Reinforced Earth Company
8614 Westwood Center Drive # 1100, Vienna VA 22182
703-821-1175

JOB NUMBER : 8079 SHEET #1 OF 43
DESG: MJB CHKD: DATE: 09/01/00

PORT OF SEATTLE

SEA-TAC INTERNATIONAL AIRPORT

REINFORCED EARTH DESIGN CALCULATIONS

CERTIFIED WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES ONLY.

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LOCATION: PROJECT NO: SHEET 1 OF

SUBJECT: DESG: CHKD: DATE:

Design Parameters

Select Backfill

$\phi = 37°$
$\gamma = .140$ Kcf
$Ka = Tan^2(45° - \phi/2) = .24858$

Random Backfill

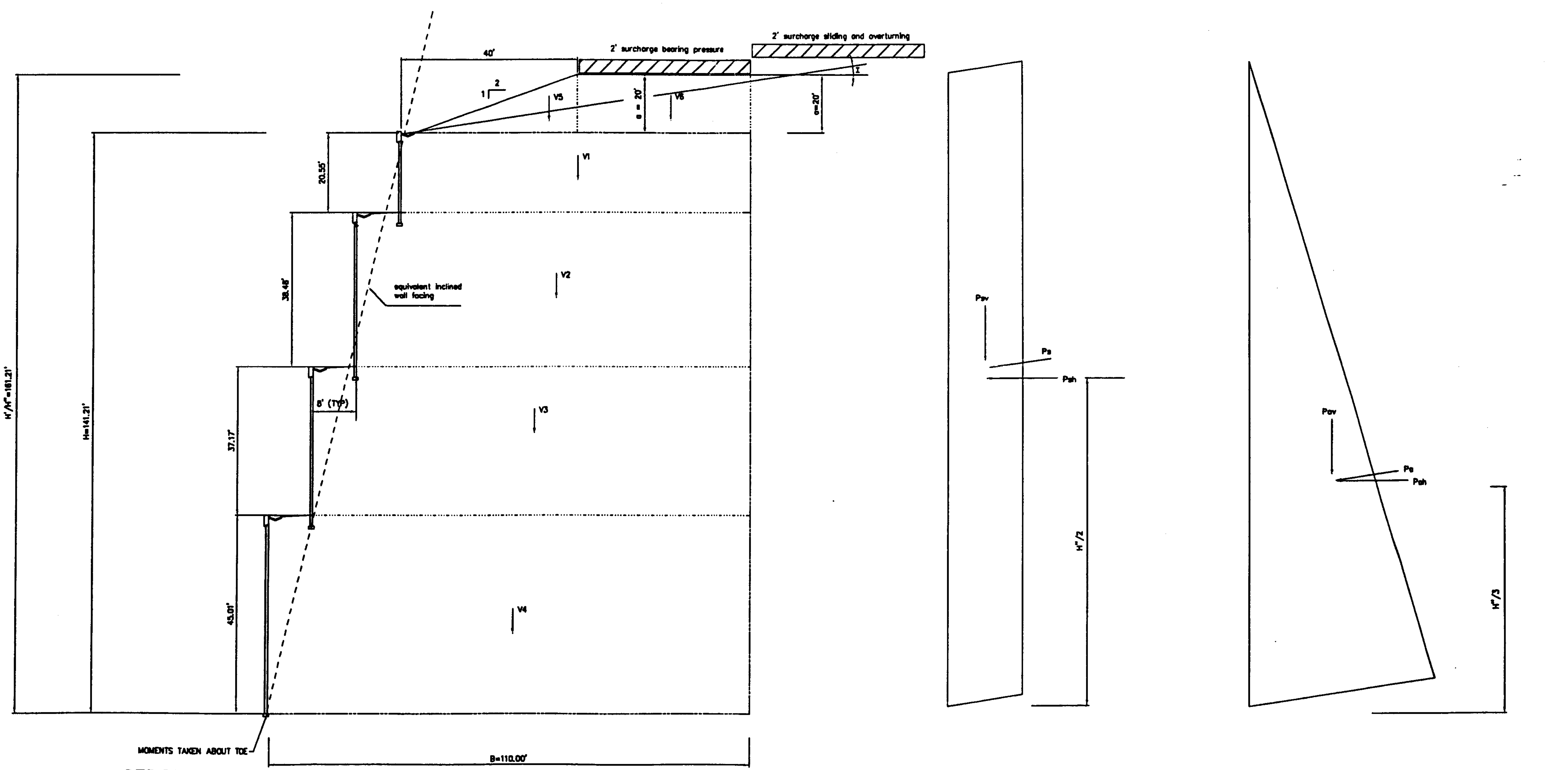
$\phi = 35°$
$\gamma = .140$ Kcf
$Ka = .27269$ (See sheet 3)

Foundation

$\phi = 35°$
Coefficient of friction @ foundation – Tan 35° = .7002

Traffic Surcharge = 2' = .280 KSF
2:1 Broken Back Slope over 40 feet.

DESIGN DRAWING EXTERNAL STABILITY AND INTERNAL STABILITY – REINF. TENSION ONLY-STATIC

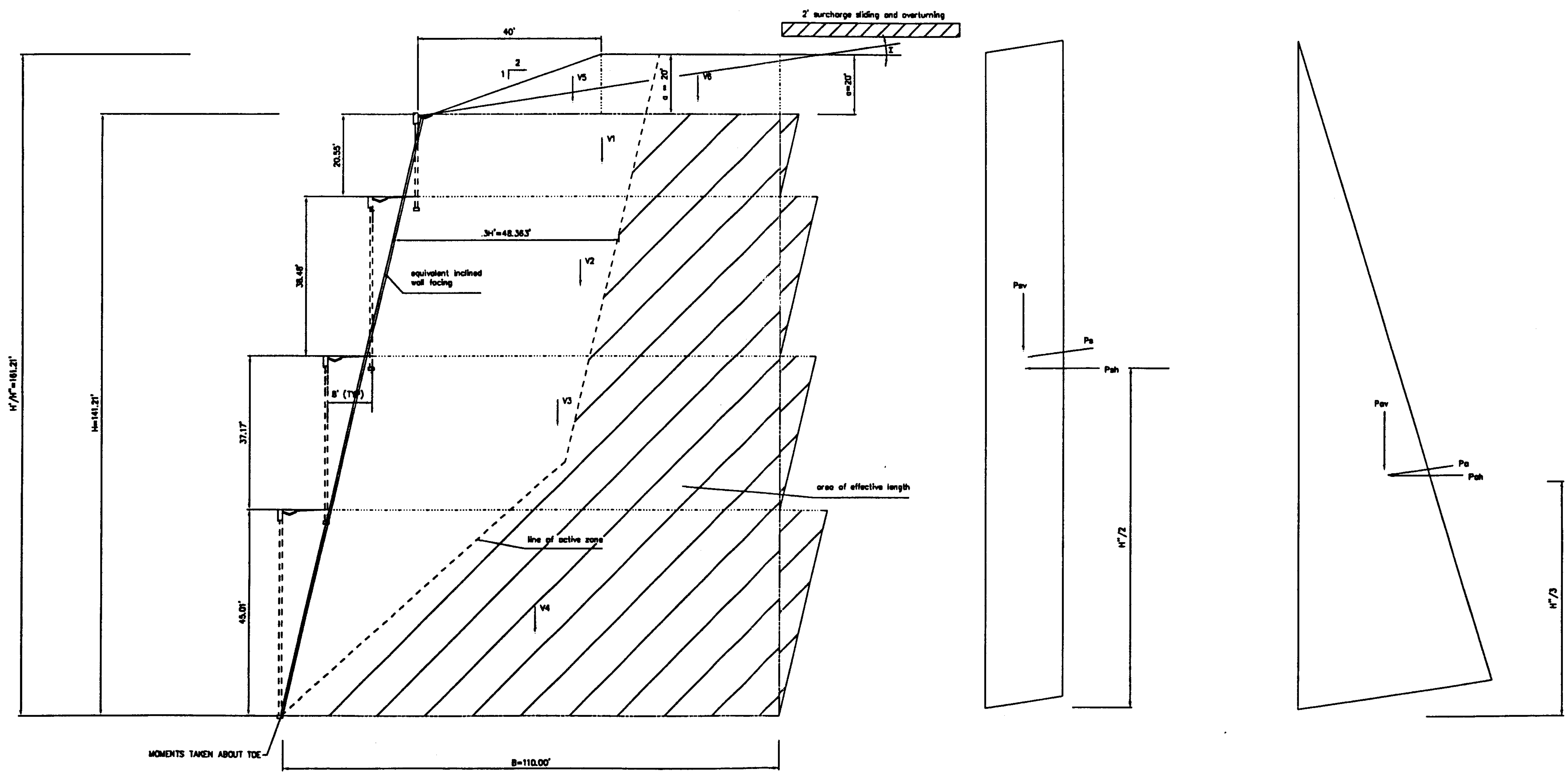


GFDGFDGFDGFD

WEST WALL 181+25

zB

DESIGN DRAWING INTERNAL STABILITY/EFFECTIVE LENGTH SAFETY FACTOR ONLY-STATIC

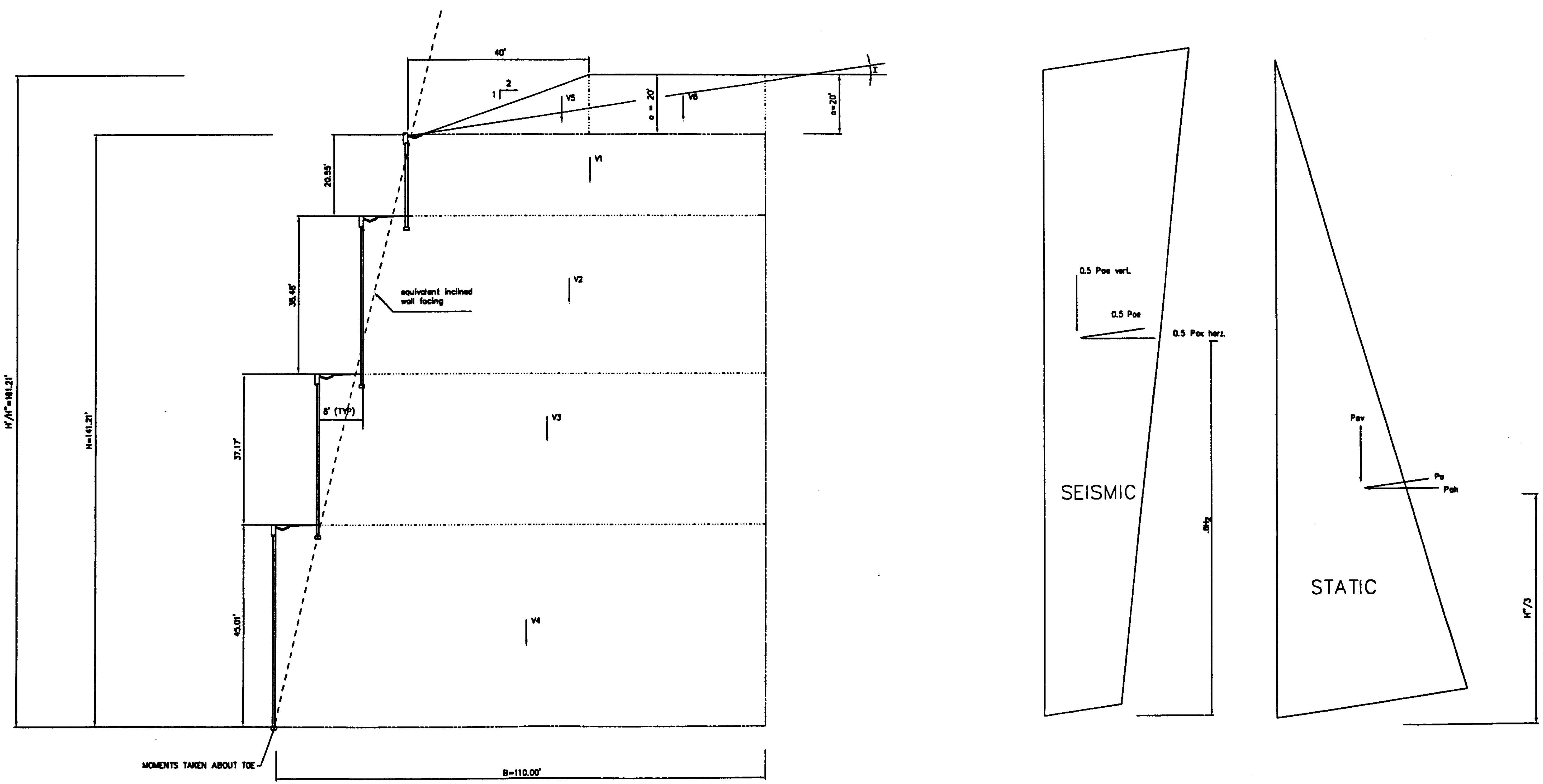


WEST WALL 181+25

AR 028059

2C

DESIGN DRAWING EXTERNAL STABILITY AND INTERNAL STABILITY – REINF. TENSION ONLY–SEISMIC

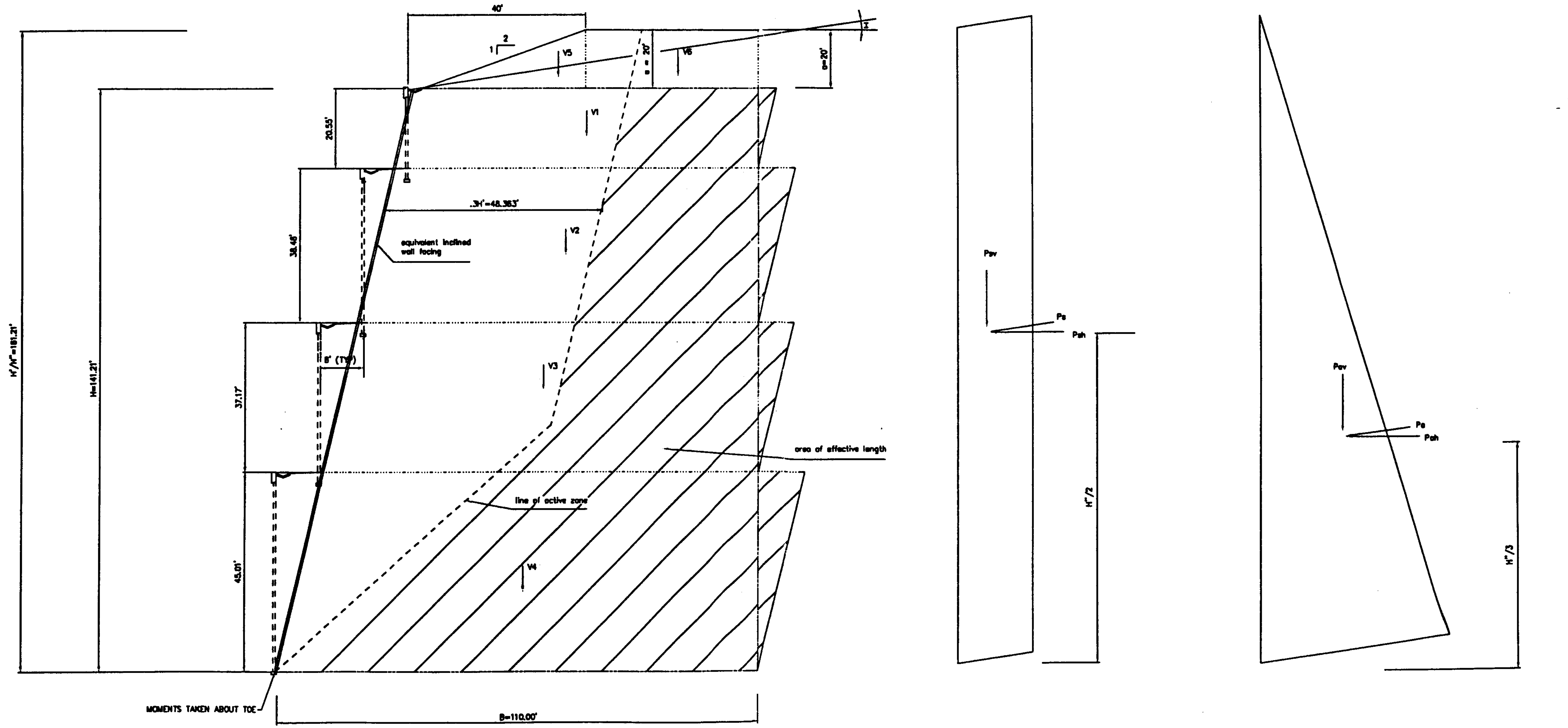


WEST WALL 181+25

AR 028060

2D

DESIGN DRAWING INTERNAL STABILITY/EFFECTIVE LENGTH SAFETY FACTOR ONLY–SEISMIC



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LOCATION: PROJECT NO: SHEET 3 OF

SUBJECT: DESG: CHKD: DATE:

Design Drawg Cont.

