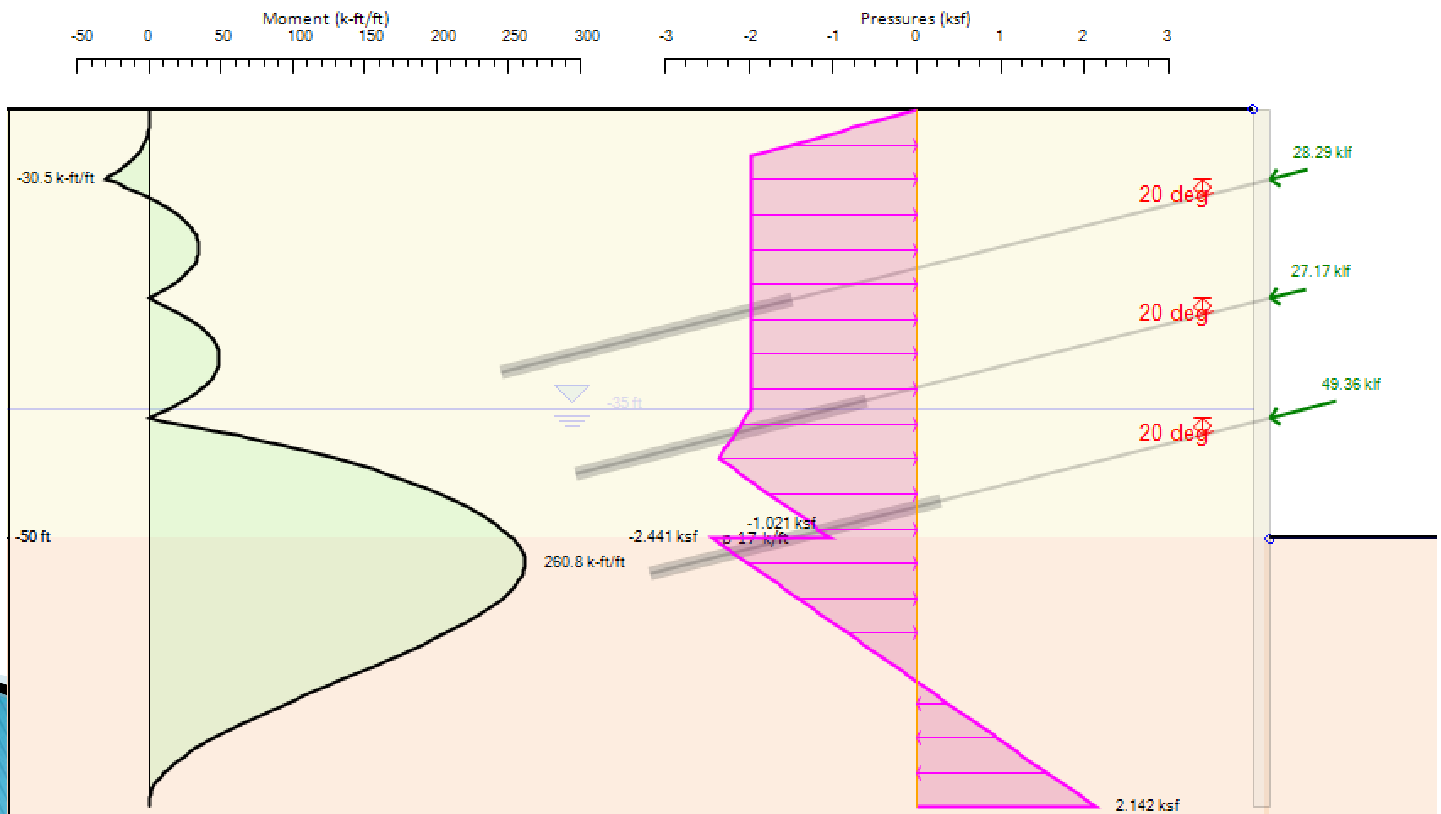
4.1.2 EC7-DA1/Comb 2.

- Geotechnical comb. – reduce soil strength
- Moment 185.7k-ft/ft ->S_{xx}= 50 in^3 – 27% greater
- Anchor: 40.83 klf -> 3% more steel
- Required wall embedment 27ft – 17% increase



4.1.3 EC7-DA2

- 1.35 on apparent, 1.4 on favorable
- Moment 260.8k-ft/ft ->S_{xx}= 69.6 in^3 – 76% greater
- Anchor: 49.36 klf -> 25.4% more steel
- Required wall embedment 31.5ft – 82% increase



