NOTE

The following independent hand-calculations for an MSE problem using AASHTO/LRFD, including a comparison with MSEW(3.0), was done by Dr. Naresh Samtani for Arizona DOT.

Dr. Naresh Samtani is a principal with NCS Consultants, LLC in Tucson, AZ. For qualifications, please see:

http://www.ncsconsultants.com/personnel/samtani.php

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August 2008

Purpose: Evaluation of maximum tensile force, T_{max}, for internal stability analysis of MSE walls using the procedures in AASHTO LRFD code (2008 Interims) and comparison with results from MSEW V3.0 Software.

References: (1) Sections 3 and 11 of AASHTO 2008 Interims

(2) MSEW V3.0 Update 9.1 Software

(3) Publication No. FHWA NHI-00-43, March 2001 "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes".

Computations: A 30-ft high wall was selected. The geometry of the wall and other relevant information is shown in Figure 1 on page 2. The reinforcement layer #s and depths are also shown on page 2. The depth of the reinforcement layers were chosen to give unsymmetrical tributary heights for the uppermost and lowermost layers. In actual practice, the reinforcement layers may be arranged differently, particularly the lowermost layer.

Job Evaluation of Tmax in MSE walls using LRFD Job # J2008-04
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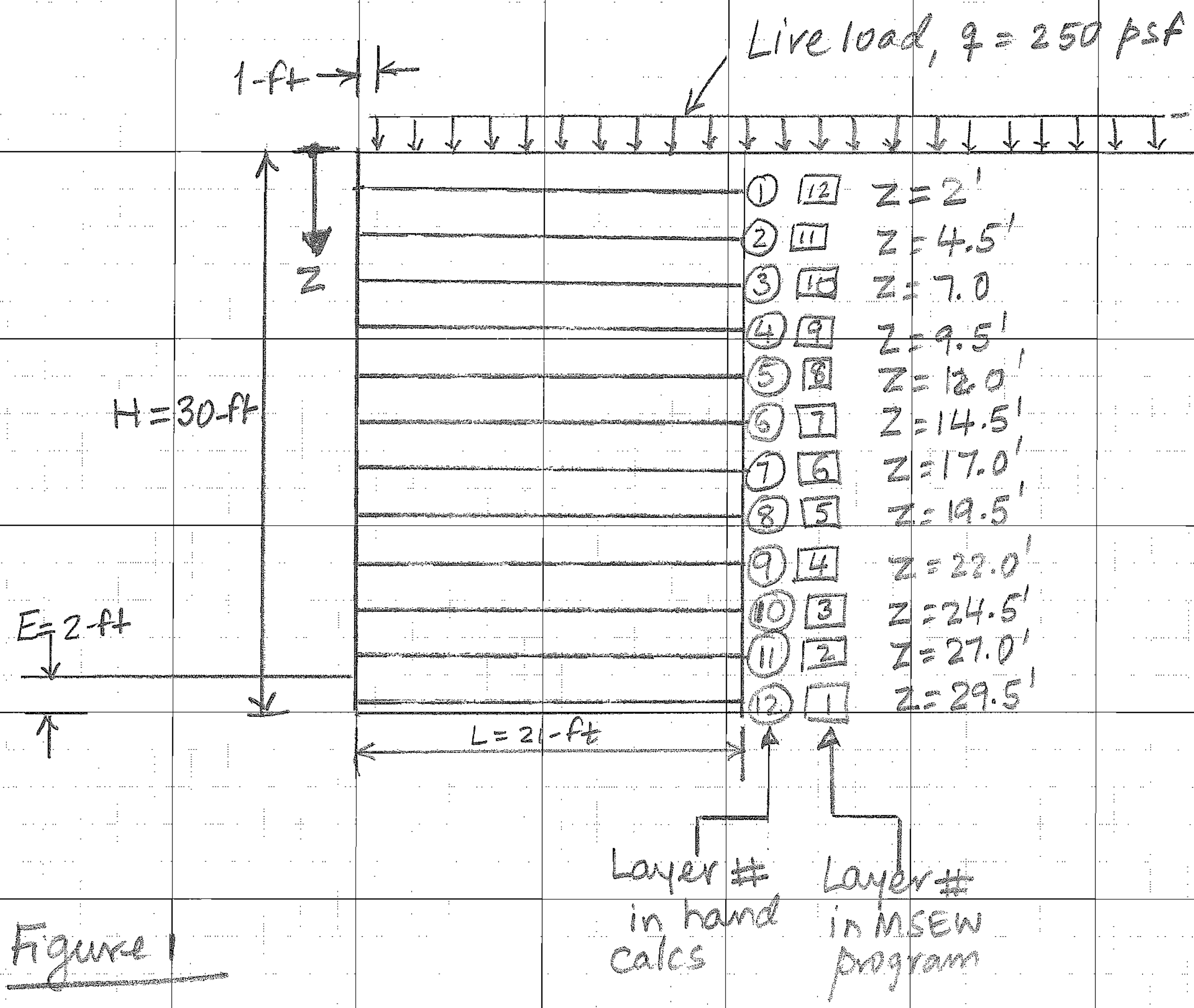


Figure 1

- Unit weight of reinforced mass, $\gamma_r = 125$ pcf
- Coefficient of Uniformity of reinforced soil, $C_u = 4.0$
- Angle of internal friction of reinforced soil, $\phi_r = 34°$
- Assume ribbed metal strips as soil reinforcement
- Disregard impact loads for purpose of these calcs (should be included in actual problems as appropriate)

A spreadsheet was developed to facilitate hand calculations. This spreadsheet is included in Attachment 1. All calculations are clearly doumented in a systematic manner in 45 columns labeled Col 1 to Col 45. The step by step calculations can be followed by systematically progressing from Col. 1 to Col. 45. Cols 1-22 are of particular interest since they deal with the derivation of Tmax and its comparison with values computed from MSEW program. The calculations in the spreadsheet are based on efuations in AASHTO 2008 and are noted in each column.

Two sets of calculations were performed. The only difference between these 2 sets was the value of load factor for live load as follows

<u>Set 1</u>: Use load factor for LL, $\gamma_{LL} = 1.75$

<u>Set 2</u>: Use load factor for LL, $\gamma_{LL} = 1.35$

The above sets of calculations were developed to evaluate alternative interpretations of application of γ_{LL} for internal stability analysis as discussed below.

Attachment 2 includes nine (9) relevant page from Reference (1) i.e. Sections 3 and 11 of AASHTO 2008 Interims. It should be noted that the figures and equations in the 9 pages in Attachment 2 are largely similar to those in Reference (3) i.e. March 2001 NHI manual except as modified for LRFD by application of load factors.

In Attachment 2, handwritten notes are included to trace the logic and interpretation of various figures and equations. In LRFD one has to be very careful about applying proper load factors to given loads based on the nature and application of the load. Furthermore, once a load factor is chosen for a given problem then it should not be changed in that particular analysis. For example, if a load factor of 1.75 is used for live loads then it should be consistently used throughout the analysis and not mixed with the load factor for earth loads which is 1.35. It is herein where alternative interpretations can occur as follows:

From Page 3 of Attachment 2

$\sigma_H = \gamma_p(\sigma_V k_r + \Delta\sigma_H)$

From Page 4 of Attachment 2

$\sigma_V = \gamma_r Z + q + \Delta\sigma_V$ for max stress (e.g. σ_H or Tmax)

$\sigma_V = \gamma_r Z + \Delta\sigma_V$ for pullout resistance, P_r

From Equations on Page 4 it is clear that q represents live load effect. The figures and equations on Page 4 of Attachment 2 are identical to the figures and equations shown on page 2 and 3 of Attachment 3 which is based on the 2002 or Allowable Stress Design (ASD) version of AASHTO. It is clear from this comparison that the equations on Page 4 of Attachment 2 are simply intended to show how various loads need to be combined. The load factor for soil based loads, σ_V, and/or other permanent loads, $\Delta\sigma_V$, is γ_p as per the equation on Page 3 of Attachment 2. Thus, the equation for vertical loads can be written as follows for calculating max stress

$$\left.\begin{aligned}\sigma_V &= \gamma_r Z + q + \Delta\sigma_V\\ \sigma_V &= \underbrace{(\gamma_r Z + \Delta\sigma_V)}_{\text{permanent or dead}} + \underbrace{q}_{\text{transient or live}}\end{aligned}\right\}\text{ unfactored}$$

From Page 3 of Attachment 2

$$\sigma_H = \gamma_p (\sigma_v k_r + \Delta\sigma_H)$$

It is important to note the definition of σ_v below this equation on Page 3 of Attachment 2. It indicates that σ_v is the "resultant of gravity forces from soil self weight .....". For the case of live load surcharges, one has to refer to the figure on Page 2 of Attachment 2 and how the Live Load is positioned to create the "extreme force effect" as noted on Page 5 of Attachment 2. Using the above discussion, the equation for horizontal stress for the case of a wall with LL surcharge can be written as follows.

$$\sigma_H = \gamma_p (\sigma_v k_r + \Delta\sigma_H) + \gamma_{LL}\, q\, k_r$$

$$\sigma_H = \gamma_p \left[(\gamma_r Z + \Delta\sigma_v) k_r + \Delta\sigma_H\right] + \gamma_{LL}\, q\, k_r$$

For $\Delta\sigma_v = 0$ and $\Delta\sigma_H = 0$

$$\boxed{\sigma_H = \gamma_p \left[(\gamma_r Z) k_r\right] + \gamma_{LL}\, q\, k_r} \qquad \text{Eq } 1$$

Alternative Interpretation

One can simply substitute the equation for σ_V from Page 4 of Attachment 2 into the equation for σ_H as follows

$$\sigma_H = \gamma_P (\sigma_V k_r + \Delta\sigma_H)$$

$$\sigma_H = \gamma_P \left[(\gamma_r Z + q + \Delta\sigma_V) K_r + \Delta\sigma_H\right]$$

For $\Delta\sigma_V = 0$ and $\Delta\sigma_H = 0$

$$\sigma_H = \gamma_P \left[(\gamma_r Z + q) K_r\right]$$

or for direct comparison with Eq 1 σ_H can be written as follows:

$$\boxed{\sigma_H = \gamma_P \left[(\gamma_r Z) K_r\right] + \gamma_P q K_r} \qquad \text{Eq } 2$$

In Eq 2, $\gamma_{LL} = \gamma_P$ for the "q" or LL term

To evaluate the effect of alternative interpretations by Eq 1 and Eq 2, the 2 sets of calculations in Attachment 1 were performed. Set 1 corresponds to Eq 1 and Set 2 corresponds to Eq 2.

Each of the two sets contains the following:

a) A printout of the spreadsheet calculations

b) A printout of the output file from MSEW program

Once the σ_H profile is developed with depth, one needs to calculate Tmax which is done as follows

$T_{max} = (\sigma_H)(\text{Tributary height}, S_v)$

The area of the σ_H diagram within the tributary height is in essence the value of Tmax for a given layer. Figure 2a and 2b show 2 different ways of approximating the σ_H profile. The "trapezoid" approach in Figure 2a approximates the σ_H profile better than the "rectangle" approach shown in Figure 2b. MSEW program appears to use the trapezoid approach since the calculations in the spreadsheets in Attachment 1 appear to match MSEW results much better when the "trapezoid" approach is used, see Cols 16-22 of the spreadsheets