$P_{aV} = P_a \sin I$; $P_{sV} = P_s \sin I$
$P_{aH} = P_a \cos I$; $P_{sH} = P_s \cos I$ } (AASHTO 96 Fig. 5.8.2C)

$I = \arctan a/2H$ = angle of equivalent infinite slope

$a = 20'$
$H = 141.21'$ $\quad I = \dfrac{20}{2(141.21)} = 4.0507°$ (AASHTO 96' Fig. 5.8.2C)

$$K_a = \cos I \left[\frac{\cos I - \sqrt{\cos^2 I - \cos^2 \phi}}{\cos I + \sqrt{\cos^2 I - \cos^2 \phi}} \right] \quad \text{(Rankine)}$$

$$= \cos 4.0507° \left[\frac{\cos 4.0507° - \sqrt{\cos^2 4.0507° - \cos^2 35°}}{\cos 4.0507° + \sqrt{\cos^2 4.0507° - \cos^2 35°}} \right]$$

$= .27269$

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LOCATION: PROJECT NO: SHEET 4 OF

SUBJECT: DESG: CHKD: DATE:

Design Drawing Cont.

Derivation of H' (AASHTO 96 FIG. 5.8.4.1A)

H' is the theoretical height of the wall. Its top boundry is the point at which the failure surface intersects the ground surface. The top of the failure surface begins a distance of .3H' from the back face of the panels.

#1.) For a 2:1 slope:

$$H' = H + \frac{.3H'}{2} \Longrightarrow H' = H/.85$$

$$H' = 141.21/.85 = 166.13$$

However: $166.13(.3) = 49.84 > 40'$
So #2 below.

#2.) If the slope ends at a point less than .3H' from the back face of panels as calculated above then:

$$H' = H + \frac{\text{Dist slope ends from back of panel}}{\text{slope}}$$

$$H' = 141.21 + 40/2 = 161.21'$$

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LOCATION: PROJECT NO: SHEET 5 OF

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Derivation of H''

H'' is the height to which lateral pressures from the ground surface at the end of the strips. If the slope extends beyond the end of the reinf. strips then the effects of traffic surcharge are not included in the calculations for either horizontal or vertical pressures.

$$H'' = H + \frac{\text{Dist. slope ends from Back of panels}}{\text{Slope}}$$

$$H'' = 141.21 + 40/2 = 161.21'$$

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LOCATION: PROJECT NO: SHEET 6 OF

SUBJECT: DESG: CHKD: DATE:

External Stability - Static

Vertical Loads and Resisting Moments @ toe of Wall

Vertical Loads	Arm	Moment (k·ft)
$V_1 = 20.55' \times 86 \times .140 = 247.42^k$	67'	16577.14
$V_2 = 38.48' \times 94 \times .140 = 506.40^k$	63'	31903.2
$V_3 = 37.17' \times 102 \times .140 = 530.79^k$	59'	31316.61
$V_4 = 45.01' \times 110 \times .140 = 693.15^k$	55'	38123.25
$V_5 = 1/2(40)(20)(.140) = 56^k$	50.67'	2837.52
$V_6 = 46 \times 20 \times .140 = 128.8^k$	87'	11205.6
$P_{SV} = 2' \times .140 \times 161.21' \times .27269 \times (\sin 4.0507°) = .87^k$	110'	95.7
$P_{aV} = 1/2(161.21)^2 \times .140 \times (.27269) \times (\sin 4.0507°) = 35.04^k$	110'	3854.4
$\Sigma = 2198.47^k$		$\Sigma = 135913.46^{k \cdot ft}$

Horizontal Loads: overturning Moments about toe of wall

	Arm	Moment
$P_{aH} = 1/2(161.21)^2 \times .140 \times .27269 \times \cos 4.0507° = 494.84^k$	53.74'	26590.77
$P_{SH} = 2 \times .140 \times 161.21 \times .27269 \times \cos 4.0507° = 12.28^k$	80.61'	989.67
507.11^k		Σ 27580.44 k·ft

Safety factors:

1) overturning $= \frac{135913.46}{27580.44} = 4.93 > 2.0$ ok

2) Sliding $= \frac{\tan 35° (2198.47)}{507.11} = 3.04 > 1.50$ ok

Bearing Pressure

Vertical Loads	Arm	Moments
2198.47^k		$135913.46^{k \cdot ft}$
$2' \times 46 \times .140 = 12.88^k$	87'	$1120.56^{k \cdot ft}$
2211.35		$137034.02^{k \cdot ft}$

Horizontal Loads

$\Sigma = 507.11^k$

Moments

$27580.44^{k \cdot ft}$

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LOCATION: PROJECT NO: SHEET 7 OF

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eccentricity (AASHTO 96 Fig 5.8.2c)

$$e = \frac{110}{2} - \frac{137034.02^{k \cdot ft} - 27580.44^{k \cdot ft}}{2211.35^{k}} = 5.50'$$ (3)

Bearing Pressure (AASHTO 96 Fig 5.8.2.c)

$$\sigma_v = \frac{\Sigma V}{B-2e} = \frac{2211.35}{110-2(5.50)} = 22.34 \text{ ksf}$$ (4)

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LOCATION: PROJECT NO:

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Seismic Design:

Kh = horizontal seismic coeff. = a_m/g

$a_m/g = (1.45 - a_o/g)\, a_o/g = (1.45 - .36)\, .36 = .3924$
(AASHTO 96 Eq. 5.8.10.1-1)

$$K_{ae} = \frac{\cos^2(\phi-\theta)}{\cos\theta\cos(\delta+\theta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-i)}{\cos(\delta+\theta)\cos i}}\right]^2}$$ (See TB-MSE-9 Sheet 11)

ϕ = angle of internal friction of soil = 35°

~~θ = arc tan $Kh/(1-Kv)$~~ Neglecting vertical accelerations in accordance with section 2.2.)

θ = arctan kh = 21.425°

δ = angle of friction between soil and structure (for standard RE design $\delta = i$)

i = backfill slope angle = 4.0507°

$$K_{ae} = \frac{\cos^2(35-21.425)}{\cos 21.425 \cos(21.425+4.0507)\left[1+\sqrt{\frac{\sin(35+4.0507)\sin(35-21.425-4.0507)}{\cos(4.0507+21.425)\cos 4.0507}}\right]^2}$$

$= .6260$

See TB-MSE-9 SH 10-12.

$\Delta K_{ae} = K_{ae} - K_a = .6260 - .27269 = .3533$

$P_{ae} = 1/2\, \gamma H^2 \Delta K_{ae}$ = Dynamic horizontal thrust

$P_{ir} = 1/2\, \gamma H^2 A_m$ = effective Inertia Force (AASHTO 96 5.8.10.1-3)

$H_{eq} = H + H/2(\tan i)/(1 - \frac{\tan i}{2})$ $= 141.21 + \left[\frac{141.21}{2}(\tan 4.0507°)\right]/\left(1 - \frac{\tan 4.0507}{2}\right)$
(AASHTO 96 Eq. 5.8.10.1-4)
$= 146.39'$

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LOCATION: PROJECT NO: SHEET 9 OF

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External Stability - Seismic

Vertical Loads	Arm	Moment
Static = 2197.60		135817.82
$P_{AE_{Vert}} = \frac{1}{2} \cdot \frac{1}{2} \cdot .140 \times (146.39)^2 \times \sin 4.0507° \times .3533 = 18.72$	$.5(146.39) = 73.20$	1370.15
$\Sigma V = 2216.32$		137187.97

Horizontal Loads	Arm	Moment
$P_{aH} = 494.83$	53.74	26590.77
$P_{ae_H} = \frac{1}{2} \cdot \frac{1}{2} \cdot (140) \cdot (146.39)^2 \times \cos 4.0507° \times .3533 = 264.33$	87.84'	23217.43
$E_i = .3924 [(\frac{1}{2}(146.39)(141.21)(.140) - W_1 - W_2 - W_3]$ $- W_1 = .140(8)(37.17) = 41.63$ $- W_2 = .140(16)(38.48) = 86.20$ $- W_3 = .140(24)(20.55) = 69.048$		
$E_i = .3924[\frac{1}{2}(146.39)(141.21)(.140) - 41.63 - 86.20 - 69.048) = 490.57^K$	70.61'	32074.46
$E_{i_2} = .3924 [\frac{20 \times 40}{2} \times .140] = 21.97^K$	$141.21 + 20/3 = 147.88'$	3249.50
$E_{i_3} = .3924 [\frac{146.39}{2} - 24 + 40] \times 20 \times .140 = 10.10^K$	$141.21 + 20/2 = 151.21$	1527.93
$\Sigma H = 1281.81^K$		Σ 86660.09

(AASHTO 96 Eq. 5.8.9.1-6)

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LOCATION: PROJECT NO: SHEET 10 OF

SUBJECT: DESG: CHKD: DATE:

~~Safety Factors~~

overturning = 137187.97 / 86660.09 = 1.58 > 1.50 ok (5)

Sliding = $\frac{\tan 35°(2216.32)}{1281.81}$ = 1.21 > 1.10 ok (6)

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LOCATION: PROJECT NO: SHEET 11 OF

SUBJECT: DESG: CHKD: DATE:

Internal Stability - static

Horizontal earth pressure at a specific reinforcing strip level is calculated in the following manner:

1) Calculate bearing pressure at that particular level based on Meyerhoff:

a) with traffic surcharge above RE volume for calculation of maximum stress.

b) without traffic surcharge above RE volume for calculation of effective length safety factor for bond

2.) Multiply bearing pressure by the coeff of lateral earth pressure, K, for that particular level in order to calculate the horizontal earth pressure at the strip level to be examined.

- K is based on the height of overburden directly above the location of maximum tension at a strip level. This location is on the failure surface.

$K = K_o - (\text{depth})\left(\frac{K_o - K_a}{20'}\right)$ to a depth of 20'

Below 20' $K = K_a$ (Sec. 5.8.4.1 AASHTO96)

- The apparent coefficient of friction, f* is a function of the average height of overburden between the failure surface and the end of the strips at a specific strip level.

$f^* = 2.0 - H_{av}\,\frac{2.0 - \tan\phi_{sel}}{20'}$ to a depth of 20'

$f^* = \tan\phi_{sel}$ below 20'

(AASHTO 96 Sec 5.8.5)

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LOCATION: PROJECT NO: SHEET 12 OF

SUBJECT: DESG: CHKD: DATE:

Internal Stability - Static @ level 120.30'

Vertical Loads

$V_1 = 247.42^k$ } See sheet 6	67'	16577.14
$V_2 = 506.40^k$ } See sheet 6	63'	31903.2
~~$V_3 = 530.79^k$~~	59'	31316.61
$V_4 = \boxed{120.3 - 20.55 - 38.48 - 37.17}(110)(.140) = 371.14^k$	55'	20412.7
$V_5 = 56^k$ } See sheet 6	50.67	2837.52
$V_6 = 128.8^k$ } See sheet 6	87'	11205.6
$P_{AV} = 1/2(.140)(120.3+20)^2(.27269)(\sin 4.0507) = 26.54^k$	110'	2919.4
$P_{SV} = 2'(.140)(140.3)(.27269)(\sin 4.0507) = .76^k$	110'	83.6
$V_S = 12.88$ (See sheet 6)	87'	1120.56

$\Sigma V = 1880.13^k$

$\Sigma M_r = 118376.33$

Horizontal Loads

$P_{SH} = 2'(.140)(140.3)(.27269)(\cos 4.0507) = 10.69^k$	70.15	$749.59^{k\cdot f}$
$S_H = 1/2(.140)(140.3)^2(.27269)(\cos 4.0507) = 374.80^k$	46.77	17528.02

$\Sigma H = 385.49^k$

$\Sigma M_o = 18277.61^k$

eccentricity @ 120.30'

$$e = \frac{110}{2} - \frac{118376.33 - 18277.61}{1880.13} = 1.76'$$

Bearing Pressure

$$\sigma_v = \frac{1880.13}{110 - 2(1.76)} = 17.66 \text{ KSF}$$

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$K = K_a$ below depth of 20'

$K = .24858$

Maximum Horizontal Stress

$\sigma_H = .24858(17.66) = 4.39^{ksf}$ 7 (AASHTO 98 eq. 5.8.4.1-3)

Reinf. Tension

$T =$ area of 2 half A panels $\times\ \sigma_H = 24.2 \times 4.39 = 106.24^K$ (AASHTO 98 eq. (5.8.4.1-4)

Tension/strip $= 106.24/17 = 6.25^K < 9.33^k$ ok 8

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Bond Calculations (excludes traffic surcharge)

$\Sigma V = 1867.25^{k}$ $\Sigma M_R = 117255.77^{k\cdot ft}$

$\Sigma H = 385.49^{k}$ $\Sigma M_o = 18277.61^{k\cdot ft}$

$$e_{bond} = \frac{110}{2} - \frac{117255.77 - 18277.61}{1867.25} = 1.99'$$

$$\sigma_{V\,bond} = \frac{1867.25}{110 - 2(1.99)} = 17.61^{ksf}$$

$$\sigma_{bond} = 17.61(.24858) = 4.378^{ksf}$$

Effective Length Safety Factor

$$SF = R/\sigma H \times A = \frac{7924.88}{4.378 \times 24.2} = 74.80 \; (9) > 1.50 \; OK$$

$$R = 2b \times l_{eff} \times h_{ave} \times \gamma \times f^{*} \times N = 0.328 \times 97.454 \times 138.37 \times .140 \times .7536 \times 17 = 7924.88^{k}$$

b = width of strip = 50mm

l_{eff} = effective length of strip = 97.454' (see sheet 15)

h_{ave} = ave. height of overburden = 138.37' (see sheet 16)

γ = unit weight = .140 kcf

f^{*} = apparent coeff. of friction = .7536 (see sheet 15)

N = Number of strips over area of 24.2^{ft^2} = 17

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LOCATION: PROJECT NO: SHEET 15 OF

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Apparent Coef. of Friction

$f^* = \tan\phi$ at depth greater than 20'

$f^* = \tan 37° = .7536$

(AASHTO 96 Sec. 5.8.5)

Effective Length

$H/2 = 141.21 \div 2 = 70.605'$

$.3H = .3(141.21) = 42.363'$

similar triangles

$$\frac{70.605}{42.363} = \frac{141.21 - 120.3}{X}$$

$X = 12.546$

$L_{eff} = 110 - 12.546 = 97.454'$

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Ave height of overburden

$$h_{av} = \left[\frac{\frac{a'+a}{2} \times \left(S - (\cancel{B-66})\right) + \left(a \times B - S\right)}{le} \right] + level$$

a = height of slope @ line of max tension = 12.546/2 = 6.273

a' = height of slope @ end of slope = 20'

S = Distance from back of panel to end of slope = 40'

H = 141.21'

