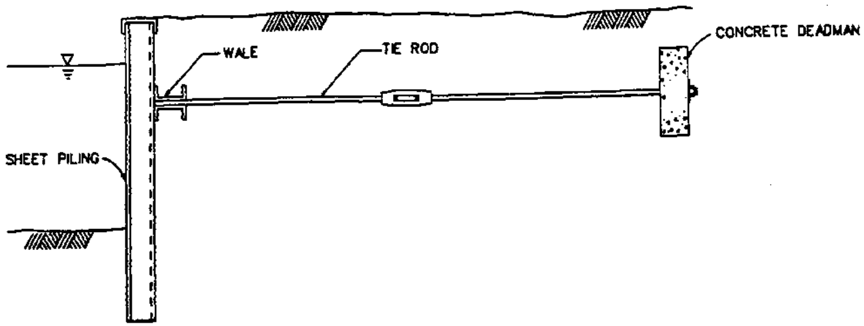
ENCE 4610 Foundation Analysis and Design

Lecture 11
Retaining Walls
Sheet Piling: Anchored Walls

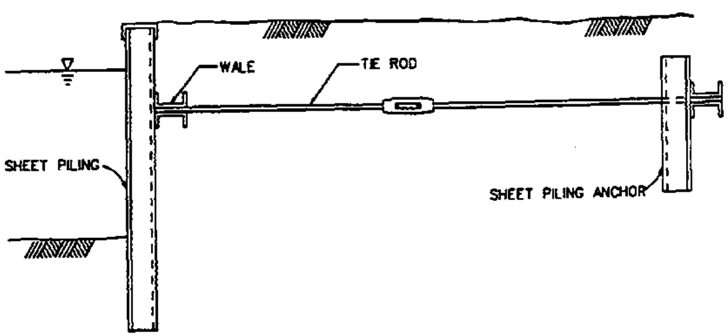
Anchored Sheet Pile Walls

- Includes an anchor or tieback at or near the head of the wall
- More than one set of anchors or tiebacks can be used
- Increases wall stability and enables taller walls to be built and sustained
- Almost a necessity with vinyl, aluminum and fiberglass sheet piles
- Not exclusive to sheet piling; also used with other types of in situ wall systems