Trapezoid Tributary area

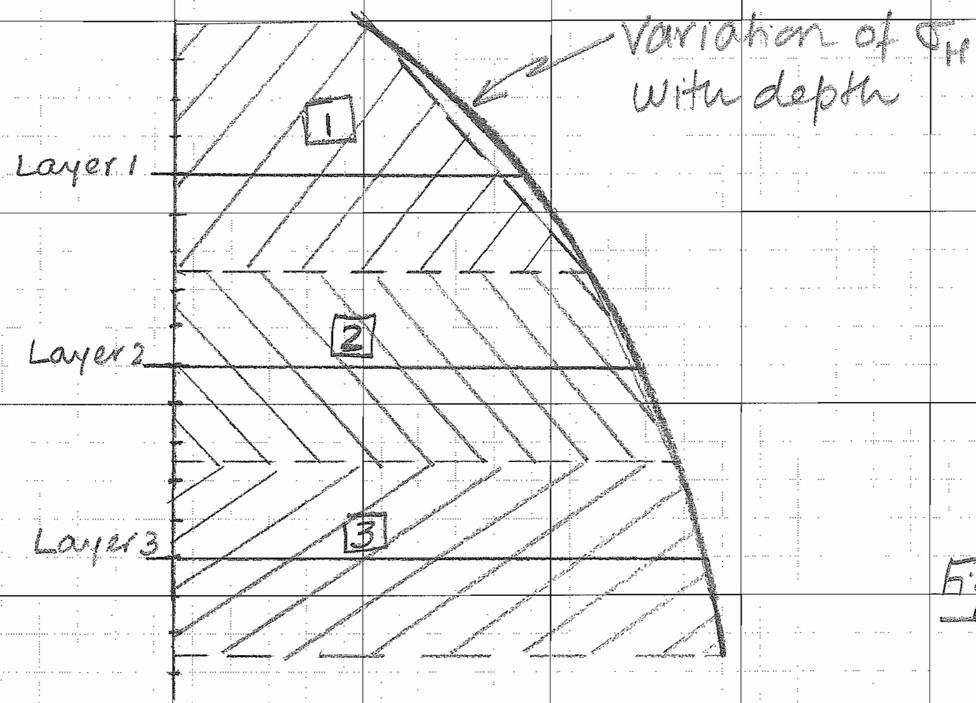


Figure 2a

Rectangular Tributary Area

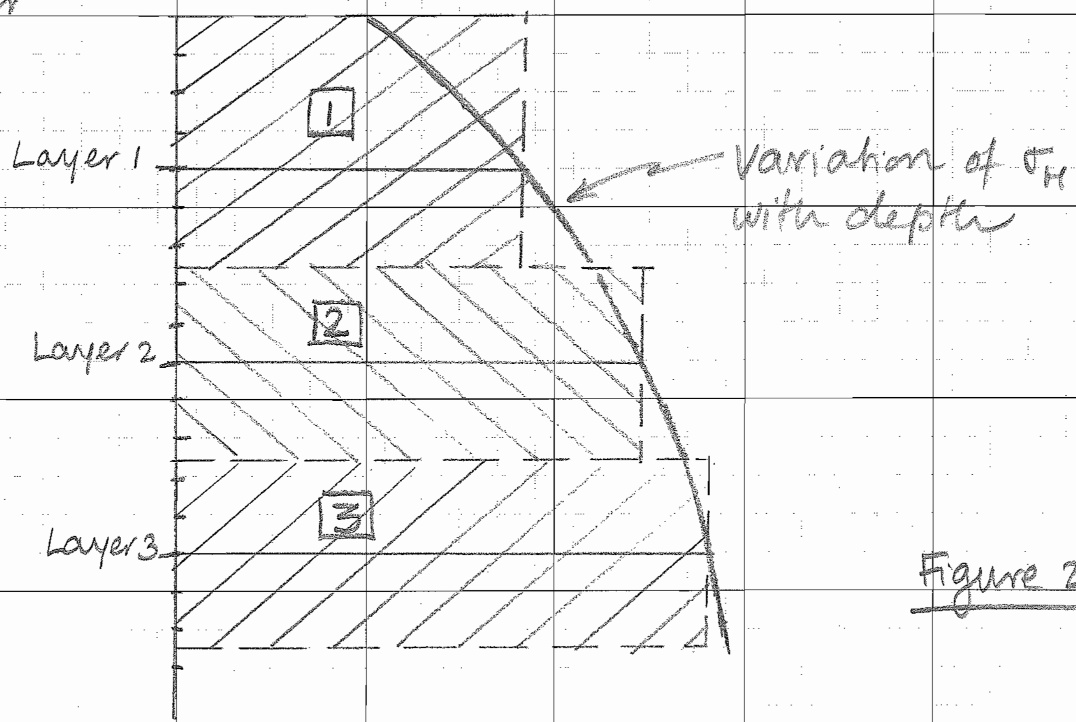
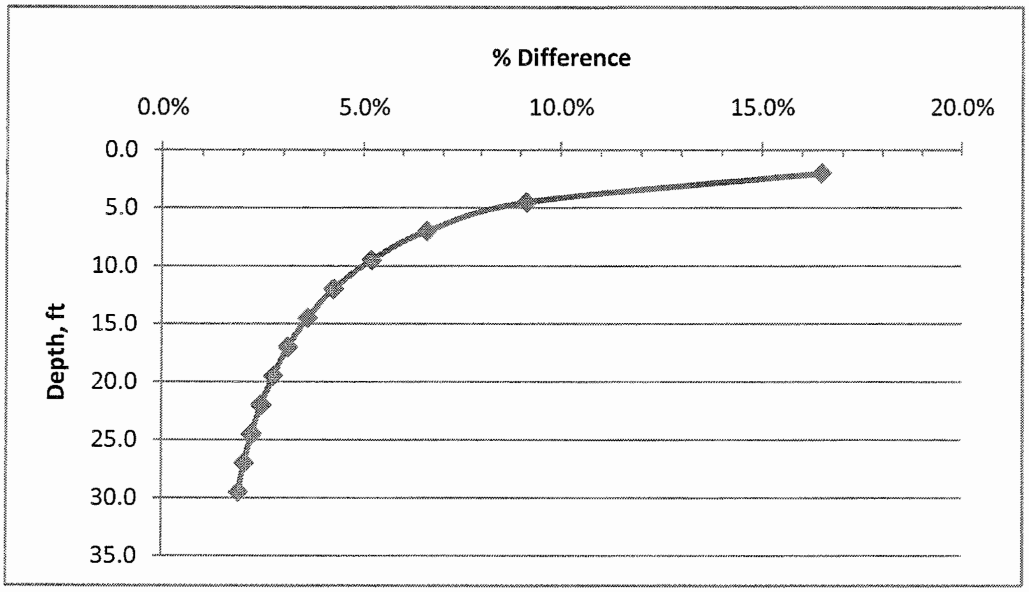


Figure 2b

The Tmax values from Col 20 of the spreadsheets for sets 1 and 2 and the % difference between the two sets are tabulated and graphed below

Table 1

Col 1	Col 2	Col 3	Col 4	Col 5
		Tmax, kips/ft		
		Set 1	Set 2	% Diff
Layer #	Z, ft	γ_{LL}=1.75	γ_{LL}=1.35	
1	2.0	1.066	0.915	16.5%
2	4.5	1.335	1.223	9.2%
3	7.0	1.746	1.638	6.6%
4	9.5	2.101	1.997	5.2%
5	12.0	2.405	2.307	4.2%
6	14.5	2.706	2.612	3.6%
7	17.0	2.978	2.888	3.1%
8	19.5	3.216	3.130	2.7%
9	22.0	3.519	3.435	2.4%
10	24.5	3.877	3.793	2.2%
11	27.0	4.235	4.151	2.0%
12	29.5	3.177	3.118	1.9%



From Table 1, it can be noted that the difference between the two alternative interpretations of application of γ_{LL} is approximately 16.5% near the top of the wall for the particular wall configuration being evaluated herein. This difference is not unique and would vary based on the configuration of the wall and superimposed loads. Nevertheless, the difference can be significant when live loads are present. Therefore, the owner agency should carefully evaluate and select the interpretation and application of γ_{LL} for internal stability evaluation and apply the chosen interpretation consistently for all MSE walls designs and all vendors.

Conclusions and Recommendations

From the above discussions and calculations, it appears that MSEW program has implemented Eq ① and not Eq ②. Furthermore, MSEW appears to use the "trapezoid" tributary area approach to calculate T_{max}. Both of these approaches appear to be valid interpretations and are therefore recommended for LRFD implementation by the Arizona Department of Transportation (ADOT).

NCS

Job Evaluation of Tmax in MSE Walls Using LRFD Job # J2008-04

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Checked by Date Sheet of

ATTACHMENT 1

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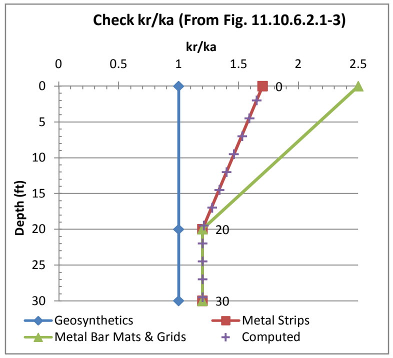
Comparison of Hand Calculations with MSEW Version 3.0, Update 9.1
(Calculations Based on LRFD AASHTO 2008 Interims)

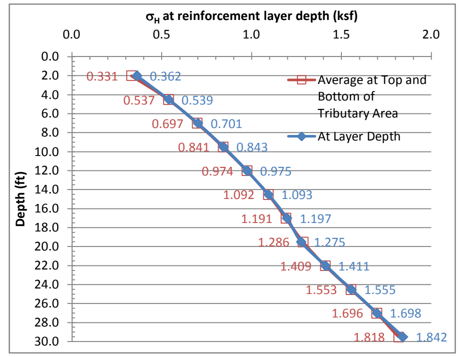
SET 1

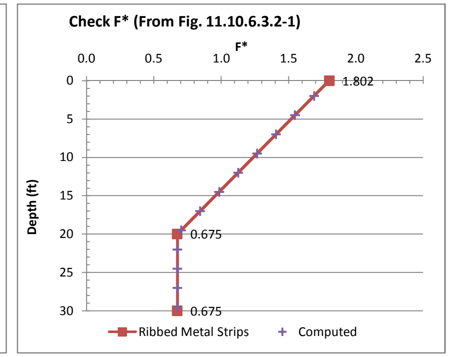
Made by:	N. Samtani
Date:	6/23/2008
Checked by:	WUF/RMP
Date:	6/24/2008
Basic Wall Parameters	
H (ft) =	30
θ (deg) =	90
δ (deg) =	0
β (deg) =	0
L (ft) =	21
$\gamma_f=\gamma_r$ (kcf) =	0.125
ϕ_f (deg) =	34
C_u (dim) =	4.0
Live Load Surcharge	
H_{eq} (ft) =	2.0
σ_{vs} (ksf) =	0.250

Load Factors	
γ_p =	1.35
γ_{LL} =	1.75
γ_{EQ} =	1.00
γ (impact)=	1.00
Resistance Factors	
Pull ϕ_p =	0.9
Str ϕ_s =	0.75
Metal Strip Information	
b (mm) =	50
b (in) =	1.969
A_c (in^2) =	0.200
F_y (ksi) =	65
T_{al} =	13 k/strip
α (dim) =	1.0
C (dim) =	2

Other Info	
# Layers =	12
Panel width	5.0 ft
Impact Load	0.0 k/ft
MSEW file name	
NCS-M Prob 1 30-ft.ben	
Calculation of k_a (dim)	
Γ =	2.431
k_a (dim) =	0.283







Col 1	Col 2	Col 2	Col 4	Col 5	Col 6	Col 7	Col 8	Col 9	Col 10	Col 11	Col 12	Col 13	Col 14	Col 15	Col 16	Col 17	Col 18	Col 19	Col 20	Col 21	Col 22
Input layer geometry		Calculate k_r		Calculate Vertical Stresses		Calculate Horizontal Stress at Reinforcement Layer Depth			Calculate tributary height parameters (e.g., top and bottom depths)			Calculate Horizontal Stress at top and bottom of tributary heights			Calculate Tmax		Data from MSEW Program Output			Comparison of Tmax with MSEW output	
Layer #	Depth from top of wall (or coping as appropriate)	Stress Ratio	k_r	Vert Stress due to soil load (i.e. w/o LL), σ_{vE}	Vert Stress w/ LL (i.e., $\Delta\sigma_{vs}$) for Tmax, σ_{vMax}	Horiz Stress due to Soil Load, σ_{HE}	σ_{HLL} (ksf)	Total Horiz Stress (w/ LL), σ_H	Trib. Height, S_v (ft)	Depth to top of Trib height	Depth to bottom of Trib height	σ_H at Z_{top}	σ_H at Z_{bot}	Average σ_H	Using Trapezoid tributary area	Using Rectangle triburary area	MSEW Layer #	Layer "Height" in MSEW program	Tmax from MSEW Program	% Tmax Diff vs. Trapezoid based σ_H (%)	% Tmax Diff vs. Rectangle based σ_H (%)
-	Z	k_r / k_a	$(k_r/k_a)(k_a)$	$(Z)(\gamma_r)$	$(\sigma_{vE}+\Delta\sigma_{vs})$	$(k_r)(\sigma_{vE})(\gamma_p)$	$(k_r)(\sigma_{vs})(\gamma_{LL})$	$(\sigma_{HE}+\sigma_{HLL})$	S_v	Z_{top}	Z_{bot}	σ_{Htop}	σ_{Hbot}	σ_{Havg}	Tmax_t	Tmax_r	-	-	Tmax_m	-	-
(dim)	(ft)	(dim)	(dim)	(ksf)	(ksf)	(ksf)	(ksf)	(ksf)	(ft)	(ft)	(ft)	(ksf)	(ksf)	(ksf)	(kips/ft)	(kips/ft)	(dim)	(ft)	(kips/ft)	(%)	(%)
1	2.0	1.650	0.467	0.250	0.500	0.158	0.204	0.362	3.250	0.00	3.25	0.210	0.452	0.331	1.077	1.176	12	28	1.066	1.0%	10.3%
2	4.5	1.590	0.450	0.563	0.813	0.342	0.197	0.539	2.500	3.25	5.75	0.452	0.622	0.537	1.342	1.347	11	25.5	1.335	0.5%	0.9%
3	7.0	1.530	0.433	0.875	1.125	0.511	0.189	0.701	2.500	5.75	8.25	0.622	0.772	0.697	1.741	1.752	10	23	1.746	-0.3%	0.3%
4	9.5	1.460	0.413	1.188	1.438	0.662	0.181	0.843	2.500	8.25	10.75	0.772	0.911	0.841	2.103	2.108	9	20.5	2.101	0.1%	0.3%
5	12.0	1.400	0.396	1.500	1.750	0.802	0.173	0.975	2.500	10.75	13.25	0.911	1.037	0.974	2.435	2.438	8	18	2.405	1.2%	1.4%
6	14.5	1.340	0.379	1.813	2.063	0.928	0.166	1.093	2.500	13.25	15.75	1.037	1.148	1.092	2.730	2.734	7	15.5	2.706	0.9%	1.0%
7	17.0	1.280	0.362	2.125	2.375	1.038	0.158	1.197	2.500	15.75	18.25	1.148	1.234	1.191	2.977	2.992	6	13	2.978	0.0%	0.5%
8	19.5	1.210	0.342	2.438	2.688	1.126	0.150	1.275	2.500	18.25	20.75	1.234	1.338	1.286	3.215	3.188	5	10.5	3.216	0.0%	-0.9%
9	22.0	1.200	0.340	2.750	3.000	1.262	0.149	1.411	2.500	20.75	23.25	1.338	1.481	1.409	3.523	3.528	4	8	3.519	0.1%	0.3%
10	24.5	1.200	0.340	3.063	3.313	1.406	0.149	1.555	2.500	23.25	25.75	1.481	1.624	1.553	3.882	3.887	3	5.5	3.877	0.1%	0.3%
11	27.0	1.200	0.340	3.375	3.625	1.549	0.149	1.698	2.500	25.75	28.25	1.624	1.768	1.696	4.240	4.245	2	3	4.235	0.1%	0.2%
12	29.5	1.200	0.340	3.688	3.938	1.693	0.149	1.842	1.750	28.25	30.00	1.768	1.868	1.818	3.181	3.223	1	0.5	3.177	0.1%	1.4%

