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Maryland Department of Transportation

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STATE HIGHWAY ADMINISTRATION

RESEARCH REPORT

LRFD RESISTANCE FACTORS FOR MARYLAND RETAINING WALLS

CIVIL AND ENVIRONMENTAL ENGINEERING DEPARTMENT
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16. Abstract

AASHTO, LRFD specifications for retaining walls were summarized and presented in this report. To carry out comparative design between ASD and LRFD specifications, three types of retaining walls that are used by Maryland SHA were analyzed by both the ASD and LRFD methods of design. This provides a guide for engineers who are not familiar with LRFD methodology but are interested in implementing it. A spread-sheet program for the design of three types of retaining walls based on AASHTO LRFD specifications were developed, which will facilitate the design of these walls for different geometry and soil properties. All analyses undertaken dealt only with the external stability of the wall, i.e., its resistance to overturning, sliding and bearing failure.

Standard cantilever walls with different heights were then analyzed and their resistance factors determined. The resistance factors determined were found to be much less than the values recommended by the AASHTO specification, i.e., the walls were originally overdesigned. By varying the base dimension of a 20 ft high wall, a reduction in cross-sectional area of the wall of up to 34% can be achieved with the wall still within the AASHTO specification. Thus, unless there is a structural reason for the current dimensions of theses cantilever walls, they can be reduced in cross-sectional area based on the geotechnical analyses undertaken, which will translate into a reduction in cost of the retaining wall.

A study was also undertaken on the effect of the life load surcharge on the resistance factors. It was found that with the larger life load surcharge recommended by the AASHTO specification for shorter walls compared to the taller walls, the resistance factors are still acceptable. However, the shorter walls have higher resistance factors, as was expected.

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LRFD Resistance Factors for Maryland Retaining Walls

Report Submitted

to

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by

M. Sherif Aggour

Civil and Environmental Engineering Department
University of Maryland
College Park, Maryland
20742

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SUMMARY

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INTRODUCTION

1.1 General Overview

The design of foundations, retaining walls, etc., has traditionally been performed using allowable stress design (ASD) in which all uncertainty in loads and material resistance is combined in a factor of safety. The factor of safety is an empirical, but arbitrary, measure used to reduce the potential for adverse performance. AASHTO and FHWA are committed to transforming the current ASD method to load and resistance factor design (LRFD). LRFD is based primarily on a rational evaluation of performance reliability. It represents an approach in which applicable failure and serviceability conditions can be evaluated considering the uncertainties associated with loads and material resistance. AASHTO no longer publishes the ASD code, only the LRFD code. Several states, including Pennsylvania, West Virginia, etc. are already using LRFD.

In the LRFD, various types of loads are multiplied by load factors and the ultimate resistance is multiplied by a resistance factor. The uncertainty in loads is represented by load factors that generally have a value greater than one, and the uncertainty in material resistance is represented by a resistance factor that generally has a value less than one. For substructure design, the majority of loads that must be supported are prescribed by the structural designer, thus geotechnical engineers have only limited control over the load side of the relationship.

In geotechnical design, the resistance factors depend on the uncertainties associated with the variability and reliability of different factors that include the extent of soil exploration and type of sampling and testing used to characterize a site; inherent soil variability; soil property measurements; the procedures or models used for design; and the measures employed to monitor the construction processes. Thus selecting resistance factors that target an acceptable probability of survival is a difficult one. However, geotechnical engineers have the opportunity to control the extent and type of sampling and testing used to characterize a site, and the procedures or models used for design.

1.2 Objective of the Study

The objective of the study was to present the procedure used in design using the LRFD. The procedure was then demonstrated by analyzing three retaining walls, of the type that are used by Maryland SHA, both by the ASD and LRFD. The study focused on global stability (i.e., the external stability that includes sliding, overturning and bearing of the wall systems).

The three retaining walls analyzed were:

1) A cantilever wall, Type A, Standard No. RW (6.03)-83-134. (Appendix A)
2) Crib Wall-Type A, RW (6.01)-79-18. (Appendix B)
3) A mechanically stabilized earth wall (MSE wall).

The results of the design were to be analyzed and the resistance factors used in those Maryland retaining walls determined.

Another objective of the study was to develop a spreadsheet program for the design of the three types of retaining walls using AASHTO LRFD specification (Appendix C). The Excel program was to be used to check the hand calculations and facilitate the design of these walls for different geometry and soils properties.

1.3 Organization of the Report

This report is divided into seven chapters. Chapter II presents the design procedure of retaining walls by LRFD using the AASHTO LRFD specifications. Chapters, III, IV, and V present the design of the cantilever retaining wall, the crib retaining wall and the MSE wall, respectively by both the ASD and LRFD. Chapter VI is the analysis of the design results and Chapter VII is the conclusion of the study.

CHAPTER II

DESIGN PROCEDURE

2.1 Introduction

This chapter presents the load and resistance factor design of retaining walls. The chapter presents AASHTO LRFD design procedures including the 2002 Interim Revisions. Tables presented in the chapter were produced from the 2002 Interim Revisions published in May 2002. The tables numbers, as shown in the AASHTO publication, were kept on the tables as it is expected that additional revisions of AASHTO publications will change some of the numbers in the tables but not the table numbers. This way it will be easier for the State to update this report.

Another reference that was utilized in this chapter is the "Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures," Federal Highway Administration, Publication No. FHWA HI-98-032, July 1998.

As stated before, in the LRFD, various types of loads are multiplied by load factors and the ultimate resistance is multiplied by a resistance factor. The uncertainty in loads is represented by load factors that generally have a value greater than one, and the uncertainty in material resistance is represented by a resistance factor that generally has a value less than one.

As used in the AASHTO LRFD specification, the basic LRFD equation is defined by:

$$\Sigma \gamma_i Q_i \le \phi R_n$$

where: γ_i = load factors, Q_i = applied load, R_n = ultimate resistance, and ϕ = resistance factor.

2.2 Load Factors

For substructure design, the majority of loads that must be supported are prescribed by the structural designer, thus geotechnical engineers have only limited control over the load side of the relationship.

Table 2.1 presents AASHTO load combinations and load factors as well as the table for the maximum and minimum load factors of the permanent loads. Based on AASHTO 2002, any structure should be evaluated for 11 cases of limit states as identified in Table 2.1 (five strength, 2 extreme event, 3 service and one fatigue). However, depending on the particular loading conditions and performance characteristics of the structure, only certain limit states need to be evaluated.

As was presented in the FHWA report, each limit state was assessed to determine its applicability for the retaining wall problem.

Strength I – applicable as it is a basic load combination

Strength II – not applicable – no special design vehicles

Strength III – not applicable – requires wind loading exceeding 90 km/hr

Strength IV – applicable – when dead loads predominate

Strength V – not applicable – again consider wind loads

Extreme Event I – not applicable – no earthquake loading

Extreme Event II – not applicable – no ice or collision loading

Service I – applicable – basic load combination

Service II – not applicable due to structure type

Service III – not applicable due to structure type

Fatigue – not applicable due to structure type

Table 2.1 Limit state, load combinations and load factors

Table 3.4.1-1 - Load Combinations and Load Factors

Load Combination Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	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 EH, EV, ES, DW DC ONLY	γ_p 1.5	-	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	γ_{EQ}	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}	-	-	-	-
FATIGUE-LL, IM & CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-	-	-

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

Type of Load	Load Factor Maximum	Minimum
DC: Component and Attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.50	0.65
EH: Horizontal Earth Pressure • Active • At-Rest	 1.50 1.35	 0.90 0.90
EL: Locked-in Erection Stresses	1.00	1.00
EV: Vertical Earth Pressure • Overall Stability • Retaining Walls and Abutments • Rigid Buried Structure • Rigid Frames • Flexible Buried Structures other than Metal Box Culverts • Flexible Metal Box Culverts	 1.00 1.30 1.35 1.95 1.50	 N/A 0.90 0.90 0.90 0.90
ES: Earth Surcharge	1.50	0.75

Consequently, only the strength I, strength IV and service I limit states apply to retaining wall design. Since we have both minimum and maximum load factors for permanent loads, for every limit state we will have a case, a, that utilizes minimum load factors and case, b, that utilizes maximum load factors.

In summary, the following are the five cases to be analyzed:

1. Strength I-a (uses min and max load factors)
2. Strength I-b (uses min and max load factors)
3. Strength IV-a (uses min and max load factors)
4. Strength IV-b (uses min and max load factors)
5. Service I

2.3 Resistance Factors

In geotechnical design, the resistance factors depend on the uncertainties associated with the variability and reliability of different factors that include the extent of soil exploration and type of sampling and testing used to characterize a site; inherent soil variability; soil property measurements; the procedures or models used for design; and the measures employed to monitor the construction processes. Thus selecting resistance factors that target an acceptable probability of survival is a difficult one. However, geotechnical engineers have the opportunity to control the extent and type of sampling and testing used to characterize a site, and the procedures or models used for design.

AASHTO 2002 interim provides the resistance factors for geotechnical design of foundations. Table 2.2 provides the resistance factors for both the bearing capacity and sliding for shallow foundations. As stated in the FHWA report "that whereas the ASD factor of safety for bearing resistance and sliding are fixed, however, the LRFD

Table 2.2 Resistance factors

Table 10.5.5-1 - Resistance Factors for Strength Limit State for Shallow Foundations

METHOD/SOIL/CONDITION			RESISTANCE FACTOR
Bearing Capacity and Passive Pressure		Sand	
		- Semiempirical procedure using SPT data	0.45
		- Semiempirical procedure using CPT data	0.55
		- Rational Method --	
		using φ_f estimated from SPT data	0.35
		using φ_f estimated from CPT data	0.45
		Clay	
		- Semiempirical procedure using CPT data	0.50
		- Rational Method --	
		using shear resistance measured in lab tests	0.60
		using shear resistance measured in field vane tests	0.60
		using shear resistance estimated from CPT data	0.50
		Rock	
		- Semiempirical procedure, Carter and Kulhawy (1988)	0.60
		Plate Load Test	0.55
Sliding		Precast concrete placed on sand	
		using φ_f estimated from SPT data	0.90
		using φ_f estimated from CPT data	0.90
		Concrete cast-in-place on sand	
		using φ_f estimated from SPT data	0.80
		using φ_f estimated from CPT data	0.80
	φ_τ	Sliding on clay is controlled by the strength of the clay when the clay shear is less than 0.5 times the normal stress and is controlled by the normal stress when the clay shear strength is greater than 0.5 times the normal stress (see Figure 1, which is developed for the case in which there is at least 6.0 IN of compacted granular material below the footing).	
		Clay (where shear resistance is less than 0.5 times normal pressure)	
		using shear resistance measured in lab tests	0.85
		using shear resistance measured in field tests	0.85
		using shear resistance estimated from CPT data	0.80
		Clay (where the resistance is greater than 0.5 times normal pressure)	0.85
		Soil on soil	1.0
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

resistance factors could possibly be increased with additional date accumulation and reliability calibration for similar soils."

2.4 Load Combination for Wall Stability

This report deals with the external stability of the wall. For the external stability to be satisfied, the wall must be safe against three modes of failures: overturning, sliding and bearing. For retaining walls, the loads to be considered are: weight of the wall, dead earth load, lateral earth pressure and life load surcharge.

The selection of load factor combination will depend on the mode of failure to be analyzed. The load factor combination that results in the maximum vertical load controls the bearing capacity consideration. Load factor combinations that include minimum vertical loads and maximum horizontal loads control the sliding resistance as well as the overturning. Having the greatest net overturning moment produces the largest resultant eccentricity.

2.4.1 Cantilever Walls

For a cantilever wall, the earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of soil to the left of the vertical plane is considered as part of the wall weight. The resultant force makes an angle d with the perpendicular to the wall, where d is the friction angle between fill and wall.

Figure 2.1 shows the load factor and combination of a cantilever wall from AASHTO LRFD publication. In Fig. 2.1.a, the load factors for sliding and eccentricity are presented and in Fig. 2.1.b the load factors for bearing resistance are presented.

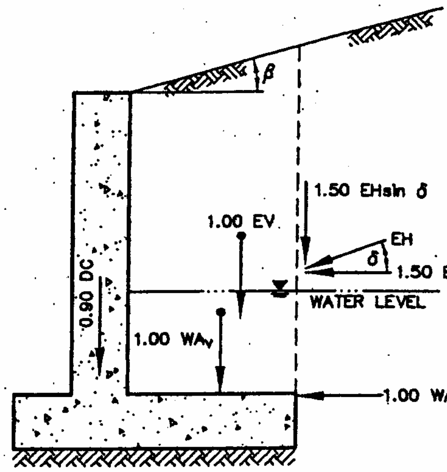


Figure C11.5.5-2 - Typical Application of Load Factors for Sliding and Eccentricity

(a)

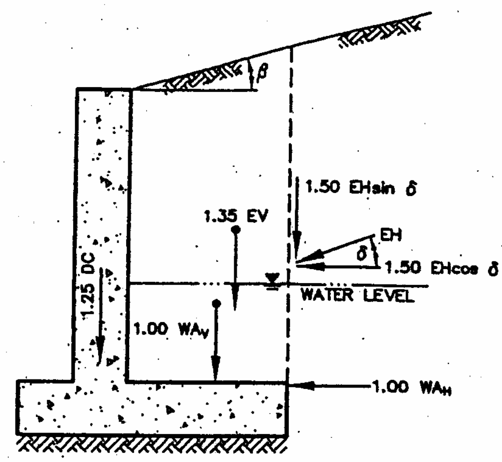


Figure C11.5.5-1 - Typical Application of Load Factors for Bearing Resistance

(b)

- Permanent Loads

DC	=	dead load of structural components and nonstructural attachments
DW	=	dead load of wearing surfaces and utilities
EH	=	horizontal earth pressure load
ES	=	earth surcharge load
EV	=	vertical pressure from dead load of earth fill

- Transient Loads

LS	=	live load surcharge
WA	=	water load and stream pressure

Fig. 2.1 Load factors and combinations for a retaining wall

2.4.2 Mechanically Stabilized Earth Walls-MSE Walls

The active earth pressure coefficients for retained backfill, i.e., fill behind the reinforced soil mass, for external stability calculations are computed with d = ß (article 11.10.5.2). Where ß is the slope angle of the backfill.