B = 110'

le = 97.454'

level = 120.30

$$h_{av} = \left[\frac{\frac{6.273+20}{2} \times \left(40 - 12.546\right) + \left(20 \times (110 - 40)\right)}{97.454} \right] + 120.30$$

$= 138.37'$

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LOCATION: PROJECT NO: SHEET 17 OF

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(see AASHTO & sheets 16-21 TB-mSEQ)

Internal Stability @ level 120.30 Seismic

Weight of Active Zone over 2 columns of panels

~~$W_a = (\gamma)(\text{Area Active zone})(9.84)$~~

$W_a = .140(161.21 \times 48.363 \times 3/4 - \frac{20(40)}{2}) \times 9.84 = 7504.41$ Kips

$E_d = a_m/g \times w = .3924(7504.41) = 2944.73$ Kips

$\Sigma N_i L_i$ = sum of Density × effective length at each level
$= 46880.22$

$N_i L_i$ @ level $120.30 = 17 \times 97.454 = 1656.718$

$$T_d = \frac{E_d \times N_i L_i}{\Sigma N_i L_i} = \frac{2944.73 \times 1656.718}{46880.72} = 104.06 \text{ Kips}$$

$$\sigma_{h\,total\,seismic} = \frac{T_d}{Area} + \sigma_{H\,max\,static} = \frac{104.06^k}{24.2 ft^2} + 4.39^{ksf}$$

$= 8.69^{ksf}$ (10)

Tension Seismic

$$T = \frac{\sigma_{H\,total} \times 24.2}{N} = \frac{8.69 \times 24.2}{17} = 12.38^k \text{ (11)}$$

Effective Length safety Factor

$f_{seismic} = .8 f_{static}$
$= .8(7924.88) = 6339.9^k$

$$SF_{Bond\,seismic} = \frac{6339.9}{12.38 \times 17} = 30.20 > 1.10 \text{ ok}$$

(12)

COMMENTS: WEST WALL 181+25 DATE: 09/05/00
JOB NUMBER : 8079 DESIGNED :

MASS STABILITY AND MAXIMUM BEARING PRESSURE

Exposed height = 122.21ft,Total height incl. Embed =141.21ft
Top Tier, Exposed Height= 20.55ft, Real Height =22.55ft, Reinf. Length = 86ft
2nd Tier from top, Exposed Height= 38.48ft, Real Height =41.18ft, Reinf. Length = 94ft
3rd Tier from top, Exposed Height= 37.17ft, Real Height =40.07ft, Reinf. Length = 102ft

STATIC MASS STABILITY

VERTICAL LOADS(Kips)		MOMENT ARM	MOMENT
693.15	4th Tier, Reinf.Length= 110.00ft	55.00 ft	38123.47 Kip-ft
530.79	3rd Tier, Reinf.Length= 102.00ft	59.00	31316.47
506.40	2nd Tier from top, Reinf.Length= 94.00ft	63.00	31903.00
247.42	Top Tier, Reinf.Length= 86.00ft	67.00	16577.27
56.00		50.67	2837.33
128.80		87.00	11205.60
35.04		110.00	3854.67
0.87		110.00	95.64
2198.47			135913.46

HORIZONTAL LOAD		MOMENT ARM	MOMENT
494.83		53.74	26590.77
12.28		80.61	989.67
507.11			27580.44

SAFETY FACTORS 1) OVERTURNING 2 4.93 >=2.00 OK
2) SLIDING 1 3.04 >=1.50 OK

BEARING PRESSURE

VERTICAL LOADS	MOMENT ARM	MOMENT
2198.47		135913.46
12.88	87.00	1120.56
2211.35		137034.02

HORIZONTAL LOADS (SAME AS FOR MASS STABILITY, static case)
BEARING PRESSURE AT TOE OF WALL= 22.34Ksf 4
ECCENTRICITY= 5.50ft<= B/6 =18.33ft, OK 3

MASS STABILITY- SEISMIC CASE

VERTICAL LOADS		MOMENT ARM	MOMENT
693.15		55.00	38123.47
530.79			31316.47
506.40			31903.00
247.42			16577.27
56.00			2837.33
128.80			11205.60
35.04	= Pa x sin(i)	110.00	3854.67
18.72	= Pae x sin(i)	110.00	2059.06
2216.32			137876.87

HORIZONTAL LOAD			
494.83	= Pa x cos(i)	53.74	26590.77
264.33	= Pae x cos(i)	87.84	23217.43
490.57	= Ei	70.61	32074.46
21.97	= Eis1	147.88	3249.50
10.10	= Eis2	151.21	1527.93
1281.81			86660.09

SAFETY FACTORS 5 1) OVERTURNING 1.59 >=1.50 OK
6 2) SLIDING 1.21 >=1.1 OK

ECCENTRICITY= 31.89ft<= B/3 =36.67ft
DESIGN TYPE : 2.00 :1 SLOPING BACKFILL OVER 40.00ft FROM BACK FACE OF WALL
EQUIV. HEIGHT L.L. SURCH.= 2.00ft or 0.28Ksf
COEFFICIENT OF ACTIVE EARTHPRESSURE = Ka = 0.2727
Select Backf'l =0.140Kcf, Phi.sel = 37.00deg., Random Backf'l = 0.140Kcf, Phi.random =35.00deg
Coefficient of friction of wall/ found. = 0.70, Area of A panel = 24.21sqft
f*= Coefficient of apparent friction = 2.00
HORIZONTAL ACCELERATION USED FOR SEISMIC DESIGN = ao/g = 0.36

JOB NUMBER : 8079 COMMENTS: WEST WALL 181+25 DESIGNED : MJB

REINFORCED EARTH INTERNAL STABILITY SUMMARY

Exposed height = 122.21ft,Total height incl. Embed =141.21ft

STEEL COST PER SQFT = $35.17

Top Tier, Exposed Height= 20.55ft, Real Height =22.55ft, Reinf. Length = 86ft
2nd Tier from top, Exposed Height= 38.48ft, Real Height =41.18ft, Reinf. Length = 94ft
3rd Tier from top, Exposed Height= 37.17ft, Real Height =40.07ft, Reinf. Length = 102ft

DESIGN TYPE : 2.00 :1 SLOPING BACKFILL OVER 40.00ft FROM BACK FACE OF WALL
EQUIV. HEIGHT L.L. SURCH.= 2.00ft or 0.28Ksf
COEFFICIENT OF ACTIVE EARTHPRESSURE = Ka = 0.2727
Select Backf'l =0.140Kcf, Phi.sel = 37.00deg., Random Backf'l = 0.140Kcf, Phi.random =35.00deg
Coefficient of friction of wall/ found. = 0.70, Area of A panel = 24.21sqft
f* = Coefficient of apparent friction = 2.00

	LEVEL	REINF. TYPE	Density	STRIP LENGTH	STATIC MAX. HORIZ. STRESS Ksf	STATIC Reinf. Tension <= 9.33Kips	STATIC EFFECT. Length Safety FACTOR >= 1.5	SEISMIC MAX. HORIZ. STRESS	SEISMIC Reinf. Tension <= 7.37Kips(1) <= 12.41Kips(2)	SEISMIC EFFECT. Length Safety FACTOR >=1.10
TOP TIER	2.250	50x4mm	4	86.00	0.65	3.95	10.12	0.86	5.23	4.44
	4.100	50x4mm	4	86.00	0.72	4.34	9.69	0.93	5.60	4.48
	6.560	50x4mm	4	86.00	0.80	4.86	9.23	1.01	6.11	4.53
	9.020	50x4mm	4	86.00	0.89	5.38	8.87	1.09	6.62	4.56
	11.480	50x4mm	5	86.00	0.98	4.72	10.73	1.28	6.19	5.30
	13.940	50x4mm	5	86.00	1.06	5.14	10.42	1.36	6.61	5.35
	16.400	50x4mm	6	86.00	1.15	4.63	12.20	1.55	6.25	6.07
	18.860	50x4mm	6	86.00	1.23	4.98	11.93	1.64	6.61	6.13
	21.320	50x4mm	6	86.00	1.32	5.33	11.71	1.73	6.96	6.18
2nd TIER	22.800	50x6mm	4	94.00	1.26	7.64	10.32	1.56	9.42	5.74
	26.060	50x6mm	4	94.00	1.38	8.33	10.01	1.67	10.13	5.74
	28.520	50x6mm	4	94.00	1.46	8.86	9.81	1.76	10.68	5.74
	30.980	50x6mm	5	94.00	1.55	7.50	12.04	1.97	9.55	6.74
	33.440	50x6mm	5	94.00	1.64	7.92	11.84	2.07	10.00	6.75
	35.900	50x6mm	5	94.00	1.72	8.34	11.65	2.16	10.44	6.76
	38.360	50x6mm	5	94.00	1.81	8.76	11.48	2.25	10.90	6.77
	40.820	50x6mm	5	94.00	1.90	9.18	11.32	2.34	11.35	6.77
	43.280	50x6mm	6	94.00	1.98	8.00	13.40	2.56	10.32	7.75
	45.740	50x6mm	6	94.00	2.07	8.35	13.24	2.65	10.71	7.76
	48.200	50x6mm	6	94.00	2.16	8.70	13.08	2.75	11.10	7.76
	50.660	50x6mm	6	94.00	2.24	9.05	12.93	2.85	11.50	7.76
	53.120	50x6mm	7	94.00	2.33	8.06	14.92	3.07	10.61	8.71
	55.580	50x6mm	7	94.00	2.42	8.36	14.76	3.17	10.95	8.72
	58.040	50x6mm	7	94.00	2.50	8.66	14.61	3.27	11.30	8.72
	60.500	50x6mm	7	94.00	2.59	8.96	14.48	3.37	11.65	8.73
3rd TIER	61.280	50x6mm	7	94.00	1.18	4.08	80.70	1.53	5.28	23.04
	63.430	50x6mm	7	102.00	2.48	8.59	20.28	3.28	11.35	11.26
	65.890	50x6mm	7	102.00	2.56	8.87	20.65	3.40	11.75	11.50
	68.350	50x6mm	7	102.00	2.64	9.15	21.00	3.51	12.14	11.74
	70.810	50x6mm	8	102.00	2.72	8.25	24.41	3.78	11.45	13.12
	73.270	50x6mm	8	102.00	2.81	8.49	24.80	3.90	11.81	13.38
	75.730	50x6mm	8	102.00	2.89	8.73	25.19	4.03	12.18	13.64
	78.190	50x6mm	9	102.00	2.97	7.98	28.77	4.32	11.61	15.02
	80.650	50x6mm	9	102.00	3.05	8.19	29.19	4.45	11.96	15.29
	83.110	50x6mm	10	102.00	3.13	7.57	32.88	4.75	11.50	16.65
	85.570	50x6mm	10	102.00	3.21	7.77	33.31	4.88	11.82	16.94
	88.030	50x6mm	10	102.00	3.29	7.96	33.74	5.02	12.15	17.21
	90.490	50x6mm	11	102.00	3.37	7.41	37.56	5.34	11.76	18.56
	92.950	50x6mm	11	102.00	3.45	7.59	38.00	5.49	12.07	18.85
	95.410	50x6mm	12	102.00	3.53	7.12	41.91	5.82	11.75	20.17
	97.870	50x6mm	12	102.00	3.61	7.29	42.45	5.97	12.04	20.49
4TH TIER	98.450	50x6mm	13	102.00	2.27	4.23	119.81	4.39	8.17	33.68
	100.620	50x6mm	13	110.00	3.55	6.61	55.76	6.31	11.75	24.19
	103.080	50x6mm	13	110.00	3.64	6.77	55.96	6.48	12.06	24.45
	105.540	50x6mm	13	110.00	3.72	6.93	56.16	6.64	12.36	24.71
	108.000	50x6mm	14	110.00	3.81	6.59	60.69	7.04	12.17	25.99
	110.460	50x6mm	15	110.00	3.90	6.29	65.24	7.45	12.02	27.23
	112.920	50x6mm	15	110.00	4.02	6.49	65.44	7.63	12.31	27.50
	115.380	50x6mm	16	110.00	4.14	6.27	70.01	8.05	12.19	28.72
	117.840	50x6mm	17	110.00	4.27	6.07	74.60	8.49	12.09	29.91
	120.300	50x6mm	17	110.00	4.39 7	6.25 8	74.80 9	8.68 10	12.36 11	30.20 12
	122.760	50x6mm	19	110.00	4.66	5.94	81.21	9.53	12.14	31.72
	125.220	50x6mm	19	110.00	4.80	6.12	81.34	9.74	12.41	32.01
	127.680	50x6mm	20	110.00	4.94	5.98	85.74	10.22	12.37	33.10
	130.140	50x6mm	21	110.00	5.08	5.86	90.14	10.71	12.34	34.18
	132.600	50x6mm	22	110.00	5.23	5.75	94.53	11.21	12.33	35.24
	135.060	50x6mm	23	110.00	5.38	5.66	98.91	11.72	12.33	36.27
	137.520	50x6mm	25	110.00	5.53	5.35	107.58	12.51	12.12	37.97
	139.980	50x6mm	25	110.00	5.68	5.50	107.62	12.76	12.36	38.29

SEISMIC DESIGN OF REINFORCED EARTH RETAINING WALLS AND BRIDGE ABUTMENTS

AASHTO Design Method
For
Reinforced Earth Structures
Subject to Seismic Forces

Technical Bulletin: MSE - 9

PART A: RETAINING WALLS

January 1995

INDEX

LIST OF FIGURES

PART A - RETAINING WALLS

1. INTRODUCTION

It is generally agreed that the stability of retaining walls exposed to earthquakes is not a matter for real concern.

In a paper delivered in 1970 at the ASCE Specialty Conference, Professors H. Bolton Seed and Robert V. Whitman said:

> "Few cases of retaining wall movement or collapse of walls located above the water table have been reported in the literature on earthquake damage. (...) it seems likely that the small number of accounts of retaining wall performance is not necessarily indicative of the lack of occurrence of wall movements: this type of damage is not particularly dramatic compared with other forms of earthquake damage and thus may often be considered of minor significance."

The same authors find confirmation of their view that the stability of retaining walls is not crucial based on the scant attention accorded to such structures in the construction codes:

> "While all investigators have concluded that the dynamic lateral pressures developed during earthquakes exceed the static pressures on earth retaining structures, a survey of a number of engineering companies highway departments and port authorities in California shows that (...) it is general practice to make no special allowance for increased lateral pressures on retaining walls (...) due to earthquake effects. This also appears to be the case in many other countries."

It is interesting to note that habits have not changed much over the last twenty years. Having recently done a survey similar to that of Seed and Whitman we note:

The seismic design of cantilever retaining walls is a subject on which there is not many guidelines. In fact, most highway departments do not design cantilever retaining walls for seismic loads. Instead they assume, based on previous performance, that static design is adequate. Conversations with the California static design is adequate. Conversations with the California Department of Transportation confirms this.

In fact, even the most detailed seismic design codes, such as the recommendations from the French Association for Seismic Engineering published in 1990, contain a few rather simplistic rules for standard retaining walls, together with extremely complex design methods for building structures.

In their ASCE communications, Seed and Whitman explained why the stability of retaining walls during earthquakes was a problem which very often resolved itself. Considering the order of magnitude of the additional stresses caused by the effects of "normal" earth tremors, and the usual values of safety coefficients, they state:

> "It should be noted that the factor of safety provided in the design of walls for static pressures may be adequate to prevent damage or detrimental movements during many earthquakes. (...) Thus where backfill and foundation soils remain stable, it is only in areas where very strong ground motions might be expected, for walls with sloping backfills or heavy surcharge pressures and for structures which are very sensitive to wall movements, that special seismic design provisions for lateral pressure effects may be necessary."

Such considerations of a very general nature obviously also apply to Reinforced Earth structures which, better than any other type of structure, are known to be able to withstand deformation without damage. Their performance record provides ample proof of this. Many Reinforced Earth structures have been built in seismic zones, usually without any special precautions or extra reinforcement for earthquakes. Some have already been tested by an actual earthquake and have been unaffected.