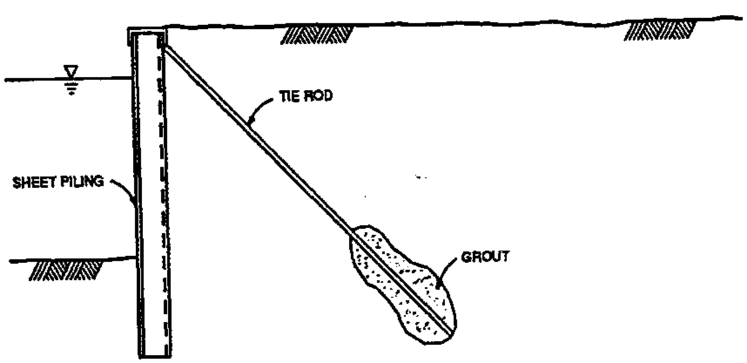
Anchored Sheet Pile Walls



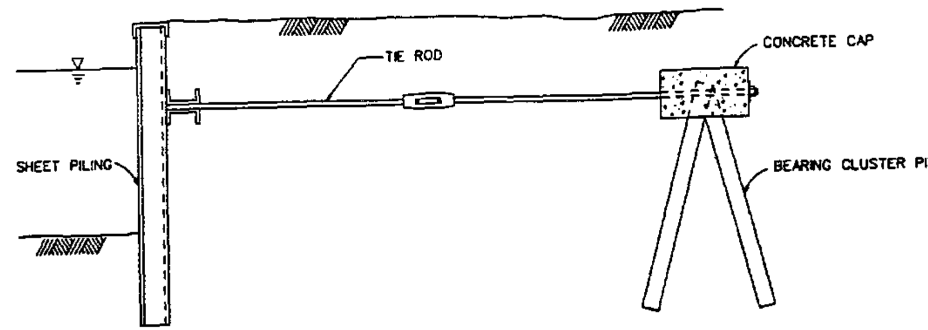
a. Tie rods and dead man



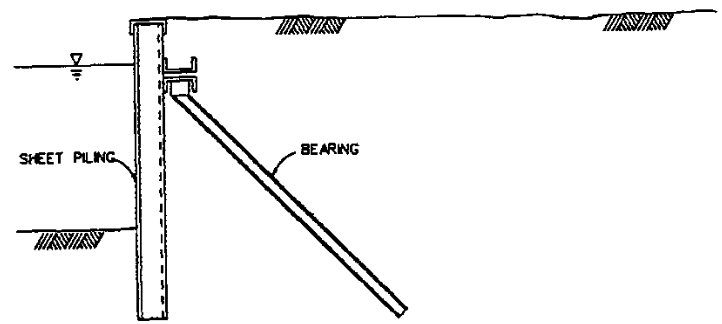
b. Tie rods and anchor wall



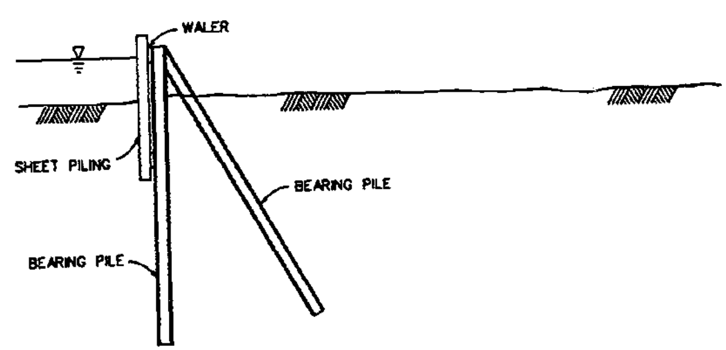
c. Tiebacks with grout anchor



d. Tie rods and A-frame

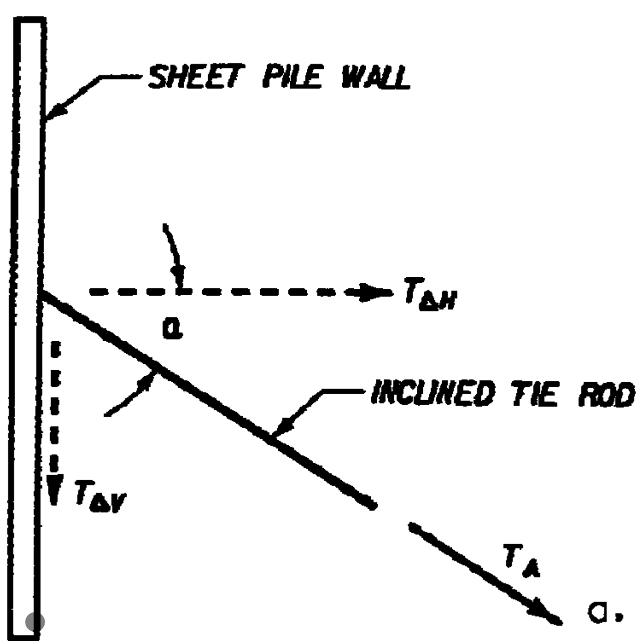


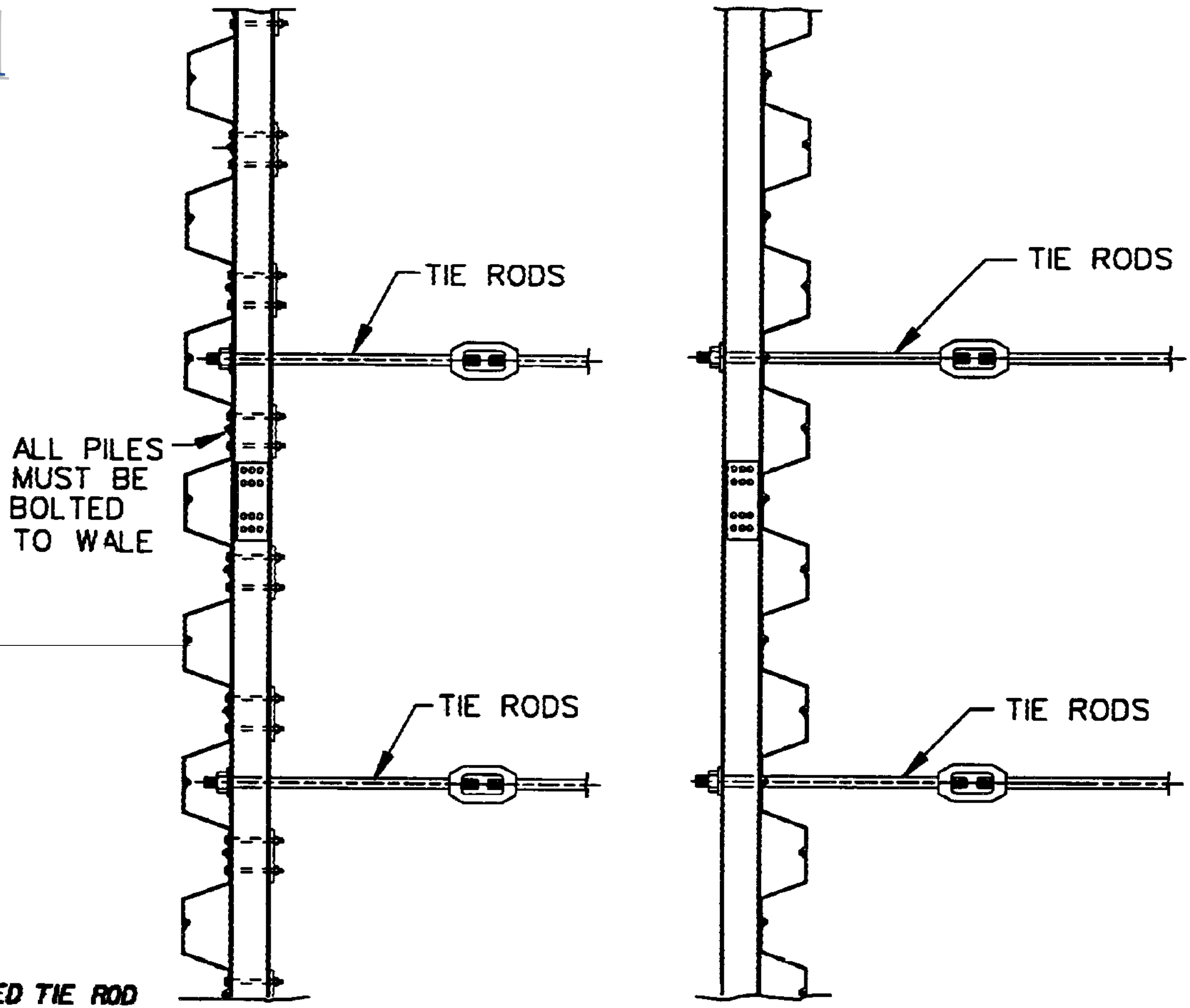
e. Steel H-pile tension anchors



f. Steel H-pile anchors

Wales and Tiebacks in Sheet Pile Walls





a. Wales inside of wall

b. Wales outside of wall

Analysis Methods for Anchored Walls

- Design Methods
  - Free Earth Support Method
    - Assumes lower end of the pile incapable of producing negative bending moments
    - Converts problem into a statically determinate one
    - Rowe's Moment Reduction Method used to take in to account flexibility of sheeting
      - Is unnecessary with SSI (soil-structure interaction) solutions such as SPW 2006
  - Fixed Earth Support Method/Blum's Method (Equivalent Beam Method)
    - Makes lower end has no angle like a cantilever beam, but no moment
    - Results in sheeting which is longer but has lower moment (and thus can be lighter)
    - Useful for some kinds of sheeting, more conservative
    - Also does not require Rowe's Moment Reduction
  - Beam on Elastic Foundation Method (generally w/plasticity considered)

- Implementation Methods
  - All of the methods used in cantilever walls can be used in anchored walls as well, with same strengths and weaknesses
  - Anchored walls are both simpler and more complex for hand solution than cantilever ones
  - Key for solution is to take moment around anchor point, solve for sheeting length and then compute anchor force
  - In this course, we will primarily use SPW2006 to analyze these walls
    - Stable wall is arrived at by varying the length of the wall
    - Solution doing this is basically free earth support method with SSI, obviating need for Rowe's moment reduction
  - Blum's Method
    - Discussed in Veruijt for simple cases
    - A more comprehensive closed-form solution is available

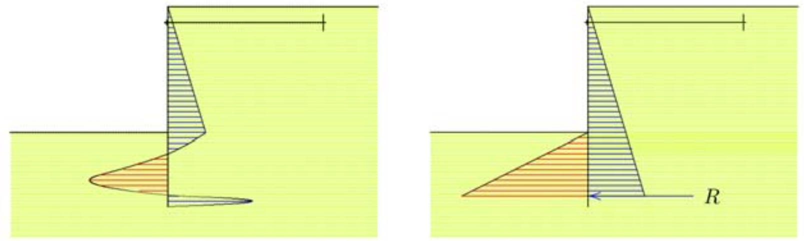
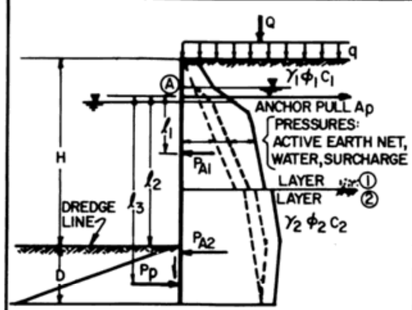


Figure 37.1: Blum's schematization.