2.4.3 Life Load Surcharge

As stated in AASHTO, 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 backfill of the wall. The effect of the surcharge can be represented by an equivalent height of soils. In ASD, the height of soils was the same for any height of wall, at a height of 2 ft. Current AASHTO LRFD design defines the equivalent height of soils as a function of the height of the walls, as shown in Table 2.3. As stated in AASHTO, linear interpolation shall be used for intermediate wall heights. Figure 2.2 shows a typical application of live load surcharge in a) for a conventional structure, and, in b) for a MSE structure from AASHTO LRFD publication.

2.5 Resistance Consideration in Wall Stability

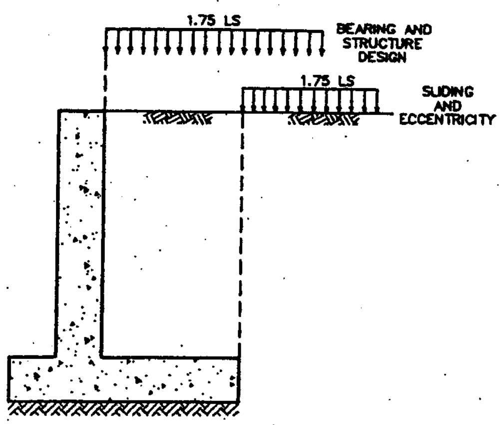
Bearing resistance shall be determined based on the highest anticipated position of the groundwater level. Because of the load eccentricity, a reduced effective width of the footing base will be used in determining the bearing capacity. The design bearing pressure on the effective width shall be assumed to be uniform.

For footings on soils, the eccentricity of the footing, evaluated based on factored loads, is less than 1/4 of the corresponding footing dimension. i.e; the location of the resultant of the reaction forces shall be within the middle one-half of the base width.

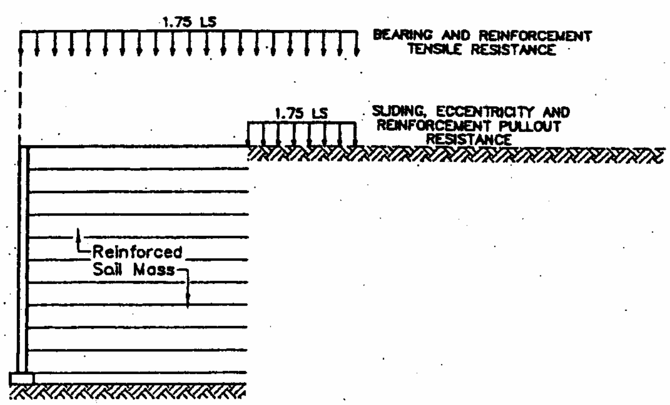
Table 2.3 Equivalent height of soil as a function of wall height

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	1.0 FT or Further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0



(a) CONVENTIONAL STRUCTURE



(b) MECHANICALLY STABILIZED EARTH STRUCTURE

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

Fig. 2.2 Typical application of live load surcharge

(The criteria for evaluating overturning in ASD requires that the eccentricity be less than 1/3 of the corresponding footing dimension).

CHAPTER III

CANTILEVER RETAINING WALL DESIGN

The Cantilever retaining wall in Fig. 3.1 is a State of Maryland Type A retaining wall section, Standard No. RW(6.03)-83-134. The wall will be backfilled with a free draining granular fill with f = 30° and ? = 110 pcf. The foundation soil has a f_f = 35° and $?_f$ = 120 pcf. Goetechnical design of the wall is undertaken by both the ASD and LRFD methods.

3.1 Allowable Stress Design (ASD)

3.1.1 Load Consideration for Geotechnical Design

(A) The Active Earth Pressure Coefficient (K_a)

$$K_a = \frac{\sin^2(q + j)}{\sin^2 q \sin(q - d)\left[1 + \sqrt{\dfrac{\sin(j + d)\sin(j - b)}{\sin(q - d)\sin(q + b)}}\right]^2}$$

For $j = 30°$ for the backfill soil

$q = 90°$ for a vertical wall and ß = 0 for a horizontal backfill

and assume $d = j = 30°$

$\sin^2(q + j) = \sin^2(90 + 30) = 0.75$

$\sin^2 q = \sin^2 90 = 1$

$\sin(q - d) = \sin(90 - 30) = 0.866$

$\sin(j + d) = \sin(30 + 30) = 0.866$

$\sin(j - b) = \sin(30 - 0) = 0.5$

$\sin(q - d) = \sin(90 - 30) = 0.866$

$\sin(q + b) = \sin(90 + 0) = 1.0$

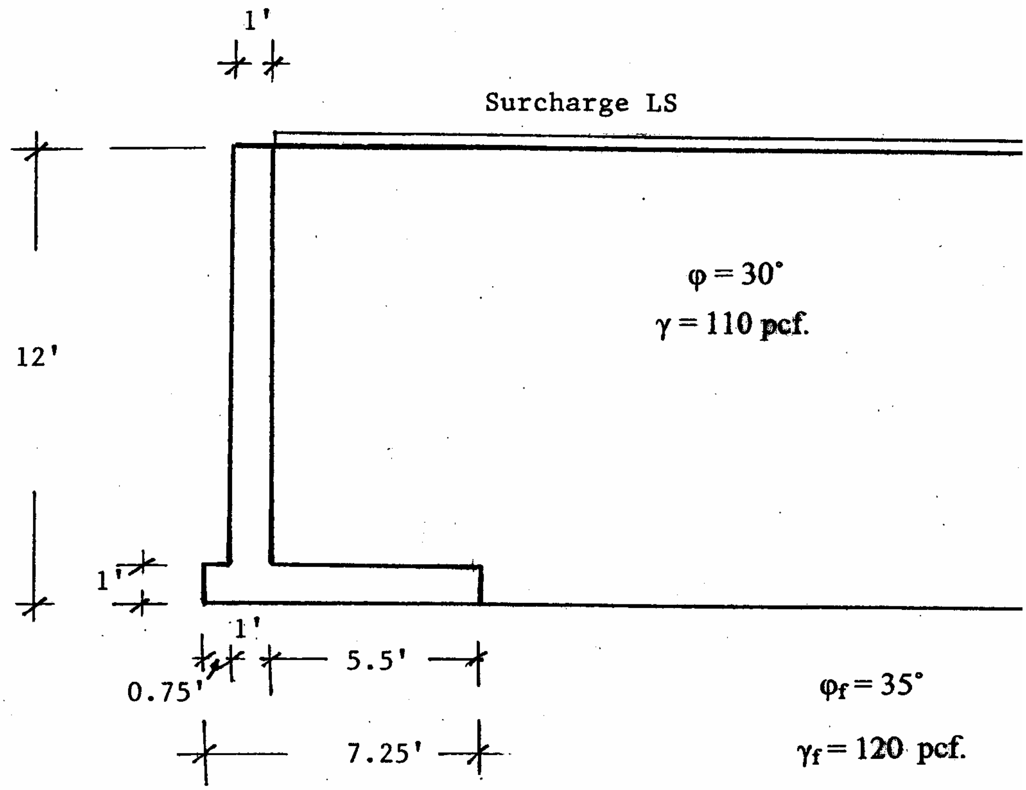


Fig. 3.1 Cantilever retaining wall analyzed

$$K_a = \frac{0.75}{1 \times 0.866\left[1+\sqrt{\frac{0.866 \times 0.5}{0.866 \times 1.0}}\right]^2}$$

$$= \frac{0.866}{[1+0.707]^2} = 0.297$$

(B) Dead Load of Structural Components (DC)

Referring to Fig. 3.1 and assuming a unit weight of concrete equal to 150 lb/ft^3.

DC_1 = 1 x 11 x 150 = 1,650 lb/ft

DC_3 = 1 x 7.25 x 150 = 1,088 lb/ft

(C) Live Load Surcharge (LS)

For 2 ft of soil surcharge and assuming $?_{soil}$ = 110 pcf

LS = 2 x 110 x 5.5 = 1210 lb/ft

Earth pressure due to surcharge

P_{LS} = 2 x 110 x 0.297 x 12 = 784.0 lb/ft

P_{LSV} = 784 sin d = 784 sin 30 = 392 lb/ft

P_{LSH} = 784 cos d = 784 cos 30 = 679 lb/ft

(D) Vertical Pressure from Dead Load of Earth Fill (EV)

EV = 5.5 x 11 x 110 = 6,655 lb/ft

(E) Lateral Earth Pressure (EH)

the active earth pressure is:

$$P_a = \frac{110 \times 12^2}{2} \times 0.297 = 2{,}352 \text{ lb/ft}$$

P_{av} = 2352 sin 30 = 1,176 lb/ft

P_{ah} = 2352 cos 30 = 2,037 lb/ft

(F) Summary of Loads and Moments

A summary of vertical loads and resisting moments is presented in Table 3.1, and of the horizontal loads and driving moments in Table 3.2.

Table 3.1 Vertical loads and resisting moments

Item	Force (V), lb	Moment arm, ft	Moment about toe, lb.ft
DC_1	1,650	1.25	2,063
DC_3	1,088	3.625	3,944
LS	1,210	4.5	5,445
EV	6,655	4.5	29,948
P_{LSV}	392	7.25	2,842
P_{av}	1,176	7.25	8,526
Total	12,171		52,768

Table 3.2 Horizontal loads and driving moments

Item	Force (H), lb	Moment arm, ft	Moment, lb.ft
P_{LSH}	679	6	4,074
P_{ah}	2,037	4	8,148
Total	2,716		12,222

3.1.2 External Stability

(A) Sliding Resistance

assuming the friction coefficient to be 2/3 tan j_f :

$$\text{F.S.} = \frac{12{,}171 \times {}^{2}\!/_{3} \times \tan 35}{2{,}716}$$

$$= \frac{5{,}681}{2{,}716} = 2.09 > 1.5 \text{ o.k.}$$

(B) Overturning Resistance

$$M_{net} = 52,768 - 12,222$$

$$= 40,546$$

$$X_0 = \frac{M_{net}}{V} = \frac{40,546}{12,171} = 3.33 \text{ ft}$$

$$e = \frac{B}{2} - X_0$$

$$= \frac{7.25}{2} - 3.33 = 0.295 \text{ ft}$$

$$\frac{B}{6} = \frac{7.25}{6} = 1.21 \qquad \text{i.e} \qquad e < \frac{B}{6} \text{ o.k.}$$

$$\text{F.S.} = \frac{52,768}{12,222} = 4.32 > 2 \text{ o.k.}$$

(C) Bearing Failure Resistance

Vertical stress,

$$s_v = \frac{V}{B - 2e}$$

$$= \frac{12,171}{7.25 - 2 \times 0.295}$$

$$= \frac{12,171}{6.66} = 1827 \text{ psf}$$

The nominal bearing resistance of cohesionless soil such as sands or gravel may be taken as (A10.6.3.1.2C)

q_{ult} = 0.5 ? B C_{w1} $N_{?m}$ + ? C_{w2} D_f N_{qm}

and $N_{?m}$ = $N_?$ $S_?$ $C_?$ $i_?$

N_{qm} = N_q S_q C_q i_q d_q

For a f of 35°, $N_?$ = 50 and N_q = 34

For no water table, C_{w1} = 1.0 and C_{w2} = 1.0

For f = 35°, $\frac{L}{B} > 10$, S_q = 1.0, $S_?$ = 1.0

For the pressure at the base of the footing

$$\frac{3 \times 120}{2{,}000} = 0.18 \text{ tsf}$$

use $C_?$ = C_q = 0.76

For H = 2,716 lb, V = 12,171 lb

i.e., $\frac{H}{V} = \frac{2{,}716}{12{,}171} = 0.223$

$i_?$ = 0.46, i_q = 0.60

For d_q use a value of 1.0

$B^\backslash$ = B – 2e = 7.25 – 2 x 0.295 = 6.66 ft

q_{ult} = 0.5 x 120 x 6.66 (50 x 1.0 x 0.76 x 0.46)

+ 110 x 1 x 3 x (34 x 1 x 0.76 x 0.6 x 1.0)

= 6,985 + 5,116

q_{ult} = 12,101 psf

F.S. = $\frac{12{,}101}{18{,}27} = 6.62 > 3$ o.k.

3.2 Load and Resistance Factor Design (LRFD)

Steps in design:

1. Calculation of the unfactored loads and resulting moments due to wall components, and earth pressures.
2. Selection of the load factors and load combinations controlling geotechnical design.
3. Calculation of the factored loads and moments by multiplying the unfactored loads and moments by the appropriate load factors and load combinations.
4. For sliding resistance, ensure that the sum of the factored lateral load components H_{total}, is less than or equal to the factored geotechnical lateral load resistance, Q_R.
5. For eccentricity (overturning), ensure that the factored resultant vertical load component is located within B/4 of the base centroid.
6. Bearing, ensure that the maximum bearing stress due to the factored load components ?q is less than or equal to the factored geotechnical bearing resistance, $f\, q_{ult}$.