	Col 23	Col 24	Col 25	Col 26	Col 27	Col 28	Col 29	Col 30	Col 31	Col 32	Col 33	Col 34	Col 35	Col 36	Col 37	Col 38	Col 39	Col 40	Col 41	Col 42	Col 43	Col 44	Col 45
	Calculate Active and Effective Lengths		Calculate Factored Pullout Resistance			Calculate Number of Strips based on no impact		Calculate Number of Strips based on consideration of impact load				Calculate Number of Strips based on consideration of tensile strength			Calculate Governing Number of Strips, horizontal spacing and coverage ratio			Calculate Provided Factored Resistances		Calculate Capacity Demand Ratios (CDRs)		CDRs from MSEW Output	
Layer #	Active Length	Effective Length	Vert Stress for Pullout Resistance	Pullout Resistance Factor	Factored Pullout Resistance per strip	Tmax for a 5-ft wide panel	N Strips - Calc per panel for pullout w/o impact load	Tmax w/ Impact	Factored Pullout Resistance	N Strips - Calc (Impact)	Governing N Strips - Pullout	Tmax for panel width including impact	N Strips - Calc based on Tensile Strength	N Strips - Actual Based on Tensile Strength	Governing Final N (Strength & Pullout)	Required Horiz rein spacing using Final N	Required Rein coverage ratio using	Provided Factored Pullout Resistance Per Panel	Provided Factored Tensile Resistance Per Panel	Capacity Demand Ratio for Pullout	Capacity Demand Ratio for Tensile Strength	CDR for pullout from MSEW	CDR for tensile strength from MSEW
-	La	Le	$\sigma_{vPullout}$ = σ_{vE}	F*	ϕ_pP_r	Tmax_p	Tmax_p / ϕ_pP_r	Tmax_i	$\phi_pP_{r\_i}$	Tmax_i / $\phi_pP_{r\_i}$	-	Tmax_p	Tmax_p / ϕ_sT_{al}	-	-	S_h (ft)	R_c	-	$\phi_pP_{r\_i}$	CDR_p	CDR_t	CDR_pm	CDR_tm
(dim)	(ft)	(ft)	(ksf)	(dim)	(kips)	(kips)	(dim)	(kips/ft)	(kips/ft)	(dim)	(dim)	(kips)	(dim)	(dim)	(dim)	(ft)	(ft)	(kips)	(kips)	(dim)	(dim)	(dim)	(dim)
1	9.00	12.00	0.250	1.689	1.50	5.38	3.59	1.077	0.52	2.07	4	5.38	0.55	1	4	1.25	0.131	6.00	39.00	1.115	7.245	1.119	7.311
2	9.00	12.00	0.563	1.548	3.09	6.71	2.17	1.342	1.08	1.24	3	6.71	0.69	1	3	1.67	0.098	9.27	29.25	1.382	4.359	1.377	4.373
3	9.00	12.00	0.875	1.407	4.36	8.71	2.00	--	--	--	2	8.71	0.89	1	2	2.50	0.066	8.72	19.50	1.002	2.240	0.992	2.233
4	9.00	12.00	1.188	1.266	5.33	10.52	1.97	--	--	--	2	10.52	1.08	2	2	2.50	0.066	10.66	19.50	1.014	1.854	1.004	1.857
5	9.00	12.00	1.500	1.126	5.99	12.17	2.03	--	--	--	3	12.17	1.25	2	3	1.67	0.098	17.97	29.25	1.476	2.403	1.469	2.427
6	9.00	12.00	1.813	0.985	6.33	13.65	2.16	--	--	--	3	13.65	1.40	2	3	1.67	0.098	18.99	29.25	1.391	2.143	1.374	2.157
7	7.80	13.20	2.125	0.844	6.99	14.89	2.13	--	--	--	3	14.89	1.53	2	3	1.67	0.098	20.97	29.25	1.409	1.965	1.371	1.960
8	6.30	14.70	2.438	0.703	7.44	16.07	2.16	--	--	--	3	16.07	1.65	2	3	1.67	0.098	22.32	29.25	1.389	1.820	1.341	1.815
9	4.80	16.20	2.750	0.675	8.88	17.62	1.98	--	--	--	2	17.62	1.81	2	2	2.50	0.066	17.76	19.50	1.008	1.107	1.002	1.108
10	3.30	17.70	3.063	0.675	10.81	19.41	1.80	--	--	--	2	19.41	1.99	2	2	2.50	0.066	21.62	19.50	1.114	1.005	1.106	1.006
11	1.80	19.20	3.375	0.675	12.92	21.20	1.64	--	--	--	2	21.20	2.17	3	3	1.67	0.098	38.76	29.25	1.828	1.380	1.812	1.378
12	0.30	20.70	3.688	0.675	15.22	15.90	1.04	--	--	--	2	15.90	1.63	2	2	2.50	0.066	30.44	19.50	1.914	1.226	1.901	1.227

AASHTO 2007 (LRFD)
MSE Wall Recertification

PROJECT IDENTIFICATION

Title:	MSE Wall Recertification
Project Number:	J2008- 04
Client:	ADOT
Designer:	N. Samtani
Station Number:	-

Description:

Problem 2, 30-ft high wall (Comparison with hand calculations with load factor for live load equal to 1.75 and load factor for soil wt equal to 1.35)

Company's information:

Name: NCS Consultants, LLC
Street: 640 W Paseo Rio Grande

Tucson, AZ 85737
Telephone #: 520-544-2786
Fax #: 520-544-3150
E-Mail: nsamtani@ncsconsultants.com

Original file path and name: P:\ADOT On-Call 07-24.5\01 - MSE Wall Re-Certifications..... .....S-M Prob 1 30-ft.BEN

Original date and time of creating this file: Thu May 29 11:55:15 2008

PROGRAM MODE: ANALYSIS of a SIMPLE STRUCTURE using METAL STRIPS as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	30.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	125.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	30.0 °
Equivalent cohesion, $c_{equiv.}$	0.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3333 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 30.14 N_γ = 22.40

SEISMICITY

Not Applicable

INPUT DATA: Metal strips
(Analysis)

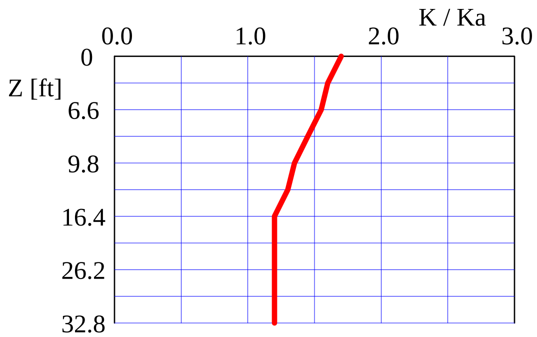
D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [kips/in ²]	65.0	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in ²]	0.20	N/A	N/A	N/A	N/A

Ribbed steel strips.
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0

Friction angle along reinforcement-soil interface,	ρ				
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.67	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

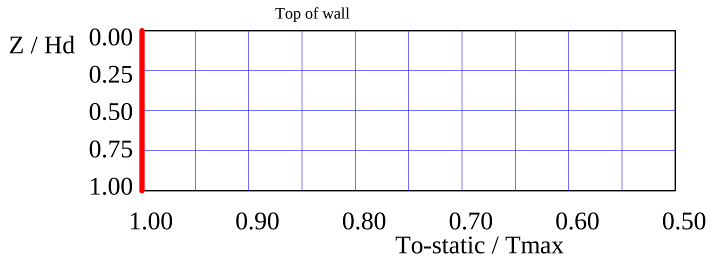
Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20



INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 1.31 ft. Horizontal distance to Center of Gravity of panel is 0.66 ft.
Average unit weight of panel is γ_f = 150.00 lb/ft ^{3}

Z / Hd	To-static / Tmax
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	---	N/A	N/A	N/A	N/A
Strength reduction at the connection, CRu = Fyc / Fy	0.90	N/A	N/A	N/A	N/A

INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	30.00	[ft]	{ Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 30.00 ft }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M S U R C H A R G E

Uniformly distributed dead load is 0.0 [lb/ft ^{2}], and live load is 250.0 [lb/ft ^{2}]

ANALYZED REINFORCEMENT LAYOUT:

SCALE:

0 5 10 15 20 25 30 [ft]

AASHTO 2007 (LRFD) Input Data

INTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2:	γ_{p-EV}	1.35		
Load factor for earthquake loads, EQ, from Table 3.4.1-1:	γ_{p-EQ}	1.00		
Load factor for live load surchrge, LS, from Figure C11.5.5-3(b): (Same as in External Stability).	γ_{p-LS}	1.75		
Load factor for dead load surchrge, ES: (Same as in External Stability).	γ_{p-ES}	1.50		
Resistance factor for reinforcement tension from Table 11.5.6-1:	ϕ	Static	Combined static/seismic	
Metal Strips:		0.75		1.00
Resistance factor for reinforcement tension in connectors from Table 11.5.6-1:	ϕ	Static	Combined static/seismic	
Metal Strips:		0.75	1.00	
Resistance factor for reinforcement pullout from Table 11.5.6-1:	ϕ	0.90	1.20	

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2 and Figure C11.5.5-2:		Static	Combined Static/Seismic	
Sliding and Eccentricity	γ_{p-EV}	1.00	γ_{p-EQ}	1.00
Bearing Capacity	γ_{p-EV}	1.35	γ_{p-EQ}	1.35

Load factor of active lateral earth pressure, EH, from Table 3.4.1-2 and Figure C11.5.5-2:	γ_{p-EH}	1.50
Load factor of active lateral earth pressure during earthquake (does not multiply P_{AE} and P_{IR}):	$(\gamma_{p-EH})_{EQ}$	1.50
Load factor for earthquake loads, EQ, from Table 3.4.1-1 (multiplies P_{AE} and P_{IR}):	γ_{p-EQ}	1.00

Resistance factor for shear resistance along common interfaces from Table 10.5.5.2.2-1:		Static	Combined Static/Seismic
Reinforced Soil and Foundation	ϕ_τ	0.90	1.00
Reinforced Soil and Reinforcement	ϕ_τ	0.90	1.00
Resistance factor for bearing capacity of shallow foundation from Table 10.5.5.2.2-1:		Static	Combined Static/Seismic
	ϕ_b	0.45	0.65

ANALYSIS: CALCULATED FACTORS (Static conditions)

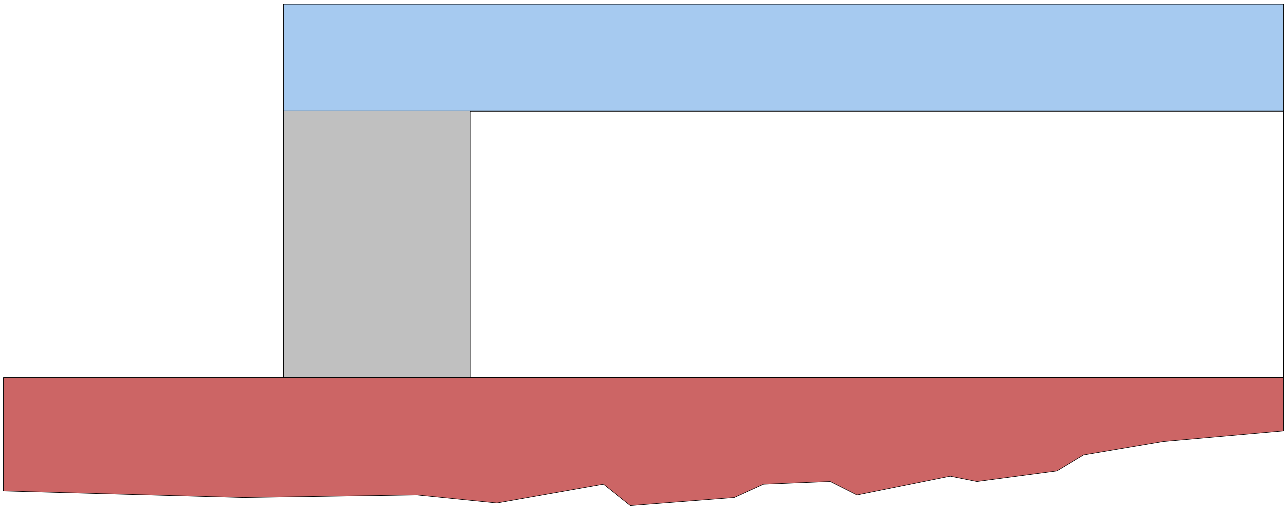
Bearing capacity, CDR = 1.23, Meyerhof stress = 7703 lb/ft².

Foundation Interface: Direct sliding, CDR = 1.259, Eccentricity, e/L = 0.2098, CDR-overturning = 2.38

METAL STRIP				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	CDR [pullout resistance]	CDR [connection break]	CDR [metal strip strength]	Metal strip strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	0.50	21.00	1	N/A	1.10	1.23	1.227	1.901	1.485	0.2035	---
2	3.00	21.00	1	N/A	1.24	1.38	1.378	1.812	1.599	0.1735	---
3	5.50	21.00	1	N/A	0.91	1.01	1.006	1.106	1.740	0.1458	---
4	8.00	21.00	1	N/A	1.00	1.11	1.108	1.002	1.903	0.1206	---
5	10.50	21.00	1	N/A	1.63	1.82	1.815	1.341	2.097	0.0976	---
6	13.00	21.00	1	N/A	1.76	1.96	1.960	1.371	2.354	0.0771	---
7	15.50	21.00	1	N/A	1.94	2.16	2.157	1.374	2.660	0.0589	---
8	18.00	21.00	1	N/A	2.18	2.43	2.427	1.469	3.060	0.0431	---
9	20.50	21.00	1	N/A	1.67	1.86	1.857	1.004	3.599	0.0296	---
10	23.00	21.00	1	N/A	2.01	2.23	2.233	0.992	4.371	0.0185	---
11	25.50	21.00	1	N/A	3.94	4.37	4.373	1.377	5.563	0.0098	---
12	28.00	21.00	1	N/A	6.58	7.31	7.311	1.119	7.649	0.0034	---

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Ultimate bearing capacity, q-ult	9447	N/A	[lb/ft ²]
Meyerhof stress, σ_V	7703.3	N/A	[lb/ft ²]
Eccentricity, e	3.00	N/A	[ft]
Eccentricity, e/L	0.143	N/A	
CDR calculated	1.23	N/A	
Base length	21.00	N/A	[ft]



SCALE:

0 5 10 15 20 25 30 [ft]

DIRECT SLIDING for GIVEN LAYOUT (for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.259

#	Metal strip Elevation [ft]	Metal strip Length [ft]	CDR Static	CDR Seismic	Metal strip Type #	Product name
1	0.50	21.00	1.485	N/A	1	---
2	3.00	21.00	1.599	N/A	1	---
3	5.50	21.00	1.740	N/A	1	---
4	8.00	21.00	1.903	N/A	1	---
5	10.50	21.00	2.097	N/A	1	---
6	13.00	21.00	2.354	N/A	1	---
7	15.50	21.00	2.660	N/A	1	---
8	18.00	21.00	3.060	N/A	1	---
9	20.50	21.00	3.599	N/A	1	---
10	23.00	21.00	4.371	N/A	1	---
11	25.50	21.00	5.563	N/A	1	---
12	28.00	21.00	7.649	N/A	1	---

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.2098; Overturning: CDR-static = 2.38

#	Metal strip Elevation [ft]	Metal strip Length [ft]	e / L Static	e / L Seismic	Metal strip Type #	Product name
1	0.50	21.00	0.2035	N/A	1	---
2	3.00	21.00	0.1735	N/A	1	---
3	5.50	21.00	0.1458	N/A	1	---
4	8.00	21.00	0.1206	N/A	1	---
5	10.50	21.00	0.0976	N/A	1	---
6	13.00	21.00	0.0771	N/A	1	---
7	15.50	21.00	0.0589	N/A	1	---
8	18.00	21.00	0.0431	N/A	1	---
9	20.50	21.00	0.0296	N/A	1	---
10	23.00	21.00	0.0185	N/A	1	---
11	25.50	21.00	0.0098	N/A	1	---
12	28.00	21.00	0.0034	N/A	1	---

RESULTS for STRENGTH [NLive Load included in calculating Tmaxctual stress)]

#	Metal strip Elevation [ft]	Coverage ratio, Rc=b/Sh	Horizontal spacing, Sh [ft]	Long-term strength Fy·Ac·Rc/b [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum CDR static	Actual calculated CDR static	Specified minimum CDR seismic	Actual calculated CDR seismic
1	0.50	0.066	2.500	3900	3177.69	N/A	N/A	1.227	N/A	N/A
2	3.00	0.098	1.670	5838	4235.42	N/A	N/A	1.378	N/A	N/A
3	5.50	0.066	2.500	3900	3877.61	N/A	N/A	1.006	N/A	N/A
4	8.00	0.066	2.500	3900	3519.80	N/A	N/A	1.108	N/A	N/A
5	10.50	0.098	1.670	5838	3216.36	N/A	N/A	1.815	N/A	N/A
6	13.00	0.098	1.670	5838	2978.84	N/A	N/A	1.960	N/A	N/A
7	15.50	0.098	1.670	5838	2706.55	N/A	N/A	2.157	N/A	N/A
8	18.00	0.098	1.670	5838	2405.36	N/A	N/A	2.427	N/A	N/A
9	20.50	0.066	2.500	3900	2100.69	N/A	N/A	1.857	N/A	N/A
10	23.00	0.066	2.500	3900	1746.25	N/A	N/A	2.233	N/A	N/A
11	25.50	0.098	1.670	5838	1335.10	N/A	N/A	4.373	N/A	N/A
12	28.00	0.131	1.250	7800	1066.84	N/A	N/A	7.311	N/A	N/A

RESULTS for PULLOUT Live Load included in calculating Tmax

NOTE: Live load is not included in calculating the overburden pressure used to assess pullout resistance.