Free Earth Analysis of Anchored Walls



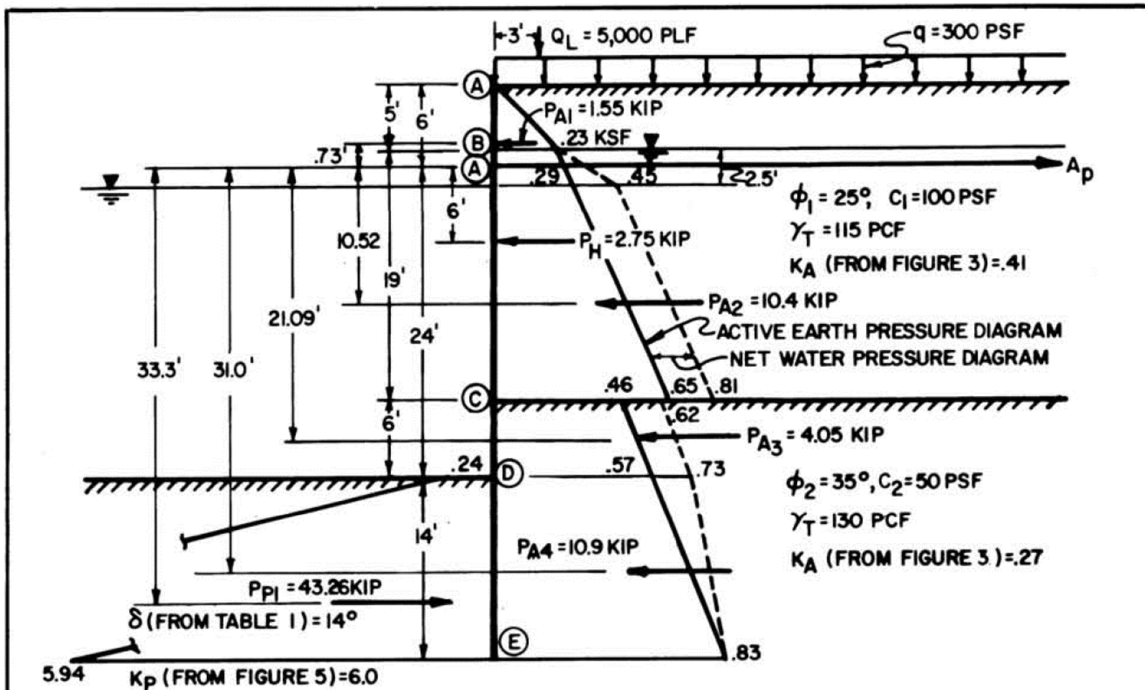
FREE EARTH SUPPORT – GENERAL CASE

1. COMPUTE PRESSURES BY METHODS OF FIGURES 2 TO 7 PASSIVE PRESSURES FOR CLEAN COARSE GRAIN SOILS INCLUDE WALL FRICTION (δ). TABLE I. FOR ACTIVE OR PASSIVE PRESSURES IN ALL OTHER SOIL TYPES, IGNORE WALL FRICTION.
2. DEPTH OF PENETRATION REQUIRED: TAKE MOMENTS ABOUT POINT (A) AND SOLVE FOR D: $P_{A1} l_1 + P_{A2} l_2 = \frac{P_P}{F_S} l_3$
   F_S = 2 TO 3 FOR COARSE GRAINED SOILS
   F_S = 1.5 TO 2 FOR FINE GRAINED SOILS
3. ANCHOR PULL: $A_p = [P_{A1} + P_{A2} - P_p/F_S]\, d$, d=ANCHOR SPACING
4. MAXIMUM BENDING MOMENT (M_{MAX}) IN SHEETING COMPUTED BY THE FREE EARTH SUPPORT METHOD AND APPLYING P_{A1}, P_{A2}, P_p/F_S AND Ap. FOR SHEETING IN SAND APPLY MOMENT REDUCTION FOR FLEXIBILITY OF FIGURE 19.
5. INCRESE PENETRATION COMPUTED (D) BY 20% TO ALLOW FOR DREDGING, SCOUR, ETC.

Notes:

A) #1, "methods of Figures 2-7" refer to methods of computing Ka and Kp and lateral earth pressures previously discussed.

B) #5, same note as with cantilever walls; however, however, consider subsequent changes in level of "dredge line" when designing wall



ACTIVE EARTH PRESSURE (SEE FIGURE 2)

INCLUDING UNIFORM SURCHARGE q

$\sigma_H = \gamma Z K_A - 2C\sqrt{K_A}$

(A), $\sigma_H = .30 \times .41 - 2 \times .10\sqrt{.41} = 0$

(B), $\sigma_H = (.30 + 5 \times .115).41 - 2 \times .10\sqrt{.41} = .23$ KSF

(C), $\sigma_H = (.30 + 5 \times .115 + 19 \times .053).41 - 2 \times .10\sqrt{.41} = .65$ KSF

(C), $\sigma_H = (.30 + 5 \times .115 + 19 \times .053).27 - 2 \times .05\sqrt{.27} = .46$ KSF

(D), $\sigma_H = .46 + 6 \times .068 \times .27 = .57$ KSF

(E), $\sigma_H = .46 + (6 + 14) \times .068 \times .27 = .83$ KSF

PRESSURE OF LINE LOAD SURCHARGE

(SEE FIGURE 11)

$m = \frac{X}{H} = \frac{3}{30} = 0.1$

$P_H = 0.55\, Q_L = 0.55 \times 5 = 2.75$ KIP

LOCATION OF RESULTANT:

$R = .60\,H = .60 \times 30 = 18'$

NET WATER PRESSURE

$\sigma_W = \gamma_W Z = .0625 \times 2.5 = .16$ KSF

PASSIVE PRESSURE

$\sigma_H = \gamma Z K_P + 2C\sqrt{K_P}$

(D), $\sigma_H = 0 + 2 \times .05\sqrt{6.0} = .24$ KSF

(E), $\sigma_H = .068 \times 14 \times 6.0 + 2 \times .05\sqrt{6.0} = 5.94$ KSF

SAFETY FACTOR AGAINST TOE FAILURE:

TAKE MOMENTS ABOUT (A)

$$F_S = \frac{\Sigma \text{ MOMENTS OF PASSIVE FORCES}}{\Sigma \text{ MOMENTS OF ACTIVE FORCES}}$$

$$= \frac{43.26 \times 33.3}{1.55 \times 0.73 + 2.75 \times 6 + 10.4 \times 10.52 + 4.05 \times 21.09 + 10.9 \times 31.0}$$

$= 2.61 > 2.5$

ANCHOR PULL

$A_p = \Sigma P_A - \Sigma P_p / F_S$

$= 1.55 + 2.75 + 10.4 + 4.05 + 10.9 - \frac{43.26}{2.5} = 12.37$ KIPS

MAXIMUM BENDING MOMENT IN SHEETING

POINT OF ZERO SHEAR:

$12.37 - 1.55 - 2.75 - .45X - .022 \times \frac{X^2}{2} = 0$

$X \cong 13.6'$ BELOW OUTSIDE WATER LEVEL

$M_{MAX} = 1.55 \times 15.7 + 12.37 \times 15.1 - 2.75 \times 9.1$
$\quad - .45 \times \frac{13.6^2}{2} - .022 \times \frac{13.6^2}{2} \times 4.52 = 86.9$ FT-KIPS

MOMENT REDUCTION:

ASSUME: f_s = 27,000 PSI, E = 30,000,000 PSI

TRY ZP 32, $I = 385.7\ IN^4$, $S = 38.3\ IN^3$

ρ = (FROM FIGURE 19) $= \frac{(H+D)^4}{EI}$

$\rho = \frac{(30 \times 12 + 14 \times 12)^4}{30{,}000{,}000 \times 385.7} = 6.7\ \frac{IN^2}{LB}$

