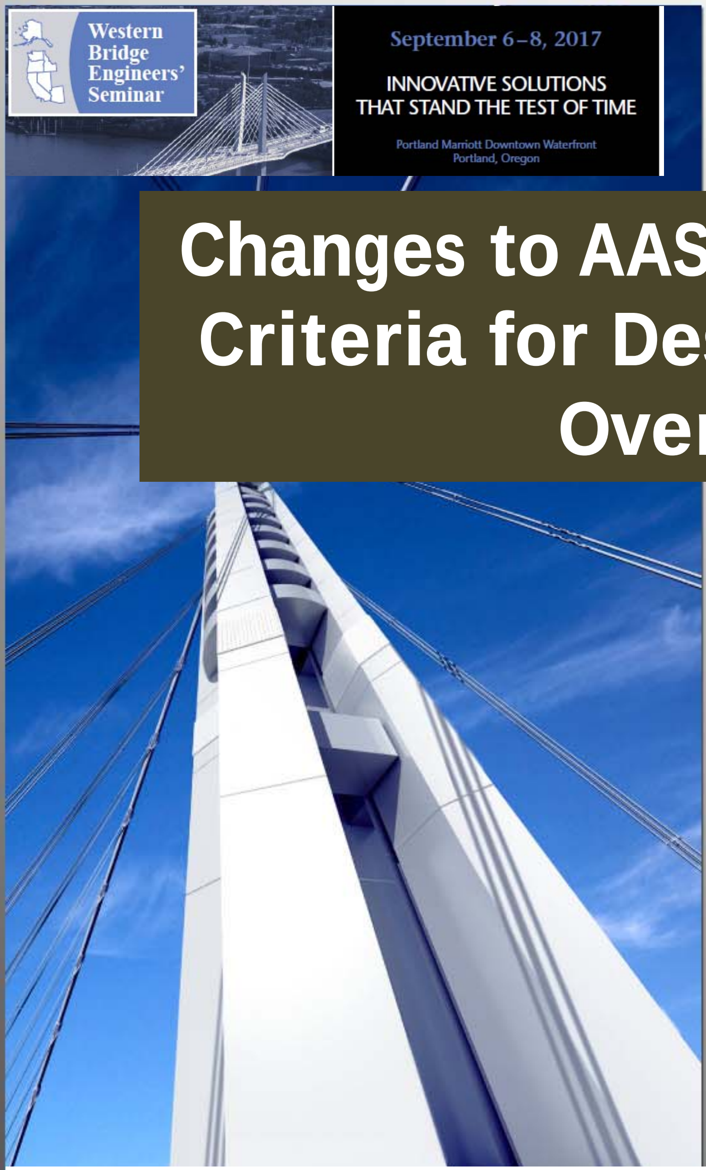


Sept 8th, 2017

Changes to AASHTO LRFD & Caltrans Design Criteria for Design of Retaining Structures Over the Last Decade

LFD → LRFD

By:

Ahilan Selladurai, P.E.,S.E.,PMP

Senior Bridge Engineer

T·Y·LIN INTERNATIONAL

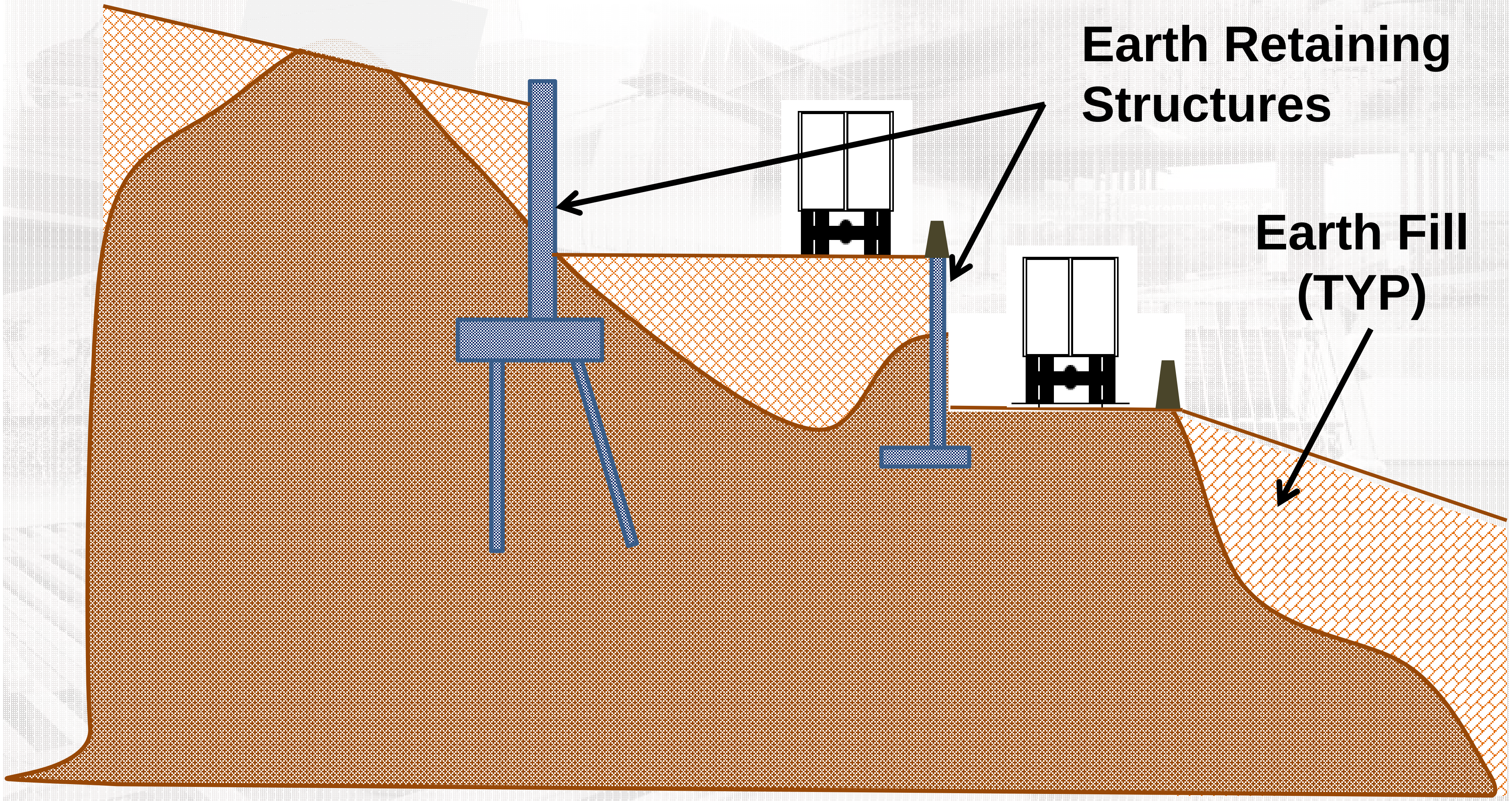
OUTLINE

- Geometry & Cost Comparison ASD/LRFD
- Standard Walls vs Special Design Walls
- LRFD Retaining Wall Design Background
- Summary of Reasons For Changes
- Other Type of Walls
- Conclusion

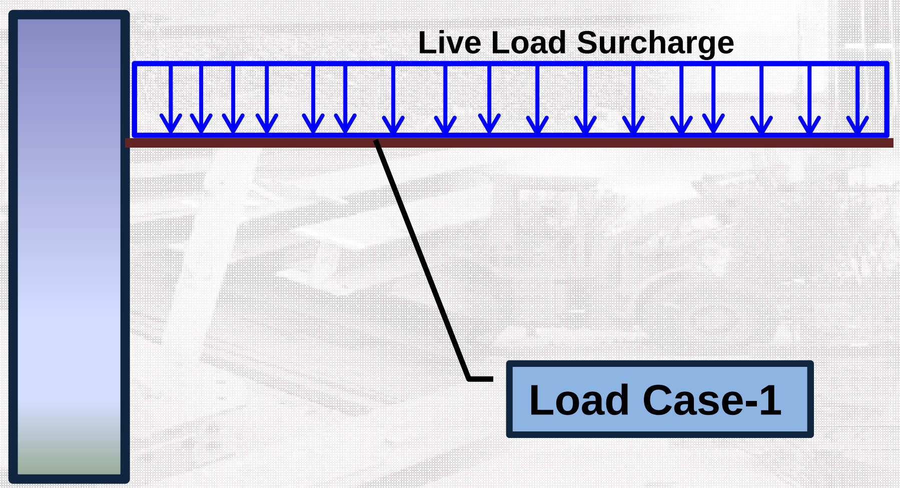
OUTLINE

- Geometry & Cost Comparison LFD/LRFD
- Standard Walls vs Special Design Walls
- LRFD Retaining Wall Design Background
- Summary of Reasons For Changes
- Other Type of Walls
- Conclusion

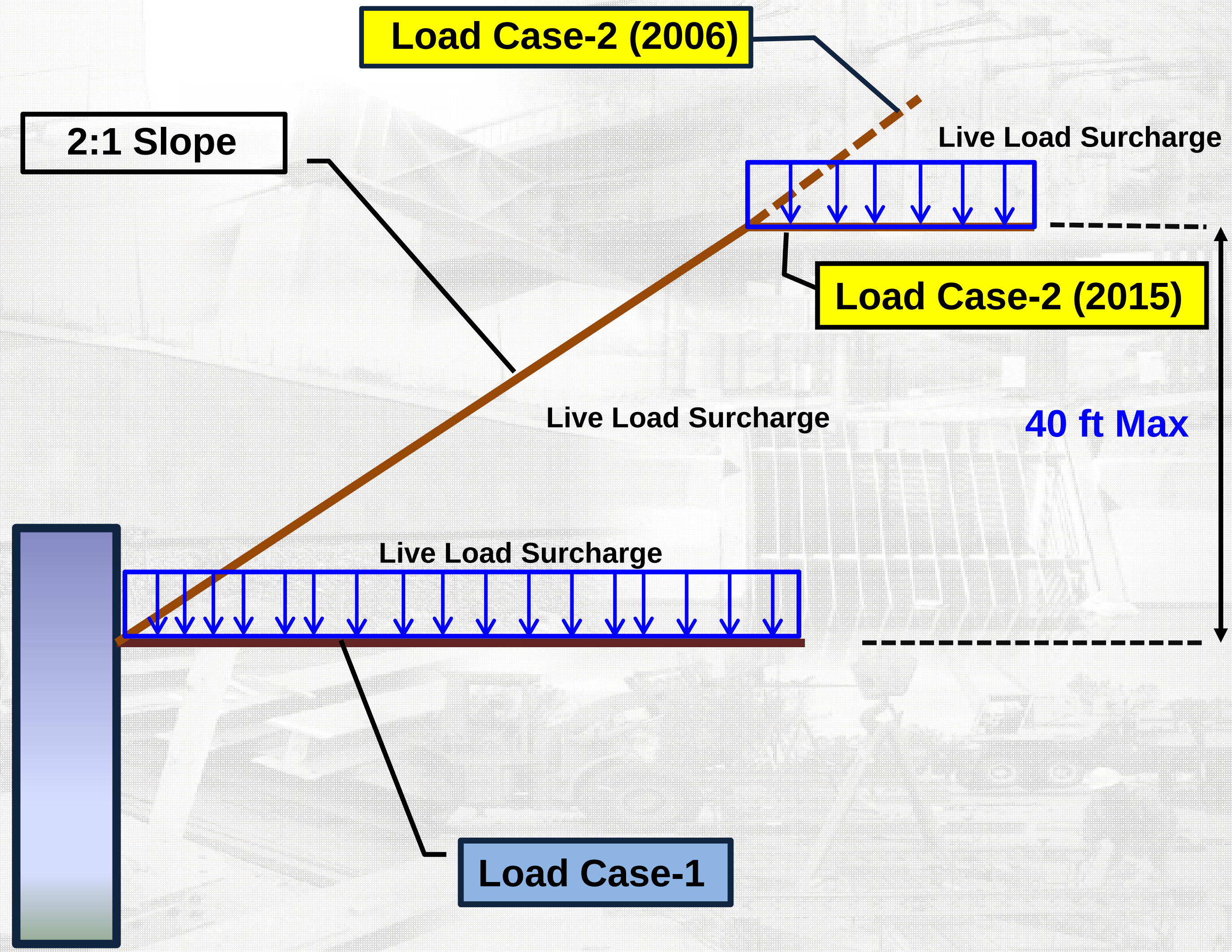
WHY EARTH RETAINING STRUCTURES?



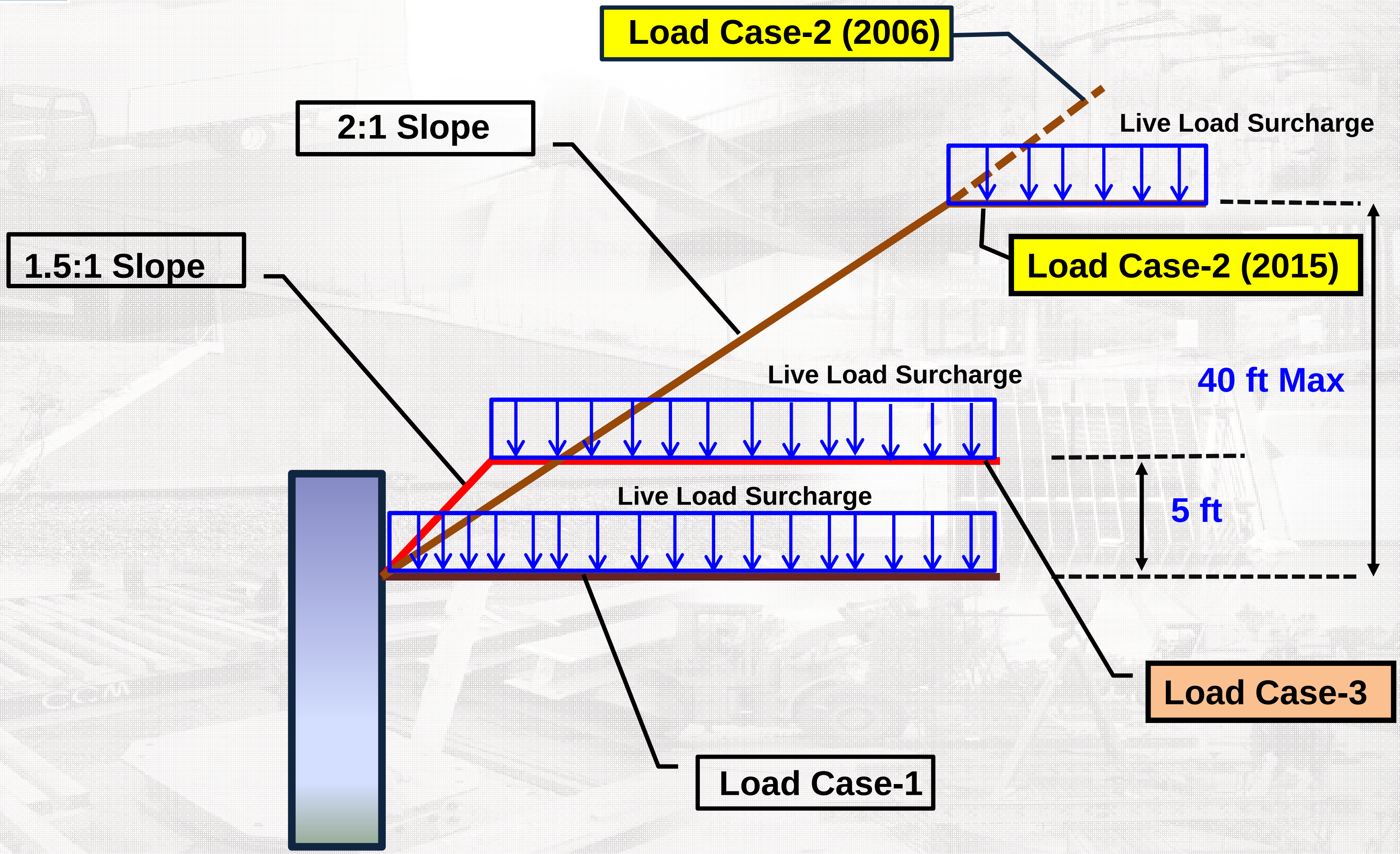
CALTRANS TYPICAL LOAD CASES



CALTRANS TYPICAL LOAD CASES



CALTRANS TYPICAL LOAD CASES



CALTRANS STD PLANS 2006

No Design Criteria on Plans!!!