3.2.1 Load Consideration for Geotechnical Design

(A) The Active Earth Pressure Coefficient (K_a)

same as for the ASD, equal to 0.297

(B) Dead Load of Structural Components (DC)

same as for the ASD

DC_1 = 1,650 lb/ft

DC_3 = 1,088 lb/ft

(C) Live Load Surcharge (LS)

from Table 3.11.6.4-2, for a wall of 12 ft, the equivalent height of surcharge is 3.2 ft.

LS = 3.2 x 110 x 5.5 = 1,936 lb/ft

Earth pressure due to surcharge

P_{LS} = 3.2 x 110 x 0.297 x 12 = 1,255 lb/ft

P_{LSV} = 1,255 sin 30 = 628 lb/ft

P_{LSH} = 1,255 cos 30 = 1,087 lb/ft

(D) Vertical Pressure from Dead Load of Earth Fill (EV) same as ASD,

EV = 6,655 lb/ft

(E) Earth Pressure (EH)

same as ASD,

P_{av} = 1,176 lb/ft

P_{ah} = 2,037 lb/ft

(F) Summary of Unfactored Loads and Moments

A summary of unfactored vertical loads and resisting moments is presented in Table 3.3, and of unfactored horizontal loads and driving moments in Table 3.4.

Table 3.3 Unfactored vertical loads and resisting moments

Item	Force (V)	Moment arm	Moment
DC_1	1,650	1.25	2,063
DC_3	1,088	3.625	3,944
EV	6,655	4.5	29,948
LS	1,936	4.5	8,712
P_{LSV}	628	7.25	4,553
P_{av}	1,176	7.25	8,526
Total	13,133		57,746

Table 3.4 Unfactored horizontal loads and driving moments

Item	Force (H)	Moment arm	Moment
P_{LSH}	1,087	6	6,522
P_{ah}	2,037	4	8,148

Total	3,124		14,670

3.2.2 Limit States and Load Factors

Strength I Limit State:

For sliding and overturning, minimum vertical loads and maximum horizontal loads (I-a) – the minimum load factors are used for those load components that contribute to the resistance (DC = 0.9 and EV = 1.0) and the maximum load factor is used for the driving force (EH = 1.5 and LS = 1.75). The live load surcharge, LS, is not applied over the heel of the wall for this case.

For bearing, maximum vertical loads (I-b) – the maximum load factors are used for all components of load for bearing (DC = 1.25, EV = 1.35, EH = 1.5 and LS = 1.75). LS is included over the heel of the wall for such an evaluation.

Strength IV Limit State:

For sliding and overturning, minimum vertical loads and maximum horizontal loads (IV-a) – will produce a case less critical then I-a since LS = 0 and DC = 1.5. Thus, no need to check such a case.

For bearing maximum vertical loads, (IV-b) – this case is to be checked and compared to strength (I-b) even though LS = 0 because the vertical load is a maximum when the factor for DC is 1.5.

Service I Limit State:

Settlement – all the applicable loads have a load factor of 1.00.

The limit states that need to be evaluated are shown in Figure 3.2. The applicable load factors are summarized in Table 3.5.

Table 3.5 Load factors

Group	$?_{DC}$	$?_{EV}$	$?_{LS}$	$?_{EH}$	Use
Strength I-a	0.9	1.0	1.75	1.5	Sliding and Eccentricity

Strength I-b	1.25	1.35	1.75	1.5	Bearing Capacity
Strength IV-a	1.5	1.0	-	1.5	Sliding and Eccentricity
Strength IV-b	1.5	1.35	-	1.5	Bearing Capacity
Service I	1.0	1.0	1.0	1.0	Settlement

3.2.3 Factored Loads and Factored Moments

Summary of factored loads and moments are presented in Tables 3.6, 3.7, 3.8 and 3.9.

Table 3.6 Factored vertical loads

Item	DC_1	DC_3	EV	LS	P_{LSV}	P_{av}	V_{total}
V (unfactored)	1,650	1,088	6,655	1,936	628	1,176	13,133
Strength I-a	1,485	979	6,655	3,388	1,099	1,764	15,370
Strength I-b	2,063	1,360	8,984	3,388	1,099	1,764	18,658
Strength IV-b	2,475	1,632	8,984	-	-	1,764	14,855
Service I	1,650	1,088	6,655	1,936	628	1,176	13,133

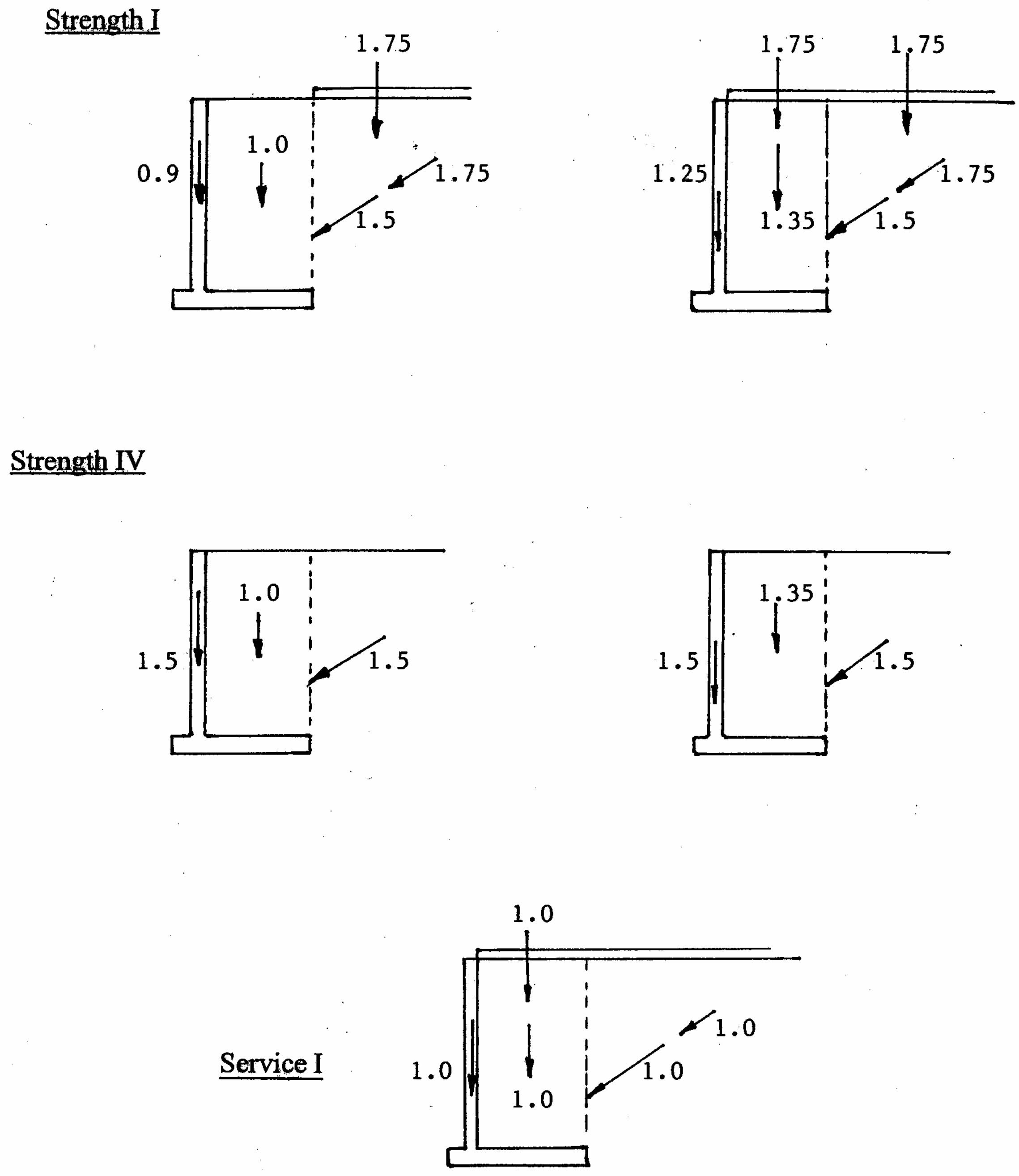


Fig. 3.2 Limit states analyzed for cantilever wall

Table 3.7 Factored horizontal loads

Item	P_{LSH}	P_{ah}	H_{total}
H (unfactored)	1,087	2,037	3,124
Strength I-a	1,902	3,055	4,957
Strength I-b	1,902	3,055	4,957
Strength IV-b	-	3,055	3,055
Service I	1,087	2,037	3,124

Table 3.8 Factored moments from vertical forces M_V

Item	DC_1	DC_3	EV	LS	P_{LSV}	P_{av}	$M_{V(total)}$
M_V (unfactored)	2,063	3,944	29,948	8,712	4,553	8,526	57,746
Strength I-a	1,857	3,550	29,948	15,246	7,968	12,789	71,358
Strength I-b	2,579	4,930	40,430	15,246	7,968	12,789	83,942
Strength IV-b	3,095	5,916	40,430	-	-	12,789	62,230
Service I	2,063	3,944	29,948	8,712	4,553	8,526	57,746

Table 3.9 Factored moments from horizontal forces M_h

Item	P_{LSH}	P_{ah}	$M_{h(total)}$
M_h (unfactored)	6,522	8,148	14,670
Strength I-a	11,414	12,222	23,636
Strength I-b	11,414	12,222	23,636
Strength IV-b	-	12,222	12,222
Service I	6,522	8,148	14,670

3.2.4 External Stability

(A) Sliding Resistance

The force due to live load surcharge (LS) over the heel is not included in the sliding evaluation.

The factored resistance, Q_R, against failure by sliding is

$Q_R = f_T \cdot Q_T$

where f_T = resistance factor for shear resistance between soil and foundation specified in Table 10.5.5-1. For concrete cast-in-place on sand f_T = 0.8.

Q_T = nominal shear resistance between soil and foundation, which is equal to V tan d, where V is the vertical force and tan d = tan f_f for concrete cast against soil.

i.e., Q_R = 0.8 V tan f_f

= 0.8 V tan 35

= 0.56 V

Table 3.10 Sliding resistance for the retaining wall

Item	V_{total}	Q_R	H_{total}
Strength I-a	11,982	6,710	4,957
Strength I-b	15,270	8,551	4,957
Strength IV-b	14,855	8,319	3,055
Service I	11,197	6,270	3,124

Because the factored sliding resistance, Q_R, is greater than the factored horizontal loading, H_{total}, the sliding resistance is satisfactory.

(B) Eccentricity (overturning)

The eccentricity of the returning wall is checked by comparing the calculated eccentricity, e, for each loading group to the maximum allowed eccentricity e_{max}. The force and moment due to live load surcharge over the heel are not included in the eccentricity (i.e., overturning) evaluation.

$$X_0 = \frac{M_v - M_h}{V} \qquad \text{(location of the resultant from the toe)}$$

$$e = \frac{B}{2} - X_0$$

$$= \frac{7.25}{2} - X_0 = 3.625 - X_o$$

$$e_{max} = \frac{B}{4} = \frac{7.25}{4} = 1.813 \text{ ft}$$

Table 3.11 Eccentricity for the retaining wall

Item	V	M_v	M_h	X_0	e	e_{max}
Strength I-a	11,982	56,112	23,636	2.71	0.915	1.813
Strength I-b	15,270	68,696	23,636	2.95	0.674	1.813
Strength IV-b	14,855	62,230	12,222	3.37	0.259	1.813
Service I	11,197	49,034	14,670	3.07	0.556	1.813

For all cases, $e < e_{max}$, i.e., the design is adequate in regard to eccentricity.

(C) Bearing Resistance

(C.1) Factored uniform Bearing Stress ?q

The adequacy for bearing capacity is developed based on a rectangular distribution of soil pressure, q, over the reduced effective area of the footing. The force and moment due to live load surcharge over the heel are included in the bearing resistance evaluation.

$$B^{\backslash} = 2\left(\frac{B}{2} - e\right)$$

$$= \mathrm{B} - 2\mathrm{e}$$

$$X_0 = \left(\frac{M_v - M_h}{V}\right)$$

$$e = \frac{B}{2} - X_0$$

i.e., $$B^{\backslash} = B - 2\left(\frac{B}{2} - X_0\right)$$

$$B^{\backslash} = 2X_0$$

The maximum factored uniform bearing stress $gq = \dfrac{V}{L^{\backslash}B^{\backslash}}$

Since $L^{\backslash}$ = 1 ft (i.e., unit length of the wall) then,

$$?\mathrm{q} = \frac{V}{1 \times 2X_0} = \frac{V}{2X_0}$$

Table 3.12 Bearing stress for the retaining wall

Item	V	M_v	M_h	X_0	gq
Strength I-a	15,370	71,358	23,636	3.10	2,479
Strength I-b	18,658	83,942	23,636	3.23	2,888
Strength IV-b	14,855	62,230	12,222	3.37	2,204
Service I	13,133	57,746	14,670	3.28	2,002

(C.2) Factored Bearing Resistance

The factored bearing resistance, q_R, is determined from:

$$q_R = f q_{ult}$$

where f = resistance factor. From Table 10.5.5-1, using the rational method and estimating the friction angle from SPT data, the resistance factor f is equal to 0.35.

q_{ult} = nominal bearing resistance

i.e., $q_R = 0.35\ q_{ult}$

The nominal bearing resistance of cohesionless soil such as sands or gravels, may be taken as (A10.6.3.1.2C)

$q_{ult} = 0.5\ ?\ B\ C_{w1}\ N_{?m} + ?\ C_{w2}\ D_f\ N_{qm}$

and $N_{?m} = N_?\ S_?\ C_?\ i_?$

$N_{qm} = N_q\ S_q\ C_q\ i_q\ d_q$

For a f of 35°, $N_? = 50$ and $N_q = 34$

For no water table, $C_{w1} = 1.0$ and $C_{w2} = 1.0$

For f = 35°, $\frac{L}{B} > 10$, $S_q = 1.0$, $S_? = 1.0$

For the pressure at the base of the footing

$$\frac{3 \times 120}{2000} = 0.18 \text{ tsf}$$

use $C_? = C_q = 0.76$

For H = 3,124 lb, V = 13,133 lb

i.e., $\frac{H}{V} = \frac{3{,}124}{13{,}133} = 0.23$

$i_? = 0.46$, $i_q = 0.60$

For d_q use a value of 1.0

$B^{\backslash} = B - 2e = 7.25 - 2 \times 0.295 = 6.66$ ft

q_{ult} = 0.5 x 120 x 6.66 (50 x 1.0 x 0.76 x 0.46)

+ 110 x 1 x 3 x (34 x 1 x 0.76 x 0.6 x 1.0)

= 6,985 + 5,116

= 12,101 psf

q_R = 0.35 x 12,101 = 4,235 psf

Because the factored bearing resistance q_R exceeds the maximum factored uniform bearing stress, gq = 2888 psf, the bearing resistance is adequate.