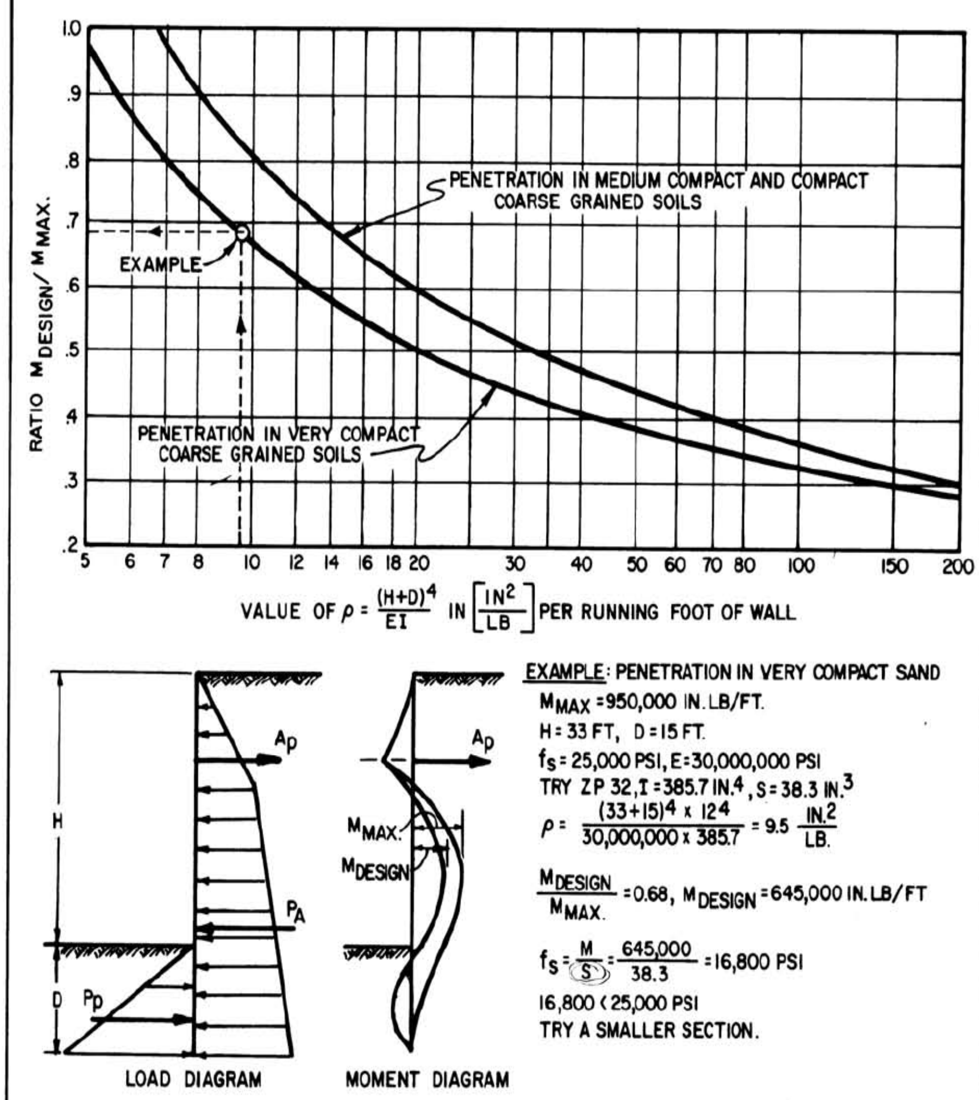
$\frac{M_{DESIGN}}{M_{MAX}} = .83$; $M_{DESIGN} = .83 \times 86.9 = 72.1$ FT-KIPS

$f_s = \frac{M}{S} = \frac{72.1 \times 1000 \times 12}{38.3} = 22{,}600$ PSI $< 27{,}000$

TRY A SMALLER SECTION

Rowe's Moment Reduction Curves

- Used to take into consideration the flexibility of the pile and its effect on relieving the actual bending moment the wall experiences
- Different set of curves for clay and sand



LEGEND

M_{MAX} = MAXIMUM POSITIVE MOMENT IN SHEETING COMPUTED BY METHODS OF FIGURE 18.

M_{DESIGN} = MAXIMUM POSITIVE MOMENT FOR DESIGN OF SHEETING.

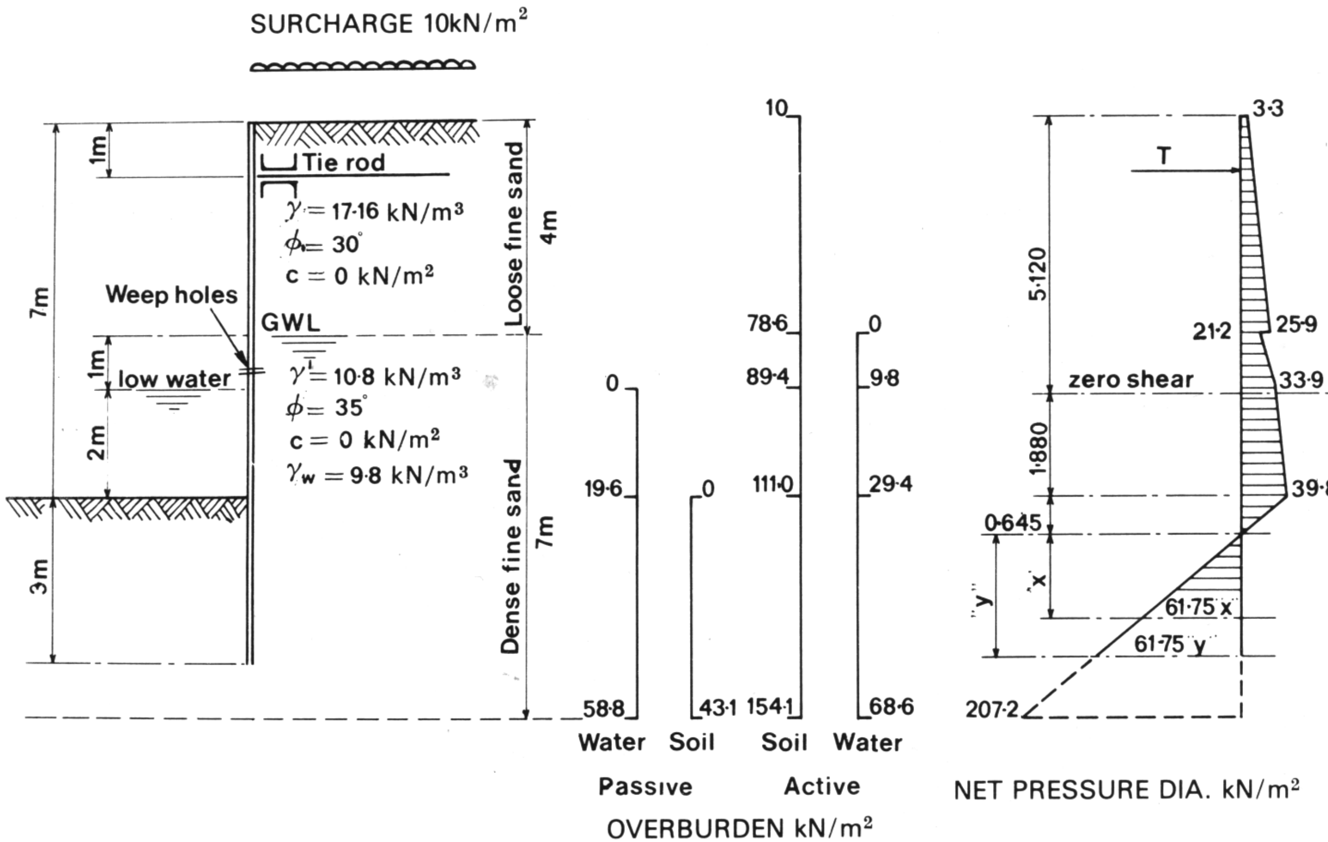
ρ = FLEXIBILITY NUMBER = $\frac{(H+D)^4}{EI}$, E = SHEETING MODULUS OF ELASTICITY, PSI
I = SHEETING MOMENT OF INERTIA, IN.^{4} PER RUNNING FOOT OF WALL.

NOTES

1. M_{DESIGN} IS OBTAINED BY SUCCESSIVE TRIALS OF SHEETING SIZE UNTIL MAX. BENDING STRESS IN SHEETING EQUALS ALLOWABLE BENDING STRESS.
2. NO REDUCTION IN $M_{MAX.}$ IS PERMITTED FOR PENETRATION IN FINE GRAINED SOILS OR LOOSE OR VERY LOOSE COARSE GRAINED SOILS.
3. FLEXIBILITY NUMBER IS COMPUTED ON THE BASIS OF LUBRICATED INTERLOCKS.