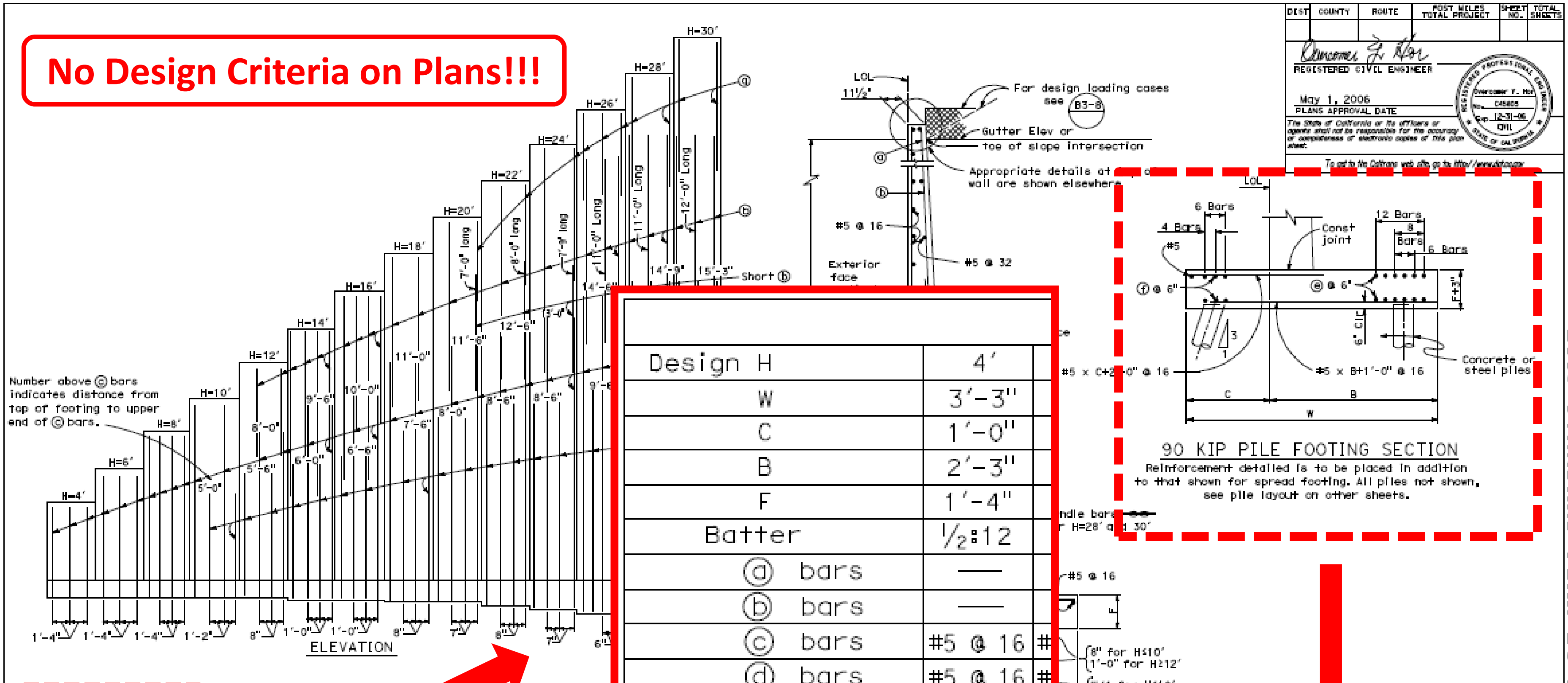


TABLE OF REINFORCING STE[...]NSIONS AND DATA

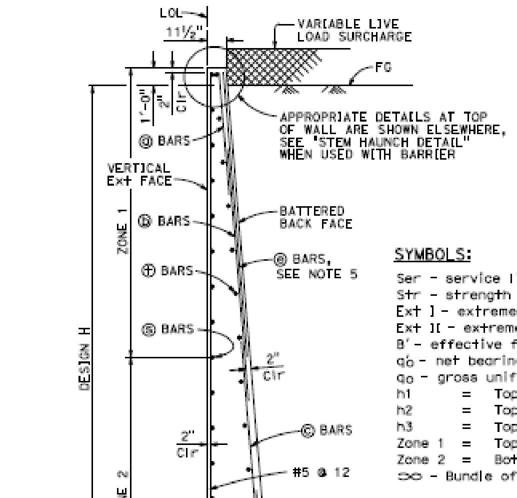
Design H		4'	6'	8'	10'	12'	[illegible]	16'	18'	20'	22'
W		3'-3"	4'-3"	5'-3"	6'-3"	[illegible]	[illegible]	9'-0"	10'-0"	11'-0"	12'-1"
C		1'-0"	1'-4"	1'-8"	[illegible]	[illegible]	2'-7"	2'-11"	3'-3"	3'-7"	3'-11"
B		2'-3"	2'-11"	3'-7"	[illegible]	[illegible]	5'-5"	6'-1"	6'-9"	7'-5"	8'-2"
F		1'-4"	1'-4"	[illegible]	[illegible]	1'-4"	1'-6"	1'-6"	1'-6"	1'-6"	1'-8"
Batter		½:12	½:12	[illegible]	½:12	½:12	½:12	½:12	½:12	½:12	½:12
ⓐ bars		—	—	—	—	—	—	—	—	#5 @ 28	#5 @ 32
ⓑ bars		—	—	—	—	#5 @ 16	#5 @ 12	#5 @ 12	#6 @ 16	#8 @ 14	#8 @ 16
ⓒ bars		#5 @ 16	#5 @ 16	#5 @ 16	#6 @ 14	#6 @ 8	#8 @ 12	#9 @ 12	#9 @ 8	#9 @ 7	#11 @ 8
ⓓ bars		#5 @ 16	#5 @ 16	#5 @ 16	#5 @ 14	#5 @ 8	#8 @ 12	#9 @ 12	#9 @ 8	#9 @ 7	#11 @ 8
Total ⓔ bars		6-#6	6-#6	6-#6	12-#6	12-#6	12-#6	12-#6	8-#6	8-#6	8-#6
Total ⓕ bars		—	—	—	6-#6	6-#6	6-#6	6-#6	6-#6	6-#6	6-#6
Loading Case I	Toe Pressure ksf	1.7	1.9	2.2	2.5	2.8	3.3	3.5	4.0	4.3	4.6
Loading Case II	Toe Pressure ksf	1.1	1.5	2.0	2.3	2.7	3.3	3.7	4.2	4.7	5.5
Loading Case III	Toe Pressure ksf	1.4	1.7	2.1	2.5	2.9	3.4	3.8	4.3	4.8	5.4

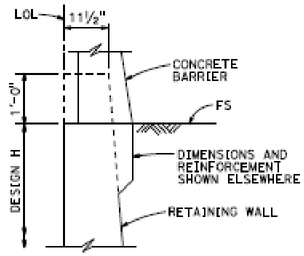
Ĩ Denotes a bundle of 2 bars

Design H		4'
W		3'-3"
C		1'-0"
B		2'-3"
F		1'-4"
Batter		½:12
ⓐ bars		—
ⓑ bars		—
ⓒ bars		#5 @ 16
ⓓ bars		#5 @ 16
Total ⓔ bars		6-#6
Total ⓕ bars		—
Loading Case I	Toe Pressure ksf	1.7
Loading Case II	Toe Pressure ksf	1.1
Loading Case III	Toe Pressure ksf	1.4

PILE OPTION

CALTRANS STD PLANS 2015





STEM HAUNCH DETAIL

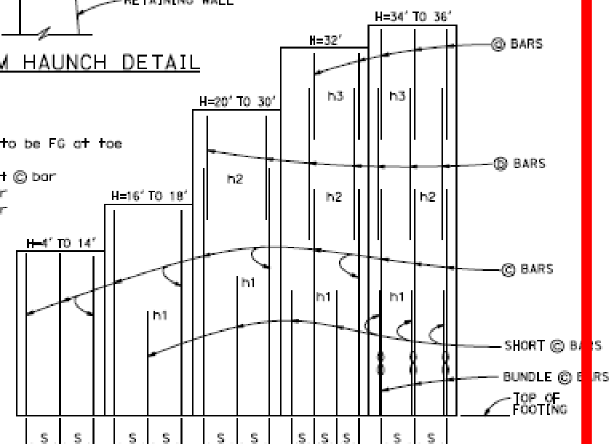
SYMBOLS:

Ser – service limit state I
Str – strength limit state I
Ext I – extreme event limit state I
Ext II – extreme event limit state II
B' – effective footing width (ft)
q'o – net bearing stress (ksf), OG assumed to be FG at toe
qo – gross uniform bearing stress (ksf)
h1 = Top of footing to top of short Ⓒ bar
h2 = Top of footing to top of Ⓒ bar
h3 = Top of footing to top of Ⓑ bar
Zone 1 = Top half of stem height
Zone 2 = Bottom half of stem height
⊃⊂ – Bundle of two bars

DESIGN CONDITIONS:

Design H may be exceeded by 6' before going to the next size. Special footing design is required where foundation material is incapable of supporting bearing stress listed in the table.

Design Criteria !!!



ELEVATION

DESIGN NOTES:

DESIGN: AASHTO LRFD Bridge Design Specifications, 4th Edition with California Amendments

LS: Varied surcharge on level ground surface

DC: Stem Architectural Treatment of thickness up to 6" of concrete (75 psf) considered

CT: 54 kip transverse force applied at He = 32", distributed over 10 feet at the top of wall and 1 : 1 distribution down and outward. Distribution below footing taken no less than 40'.

SEISMIC: $K_h = 0.2$, $K_v = 0.0$

SOIL: $\phi = 34°$, $\gamma = 120$ pcf

REINFORCED CONCRETE: $f'c = 3{,}600$ psi, $fy = 60{,}000$ psi

LOAD COMBINATIONS AND LIMIT STATES:

- Service I: $Q = 1.00DC+1.00EV+1.00EH+1.00LS$
- Strength I: $Q = \alpha DC+\beta EV+\eta EH+1.75LS$
- Extreme I: $Q = 1.00DC+1.00EV+1.00EH+1.00EQD+1.00EQE$
- Extreme II: $Q = 1.00DC+1.00EV+1.00EH+1.00CT$

Where:
- Q: Force Effects
- α: 1.25 or 0.90, Whichever Controls Design
- β: 1.35 or 1.00, Whichever Controls Design
- η: 1.50 or 0.90, Whichever Controls Design
- DC: Dead Load of Structure Components
- EH: Horizontal Earth Fill Pressure
- EV: Vertical Earth Pressure from Earth Fill Weight
- LS: Live Load Surcharge
- EQE: Seismic Earth Pressure
- EQD: Soil and Structural and Nonstructural Components Inertia
- CT: Vehicular Collision Force

TABLE OF REINFORCING STEEL, DIMENSIONS AND DATA

DESIGN H	4'	6'	8'	10'	12'	14'	16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'
W	6'-10"	7'-0"	7'-3"	7'-7"	8'-4"	9'-7"	10'-9"	12'-0"	13'-3"	14'-6"	15'-9"	17'-1"	18'-5"	19'-10"	21'-2"	22'-7"	24'-0"
C	2'-2"	2'-3"	2'-3"	2'-4"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"	5'-5"	6'-0"	6'-6"	7'-2"	7'-8"	8'-2"	9'-0"
B	4'-8"	4'-9"	5'-0"	5'-3"	5'-10"	6'-7"	7'-3"	8'-0"	8'-9"	9'-6"	10'-4"	11'-1"	11'-11"	12'-8"	13'-6"	14'-5"	15'-0"
F	1'-4"	1'-4"	1'-4"	1'-4"	1'-6"	1'-8"	1'-8"	1'-9"	1'-9"	1'-11"	2'-2"	2'-5"	2'-10"	3'-3"	3'-6"	4'-0"	4'-3"
BATTER	½: 12	½: 12	½: 12	½: 12	½: 12	½: 12	½: 12	½: 12	½: 12	½: 12	⅝: 12	⅝: 12	¾: 12	⅞: 12	1 : 12	1 : 12	1 : 12
SPACING "S"	9"	9"	9"	9"	9"	7"	6"	5"	6"	6"	6"	6"	6"	6"	6"	10"	8"
Ⓐ BARS	–	–	–	–	–	–	–	–	–	–	–	–	–	–	#7	#7	#6
Ⓑ BARS	–	–	–	–	–	–	–	–	#7	#7	#7	#7	#7	#7	#9	#9	#8
Ⓒ BARS	#6	#6	#6	#6	#6	#6	#7	#7	#8	#9	#9	#10	#10	#10	#11	#11	#11
Ⓓ BARS	#5	#5	#6	#6	#6	#6	#9	#8	#8	#9	#9	#10	#10	#10	#11	#11	#11
h1	–	–	–	–	–	–	5'-9"	5'-10"	8'-0"	9'-0"	10'-1"	11'-0"	12'-1"	13'-0"	13'-0"	12'-7"	11'-6"
h2	–	–	–	–	–	–	–	–	10'-5"	13'-0"	14'-7"	17'-6"	19'-0"	20'-5"	19'-0"	18'-0"	20'-2"
h3	–	–	–	–	–	–	–	–	–	–	–	–	–	–	21'-2"	21'-10"	24'-0"
ZONE 1 Ⓢ BARS	#5 @ 18	#5 @ 18	[illegible]5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12
ZONE 2 Ⓢ BARS	#5 @ 18	#5 @ 18	[illegible]5 @ 18	#5 @ 18	#5 @ 18	#5 @ 18	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#5 @ 12	#6 @ 12	#6 @ 12	#6 @ 12	#7 @ 12	#7 @ 12
ZONE 1 Ⓕ BARS	#4 @ 18	#4 @ 18	[illegible] @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 12	#4 @ 12	#4 @ 18	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12
ZONE 2 Ⓕ BARS	#4 @ 18	#4 @ 18	[illegible] @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 18	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#5 @ 12	#5 @ 12	#5 @ 12
Ser: B', q'o	6.8, 0.7	6.5, 1.0	[illegible].2, 1.3	6.0, 1.6	6.3, 2.0	7.5, 2.1	8.6, 2.2	9.8, 2.3	11.0, 2.4	12.1, 2.5	13.2, 2.8	14.4, 2.9	15.5, 3.1	16.8, 3.3	18.0, 3.5	19.2, 3.7	20.6, 3.7
Str: B', qo	6.6, 1.6	5.0, 1.8	[illegible]6, 2.3	3.0, 3.3	3.2, 4.0	4.3, 3.8	5.3, 3.7	6.4, 3.7	7.4, 3.8	8.2, 4.1	9.0, 4.4	9.9, 4.6	10.7, 4.9	11.7, 5.2	12.6, 5.4	13.6, 5.8	14.6, 5.9
Ext I: B', qo	5.2, 1.1	4.7, 1.5	[illegible]9, 2.2	3.1, 3.4	2.8, 4.8	3.2, 5.3	3.6, 5.7	4.1, 6.1	4.6, 6.4	5.0, 6.9	5.3, 7.6	5.8, 8.1	6.1, 8.9	6.7, 9.4	7.1, 10.0	7.5, 10.7	8.2, 10.9
Ext II: B', qo	2.6, 2.2	2.7, 2.6	[illegible]8, 3.1	2.9, 3.6	3.7, 3.6	5.2, 3.3	6.7, 3.1	8.3, 3.0	9.8, 3.0	11.2, 3.1	12.5, 3.2	13.9, 3.4	15.2, 3.6	16.7, 3.8	18.0, 4.0	19.3, 4.2	20.8, 4.3

DESIGN H	4'	6'
W	6'-10"	7'-0"
C	2'-2"	2'-3"
B	4'-8"	4'-9"
F	1'-4"	1'-4"
BATTER	½: 12	½: 12
SPACING "S"	9"	9"
Ⓐ BARS	–	–
Ⓑ BARS	–	–
Ⓒ BARS	#6	#6
Ⓓ BARS	#5	#5
h1	–	–
h2	–	–
h3	–	–
ZONE 1 Ⓢ BARS	#5 @ 18	#5 @ 18
ZONE 2 Ⓢ BARS	#5 @ 18	#5 @ 18
ZONE 1 Ⓕ BARS	#4 @ 18	#4 @ 18
ZONE 2 Ⓕ BARS	#4 @ 18	#4 @ 18
Ser: B', q'o	6.8, 0.7	6.5, 1.0
Str: B', qo	6.6, 1.6	5.0, 1.8
Ext I: B', qo	5.2, 1.1	4.7, 1.5
Ext II: B', qo	2.6, 2.2	2.7, 2.6

NO PILE OPTION ?

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS

REGISTERED CIVIL ENGINEER

October 30, 2015

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL CIVIL ENGINEER — Gary Wong — C59298 — Exp. 6-30-16 — CIVIL — STATE OF CALIFORNIA

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

RETAINING WALL TYPE 1 (CASE 1)

NO SCALE

B3-1A

Western Bridge Engineers' Seminar

GEOMETRY COMPARISION CALTRANS STD PLANS 2006 vs 2015

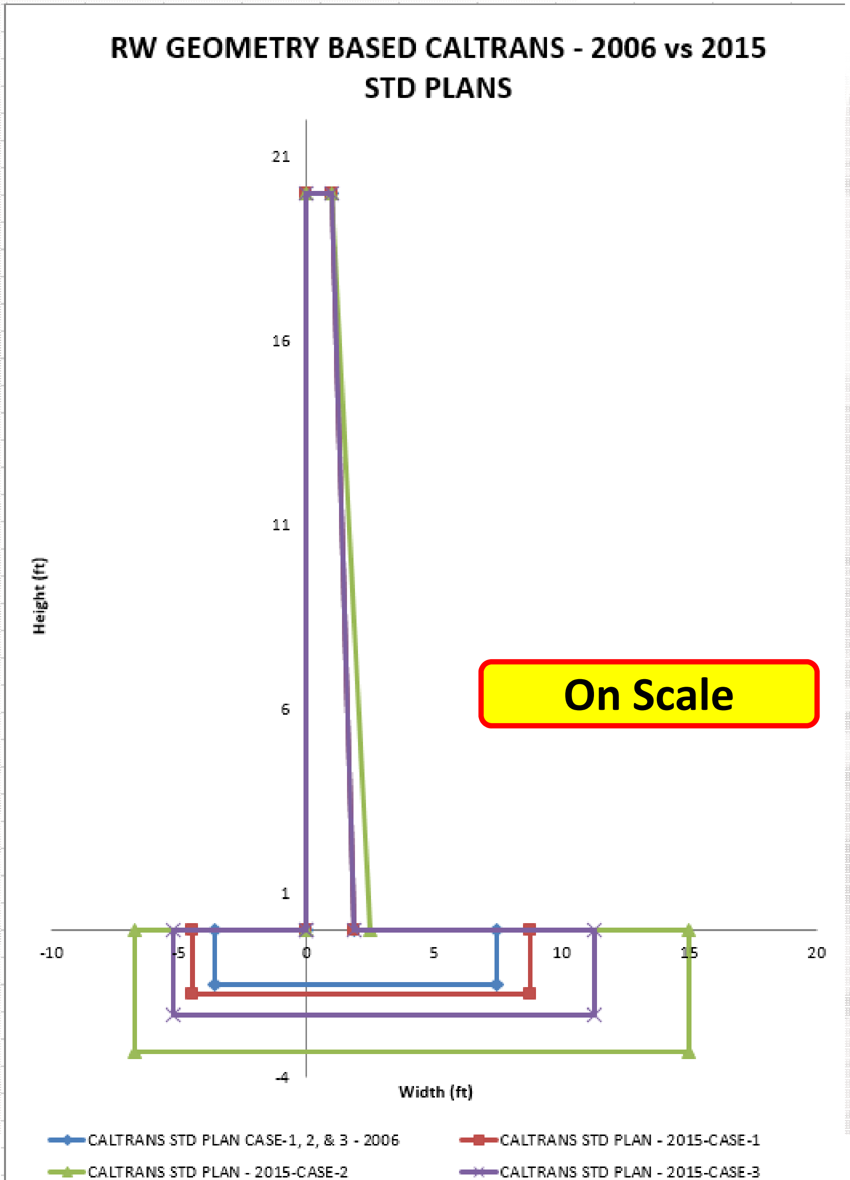
T·Y·LIN INTERNATIONAL

Description	CALTRANS STD PLAN H = 20 FT			
	2006 All Load Cases	2015 Load Case-1	2015 Load Case-2	2015 Load Case-3
H - Height -ft	20.00	20.00	20.00	20.00
W -Footing Width -ft	11.00	13.25	21.75	16.50
T -Wall Top -ft	1.00	1.00	1.00	1.00
Batter (1: xx)	½:12	½:12	5/8:12	½:12
F -Footing thickness -ft	1.50	1.75	3.33	2.33
C -Toe to Back Face -ft	3.58	4.50	6.75	5.25

Western Bridge Engineers' Seminar

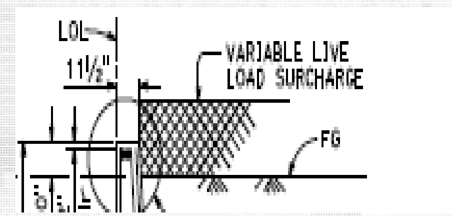
T·Y·LIN INTERNATIONAL

GEOMETRY COMPARISION CALTRANS STD PLANS 2006 vs 2015

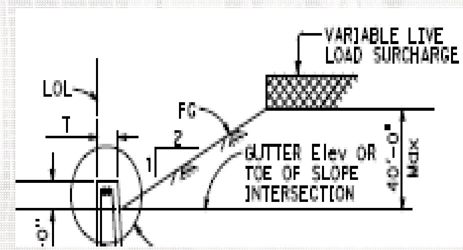


On Scale

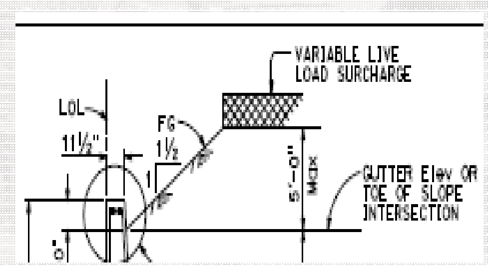
LOAD CASE - 1



LOAD CASE - 2



LOAD CASE - 3



Western Bridge Engineers' Seminar

QUANTITY & COST COMPARISION ASD/LRFD – CALTRANS STD PLANS 2006 vs 2015

T·Y·LIN INTERNATIONAL

EXCAVATION PER LINEAR FT OF RETAINING WALL				
Description	CALTRANS STD PLAN			
	2006 All Load Cases	2015 Load Case-1	2015 Load Case-2	2015 Load Case-3
Excavation - CY	46.00	58.00	127.00	81.00
Increase - %	0 %	26 %	178 %	76 %