#	Metal strip Elevation [ft]	Coverage Ratio Rc=b/Sh	Tmax [lb/ft]	Tmd [lb/ft]	Le [ft] (see NOTE)	La [ft]	Avail.Static Pullout, Pr [lb/ft]	Specified Static CDR	Actual Static CDR	Avail.Seism. Pullout, Pr [lb/ft]	Specified Seismic CDR	Actual Seismic CDR
1	0.50	0.066	3177.7	N/A	20.70	0.30	6040.4	N/A	1.901	N/A	N/A	N/A
2	3.00	0.098	4235.4	N/A	19.20	1.80	7676.5	N/A	1.812	N/A	N/A	N/A
3	5.50	0.066	3877.6	N/A	17.70	3.30	4289.5	N/A	1.106	N/A	N/A	N/A
4	8.00	0.066	3519.8	N/A	16.20	4.80	3525.4	N/A	1.002	N/A	N/A	N/A
5	10.50	0.098	3216.4	N/A	14.70	6.30	4312.0	N/A	1.341	N/A	N/A	N/A
6	13.00	0.098	2978.8	N/A	13.20	7.80	4084.4	N/A	1.371	N/A	N/A	N/A
7	15.50	0.098	2706.6	N/A	12.00	9.00	3718.3	N/A	1.374	N/A	N/A	N/A
8	18.00	0.098	2405.4	N/A	12.00	9.00	3533.6	N/A	1.469	N/A	N/A	N/A
9	20.50	0.066	2100.7	N/A	12.00	9.00	2110.0	N/A	1.004	N/A	N/A	N/A
10	23.00	0.066	1746.3	N/A	12.00	9.00	1732.6	N/A	0.992	N/A	N/A	N/A
11	25.50	0.098	1335.1	N/A	12.00	9.00	1838.5	N/A	1.377	N/A	N/A	N/A
12	28.00	0.131	1066.8	N/A	12.00	9.00	1193.3	N/A	1.119	N/A	N/A	N/A

RESULTS for CONNECTION (static conditions)

Live Load included in calculating Tmax

#	Metal strip Elevation [ft]	Coverage ratio Rc=b/Sh	Horizontal spacing, Sh [ft]	Connection force, To [lb/ft]	Reduction factor for connection break, CRu	Long-term connection strength,Tac (break criterion) [lb/ft]	Metal strip long-term strength, [lb/ft]	CDR connection break Specified	CDR connection break Actual	CDR Metal strip strength Specified	CDR Metal strip strength Actual	Product name
1	0.50	0.066	2.500	3178	0.90	3510	3900	N/A	1.10	N/A	1.23	---
2	3.00	0.098	1.670	4235	0.90	5254	5838	N/A	1.24	N/A	1.38	---
3	5.50	0.066	2.500	3878	0.90	3510	3900	N/A	0.91	N/A	1.01	---
4	8.00	0.066	2.500	3520	0.90	3510	3900	N/A	1.00	N/A	1.11	---
5	10.50	0.098	1.670	3216	0.90	5254	5838	N/A	1.63	N/A	1.82	---
6	13.00	0.098	1.670	2979	0.90	5254	5838	N/A	1.76	N/A	1.96	---
7	15.50	0.098	1.670	2707	0.90	5254	5838	N/A	1.94	N/A	2.16	---
8	18.00	0.098	1.670	2405	0.90	5254	5838	N/A	2.18	N/A	2.43	---
9	20.50	0.066	2.500	2101	0.90	3510	3900	N/A	1.67	N/A	1.86	---
10	23.00	0.066	2.500	1746	0.90	3510	3900	N/A	2.01	N/A	2.23	---
11	25.50	0.098	1.670	1335	0.90	5254	5838	N/A	3.94	N/A	4.37	---
12	28.00	0.131	1.250	1067	0.90	7020	7800	N/A	6.58	N/A	7.31	---

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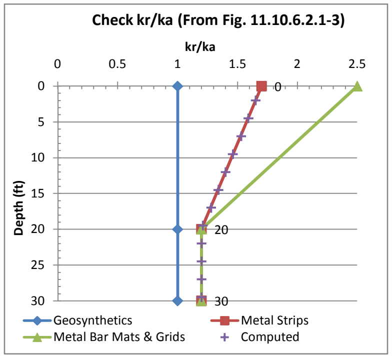
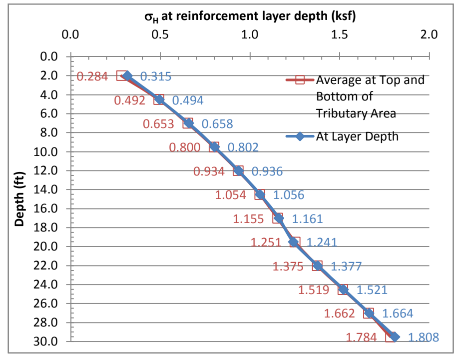
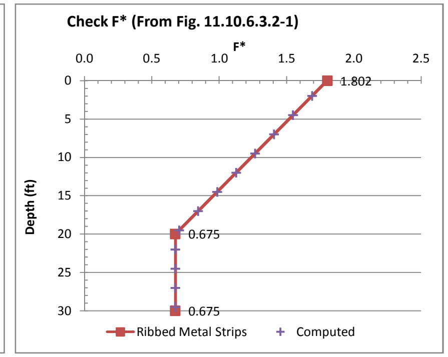
Comparison of Hand Calculations with MSEW Version 3.0, Update 9.1
(Calculations Based on LRFD AASHTO 2008 Interims)

SET 2

Made by:	N. Samtani	**Load Factors**		**Other Info**	
Date:	6/23/2008	γ_P =	1.35	# Layers =	12
Checked by:	WUF/RMP	γ_{LL} =	1.35	Panel width	5.0 ft
Date:	6/24/2008	γ_{EQ} =	1.00	Impact Load	0.0 k/ft
Basic Wall Parameters		γ (impact)=	1.00		
H (ft) =	30	**Resistance Factors**			
θ (deg) =	90	Pull ϕ_p =	0.9		
δ (deg) =	0	Str ϕ_s =	0.75		
β (deg) =	0	**Metal Strip Information**			
L (ft) =	21	b (mm) =	50		
γ_f=γ_r (kcf) =	0.125	b (in) =	1.969		
ϕ_f (deg) =	34	A_c (in^2) =	0.200	**MSEW file name**	
C_u (dim) =	4.0	F_y (ksi) =	65	NCS-M Prob 2 30-ft.ben	
Live Load Surcharge		T_{al} =	13 k/strip	**Calculation of k_a (dim)**	
H_{eq} (ft) =	2.0	α (dim) =	1.0	Γ =	2.431
σ_{vs} (ksf) =	0.250	C (dim) =	2	k_a (dim) =	0.283







Col 1	Col 2	Col 2	Col 4	Col 5	Col 6	Col 7	Col 8	Col 9	Col 10	Col 11	Col 12	Col 13	Col 14	Col 15	Col 16	Col 17	Col 18	Col 19	Col 20	Col 21	Col 22
Input layer geometry		Calculate k_r		Calculate Vertical Stresses		Calculate Horizontal Stress at Reinforcement Layer Depth			Calculate tributary height parameters (e.g., top and bottom depths)			Calculate Horizontal Stress at top and bottom of tributary heights			Calculate Tmax		Data from MSEW Program Output			Comparison of Tmax with MSEW output	
Layer #	Depth from top of wall (or coping as appropriate)	Stress Ratio	k_r	Vert Stress due to soil load (i.e. w/o LL), σ_{vE}	Vert Stress w/ LL (i.e., $\Delta\sigma_{vs}$) for Tmax, σ_{vMax}	Horiz Stress due to Soil Load, σ_{HE}	σ_{HLL} (ksf)	Total Horiz Stress (w/ LL), σ_H	Trib. Height, S_v (ft)	Depth to top of Trib height	Depth to bottom of Trib height	σ_H at Z_{top}	σ_H at Z_{bot}	Average σ_H	Using Trapezoid tributary area	Using Rectangle tributary area	MSEW Layer #	Layer "Height" in MSEW program	Tmax from MSEW Program	% Tmax Diff vs. Trapezoid based σ_H (%)	% Tmax Diff vs. Rectangle based σ_H (%)
-	Z	k_r / k_a	$(k_r/k_a)(k_a)$	$(Z)(\gamma_r)$	$(\sigma_{vE}+\Delta\sigma_{vs})$	$(k_r)(\sigma_{vE})(\gamma_P)$	$(k_r)(\sigma_{vs})(\gamma_{LL})$	$(\sigma_{HE}+\sigma_{HLL})$	S_v	Z_{top}	Z_{bot}	σ_{Htop}	σ_{Hbot}	σ_{Havg}	Tmax_t	Tmax_r	-	-	Tmax_m	-	-
(dim)	(ft)	(dim)	(dim)	(ksf)	(ksf)	(ksf)	(ksf)	(ksf)	(ft)	(ft)	(ft)	(ksf)	(ksf)	(ksf)	(kips/ft)	(kips/ft)	(dim)	(ft)	(kips/ft)	(%)	(%)
1	2.0	1.650	0.467	0.250	0.500	0.158	0.158	0.315	3.250	0.00	3.25	0.162	0.406	0.284	0.924	1.024	12	28	0.915	0.9%	11.9%
2	4.5	1.590	0.450	0.563	0.813	0.342	0.152	0.494	2.500	3.25	5.75	0.406	0.577	0.492	1.229	1.235	11	25.5	1.223	0.5%	1.0%
3	7.0	1.530	0.433	0.875	1.125	0.511	0.146	0.658	2.500	5.75	8.25	0.577	0.729	0.653	1.633	1.644	10	23	1.638	-0.3%	0.4%
4	9.5	1.460	0.413	1.188	1.438	0.662	0.139	0.802	2.500	8.25	10.75	0.729	0.871	0.800	2.000	2.004	9	20.5	1.997	0.2%	0.4%
5	12.0	1.400	0.396	1.500	1.750	0.802	0.134	0.936	2.500	10.75	13.25	0.871	0.998	0.934	2.336	2.339	8	18	2.307	1.2%	1.4%
6	14.5	1.340	0.379	1.813	2.063	0.928	0.128	1.056	2.500	13.25	15.75	0.998	1.110	1.054	2.635	2.639	7	15.5	2.612	0.9%	1.0%
7	17.0	1.280	0.362	2.125	2.375	1.038	0.122	1.161	2.500	15.75	18.25	1.110	1.199	1.155	2.887	2.902	6	13	2.888	0.0%	0.5%
8	19.5	1.210	0.342	2.438	2.688	1.126	0.115	1.241	2.500	18.25	20.75	1.199	1.304	1.251	3.129	3.103	5	10.5	3.130	0.0%	-0.9%
9	22.0	1.200	0.340	2.750	3.000	1.262	0.115	1.377	2.500	20.75	23.25	1.304	1.447	1.375	3.438	3.443	4	8	3.435	0.1%	0.2%
10	24.5	1.200	0.340	3.063	3.313	1.406	0.115	1.521	2.500	23.25	25.75	1.447	1.590	1.519	3.797	3.802	3	5.5	3.793	0.1%	0.2%
11	27.0	1.200	0.340	3.375	3.625	1.549	0.115	1.664	2.500	25.75	28.25	1.590	1.734	1.662	4.155	4.160	2	3	4.151	0.1%	0.2%
12	29.5	1.200	0.340	3.688	3.938	1.693	0.115	1.808	1.750	28.25	30.00	1.734	1.834	1.784	3.121	3.163	1	0.5	3.118	0.1%	1.4%