In Friuli, Italy, four small Reinforced Earth walls 15 to 20 feet in height were at the epicenter of the 1976 earthquake (6.4 Richter magnitude). The design of these walls was based on the minimum requirements for static conditions only. There was no additional reinforcement density or length provided, yet no damage occurred to these walls.

In Japan, most structures are located in a seismic zone; design calculations include a check for earthquake effects, but the final design will, in practice, be based on the routine static approach. In 1983, a serious 7.7 Richter magnitude earthquake occurred in the Akita area, causing considerable damage to buildings, bridges, and port installations. None of the 24 local Reinforced Earth structures suffered any damage. (Report available).

In 1989, the Loma Prieta Earthquake, a severe 7.1 Richter magnitude event, shook the San Francisco area, causing serious damage to bridges and buildings. Only three privately owned walls out of the 20 Reinforced Earth structures located in the area were designed for earthquake loading conditions. the remaining Reinforced Earth structures, with the exception of one, are owned by Caltrans who has no earthquake design requirements for retaining walls. All 20 of the Reinforced Earth structures whether designed for earthquake resistance or not, performed without any damage. (Report available)

In 1994, the Northridge earthquake, a severe 6.7 Richter magnitude event, shook the densely populated San Fernando Valley, 20 miles northwest of Los Angeles. Severe damage occurred to buildings, bridges and freeways. Twenty-one Reinforced earth walls and 2 Reinforced Earth bridge abutments were located within the effected area. One-half of the walls and the two bridge abutments, were designed for seismic loads; the others were not. The Reinforced Earth structures performed extremely well, with only superficial damage to one wall, whether specifically designed for earthquake loads or not. (Report available)

These observations confirm that, since no particular provisions for earthquake effects are normally required when designing conventional retaining structures, they may be even less necessary for Reinforced Earth retaining structures due to their outstanding performance record, inherent strength, flexibility, and high degree of damping. And yet, we have always applied special design rules to Reinforced Earth structures built in recognized seismic zones. The practical design method presented in this report, and adopted by the AASHTO technical committee in 1992, is the result of research carried out over fifteen years with the assistance of leading experts. Tests on reduced-scale models, measurements in full-scale test structures subjected to vibration, research led by specialists, such as the late Professor Seed[1], assembling and processing the research results, and finally, in 1989, a series of dynamic finite element computations enabled us to further refine

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1 The late Professor H. Bolton Seed of the University of California at Berkeley is frequently cited in this report. It was the review and evaluation he performed together with Professor James K. Mitchell which helped us develop an understanding for how a Reinforced Earth structure will react to seismic motion. On the basis of his great experience and sure instincts, Professor Seed proposed a number of simple rules in this synthesis; our finite element models have since provided resounding confirmation of their validity.

our seismic design method. The practical design method presented in this report explains in detail, the method outlined in the 1994 AASHTO interim specifications for highway bridges.

It should be noted that it is rare for seismic design calculations to result in a significant increase in reinforcements in a Reinforced Earth structure. However, this design method allows us to make such decisions, where advisable, for particularly earthquake-prone regions with high acceleration coefficients, or in the case of structures with special geometry or loading conditions.

2. GENERAL

2.1 Forward

As is customary, the design method distinguishes between the verification of safety factors for external stability and those relating to internal stability.
Verification of safety factors with respect to sliding and overturning for external stability will follow relevant rules and regulations set forth in the 1994 AASHTO Interim Specifications for design of highway bridges.

The method for calculating internal stability, also outlined in the 1994 AASHTO Interim Specifications, is based on a specific analysis of the behavior of Reinforced Earth structures exposed to seismic forces. It must therefore be strictly adhered to, totally disregarding calculation methods developed for other types of structures.

2.2 Dynamic forces - Definitions

Dynamic forces, or more accurately, pseudo-static forces play a role in these calculations. The type of pseudo-static force to be considered depends on whether one is concerned with external stability or internal stability.

2.2.1 External Stability (Figure 1)

From the applied horizontal seismic accelerations, two supplementary horizontal forces develop:

P_{ae} = an increase in pressure from the earth retained by the structure.

P_{ir} = an overall inertia load, proportional to the weight of the effective Reinforced Earth mass.

An upward or downward variation in the weight of the structure is possible due to vertical accelerations. However, the vertical accelerations are considered secondary compared to the horizontal accelerations and are therefore generally ignored.

In the paper delivered in 1970 at the ASCE Specialty Conference, Professors H. Bolton Seed and Robert V. Whitman stated:

> "Since for most earthquakes the horizontal acceleration components are considerably greater than the vertical acceleration components, it seems reasonable to conclude that in such cases the influence of the vertical acceleration component K_v can be neglected for practical purposes."

2.2.2 Internal Stability (Figure 2)

Only one supplementary horizontal force is included:

P_i = an internal dynamic force, the sum, in fact, of the additional tensile forces occurring in the reinforcing strips which is simply equal to the inertia of the active zone.

2.3 The Accelerations to be Taken Into Account

- The dynamic or pseudo-static forces are functions of "A_m", the average maximum horizontal acceleration occurring in the Reinforced Earth structure and the ground behind the structure (the term "maximum" is with respect to time, while "average" relates to the height of the structure).

- The acceleration "A_m" is related to the maximum horizontal acceleration "A" which is presumed to occur at the level of the free surface of the natural ground at the site, for a given earthquake and class of risk.

This acceleration "A" (known as the "free field" acceleration), having been somewhat influenced by the presence of the Reinforced Earth structure on the site, becomes gradually greater towards the surface of the reinforced backfill (Figures 3a and 3b). On average, the greater the acceleration "A" the less pronounced the amplification with height. In practical terms, for any site where:

$$0.05 < A < 0.45$$

the average maximum horizontal acceleration, A_m, in the Reinforced Earth structure and the ground behind can be taken as:

$$A_m = (1.45 - A)A \quad \text{(Figure 4)}$$

The free field acceleration "A" is a function of the structure's location with respect to an active fault and the nature of the foundation soils. If the value of "A" is not indicated by the owner or their agent, the value can be assumed as the acceleration coefficient "A" obtained from figure 1-5 of the 1991 AASHTO interim specifications for highway bridges (See appendix). Note, the accelerations given on the contour map are expressed as percent of gravity. Therefore, these values must be divided by 100 to obtain the decimal percent acceleration to be used in the design calculations.

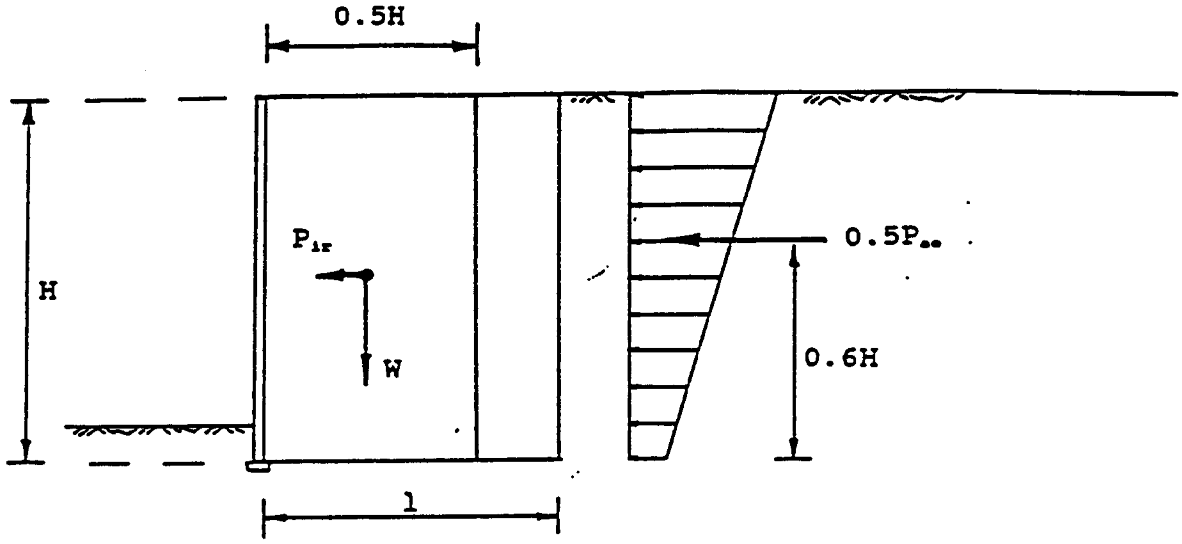


Figure 1: External Stability, Supplementary Forces

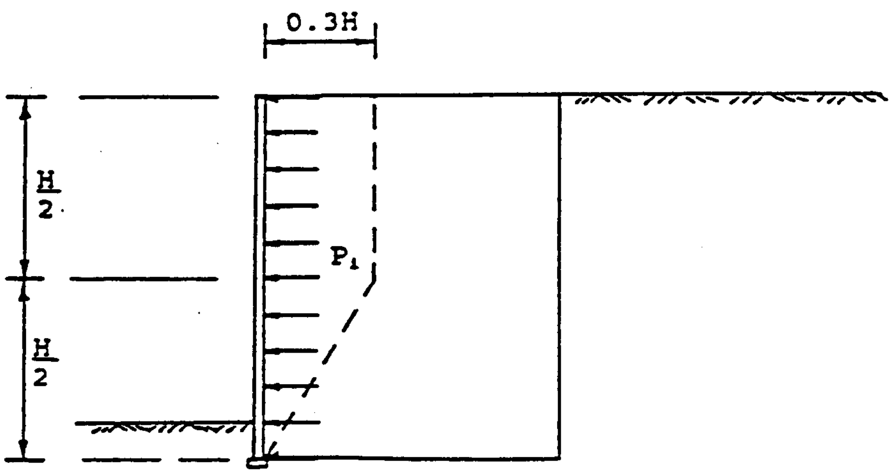


Figure 2: Internal Stability, Supplementary Force

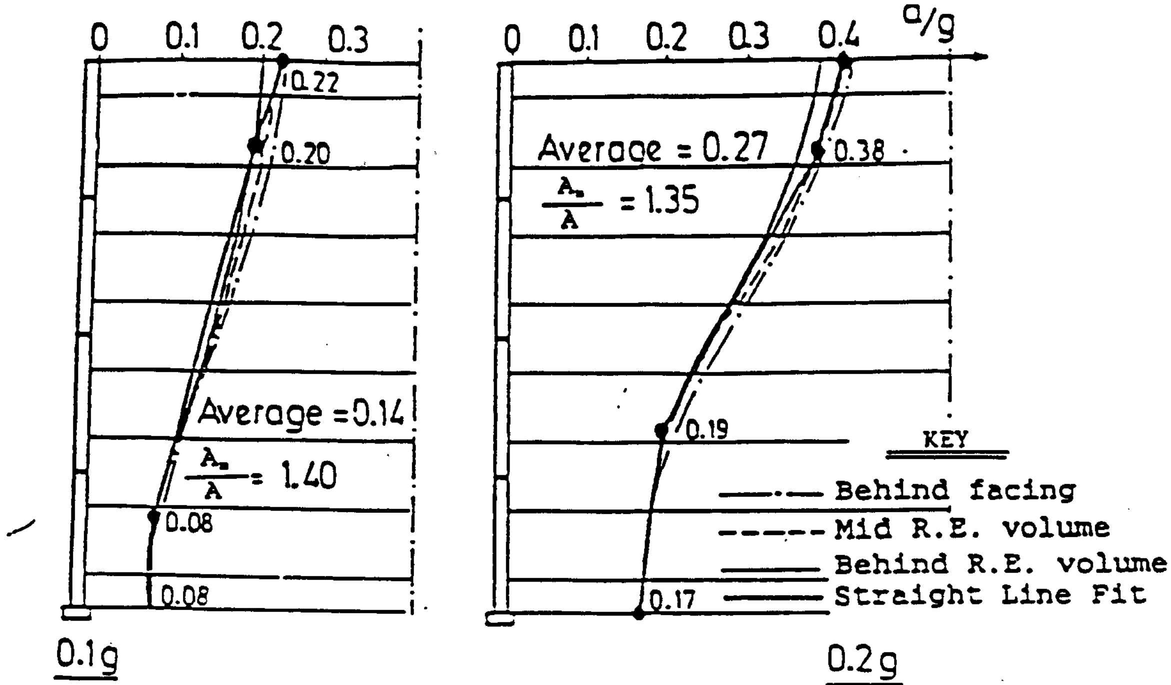


Figure 3a: Maximum accelerations within and behind the Reinforced Earth volume, 19.7 ft. wall (Superflush)

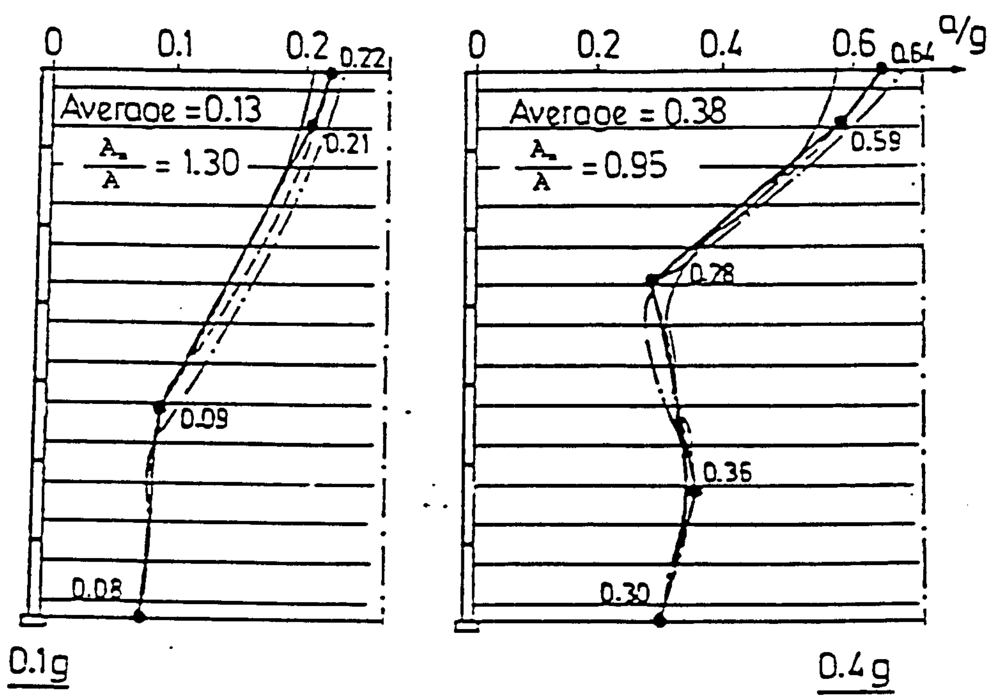


Figure 3b: Maximum accelerations within and behind the Reinforced Earth volume, 34.5 ft. wall (Superflush)

2.4 Load Combination

Seismic loads are generally considered to be accidental in nature, with a single degree of aggressiveness and no load factor. The combined loads to be taken into account when verifying the stability of the structure, both externally and internally, fall under AASHTO service load group VII. Group VII considers dead load, earth pressure, buoyancy, stream flow pressure, and the earthquake forces. Live loads are not considered in a seismic analysis.

Table 3.22.1a and the applicable text, of the AASHTO standard specifications for highway bridges are presented in the appendix for reference.