BACKFILL PER LINEAR FT OF RETAINING WALL				
Description	CALTRANS STD PLAN			
	2006 All Load Cases	2015 Load Case-1	2015 Load Case-2	2015 Load Case-3
Backfill - CY	26.00	31.00	50.00	38.00
Increase - %	0 %	20 %	95 %	50 %

QUANTITY & COST COMPARISION ASD/LRFD – CALTRANS STD PLANS 2006 vs 2015

CONCRETE PER LINEAR FT OF RETAINING WALL				
Description	CALTRANS STD PLAN			
	2006 All Load Cases	2015 Load Case-1	2015 Load Case-2	2015 Load Case-3
Concrete - CY	237	244	300	260
Increase - %	0 %	3%	27%	10 %

STEEL PER LINEAR FT OF RETAINING WALL				
Description	CALTRANS STD PLAN			
	2006 All Load Cases	2015 Load Case-1	2015 Load Case-2	2015 Load Case-3
Steel - lbs	304	287	376	330
Increase - %	0 %	- 6 %	24 %	9 %

COST COMPARISION ASD/LRFD – CALTRANS STD PLANS 2006 vs 2015

Concrete	$ 500.00 /CY
Steel	$ 2.00 /CY
Excavation	$ 50.00 /CY
Backfill	$ 40.00 /CY

COST PER LINEAR FT OF RETAINING WALL				
Description	CALTRANS STD PLAN			
	2006 All Load Cases	2015 Load Case-1	2015 Load Case-2	2015 Load Case-3
Cost per ft	$ 5120	$ 5240	$ 6600	$ 5660
Increase - %	0.00%	2.5 %	29 %	11 %

+ ROW Cost if Applicable

Western Bridge Engineers' Seminar

OUTLINE

T·Y·LIN INTERNATIONAL

- Geometry & Cost Comparison ASD/LRFD
- **Standard Walls vs Special Design Walls**
- LRFD Retaining Wall Design Background
- Summary of Reasons For Changes
- Other Type of Walls
- Conclusion

2006 - STANDARD PLAN WALL & XS SHEETS DESIGN CRITERIA

2006 STD PLAN

No Criteria is shown on Std Plans

From Our Past Practice,

- K_h = 0.3 (50% of Site Adjusted PGA = 0.6 g)
- Seismic Force @ 0.6 H
- Bearing Pressure – with single number of Allowable Bearing Pressure
- One Std Plan for All 3 Loading Cases

Earth Pressure Application - Caltrans

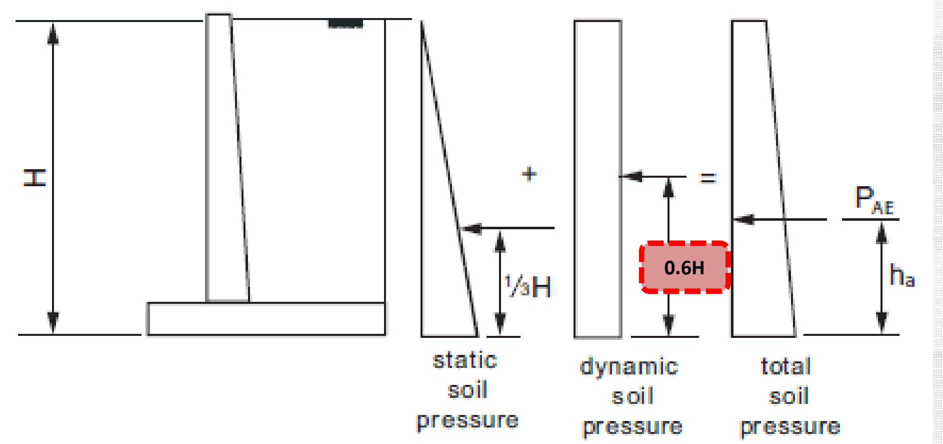


Figure 3 Seismic Loading
(Reference: 2010 California Amendments)

Earth Pressure Application - Caltrans

Western Bridge Engineers' Seminar

2015 - STANDARD PLAN WALL & XS SHEETS DESIGN CRITERIA

T·Y·LIN INTERNATIONAL

2015 STD PLAN

DESIGN NOTES:

DESIGN: AASHTO LRFD Bridge Design Specifications, 4th Edition with California Amendments

LS: Varied surcharge on level ground surface

DC: Stem Architectural Treatment of thickness up to 6" of concrete (75 psf) considered

CT: 54 kip transverse force applied at He = 32", distributed over 10 feet at the top of wall and 1 : 1 distribution down and outward. Distribution below footing taken no less than 40'.

SEISMIC: $k_h = 0.2$, $k_v = 0.0$

SOIL: $\phi = 34°$, $\gamma = 120$ pcf

REINFORCED CONCRETE: f'c = 3,600 psi, fy = 60,000 psi

LOAD COMBINATIONS AND LIMIT STATES:

Service I	Q = 1.00DC+1.00EV+1.00EH+1.00LS
Strength I	Q = αDC+βEV+ηEH+1.75LS
Extreme I	Q = 1.00DC+1.00EV+1.00EH+1.00EQD+1.00EQE
Extreme II	Q = 1.00DC+1.00EV+1.00EH+1.00CT

Where:

Q:	Force Effects
α:	1.25 or 0.90, Whichever Controls Design
β:	1.35 or 1.00, Whichever Controls Design
η:	1.50 or 0.90, Whichever Controls Design
DC:	Dead Load of Structure Components
EH:	Horizontal Earth Fill Pressure
EV:	Vertical Earth Pressure from Earth Fill Weight
LS:	Live Load Surcharge
EQE:	Seismic Earth Pressure
EQD:	Soil and Structural and Nonstructural Components Inertia
CT:	Vehicular Collision Force

Different Std Plan for Each load Cases

2014 Dated XS SHEETS

DESIGN DATA

Design: AASHTO LRFD Bridge Design Specifications, 4th edition with California Amendments

WS: 33 psf on sound wall

LS: Varied surcharge on level ground surface

EQE: Mononabe-Okabe Method, $K_h = 0.3$, $K_v = 0.0$

Soil: $\phi = 34°$, $\gamma = 120$ pcf

Reinforced Concrete: f'c = 3600 psi, fy = 60,000 psi

Load Combinations and Limit States

Service I	Q=1.00DC+1.00EV+1.00EH+1.00LS+0.30WS
Service II	Q=1.00DC+1.00EV+1.00EH+1.00WS
Strength I	Q=αDC+βEV+1.50EH+1.75LS
Strength III	Q=αDC+βEV+1.50EH+1.40WS
Strength V	Q=αDC+βEV+1.50EH+1.35LS+0.40WS
Extreme I	Q=1.00DC+1.00EV+1.00EH+1.00EQD+1.00EQE

Earth Pressure Application - Caltrans

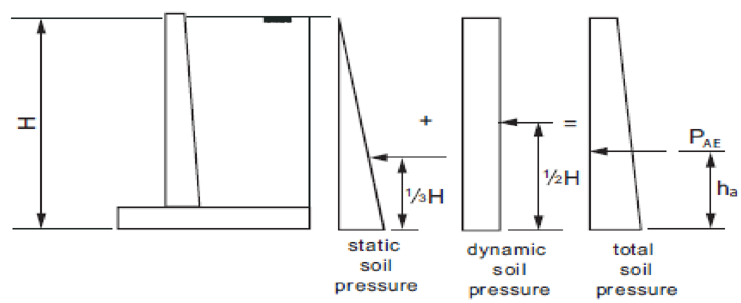


Figure 3 Seismic Loading
(Reference: 2010 California Amendments)

SPECIAL DESIGN ?

Geometry Constrains ?

Seismic $K_h \geq 0.2$?

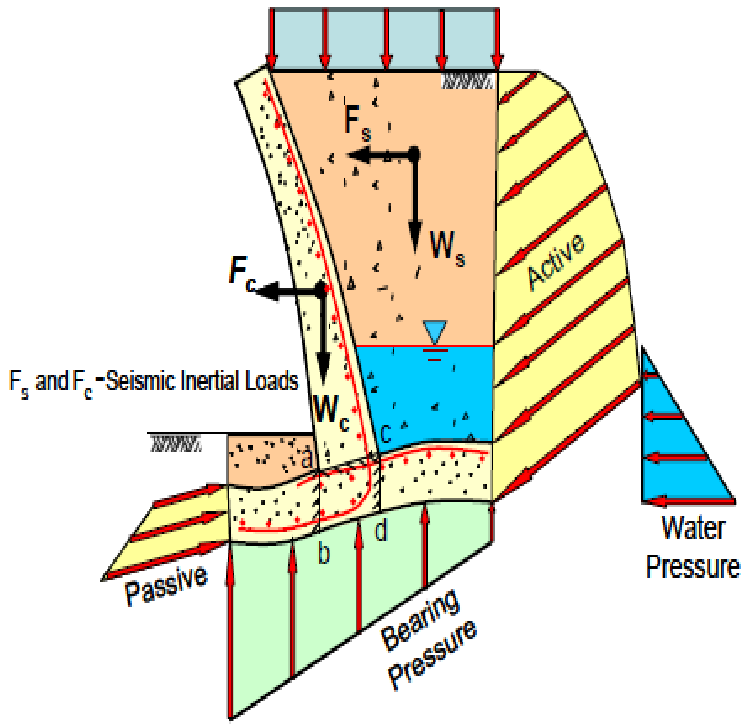
Bearing Pressure Capacity – NG ?

SPECIAL DESIGN

OUTLINE

- Geometry & Cost Comparison ASD/LRFD
- Standard Walls vs Special Design Walls
- **LRFD Retaining Wall Design Background**
- Summary of Reasons For Changes
- Other Type of Walls
- Conclusion

LRFD DESIGN CRITERIA



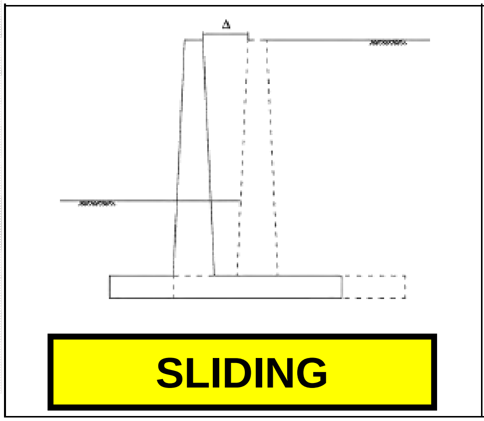
a. Loading

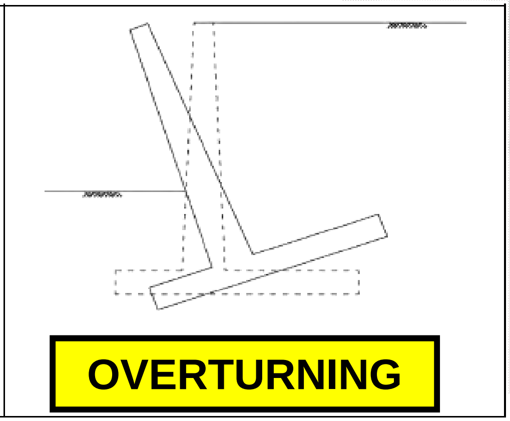
Strength/ Service/ Extreme Load Cases

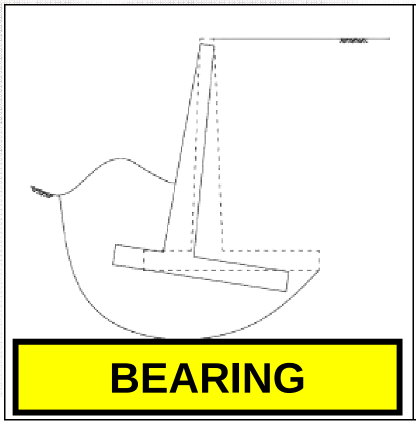
b. External Stability

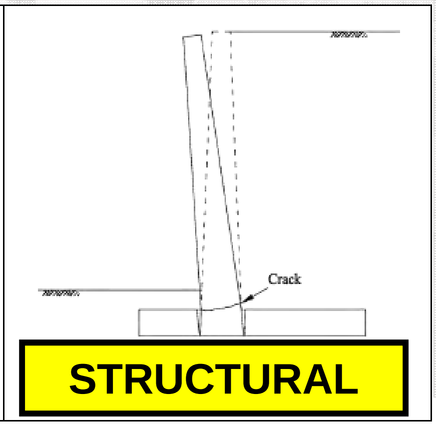
c. Structural Design

REF: FHWA-NHI-11-032





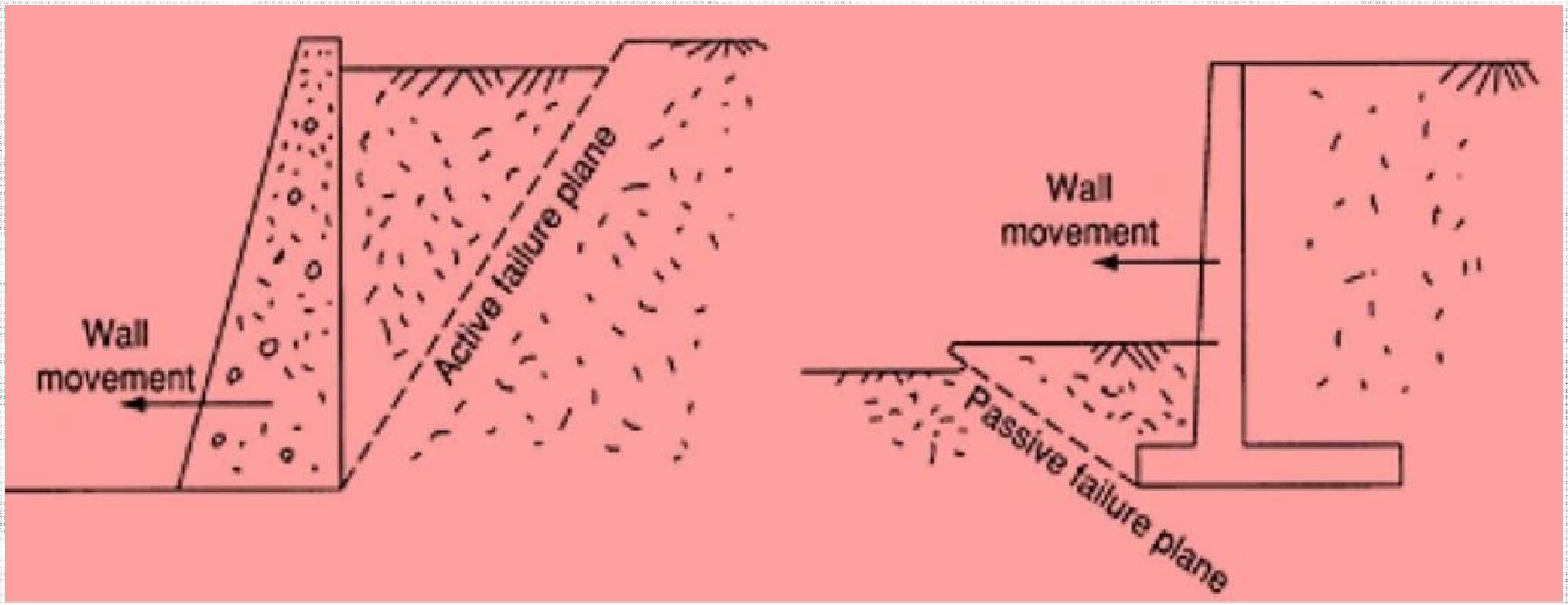




STATIC EARTH PRESSURE

a. **Active Earth Pressure**

b. **Passive Earth Pressure**



STATIC EARTH PRESSURE

Rankine Theory

$$K_a = \cos\beta \frac{\cos\beta - (\cos^2\beta - \cos^2\phi)^{1/2}}{\cos\beta + (\cos^2\beta - \cos^2\phi)^{1/2}}$$

$$K_p = \cos\beta \frac{\cos\beta + (\cos^2\beta - \cos^2\phi)^{1/2}}{\cos\beta - (\cos^2\beta - \cos^2\phi)^{1/2}}$$

For the case where β is 0, the above equations simplify to

$$K_a = \tan^2\left(45 - \frac{\phi}{2}\right) = \frac{1 - \sin(\phi)}{1 + \sin(\phi)}$$

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right) = \frac{1 + \sin(\phi)}{1 - \sin(\phi)}$$

Coulomb Theory

$$K_a = \frac{\cos^2(\phi - \theta)}{\cos^2\theta \cos(\delta + \theta)\left(1 + \sqrt{\frac{\sin(\delta+\phi)\sin(\phi-\beta)}{\cos(\delta+\theta)\cos(\beta-\theta)}}\right)^2}$$

$$K_p = \frac{\cos^2(\phi + \theta)}{\cos^2\theta \cos(\delta - \theta)\left(1 - \sqrt{\frac{\sin(\delta+\phi)\sin(\phi+\beta)}{\cos(\delta-\theta)\cos(\beta-\theta)}}\right)^2}$$

Trial Wedge Method

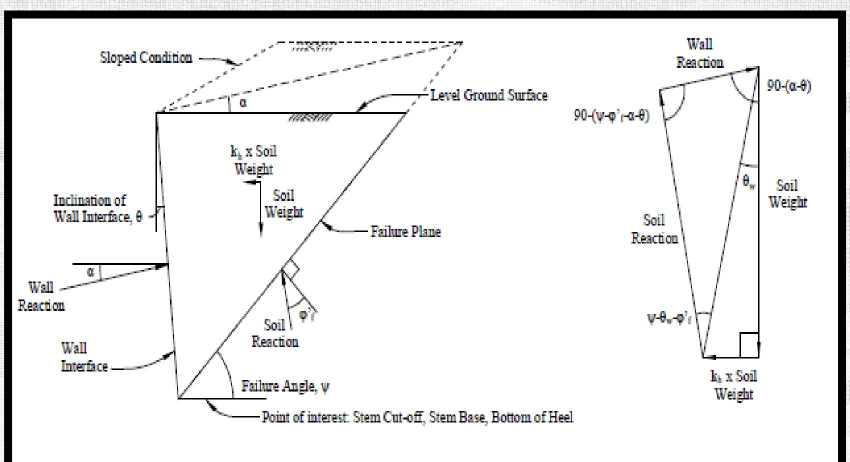
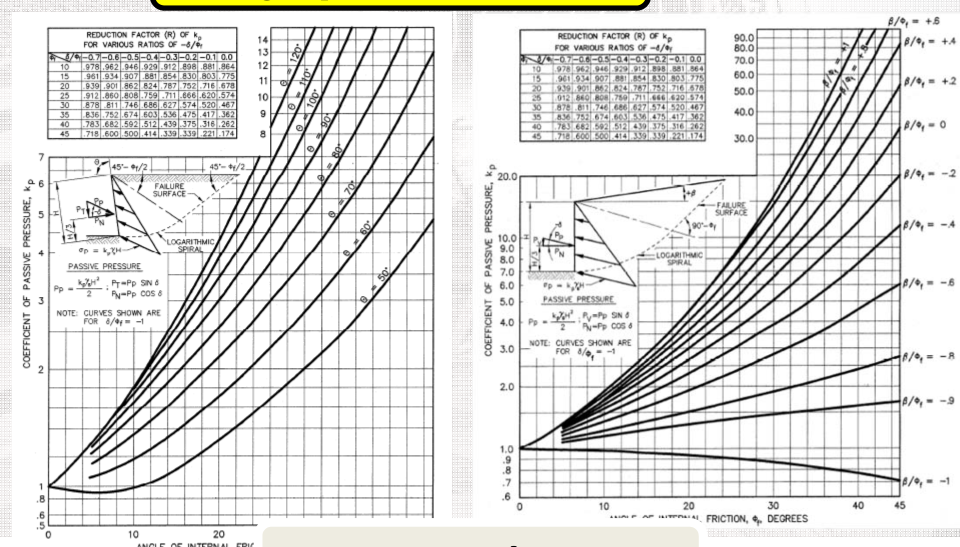


Figure 1.1.1-5 Trial Wedge Terminology

Log Spiral Method



AASHTO –Chapter-3

STATIC EARTH PRESSURE

Western Bridge Engineers' Seminar

T·Y·LIN INTERNATIONAL

Why?