3.3 Summary of the ASD and LRFD for the Cantilever Retaining Wall

The results of the analysis for both the ASD and LRFD are summarized in Table 3.13.

Table 3.13 Summary of cantilever wall design by ASD and LRFD

Performance Limit	ASD		LRFD	
	Required F.S./ Eccentricity	Actual	Factored Resistance	Factored Loading
Eccentricity	$e = \frac{B}{6} < 1.21$ (F.S. > 2)	e = 0.295 (F.S. = 4.32)	$e = \frac{B}{4} < 1.813$	e = 0.915
Sliding Resistance	F.S. > 1.5	F.S. = 2.09	6,710 lb/ft	4,957 lb/ft
Bearing Resistance	F.S. > 3	F.S. = 6.62	4,235 psf	2,888 psf

As was expected, both the LRFD and ASD produce an acceptable design for the wall.

CHAPTER IV

CRIB RETAINING WALL DESIGN

The crib retaining wall in Figure 4.1 is a state of Maryland Type A retaining wall section, Standard No. RW(6.01)-79-18. The wall is to be backfilled with a free draining granular fill. The unit weight of the soil and the concrete members, $?_{s+c}$ = 120 pcf. The backfill soil has a unit weight $?_b$ = 110 pcf and f_b = 30°. The foundation soil has a f_f = 30°. Geotechnical design of the wall is undertaken by both the ASD and LRFD methods.

4.1 Allowable Stress Design (ASD)

4.1.1 Load Consideration for Geotechnical Design

(A) The Active Earth Pressure Coefficient (K_a)

$$K_a = \frac{\sin^2(q + j)}{\sin^2 q \sin(q - d)\left[1 + \sqrt{\frac{\sin(j + d)\sin(j - b)}{\sin(q - d)\sin(q + b)}}\right]^2}$$

let $q^{\backslash}$ be the crib tilt, then

$\tan q^{\backslash} = \frac{2}{12}$, thus $q^{\backslash} = 9.46°$

let q be the crib angle with the horizontal, then

$q = 90 + q^{\backslash}$

$q = 90 + 9.46 = 99.46°$

let b be the slope angle with the horizontal, then

$\tan b = \frac{1}{2}$ $\qquad b = 26.56°$

For $j_b = 30°$ for the backfill soils and

assume $d = \frac{2}{3} \times 30 = 20°$

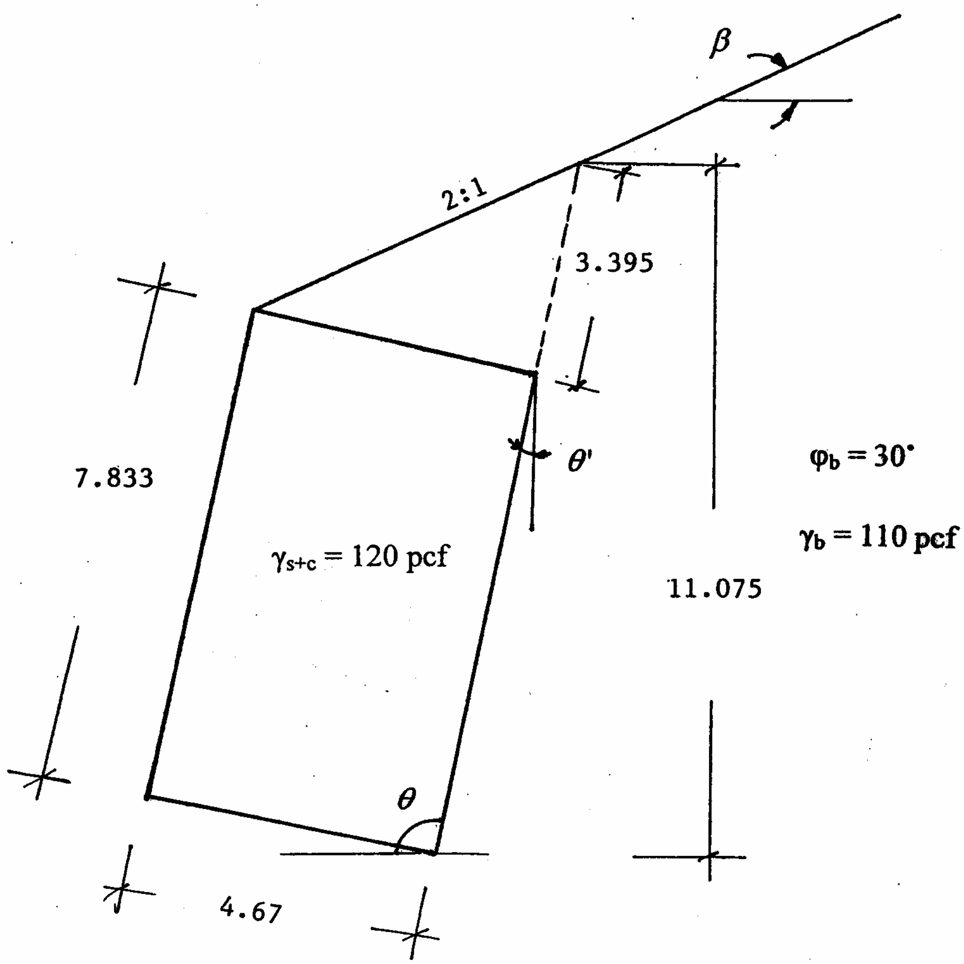


Fig. 4.1 Crib retaining wall analyzed

$$\sin^2(q + j) = \sin^2(99.46° + 30°) = 0.596$$

$$\sin^2 q = \sin^2 99.46 = 0.973$$

$$\sin(q - d) = \sin(99.46 - 20) = 0.983$$

$$\sin(j + d) = \sin(30 + 20) = 0.766$$

$$\sin(j - b) = \sin(30 - 26.56) = 0.06$$

$$\sin(q - d) = \sin(99.46 - 20) = 0.983$$

$$\sin(q + b) = \sin(99.46 + 26.56) = 0.809$$

$$K_a = \frac{0.596}{0.973 \times 0.983\left[1 + \sqrt{\dfrac{0.766 \times 0.06}{0.983 \times 0.809}}\right]^2}$$

$$= \frac{0.623}{[1 + 0.240]^2} = 0.405$$

(B) Dead Load of Wall (DC)

Referring to Figure 4.1 and assuming an average unit weight of the soil and the concrete members, $?_{s+c}$, equal to 120 lb/ft^3.

W = 4.67 x 7.833 x 120 = 4,390 lb/ft

W_x = W sin $q^{\backslash}$ = 4,390 sin 9.46 = 722 lb/ft

W_y = W cos $q^{\backslash}$ = 4,390 cos 9.46 = 4,330 lb/ft

(C) Vertical Pressure from Dead Load of Earth Fill (EV)

assuming $?_b$ = 110 lb/ft^3

EV = ½ x 4.67 x 3.395 x 110 = 872 lb/ft

EV_x = EV sin $q^{\backslash}$ = 872 sin 9.46 = 143 lb/ft

EV_y = EV cos $q^{\backslash}$ = 872 cos 9.46 = 860 lb/ft

(D) Lateral Earth Pressure (EH)

For a height of 11.075 ft, K_a = 0.405 and $?_b$ = 110 pcf

P_A = ½ x 110 x 11.075^2 x 0.405 = 2,732 lb/ft

$P_{AX} = P_A$ cos d = 2,732 cos 20 = 2,567 lb/ft

$P_{AY} = P_A$ sin d = 2,732 sin 20 = 934 lb/ft

(E) Summary of Loads and Moments

A summary of vertical loads and resisting moments is presented in Table 4.1, and of the horizontal loads and driving moments in Table 4.2.

Table 4.1 Vertical loads and resisting moments

Item	Force, lb	Moment arm, ft	Moment, lb.ft
W_y	4,330	2.335	10,111
EV_y	860	3.113	2,677
P_{AY}	934	4.67	4,362
Total	6,124		17,150

Table 4.2 Horizontal loads and driving moments

Item	Force, lb	Moment arm, ft	Moment, lb.ft
P_{Ax}	2,567	$\frac{11.075}{3\cos q} = 3.743$	9,608
$-W_x$	-722	3.917	-2,828
$-EV_x$	-143	8.965	-1,282
Total	1,702		5,498

4.1.2 External Stability

(A) Sliding Resistance

assuming the friction coefficient to be the smallest of tan j_b and tan j_f;

$$\text{F.S.} = \frac{6{,}124 \tan j_f}{1{,}702}$$

$$= 2.08 \quad > 1.5 \text{ o.k.}$$

(B) Overturning Resistance

M_{net} = 17,150 – 5,498

= 11,652

$$X_0 = \frac{M_{net}}{V} = \frac{11{,}652}{6{,}124} = 1.903$$

$$e = \frac{B}{2} - X_0$$

$$= \frac{4.62}{2} - 1.903 = 0.432$$

$$\frac{B}{6} = \frac{4.67}{6} = 0.778 \qquad \text{i.e} \qquad e < \frac{B}{6} \text{ o.k.}$$

$$\text{F.S.} = \frac{17{,}150}{5{,}498} = 3.12 \; > 2.0 \text{ o.k.}$$

(C) Bearing Resistance

$$s_y = \frac{V}{B - 2e}$$

$$= \frac{6{,}124}{4.67 - 2 \times 0.432}$$

$$= \frac{6{,}124}{3.806} = 1{,}609 \text{ psf}$$

The nominal bearing resistance of cohesionless soil, such as sands or gravels, based on SPT results was calculated from AASHTO equation (10.6.3.1.3b-1)

$$q_{ult} = \frac{N \times B}{10}\left(C_{w1} + C_{w2}\frac{D}{B}\right)R_i \qquad \text{in TSF}$$

For $\frac{H}{V} = \frac{1{,}702}{6{,}124} = 0.28, R_i = 0.56$

assuming N = 12

For no water table, $C_{w1} = C_{w2} = 1.0$

$$q_{ult} = \frac{12 \times 4.67}{10}\left(1 + 1 \times \frac{3}{4.67}\right) \times 0.56$$

$= 10{,}308$ psf

$$\text{F.S.} = \frac{10{,}308}{1{,}609} = 6.4 > 3 \quad \text{o.k.}$$

4.2 Load and Resistance Factor Design (LRFD)

Steps in design:

1. Calculation of the unfactored loads and resulting moments due to wall components, and earth pressures.
2. Selection of the load factors and load combinations controlling geotechnical design.
3. Calculation of the factored loads and moments by multiplying the unfactored loads and moments by the appropriate load factors and load combinations.
4. For sliding resistance, ensure that the sum of the factored lateral load components H_{total}, is less than or equal to the factored geotechnical lateral load resistance, Q_R.
5. For eccentricity (overturning), ensure that the factored resultant vertical load component is located within B/4 of the base centroid.
6. For bearing, ensure that the maximum bearing stress due to the factored load components, ?q, is less than or equal to the factored geotechnical bearing resistance, $f\, q_{ult}$.

4.2.1 Load Consideration for Geotechnical Design

(A) The Active Earth Pressure Coefficient (K_a)

same as for the ASD, equal to 0.405

(B) Dead Load of Structural Components (DC)

same as for the ASD

$W = 4390$ lb/ft

$W_x = 722$ lb/ft

$W_y = 4330$ lb/ft

(C) Vertical Pressure from Dead Load of Earth Fill (EV)

same as ASD,

$EV = 872$ lb/ft

$EV_x = 143$ lb/ft

$EV_y = 860$ lb/ft

(D) Earth Pressure (EH)

same as ASD,

$P_A = 2{,}732$ lb/ft

$P_{Ax} = 2{,}567$ lb/ft

$P_{Ay} = 934$ lb/ft

(E) Summary of Unfactored Loads and Moments

A summary of unfactored vertical loads and resisting moments is presented in Table 4.3, and of unfactored horizontal loads and driving moments in Table 4.4.

Table 4.3 Unfactored vertical loads and resisting moments

Item	Force, lb	Moment arm, ft	Moment, lb.ft
W_y	4,330	2.335	10,111
EV_y	860	3.113	2,677
P_{Ay}	934	4.67	4,362
Total	6,124		17,150

Table 4.4 Unfactored horizontal loads and driving moments

Item	Force	Moment arm	Moment
P_{Ax}	2,567	3.743	9,608
$-W_x$	-722	3.917	-2,828
$-EV_x$	-143	8.965	-1,282
Total	1,702		5,498

4.2.2 Limit States and Load Factors

Strength I Limit State:

For sliding and overturning, minimum vertical loads and maximum horizontal loads (I-a) – the minimum load factors are used for those load components thah contribute to the resistance (DC = 0.9 and EV = 1.0) and the maximum load factor is used for the driving force (EH = 1.5).