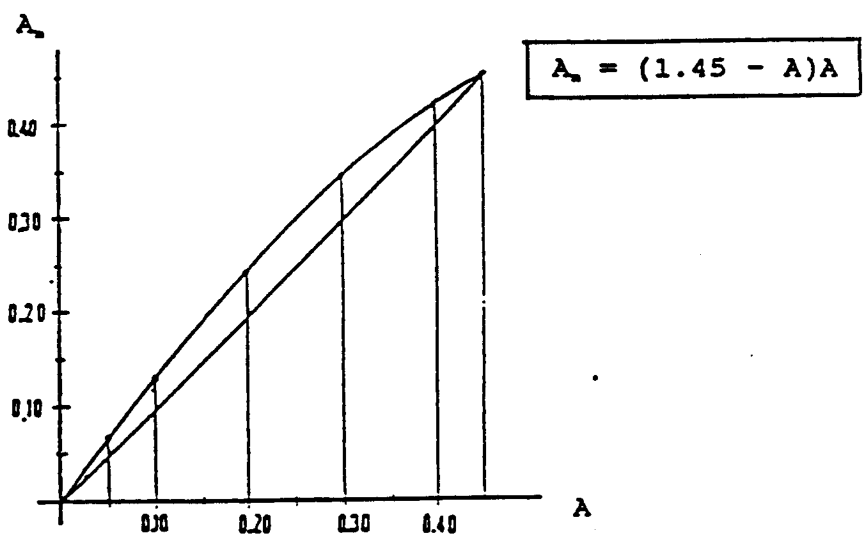


Figure 4: Average maximum acceleration, A_m, depending on the "free field" acceleration, A

2.5 Factors of Safety and Allowable Stress

Increased allowable stress and reduced factors of safety are acceptable during seismic events due to the temporary nature of the loading condition. It is generally acceptable to allow 133% of the allowable static stresses and 75% of the required static safety factors for dynamic conditions associated with an earthquake event.

External Stability	Static	Seismic
F.S. with respect to base sliding:	1.5	1.1
F.S. with respect to overturning:	2.0	1.5
F.S. with respect to bearing capacity:	2.0	Note 1

Internal Stability	Static	Seismic
Reinforcement Tensile Stress: (see note 2)	0.55 F_y (36 ksi)	0.73 F_y (48 ksi)
F.S. with respect to bond of Reinforcing Strips:	1.5	1.1

Note 1: A factor of safety of 2.0 with respect to foundation bearing capacity is considered acceptable for static conditions. Eccentricity of the structure and applied bearing pressure are not determined during a seismic event due to the temporary and transient nature of the loading condition. Bearing pressure at the toe of the structure during a seismic event should not vary appreciably from the static case. However, this commentary shall serve as a reminder that it may be necessary to check that an earthquake will not alter the inherent strength characteristics of the foundation soils.

Note 2: The reinforcement tensile stress presented above is the allowable reinforcement tensile stress at the end of the design service life. At time zero, the allowable tensile stress is considerably less to allow for a minimum sacrificial reinforcement thickness of 1.42mm for a 75 year service life and 1.77mm for a 100 year service life.

3. EXTERNAL STABILITY

3.1 Seismic coefficients

Two "seismic coefficients", K_h and K_v, must be defined before the dynamic horizontal thrust, P_{ae}, and the structure's inertia load, P_{ir} can be calculated. These coefficients are applied simultaneously and uniformly to all parts of the structure, i.e. to the retaining structure itself and to the ground behind the structure.

For gravity structures such as Reinforced Earth, the values assigned to these coefficients are:

$$K_h = A_m$$

$$K_v = 0.5K_h = 0.5A_m$$

The value selected for the seismic coefficient, k_h, equal to the average maximum horizontal acceleration, A_m, should be conservative. The use of one-half the dynamic thrust, $0.5P_{ae}$, as shown in Figure 1 takes into account the fact that particle acceleration is not at its maximum everywhere at the same moment, either in the wall, or in the ground it retains, and that some small horizontal displacement leading to stress release is acceptable. This is consistent with the recommendations of Professors Seed and Mitchell in their report, <u>Earthquake Resistant Design of Reinforced Earth Walls</u>, dated December 1981.

3.2 Determining the Dynamic Horizontal Thrust, P_{ae}

The additional dynamic horizontal thrust, P_{ae}, has the effect of increasing the static force, P. Stability computations shall be made by considering, in addition to static forces, the horizontal inertial force (P_{ir}) acting simultaneously with <u>50 percent</u> of the dynamic horizontal thrust ($0.5P_{ae}$). The dynamic horizontal thrust P_{ae} shall be evaluated using the pseudo-static Mononabe-Okabe method and shall be applied to the vertical rear boundary of the effective reinforced earth mass at a height of 0.6H from the base and the horizontal inertial force shall be applied at mid-height of the structure.

To find P_{ae}, we use the Mononabe-Okabe formula:

$$P_{ae} = 1/2\ \gamma\ H^2 \Delta K_{ae}$$

where:

$$\Delta K_{ae} = (1-K_v)K_{ae} - K_a$$

K_{ae} is a total earth pressure coefficient, including the seismic effect, and K_a is the static earth pressure coefficient. By effect, and K_a is the static earth pressure coefficient. By subtracting K_a from K_{ae}, we obtain ΔK_{ae} which represents the incremental increase in the earth pressure due to the earthquake event.

Calculation of the total earth pressure coefficient, K_{ae}, for a vertical wall, using the Mononabe-Okabe equation is as follows:

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

If $i > (\phi - \theta)$, then $(\phi - \theta - i)$ is assumed to be zero. The above relationship becomes:

$$K_{ae} = \frac{\cos^2(\phi - \theta)}{\cos\theta \cos(\delta + \theta)}$$

Calculation of the static earth pressure coefficient, K_a, for any backfill slope angle, i, is:

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

where:

ϕ = angle of internal friction of the soil

θ = arc tan $K_h/(1 - K_v)$

δ = angle of friction between soil and structure (Note for standard RE design, $\delta = i$)

k_h = horizontal seismic coefficient

k_v = vertical seismic coefficient

i = backfill slope angle

Neglecting vertical accelerations in accordance with section 2.2.1

$$\theta = \text{arc tan } K_h = \text{arc tan } A_m$$

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

3.2.1 Vertical Wall With Horizontal Backfill (figure 5)

For a vertical wall, with a horizontal backfill having an angle of internal friction of 30°, a free field acceleration equal to 0.4g, the value of P_{ae} may be calculated as follows:

$$P_{ae} = 0.375\ \gamma\ H^2 A_m$$

For other accelerations or for materials of differing shear strength, the value of P_{ae} may be calculated by computing the difference between K_{ae} and K_a to determine the seismic earth pressure coefficient, ΔK_{ae}. Therefore, the value of P_{ae} may be calculated as follows:

$$P_{ae} = 1/2\ \gamma\ H^2\ \Delta\ K_{ae} = 1/2\ \gamma\ H^2\ (K_{ae} - K_a)$$

In either case, one-half of the resultant dynamic thrust, $0.5P_{ae}$, is applied horizontally at 0.6H above the base of wall as shown in figure 5.

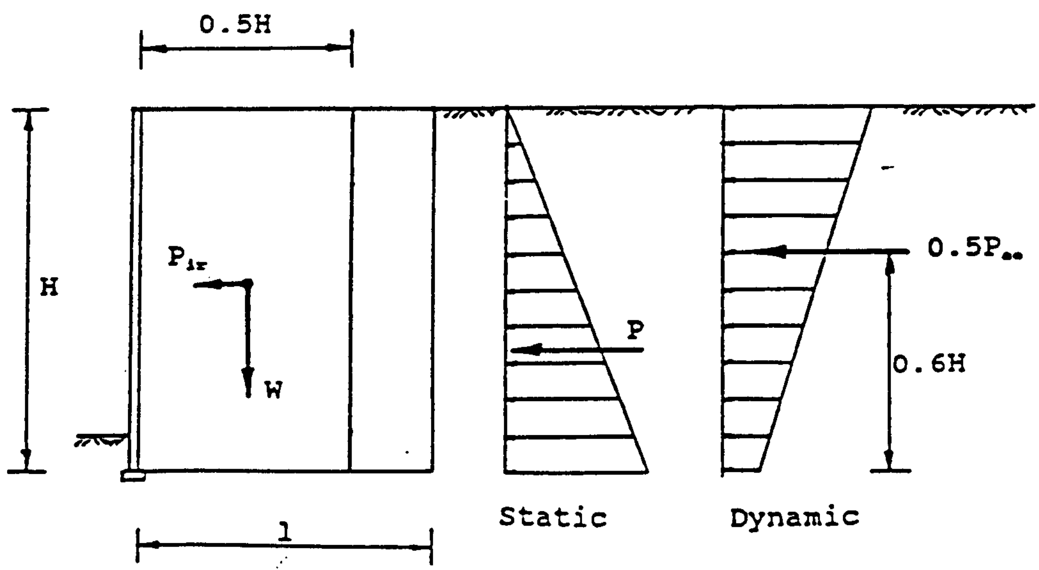


Figure 5: External stability - level surcharge condition

3.2.2 Vertical Wall With Sloping Backfill (Figures 6a and 6b)

For vertical walls with sloping backfill, the resultant seismic force, P_{ae}, is always calculated by working out the difference between K_{ae} and K_a to determine the seismic earth pressure coefficient, ΔK_{ae}. The procedure allows for the actual shear strength and slope angle of the soil being retained.

One-half of the resultant seismic force, $0.5P_{ae}$, is applied at $0.6H_2$ above the base of wall, acting parallel to the actual infinite slope or equivalent infinite slope at an angle of i with respect to the horizontal.

3.3 Effective Inertia Force P_{ir}

The effective inertia force, P_{ir}, is a horizontal load acting at the center of gravity of the effective mass. For a horizontal backfill condition (Figure 5), with W being the total weight of the effective mass, the effective inertia force is equal to:

$$P_{ir} = K_h W = 0.5\ \gamma\ H^2\ A_m$$

For a sloping surcharge condition (Figures 6a and 6b), the supplementary inertia force, P_{is}, caused by any soil situated above the effective mass shall be included in the computation. Therefore, the total inertia force becomes:

$$P_{ir} + P_{is} = K_h (W + W_s) = 0.5\ \gamma\ H_2\ A_m\ [H_t + 0.5\ (H_2 - H_t)]$$

where:

$$H_2 = H_t + \frac{0.5\ H_t \tan i}{1 - 0.5 \tan i}$$

In either case, the weight of the facing panels is omitted from the calculations as in the case for routine static stability calculations.

3.4 Performing the External Stability Calculations

The static stability of the structure is determined as normal, utilizing the minimum reinforcement lengths necessary to satisfy the required factors of safety for sliding, overturning and bearing, including a check of structure eccentricity (see section 2.5). In addition, the minimum reinforcement length for static stability should satisfy the minimum reinforcement length requirements of the project specifications.

The static thrust, P, is applied to the imaginary vertical rear boundary at the end of the reinforcements as shown in figures 5, 6a and 6b. Next, it is necessary to determine the geometry of the effective mass of the structure for the dynamic condition, which extends a distance of 0.5 H_2 behind the wall facing. Then, one-half of the dynamic thrust, $0.5P_{ae}$, is applied to the imaginary vertical rear boundary at 0.5 H_2 behind the wall facing acting simultaneously with the inertia of the effective mass, P_{ir} and P_{is}, if applicable. The dynamic forces are in addition to the static force used to determine the minimum reinforcement length required for static stability. See figures 5, 6a and 6b.

If the reinforcement length is required to be increased for adequate stability during the dynamic condition, the applied thrusts, P, 0.5 P_{ae}, P_{ir} and P_{is} are <u>NOT</u> changed. Only the resistance of the reinforced mass is increased as required to achieve the required stability safety. This procedure is logical since there is <u>no</u> reason for the applied thrusts from the embankment to increase just because the reinforcements get lengthened.

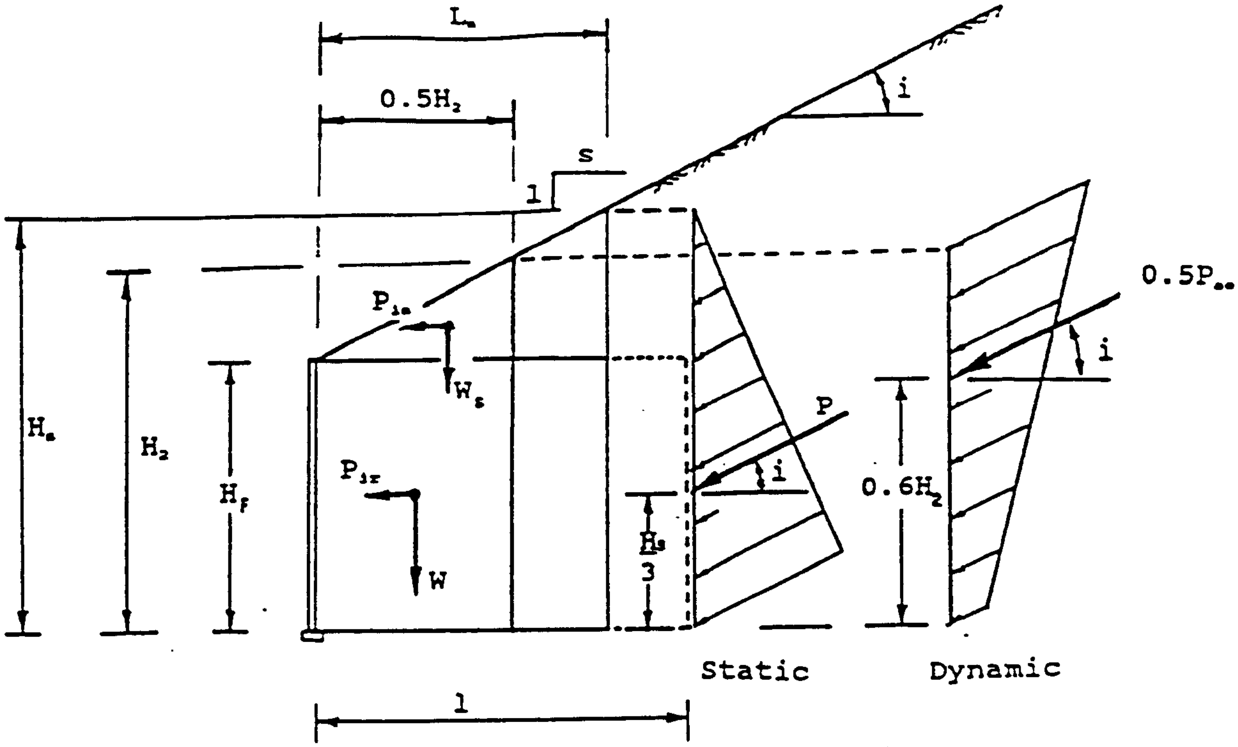


Figure 6a: External Stability - Infinite Slope Condition
NOTE: The reinforcement length, L, may need to be increased for stability, however the applied thrusts do NOT increase and remain applied to their respective imaginary vertical boundaries as shown.