1. Any friction angle/Slope
2. Broken Slope
3. Cohesive Soils
4. Multi-layer Soil

Why?

1. Curve Failure Surface
2. Wedge solutions are unconservative for large friction angles
3. 5% H Movement required to mobilize full passive pressure

Trial Wedge Method

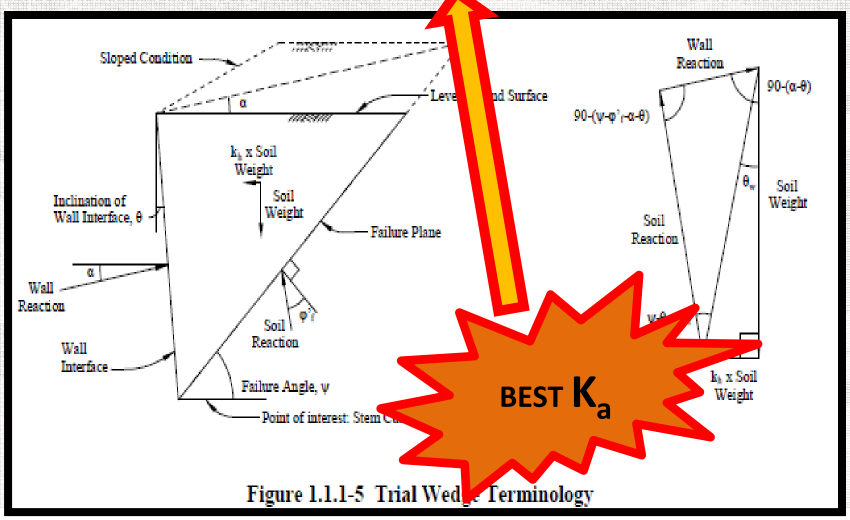
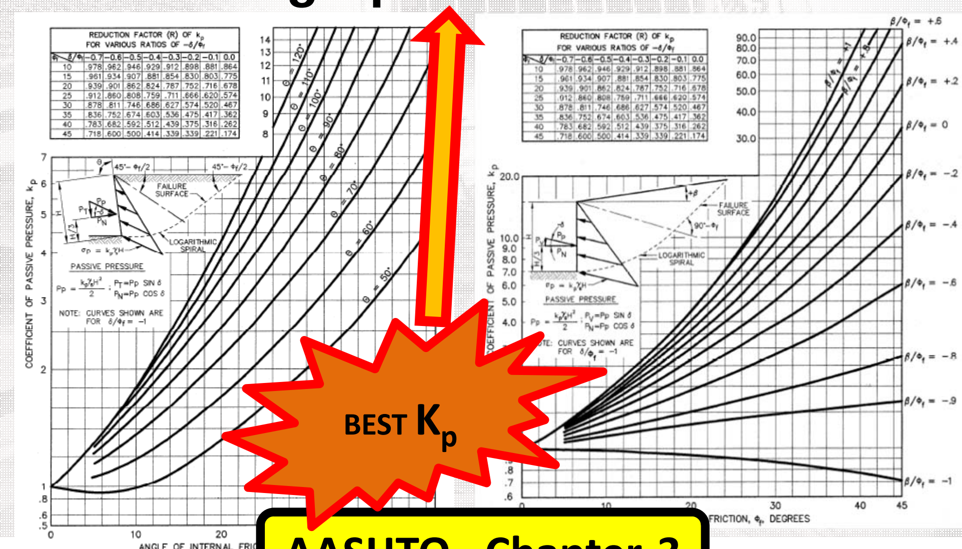


Figure 1.1.1-5 Trial Wedge Terminology

BEST K_a

Log Spiral Method

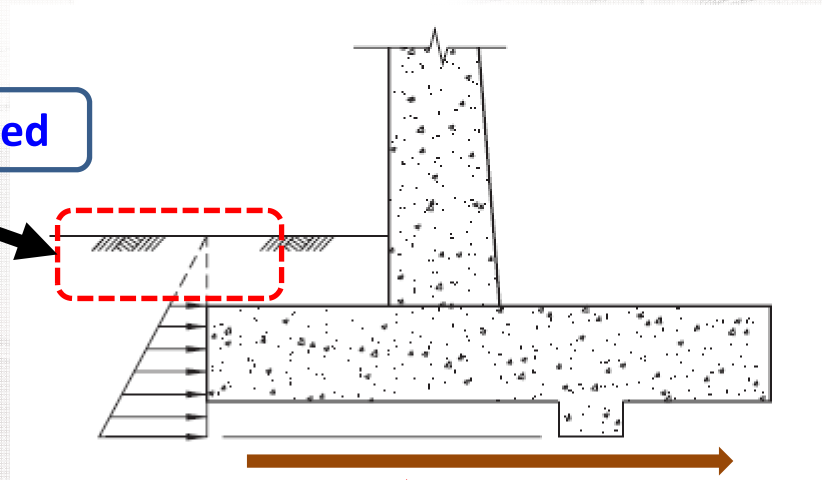


BEST K_p

AASHTO –Chapter-3

PASSIVE EARTH PRESSURE & FRICTION FORCE

Resistance Not Considered



FRICTION FORCE

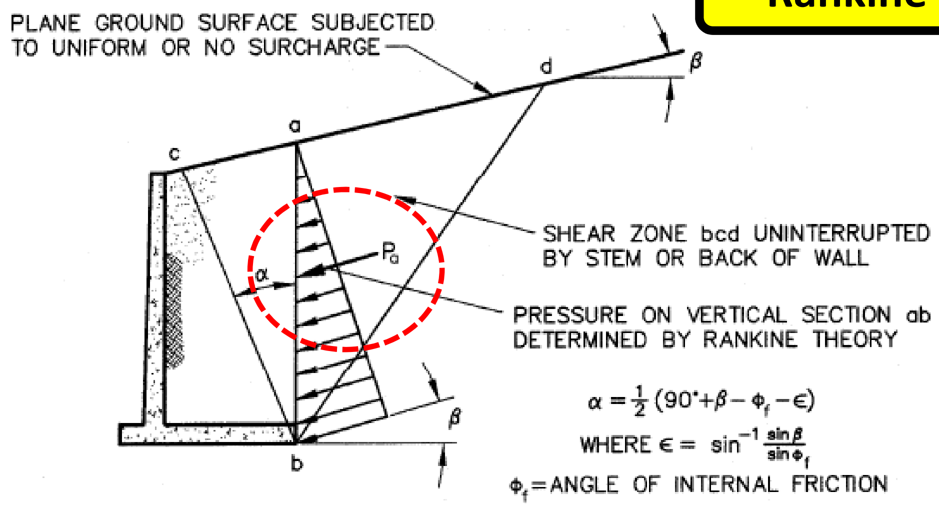
FRICTION FORCE:

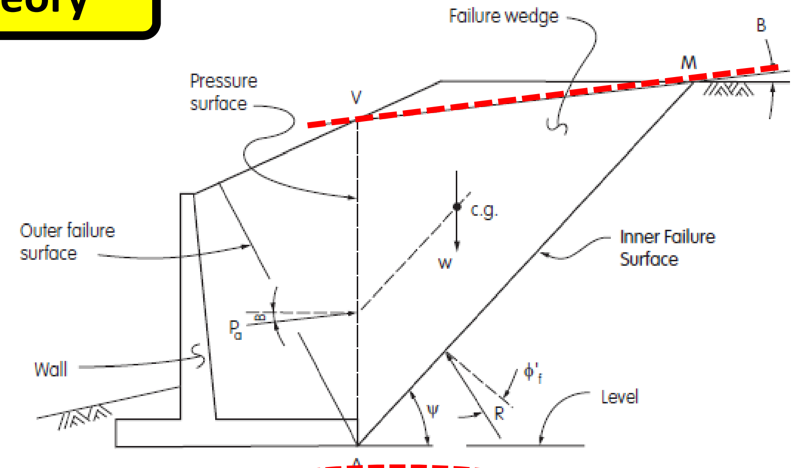
1. Based on Soil-Footing Friction Force
2. Same Friction Coefficient for Entire Length of Wall

$$\mu = \tan\left(\frac{2}{3}\ \mathit{Friction\ Ange}\right)$$

DIRECTION OF EARTH PRESSURE

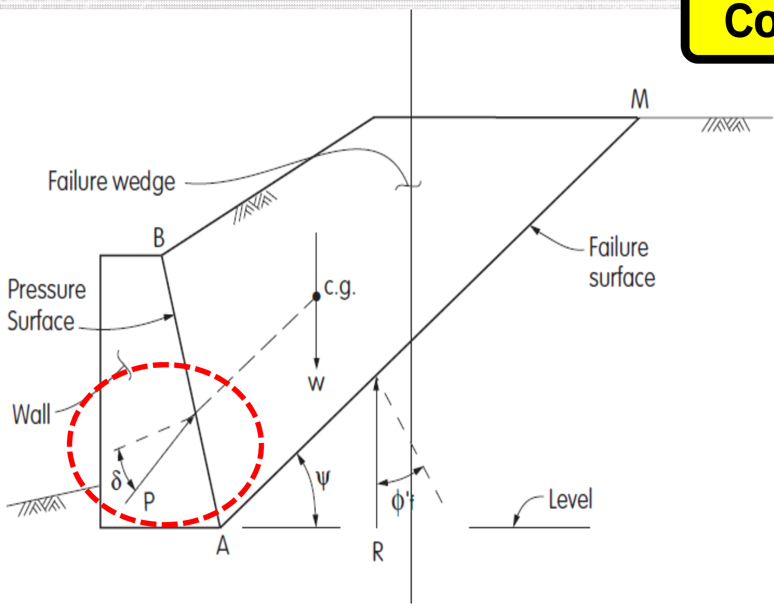
Rankine Theory

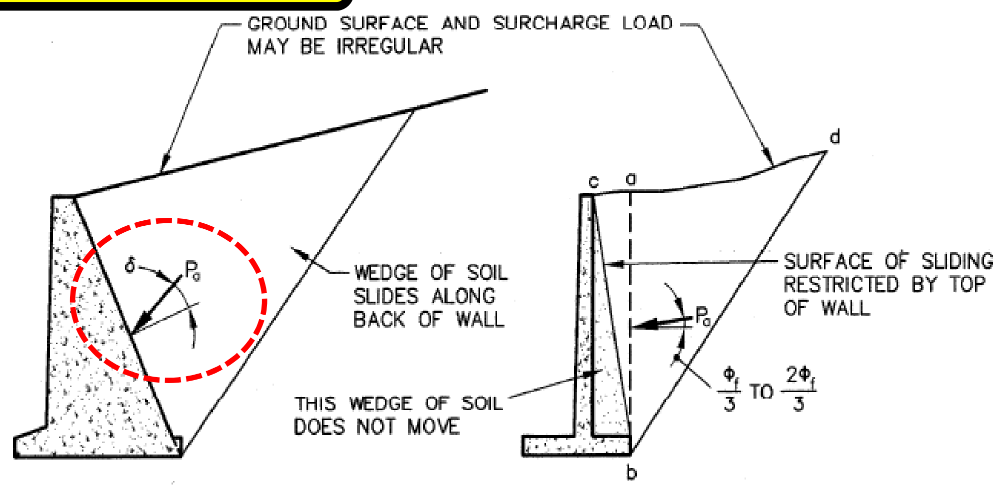




The direction of, P_a, is parallel to a line, VM

Coulomb Theory





SURCHARGE LOAD

Different in LFD & LRFD

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

NEW IN LRFD

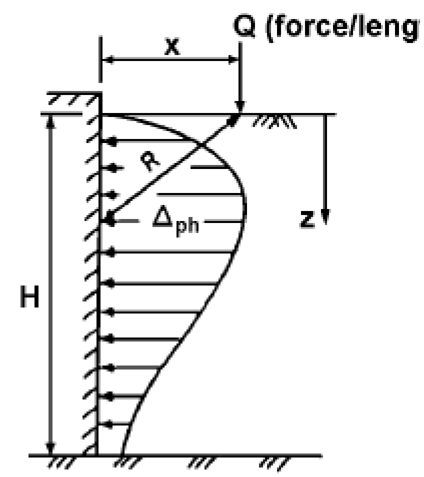
Table 3.11.6.4-2—Equivalent H... ...lar Loading on Retaining Walls Paral...

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

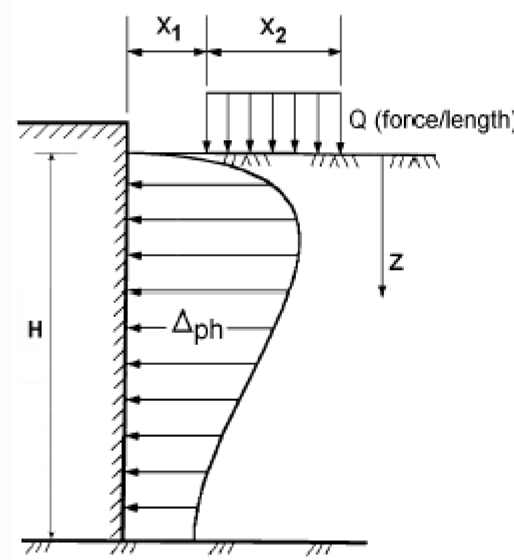
The load factor for both vertical and horizontal components of live load surcharge shall be taken as specified in Table 3.4.1-1 for live load surcharge.

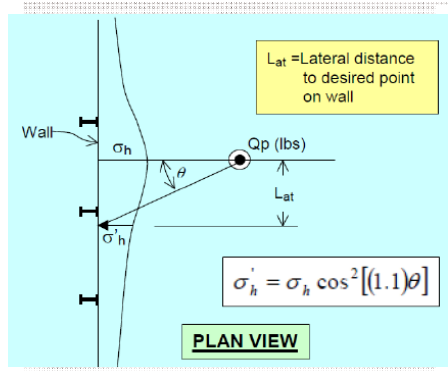
LFD – 2ft Surcharge Always

Point Load



Uniform Load





$$\Delta_{ph} = \frac{Q}{\pi Z}\left(\frac{1}{A^3} - \frac{1-2\nu}{A+\frac{Z}{X_2}} - \frac{1}{B^3} + \frac{1-2\nu}{B+\frac{Z}{X_1}}\right) \quad (3.11.6.2\text{-}4)$$

in which:

$$A = \sqrt{1+\left(\frac{Z}{X_2}\right)^2} \quad (3.11.6.2\text{-}5)$$

$$B = \sqrt{1+\left(\frac{Z}{X_1}\right)^2} \quad (3.11.6.2\text{-}6)$$

$$\Delta_{ph} = \frac{4Q}{\pi}\frac{X^2 Z}{R^4} \quad (3.11.6.2\text{-}3)$$

SURCHARGE LOAD CONT

BACKFACE OF WALL

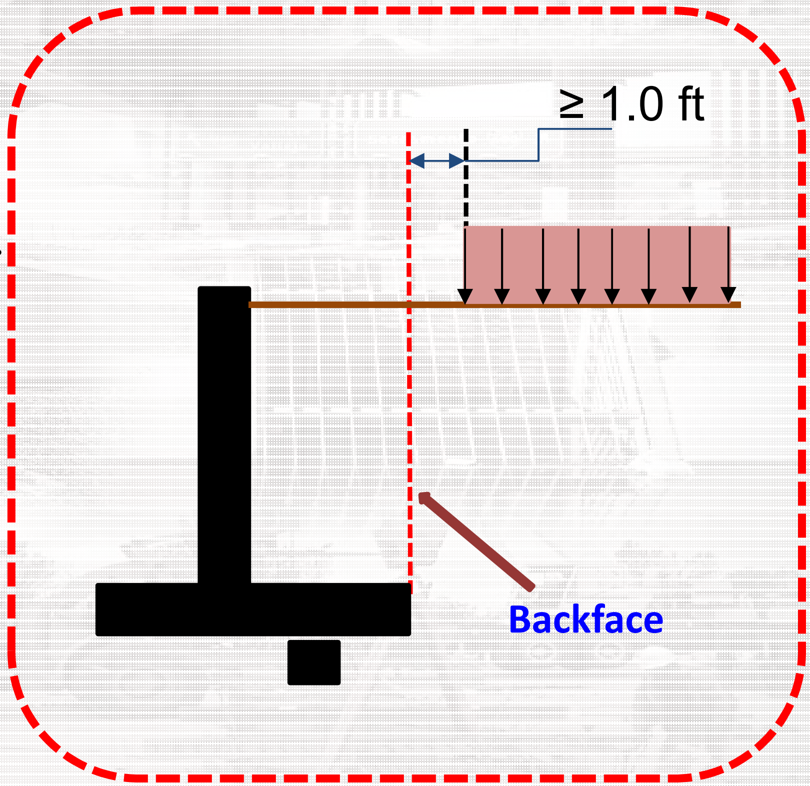
Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

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

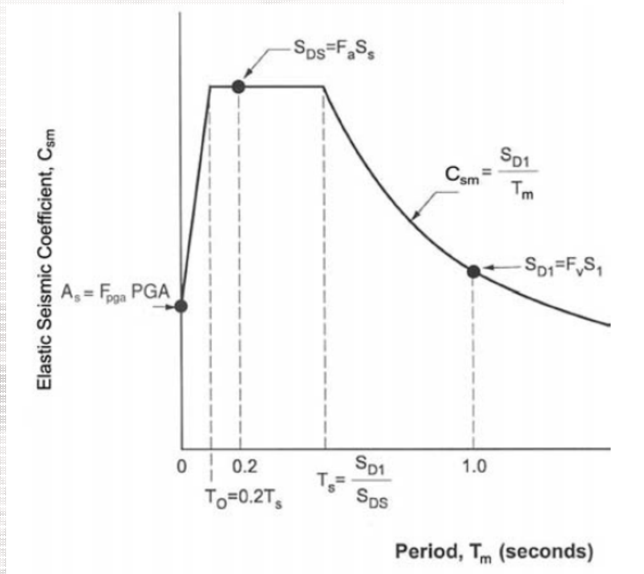
Table 3.11.6.4-2—Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic

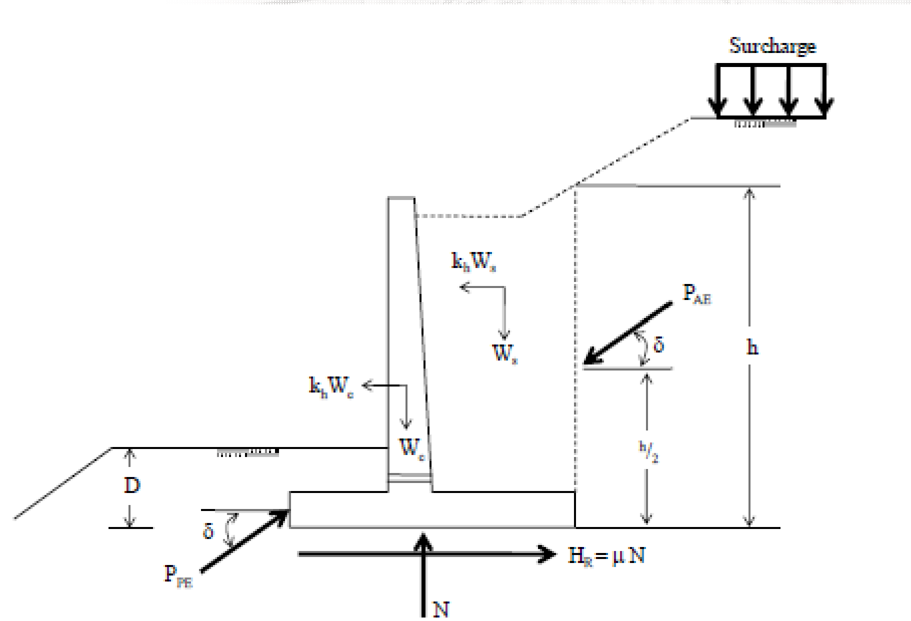
Retaining Wall Height (ft)	h_{eq} (ft) Distance from wall backface to edge of traffic	
	0.0 ft	1.0 ft or Further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

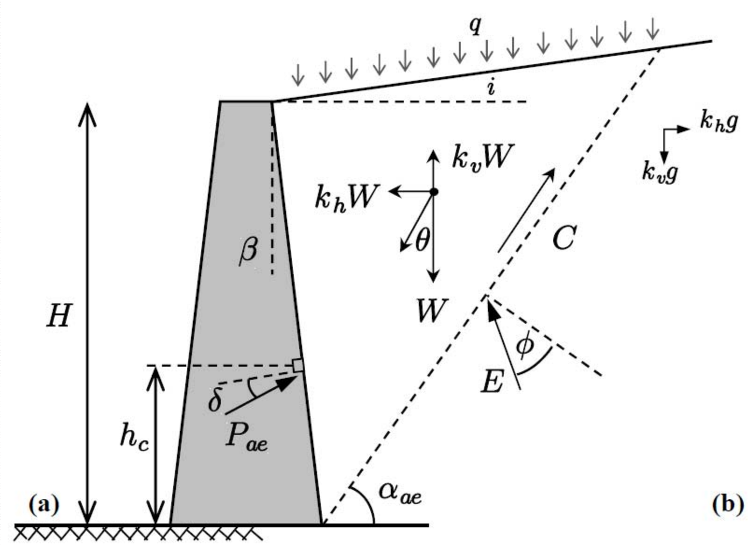
The load factor for both vertical and horizontal components of live load surcharge shall be taken as specified in Table 3.4.1-1 for live load surcharge.