For bearing, maximum vertical loads (I-b) – the maximum load factors are used for all components of load for bearing (DC = 1.25, EV = 1.35, and EH = 1.5).

Strength IV Limit State:

For sliding and overturning, minimum vertical loads and maximum horizontal loads (IV-a) – this is the same case as (I-a) however since DC = 1.5 it is not as critical.

For bearing, maximum vertical loads (IV-b) – this case will have DC = 1.5, EV = 1.35 and EH = 1.5, thus will be more critical than (I-b).

Service I Limit State:

Settlement – all the applicable loads have a load factor of 1.00.

The limit states that need to be evaluated are shown in Fig. 4.2. The applicable load combinations and load factors are summarized in Table 4.5.

Table 4.5 Load factors

Group	$?_{DC}$	$?_{EV}$	$?_{EH}$	Use
Strength I-a	0.9	1.0	1.5	Sliding and Eccentricity
Strength I-b	1.25	1.35	1.5	Bearing Capacity
Strength IV-a	1.5	1.0	1.5	Sliding and Eccentricity
Strength IV-b	1.5	1.35	1.5	Bearing Capacity
Service I	1.0	1.0	1.0	Settlement

4.2.3 Factored Loads and Factored Moments

A summary of factored loads and moments is presented in Tables 4.6, 4.7, 4.8, and 4.9.

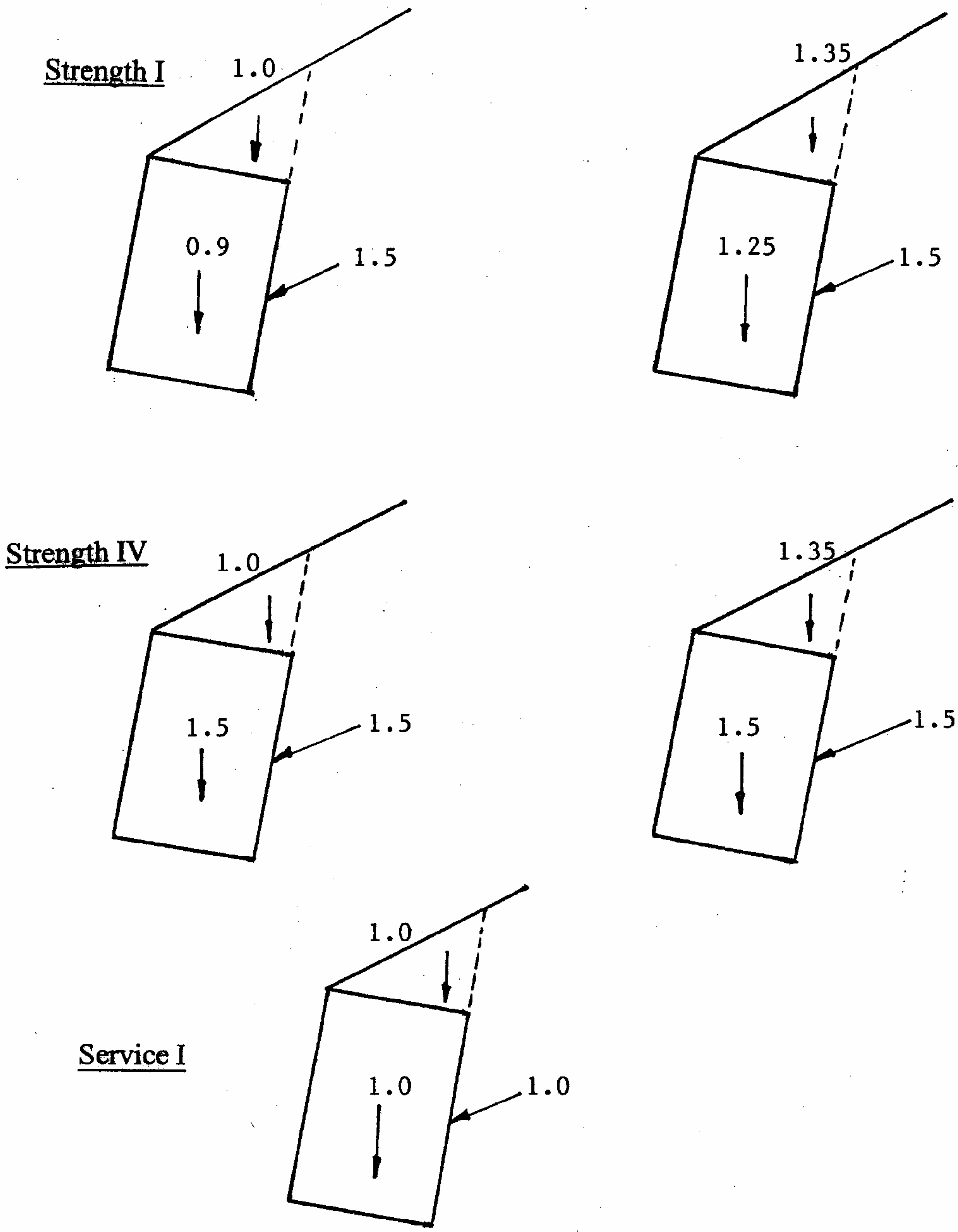


Fig. 4.2 Limit states analyzed for crib wall

Table 4.6 Factored vertical loads

Item	W_y	EV_y	P_{Ay}	V_{total}
V (unfactored)	4,330	860	934	6,124
Strength I-a	3,897	860	1,401	6,158
Strength IV-b	6,495	1,161	1,401	9,057
Service I	4,330	860	934	6,124

Table 4.7 Factored horizontal loads

Item	P_{Ax}	$-W_x$	$-EV_x$	H_{total}
H (unfactored)	2,567	-722	-143	1,702
Strength I-a	3,850	-650	-143	3,057
Strength IV-b	3,850	-1,083	-193	2,574
Service I	2,567	-722	-143	1,702

Table 4.8 Factored moments from vertical forces M_V

Item	W_y	EV_y	P_{Ay}	$M_{V(total)}$
M_V (unfactored)	10,111	2,677	4,362	17,150
Strength I-a	9,100	2,677	6,543	18,320
Strength IV-b	15,167	3,614	6,543	25,324
Service I	10,111	2,677	4,362	17,150

Table 4.9 Factored moments from horizontal forces M_h

Item	P_{Ax}	$-W_x$	$-EV_x$	$M_{h(total)}$
M_h (unfactored)	9,608	-2,828	-1,282	5,498
Strength I-a	14,412	-2,545	-1,282	10,585
Strength IV-b	14,412	-4,242	-1,731	8,439
Service I	9,608	-2,828	-1,282	5,498

4.2.4 External Stability

(A) Sliding Resistance

The factored resistance against failure by sliding, Q_R, is

$Q_R = f_T \cdot Q_T$

where f_T = resistance factor for sliding of soil and against soil. From

Table 10.5.5-1, f_T = 1.0.

Q_T = nominal shear resistance between soil and foundation, which is equal

to V tan d, where V is the vertical force and tan d is the lesser of tan f_b or

tan f_f

i.e., $Q_R = V \tan j_f$

$= V \tan 30$

$= 0.577 V$

Table 4.10 Sliding resistance for the wall

Item	V_{total}	Q_R	H_{total}
Strength I-a	6,158	3,553	3,057
Strength IV-b	9,057	5,226	2,574
Service I	6,124	3,534	1,702

Because the factored sliding resistance, Q_R, is greater than the factored horizontal loading, H_{total}, the sliding resistance is satisfactory.

(B) Eccentricity

X_0 = location of the resultant from toe of wall = $\dfrac{M_v - M_h}{V}$

$$e = \text{ eccentricity } = \frac{B}{2} - X_0$$

$$= \frac{4.67}{2} - X_0$$
$$= 2.335 - X_o$$

The location of the resultant must be in the middle half of the base

$$e_{max} = \frac{B}{4} = \frac{4.67}{4} = 1.168 \text{ ft}$$

Table 4.11 Eccentricity for the wall

Item	V	M_v	M_h	X_0	e	e_{max}
Strength I-a	6,158	18,320	10,585	1.256	1.079	1.168
Strength IV-b	9,057	25,324	8,439	1.864	0.471	1.168
Service I	6,124	17,150	5,498	1.903	0.432	1.168

for all cases, $e < e_{max}$, i.e., the design is adequate in regard to eccentricity.

(C) Bearing Resistance

(C.1) Factored Uniform Bearing Stress ?q

$$B^{\backslash} = B - 2e$$

$$e = \frac{B}{2} - X_0$$

i.e., $B^{\backslash} = 2X_0$

The maximum factored uniform bearing stress $gq = \frac{V}{L^{\backslash}B^{\backslash}}$

Since $L^{\backslash}$ = 1 ft (i.e., unit length of the wall) then

$$?q = \frac{V}{1 \times 2X_0} = \frac{V}{2X_0}$$

Table 4.12 Bearing stress for the wall

Item	V	X_0	gq
Strength I-a	6,158	1.256	2,451
Strength IV-b	9,057	1.864	2,429
Service I	6,124	1.903	1,609

(C.2) Factored Bearing Resistance

The factored bearing resistance, q_R, is determined from $q_R = f\, q_{ult}$

where f = resistance factor. From Table 10.5.5-1 based on an semiempirical procedure using SPT data, the resistance factor is 0.45. Again;

$$q_{ult} = \frac{N \times B}{10}\left(C_{w1} + C_{w2}\frac{D}{B}\right)R_i \qquad \text{in TSF}$$

For $\frac{H}{V} = \frac{1,702}{6,124} = 0.28, R_i = 0.56$

assuming N = 12

For no water table, $C_{w1} = C_{w2} = 1.0$

$$q_{ult} = \frac{12 \times 4.67}{10}\left(1 + 1 \times \frac{3}{4.67}\right) \times 0.56$$

= 10,308 psf

$q_R = 0.45 \times 10{,}308 = 4{,}639$ psf

Because the factored bearing resistance q_R, exceeds the maximum factored uniform bearing stress, ?q = 2451, the bearing resistance is adequate.

4.3 Summary of the ASD and LRFD for the Crib Wall

The results of the analysis for both ASD and LRFD are summarized in Table 4.13.

Table 4.13 Summary of crib wall design by ASD and LRFD

Performance Limit	ASD		LRFD	
	Required F.S./ Eccentricity	Actual	Factored Resistance	Factored Loading
Eccentricity	$e = \frac{B}{6} < 0.778$ F.S. > 2	e = 0.432 (F.S. = 3.12)	$e = \frac{B}{4} < 1.168$	e = 1.079
Sliding Resistance	F.S. > 1.5	F.S. = 2.08	3,553 lb/ft	3,057 lb/ft
Bearing Resistance	F.S. > 3	F.S. = 6.4	4,639 psf	2,451 psf

Both the LRFD and ASD produce an acceptable design for the wall.

CHAPTER V

MECHANICALLY STABILIZED EARTH (MSE) WALL DESIGN

The retaining wall shown in Fig 5.1 is an example of an MSE wall with a geogrid reinforcement. The wall is to be backfilled with a free draining granular fill with a $f_b = 30°$ and $?_b$ = 110 pcf. The foundation soil has a $f_f = 35°$ and $?_f$ = 120 pcf and the reinforced wall has a f_r = 30° and $?_r$ = 110 pcf. Goetechnical design of the wall is undertaken by both the ASD and LRFD methods.

5.1 Allowable Stress Design (ASD)

5.1.1 Load Consideration for Geotechnical Design

(A) The Active Earth Pressure Coefficient (K_a)

$$K_a = \frac{\sin^2(q + j)}{\sin^2 q \sin(q - d)\left[1 + \sqrt{\frac{\sin(j + d)\sin(j - b)}{\sin(q - d)\sin(q + b)}}\right]^2}$$

$j = 30°$ for the backfill soil

$q = 90°$ for a vertical wall and $b = 15°$ for the sloping backfill

and $d = b = 15°$ (AASHTO 11.10.5.2)