5. Compare requirements

Case	Moment	S_{xx}	Increase in S_{xx}	Wall Embedment		Supports	Area of steel	
	(k-ft)	(in^3/ft)	(%)	(ft)	(%)	(klf)	(in^2/ft)	(%)
ASD	98.3	39.32	0.0%	23	0.0%	29.62	0.183	0.0%
AASHTO LRFD Ia	206.4	55.04	40.0%	28	21.7%	44.8	0.207	13.4%
EC7 DA1/1	177.2	47.25	20.2%	23	0.0%	43.6	0.202	10.4%
EC7 DA1/2	185.7	49.52	25.9%	27	17.4%	40.8	0.189	3.3%
EC7 DA2	260.8	69.55	76.9%	31.5	37.0%	49.4	0.229	25.1%

- Ultimate moments 180% to 265% of ASD
- Reactions 137% to 167%

6. Elastoplastic & LRFD or EC7

- EC7 & LRFD calibrated on conventional methods
- Performance of deep excavations depends on construction staging.
- So if you are not getting realistic displacements in an analysis, or a realistic response are EC7 or LRFD results valid for design?

6.1 How to apply LRFD in ELPL analyses

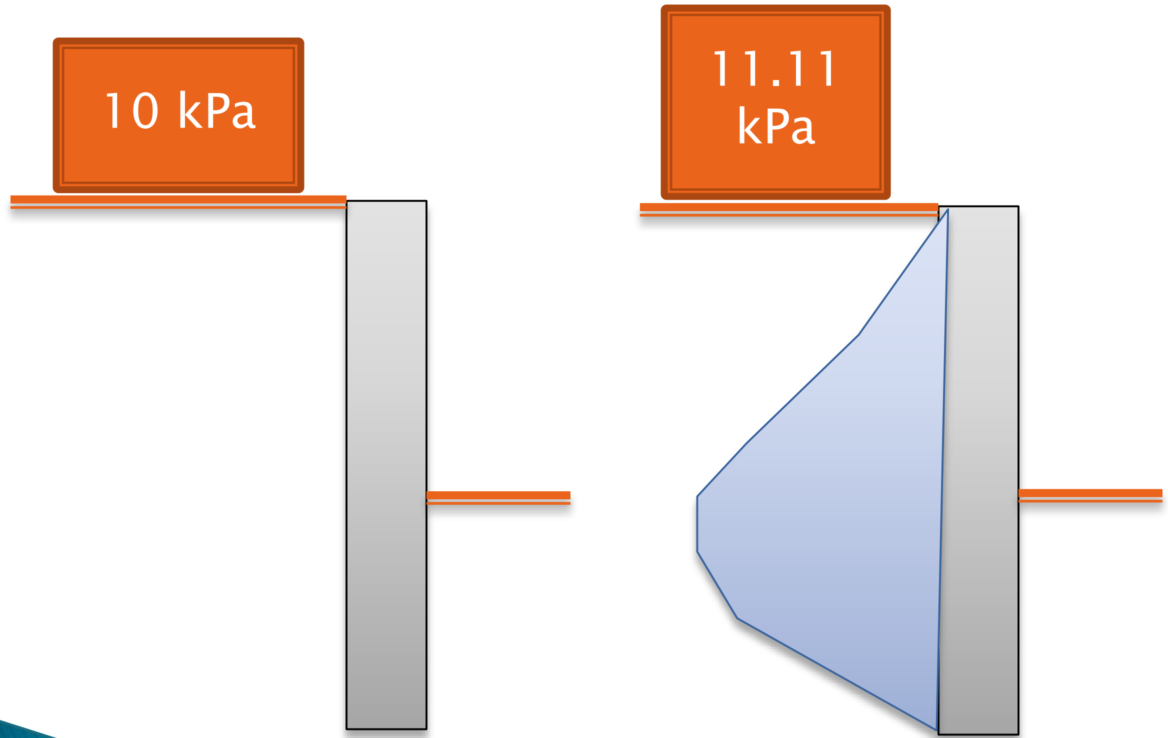
- One approach, adjust Ka, Kp, c' or Su to produce equivalent effect
- Multiply surcharges by load factors
- AASHTO LRFD does not specifically mention what to do….
- Some engineers, standardize initial the analysis by the unfavorable lateral earth pressure factor (1.35), and then multiply results times that factor at the very end

6.2 EC7 and Elastoplastic Analyses

- EC7 in allows the effect of actions to be multiplied.
- In some combinations (DA 1/1) this would lead to a reasonable approach
- Design Approaches that have soil strength adjusted (M2) may produce unrealistic wall response
- M2 combinations may still show where a structure is sensitive
- M2 combinations might not work well (compared to ASD) if you have clays (especially softer clays)