SEISMIC EARTH PRESSURE







What is K_h?

50% of Adj. PGA ?

CURRENT RW SEISMIC CRITERIA

K_h = Coefficient of Horizontal Acceleration

AASHTO LRFD 11.5.4.2

11.5.4.2—Extreme Event I, No Analysis

A seismic design shall not be considered mandatory for walls located in Seismic Zones 1 through 3, or for walls at sites where the site adjusted peak ground acceleration, A_s, is less than or equal to 0.4*g*, unless one or more of the following is true:

- Liquefaction induced lateral spreading or slope failure, or seismically induced slope failure, due to the presence of sensitive clays that lose strength during the seismic shaking, may impact the stability of the wall for the design earthquake.
- The wall supports another structure that is required, based on the applicable design code or specification for the supported structure, to be designed for seismic loading and poor seismic performance of the wall could impact the seismic performance of that structure.

The no-seismic-analysis option should be limited to internal and external seismic stability design of the wall. If the wall is part of a bigger slope, overall seismic stability of the wall and slope combination should still be evaluated.

These no-seismic-analysis provisions shall not be considered applicable to walls functioning as support piers for bridges.

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: ALL STAFF
Geotechnical Services
Division of Engineering Services

Date: June 13, 2013

From: PHILIP J. STOLARSKI
State Materials Engineer
Deputy Division Chief
Materials Engineering and Testing Services
and Geotechnical Services
Division of Engineering Services

Subject: **Seismic Design and Selection of Standard Retaining Walls**

When providing geotechnical recommendations for type selection of retaining walls during planning and design phases, the job site should be evaluated to ensure seismic design criteria used for development of the LRFD standard plans are applicable.

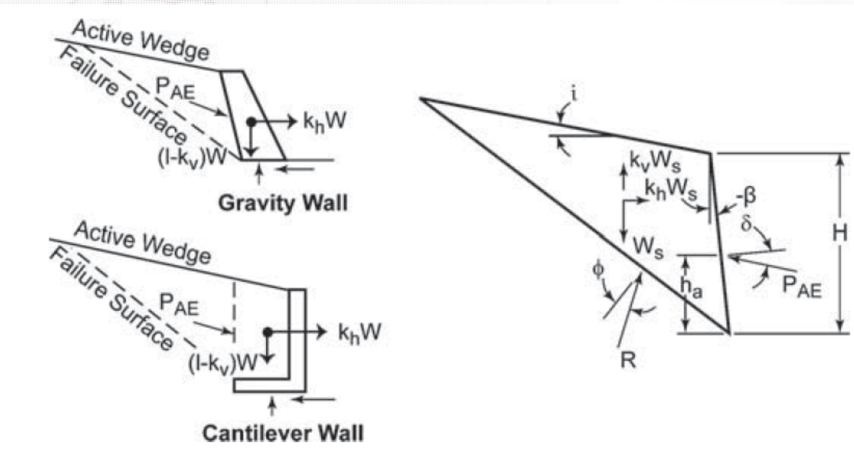
According to standard plan sheets dated April 2012, the seismic criteria threshold for standard retaining walls are; Coefficient of Horizontal Acceleration, $k_h = 0.2$ and Coefficient of Vertical Acceleration $k_v = 0.0$, except for concrete retaining walls supporting soundwalls where $k_h = 0.3$ and $k_v = 0.0$ are used. The $k_h = 0.2$ is roughly based on using 1/3 Peak Ground Acceleration (PGA), therefore, at sites where the PGA is equal to or less than 0.6g, the retaining walls shown in the Standard Plans are applicable. For sites with PGA greater than 0.6g, the standard plans are not applicable, and DES/Structure Design should design the retaining walls as special design walls. Include the seismic assessment in geotechnical reports to the District Project Engineer as early as possible during planning or design phases of the project development process, so that appropriate functional units can be notified and resources be allocated.

Adj PGA ≤ 0.4 g => NON SEISMIC WALL

K_h = 1/3* Adj PGA

SEISMIC EARTH PRESSURE

Mononobe-Oakabe Method



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

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

Seismic Active Earth Pressure

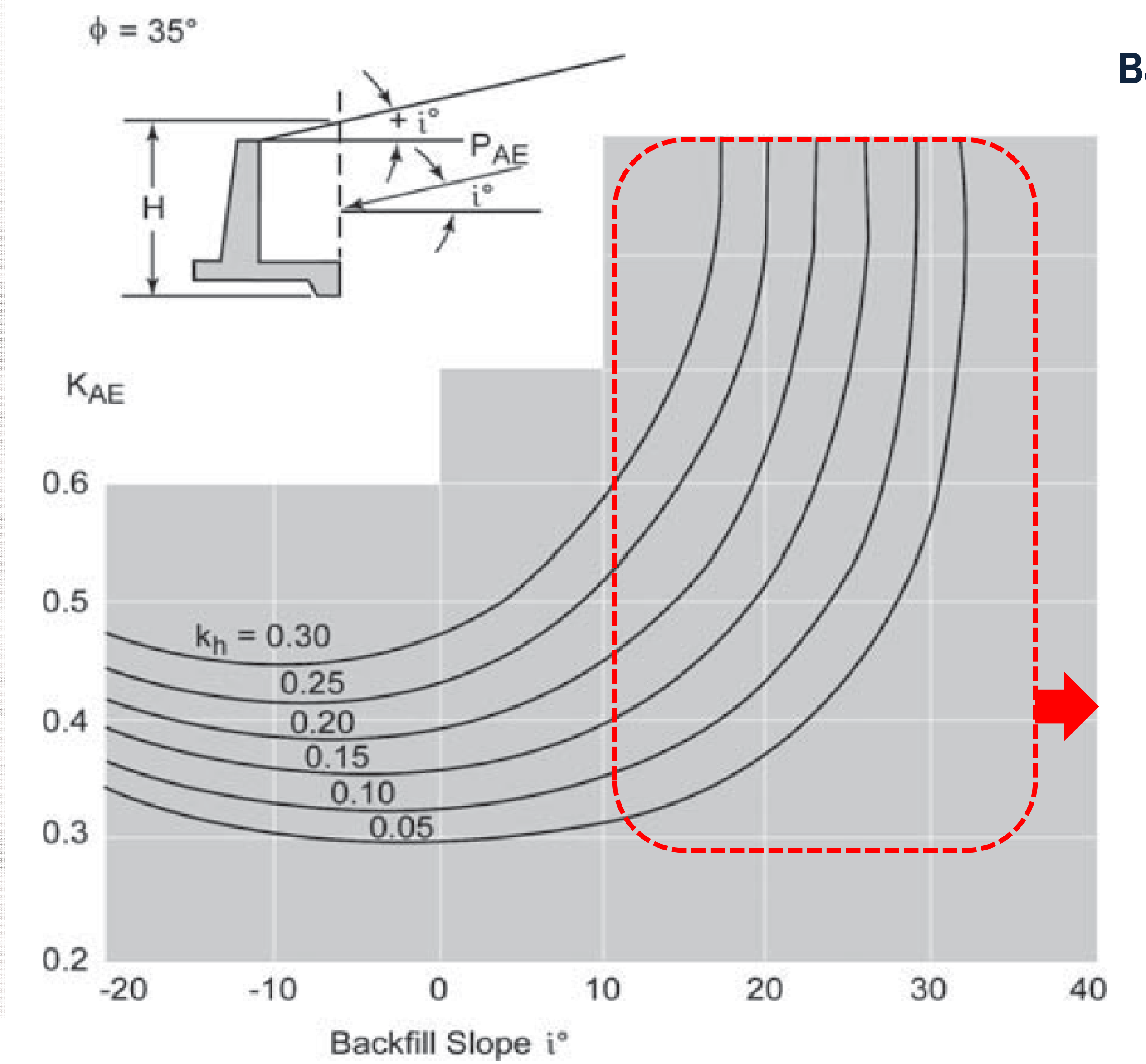
$$P_{AE} = 0.5\,\gamma\,H^2(1-k_v)K_{AE}$$

Seismic Passive Earth Pressure

$$P_{PE} = 0.5\,\gamma\,H^2(1-k_v)K_{PE}$$

γ = unit weight of soil (ksf)
H = height of wall (ft)
ϕ = friction angle of soil (°)
θ = arc tan $(k_h/(1-k_v))$(°)
δ = angle of friction between soil and wall (°)
k_h = horizontal acceleration coefficient (dim.)
k_v = vertical acceleration coefficient (dim.)
i = backfill slope angle (°)
β = slope of wall to the vertical, negative as shown (°)

SEISMIC EARTH PRESSURE vs BACKFILL SLOPE



Based on Mononobe-Oakabe (M-O) Equation

- **Only Cohesion less Soils**
- **Only Constant Slope**
- **Not to use for Passive Seismic Earth Pressure (AASHTO 11.6.5.5)**
- **Modest Increase with Slope Angle & excessively Conservative**

REF: NCHRP Report – NR611

Mononobe Oakabe Theory - Seismic Modification

$$P_{AE}=\frac{W\left[(1-k_v)\tan(\alpha-\phi)+k_h\right]-C\,L\,\left[\sin\alpha\tan(\alpha-\phi)+\cos\alpha\right]-C_A\,H\left[\tan(\alpha-\phi)\cos\omega+\sin\omega\right]}{\left[1\;+\;\tan(\delta+\omega)\tan(\alpha-\phi)\right]*\cos(\delta+\omega)}\qquad\text{A11.3.2-1}$$

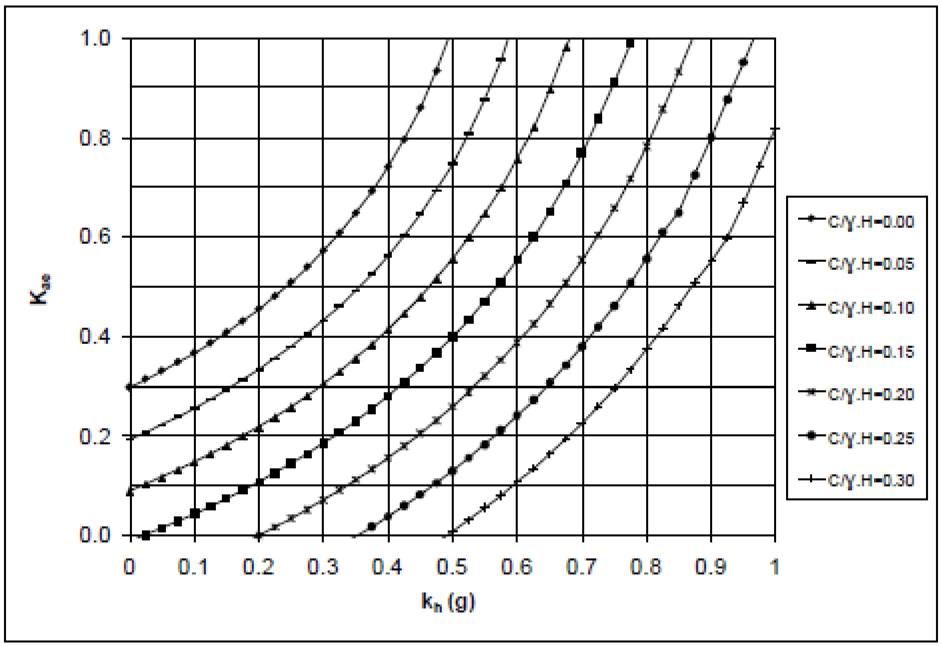


Figure A11.3.2-2—Seismic Active Earth Pressure Coefficient for ϕ = 30 degrees (c = soil cohesion, γ = soil unit weight, and H = retaining wall height)

Note: $k_h = A_s = k_{h0}$ for wall heights greater than 20 ft. This could be H or h as defined in Figure A11.3.1-1.

ϕ = 30 degrees

REF: AASHTO LRFD Chapt. 11

RETAINING WALL DESIGN

Mononobe Oakabe Theory - Seismic Modification

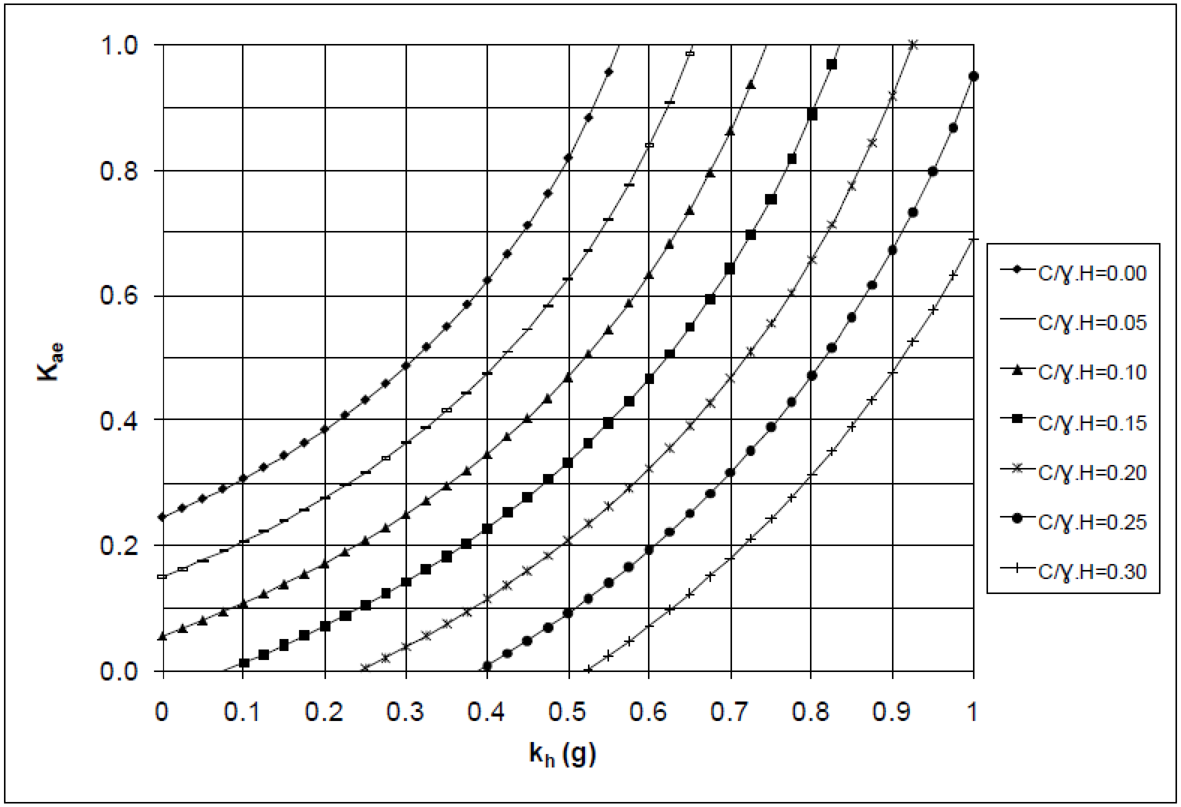


Figure A11.3.2-3—Seismic Active Earth Pressure Coefficient for ϕ = 35 degrees (c = soil cohesion, γ = soil unit weight, and H = retaining wall height)

$\phi = 35$ degrees

REF: AASHTO LRFD Chapt. 11

RETAINING WALL DESIGN

Mononobe Oakabe Theory - Seismic Modification

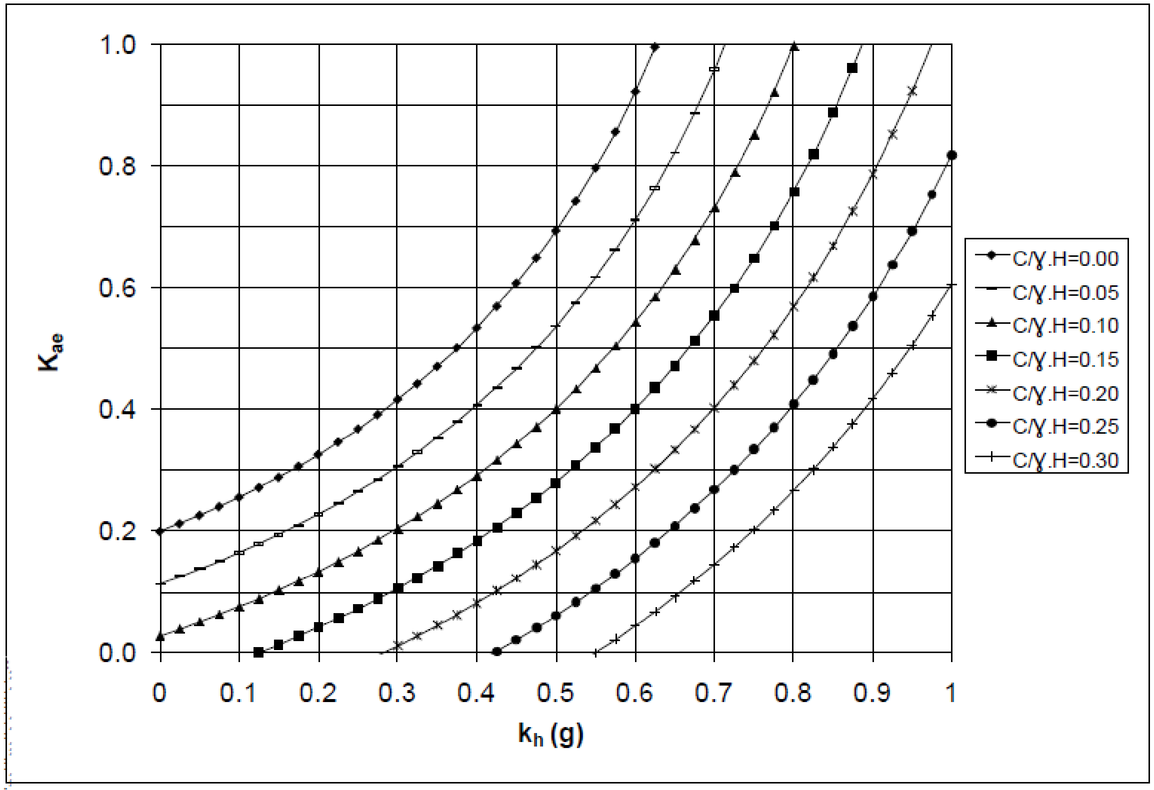


Figure A11.3.2-4—Seismic Active Earth Pressure Coefficient for ϕ = 40 degrees (c = soil cohesion, γ = soil unit weight, and H = retaining wall height)