	Col 23	Col 24	Col 25	Col 26	Col 27	Col 28	Col 29	Col 30	Col 31	Col 32	Col 33	Col 34	Col 35	Col 36	Col 37	Col 38	Col 39	Col 40	Col 41	Col 42	Col 43	Col 44	Col 45
	Calculate Active and Effective Lengths		Calculate Factored Pullout Resistance			Calculate Number of Strips based on no impact		Calculate Number of Strips based on consideration of impact load				Calculate Number of Strips based on consideration of tensile strength			Calculate Governing Number of Strips, horizontal spacing and coverage ratio			Calculate Provided Factored Resistances		Calculate Capacity Demand Ratios (CDRs)		CDRs from MSEW Output	
Layer #	Active Length	Effective Length	Vert Stress for Pullout Resistance	Pullout Resistance Factor	Factored Pullout Resistance per strip	Tmax for a 5-ft wide panel	N Strips - Calc per panel for pullout w/o impact load	Tmax w/ Impact	Factored Pullout Resistance	N Strips - Calc (Impact)	Governing N Strips - Pullout	Tmax for panel width including impact	N Strips - Calc based on Tensile Strength	N Strips - Actual Based on Tensile Strength	Governing Final N (Strength & Pullout)	Required Horiz rein spacing using Final N	Required Rein coverage ratio using	Provided Factored Pullout Resistance Per Panel	Provided Factored Tensile Resistance Per Panel	Capacity Demand Ratio for Pullout	Capacity Demand Ratio for Tensile Strength	CDR for pullout from MSEW	CDR for tensile strength from MSEW
-	La	Le	$\sigma_{vPullout}$ = σ_{vE}	F*	ϕ_pP_r	Tmax_p	Tmax_p / ϕ_pP_r	Tmax_i	$\phi_pP_{r\_i}$	Tmax_i / $\phi_pP_{r\_i}$	-	Tmax_p	Tmax_p / ϕ_sT_{al}	-	-	S_h (ft)	R_c	-	$\phi_pP_{r\_i}$	CDR_p	CDR_t	CDR_pm	CDR_tm
(dim)	(ft)	(ft)	(ksf)	(dim)	(kips)	(kips)	(dim)	(kips/ft)	(kips/ft)	(dim)	(dim)	(kips)	(dim)	(dim)	(dim)	(ft)	(ft)	(kips)	(kips)	(dim)	(dim)	(dim)	(dim)
1	9.00	12.00	0.250	1.689	1.50	4.62	3.08	0.924	0.52	1.78	4	4.62	0.47	1	4	1.25	0.131	6.00	39.00	1.299	8.443	1.304	8.523
2	9.00	12.00	0.563	1.548	3.09	6.15	1.99	1.229	1.08	1.14	2	6.15	0.63	1	2	2.50	0.066	6.18	19.50	1.005	3.172	1.004	3.188
3	9.00	12.00	0.875	1.407	4.36	8.17	1.87	--	--	--	2	8.17	0.84	1	2	2.50	0.066	8.72	19.50	1.068	2.388	1.058	2.381
4	9.00	12.00	1.188	1.266	5.33	10.00	1.88	--	--	--	2	10.00	1.03	2	2	2.50	0.066	10.66	19.50	1.066	1.950	1.056	1.952
5	9.00	12.00	1.500	1.126	5.99	11.68	1.95	--	--	--	2	11.68	1.20	2	2	2.50	0.066	11.98	19.50	1.026	1.670	1.023	1.690
6	9.00	12.00	1.813	0.985	6.33	13.18	2.08	--	--	--	3	13.18	1.35	2	3	1.67	0.098	18.99	29.25	1.441	2.220	1.423	2.235
7	7.80	13.20	2.125	0.844	6.99	14.44	2.07	--	--	--	3	14.44	1.48	2	3	1.67	0.098	20.97	29.25	1.453	2.026	1.414	2.021
8	6.30	14.70	2.438	0.703	7.44	15.64	2.10	--	--	--	3	15.64	1.60	2	3	1.67	0.098	22.32	29.25	1.427	1.870	1.378	1.865
9	4.80	16.20	2.750	0.675	8.88	17.19	1.94	--	--	--	2	17.19	1.76	2	2	2.50	0.066	17.76	19.50	1.033	1.134	1.026	1.135
10	3.30	17.70	3.063	0.675	10.81	18.98	1.76	--	--	--	2	18.98	1.95	2	2	2.50	0.066	21.62	19.50	1.139	1.027	1.131	1.028
11	1.80	19.20	3.375	0.675	12.92	20.77	1.61	--	--	--	2	20.77	2.13	3	3	1.67	0.098	38.76	29.25	1.866	1.408	1.849	1.407
12	0.30	20.70	3.688	0.675	15.22	15.61	1.03	--	--	--	2	15.61	1.60	2	2	2.50	0.066	30.44	19.50	1.950	1.249	1.937	1.251

AASHTO 2007 (LRFD)
MSE Wall Recertification

PROJECT IDENTIFICATION

Title:	MSE Wall Recertification
Project Number:	J2008- 04
Client:	ADOT
Designer:	N. Samtani
Station Number:	-

Description:

Problem 2, 30-ft high wall (Comparison with hand calculations with live and soil wt load factors both equal to 1.35)

Company's information:

Name: NCS Consultants, LLC
Street: 640 W Paseo Rio Grande

Tucson, AZ 85737
Telephone #: 520-544-2786
Fax #: 520-544-3150
E-Mail: nsamtani@ncsconsultants.com

Original file path and name: P:\ADOT On-Call 07-24.5\01 - MSE Wall Re-Certifications.....
.....S-M Prob 2 30-ft.BEN

Original date and time of creating this file: Thu May 29 11:55:15 2008

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using METAL STRIPS as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	125.0 lb/ft ³
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ³
Design value of internal angle of friction, ϕ	30.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	125.0 lb/ft ³
Equivalent internal angle of friction, $\phi_{equiv.}$	30.0 °
Equivalent cohesion, $c_{equiv.}$	0.0 lb/ft ²

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3333 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 30.14 N_γ= 22.40

SEISMICITY

Not Applicable

INPUT DATA: Metal strips
(Analysis)

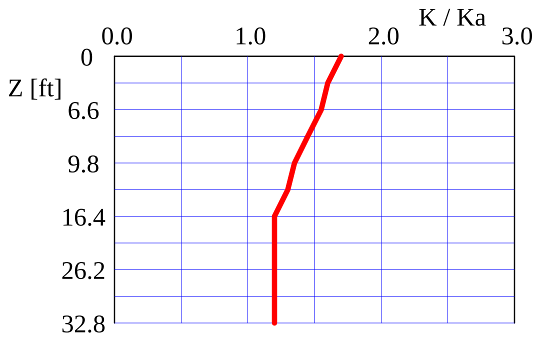
D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [kips/in ²]	65.0	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in ²]	0.20	N/A	N/A	N/A	N/A

Ribbed steel strips.
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0

Friction angle along reinforcement-soil interface,	ρ				
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.67	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

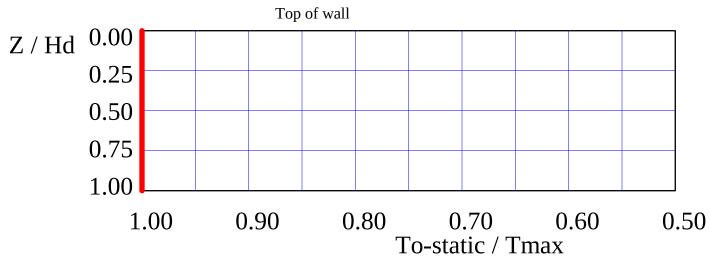
Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20



INPUT DATA: Facia and Connection
(Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 1.31 ft. Horizontal distance to Center of Gravity of panel is 0.66 ft.
Average unit weight of panel is γ_f = 150.00 lb/ft ³

Z / Hd	To-static / Tmax
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	---	N/A	N/A	N/A	N/A
Strength reduction at the connection, CRu = Fyc / Fy	0.90	N/A	N/A	N/A	N/A

INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	30.00	[ft]	{ Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 30.00 ft }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M S U R C H A R G E

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:

SCALE:

0 5 10 15 20 25 30 [ft]

AASHTO 2007 (LRFD) Input Data

INTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2:	$\gamma_{p\text{-}EV}$	1.35		
Load factor for earthquake loads, EQ, from Table 3.4.1-1:	$\gamma_{p\text{-}EQ}$	1.00		
Load factor for live load surchrge, LS, from Figure C11.5.5-3(b): (Same as in External Stability).	$\gamma_{p\text{-}LS}$	1.35		
Load factor for dead load surchrge, ES: (Same as in External Stability).	$\gamma_{p\text{-}ES}$	1.50		
Resistance factor for reinforcement tension from Table 11.5.6-1:	ϕ	Static	Combined static/seismic	
Metal Strips:		0.75		1.00
Resistance factor for reinforcement tension in connectors from Table 11.5.6-1:	ϕ	Static	Combined static/seismic	
Metal Strips:		0.75	1.00	
Resistance factor for reinforcement pullout from Table 11.5.6-1:	ϕ	0.90	1.20	

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2 and Figure C11.5.5-2:		Static		Combined Static/Seismic
Sliding and Eccentricity	$\gamma_{p\text{-}EV}$	1.00	$\gamma_{p\text{-}EQ}$	1.00
Bearing Capacity	$\gamma_{p\text{-}EV}$	1.35	$\gamma_{p\text{-}EQ}$	1.35

Load factor of active lateral earth pressure, EH, from Table 3.4.1-2 and Figure C11.5.5-2:	$\gamma_{p\text{-}EH}$	1.50
Load factor of active lateral earth pressure during earthquake (does not multiply P_{AE} and P_{IR}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.50
Load factor for earthquake loads, EQ, from Table 3.4.1-1 (multiplies P_{AE} and P_{IR}):	$\gamma_{p\text{-}EQ}$	1.00

Resistance factor for shear resistance along common interfaces from Table 10.5.5.2.2-1:		Static	Combined Static/Seismic
Reinforced Soil and Foundation	ϕ_τ	0.90	1.00
Reinforced Soil and Reinforcement	ϕ_τ	0.90	1.00
Resistance factor for bearing capacity of shallow foundation from Table 10.5.5.2.2-1:		Static	Combined Static/Seismic
	ϕ_b	0.45	0.65

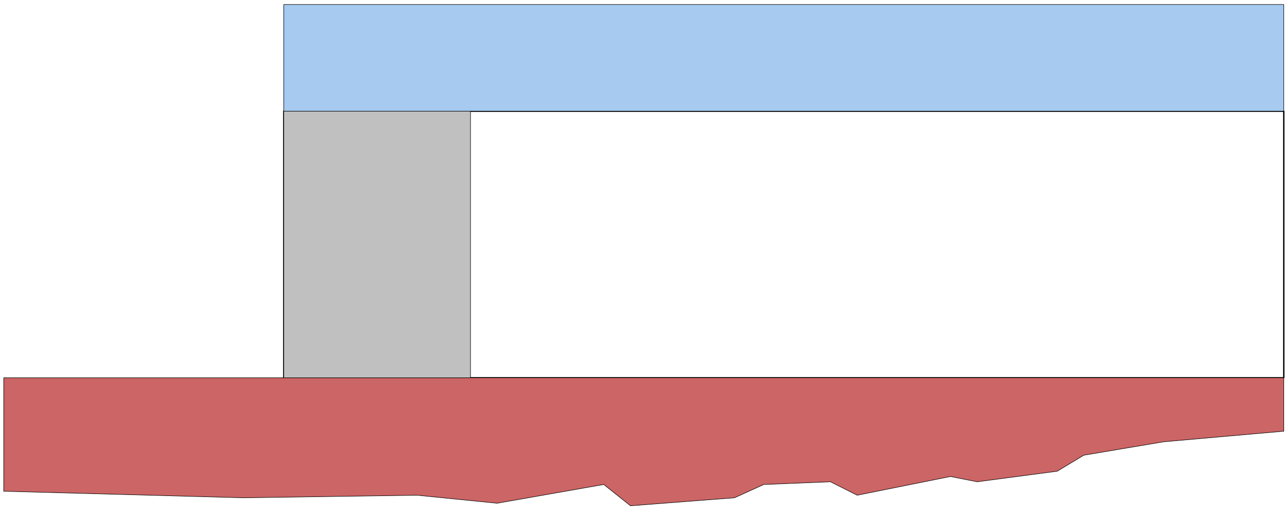
ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 1.27, Meyerhof stress = 7487 lb/ft².
Foundation Interface: Direct sliding, CDR = 1.299, Eccentricity, e/L = 0.2007, CDR-overturning = 2.49

METAL STRIP				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	CDR [pullout resistance]	CDR [connection break]	CDR [metal strip strength]	Metal strip strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	0.50	21.00	1	N/A	1.13	1.25	1.251	1.937	1.533	0.1945	---
2	3.00	21.00	1	N/A	1.27	1.41	1.407	1.849	1.654	0.1653	---
3	5.50	21.00	1	N/A	0.93	1.03	1.028	1.131	1.806	0.1384	---
4	8.00	21.00	1	N/A	1.02	1.14	1.135	1.026	1.982	0.1139	---
5	10.50	21.00	1	N/A	1.68	1.87	1.865	1.378	2.194	0.0918	---
6	13.00	21.00	1	N/A	1.82	2.02	2.021	1.414	2.475	0.0720	---
7	15.50	21.00	1	N/A	2.01	2.23	2.235	1.423	2.817	0.0545	---
8	18.00	21.00	1	N/A	1.52	1.69	1.690	1.023	3.269	0.0395	---
9	20.50	21.00	1	N/A	1.76	1.95	1.952	1.056	3.893	0.0267	---
10	23.00	21.00	1	N/A	2.14	2.38	2.381	1.058	4.811	0.0164	---
11	25.50	21.00	1	N/A	2.87	3.19	3.188	1.004	6.295	0.0084	---
12	28.00	21.00	1	N/A	7.67	8.52	8.523	1.304	9.106	0.0028	---

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Ultimate bearing capacity, q-ult	9544	N/A	[lb/ft ²]
Meyerhof stress, σ_V	7486.7	N/A	[lb/ft ²]
Eccentricity, e	2.93	N/A	[ft]
Eccentricity, e/L	0.139	N/A	
CDR calculated	1.27	N/A	
Base length	21.00	N/A	[ft]



SCALE:

0 5 10 15 20 25 30 [ft]

DIRECT SLIDING for GIVEN LAYOUT
(for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.299

#	Metal strip Elevation [ft]	Metal strip Length [ft]	CDR Static	CDR Seismic	Metal strip Type #	Product name
1	0.50	21.00	1.533	N/A	1	---
2	3.00	21.00	1.654	N/A	1	---
3	5.50	21.00	1.806	N/A	1	---
4	8.00	21.00	1.982	N/A	1	---
5	10.50	21.00	2.194	N/A	1	---
6	13.00	21.00	2.475	N/A	1	---
7	15.50	21.00	2.817	N/A	1	---
8	18.00	21.00	3.269	N/A	1	---
9	20.50	21.00	3.893	N/A	1	---
10	23.00	21.00	4.811	N/A	1	---
11	25.50	21.00	6.295	N/A	1	---
12	28.00	21.00	9.106	N/A	1	---

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.2007; Overturning: CDR-static = 2.49