6.2. Example of "effect" on actions

- DA 1/1 – LF= 1.5 Permanent, 1.35 Earth
- 10 kpa x 1.5/1.35 = 11.11 kPa



- x 1.35 = Design Values

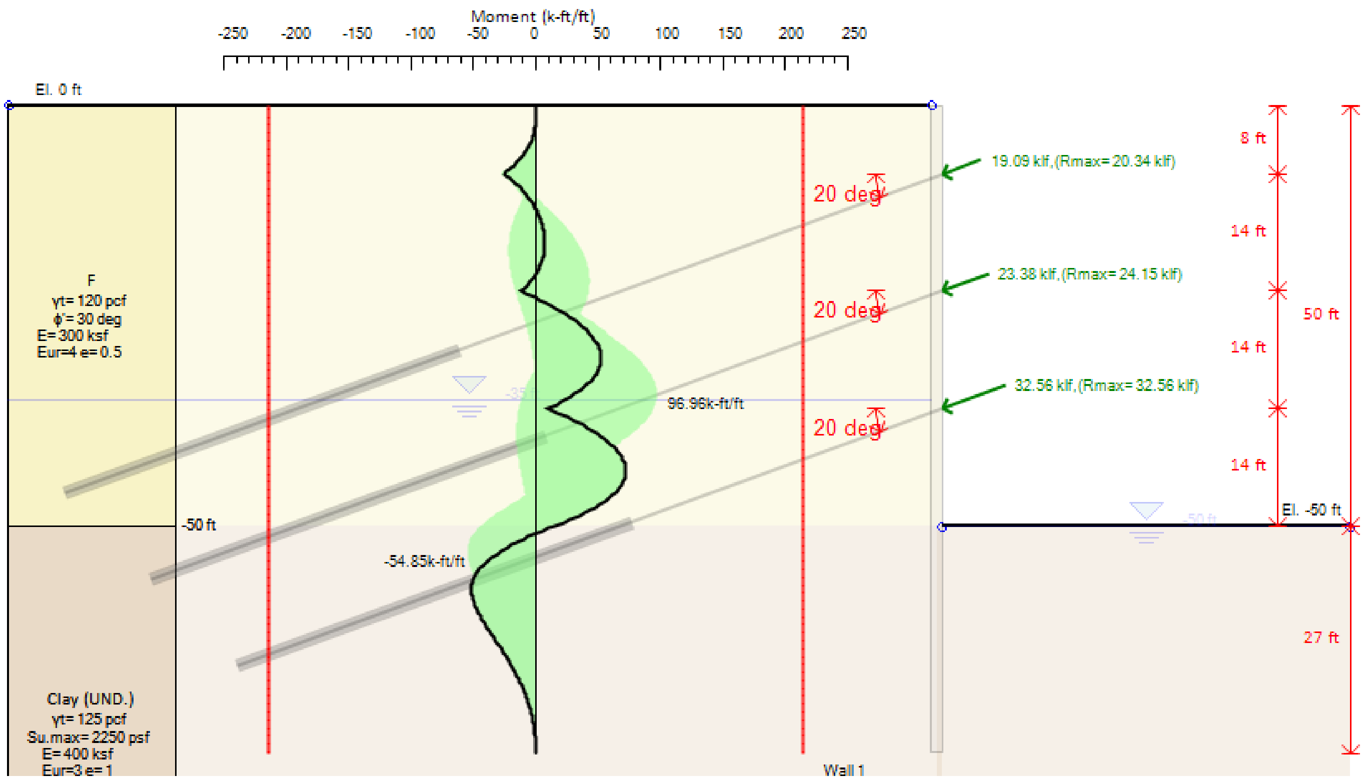
6.3 Compare NL Results

Design Case	Wall Displacement	Max. Wall Moment		Max. Support Reaction	
		Moment	Percent of service case	Reaction	Percent of service case
	(in)	(k-ft/ft)	-	(k/ft)	-
Service case	1.63	91.1	100.0%	31.8	100.0%
AASHTO LRFD (2010): Strength Ia	6.13	198.7	218.2%	43.4	136.3%
EC7, 2007: DA-1, Comb. 1: A1 + M1 + R1	1.63	122.9	135.0%	43.0	135.0%
EC7, 2007: DA-1, Comb. 2: A2 + M2 + R1	5.07	177.9	195.3%	41.3	129.9%
EC7, 2007: DA-2: A1 + M1 + R2	2.53	156.9	172.3%	46.2	145.2%