$\sin^2(q + j) = \sin^2(90 + 30) = 0.75$

$\sin^2 q = \sin^2 90 = 1$

$\sin(q - d) = \sin(90 - 15) = 0.966$

$\sin(j + d) = \sin(30 + 15) = 0.707$

$\sin(j - b) = \sin(30 - 15) = 0.259$

$\sin(q - d) = \sin(90 - 15) = 0.966$

$\sin(q + b) = \sin(90 + 15) = 0.966$

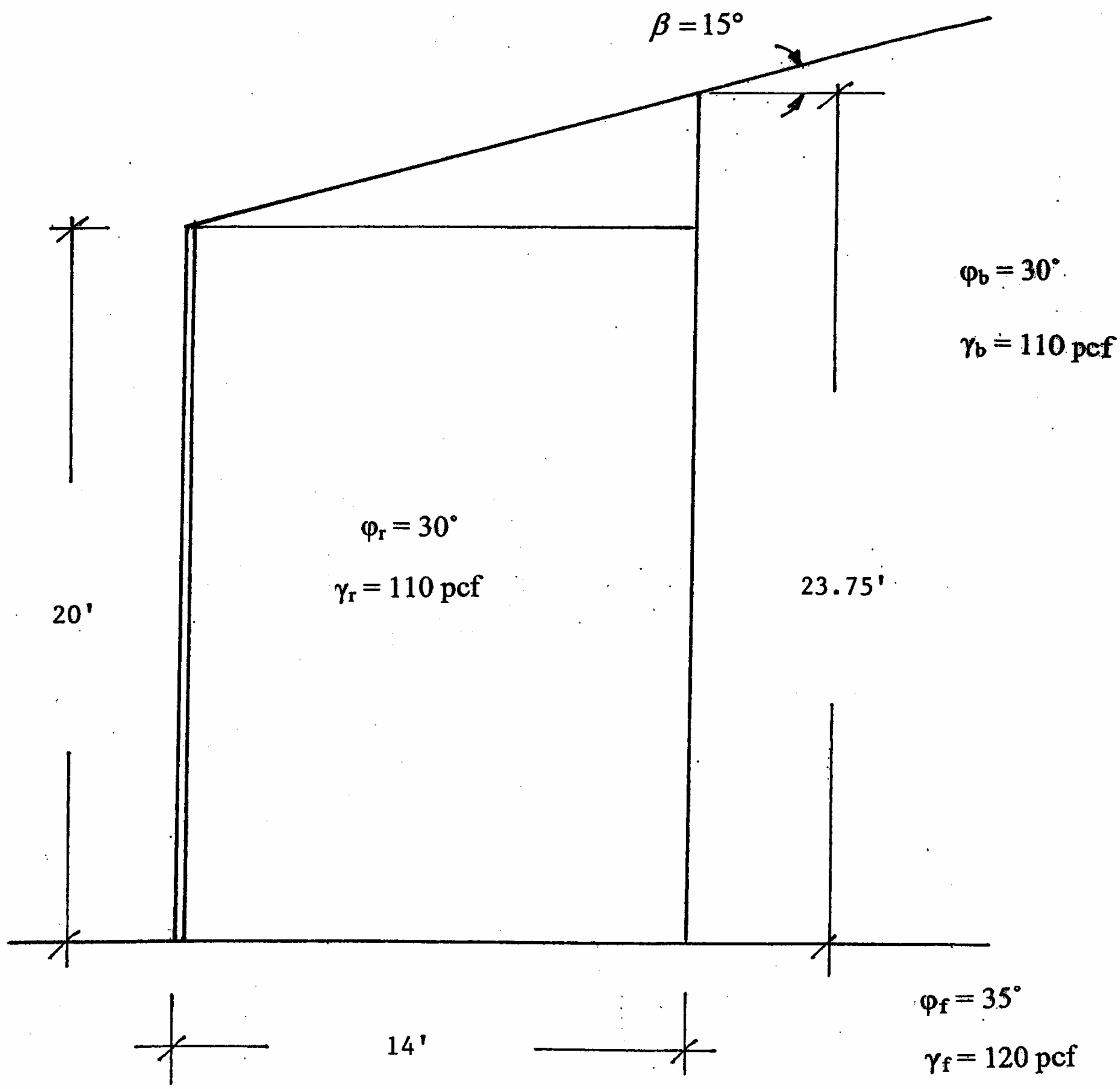


Fig. 5.1 Mechanically stabilized earth (MSE) wall analyzed

$$K_a = \frac{0.75}{1 \times 0.966\left[1 + \sqrt{\frac{0.707 \times 0.259}{0.966 \times 0.966}}\right]^2}$$

$$= \frac{0.776}{[1 + 0.443]^2} = 0.373$$

(B) Vertical Pressure from Earth Fill (EV)

Assuming the unit weight of the reinforced soil $?_r$ to be 110 lb/ft^3, the weight of the reinforced soil is:

EV_1 = 14 x 20 x 110 = 30,800 lb/ft

EV_2 = ½ x 3.75 x 14 x 110 = 2,888 lb/ft

(C) Lateral Earth Pressure (EH)

For a height of 23.75 ft, K_a = 0.373 and $?_b$ = 110 pcf, the active earth pressure is:

P_A = ½ x 110 x 23.75^2 x 0.373 = 11,572 lb/ft

P_{Ax} = P_A cos ß = 11572 x 0.966 = 11,178 lb/ft

P_{Ay} = P_A sin ß = 11572 x 0.259 = 2,995 lb/ft

(D) Summary of Loads and Moments

A summary of vertical loads and resisting moments is presented in Table 5.1, and of horizontal loads and driving moments in Table 5.2.

Table 5.1 Vertical loads and resisting moments

Item	Force, lb	Moment arm, ft	Moment, lb.ft
EV_1	30,800	7	215,600
EV_2	2,888	9.333	26,955
P_{Ay}	2,995	14	41,930
Total	36,683		284,485

Table 5.2 Horizontal loads and driving moments

Item	Force, lb	Moment arm, ft	Moment, lb.ft
P_{Ax}	11,178	$\frac{23.75}{3}$	88,493

5.1.2 External Stability

(A) Sliding Resistance

assuming the friction coefficient to be the smallest of tan f_r and tan f_f;

$$\text{F.S.} = \frac{36{,}683 \tan 30}{11{,}178}$$

$$= \frac{21{,}179}{11{,}178} = 1.89 \quad > 1.5 \text{ o.k.}$$

(B) Overturning Resistance

$$M_{net} = 284{,}485 - 88{,}493$$

$$= 195{,}992$$

$$X_0 = \frac{M_{net}}{V} = \frac{195{,}992}{36{,}683} = 5.343 \text{ ft}$$

$$e = \frac{B}{2} - X_0$$

$$= \frac{14}{2} - 5.343 = 1.657 \text{ ft}$$

$$\frac{B}{6} = \frac{14}{6} = 2.333 \qquad \text{i.e.,} \qquad e < \frac{B}{6} \text{ o.k.}$$

$$\text{F.S.} = \frac{284{,}485}{88{,}493}$$

$$= 3.215 \qquad > 2 \text{ o.k.}$$

(C) Bearing Failure Resistance

Vertical stress, $s_v = \dfrac{V}{B - 2e}$

$$= \frac{36{,}683}{14 - 2 \times 1.657}$$

s_v = 3,432 psf

The nominal bearing resistance of cohesionless soil such as sands or gravels based on SPT results was calculated from AASHTO equation (10.6.3.1.3b-1).

$$q_{ult} = \frac{N \times B}{10}\left(C_{w1} + C_{w2}\frac{D}{B}\right)R_i \qquad \text{in TSF}$$

For $\dfrac{H}{V} = \dfrac{11{,}178}{36{,}683} = 0.3, R_i = 0.52$

assuming N = 12

For no water table, $C_{w1} = C_{w2} = 1.0$

$$q_{ult} = \frac{12 \times 14}{10}\left(1 + 1 \times \frac{3}{14}\right) \times 0.52$$

$$= 21{,}216 \text{ psf}$$

$$\text{F.S.} = \frac{21{,}216}{3{,}432} = 6.18 > 3 \quad \text{o.k.}$$

5.2 Load and Resistance Factor Design (LRFD)

Steps In Design:

1. Calculation of the unfactored loads and resulting moments due to wall components and earth pressures.
2. Selection of the load factors and load combinations controlling geotechnical design.
3. Calculation of the factored loads and moments by multiplying the unfactored loads and moments by the appropriate load factors and load combinations.
4. For sliding resistance, ensure that the sum of the factored lateral load components H_{total}, is less than or equal to the factored geotechnical lateral load resistance, Q_R.
5. For eccentricity (overturning), ensure that the factored resultant vertical load component is located within B/4 of the base centroid.
6. For bearing, ensure that the maximum bearing stress due to the factored load components ?q, is less than or equal to the factored geotechnical bearing resistance, $f\ q_{ult}$.

5.2.1 Load Consideration for Geotechnical Design

(A) The Active Earth Pressure Coefficient (K_a)

same as for the ASD, equal to 0.373

(B) Vertical Pressure from Earth Fill (EV)

same as ASD

EV_1 = 30,800 lb/ft

EV_2 = 2,888 lb/ft

(C) Lateral Earth Pressure (EH)

same as ASD

P_A = 11,572 lb/ft

P_{Ax} = 11,178 lb/ft

P_{Ay} = 2,995 lb/ft

(D) Summary of Unfactored Loads and Moments

A summary of unfactored vertical loads and resisting moments is presented in Table 5.3, and of unfactored horizontal loads and driving moments in Table 5.4.

Table 5.3 Unfactored vertical loads and resisting moments

Item	Force, lb	Moment arm, ft	Moment, lb.ft
EV_1	30,800	7	215,600
EV_2	2,888	9.333	26,955
P_{Ay}	2,995	14	41,930
Total	36,683		284,485

Table 5.4 Unfactored horizontal loads and driving Moments

Item	Force	Moment arm	Moment
P_{Ax}	11,178	$\frac{23.75}{3}$	88,493

5.2.2 Limit States and Load Factors

<u>Strength I Limit State</u>:

For sliding and overturning, minimum vertical loads and maximum horizontal loads (I-a) – the minimum load factors are used for those load components that contribute to the resistance (EV = 1.0) and the maximum load factor is used for the driving force (EH = 1.5).

For bearing, maximum vertical loads (I-b) – the maximum load factors are used for all components of load for bearing (EV = 1.35 and EH = 1.5).

<u>Strength IV Limit State</u>:

For sliding and overturning, minimum vertical loads and maximum horizontal loads (IV-a) – this is the same case as (I-a).

For bearing, maximum vertical loads, (IV-b) – this is the same case as (I-b).

Service I Limit State:

Settlement – all the applicable loads have a load factor of 1.00.

The limit states that need to be evaluated are shown in Fig. 5.2. The applicable load combinations and load factors are summarized in Table 5.5.

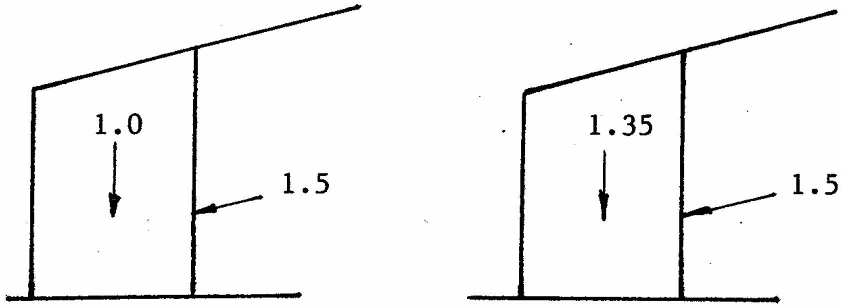
Table 5.5 Load factors

Group	?$_{EV}$	?$_{EH}$	Use
Strength I-a	1.0	1.5	Sliding and Eccentricity
Strength I-b	1.35	1.5	Bearing Capacity
Strength IV-a	1.0	1.5	Sliding and Eccentricity
Strength IV-b	1.35	1.5	Bearing Capacity
Service I	1.0	1.0	Settlement

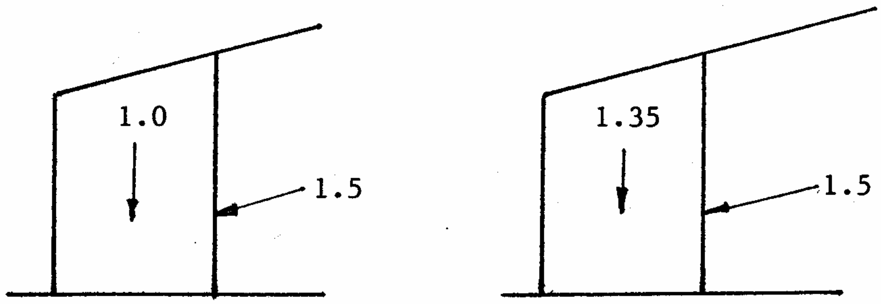
5.2.3 Factored Loads and Factored Moments

A summary of factored loads and moments is presented in Tables 5.6, 5.7, 5.8, and 5.9.

Strength I



Strength IV



Service I

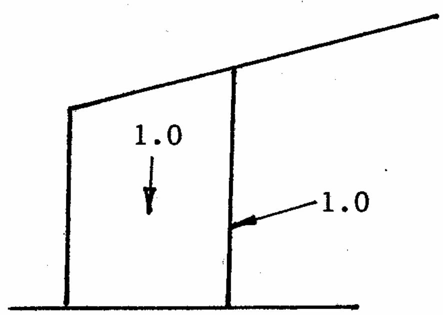


Fig. 5.2 Limit states analyzed for MSE wall

Table 5.6 Factored vertical loads

Item	EV_1	EV_2	P_{Ay}	V_{total}
V (unfactored)	30,800	2,888	2,995	36,683
Strength I-a	30,800	2,888	4,493	38,181
Strength I-b	41,580	3,899	4,493	49,972
Service I	30,800	2,888	2,995	36,683

Table 5.7 Factored horizontal loads

Item	P_{Ax}
H (unfactored)	11,178
Strength I-a	16,767
Strength I-b	16,767
Service I	11,178

Table 5.8 Factored moments from vertical forces M_V

Item	EV_1	EV_2	P_{Ay}	$M_{V(total)}$
M_V (unfactored)	215,600	26,955	41,930	284,485
Strength I-a	215,600	26,955	62,895	305,450
Strength I-b	291,060	36,389	62,895	390,344
Service I	215,600	26,955	41,930	284,485

Table 5.9 Factored moments from horizontal forces M_h

Item	P_{Ax}	$M_{h(total)}$
M_h (unfactored)	88,493	88,493
Strength I-a	132,740	132,740
Strength I-b	132,740	132,740
Service I	88,493	88,493

5.2.4 External Stability

(A) Sliding Resistance

The factored resistance against failure by sliding, Q_R, is:

$Q_R = f_T \cdot Q_T$

where f_T = resistance factor for sliding of soil against soil. From Table 10.5.5-1, f_T = 1.0.

Q_T = nominal shear resistance between soil and foundation, which is equal to V tan d, where V is the vertical force and tan d is the lesser of tan j_r or tan j_f.

i.e., Q_R = V tan f_f

= V tan 30

= 0.577 V

Table 5.10 Sliding resistance for the wall

Item	V_{total}	Q_R	H_{total}
Strength I-a	38,181	22,030	16,767
Strength I-b	49,972	28,834	16,767
Service I	36,683	21,166	11,178

Because the factored sliding resistance, Q_R, is greater than the factored horizontal loading, H_{total}, the sliding resistance is satisfactory.