#	Metal strip Elevation [ft]	Metal strip Length [ft]	e / L Static	e / L Seismic	Metal strip Type #	Product name
1	0.50	21.00	0.1945	N/A	1	---
2	3.00	21.00	0.1653	N/A	1	---
3	5.50	21.00	0.1384	N/A	1	---
4	8.00	21.00	0.1139	N/A	1	---
5	10.50	21.00	0.0918	N/A	1	---
6	13.00	21.00	0.0720	N/A	1	---
7	15.50	21.00	0.0545	N/A	1	---
8	18.00	21.00	0.0395	N/A	1	---
9	20.50	21.00	0.0267	N/A	1	---
10	23.00	21.00	0.0164	N/A	1	---
11	25.50	21.00	0.0084	N/A	1	---
12	28.00	21.00	0.0028	N/A	1	---

RESULTS for STRENGTH [NLive Load included in calculating Tmaxctual stress)]

#	Metal strip Elevation [ft]	Coverage ratio, Rc=b/Sh	Horizontal spacing, Sh [ft]	Long-term strength Fy·Ac·Rc/b [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum CDR static	Actual calculated CDR static	Specified minimum CDR seismic	Actual calculated CDR seismic
1	0.50	0.066	2.500	3900	3118.32	N/A	N/A	1.251	N/A	N/A
2	3.00	0.098	1.670	5838	4150.61	N/A	N/A	1.407	N/A	N/A
3	5.50	0.066	2.500	3900	3792.80	N/A	N/A	1.028	N/A	N/A
4	8.00	0.066	2.500	3900	3434.99	N/A	N/A	1.135	N/A	N/A
5	10.50	0.098	1.670	5838	3130.00	N/A	N/A	1.865	N/A	N/A
6	13.00	0.098	1.670	5838	2888.59	N/A	N/A	2.021	N/A	N/A
7	15.50	0.098	1.670	5838	2612.62	N/A	N/A	2.235	N/A	N/A
8	18.00	0.066	2.500	3900	2307.46	N/A	N/A	1.690	N/A	N/A
9	20.50	0.066	2.500	3900	1997.47	N/A	N/A	1.952	N/A	N/A
10	23.00	0.066	2.500	3900	1638.08	N/A	N/A	2.381	N/A	N/A
11	25.50	0.066	2.500	3900	1223.31	N/A	N/A	3.188	N/A	N/A
12	28.00	0.131	1.250	7800	915.19	N/A	N/A	8.523	N/A	N/A

RESULTS for PULLOUT Live Load included in calculating Tmax

NOTE: Live load is not included in calculating the overburden pressure used to assess pullout resistance.

#	Metal strip Elevation [ft]	Coverage Ratio Rc=b/Sh	Tmax [lb/ft]	Tmd [lb/ft]	Le [ft] (see NOTE)	La [ft]	Avail.Static Pullout, Pr [lb/ft]	Specified Static CDR	Actual Static CDR	Avail.Seism. Pullout, Pr [lb/ft]	Specified Seismic CDR	Actual Seismic CDR
1	0.50	0.066	3118.3	N/A	20.70	0.30	6040.4	N/A	1.937	N/A	N/A	N/A
2	3.00	0.098	4150.6	N/A	19.20	1.80	7676.5	N/A	1.849	N/A	N/A	N/A
3	5.50	0.066	3792.8	N/A	17.70	3.30	4289.5	N/A	1.131	N/A	N/A	N/A
4	8.00	0.066	3435.0	N/A	16.20	4.80	3525.4	N/A	1.026	N/A	N/A	N/A
5	10.50	0.098	3130.0	N/A	14.70	6.30	4312.0	N/A	1.378	N/A	N/A	N/A
6	13.00	0.098	2888.6	N/A	13.20	7.80	4084.4	N/A	1.414	N/A	N/A	N/A
7	15.50	0.098	2612.6	N/A	12.00	9.00	3718.3	N/A	1.423	N/A	N/A	N/A
8	18.00	0.066	2307.5	N/A	12.00	9.00	2360.4	N/A	1.023	N/A	N/A	N/A
9	20.50	0.066	1997.5	N/A	12.00	9.00	2110.0	N/A	1.056	N/A	N/A	N/A
10	23.00	0.066	1638.1	N/A	12.00	9.00	1732.6	N/A	1.058	N/A	N/A	N/A
11	25.50	0.066	1223.3	N/A	12.00	9.00	1228.1	N/A	1.004	N/A	N/A	N/A
12	28.00	0.131	915.2	N/A	12.00	9.00	1193.3	N/A	1.304	N/A	N/A	N/A

RESULTS for CONNECTION (static conditions)

Live Load included in calculating Tmax

#	Metal strip Elevation [ft]	Coverage ratio Rc=b/Sh	Horizontal spacing, Sh [ft]	Connection force, To [lb/ft]	Reduction factor for connection break, CRu	Long-term connection strength,Tac (break criterion) [lb/ft]	Metal strip long-term strength, [lb/ft]	CDR connection break Specified	CDR connection break Actual	CDR Metal strip strength Specified	CDR Metal strip strength Actual	Product name
1	0.50	0.066	2.500	3118	0.90	3510	3900	N/A	1.13	N/A	1.25	---
2	3.00	0.098	1.670	4151	0.90	5254	5838	N/A	1.27	N/A	1.41	---
3	5.50	0.066	2.500	3793	0.90	3510	3900	N/A	0.93	N/A	1.03	---
4	8.00	0.066	2.500	3435	0.90	3510	3900	N/A	1.02	N/A	1.14	---
5	10.50	0.098	1.670	3130	0.90	5254	5838	N/A	1.68	N/A	1.87	---
6	13.00	0.098	1.670	2889	0.90	5254	5838	N/A	1.82	N/A	2.02	---
7	15.50	0.098	1.670	2613	0.90	5254	5838	N/A	2.01	N/A	2.23	---
8	18.00	0.066	2.500	2307	0.90	3510	3900	N/A	1.52	N/A	1.69	---
9	20.50	0.066	2.500	1997	0.90	3510	3900	N/A	1.76	N/A	1.95	---
10	23.00	0.066	2.500	1638	0.90	3510	3900	N/A	2.14	N/A	2.38	---
11	25.50	0.066	2.500	1223	0.90	3510	3900	N/A	2.87	N/A	3.19	---
12	28.00	0.131	1.250	915	0.90	7020	7800	N/A	7.67	N/A	8.52	---

NCS

Job Evaluation of Tmax in MSE Walls Using LRFD Job # J2008-04

Computed by N. Samtani Date 6/23/08 Page of

Checked by Date Sheet of

ATTACHMENT 2

Page 1

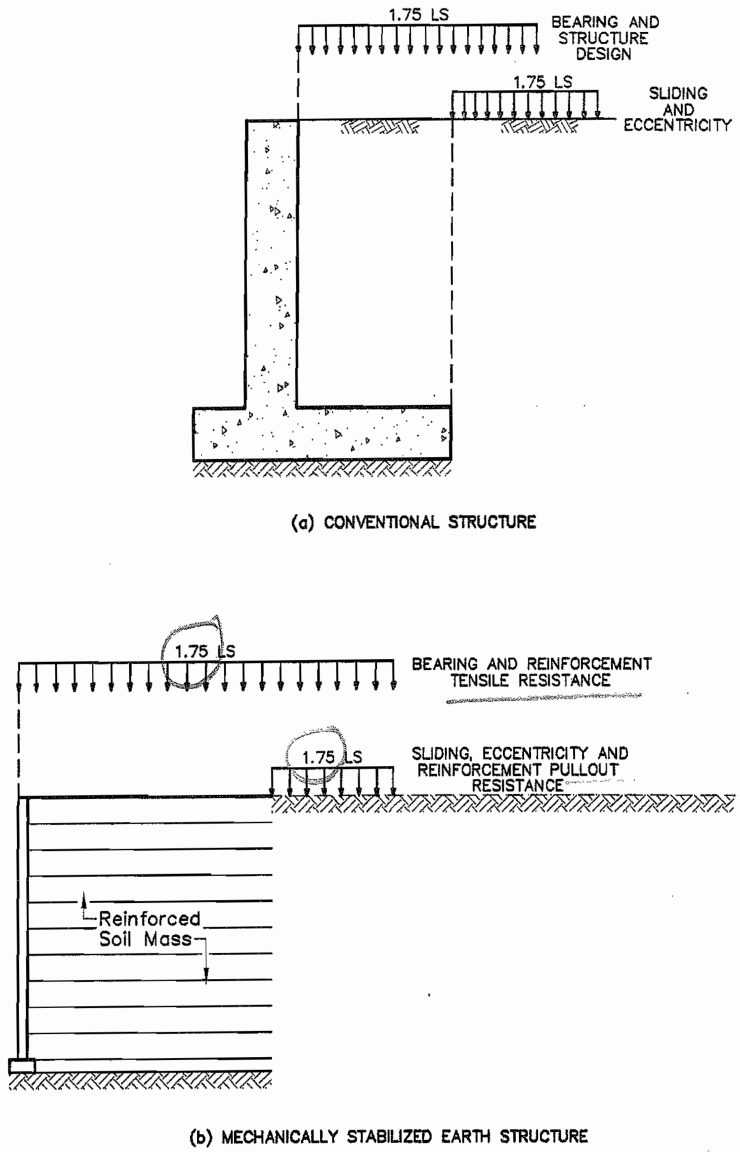
2008 Interim Revisions

AASHTO LRFD Bridge Design Specifications

Customary U.S. Units
4th Edition
2007

American Association of State Highway and Transportation Officials

Page 2



This figure shows Load factor for LL to be 1.75 and implies its use for internal stability calcs since it clearly says "...reinforcement tensile resistance" and "pullout resistance"

Figure C11.5.5-3 Typical Application of Live Load Surcharge.

11.5.6 Resistance Factors

Resistance factors for geotechnical design of foundations are specified in Tables 10.5.5-1 through 10.5.5-3 and Table 1.

If methods other than those prescribed in these Specifications are used to estimate resistance, the resistance factors chosen shall provide the same reliability as those given in Tables 10.5.5-1, 10.5.5-3, and Table 1.

Vertical elements, such as soldier piles, tangent-piles and slurry trench concrete walls shall be treated as either shallow or deep foundations, as appropriate, for purposes of estimating bearing resistance, using procedures described in Articles 10.6, 10.7, and 10.8.

C11.5.6

The resistance factors given in Table 1, other than those referenced back to Section 10, were calculated by direct correlation to allowable stress design rather than reliability theory.

Since the resistance factors in Table 1 were based on direct correlation to allowable stress design, the differences between the resistance factors for tensile resistance of metallic versus geosynthetic reinforcement are based on historical differences in the level of safety applied to reinforcement designs for these two types of reinforcements. See Article C11.10.6.2.1 for additional comments regarding the differences between the resistance factors for metallic versus geosynthetic reinforcement.

11.10.6.2.1 Maximum Reinforcement Loads

Maximum reinforcement loads shall be calculated using the Simplified Method approach. For this approach, the load in the reinforcements shall be obtained by multiplying the vertical earth pressure at the reinforcement by a lateral earth pressure coefficient, and applying the resulting lateral pressure to the tributary area for the reinforcement.

Other widely accepted and published design methods for calculation of reinforcement loads may be used at the discretion of the wall owner or approving agency, provided the designer develops method-specific resistance factors for the method employed.

The factored horizontal stress, σ_H, at each reinforcement level shall be determined as:

$$\sigma_H = \gamma_P \left(\sigma_v k_r + \Delta\sigma_H\right) \qquad (11.10.6.2.1\text{-}1)$$

where:

γ_P = the load factor for vertical earth pressure EV from Table 3.4.1-2

k_r = horizontal pressure coefficient (dim.)

This indicates use of only soil based loads and does not talk @ Live loads.

σ_v = pressure due to resultant of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present (ksf)

$\Delta\sigma_H$ = horizontal stress at reinforcement level resulting from any applicable concentrated horizontal surcharge load as specified in Article 11.10.10.1 (ksf)

Vertical stress for maximum reinforcement load calculations shall be determined as shown in Figures 1 and 2.

C11.10.6.2.1

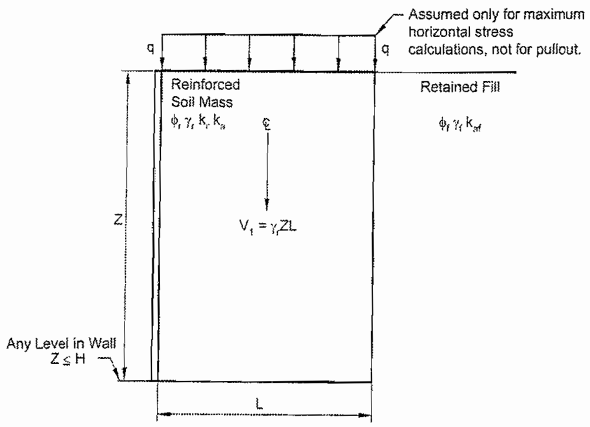
The design specifications provided herein assume that the wall facing combined with the reinforced backfill acts as a coherent unit to form a gravity retaining structure. Research by Allen and Bathurst (*2001*) indicates that reinforcement load is linear with reinforcement spacing to a reinforcement vertical spacing of 2.7 ft. or more, though a vertical spacing of this magnitude should not be attempted unless the facing is considered to be adequately stiff to prevent excessive bulging between layers (see Article C11.10.2.3.2).

These MSE wall specifications also assume that inextensible reinforcements are not mixed with extensible reinforcements within the same wall. MSE walls which contain a mixture of inextensible and extensible reinforcements are not recommended.

The calculation method for T_{max} is empirically derived, based on reinforcement strain measurements, converted to load based on the reinforcement modulus, from full scale walls at working stress conditions. The load factor EV, on the other hand, was determined in consideration of vertical earth pressure exerted by a soil mass without inclusions, and was calibrated to address uncertainties implied by allowable stress design for external stability for walls. EV is not directly applicable to internal reinforcement loads in MSE walls, since the calibration of EV was not performed with internal stability of a reinforced system in mind.

The use of EV for the load factor in this case should be considered an interim measure until research is completed to quantify load prediction bias and uncertainty.

Sloping soil surcharges are taken into account through an equivalent uniform surcharge and assuming a level backslope condition. For these calculations, the depth Z is referenced from the top of the wall at the wall face, excluding any copings and appurtenances.