- Ultimate moments 135% to 218.2% of SLS
- Reactions 130% to 145%

6.4 Let's look at clays

- Lower soil replace with clay Su= 2250 psf



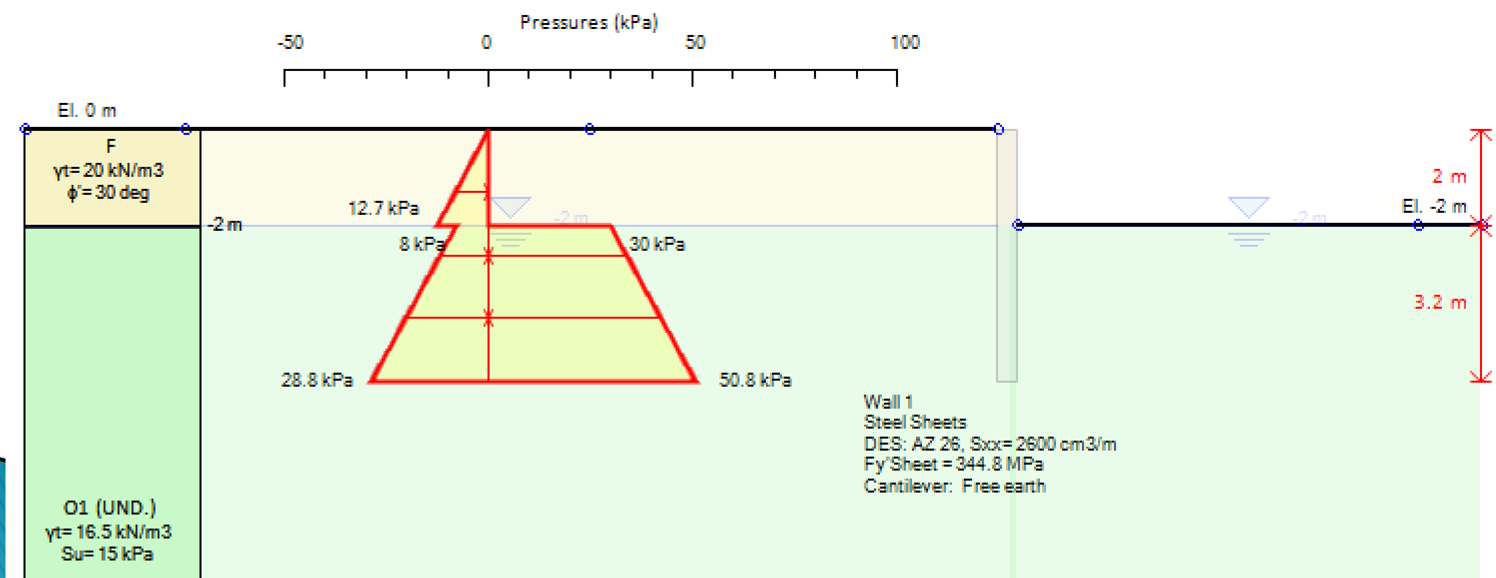
6.5 Clays (medium stiff)

Design Case	Wall Displacement	Max. Wall Moment		Max. Support Reaction	
		Moment	Percent of service case	Reaction	Percent of service case
	(in)	(k-ft/ft)	-	(k/ft)	-
Service case	1.88	97.0	100.0%	32.6	100.0%
AASHTO LRFD (2010): Strength Ia	8.04	205.8	212.3%	40.0	122.9%
EC7, 2007: DA-1, Comb. 1: A1 + M1 + R1	1.88	130.9	135.0%	44.0	135.0%
EC7, 2007: DA-1, Comb. 2: A2 + M2 + R1	7.29	177.9	183.5%	40.5	124.4%
EC7, 2007: DA-2: A1 + M1 + R2	2.49	155.7	160.6%	44.0	135.2%

- Ultimate moments 135% to 212.2% of SLS
- Reactions 122% to 135% (support prestress makes a difference)

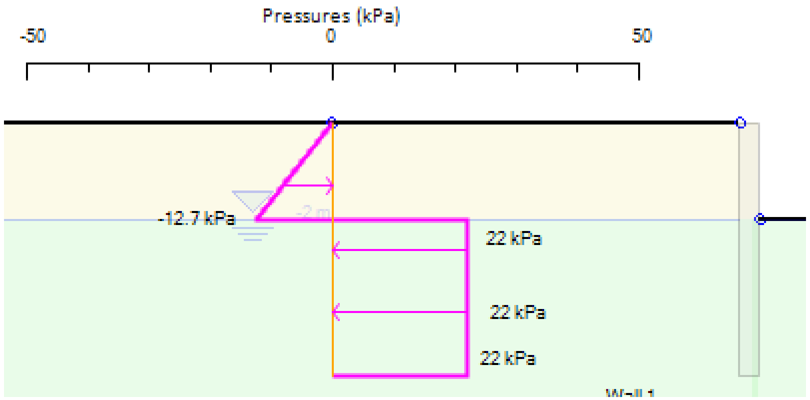
6.6.1 Let's look at soft clays

- 2.0m excavation (6.5ft)
- 2.0m fill, 30 degrees, 19 kN/m^3
- Soft clay below S_u= 15kPa (313 psf)
- Free earth method, required wall embedment 3.2m (FS>1.5)