Free Earth Support—Example

Anchored Wall Example



Active Pressure $p_a = \gamma.h.K_a - 2c\sqrt{K_a} + p_w$ kN/m²

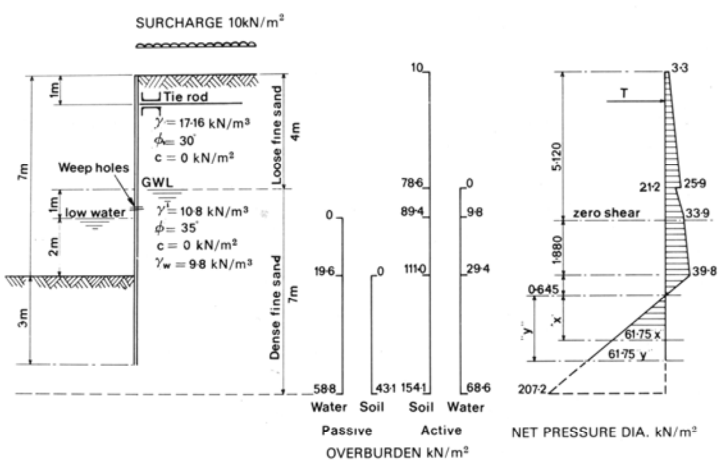
Passive Pressure $p_p = \gamma.h.K_p + 2c\sqrt{K_p} + p_w$ kN/m²

The coefficients of earth pressure are obtained from the tables in the section on earth and water pressures.

Loose fine sand: $K_a = 0{\cdot}33$ $K_p = 4{\cdot}9$; Compact find sand: $K_a = 0{\cdot}27$ $K_p = 6{\cdot}0$

Anchored Wall Example

Free Earth Support—Example



Active Pressure $p_a = \gamma.h.K_a - 2c\sqrt{K_a} + p_w$ kN/m²

Passive Pressure $p_p = \gamma.h.K_p + 2c\sqrt{K_p} + p_w$ kN/m²

The coefficients of earth pressure are obtained from the tables in the section on earth and water pressures.

Loose fine sand: $K_a = 0{\cdot}33$ $K_p = 4{\cdot}9$; Compact find sand: $K_a = 0{\cdot}27$ $K_p = 6{\cdot}0$

p_a at ground level	= 10 x 0·33	= 3·3 kN/m²
p_a at 4 m below ground level in loose fine sand	= 78·6 x 0·33	= 25·9 kN/m²
p_a at 4 m below ground level in dense fine sand	= 78·6 x 0·27	= 21·2 kN/m²
p_a at 5 m below ground level in dense fine sand	= 89·4 x 0·27 + 9·8	= 33·9 kN/m²
p_a at 7 m below ground level in dense fine sand	= 111·0 x 0·27 + 29·4	= 59·4 kN/m²
p_a at 11 m below ground level in dense fine sand	= 154·1 x 0·27 + 68·6	= 110·2 kN/m²
p_w at 7 m below ground level		= 19·6 kN/m²
p_p at 11 m below ground level in dense fine sand	= 43·1 x 6·0 + 58·8	= 317·4 kN/m²
Nett p_a at 7 m below ground level in dense fine sand	= 59·4 — 19·6	= 39·8
Nett p_p at 11 m below ground level in dense fine sand	= 317·4 — 110·2	= 207·2

Moments of Active Pressure about Tie Rod

ΣM ABOUT T = 0

MACTIVE − MPASSIVE = 0

3·3 x 4·000 x 0·5 x 0·333	= 2·2
25·9 x 4·000 x 0·5 x 1·667	= 86·4
21·2 x 1·000 x 0·5 x 3·333	= 35·3
33·9 x 1·000 x 0·5 x 3·667	= 62·2
33·9 x 2·000 x 0·5 x 4·667	= 158·2
39·8 x 2·000 x 0·5 x 5·333	= 212·3
39·8 x 0·645 x 0·5 x 6·215	= 79·8
	636·4 kN/m

Total Active Pressure = 172·5 kN

For stability the moment of Passive Pressure about the tie rod must be equal to, or not less than, the moment of Active Pressure about the tie rod.

SOLVE FOR MIN. DEPTH D,

$$\therefore \quad \frac{61{\cdot}75x^2}{2}\left(6{\cdot}645 + \frac{2x}{3}\right) = 636{\cdot}4, \quad x \simeq 1{\cdot}63 \text{ m}$$

Total Net Passive Pressure = 61·75 x 1·63² x 0·5 = 82·0 kN   TOTAL MIN. PASSIVE

Moment of Nett Passive Pressure about tie rod = 82·0 x 7·732 = 634·0 kN/m   CHECK $M_A - M_P = 0$

Total Net Active Pressure = 172·5 kN

Force in Walings and Tie Rods = 172·5 — 82·0 = 90·5 kN/m run of wall $\Sigma F_H = 0$ $T = P_A - P_P$

Zero Shear (maximum bending moment) occurs at 5·120 m below ground level.

Moments about and below level of zero shear.

TAKE MOMENTS ABOVE AND BELOW ZERO SHEAR

82·0 x 3·612	= 296·2
—34·3 x 1·880 x 0·5 x 0·627	= —20·2
—39·8 x 1·880 x 0·5 x 1·250	= —46·8
—39·8 x 0·645 x 0·5 x 2·095	= —26·9
	202·3 kN/m

B.M. on piles = 202·3 kNm/m run of wall

$$Z \text{ reqd.} = \frac{202{\cdot}3 \times 1000 \times 100}{125 \times 100} = 1618 \text{ cm}^3\text{/m run of wall}$$

Use Larssen No. 3/20 or Frodingham 3N

Penetration required to give a factor of safety of 2 against rotation of wall about the tie rod = y + 0·645 + 7·000

SAFETY FACTOR

$$\text{To find "y"} \quad \frac{61{\cdot}75y^2}{2}\left(6{\cdot}645 + \frac{2y}{3}\right) = 2 \times 636{\cdot}4, \quad y \simeq 2{\cdot}24 \text{ m}$$

Moments of Net Passive Pressure with 2·24 + 0·645 = 2·885 m cut-off
2·24 x 61·75 x 2·24 x 0·5 x 8·138 = 1260·7 kN/m

$$\text{S.F. against rotation} = \frac{1260{\cdot}7}{636{\cdot}4} = 1{\cdot}98$$

Penetration reqd. = 0·645 + 2·24 + 7 = 9·885 m

Use Larssen No. 3/20 or Frodingham No. 3N x 10·0 m long in Grade 43A steel

Anchored Wall Example

Loading Step : 1 Left Side

No.	Soil Name	H	Wd	Ws	Zw	Cap	q	c	Ka	Kp	Kn	Dw
		m	kN/m³	kN/m³	m	m	kN/m²	kN/m²	--	--	--	m
1	Loose Fine Sanc	1.000	0.000	10.000	5.000	0.000	0.000	0.000	1.000	1.000	1.000	1.000
2	Loose Fine Sanc	3.000	0.000	10.000	5.000	0.000	0.000	0.000	1.000	1.000	1.000	1.000
3	Dense Fine San	1.000	0.000	10.000	5.000	0.000	0.000	0.000	1.000	1.000	1.000	1.000
4	Dense Fine San	2.000	0.000	10.000	5.000	0.000	0.000	0.000	1.000	1.000	1.000	1.000
5	)ense Fine Sand	5.000	20.600	20.600	5.000	0.000	0.000	0.000	0.270	4.000	0.426	0.018