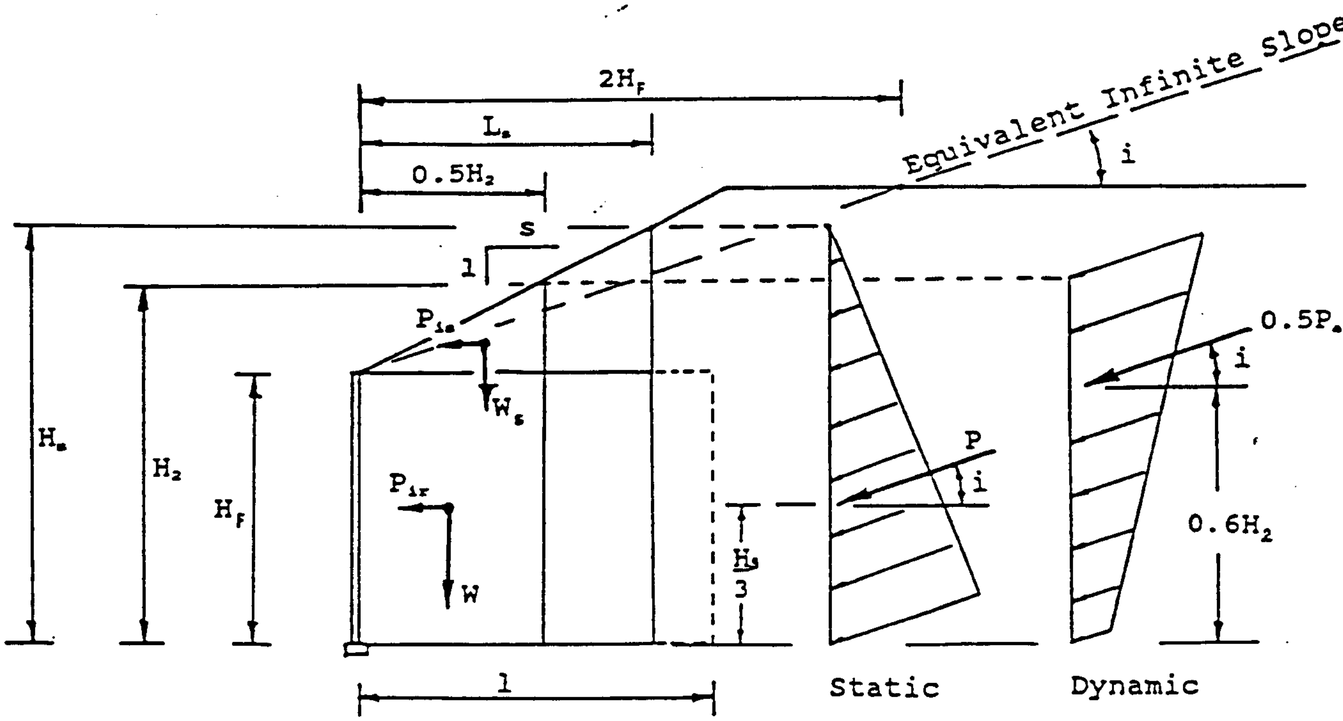


Figure 6b: External Stability - Broken Back Slope Condition

4. INTERNAL STABILITY

4.1 The Internal Dynamic Load, P_i

The internal dynamic load, P_i, which is distributed among the reinforcing strips and is added to the static tensile forces, is equal to the weight of the actual active zone (not the bilinear approximation), including any additional soil surcharge on top, multiplied by the average maximum horizontal acceleration, A_m.

Since calculations are generally performed using the bilinear envelope (Figure 7a) and not the actual active zone consisting of soil located inside the actual line of maximum tension (potential failure surface), a correction factor of 0.67 is required to adjust the volume of the active zone in the calculations.

For example, let W_a be the weight of fill in the bilinear active zone envelope (figure 7a), the internal dynamic load, P_i becomes:

$$P_i = 0.67W_a\ A_m$$

The geometry of the actual active zone, as verified by the dynamic F.E.M. results, is identical to that for static calculations. In the case of a basic structure with no additional soil surcharge load, the active zone envelope volume, V_a, is as shown in figure 7b and is equal to:

$$V_a = 0.75\ (0.3H\ X\ H) = 0.225H^2$$

Therefore, the internal dynamic load, P_i, becomes:

$$P_i = 0.67\ (0.225H^2)\ \gamma\ A_m = 0.15\ \gamma\ H^2A_m$$

4.2 Distribution of Dynamic Load P_i Among the Reinforcing Strips

The dynamic load, P_i is added to the maximum tensile forces, T_m, induced in the reinforcing strips by static loads, i.e.: the structure's own weight, applied static earth pressure, and the supplementary loads and pressures due to any dead load surcharge. The other loads of dynamic origin, $0.5P_{AE}$, or P_{IR}, are not taken into account in the calculation of the maximum tensile force T_m (figure 8a).

The dynamic load, P_i, is distributed among the individual reinforcing strips in proportion to their "resistant area", obtained by multiplying their width times their embedment length in the resistant zone.

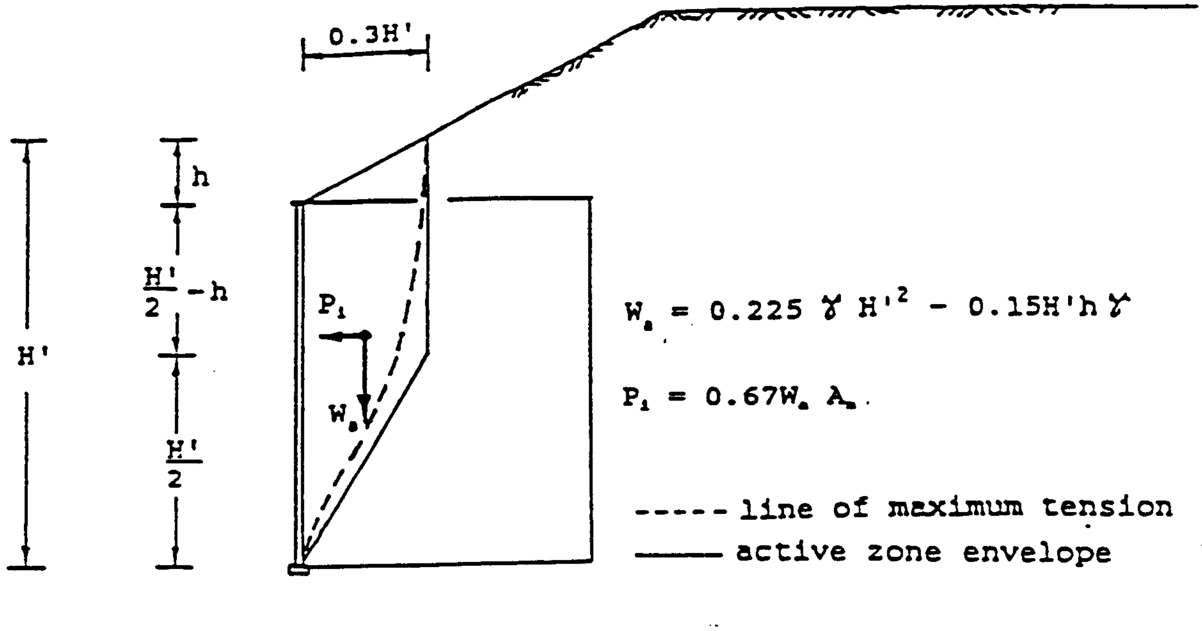


Figure 7a: Internal Stability - Sloping Condition

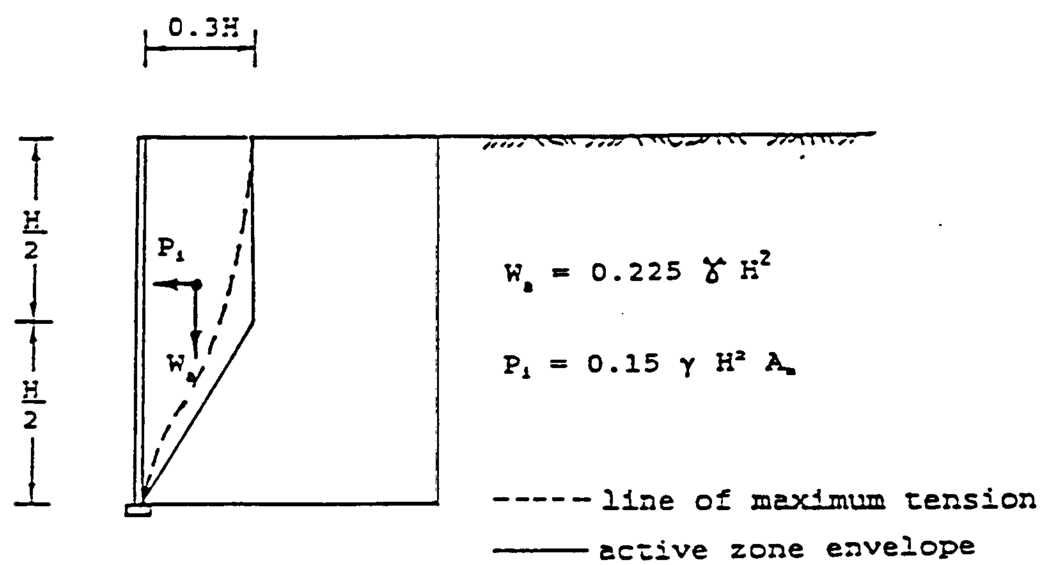


Figure 7b: Internal Stability - Level Condition

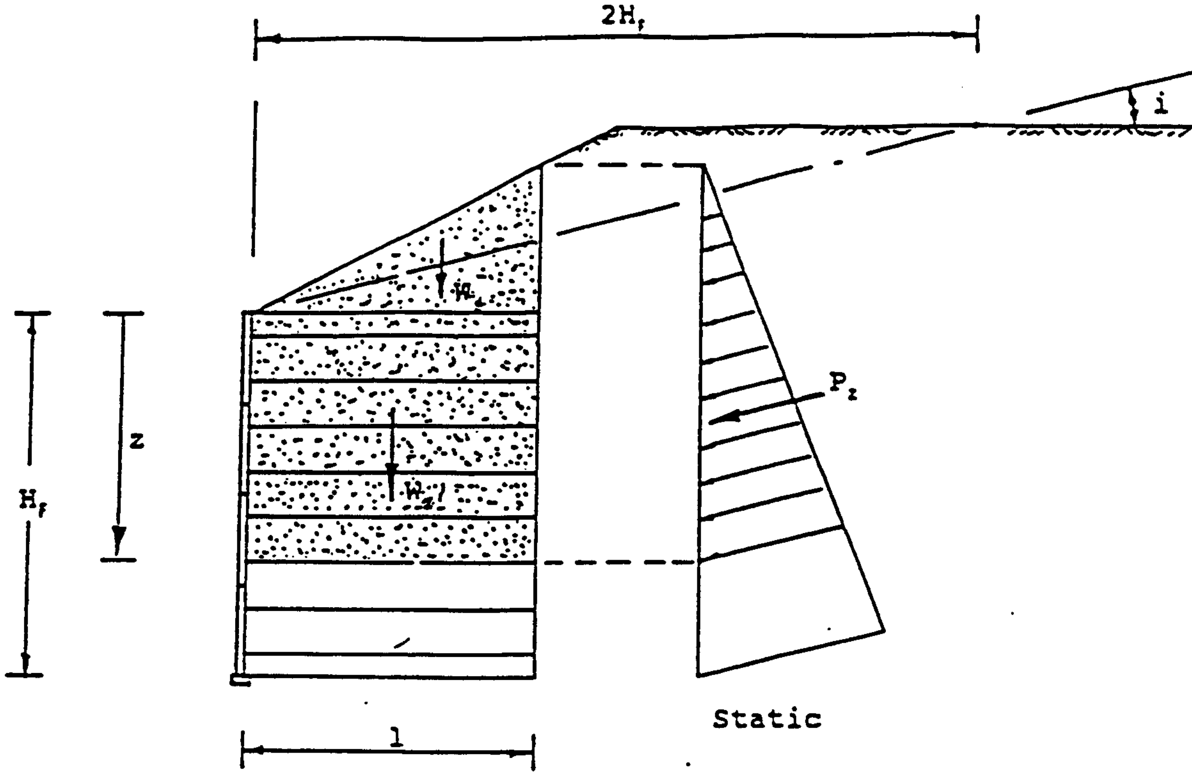


Figure 8a: Internal Stability - Loads Included in the Calculation of T_m

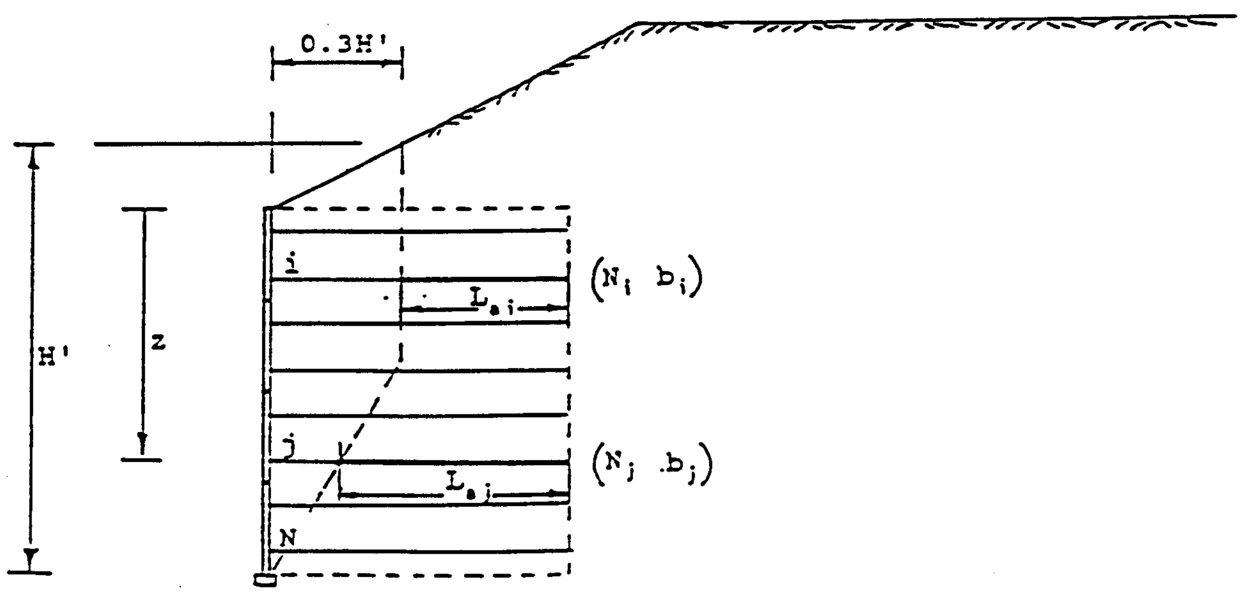


Figure 8b: Distribution of Dynamic Load Among the Strips

Thus in layer J (figure 8b), a reinforcing strip of width b_j, having a resistant length L_{aj}, the static tensile force, T_m, will be increased by an increment of the total dynamic load, ΔT_d equal to:

$$\Delta T_d = \left[\frac{b_j L_{aj}}{\sum_{i=1}^{i=N} n_i \, b_i \, L_{ai}} \right] P_i \text{ X } 9.84'$$

Where n_i is the number of reinforcing strips across two columns of panels (9.84') in layer i, and N is the total number of layers of reinforcing strips in the section of structure under investigation (figure 8b).

Therefore, the maximum tensile force in a reinforcing strip during the dynamic event becomes:

$$T_{dm} = T_m + \Delta \, T_d$$

4.3 Comparison of Calculated Dynamic Increment of Tensile Loads With F.E.M. Results

Figures 8c and 8d present a comparison of the maximum dynamic increment of tensile loads calculated by the above procedure with those determined in the dynamic finite element study. The 19.7 foot high wall (Figure 8c) and the 34.5 foot high wall (figure 8d) consist of vertical walls founded on rock subjected to the 1957 Golden Gate Accelerogram. Three peak rock accelerations, 0.1g, 0.2g and 0.4g were examined.

The Reinforced Earth Backfill material was assigned a unit weight of 125 pcf, a shear strength of 36 degrees and no cohesion. The random backfill material being retained by the Reinforced Earth structure was assigned a unit weight of 125 pcf, a shear strength of 30 degrees and no cohesion.

The facing panels consisted of 7 inch thick, discrete facing panels, 4.92 feet in height, with a unit weight and strength representative of reinforced concrete.

The maximum dynamic increment of tensile loads, as determined utilizing the following equation, is conservative with respect to the F.E.M. results:

$$P_i = 0.15 \, \gamma \, H^2 \, A_m$$

The calculation procedure, which takes into account only the inertia of the soil within the actual active zone is compared to finite element results which include the inertia of the facing panels. Therefore, based on this conservatism, there is no need to

include the facing panel weight in the calculations.
Also note in figures 8c and 8d that the level of conservatism of the calculated dynamic increment with respect to the F.E.M. results increases with increasing peak foundation acceleration. In other words, increased conservatism will be provided in structures located in more seismically active areas of the country, having higher acceleration coefficients.