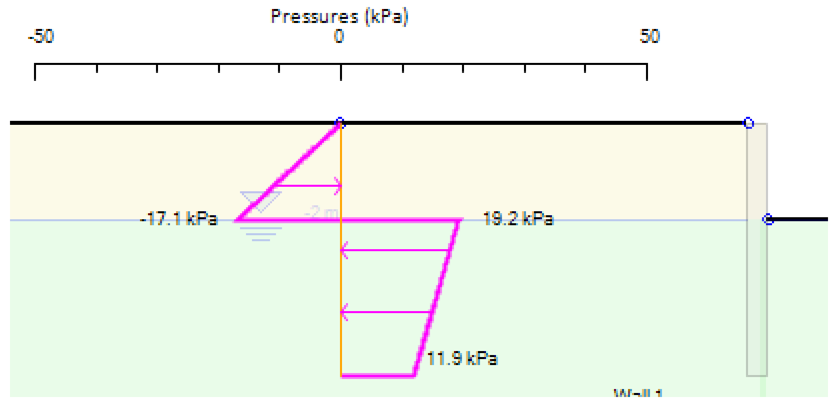


6.6.2 Soft clay example

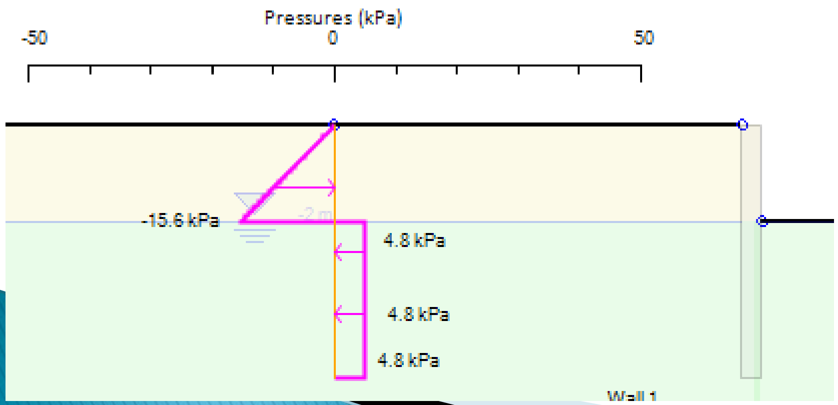
SLS (3.2m)



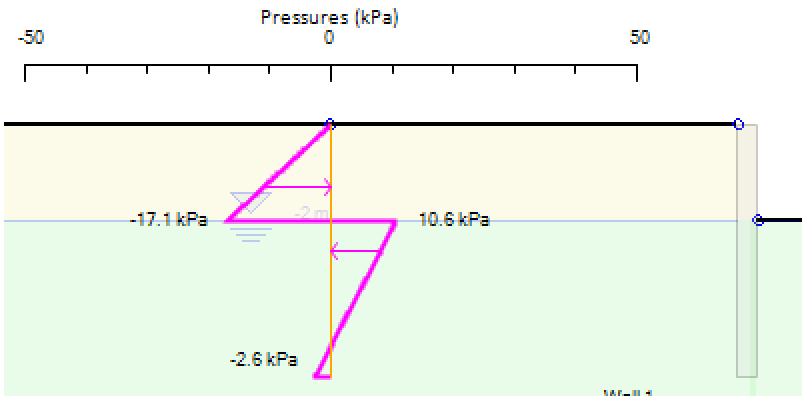
EC7 DA 1/1 (4.5m)



EC7 DA 1/2 (12.5m+)

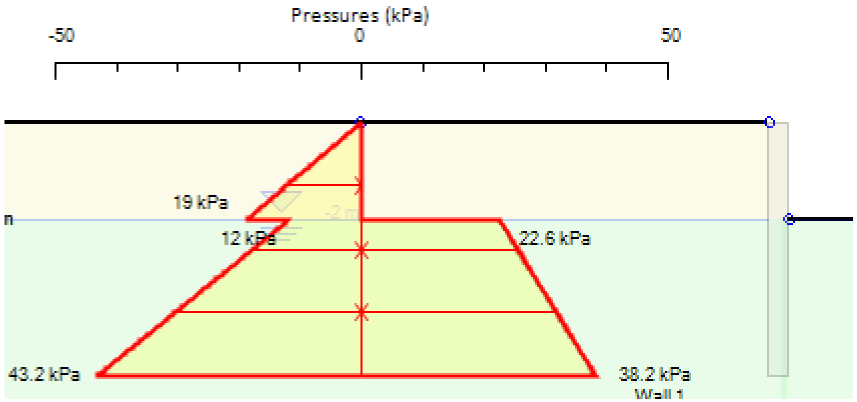


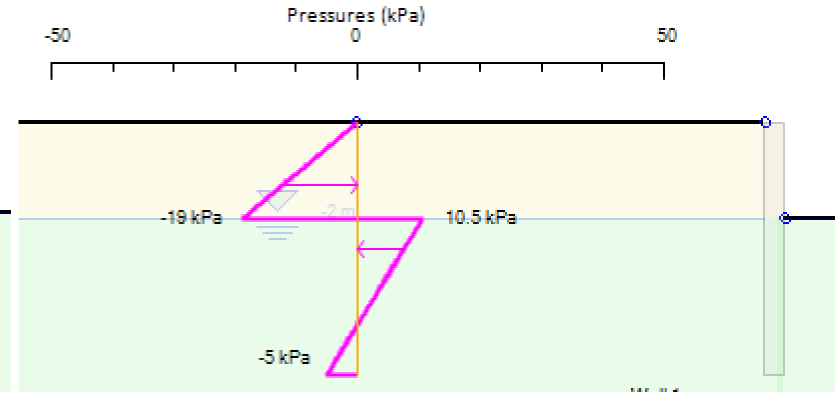
EC7 DA 2 (Unstable)