$\phi = 40$ degrees

REF: AASHTO LRFD Chapt. 11

RETAINING WALL DESIGN

Seismic Passive Pressure-Modification for Cohesion

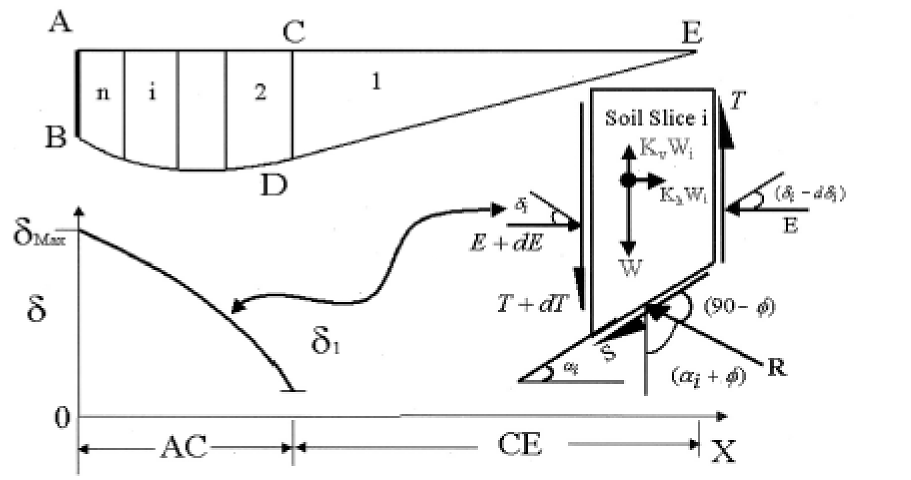


Figure A11.4-1—Limits and Shape Seismic Interslice Force Function (reported in Anderson et al., 2008)

$$dE_i = \frac{W_i(1-K_v)[\tan(\alpha_i+\phi)-K_h] + C\,L_i\,[\sin\alpha_i\tan(\alpha_i+\phi)+\cos\alpha_i]}{[1-\tan\delta_i\tan(\alpha_i-\phi)]*\cos\delta_i} \quad \text{(A11.4-1)}$$

$$P_pn = \frac{\sum_1^i dE}{[1-\tan\delta_w\tan(\alpha_w-\phi)]*\cos\delta_w} \quad \text{(A11.4-2)}$$

$$K_pn = \frac{2P_p}{\gamma h^2} \quad \text{(A11.4-3)}$$

REF: AASHTO LRFD Chapt. 11

RETAINING WALL DESIGN

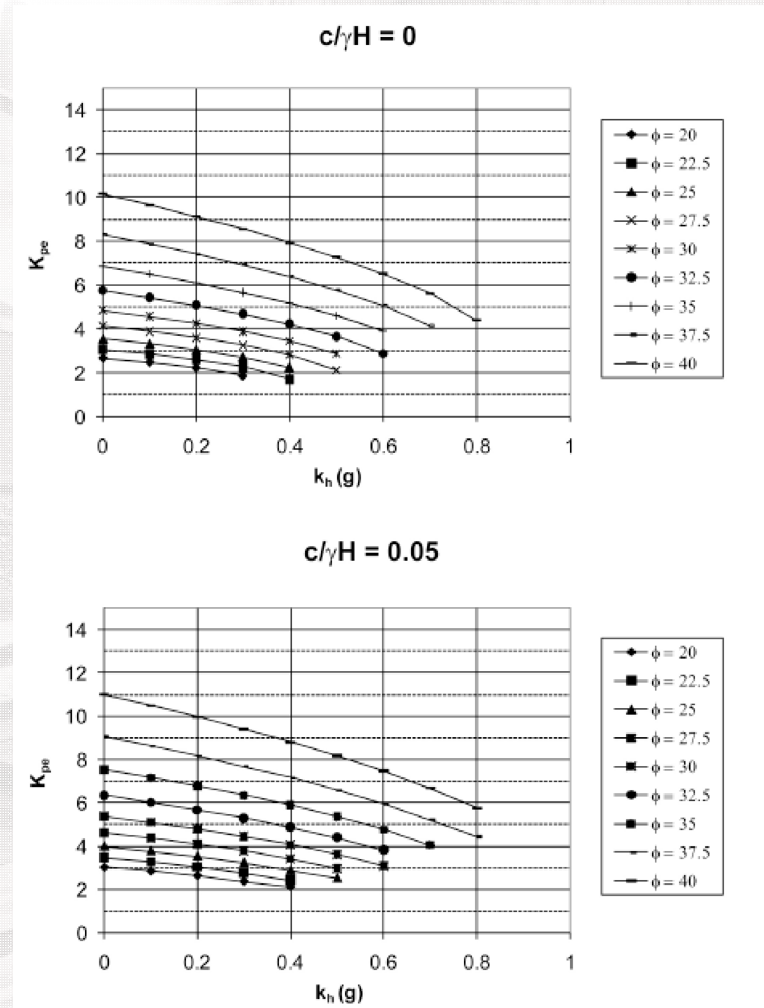
Seismic Passive Pressure-Modification for Cohesion

Based on Log Spiral Procedure

C = Cohesion

H = Height of Wall

g =Soil Unit Weight



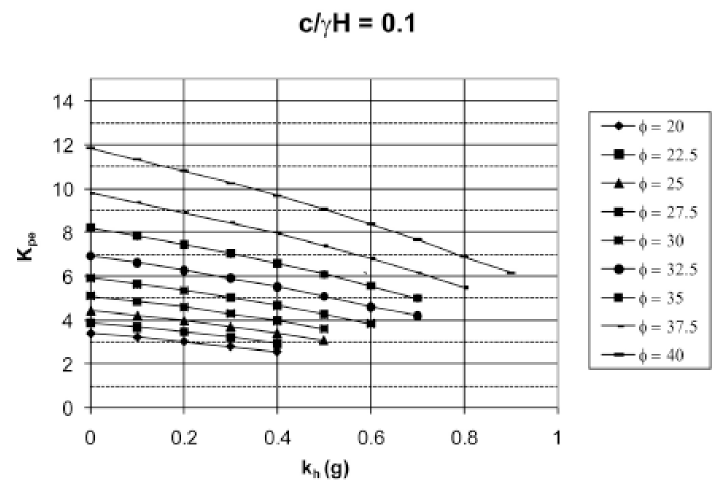
REF: AASHTO LRFD Chapt. 11

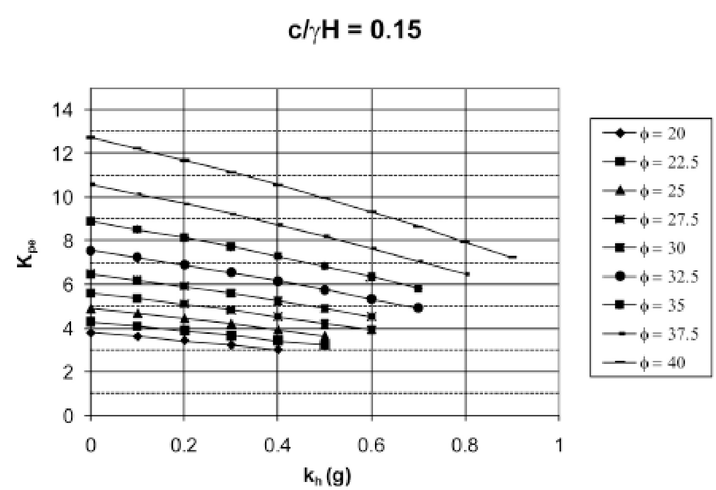
Western Bridge Engineers' Seminar

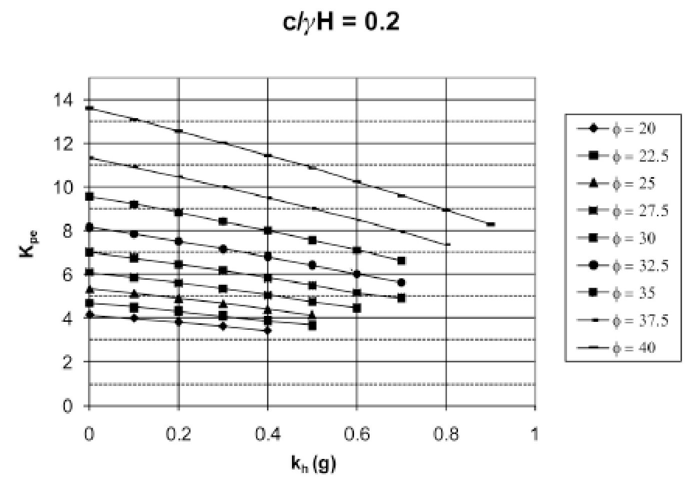
RETAINING WALL DESIGN

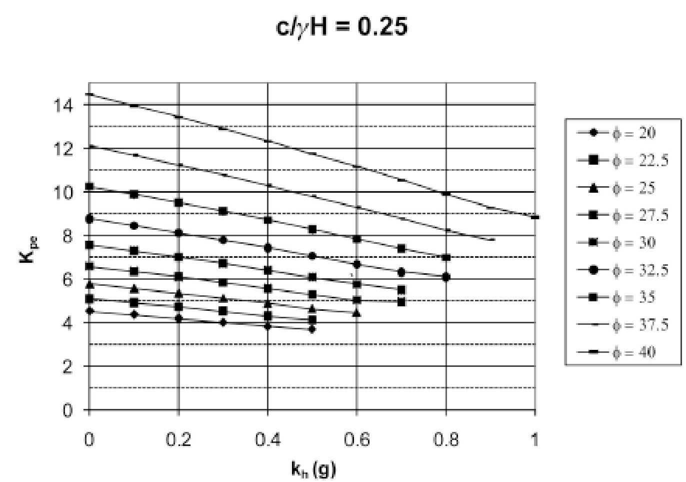
T·Y·LIN INTERNATIONAL

Seismic Passive Pressure-Modification for Cohesion



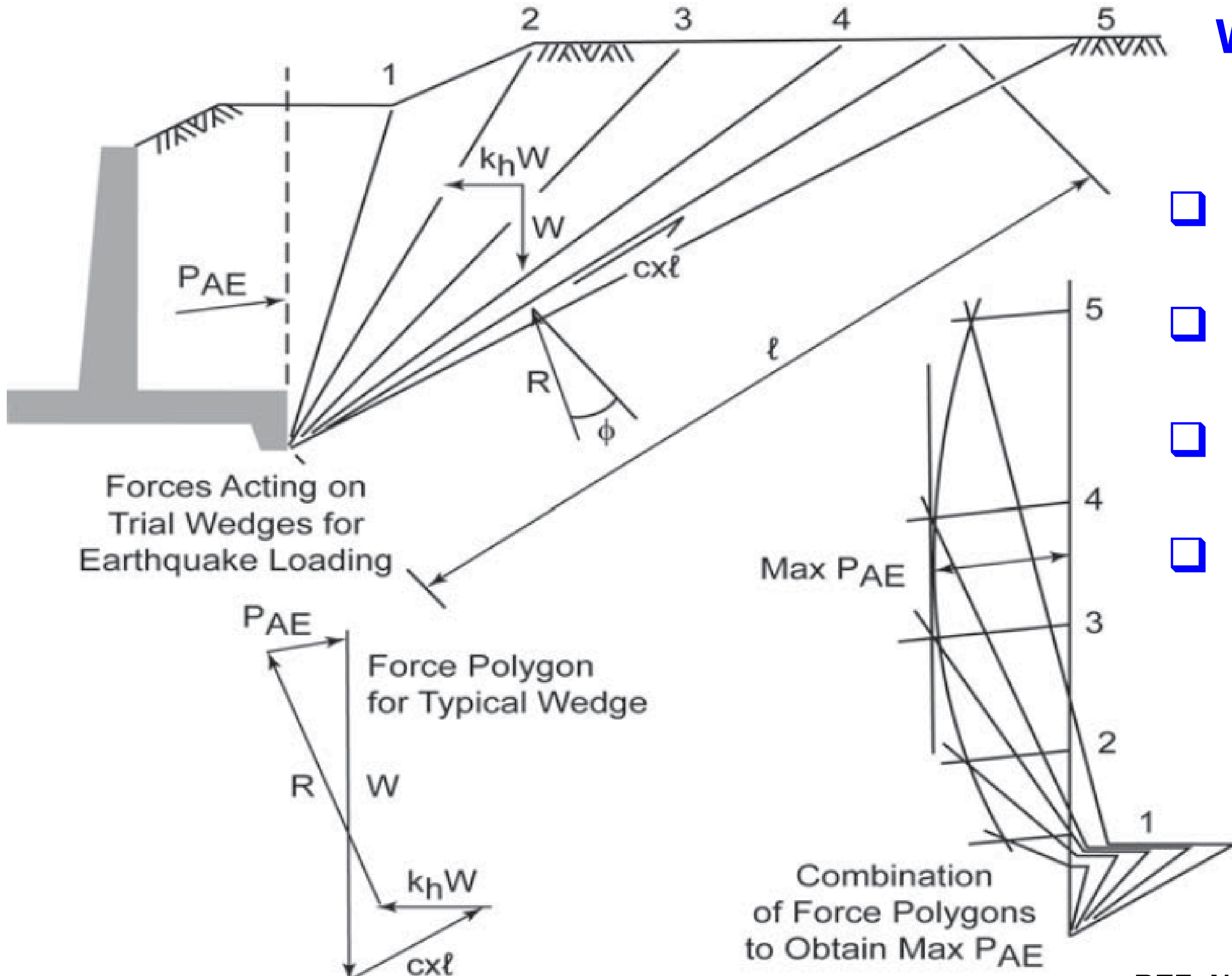






REF: AASHTO LRFD Chapt. 11

TRIAL WEDGE - SEISMIC



Why Trial Wedge Method?

- Broken Slope
- Cohesion
- Any Geometry
- Multilayer Soil

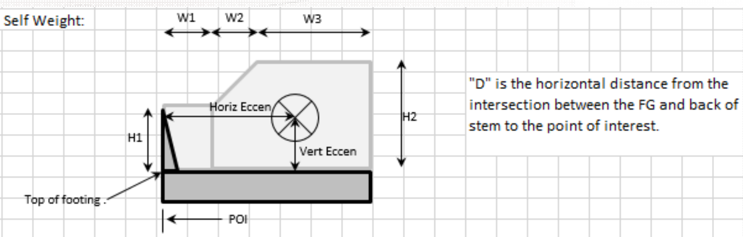
REF: NCHRP Report – NR611

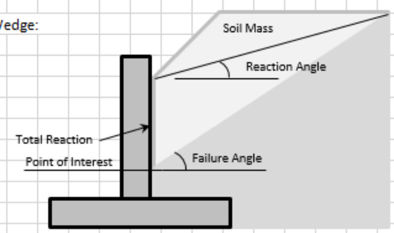
TRAIL WEDGE – NOT A BLACKBOX

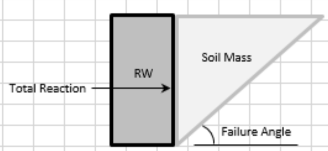
Western Bridge Engineers' Seminar

T.Y.LIN INTERNATIONAL

EXCEL MACRO







Minimum trail wedge failure angle: 0.4445 rad

Trial wedge iteration step size: 0.01 rad

Run Trial Wedge Analysis

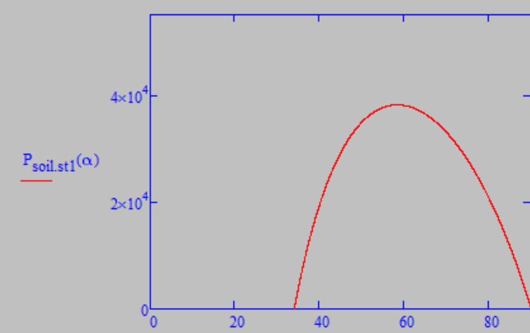
Static and Seismic Trial Wedge analysis and Soil Weight

Stem Static Soil Pressure (Strength/Service):

H	POI	Soil H	Angle to Toe	Angle to Top	Failure Angle	React Angle	Wt of Mass	Cohes Length	Total React.	P_{Ls}	P_{As}
(ft)	(ft)	(ft)	(rad)	(rad)		(rad)	(K)	(ft)	(K/ft)	(kips)	(kips)
31.0	0.00	31.00	1.326	1.113	1.040	0.211	42.20	41.7	18.91	0.25	0.12
	2.61	28.39	1.298	1.078	1.005	0.211	38.60	39.6	15.89	0.25	0.12
	7.50	23.50	1.232	1.000	1.035	0.220	23.97	32.0	10.49	0.00	0.00
	12.40	18.60	1.136	0.902	1.025	0.138	14.78	24.0	6.45	0.00	0.00
	15.50	15.50	1.051	0.827	1.015	0.050	10.17	18.9	4.46	0.00	0.00
	21.70	9.30	0.784	0.637	0.990	0.000	3.78	11.1	1.58	0.00	0.00
	24.80	6.20	0.576	0.520	0.870	0.000	2.11	8.1	0.60	0.00	0.00

MATHCAD – STATIC EARTH

$$P_{soil.st1}(\alpha) := \frac{W_s(\alpha)\cdot\sin(\alpha-\phi_1) - F_c(\alpha)\cdot\cos(\phi_1)}{\cos(\alpha-\phi_1-\delta_1)}$$