(B) Eccentricity

X_0 = location of the resultant from toe of wall $= \dfrac{M_v - M_h}{V}$

e = eccentricity = $\dfrac{B}{2} - X_0$

$$= \frac{14}{2} - X_0$$
$$= 7 - X_o$$

The location of the resultant must be in the middle half of the base

$e_{max} = \dfrac{B}{4} = \dfrac{14}{4} = 3.5$

Table 5.11 Eccentricity for the wall

Item	V	M_v	M_h	X_0	e	e_{max}
Strength I-a	38,181	305,450	132,740	4.523	2.477	3.5
Strength I-b	49,972	390,344	132,740	5.155	1.845	3.5
Service I	36,683	284,485	88,493	5.343	1.657	3.5

for all cases, $e < e_{max}$, i.e., the design is adequate in regard to eccentricity.

(C) Bearing Resistance

(C.1) Factored Uniform Bearing Stress ?q

$B^{\backslash} = B - 2e$

$e = \dfrac{B}{2} - X_0$

i.e., $B^{\backslash} = 2X_0$

The maximum factored uniform bearing stress $gq = \dfrac{V}{L^{\backslash}B^{\backslash}}$

Since $L^{\backslash}$ = 1 ft (i.e., unit length of the wall) then,

$$?q = \frac{V}{1 \times 2X_0} = \frac{V}{2X_0}$$

Table 5.12 Bearing stress for the wall

Item	V	X_0	gq
Strength I-a	38,181	4.523	4,221
Strength I-b	49,972	5.155	4,847
Service I	36,683	5.343	3,433

(C.2) Factored Bearing Resistance

The factored bearing resistance, q_R is determined from;

$q_R = f \, q_{ult}$

where f = resistance factor. From Table 10.5.5-1 based on an semiempirical procedure using SPT data, the resistance factor is 0.45. Since the wall height is 20 ft, the forces for Service I is the same as ASD solution. i.e., q_{ult} = 21,216 psf

q_R = 0.45 x 21,216 = 9,547 psf

Because the factored bearing resistance, q_R, exceeds the maximum factored uniform bearing stress, ?q = 4,847 psf, the bearing resistance is adequate.

5.3 Summary of the ASD and LRFD for the MSE Wall

The results of the analysis for both the ASD and LRFD are summarized in Table 5.13.

Table 5.13 Summary of MSE wall design by ASD and LRFD

Performance Limit	ASD		LRFD	
	Required F.S./ Eccentricity	Actual	Factored Resistance	Factored Loading
Eccentricity	$e = \frac{B}{6} < 2.333$ F.S. > 2	e = 1.657 (F.S. = 3.215)	$e = \frac{B}{4} < 3.5$	e = 2.477
Sliding Resistance	F.S. > 1.5	F.S. = 1.89	22,030 lb/ft	16,767 lb/ft
Bearing Resistance	F.S. > 3	F.S. = 6.18	9,547 psf	4,847 psf

Both the LRFD and ASD produce an acceptable design for the wall.

CHAPTER VI

ANALYSIS OF DESIGN RESULTS

6.1 Introduction

The three types of Maryland walls satisfy both the ASD and LRFD specifications. In analyzing the results obtained, several questions come to mind and need to be responded to, these are: 1) What is the effect of varying the resistance factor? We cannot vary the load factors, since they are provided to us by the structural engineer. 2) What is the effect of the Life Load surcharge on the design? AASHTO 2002 has introduced a large equivalent height of soil for shorter walls. 3) Are the walls overdesigned according to the LRFD? Can we show that smaller dimensions of walls can be used.

6.2 Effect of Varying the Resistance Factors

The resistance factors provided by AASHTO 2002 can be analyzed with respect to the three requirements for stability, sliding, overturning and bearing.

6.2.1 Sliding on Granular Soil

Using the results from the standard penetration testing, which is the practice of MD SHA, according to AASHTO specifications for precast concrete sliding on sand uses a resistance factor of 0.9 and for cast-in-place concrete sliding on sand use a factor of 0.8.

6.2.2 Eccentricity (overturning)

AASHTO requires that the eccentricity of the footing evaluated based on factored loads, is less than ¼ of the corresponding footing dimension.

6.2.3 Bearing

AASHTO requires that when using semiempirical procedures using SPT data a resistance factor of 0.45 be used and when using a rational method using f estimated from SPT data the resistance factor becomes 0.35. AASHTO recommends higher values if using CPT data. Thus, a recommendation is to use CPT data if at all possible in MD SHA design.

In summary, there is a very small range of variation in AASHTO specifications for the resistance factors.

6.3 Effect of Life Load Surcharge

As indicated in Section 2.4.3, life load surcharge can be represented by an equivalent height of soils. In ASD, the height of soils was the same for any height of wall, at a height of 2 ft. Current AASHTO LRFD specifications define the equivalent height of soils as a function of the height of wall, as shown in Table 2.3. The table shows that for a height of wall of 5 ft, the equivalent height of soil is 5 ft. Only when the height of a wall is 20 ft or higher, does the height of soil become 2 ft. This means that walls shorter than 20 ft will be subject to a higher pressure than was used previously. In this section a study was undertaken to analyze the effect of different surcharge loadings on the stability of the wall.

6.3.1 Effect of Surcharge on Eccentricity

To study such an effect, wall heights of 6, 10, 12, 14, 16 and 20 ft, as shown in Table 6.1, were analyzed twice. Once with a constant surcharge of 2 ft and once with a surcharge based on AASHTO 2002 specification, Table 2.3. The walls were Maryland Type A retaining walls, Standard No. RW(6.03)-83-134.

Table 6.1 Wall analyzed

Height H	E	B	A	C	D
6	1.0	0.75	1.0	2.75	4.5
10	1.0	0.75	1.0	4.5	6.25
12	1.0	0.75	1.0	5.5	7.25
14	1.25	1.0	1.25	6.0	8.25
16	1.25	1.0	1.25	6.75	9
20	1.75	1.25	1.75	7.75	10.75

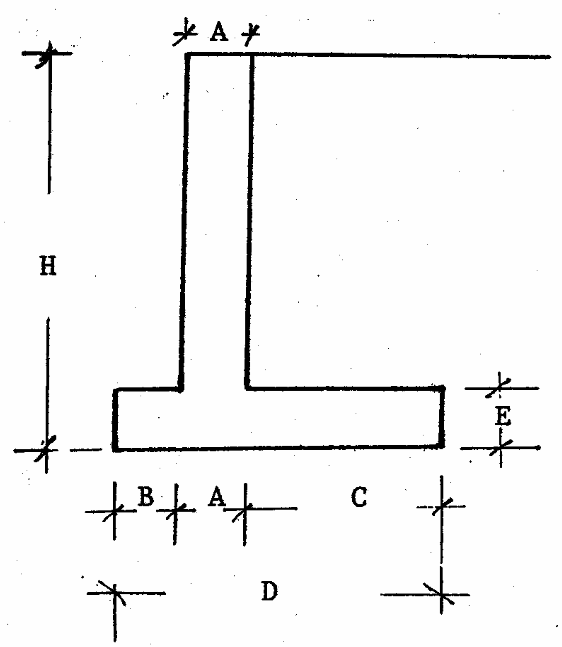


Table 6.2.a shows the case with a 2 ft surcharge and Table 6.2.b shows the case with an AASHTO 2002 surcharge. As can be seen from both tables, in both cases all walls satisfy AASHTO specifications. However, as expected for the shortest wall at 6 ft, the actual eccentricity is 50% of the limit eccentricity for AASHTO surcharge but is only 22% of the limit eccentricity for the 2 ft surcharge.

Table 6.2.a Effects of wall height on eccentricity, surcharge 2 ft

Wall Height	Actual Eccentricity (ft)	Limit Eccentricity (ft)	$\frac{\text{Actual Eccentricity}}{\text{Limit Eccentricity}} \times 100$
6	0.251	1.125	22
10	0.577	1.563	37
12	0.707	1.813	39
14	0.780	2.063	38
16	0.961	2.250	43
20	1.216	2.688	45

Table 6.2.b Effect of wall height on eccentricity, surcharge based on AASHTO 2002

Wall Height	Surcharge in ft	Actual Eccentricity (ft)	Limit Eccentricity (ft)	$\frac{\text{Actual Eccentricity}}{\text{Limit Eccentricity}} \times 100$
6	4.7	0.560	1.125	50
10	3.5	0.820	1.563	52
12	3.2	0.915	1.813	50
14	2.9	0.948	2.063	46
16	2.6	1.080	2.250	48
20	2.0	1.216	2.688	45

6.3.2 Effect of Surcharge on Sliding Resistance

Table 6.3.a shows the case for a surcharge of 2 ft and Table 6.3.b shows the case for an AASHTO 2002 surcharge. As can be seen from both tables, in both cases all walls satisfy AASHTO specifications. However, as expected for the 6 ft wall, the factored horizontal loading is 95% of the factored resistance for the AASHTO 2002 surcharge and only 68% for the 2 ft surcharge.

Table 6.3.a Effect of wall height on sliding resistance, surcharge 2 ft

Wall Height	Factored Loading (kip)	Factored Resistance (kip)	Factored Load. x 100 / Factored Resist.	Actual Resist. Factor
6	1.359	2.005	68	0.54
10	3.114	4.656	67	0.54
12	4.246	6.481	66	0.53
14	5.549	8.493	65	0.52
16	7.021	10.651	66	0.53
20	10.475	15.941	66	0.53

Table 6.3.b Effect of wall height on sliding resistance, surcharge based on AASHTO 2002

Wall Height	Surcharge in ft	Factored Loading (kip)	Factored Resistance (kip)	Factored Load. x 100 / Factored Resist.	Actual Resist. Factor
6	4.7	2.161	2.265	95	0.76
10	3.5	3.857	4.896	79	0.63
12	3.2	4.960	6.712	74	0.59
14	2.9	6.173	8.695	71	0.57
16	2.6	7.496	10.804	70	0.56
20	2.0	10.425	15.941	66	0.53

The resistance factors determined were in the range of 0.52 to 0.76, where as AASHTO allows a resistance factor of 0.8.

6.3.3 Effect of Surcharge on Bearing Capacity

Bearing capacity is a function of the site the wall will be built on. The site assumed for this analysis is a granular soil.

The bearing capacity in sand based on SPT results was calculated from AASHTO equation (10.6.3.1.3b-1)

$$q_{ult} = \frac{N.B}{10}\left(C_{w1} + C_{w2}\frac{D}{B}\right)R_i \qquad \text{in TSF}$$

where: N = corrected SPT blow count

B = footing width

C_{w1}, C_{w2} = correction factor for groundwater effect

D = depth of footing

R_i = reduction factor accounting for the effect of load inclination

For the walls analyzed, N was assumed to equal 12, C_{w1} and C_{w2} are both equal to 1.0 as there is no water table encountered at the site and R_i determined from AASHTO, Table 10.6.3.1.3b-2. The resistance factor based on the semiempirical procedure using SPT data is 0.45.

Again Table 6.4.a shows the bearing capacity for the 2 ft surcharge and Table 6.4.b shows the case for the AASHTO 2002 surcharge.

Table 6.4.a Effect of wall height on bearing capacity, surcharge 2 ft

Wall Height	Factored Bearing Stress	Factored Bearing Resistance	Bearing Stress x 100 / Bearing Resistance	Actual Resist. Factor
6	1,237	4,453	28	0.13
10	2,139	5,494	39	0.18
12	2,579	5,868	44	0.20
14	2,900	6,684	43	0.19
16	3,385	6,867	49	0.22
20	4,224	7,870	54	0.24

Table 6.4.b Effect of wall height on bearing capacity, surcharge based on AASHTO 2002

Wall Height	Factored Bearing Stress	Factored Bearing Resistance	Bearing Stress x 100 / Bearing Resistance	Actual Resist. Factor
6	1,669	3,643	46	0.21
10	2,489	4,695	53	0.24
12	2,887	6,091	47	0.21
14	3,130	6,684	47	0.21
16	3,554	6,867	52	0.23
20	4,224	7,870	54	0.24

As can be seen from both tables, in both cases all walls satisfy AASHTO specifications. However, as expected, for the 6 ft wall, the bearing stress is 46% of the bearing resistance for the AASHTO 2002 surcharge and only 28% for the 2 ft surcharge. The resistance factors determined were in the range of 0.13 to 0.24, where as AASHTO allows a resistance factor of 0.45.

In summary, all the walls are overdesigned. When we back-calculate the resistances factors of the existing design we find it to be much smaller than AASHTO specification. A reduction in the current wall dimensions can thus be undertaken. A reduction in the size of the walls will translate into a reduction in cost of the retaining walls.

6.4 Design Optimization

All the design results according to the LRFD indicated that the walls are over designed. Even with the AASHTO 2002 surcharge, the walls are still overdesigned. To get a perspective of how much are the walls overdesigned, a 20 ft high wall was analyzed twice. Once with its regular dimension of a base of 10.75 ft and again with a new dimension of a base of 8.25 ft (the same as a 14 ft high wall). Table 6.5 shows the results of both designs.

Table 6.5 Effect of the base size on the wall stability (20 ft Wall)

Base Width (ft)	Eccentricity		Sliding		Bearing		Wall Area in ft^2
	Actual	Limit	Loading	Resistance	Stress	Resistance	
8.25	1.979	2.063	10.425	12.872	5,573	6,441	33.75
10.75	1.216	2.688	10.475	15.941	4,224	7,870	50.75

As shown in Table 6.5 both base widths satisfy the eccentricity, sliding and bearing of the wall. However, the wall with a base of 8.25 ft has an area of 33.25 ft^2 and the one with a base of 10.75 ft has an area of 50.75 ft^2. That is, by using the 8.25 ft base we reduced the wall cross-sectional area by 34% of the original area of the wall. Such a reduction in area of the wall will no doubt translate into a reduction in cost of the wall.