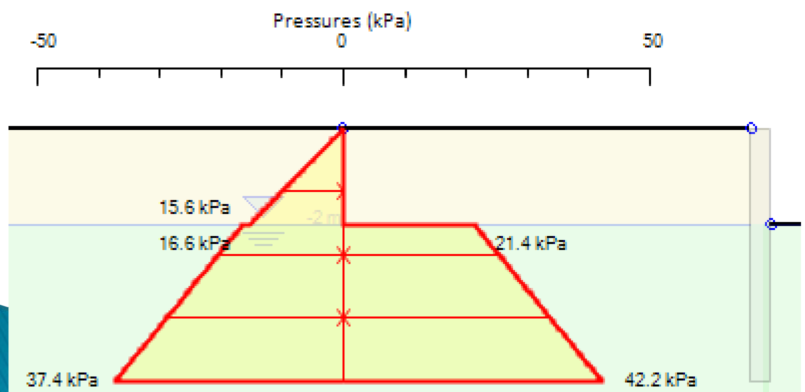
6.6.3 Soft clay example cont.

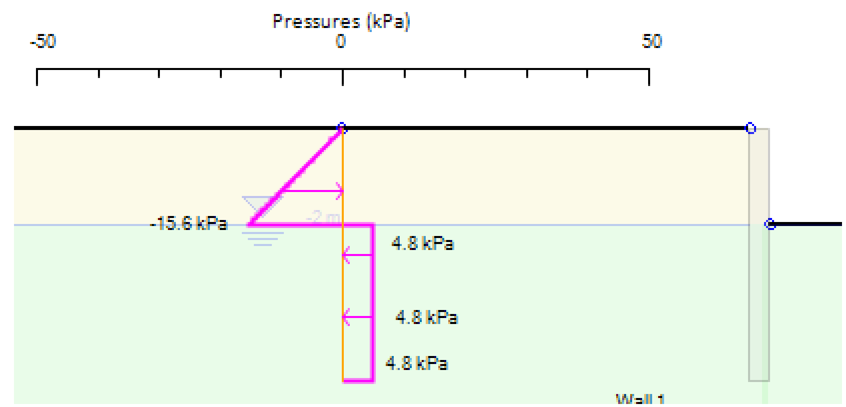
AASHTO Ia (Unstable)