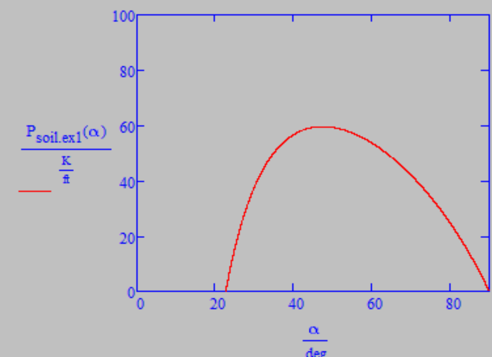


$$f(\alpha) := \frac{d}{d\alpha}P_{soil.st1}(\alpha) \qquad \frac{\alpha}{deg}$$

$$\alpha_{sta.fail} := root(f(\alpha), \alpha, 30deg, 90deg) = 58.352\cdot deg$$

MATHCAD – SEISMIC EARTH

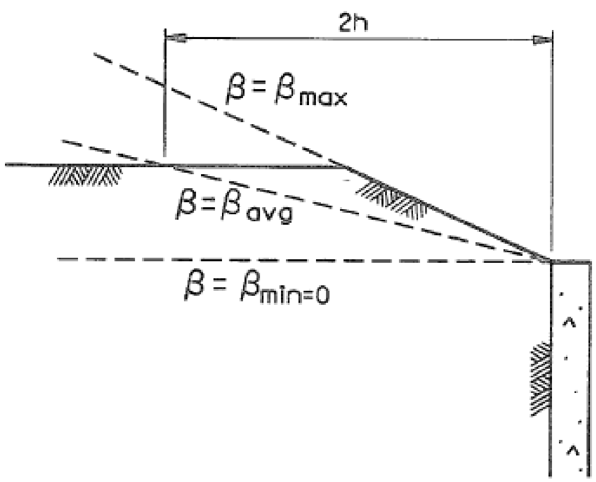
$$P_{soil.ex1}(\alpha) := \frac{W_s(\alpha)\cdot\sin(\alpha-\phi_1) + W_s(\alpha)\cdot K_h\cdot\cos(\alpha-\phi_1) - F_c(\alpha)\cdot\cos(\phi_1)}{\cos(\alpha-\phi_1-\delta_1)}$$



$$d(\alpha) := \frac{d}{d\alpha}P_{soil.ex1}(\alpha)$$

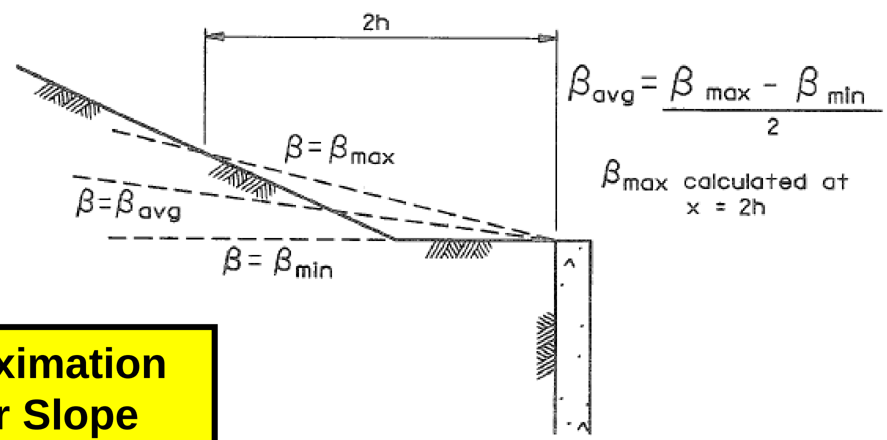
$$\alpha_{ext.fail} := root(d(\alpha), \alpha, 30deg, 90deg) = 47.433\cdot deg$$

COHESION & IRREGULAR BACKFILL



$\beta_{avg} = \beta$ at $x = 2h$

h = Wall height



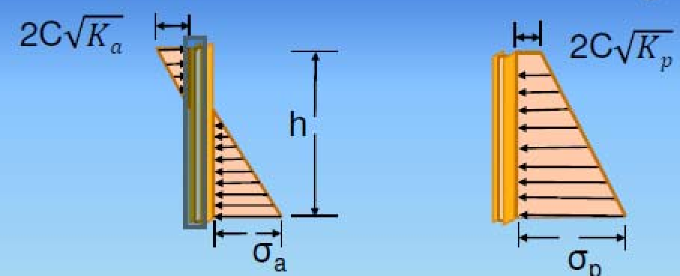
Critical value of α is between α calculated using β_{min} and β_{max}. Use β_{avg} for first trial.

Slope Approximation for Irregular Slope

Rankine's theory was modified to include Cohesion with the following formulas:

$$\sigma_a = \gamma * h * K_a - 2C\sqrt{K_a}$$

$$\sigma_p = \gamma * h * K_p + 2C\sqrt{K_p}$$

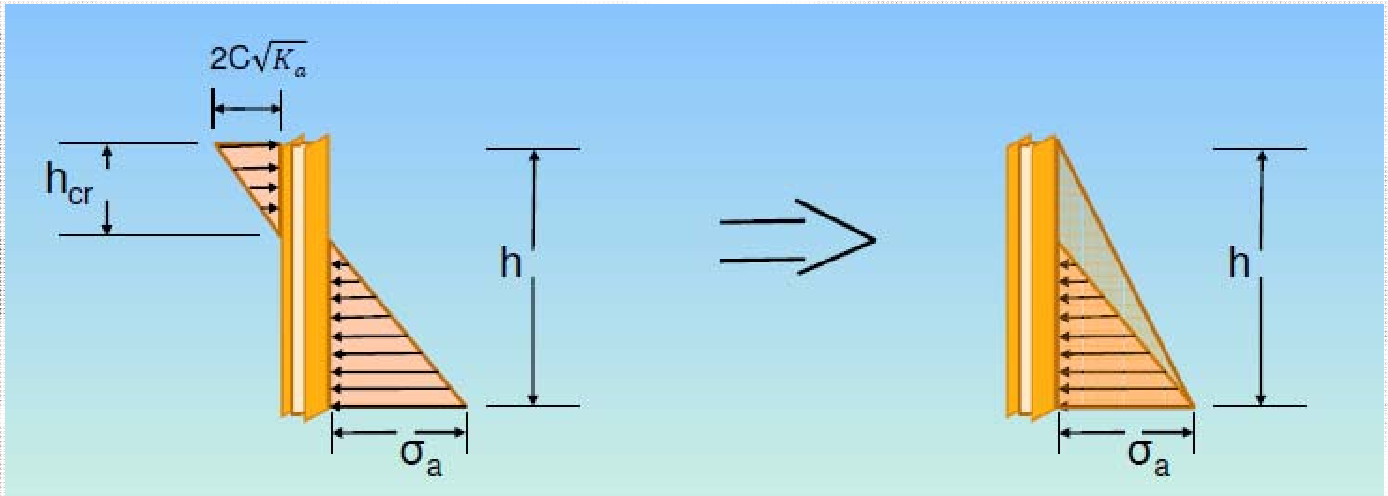


- **The active lateral earth pressure (σ_a) should not be less than 0.25 times the vertical stress ($\sigma_v = \gamma * h$) at any depth[1].**
- **$K_{apparent} = \sigma_a / (\gamma * h) \geq 0.25$**
- Laboratory testing should be required to support soil parameters resulting in K≤.25

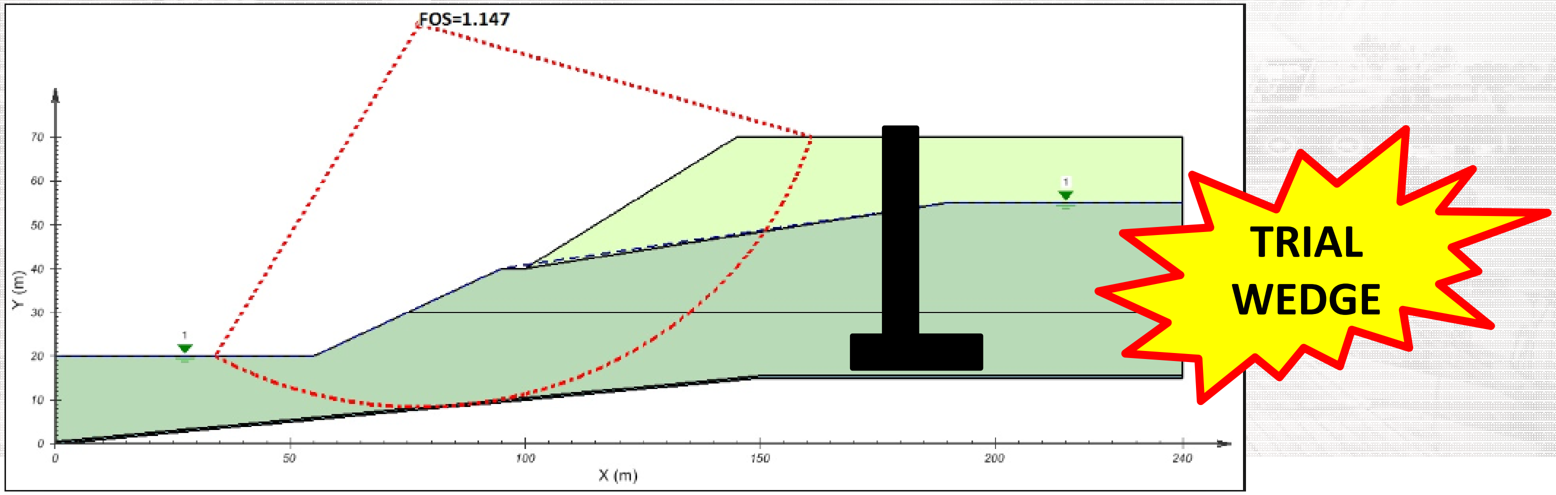
Bells Relationship for Cohesion

COHESION & MULTI LAYER SOIL

Soil Pressure Distribution for Cohesive soil



Multi-Layer Soil



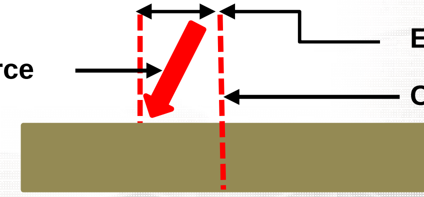
OUTLINE

- Geometry & Cost Comparison ASD/LRFD
- Standard Walls vs Special Design Walls
- LRFD Retaining Wall Design Background
- **Summary of Reasons For Changes**
- Other Type of Walls
- Conclusion

REASONS FOR CHANGES

- **LRFD/CALTRANS Eccentricity Limit Requirements**
- **LRFD/CALTRANS Live Load Surcharge Load Application Requirements**
- **LRFD/CALTRANS Seismic Load Application Requirements**
- **LRFD/CALTRANS Seismic Inertia Force Application Requirements**
- **LRFD/CALTRANS Strength/Service/Extreme Load Combination with Max-Min Factor Design Requirements**
- **LRFD/CALTRANS Rectangular Bearing Pressure**

ECCENTRICITY LIMITS



AASHTO	Service	Strength	Seismic
2007	NA	W/2 Soil 3W/4-Rock	Both Soil and Rock 2W/3 – EQ=0.0 8W/10 – EQ=1.0
2010	NA	W/2 Soil 3W/4-Rock	Both Soil and Rock 2W/3 – EQ=0.0 8W/10 – EQ=1.0
2012	NA	2W/3 Soil 9W/10-Rock	Both Soil and Rock 2W/3 – EQ=0.0 8W/10 – EQ=1.0
2014	NA	2W/3 Soil 9W/10-Rock	Both Soil and Rock 2W/3 – EQ=0.0 8W/10 – EQ=1.0

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

CALTRANS – SERVICE ONLY – Limit W/6

REASONS FOR CHANGES

- LRFD/CALTRANS Eccentricity Limit Requirements
- **LRFD/CALTRANS Live Load Surcharge Load Application Requirements**
- LRFD/CALTRANS Seismic Load Application Requirements
- LRFD/CALTRANS Seismic Inertia Force Application Requirements
- LRFD/CALTRANS Strength/Service/Extreme Load Combination with Max-Min Factor Design Requirements
- LRFD/CALTRANS Rectangular Bearing Pressure

SURCHARGE LOAD

LFD – 2 ft or 240 psf Surcharge Always

Different in LFD & LRFD

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

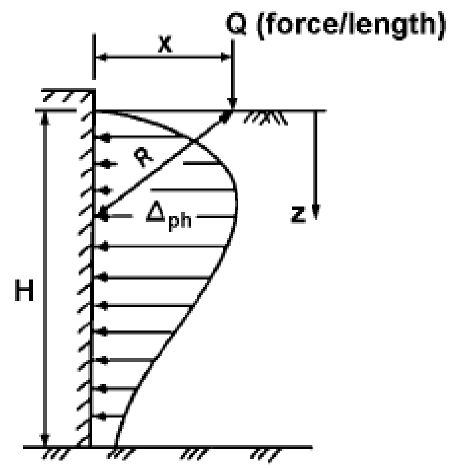
Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0

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

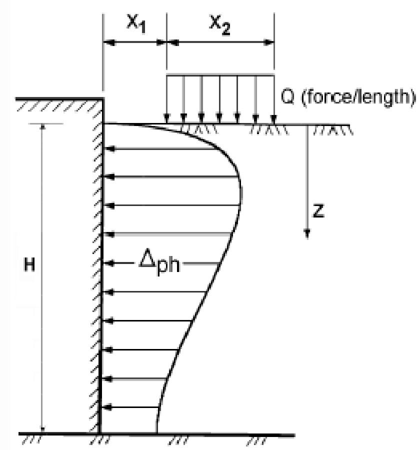
Retaining Wall Height (ft)	h_{eq} (ft) Distance from wall backface to edge of traffic	
	0.0 ft	1.0 ft or Further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

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.

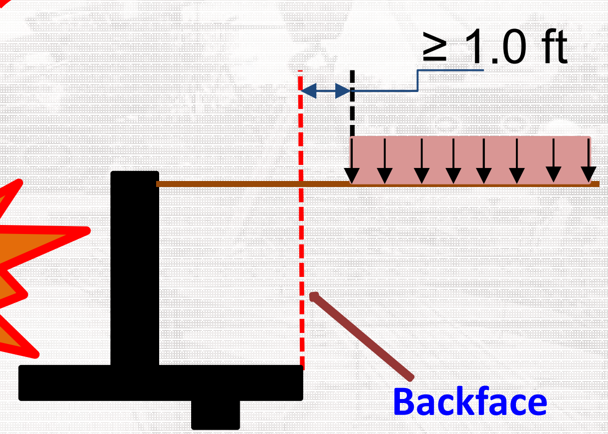
Point Load



Uniform Load



NEW IN LRFD

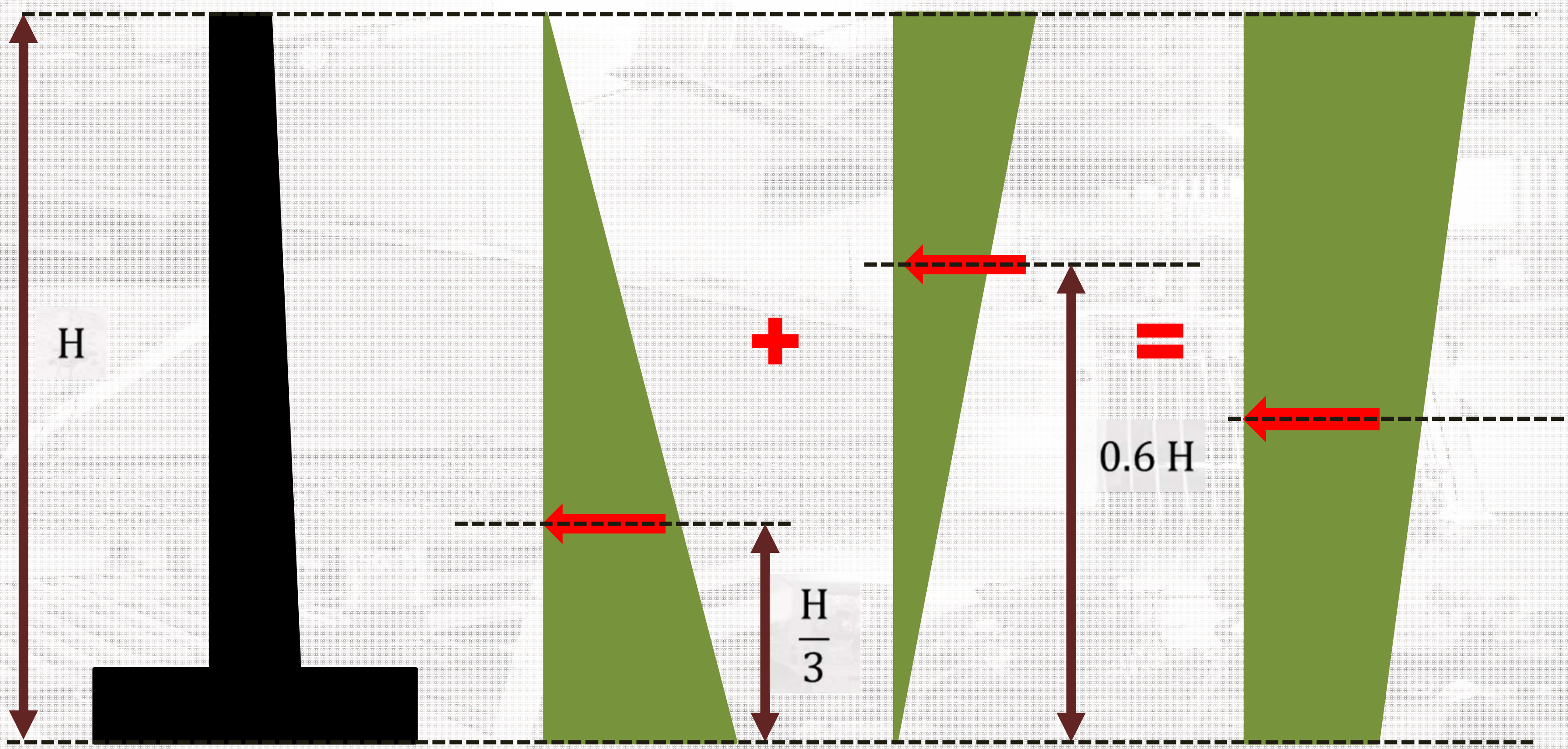


REASONS FOR CHANGES

- LRFD/CALTRANS Eccentricity Limit Requirements
- LRFD/CALTRANS Live Load Surcharge Load Application Requirements
- **LRFD/CALTRANS Seismic Load Application Requirements**
- LRFD/CALTRANS Seismic Inertia Force Application Requirements
- LRFD/CALTRANS Strength/Service/Extreme Load Combination with Max-Min Factor Design Requirements
- LRFD/CALTRANS Rectangular Bearing Pressure