CHAPTER VII

CONCLUSIONS

AASHTO, LRFD specifications for retaining walls were summarized and presented in this report. A comparative design between ASD and LRFD specifications was carried out by analyzing, three types of retaining walls, of the type that are used by Maryland SHA were analyzed by both the ASD and LRFD methodology. This provides a guide to a designer who is familiar with ASD methodology and is not familiar with LRFD methodology but is interested in implementing it. A spreadsheet program for the design of those three types of retaining walls based on AASHTO LRFD specifications was also developed. The Excel program was to be used to check the hand calculations and facilitate the design of these walls for different geometries and soil properties.

In all three walls, only external stability that includes sliding, overturning and bearing of the wall systems were considered.

Six standard cantilever walls (MD Standard No. RW(6.03)-83-134) that varied from a height of 6 ft to 20 ft were also analyzed by the LRFD to determine their resistance factors. It was found that the values of the actual resistance factors are much lower than the AASHTO recommended values. This indicated that those walls are overdesigned from the geotechnical point of view. To check this point further, a cantilever wall of a height of 20 ft was analyzed twice, once with a base width of 10.75 ft as is recommended in MD SHA and again with a width of 8.25 ft. Both walls were safe, however, the wall with a base of 8.25 ft led to a reduction in the cross-sectional area of the wall by 34%. This with no doubt translates into a reduction in cost of the wall. Thus, unless there is a structural reason for the dimensions of these cantilever walls, they can be reduced in size based on the geotechnical analyses undertaken.

Current AASHTO LRFD defined the life load surcharge as an equivalent height of soil that is a function of the height of the wall. In this definition, a wall of a height of 5 ft will be subjected to a life load surcharge equivalent to a 5 ft height of soils, and for a wall of 20 ft, the life load surcharge is equivalent to a 2 ft height of soils. Such a criteria will penalize the shorter walls compared to the previous definition of 2 ft height of soil for walls of any height. For this reason all six cantilever walls were analyzed for the old and new criteria. In all cases, the walls analyzed satisfied both criteria with the shorter walls showing higher resistance factors than the taller ones as was expected.

REFERENCES

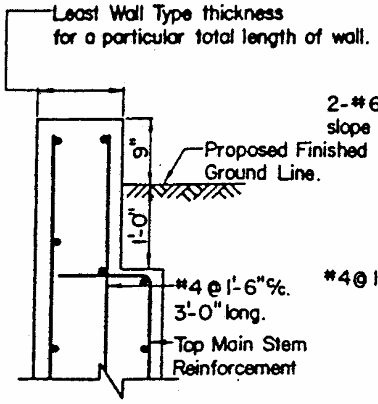
"AASHTO LRFD Bridge Design Specifications," 2002 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, D.C., May 2002.

Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures, Federal Highway Administration, FHWA HI-98-032, July 1998.

Standard Specifications for Highway Bridges, 17th edition, American Association of State Highway and Transportation Officials, Washington, D.C., 2002.

APPENDIX A

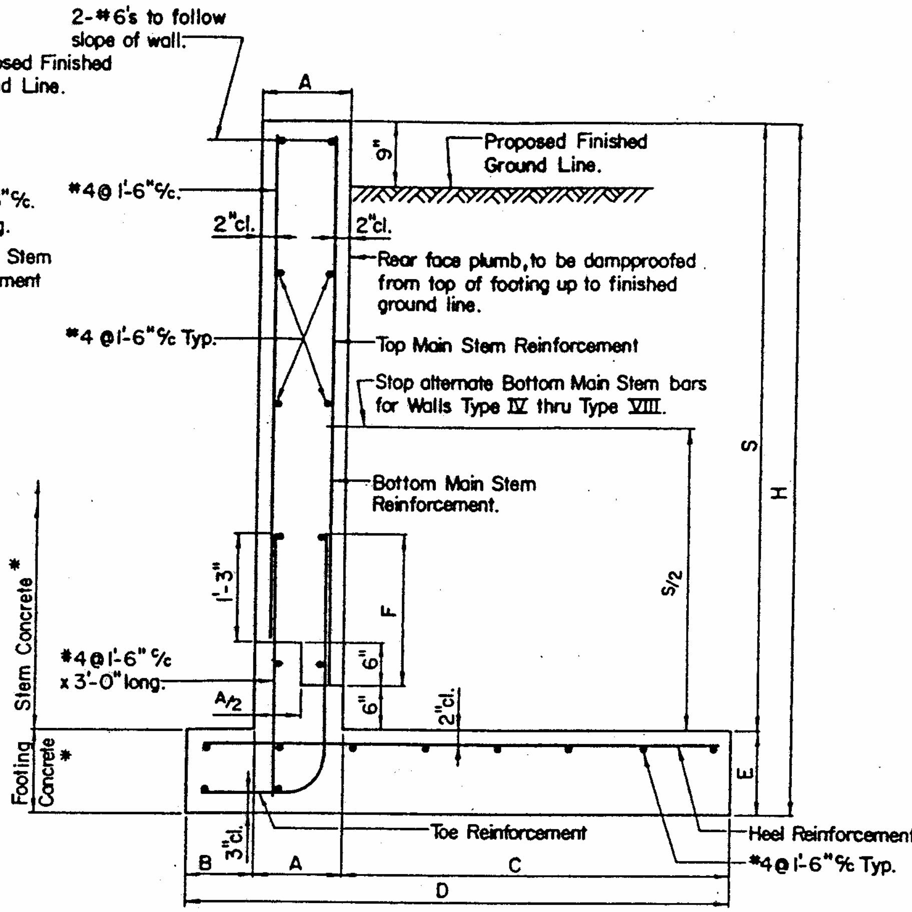
Cantilever Retaining Wall



DETAIL A

Scale: None

(See note 2 below).



* Where specific footing concrete and stem concrete items are set up in the proposal for a particular wall these shall be the pay limits. Where no specific items have been set up in the proposal, the cost of wall shall be included in the main structure bid price, i.e.; box culvert, where wings are included in box culvert item; retaining wall, where no separate pay items are established; etc.

TYPICAL SECTION

Scale: 1/2" = 1'-0"

Wall Type	H	A	B	C	D	E	Toe Reinf.	Heel Reinf.	Top Main Stem Reinf.	Bottom Main Stem Reinf.	F
A-I	6'-0"	1'-0"	9"	2'-9"	4'-6"	1'-0"	#5@1'-0"c/c	#5@1'-0"c/c	#5@1'-0"c/c	#5@1'-0"c/c	1'-9"
A-II	8'-0"	1'-0"	9"	3'-6"	5'-3"	1'-0"	#5@1'-0"c/c	#5@1'-0"c/c	#5@1'-0"c/c	#5@1'-0"c/c	1'-9"
A-III	10'-0"	1'-0"	9"	4'-6"	6'-3"	1'-0"	#5@10"c/c	#5@10"c/c	#5@10"c/c	#5@10"c/c	1'-9"
A-IV	12'-0"	1'-0"	9"	5'-6"	7'-3"	1'-0"	#5@6"c/c	#5@6"c/c	#5@1'-0"c/c	#5@6"c/c	1'-9"
A-V	14'-0"	1'-3"	1'-0"	6'-0"	8'-3"	1'-3"	#6@7"c/c	#6@7"c/c	#6@1'-2"c/c	#6@7"c/c	2'-1"
A-VI	16'-0"	1'-3"	1'-0"	6'-9"	9'-0"	1'-3"	#6@5"c/c	#6@5"c/c	#6@10"c/c	#6@5"c/c	2'-7"
A-VII	18'-0"	1'-6"	1'-3"	7'-3"	10'-0"	1'-6"	#7@6"c/c	#7@6"c/c	#7@1'-0"c/c	#7@6"c/c	2'-10"
A-VIII	20'-0"	1'-9"	1'-3"	7'-9"	10'-9"	1'-9"	#8@7"c/c	#8@7"c/c	#8@1'-2"c/c	#8@7"c/c	3'-8"

Notes:

1. An "Excellent Soil Condition" is that foundation material that can support a safe bearing pressure greater than 5k/square foot.
2. If in the length of a wall the type of wall changes and provides for a different thickness of stem, then "Detail A" shall be utilized for all walls of greater than the least wall thickness.
3. Contractor has option of lapping stem reinforcement with toe reinforcement and/or dowels as shown; or by extending the toe and/or dowel reinforcement with no splicing. However no additional compensation to Contractor will be allowed for whichever alternative is selected.

APPROVAL	
E.S. [signature] ASST. CHIEF ENGR. BRIDGE DEVEL.	
DATE 6/8/83	
REVISIONS	
SHA	FHWA
4-17-85	6-21-85
10-16-85	
12-6-88	

FHWA APPROVAL DATE: 6-21-85

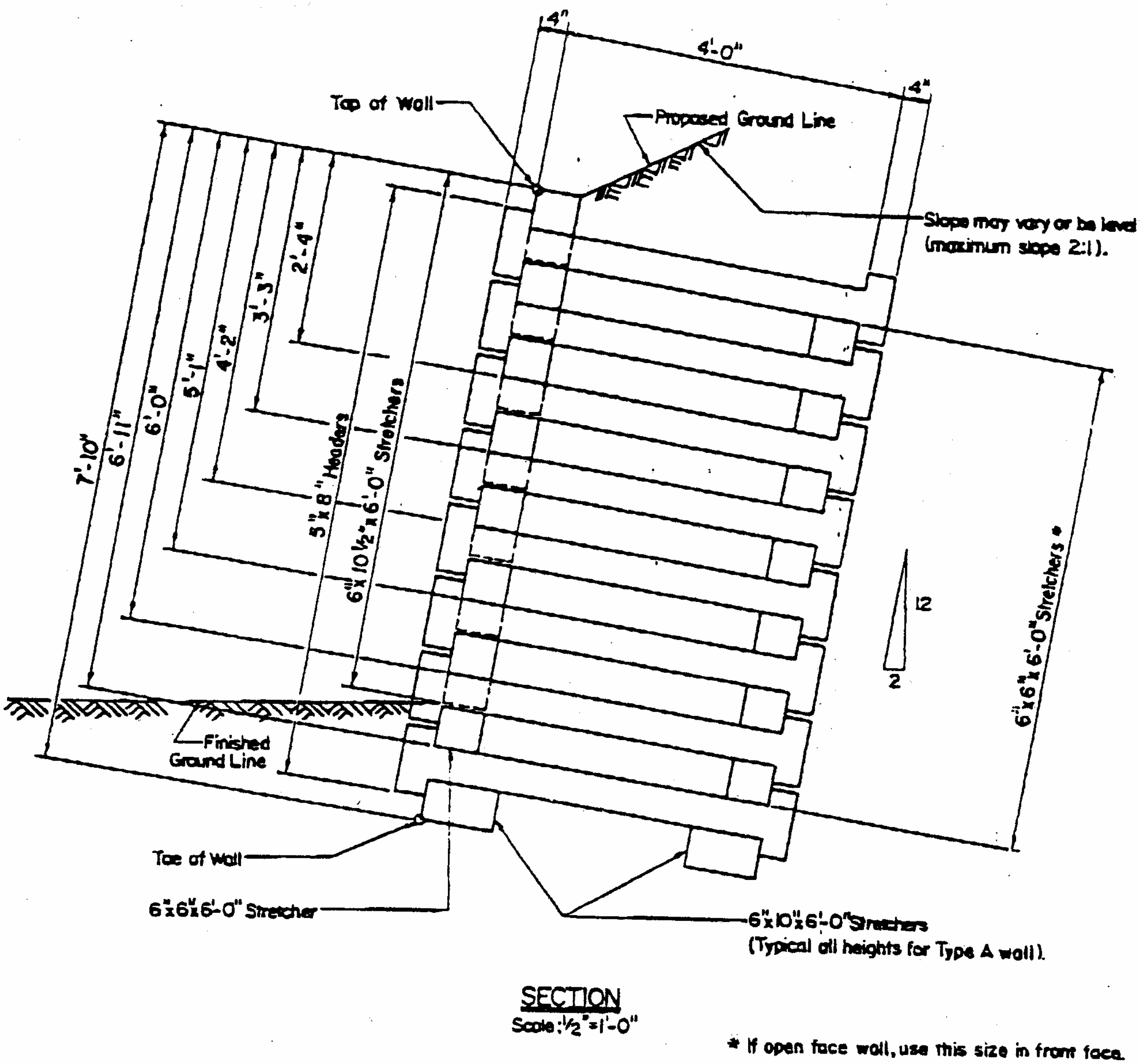
STATE OF MARYLAND
DEPARTMENT OF TRANSPORTATION
STATE HIGHWAY ADMINISTRATION
DIVISION OF BRIDGE DEVELOPMENT
TYPE A RETAINING WALL SECTION
(FOR EXCELLENT SOIL CONDITION AND TWO FOOT SURCHARGE)

STANDARD NO. RW(6.03)-83-134 SHEET 1 OF 1

RETAINING WALLS

APPENDIX B

Crib Retaining Wall



APPROVAL		STATE OF MARYLAND DEPARTMENT OF TRANSPORTATION STATE HIGHWAY ADMINISTRATION DIVISION OF BRIDGE DEVELOPMENT CRIB WALL SECTION - TYPE A
ASST. CHIEF ENGR BRIDGE DEVEL. DATE 8/21/77		
REVISIONS		
SHA	FHWA	
FHWA APPROVAL DATE: 12-19-79		STANDARD NO. RW(6.01)-79-18 SHEET 2 OF 7

RETAINING WALLS

APPENDIX C

Spreadsheet Program