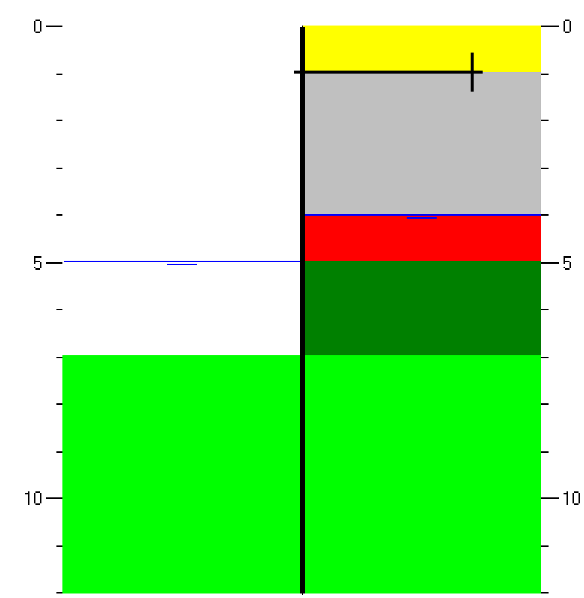
Loading Step : 1 Right Side

No.	Soil Name	H	Wd	Ws	Zw	Cap	q	c	Ka	Kp	Kn	Dw
		m	kN/m³	kN/m³	m	m	kN/m²	kN/m²	--	--	--	m
1	Loose Fine Sanc	1.000	17.160	17.160	4.000	0.000	10.000	0.000	0.333	3.260	1.000	0.056
2	Loose Fine Sanc	3.000	17.160	17.160	4.000	0.000	10.000	0.000	0.333	3.260	0.500	0.056
3	Dense Fine San	1.000	20.600	20.600	1.000	0.000	10.000	0.000	0.270	4.000	0.426	0.018
4	Dense Fine San	2.000	20.600	20.600	1.000	0.000	10.000	0.000	0.270	4.000	0.426	0.018
5	)ense Fine Sand	5.000	20.600	20.600	1.000	0.000	10.000	0.000	0.270	4.000	0.426	0.018

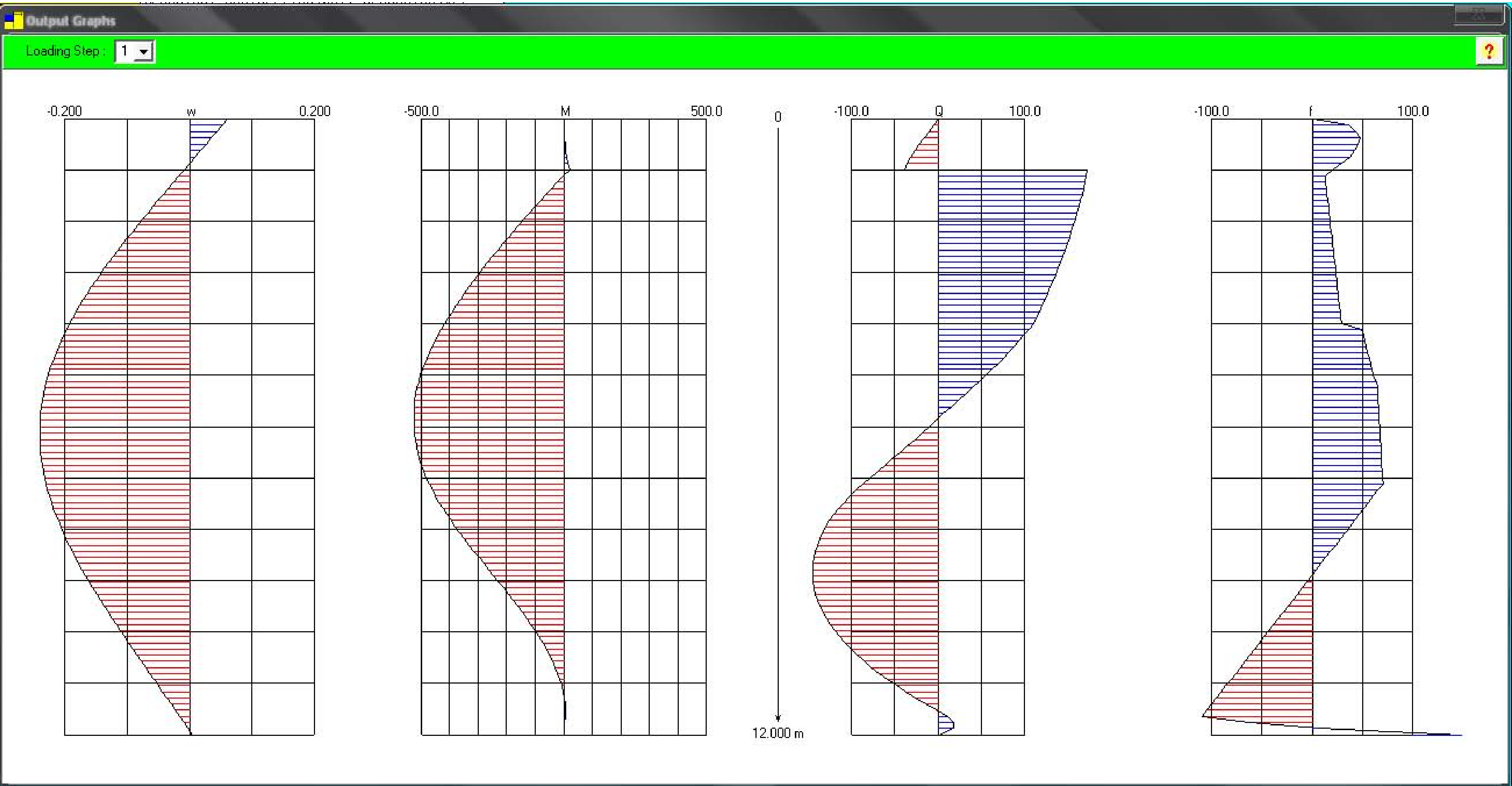
Loading Step : 1

No.	Depth	Fx	Fa	Dw
	m	kN/m	kN/m	m
0	0.000	0.000	0.000	1.000
1	1.000	0.000	500.000	-0.020
2	4.000	0.000	0.000	1.000
3	5.000	0.000	0.000	1.000
4	7.000	0.000	0.000	1.000
5	12.000	0.000	0.000	1.000

Loading Step : 1



Anchored Wall Example



Estimation of Maximum Moment using SPW 2006

- Sheet pile database has an unusual method of input for sheet pile properties
  - Modulus of elasticity "E"
  - "EI" product of modulus of elasticity and moment of inertia per meter of wall
  - "h" distance from neutral axis to face of sheeting = half of depth (this is different from way it's presented in tables)
- Conventional beam theory
  - $\sigma_{max} = (Mc)/I$
  - σ_{max} based on failure criterion, usually = σ_{yield}/FS
  - Maximum permissible stresses given for a variety of materials in previous slide set
- Substitutions using values in SPW 2006
  - I = "EI"/E
  - c = h
  - $\sigma_{max} = (MEh)/(\text{"EI"})$
  - $M_{max} = \sigma_{max}\text{"EI"}/Eh$
- To determine whether sheeting meets structural requirements:
  - Determine σ_{max} based on alloy
  - Select section, which gives you "EI," E, h.
  - Determine maximum permissible moment for section using formula above
  - If moment from SPW2006 is greater, use heavier (greater I, h) section
- Valid for both cantilever and anchored walls
- For previous example:
  - Maximum moment = 520 kN-m/m
  - PZ-22 used in the example
    - h=.1145 m
    - EI = 24150.0 kN-m^2/m
    - E = 210 GPa
    - σ_{max} = 220 MPa (ASTM A 572 Gr. 50)
    - $M_{max} = \sigma_{max}\text{"EI"}/Eh$ = (24150)(220)(1000)/(210)(1000000)(.1145) = 221 kN-m/m
    - This section is obviously too small; you will need to input a heavier section until you exceed the maximum moment