EC7 DA 1/2



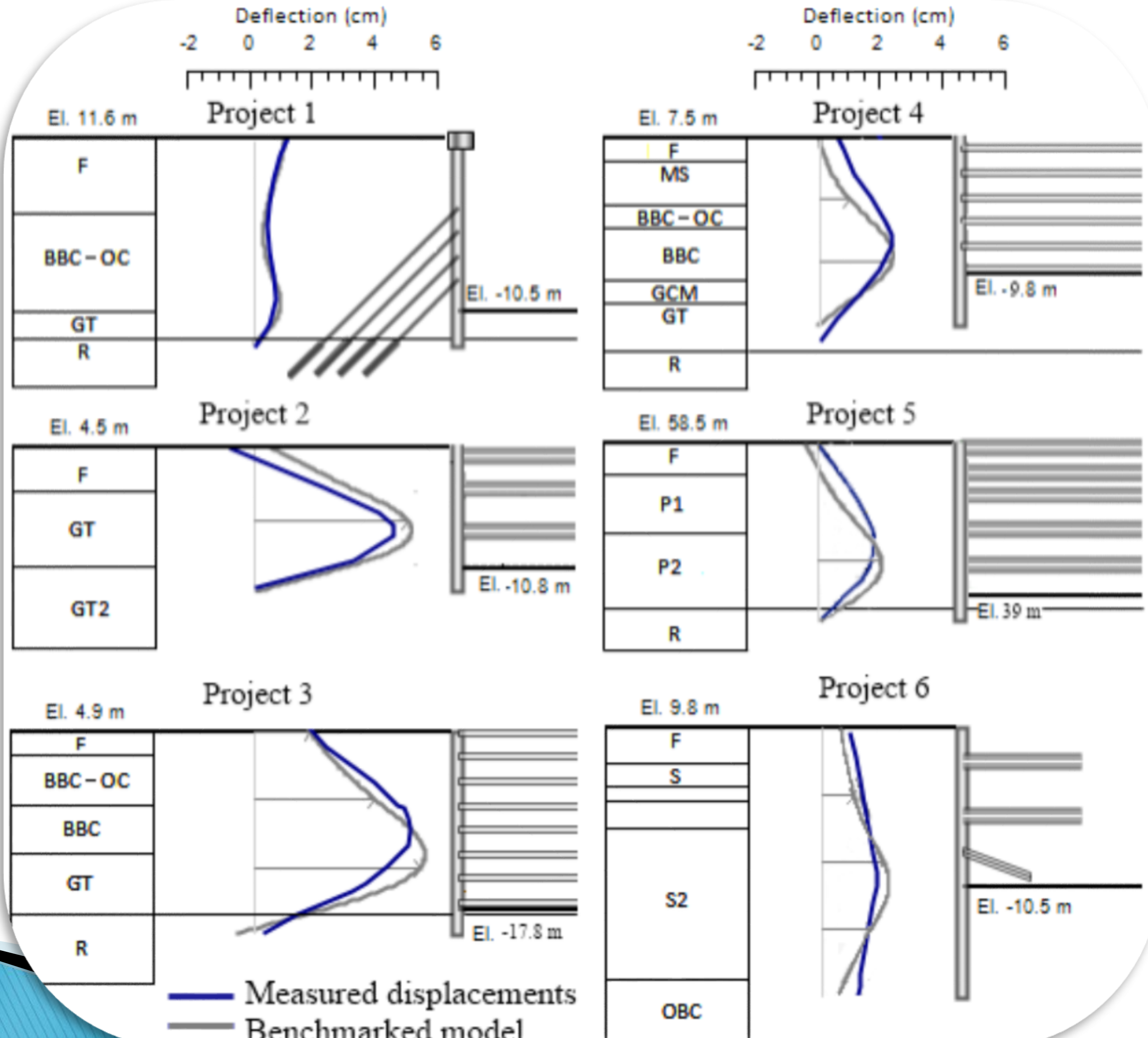


7.1 Benchmarked 6 excavations

22.1m
(72.5ft)

15.3m
(50ft)

22.7m
(75ft)



17.3m
(57ft)

19.5m
(64ft)

20.3m
(67ft)

7.2 Result summary, Part I

Project ID Case[1,2]	δ_h[5] (cm)	FS_{PAS}[6]	M_{NL}[7] (kN-m/m)	FS[8] M_{NL}	R_{NL}[9] (kN/m)	FS[10] R_{NL}	M_{LEM} (kN-m/m)	$\frac{M_{LEM}}{M_{NL}}$	R_{LEM} (kN/m)	$\frac{R_{LEM}}{R_{NL}}$
	Non-linear analysis[3]						Limit equilibrium analysis[4]			
	Wall critical results				Supports		Wall results		Supports	
1	1.29	3.131	1332.9	1.00	1169.5	1.00	533.6	0.40	621.6	0.53
1-I	2.01	2.25	1741.8	1.31	1176.5	1.01	720.3	0.54	786.0	0.67
1-II	2.02	2.182	1857.5	1.39	1178.8	1.01	720.3	0.54	797.8	0.68
1-E1/1	1.29	3.131	1799.4	1.35	1578.8	1.35	720.3	0.54	839.1	0.72
1-E1/2	1.96	2.631	1722.4	1.29	1173.9	1.00	716.9	0.54	782.5	0.67
1-E2	1.29	2.268	1889.7	1.42	1572.4	1.34	720.3	0.54	839.1	0.72
1-E3	2.00	2.472	1863.5	1.40	1176.3	1.01	716.9	0.54	794.2	0.68
1-E2*	1.29	1.14	1799.4	1.35	1578.8	1.35	720.3	0.54	839.1	0.72
2	5.06	1.05	919.4	1.00	480.5	1.00	444.8	0.48	538.1	1.12
2-I	7.23	1.006	1185.6	1.29	620.0	1.29	553.0	0.47	573.3	0.92
2-II	7.17	1.006	1182.1	1.29	619.2	1.29	553.0	0.47	576.5	0.93
2-E1/1	5.06	1.05	1241.1	1.35	648.7	1.35	654.2	0.53	726.4	1.12
2-E1/2	6.2	1.029	1095.2	1.19	574.8	1.20	507.4	0.46	585.8	1.02
2-E2	5.76	1.029	1393.6	1.52	724.8	1.51	838.6	0.60	726.4	1.00
2-E3	6.18	1.032	1090.6	1.19	574.2	1.19	507.4	0.47	589.1	1.03
2-E2*	5.06	1.05	1241.1	1.35	648.7	1.35	838.6	0.68	726.4	1.12
3	5.28	1.415	1950.9	1.00	424.9	1.00	411.2	0.21	666.8	1.57
3-I	7.54	1.036	2697.3	1.38	548.8	1.29	584.3	0.30	698.3	1.64
3-II	7.54	1.036	2697.3	1.38	548.8	1.29	584.3	0.30	698.3	1.64
3-E1/1	5.32	1.413	2642.9	1.35	576.2	1.36	556.0	0.29	902.4	2.12
3-E1/2	7.09	1.098	2579.4	1.32	520.4	1.22	439.6	0.23	737.2	1.73
3-E2	6.08	1.055	3157.8	1.62	606.0	1.43	753.7	0.39	902.4	2.12
3-E3	7.2	1.096	2600.8	1.33	531.3	1.25	441.7	0.23	742.3	1.75
3-E2*	5.32	0.656	2642.9	1.35	576.2	1.36	753.7	0.39	902.4	2.12