4.4 Tension at the Reinforcing Strip Connection to the Facing

The magnitude of tension at the reinforcing strip connection to the facing is a function of the maximum reinforcement tension at the potential failure surface and the facing type.

We know from previous studies that if the facing consists of flexible steel elements, or wire, for example, the static tension at the connection, T_o., is equal to 75 percent of the maximum reinforcement tension, Tm, over the full height of wall.

When discrete concrete facing panels, approximately 5 foot by 5 foot in dimension are used, the ratio of T_o/T_m is 85 percent from the top of the wall to a depth of 60 percent of the wall height and then increases linearly to 100% at the toe of wall.

When full height facing panels are used, the static tension at the connection is equal to the maximum tension over the full wall height.

We have learned from dynamic finite element studies that the dynamic increment of tensile force is also less at the connection in comparison to the maximum dynamic increment, ΔTd.

Therefore, at the facing, if the static tensile force at the connection is T_o and the maximum tensile force is T_m, we can calculate the total force at the connection including the superimposed dynamic load, Δ_d, as follows:

$$T_{do} = \frac{T_o}{T_m} (T_m + \Delta T_d)$$

Since the connection of the reinforcement to the facing is specifically designed to be stronger than the gross section of the reinforcement (with allowance for sacrificial metal thickness), it will <u>NOT</u> control the number of reinforcements needed in the wall. The maximum reinforcement tension occurring at the line of maximum tension (or potential failure surface) will be compared with the allowable reinforcement tension for the static and dynamic condition.

Therefore $T_m + \Delta T_d$ must be less than or equal to 73 percent of the yield stress of the steel times the reduced cross sectional area of the reinforcement (section 2.5).

$$T_m + \Delta T_d \leq 0.73\ F_y\ X\ A_{rs}$$

4.5 Reinforcing Strip Pull-out Resistance During Earthquakes

A series of pullout tests were performed on a full scale test wall subjected to vibrations. The vibrations were induced by vibratory compaction equipment placed in a cradle at the top of wall.

Several pullout tests were performed in the presence of vertical vibrations more severe than an earthquake would impose. Vertical accelerations ranged from 0.2g to 1.2g during the pullout tests. The test results show a maximum 20 percent reduction in the pullout resistance, R, of the reinforcing strips for vertical accelerations that may be considered typical for earthquake events. This reduced pullout resistance is not due to a reduction in the friction coefficient between the reinforcing strips and soil, but, is due to reduced vertical stress (overburden) on the strips caused by the vertical accelerations.

Therefore, for convenience in the analysis of Reinforced Earth structures considering earthquake effects, a 20 percent reduction of the calculated static pullout resistance of the reinforcing strips will be used for the dynamic pullout resistance to conservatively take into account any reduced vertical stress on the strips due to vertical accelerations inherent in earthquake events.

$$R_{seismic} = 0.8\ R_{static}$$

As we have already seen, the width of the active zone is not dependent on A_m. Therefore, for each reinforcing strip level, adherence is checked over the usual length as in the static condition. The calculated factor of safety with respect to bond is compared with the allowable safety factor for the seismic condition (Section 2.5).

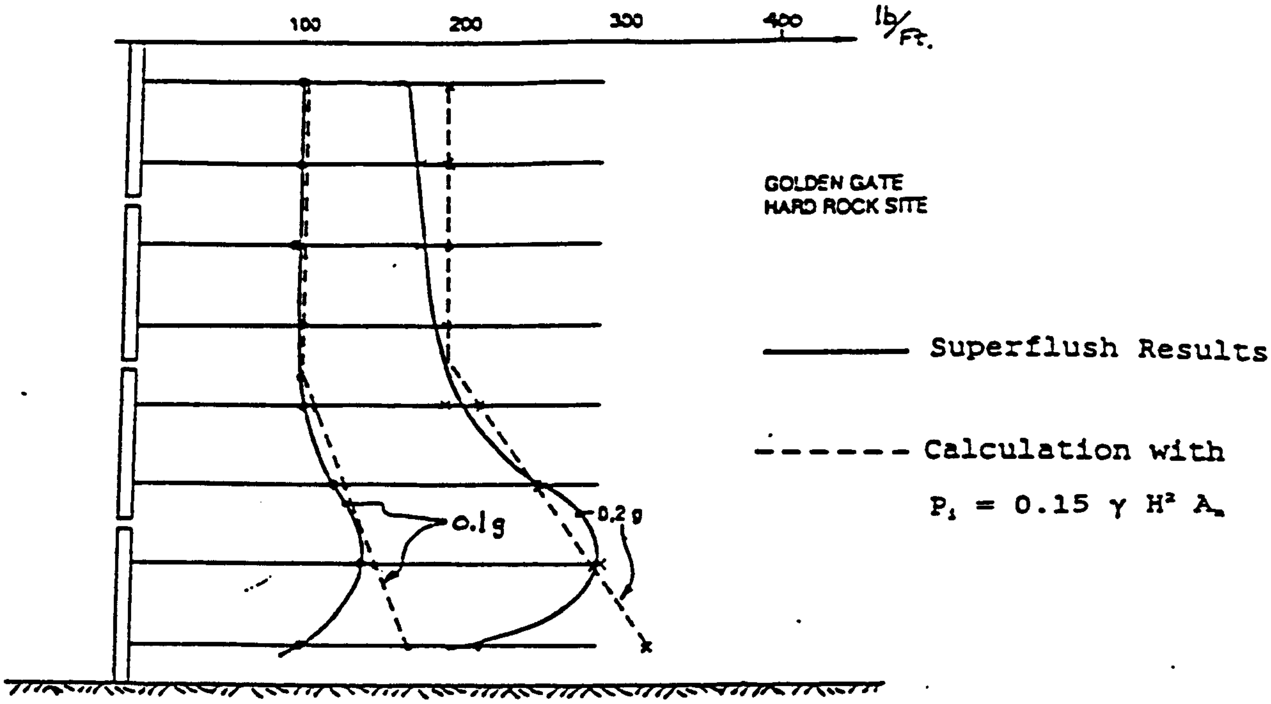


Figure 8c: Maximum dynamic increment of tensile loads 19.7 ft. wall (Superflush)

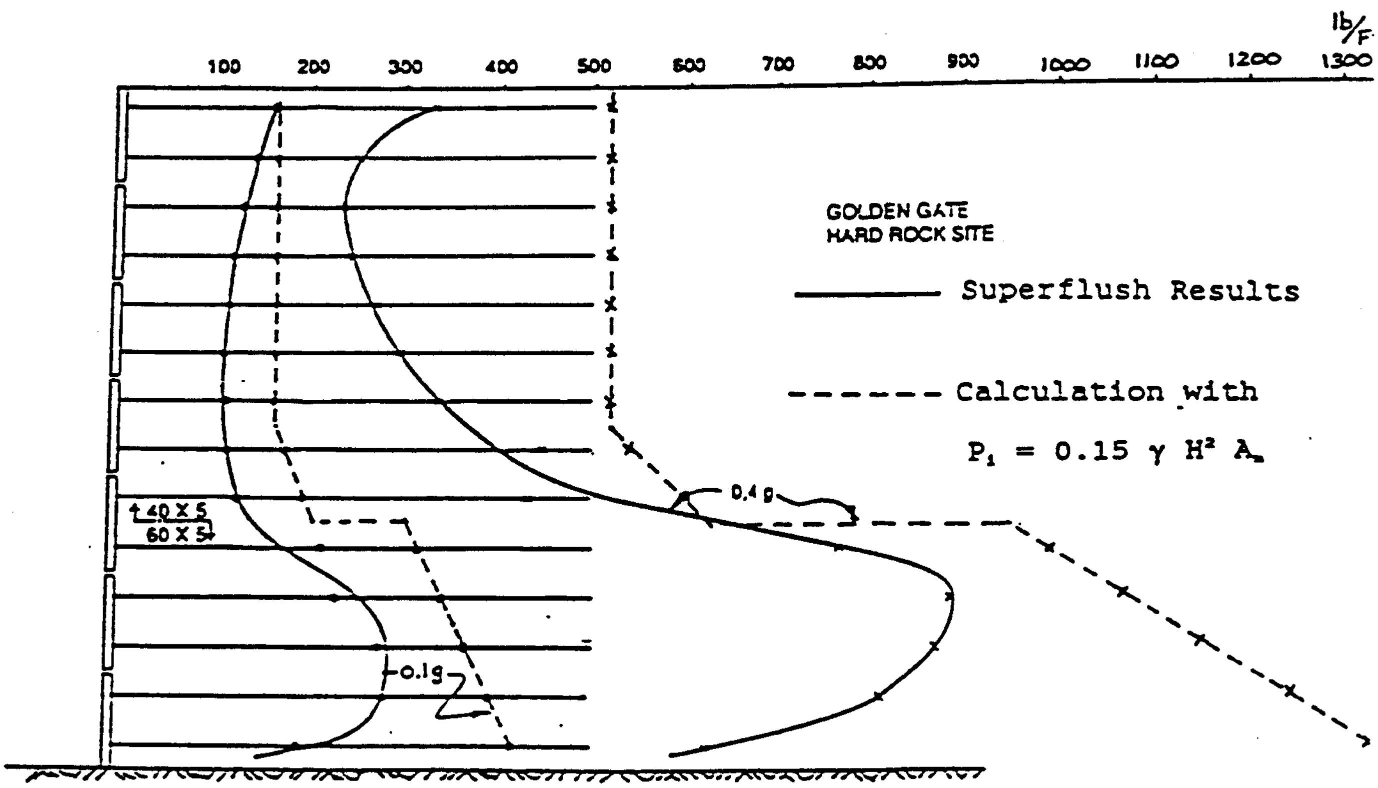


Figure 8d: Maximum dynamic increment of tensile loads 34.5 ft. wall (Superflush)

Quality Control Plan for Reinforced Earth Company
Design of MSE Walls for Sea-Tac Airport

Submitted as part of the 30% Design, August 31, 2000

Design of the Reinforced Earth walls for Sea-Tac Airport includes extensive quality control measures applied throughout the process. As stated in the Quality Assurance Plan, the design will be based on national and Reinforced Earth Company (RECo) standard references and will be performed by experienced engineers supported by US and international experts in both Reinforced Earth technology and high wall design. This Quality Control Plan discusses the work in terms of its seven phases: Kick-off Meeting, 30% Design, 30% Peer Review, 60% Design, 90% Design, 90% Peer Review and the 100% Submission.

Kick-Off Meeting (Completed)

The Kick-off Meeting had three parts – a face-to-face introductory meeting in Seattle, WA, at HNTB's office (May 20, 2000), a technical teleconference involving HNTB and Hart Crowser (geotechnical consultant) in Seattle and The Reinforced Earth Company in Vienna, VA (July 26, 2000) and a face-to-face technical meeting among RECo team members and other RECo expert staff on July 28, 2000. Both at the Seattle meeting and in the teleconference, working relationships and lines of communication were established between the principal members of RECo's team and the members of the HNTB/Hart Crowser team. The project requirements were reviewed, including critical milestone dates and the tasks to be performed by those dates. The design process was discussed in detail, including the need to determine values for certain design parameters and the identification of other decisions (such as backfill material type and properties) that had to be made prior to starting detailed design. It was agreed during the July 26 teleconference that future weekly teleconferences would be held throughout the design process.

The results of the Seattle Kick-off Meeting and the teleconference were conveyed to the RECo team on July 28 in Vienna. That meeting was attended by team members Sherif Aziz (State of Washington P.E. Review), Melissa Berkebile (Project Engineer and Designer), Roger Bloomfield (Contract Manager and COO), John Sankey (Project Manager, Project and Geotechnical Review and Soiltech Contact), Kim Truong (High Wall Engineer) and RECo expert staff. Pierre Segrestin, Soiltech International Expert Review, was briefed by telephone.

30% Design (In Progress)

The 30% Design phase includes project start-up and the following three steps:

1. Adaptation of RECo's design procedures and methods to the needs of high wall design. Specifically, the High Wall Engineer prepares modified Excel spreadsheet

30% Design (In Progress - continued)

programs for high wall design, testing repeatedly that the programs accurately evaluate walls up to the heights required at Sea-Tac. Simultaneously, the Project Engineer uses project-specific input parameters to produce hand calculations that document the programmed design method. These calculations will be used by both RECo and outside reviewers in checking the design.

2. The Project Engineer uses information provided by HNTB to prepare preliminary layout ("wall envelope") drawings of the West, North and South Walls. Close coordination is an essential part of the Quality Control process at this stage of design, as new information is being developed by Hart Crowser regarding foundation soil strength (including planned ground improvement), MSE backfill (Zone B_2) properties, and the characteristics of the common embankment material (Zone C_1) to be used behind the reinforced zone. The preliminary drawings are reviewed with HNTB and Hart Crowser to determine if modifications to the original wall envelope are required.

   Based on data provided by HNTB, the RECo Project Engineer develops the initial (optimum) MSE wall design. This design is the starting point for the iterative process of matching the wall with the site soil properties and determining all aspects of wall stability. Specifically, the initial (optimum) design determines an embedment depth and an applied bearing pressure. Embedment is a critical factor affecting bearing capacity, global stability and local ground (seismic) stability, all of which must be checked for conformance to required factors of safety.

   - Bearing Capacity: Since bearing capacity generally increases with depth, the embedment required by these very high walls would typically be beneficial. At this site, however, soil strength variations with depth, plus the presence of groundwater above the (preliminary design) foundation elevation, tend to reduce bearing capacity. Ground improvement may be required to achieve the necessary bearing capacity at this site.

   - Global Stability: Global stability is the mass stability of the entire embankment and foundation external to the MSE structure (including common embankment behind and in front of the wall and natural/improved soil beneath the wall). The deeper the embedment, the greater the confinement due to the soil in front of the wall. Deeper embedment also lengthens the critical sliding surface that passes beyond the reinforced cross section (it is generally accepted that, in the global stability analysis of an MSE wall, the critical slip circle is forced outside the reinforced cross section by the presence of the steel reinforcements).

30% Design (In Progress - continued)

- Local Ground (Seismic) Stability: Geotechnical reports prepared by Hart Crowser indicate that certain sand substrata may be prone to liquefaction and require either overexcavation and replacement or improvement in place. The depth of wall embedment and/or confinement imposed will affect the extent of liquefaction mitigation.

3. Review and submission of preliminary design. The Project Engineer prepares computer-generated calculations to document the preliminary design, backed up by the hand calculations discussed above. The wall layout as presented shows top and bottom elevations, the layout of panels, and the densities and lengths of the earth reinforcements (panel types will be designated later as part of the 60% Design Phase). Standard details, preliminary coping and barrier details, typical sections, and general notes are included.

   Prior to submission, the preliminary design receives the following reviews:

   - Tall Wall Review: Check that the wall design is consistent with the tall wall design method, that the offsets and embedments of wall tiers are satisfactory, and that calculated bearing pressures are consistent with the stated bearing capacities. Provide sample tall wall calculations.
   - Project Manager Review: Perform an overall review of the engineering work to date.
   - Architectural/Appearance Review (separate submittal). Verify that the work to date is consistent with the overall appearance plan for this project and that the proposed facing panel architectural details are both economical and consistent with MSE wall manufacturing processes.
   - International Review. Inform and consult with tall wall experts as needed.

30% Peer Review

The 30% Peer Review phase consists primarily of HNTB and Hart Crowser reviewing RECo's submission, followed by RECo's response to questions and a discussion of the needed revisions (processing those revisions will be part of the 60% Design phase). It is expected that changes will be needed to the wall embedment, the ground improvement plans, or to both; therefore, embedment, bearing pressure, bearing capacity, global, and seismic stability must be discussed among all parties.