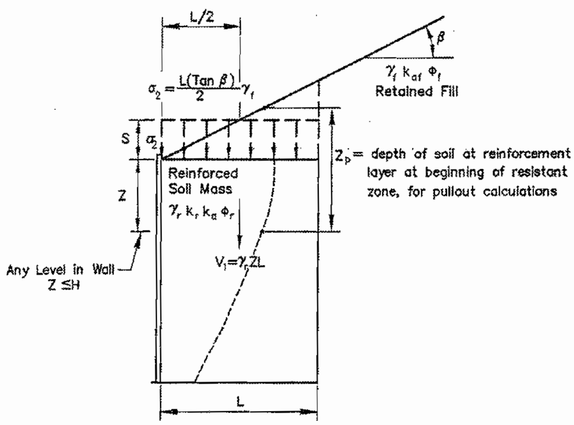
Max Stress: $\sigma_v = \gamma_r Z + q + \Delta\sigma_v$

Pullout: $\sigma_v = \gamma_r Z + \Delta\sigma_v$

Note: $\Delta\sigma_v$ is determined from Figure 11.10.10.1-1
H is the total wall height at the face.

Figure 11.10.6.2.1-1 Calculation of Vertical Stress for Horizontal Backslope Condition, Including Live Load and Dead Load Surcharges for Internal Stability Analysis.

This figure does not show any load factors. It is simply intended to show which loads should be considered for internal stability analysis. The designer should use appropriate load factors for each component of the load. e.g. for soil wt. based load (γZ) $\gamma_p = 1.35$ for Live loads (q) $\gamma = 1.75$ and so on



Max Stress: $S = (1/2)L\tan\beta$

$\sigma_v = \gamma_r Z + (1/2)L(\tan\beta)\gamma_f$

Determine k_{af} using a slope angle of β
Determine k_r from Figure 3
Pullout: $\sigma_v = \gamma_r Z_p$ and $Z_p \ge Z + S$

Note: H is the total height of the wall at the face.

Figure 11.10.6.2.1-2 Calculation of Vertical Stress for Sloping Backslope Condition for Internal Stability Analysis.

The lateral earth pressure coefficient k_r is determined by applying a multiplier to the active earth pressure coefficient, k_a, determined using Eq. 3.11.5.3-1, assuming no wall friction, i.e., $\delta = \beta$.

Since it is assumed that $\delta = \beta$, and β is assumed to always be zero for internal stability, for a vertical wall, the Coulomb equation simplifies mathematically to the simplest form of the Rankine equation.

The k_a multiplier shall be determined as shown in Figure 3.

The applied factored load to the reinforcements, T_{max}, shall be determined using a load per unit of wall width basis as follows:

$$T_{\max} = \sigma_H S_v \tag{11.10.6.2.1-2}$$

where:

σ_H = factored horizontal soil stress at the reinforcement (ksf)

S_v = vertical spacing of the reinforcement (ft.)

A vertical spacing, S_v, greater than 2.7 ft. should not be used without full scale wall data (e.g., reinforcement loads and strains, and overall deflections) that support the acceptability of larger vertical spacing.

* Live loads shall be positioned for extreme force effect. The provisions of Article 3.11.6 shall apply.

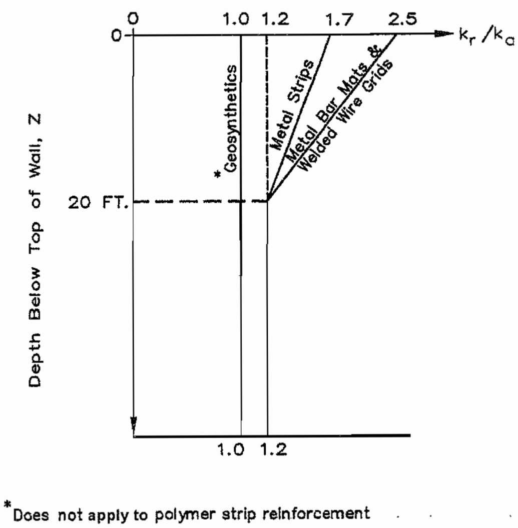


Figure 11.10.6.2.1-3 Variation of the Coefficient of Lateral Stress Ratio k_r/k_a with Depth in a Mechanically Stabilized Earth Wall.

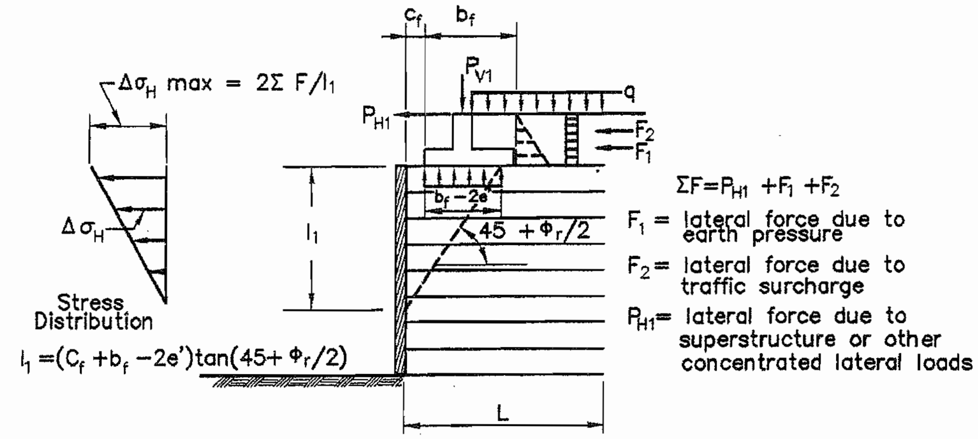
$$k_a = \tan^2\left(45 - \frac{\phi'_f}{2}\right) \tag{C11.10.6.2.1-1}$$

If the wall face is battered, the following simplified form of the Coulomb equation can be used:

$$k_a = \frac{\sin^2(\theta + \phi'_f)}{\sin^3\theta\left(1 + \frac{\sin\phi'_f}{\sin\theta}\right)^2} \tag{C11.10.6.2.1-2}$$

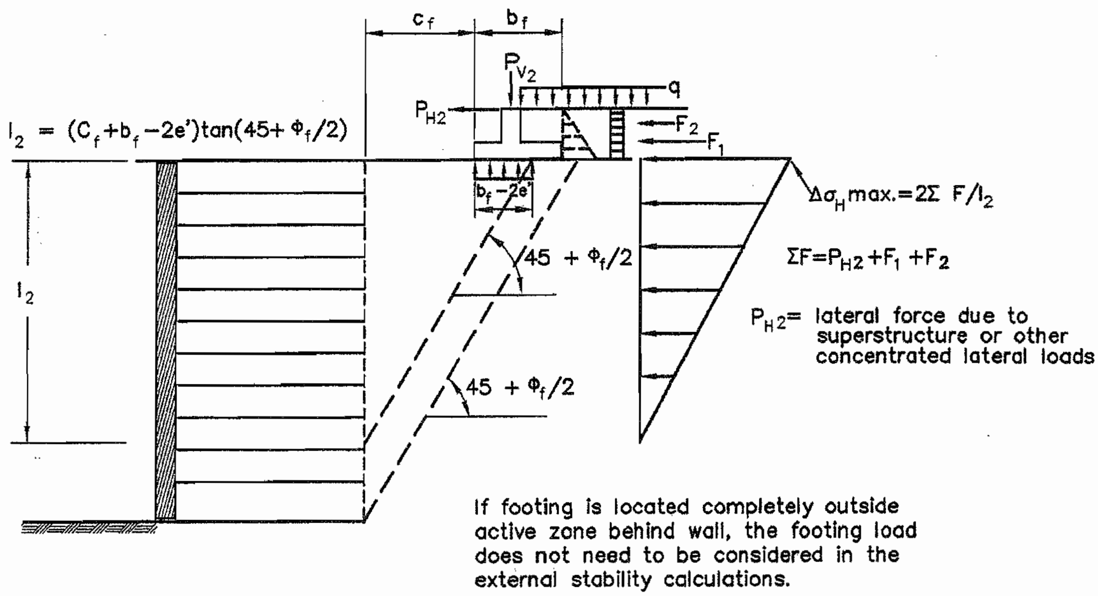
with variables as defined in Figure 3.11.5.3-1.

Based on Figure 3, the k_a multiplier is a function of the reinforcement type and the depth of the reinforcement below the wall top. Multipliers for other reinforcement types can be developed as needed through analysis of measurements of reinforcement load and strain in full scale structures.



e' = eccentricity of load on footing (see Figure 11.10.10.1-1 for example of how to calculate this)

a. Distribution of Stress for Internal Stability Calculations.



b. Distribution of Stress for External Stability Calculations.

Figure 3.11.6.3-2 Distribution of Stress from Concentrated Horizontal Loads.

3.11.6.4 Live Load Surcharge (*LS*)

A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall. If the surcharge is for a highway, the intensity of the load shall be consistent with the provisions of Article 3.6.1.2. If the surcharge is for other than a highway, the Owner shall specify and/or approve appropriate surcharge loads.

The increase in horizontal pressure due to live load surcharge may be estimated as:

C3.11.6.4

The tabulated values for h_{eq} were determined by evaluating the horizontal force against an abutment or wall from the pressure distribution produced by the vehicular live load of Article 3.6.1.2. The pressure distributions were developed from elastic half-space solutions using the following assumptions:

- Vehicle loads are distributed through a two-layer system consisting of pavement and soil subgrade

$$\Delta_p = k\gamma_s h_{eq} \quad (3.11.6.4\text{-}1)$$

where:

Δ_p = constant horizontal earth pressure due to live load surcharge (ksf)

γ_s = total unit weight of soil (kcf)

k = coefficient of lateral earth pressure

h_{eq} = equivalent height of soil for vehicular load (ft.)

Equivalent heights of soil, h_{eq}, for highway loadings on abutments and retaining walls may be taken from Tables 1 and 2. Linear interpolation shall be used for intermediate wall heights.

The wall height shall be taken as the distance between the surface of the backfill and the bottom of the footing along the pressure surface being considered.

Table 3.11.6.4-1 Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic.

Abutment Height (ft.)	h_{eq} (ft.)
5.0	4.0
10.0	3.0
≥20.0	2.0

Table 3.11.6.4-2 Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic.

Retaining Wall Height (ft.)	h_{eq} (ft.) Distance from wall backface to edge of traffic: 0.0 ft.	h_{eq} (ft.) Distance from wall backface to edge of traffic: 1.0 ft. or Further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

The load factor for both vertical and horizontal components of live load surcharge shall be taken as specified in Table 3.4.1-1 for live load surcharge. *

- Poisson's ratio for the pavement and subgrade materials are 0.2 and 0.4, respectively
- Wheel loads were modeled as a finite number of point loads distributed across the tire area to produce an equivalent tire contact stress
- The process for equating wall moments resulting from the elastic solution with the equivalent surcharge method used a wall height increment of 0.25 ft.

The value of the coefficient of lateral earth pressure k is taken as k_o, specified in Article 3.11.5.2, for walls that do not deflect or move, or k_a, specified in Articles 3.11.5.3, 3.11.5.6 and 3.11.5.7, for walls that deflect or move sufficiently to reach minimum active conditions.

The analyses used to develop Tables 1 and 2 are presented in Kim and Barker (*1998*).

The values for h_{eq} given in Tables 1 and 2 are generally greater than the traditional 2.0 ft. of earth load historically used in the AASHTO specifications, but less than those prescribed in previous editions (i.e., before 1998) of this specification. The traditional value corresponds to a 20.0-kip single unit truck formerly known as an H10 truck, Peck et al. (*1974*). This partially explains the increase in h_{eq} in previous editions of this specification. Subsequent analyses, i.e., Kim and Barker (*1998*) show the importance of the direction of traffic, i.e., parallel for a wall and perpendicular for an abutment on the magnitude of h_{eq}. The magnitude of h_{eq} is greater for an abutment than for a wall due to the proximity and closer spacing of wheel loads to the back of an abutment compared to a wall.

The backface of the wall should be taken as the pressure surface being considered. Refer to Article C11.5.5 for application of surcharge pressures on retaining walls.

3.11.6.5 Reduction of Surcharge

If the vehicular loading is transmitted through a structural slab, which is also supported by means other than earth, a corresponding reduction in the surcharge loads may be permitted.

C3.11.6.5

This Article relates primarily to approach slabs which are supported at one edge by the backwall of an abutment, thus transmitting load directly thereto.

Table 3.4.1-1 Load Combinations and Load Factors.

Use this column to obtain Load factors for Live Load

Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time *EQ*	*IC*	*CT*	*CV*
STRENGTH I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
STRENGTH V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
EXTREME EVENT I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—
EXTREME EVENT II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00
SERVICE I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—
SERVICE III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—
FATIGUE— *LL, IM* & *CE* ONLY	—	0.75	—	—	—	—	—	—	—	—	—	—	—

Table 3.4.1-2 Load Factors for Permanent Loads, γ_p.

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (*1999*) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments	for soil based loads	1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts		1.95	0.90
• Flexible Metal Box Culverts		1.50	0.90
ES: Earth Surcharge		1.50	0.75

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Table 3.4.1-3 Load Factors for Permanent Loads Due to Superimposed Deformations, γ_p.

Bridge Component	PS	CR, SH
Superstructures—Segmental Concrete Substructures supporting Segmental Superstructures (see 3.12.4, 3.12.5)	1.0	See γ_P for *DC*, Table 2
Concrete Superstructures—non-segmental	1.0	1.0
Substructures supporting non-segmental Superstructures • using I_g • using $I_{effectuve}$	 0.5 1.0	 0.5 1.0
Steel Substructures	1.0	1.0

Where prestressed components are used in conjunction with steel girders, the force effects from the following sources shall be considered as construction loads, *EL*:

- In conjunction with longitudinal prestressing of a precast deck prior to making the deck sections composite with the girders, the friction between the precast deck sections and the steel girders.
- When longitudinal post-tensioning is performed after the deck becomes composite with the girders, the additional forces induced in the steel girders and shear connectors.
- The effects of differential creep and shrinkage of the concrete.
- The Poisson effect.

The most common applications of prestressed concrete in steel girder bridges are transverse post-tensioning of the deck and integral pier caps in which the tendons penetrate the girder webs. When a composite deck is prestressed longitudinally, the shear connectors transfer force to the steel. The effect of shrinkage and long-term creep around the shear connectors should be evaluated to ensure that the composite girder is able to recognize the prestressing over the life of the bridge. The contribution of long-term deformations in closure pours between precast deck panels which have been aged to reduce shrinkage and creep may need evaluation.

The Poisson effect recognizes the bulging of concrete when subjected to prestressing. When used in pier caps, post-tensioning causes a transverse Poisson tensile stress resulting in a longitudinal stress in the steel girders.