Tiebacks and Wales

Tieback Formulae:

$$T_{ah} = TS$$

where

- T_{ah} =load per tieback
- T =tieback load per foot of wall
- S =spacing between tiebacks

$$K_{ah} = 1/4\,\frac{E_{ah}\pi\, {d_{ah}}^2}{L_{ah}}$$

where

- K_{ah} =tieback stiffness per tieback
- E_{ah} =tieback modulus of elasticity
- d_{ah} =tieback diameter
- L_{ah} =tieback length

$$M_{\max} = 1/10\,TS$$

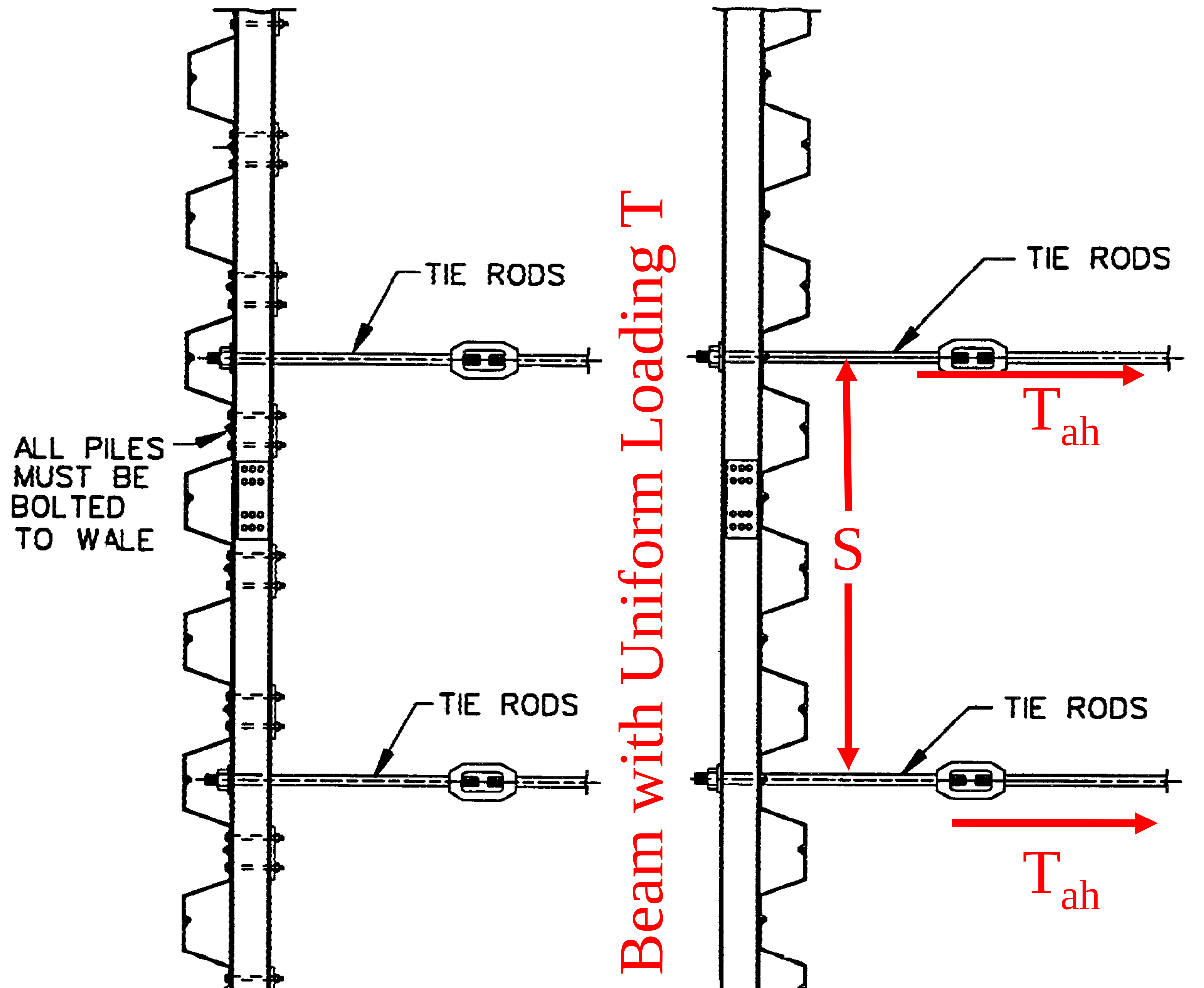
where

- M_{max} =maximum moment on wale

$$K = \frac{K_{ah}}{S}$$

where

- K =combined stiffness per unit length of wall of tiebacks

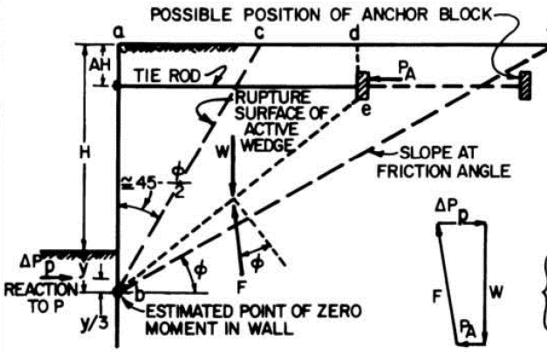


Can be considered either as a beam with rigid or flexible supports

a. Wales inside of wall   b. Wales outside of wall

Anchor Design (NAVFAC DM 7.02)

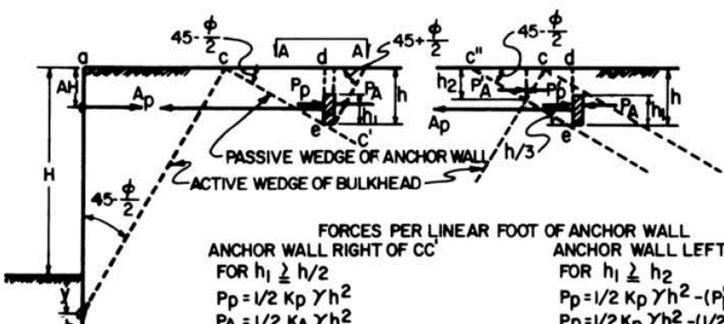
EFFECT OF ANCHOR LOCATION RELATIVE TO THE WALL



ANCHOR BLOCK LEFT OF bc PROVIDES NO RESISTANCE.
ANCHOR BLOCK RIGHT OF bf PROVIDES FULL RESISTANCE WITH NO LOAD TRANSFERRED TO WALL.
ANCHOR BLOCK BETWEEN bc AND bf PROVIDES PARTIAL RESISTANCE AND TRANSFERS LOAD ΔP_p TO BASE OF WALL.

VECTOR DIAGRAM FOR FREE BODY abed
WHERE P_A = ACTIVE FORCE ON BACK OF de AT ANCHOR BLOCK.