7.3 Result summary, Part II

Project ID Case[1,2]	δ_h[5] (cm)	FS_{PAS}[6]	M_{NL}[7] (kN-m/m)	FS[8] M_{NL}	R_{NL}[9] (kN/m)	FS[10] R_{NL}	M_{LEM} (kN-m/m)	M_{LEM}/M_{NL}	R_{LEM} (kN/m)	R_{LEM}/R_{NL}
	Non-linear analysis[3]						Limit equilibrium analysis[4]			
	Wall critical results				Supports		Wall results		Supports	
4	2.33	1.919	1144.9	1.00	273.9	1.00	216.5	0.19	414.9	1.51
4-I	3.37	1.326	1587.8	1.39	391.1	1.43	228.8	0.20	450.3	1.64
4-II	3.37	1.326	1587.8	1.39	391.1	1.43	228.8	0.20	450.3	1.64
4-E1/1	2.33	1.919	1545.6	1.35	369.7	1.35	292.3	0.26	560.1	2.05
4-E1/2	3.00	1.336	1447.6	1.26	355.9	1.30	232.8	0.20	461.7	1.69
4-E2	2.54	1.323	1689.5	1.48	386.9	1.41	292.3	0.26	560.1	2.05
4-E3	3.00	1.336	1447.6	1.26	355.9	1.30	232.8	0.20	461.7	1.69
4-E2*	2.33	0.838	1545.6	1.35	369.7	1.35	292.3	0.26	560.1	2.05
5	2.43	4.5	1748.6	1.00	626.3	1.00	355.5	0.20	572.7	0.91
5-I	3.76	3.09	2409.5	1.38	693.3	1.11	384.1	0.22	627.8	1.00
5-II	3.76	3.09	2409.5	1.38	693.3	1.11	384.1	0.22	627.8	1.00
5-E1/1	2.43	4.5	2360.6	1.35	845.6	1.35	480.0	0.27	773.1	1.23
5-E1/2	3.32	3.381	2208.5	1.26	635.3	1.01	361.2	0.21	580.7	0.93
5-E2	2.74	3.279	2565.1	1.47	794.5	1.27	480.0	0.27	773.1	1.23
5-E3	3.32	3.381	2208.5	1.26	635.3	1.01	361.2	0.21	580.7	0.93
5-E2*	2.43	1.587	2360.6	1.35	845.6	1.35	480.0	0.27	773.1	1.23
6	2.85	3.352	838.9	1.00	592.2	1.00	357.6	0.43	501.1	0.85
6-I	4.47	2.211	795.5	0.95	702.2	1.19	383.1	0.46	539.0	0.91
6-II	4.47	2.211	795.5	0.95	702.2	1.19	383.1	0.46	539.0	0.91
6-E1/1	2.85	3.352	1132.5	1.35	799.4	1.35	482.8	0.58	676.5	1.14
6-E1/2	4.19	1.878	781.2	0.93	679.6	1.15	403.4	0.48	569.8	0.96
6-E2	2.89	2.38	1044.5	1.25	808.8	1.37	482.8	0.58	676.5	1.14
6-E3	4.19	1.878	781.2	0.93	679.6	1.15	403.4	0.48	569.8	0.96
6-E2*	2.85	1.613	1132.5	1.35	799.4	1.35	482.8	0.58	676.5	1.14

7.4 EC7-LRFD Conclusion

- Non-linear analysis:
  - Overall FS on wall bending moments: 0.93 to 1.62
  - Overall FS on support reactions: 1.01 to 1.43
- Limit-equilibrium analysis with FHWA:
  - Bending moments 19 to 66% of non-linear analysis bending moment!
  - Support reactions from 53% to 178% of non-linear analysis results.

7.5 General experience

- LEM selection of appropriate method
- LEM for back-checks
- Non-linear analysis methods are encouraged
- ULS and LRFD approaches may produce inconsistent results with allowable design experience.
- Attention to details!!!
- Breath some AIR (Always Inspect Results)

8. What is next

- Third week: March 10, 11, 12

  Optimization of excavations

Thank you!

For attending this webinar.

dimitrios@deepexcavation.com

Design example:

To be e-mailed later this week

Connect on LinkedIn

DEEP EXCAVATION

RELIABLE GEOEXPERTISE