The load factor for live load in Extreme Event Load Combination I, γ_{EQ}, shall be determined on a project-specific basis.

Past editions of the Standard` Specifications used $\gamma_{EQ} = 0.0$. This issue is not resolved. The possibility of partial live load, i.e., $\gamma_{EQ} < 1.0$, with earthquakes should be considered. Application of Turkstra's rule for combining uncorrelated loads indicates that $\gamma_{EQ} = 0.50$ is reasonable for a wide range of values of average daily truck traffic (ADTT).

A load factor for passive lateral earth pressure is not given in Table 2 because, strictly speaking, passive lateral earth pressure is a resistance and not a load. For discussion of the selection of a passive lateral earth pressure resistance factor see Article 10.5.5.2.2.

Job. Evaluation of Tmax in MSE Walls Using LRFD Job #. J2008-04.
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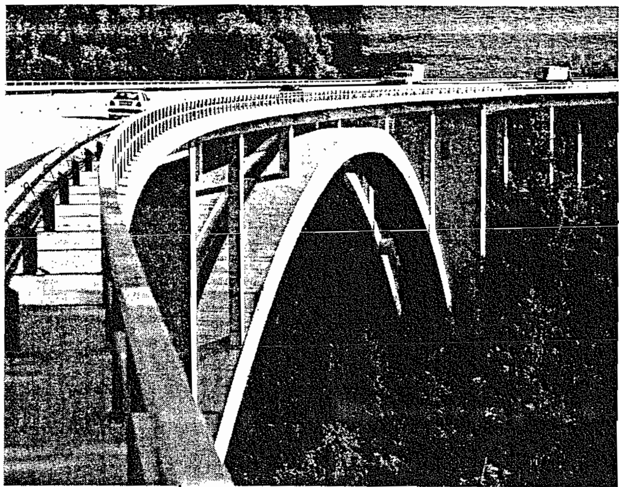
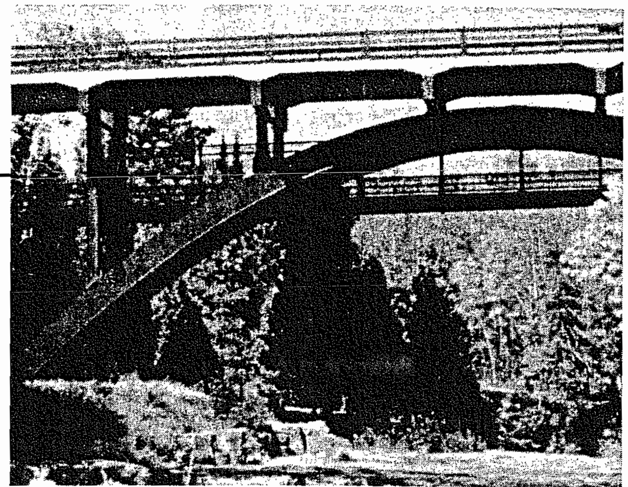
ATTACHMENT 3

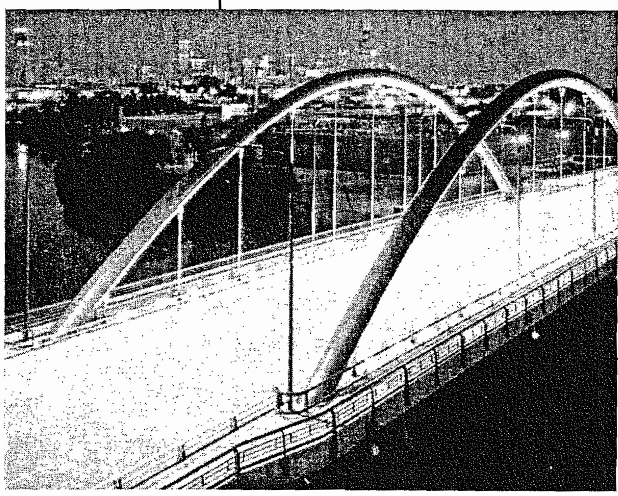
Wolfgang Fritz
Page 1

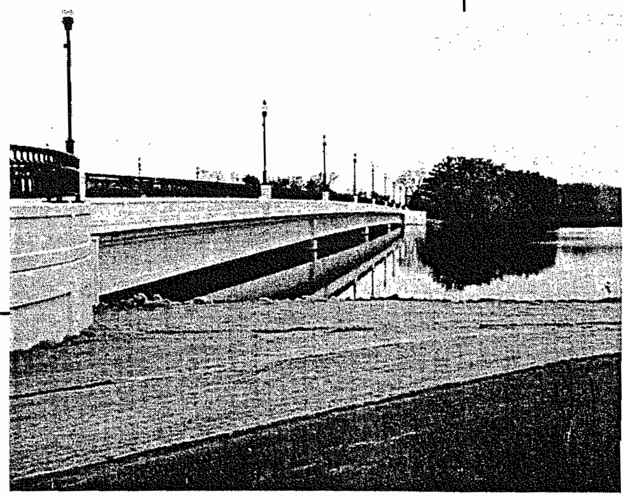
Standard Specifications for Highway Bridges

17th Edition – 2002









Upper right-hand and lower left-hand pictures courtesy of the National Steel Bridge Alliance.
Lower right-hand picture courtesy of William Oliva and Scott Becker.

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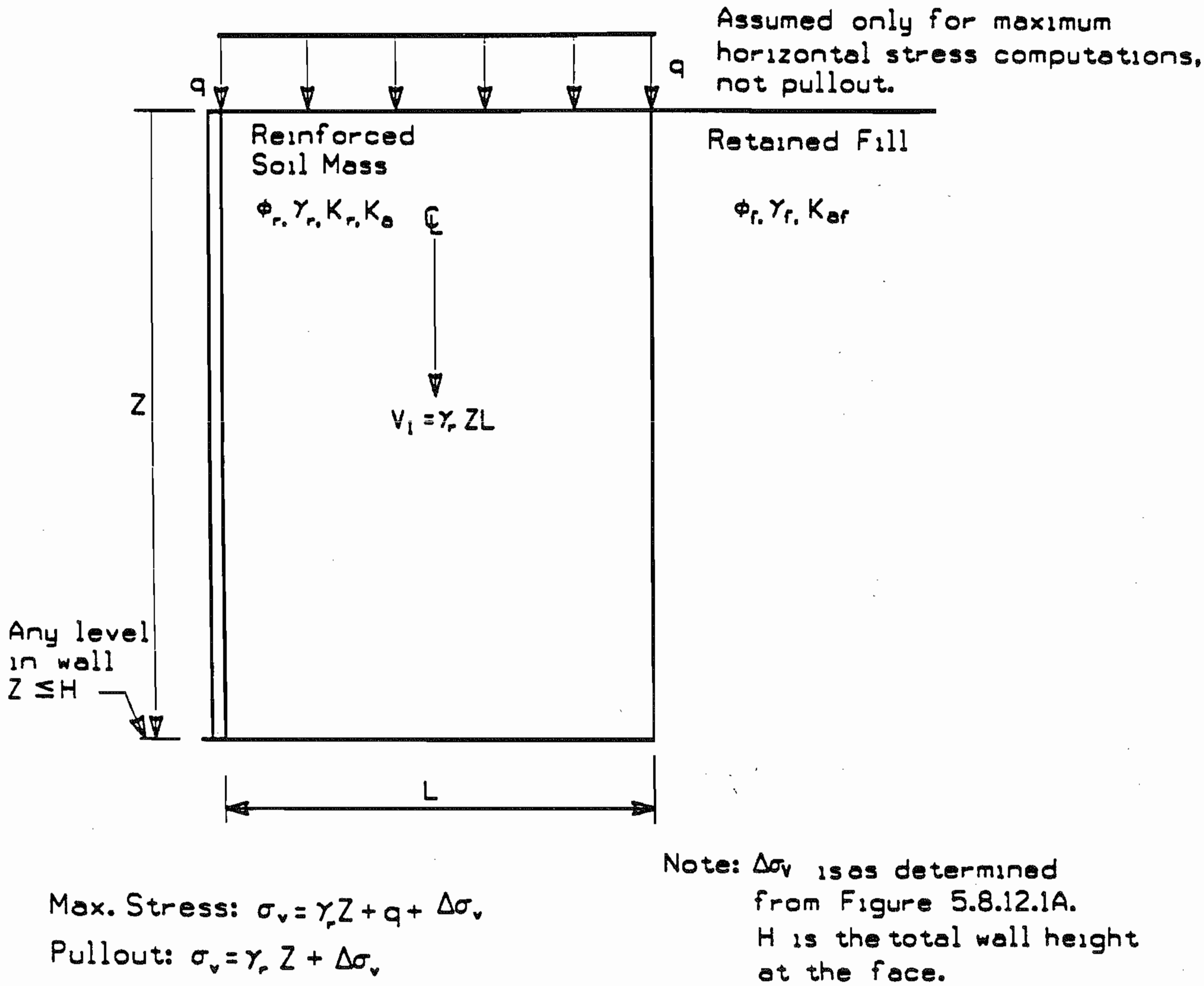


FIGURE 5.8.4.1A Calculation of Vertical Stress for Horizontal Backslope Condition, Including Live Load and Dead Load Surcharges for Internal Stability Design

not be used without full scale wall data (e.g., reinforcement loads and strains, and overall deflections) which supports the acceptability of larger vertical spacings.

These MSE wall specifications also assume that inextensible reinforcements are not mixed with extensible reinforcements within the same wall. MSE walls which contain a mixture of inextensible and extensible reinforcements are not recommended.

5.8.4.2 Determination of Reinforcement Tensile Load at the Connection to the Wall Face

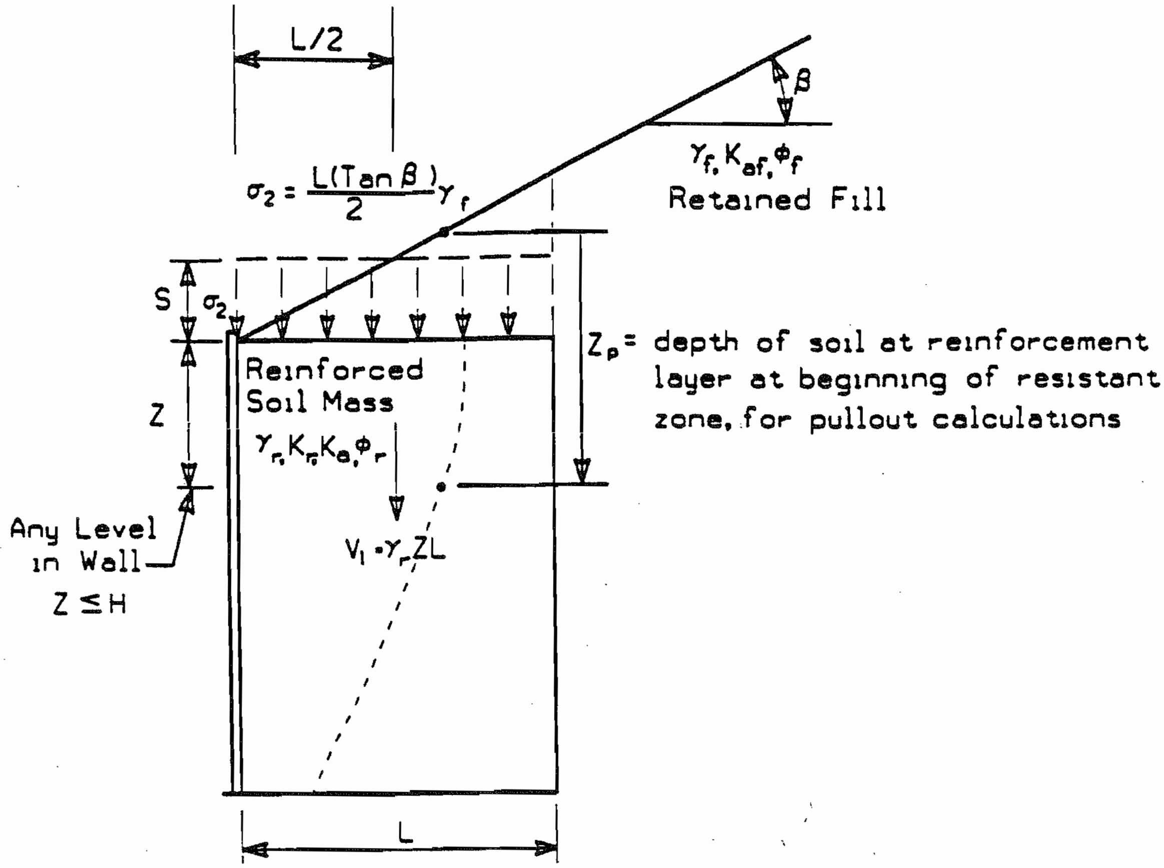
The tensile load applied to the soil reinforcement connection at the wall face, T_0, shall be equal to T_{max} for all wall systems regardless of facing and reinforcement type.

5.8.5 Determination of Reinforcement Length Required for Internal Stability

5.8.5.1 Location of Zone of Maximum Stress

The location of the zone of maximum stress for inextensible and extensible wall systems, which forms the boundary between the active and resistant zones, is determined as shown in Figure 5.8.5.1A. For all wall systems, the zone of maximum stress shall be assumed to begin at the back of the facing elements at the toe of the wall.

For extensible wall systems with a face batter of less than 10° from the vertical, the zone of maximum stress should be determined using the Rankine method.



Max Stress: $S = \frac{1}{2} L \, \mathrm{Tan}\,\beta$

$\sigma_v = \gamma_r Z + \frac{1}{2} L(\mathrm{Tan}\,\beta)\gamma_f$

with K_a determined using a slope angle of $0°$.

Determine K_r from Figure 5.8.4.1C.

Pullout: $\sigma_v = \gamma_r Z_p$, and $Z_p \geq Z + S$

Note: H is the total height of the wall at the face.

FIGURE 5.8.4.1B Calculation of Vertical Stress for Sloping Backslope Condition for Internal Stability Design

Since the Rankine method cannot account for wall face batter or the effect of concentrated surcharge loads above the reinforced backfill zone, the Coulomb method shall be used for walls with extensible reinforcement in cases of significant batter (defined as 10° from vertical or more) and concentrated surcharge loads to determine the location of the zone of maximum stress.

5.8.5.2 Soil Reinforcement Pullout Design

The reinforcement pullout resistance shall be checked at each level against pullout failure for internal stability. Only the effective pullout length which extends beyond the theoretical failure surfaces shall be used in this computation. Note that traffic loads are neglected in pullout calculations (see Figure 5.8.4.1.A).