CONTINUOUS ANCHOR WALL LOCATED BETWEEN RUPTURE SURFACE AND SLOPE AT FRICTION ANGLE



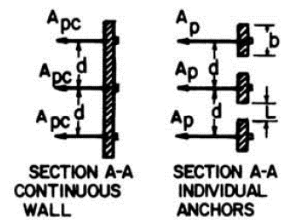
FORCES PER LINEAR FOOT OF ANCHOR WALL

ANCHOR WALL RIGHT OF CC'
FOR $h_1 \geq h/2$
$P_P = 1/2\ K_P \gamma h^2$
$P_A = 1/2\ K_A \gamma h^2$
K_P OBTAINED FROM FIGURE 5
USING $-\delta/\phi = 0.5$

ANCHOR WALL LEFT OF CC'
FOR $h_1 \geq h_2$
$P_P = 1/2\ K_P \gamma h^2 - (P'_P - P'_A)$
$P_P = 1/2 K_P \gamma h^2 - (1/2 K_P \gamma h_2^2 - 1/2 K_A \gamma h_2^2)$
$P_A = 1/2\ K_A \gamma h^2$
K_A IS OBTAINED FROM FIGURE 3

FIGURE 20
Design Criteria for Deadman Anchorage

EFFECT OF DEPTH AND SPACING OF ANCHOR BLOCKS



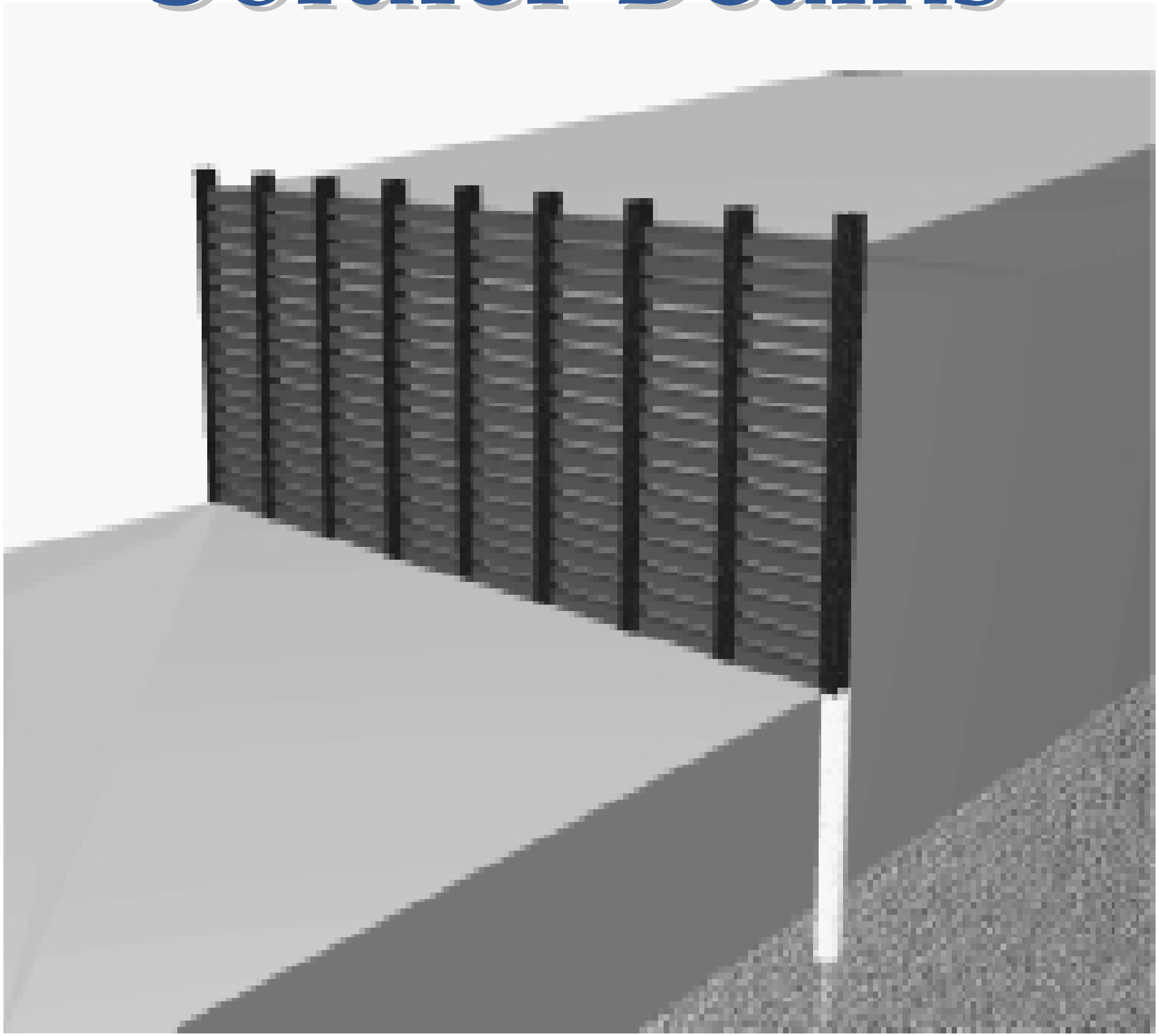
ANCHOR RESISTANCE FOR $h_1 \geq \frac{h}{2}$
1. CONTINUOUS WALL:
ULTIMATE $A_{pc}/d = P_p - P_A$ WHERE A_{pc}/d IS ANCHOR RESISTANCE AND P_p, P_A TAKEN PER LINEAL FOOT OF WALL.
2. INDIVIDUAL ANCHORS:
IF $d > b+h$, ULTIMATE $A_p = b(P_p - P_A) + 2P_o \tan\phi$, WHERE P_o = RESULTANT FORCE OF SOIL AT REST ON VERTICAL AREA cde OR c"de.
IF $d = h+b$, A_p/d IS 70% OF A_{pc}/d FOR CONTINUOUS WALL.
L FOR THIS CONDITION IS L' AND L' = h.
IF $d < h+b$, $A_p/d = A_{pc}/d - \frac{L}{L'}(.3\ A_{pc}/d)$, L' = h.

ANCHOR RESISTANCE FOR $h_1 < \frac{h}{2}$
ULTIMATE A_p/d OR A_{pc}/d EQUALS BEARING CAPACITY OF STRIP FOOTING OF WIDTH h_1 AND SURCHARGE LOAD $\gamma(h - \frac{h_1}{2})$, SEE FIGURE 1, CHAPTER 4
USE FRICTION ANGLE ϕ': WHERE $\tan\phi' = 0.6 \tan\phi$.

GENERAL REQUIREMENTS:
1. ALLOWABLE VALUE OF A_p AND A_{pc} = ULTIMATE VALUE/2, FACTOR OF SAFETY OF 2 AGAINST FAILURE.
2. VALUES OF K_A AND K_P ARE FOR COHESIONLESS MATERIALS. IF BACKFILL HAS BOTH ϕ AND c STRENGTHS, COMPUTE ACTIVE AND PASSIVE FORCES ACCORDING TO FIGURES 7 AND 9 FINE GRAINED SOILS OF MEDIUM TO HIGH PLASTICITY SHOULD NOT BE USED AT THE ANCHORAGE.
3. SOILS WITHIN PASSIVE WEDGE OF ANCHORAGE SHALL BE COMPACTED TO NO LESS THAN 90% OF MAX. UNIT WEIGHT (ASTM D698 TEST).
4. TIE ROD IS DESIGNED FOR ALLOWABLE A_p OR A_{pc}. TIE ROD CONNECTIONS TO WALL AND ANCHORAGE ARE DESIGNED FOR 1.2 (ALLOWABLE A_p OR A_{pc}).
5. TIE ROD CONNECTION TO ANCHORAGE IS MADE AT THE LOCATION OF THE RESULTANT EARTH PRESSURES ACTING ON THE VERTICAL FACE OF THE ANCHORAGE.

FIGURE 20 (continued)
Design Criteria for Deadman Anchorage

Soldier Beams



Questions