60% Design

The 60% Design phase will begin by addressing the issues raised during the 30% Peer Review and making the required revisions. These will include revisions to the preliminary design to accommodate embedment changes (resulting from the ongoing geotechnical evaluation by Hart Crowser) and adding notes and details associated with any planned ground improvement. The wall layouts (including elevation views of panels and lengths and densities of earth reinforcements) will also be revised as required by changes to, or by new information from, the site plan. Information will be added to the drawings as necessary upon direction by HNTB and/or Hart Crowser.

Additional RECo activities will include detailed design of the individual wall facing panels, detailed design of copings and barriers and revising of the general notes on the plans. Calculations will be prepared as needed to support the ongoing design work (complete calculations will be part of the 90% submission) and the specifications covering the wall materials, panel finishes and the construction process will be written.

Prior to submission, the 60% design receives the following reviews:

- Geotechnical Review: Check bearing pressures against revised bearing capacities resulting from Hart Crowser's analyses and the planned ground improvement. Recheck external, global and seismic stabilities. Review loading conditions, grading, drainage, groundwater and other factors that could affect wall stability and long-term performance.
- Tall Wall Review: Thoroughly check in-house working version of design calculations against actual wall drawings. Check offsets and embedments of wall tiers. Check reasonableness of bearing pressures resulting from high wall design method. Verify that any remaining foundation questions are addressed in the Geotechnical Review.
- Project Manager and P.E. Review: Overall review of engineering work to date.
- Construction Review: Check plans to verify constructibility of walls. Verify presence of sufficient details for contractor to perform the work properly. Check that notes are consistent with details on drawings and that they provide proper instruction and guidance to contractor.
- Specification Review: Review draft specifications for consistency with wall design and with any special site conditions or requirements. Check conformance of specifications to all national and RECo standard specifications and references as stated in the Quality Assurance Plan.

60% Design (continued)

- Architectural/Appearance Review: Verify that wall appearance information on drawings is consistent with the overall appearance plan for this project.
- International Review: Incorporate results of ongoing participation and review by international tall wall experts.

90% Design

Since all major engineering decisions are expected to be made before completion of the 60% Design, the 90% Design phase will be primarily one of finalizing, checking and submitting.

- All comments on the 60% Design will be incorporated and final plans and specifications will be submitted, including final versions of all notes and details. An agreement must be reached between RECo and HNTB which permits RECo's corporate design responsibility to be included as part of the professional engineer's signing and sealing of the drawings.
- Detailed calculations will be prepared in final submission format to document the complete wall design.
- The construction quality control manual for manufacture and installation of Reinforced Earth walls (no MSE-generic version exists) will be submitted.
- If necessary, further recommendations will be made regarding ground improvement beneath the MSE walls.
- Recommendations regarding instrumentation and monitoring of walls to confirm performance criteria will be made. Details may be added to the MSE plans as necessary to address the requirements of instrumentation and monitoring.
- Consultation with Hart Crowser on compound stability analysis.
- Consultation with Hart Crowser on MSE wall material properties to be used in deformation analysis (FLAC input parameters).

Final review and checking at the 90% phase mirrors that at the 60% phase, namely:

- Geotechnical,
- Tall Wall and International,
- State of Washington P.E. (subject to responsibility agreement discussed above),

90% Design (continued)

- Construction,
- Specifications, and
- Architectural.

90% Peer Review

This review by HNTB and Hart Crowser is a final check of all designs, drawings, calculations, specifications and supporting materials. Frequent interaction with RECo personnel will resolve any problems or conflicts uncovered by the reviewers.

100% Submission
The final project submission will include reproducible plans and specifications bearing a Washington P.E. stamp and final copies of all other supporting documents which have been changed since the 90% submission.

Quality Assurance Plan for Reinforced Earth Company Design of MSE Walls for Sea-Tac Airport

Submitted as part of the 30% Design, August 31, 2000

Quality of design for the Sea-Tac Airport MSE walls will be assured by following the requirements of

- National and Reinforced Earth Company (RECo) standard references,
- HNTB drawings, and
- Hart Crowser memoranda and reports,

except where those requirements are modified by the HNTB-led design team to meet specific project conditions. RECo's design will be produced by qualified engineers whose experience is appropriate to the needs of the project, supported by expert assistance and review provided by Soiltech*, RECo's international center for technology expertise, research and development.

National Standard References

- 1996 AASHTO Standard Specifications for Highway Bridges, Section 5.8 and other cross-referenced sections.
- 1997 and 1998 Interim Revisions to AASHTO Standard Specifications where applicable and appropriate to the needs of the project.

RECo Standard References

- Reinforced Earth Company Design Manual, July 2000.
- Reinforced Earth Company Technical Bulletins.
  - ♦ MSE-1, Service Life, Allowable Reinforcement Stress and Metal Loss Rates to be Used in the Design of Permanent MSE Structures, February 1995.
  - ♦ MSE-6, Apparent Coefficient of Friction, f*, to be Used in the Design of Reinforced Earth Structures, October 1995.
  - ♦ MSE-7, Minimum Embedment Requirements for MSE Structures, October 1995.
  - ♦ MSE-9, AASHTO Design Method for Reinforced Earth Structures Subject to Seismic Forces, January 1995.
- Reinforced Earth Company Technical Memos as required.
- Terre Armee Internationale (TAI*) Technical Reports as required.

- Reinforced Earth Company spreadsheets and hand calculations that document computer program calculations.

*Soiltech is the new name for the center of expertise formerly known as TAI.

HNTB Drawings and Hart Crowser Memoranda and Reports

- 30% Contract Drawings from HNTB for all three walls, undated.
- Hart Crowser Report to Port of Seattle and HNTB, "Subsurface Conditions Data Report, North Safety Area, Third Runway Embankment, Sea-Tac International Airport," March 20, 2000.
- Hart Crowser Report to Port of Seattle and HNTB, "Subsurface Conditions Data Report, South MSE Wall and Embankment, Third Runway Project, Sea-Tac International Airport," April 7, 2000.
- Hart Crowser Report to HNTB, "Preliminary Stability and Settlement Analyses, Subgrade Improvements, MSE Wall Support, Third Runway Project," June 2000.
- Hart Crowser Report to Port of Seattle and HNTB, "Subsurface Conditions Data Report, West MSE Wall, Third Runway Embankment, Sea-Tac International Airport," June 2000.
- Hart Crowser Memorandum to Jim Thompson, HNTB, "Geotechnical Input to MSE Wall and Reinforced Slope Design, Third Runway Embankment, June 22, 2000, Revised August 21, 2000.

Qualified RECo Engineers

- John Sankey, P.E., Project Manager, Project and Geotechnical Review, and Soiltech Contact
- Melissa Berkebile, Project Engineer and Designer
- Kim Truong, P.E., High Wall Engineer
- Sherif Aziz, P.E., State of Washington P.E. Review
- Paul Frankenberger, P.E., Regional Manager, Architectural/Appearance Review
- Don Grabner, Construction
- Roger Bloomfield, P.E., Contract Manager and COO
- Pierre Segrestin, Soiltech International Expert Review

Barry
FYI

The Reinforced Earth Company

Melissa Berkebile
(703)821-1175 ext. 284
(703)734-5794 (FAX)

8614 Westwood Center Drive Suite 1100 Vienna, Virginia 22182-2233

FAX MESSAGE

TO: Jamie Beaver
DATE: Sept 11, 2000
FROM: Melissa Berkebile
FAX #: 206-328-5581
RE: Corrosion Calculations.
PAGES (includ. cover): 3

Jamie.
Attached are the corrosion calculations per your request. Please call if you have any questions.

Melissa

JOB NUMBER: 8079
JOB NAME: SEATAC AIRPORT
OWNER : CITY OF SEATTLE

DESIGNED BY:

CORROSION OF REINFORCEMENT.

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure = [redacted] yrs

Zinc (first 2 years):	[redacted]	µm/yr
Zinc (subsequent years):	[redacted]	µm/yr
Carbon Steel:	[redacted]	µm/yr
Carbon Steel (75 - 100 yrs)	[redacted]	µm/yr

RESULTS:	
Section A-A (Tie Strip)	10.80 Kips
Section B-B (TS at bolt hole)	12.74 Kips
Section B-B (RS at bolt hole)	13.28 Kips
Section B-B (Bolt)	9.33 Kips, CONTROLS
Section C-C (Reinf. Strip)	11.04 Kips

CO

Mechanical Properties of Reinforcement Hardware

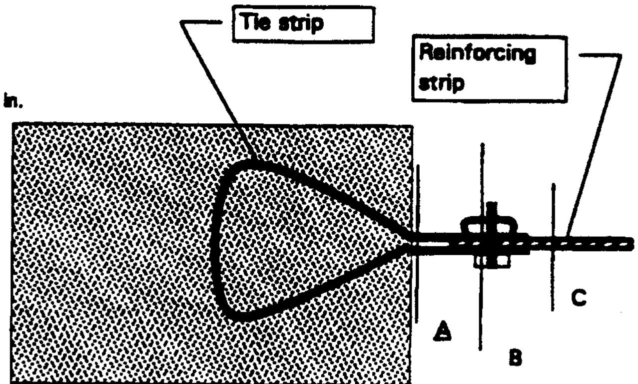
	HA Strip		Tie Strips	
Thickness t =	[redacted]	mm	[redacted]	mm
Width w=	[redacted]	mm	[redacted]	mm
A =	300	mm^2	227.5	mm^2
F_u =	80	Ksi	65	Ksi
F_y =	65	Ksi	50	Ksi

Thickness of Zinc Coat.

Zinc coat t = [redacted] µm
Life of Zinc: 16 yrs

Bolt Set.

Ø = 12.7 mm = 0.50 in.
Fv = 19 Ksi



Structural Connection of ReStrip To Panel

Section A-A (Tie Strip Only)
2 Tie Strip plates
At end of design life, carbon steel loss : 1.008mm/side
Remaining Thickness (1 plate) = 4.550- 2 x 1.008 =2.534mm
As (2 plates) = 2.534x 2 x 50 =253.400mm2 or 0.393in2
Ft = 0.55 Fy = 0.55 x 50.00Ksi =27.50Ksi
Allowable Tension = 10.80Kips

Section B-B (Tie Strip at Bolt Holes)
2 tie strip plates with 14.3 in bolt holes
At end of design life, carbon steel loss : 1.008mm/side
Remaining Thickness (1 plate, corrosion on 1 side) = 4.550 - 1.008 =3.542mm
A (2 plates) = 3.542x 2 x 50 =252.899mm2 or 0.392in2
Ft = 0.50 Fu = 0.50 x 65.00Ksi =32.50Ksi
Allowable Tension = 12.74Kips

Section B-B (Reinforcing Strip at Bolt Holes)
At end of design life, carbon steel loss : 0mm/side
Thickness = 6.00mm
As = 6.00 x (50.000 - 5.630)=214.20mm2 or 0.33 in2
Ft = 0.50 Fu = 0.50 x 80.0Ksi =40.00Ksi
Allowable Tension = 13.28Kips

Section B-B (Shear Strength of Bolt)
At end of design life, carbon steel loss : 0mm/side
Fv = 1.25 x Fy = 1.25 x 19.00Ksi = 23.75Ksi allowable (thread excluded from shear plane, AASHTO 10.32.
Area of Bolt = 126.68mm2
Allowable Shear = 9.33Kips ← Reco is deferring to shear on bolt as control, but for our compound analysis, this would not govern; this would,

Section C-C (Reinforcing Strip)
At end of design life, carbon steel loss : 1.008mm/side
Remaining Thickness = 6.000- 2 x 1.008 =3.984mm
As = 3.984x 50 =199.200 mm2 or 0.309in2
Ft = 0.55 Fy = 0.55 x 65.00Ksi =35.75Ksi
Allowable Tension = 11.04Kips

JOB NUMBER: 8079 **DESIGNED BY:**
JOB NAME: SEATAC AIRPORT
OWNER : CITY OF SEATTLE

CORROSION OF REINFORCEMENT.

Galvanization and Carbon Steel Loss Rates:

Design Life of Structure =	[redacted]	yrs
Zinc (first 2 years):	[redacted]	µm/yr
Zinc (subsequent years):	[redacted]	µm/yr
Carbon Steel:	[redacted]	µm/yr
Carbon Steel (75 - 100 yrs)	[redacted]	µm/yr

RESULTS:	
Section A-A (Tie Strip)	6.03 Kips
Section B-B (TS at bolt hole)	8.71 Kips
Section B-B (RS at bolt hole)	8.85 Kips
Section B-B (Bolt)	9.33 Kips
Section C-C (Reinf. Strip)	5.50 Kips, CONTROLS

Mechanical Properties of Reinforcement Hardware

	HA Strip		Tie Strips	
Thickness t =	[redacted]	mm	[redacted]	mm
Width w=	[redacted]	mm.	[redacted]	mm
A =	200	mm^2	171.5	mm^2
F_u =	80	Ksi	65	Ksi
F_y =	65	Ksi	50	Ksi

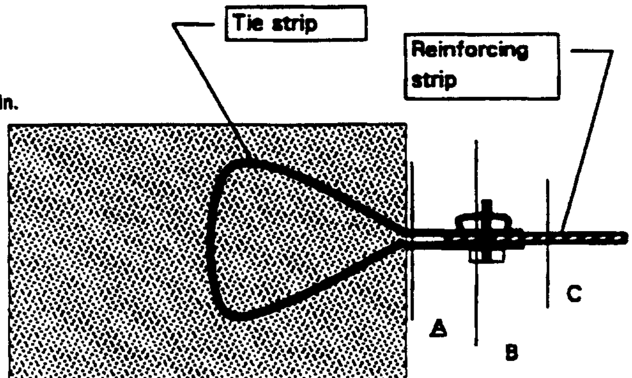
— smaller tie strips than preceding example

Thickness of Zinc Coat

Zinc coat t = [redacted] µm
Life of Zinc: 16 yrs

Bolt Set.

Ø =	12.7	mm	= 0.50 in.
Fv =	19	Ksi	



Structural Connection of ReStrip To Panel

Section A-A (Tie Strip Only)
2 Tie Strip plates
At end of design life, carbon steel loss : 1.008mm/side
Remaining Thickness (1 plate) = 3.430- 2 x 1.008 =1.414mm
As (2 plates) = 1.414x 2 x 50 =141.400mm2 or 0.219in2
Ft = 0.55 Fy = 0.55 x 50.00Ksi =27.50Ksi
Allowable Tension = 6.03Kips

Section B-B (Tie Strip at Bolt Holes)
2 tie strip plates with 14.3 in bolt holes
At end of design life, carbon steel loss : 1.008mm/side
Remaining Thickness (1 plate, corrosion on 1 side) = 3.430 - 1.008 =2.422mm
A (2 plates) = 2.422x 2 x 50 =172.931mm2 or 0.268in2
Ft = 0.50 Fu = 0.50 x 65.00Ksi =32.50Ksi
Allowable Tension = 8.71Kips

Section B-B (Reinforcing Strip at Bolt Holes)
At end of design life, carbon steel loss : 0mm/side
Thickness = 4.00mm
As = 4.00 x (50.000 - 5.630)=142.80mm2 or 0.22 in2
Ft = 0.50 Fu = 0.50 x 80.0Ksi =40.00Ksi
Allowable Tension = 8.85Kips

Section B-B (Shear Strength of Bolt)
At end of design life, carbon steel loss : 0mm/side
Fv = 1.25 x Fy = 1.25 x 19.00Ksi = 23.75Ksi allowable (thread excluded from shear plane, AASHTO 10.32.
Area of Bolt = 126.68mm2
Allowable Shear = 9.33Kips

Section C-C (Reinforcing Strip)
At end of design life, carbon steel loss : 1.008mm/side
Remaining Thickness = 4.000- 2 x 1.008 =1.984mm
As = 1.984x 50 =99.200 mm2 or 0.154in2
Ft = 0.55 Fy = 0.55 x 65.00Ksi =35.75Ksi
Allowable Tension = 5.50Kips

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