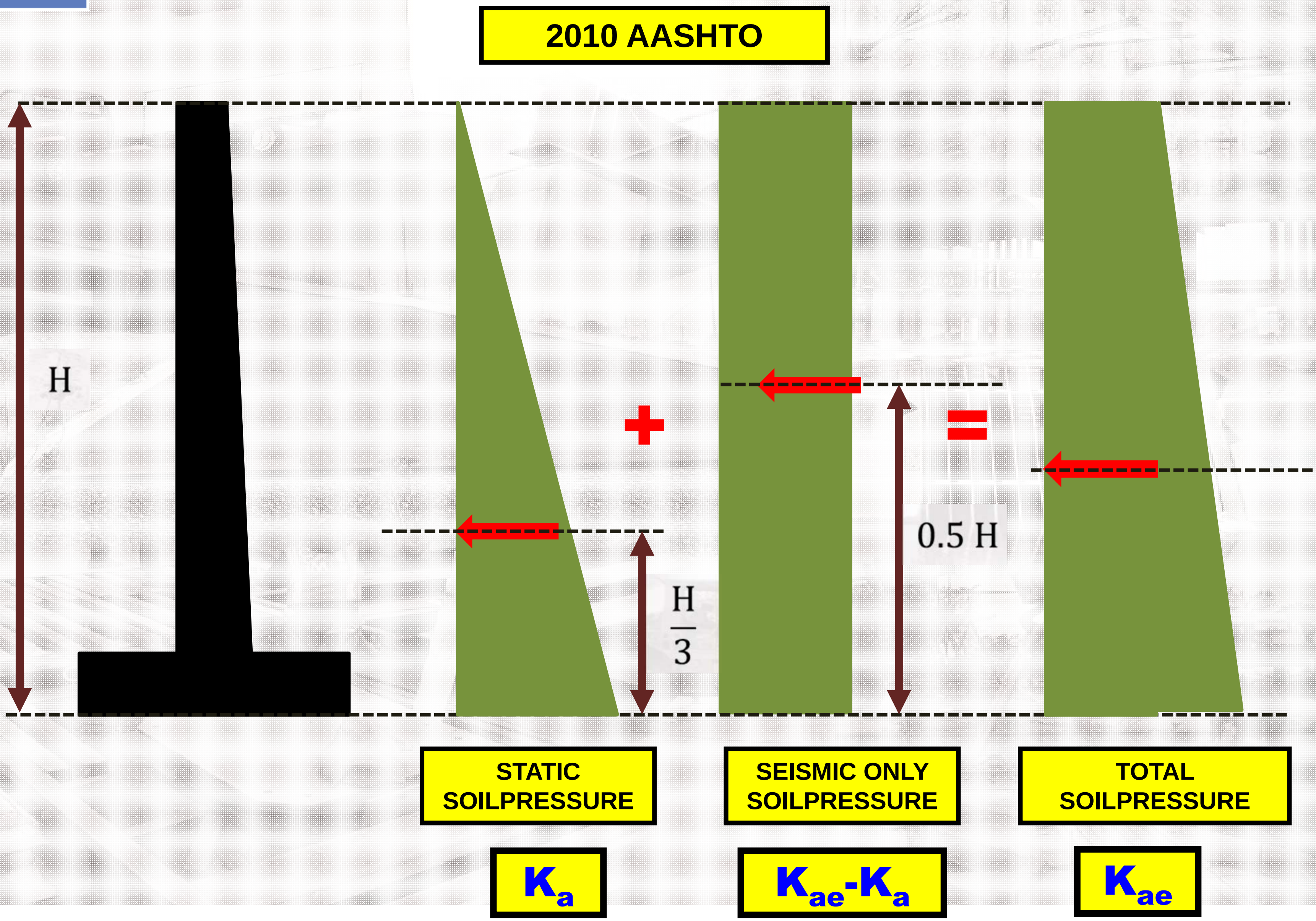
SEISMIC LOAD LOCATION

2007 AASHTO



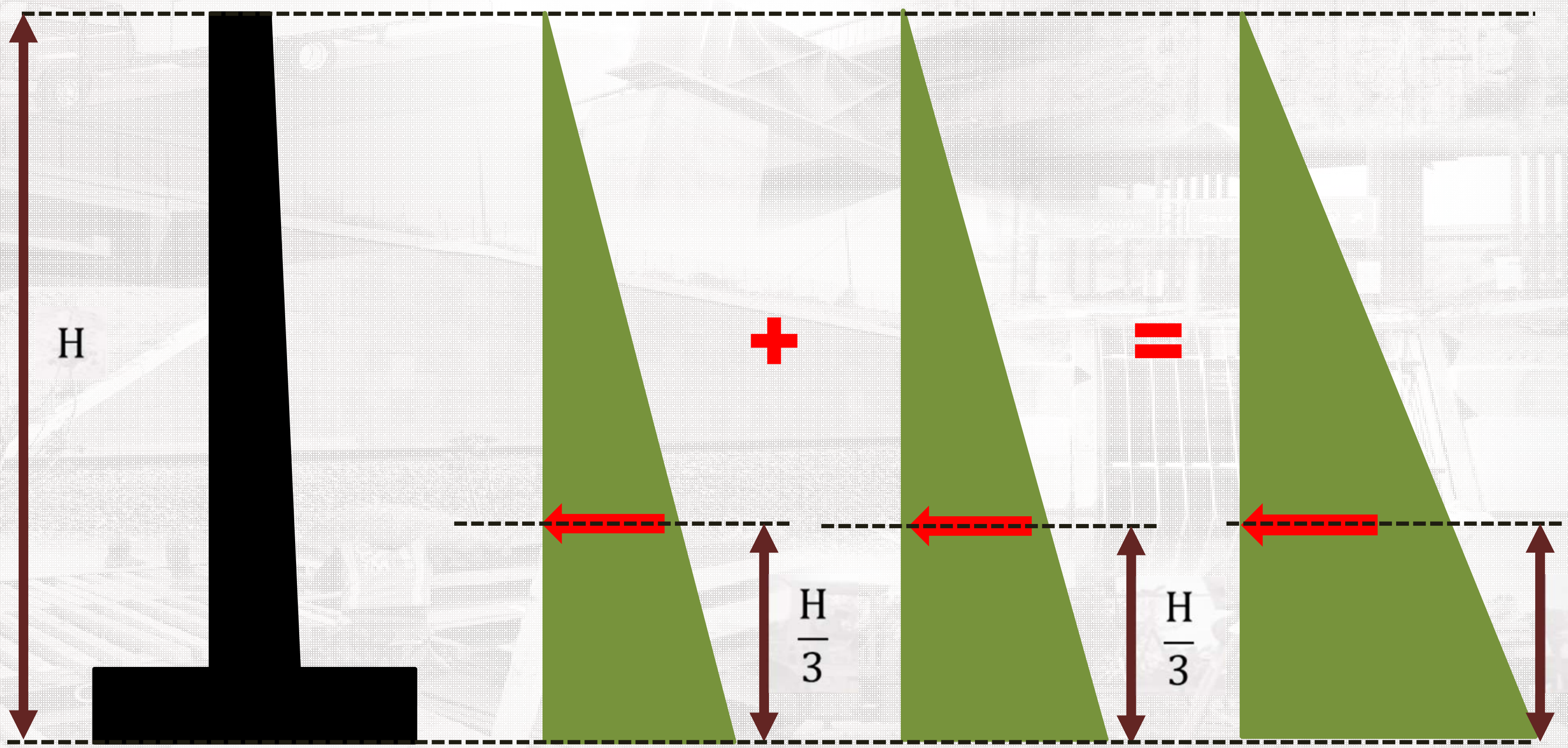
STATIC SOILPRESSURE	SEISMIC ONLY SOILPRESSURE	TOTAL SOILPRESSURE
K_a	$K_{ae}-K_a$	K_{ae}

SEISMIC LOAD LOCATION



SEISMIC LOAD LOCATION

2012 & 2014 AASHTO



STATIC SOILPRESSURE	SEISMIC ONLY SOILPRESSURE	TOTAL SOILPRESSURE
K_a	$K_{ae}-K_a$	K_{ae}

Western Bridge Engineers' Seminar

REASONS FOR CHANGES

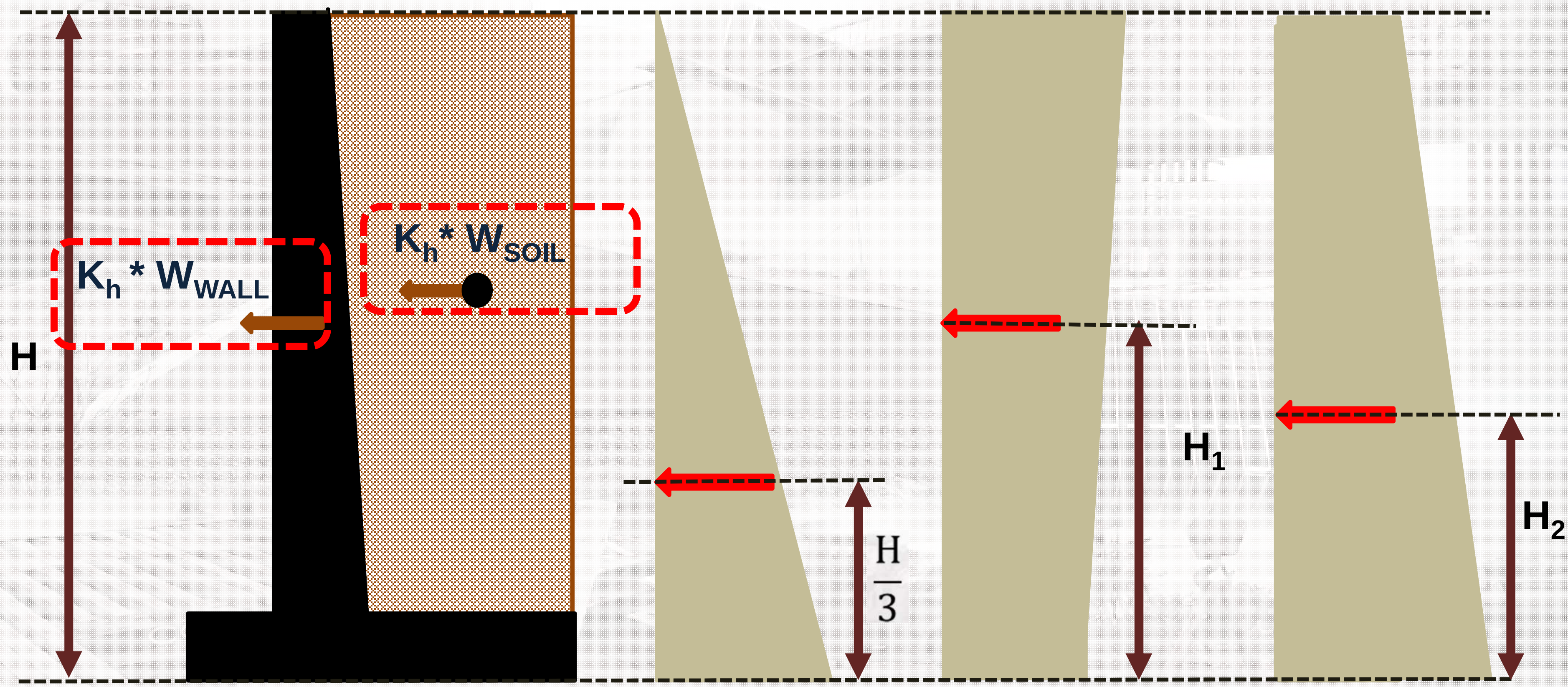
T·Y·LIN INTERNATIONAL

- LRFD/CALTRANS Eccentricity Limit Requirements
- LRFD/CALTRANS Live Load Surcharge Load Application Requirements
- LRFD/CALTRANS Seismic Load Application Requirements
- **LRFD/CALTRANS Seismic Inertia Force Application Requirements**
- LRFD/CALTRANS Strength/Service/Extreme Load Combination with Max-Min Factor Design Requirements
- LRFD/CALTRANS Rectangular Bearing Pressure

Western Bridge Engineers' Seminar

SEISMIC INERTIA LOAD

CONCEPT

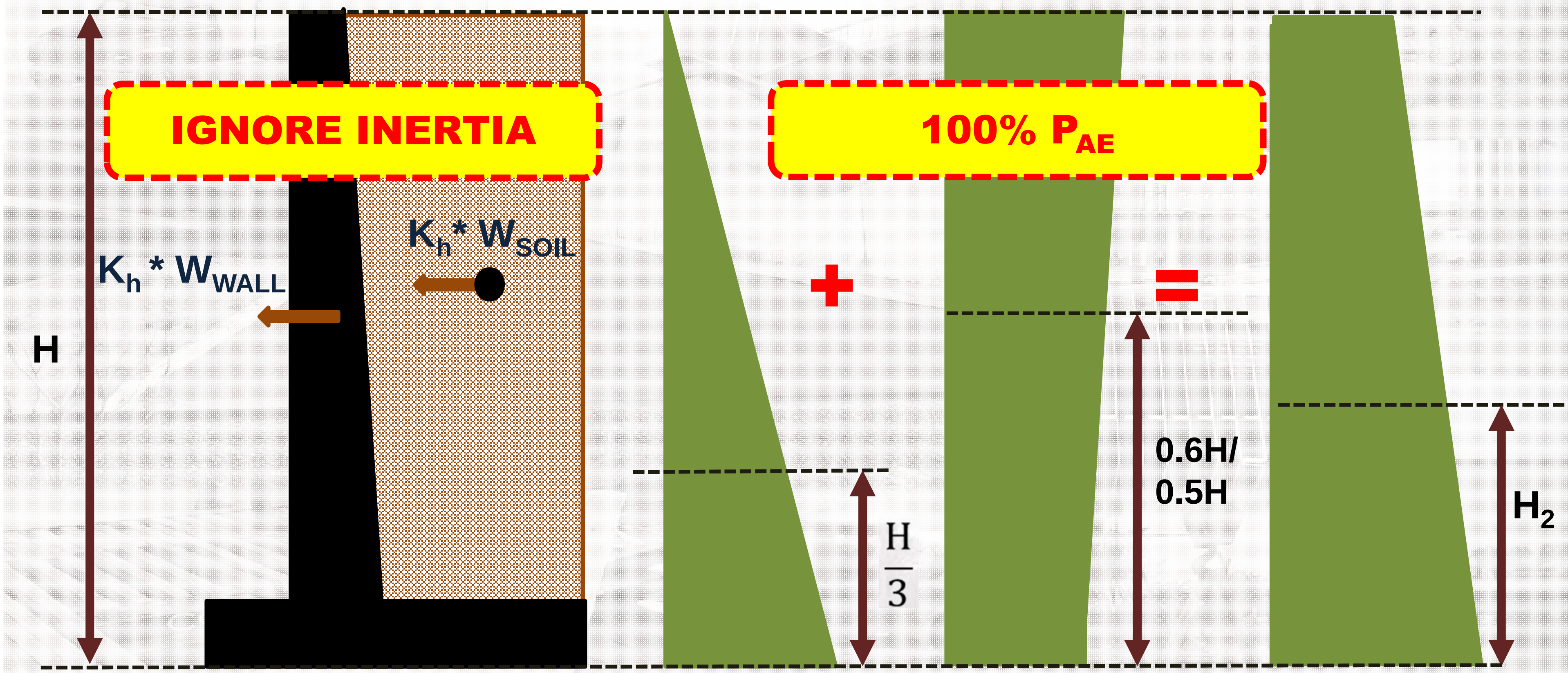


STATIC SOILPRESSURE	SEISMIC ONLY SOILPRESSURE	TOTAL SOILPRESSURE
K_a	$K_{ae}-K_a$	K_{ae}

1. **WALL INERTIA FORCE**
2. **SOIL MASS OVER HEEL INERTIA FORCE**

SEISMIC INERTIA LOAD

BEFORE 2012 AASHTO



STATIC SOILPRESSURE	SEISMIC ONLY SOILPRESSURE	TOTAL SOILPRESSURE
K_a	K_{ae}-K_a	K_{ae}

AASHTO-2007

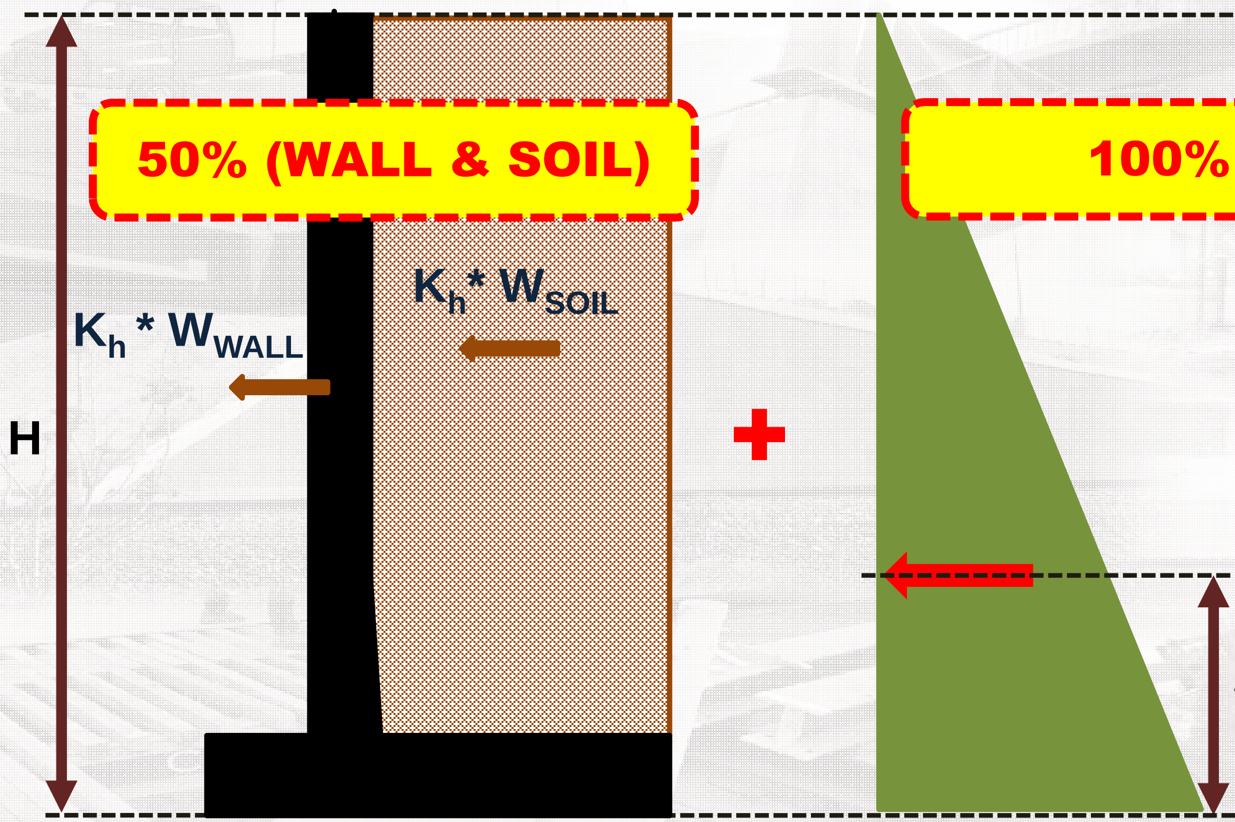
11.8.6 Seismic Design

The effect of earthquake loading shall be investigated using the Extreme Event I limit state of Table 3.4.1-1 with resistance factor ϕ=1.0 and load factor γ_p =1.0 and an accepted methodology.

In general, the pseudo-static approach developed by Mononobe and Okabe may be used to estimate the equivalent static forces provided the maximum lateral earth pressure, active and passive are computed using a seismic coefficient k_h=0.5A. Forces resulting from wall inertia effects may be ignored in estimating the seismic lateral earth pressure. Refer to Appendix A.

SEISMIC INERTIA LOAD COMBINATIONS

2012 & 2014 AASHTO



AASHTO 2014: 11.6.5

$$P_{IR} = k_h(W_w + W_s) \quad (11.6.5.1\text{-}1)$$

and where:

P_{AE} = dynamic lateral earth pressure force

P_{IR} = horizontal inertial force due to seismic loading of the wall mass

k_h = seismic horizontal acceleration coefficient

W_w = the weight of the wall

W_s = the weight of soil that is immediately above the wall, including the wall heel

TOTAL SEISMIC SOILPRESSURE

- Combine 100 percent of the seismic earth pressure P_{AE} with 50 percent of the wall inertial force P_{IR} and
- Combine 50 percent of P_{AE} but no less than the static active earth pressure force (i.e., F_1 in Figure 11.10.5.2-1), with 100 percent of the wall inertial force P_{IR}.

INERTIA LOAD CASE-1

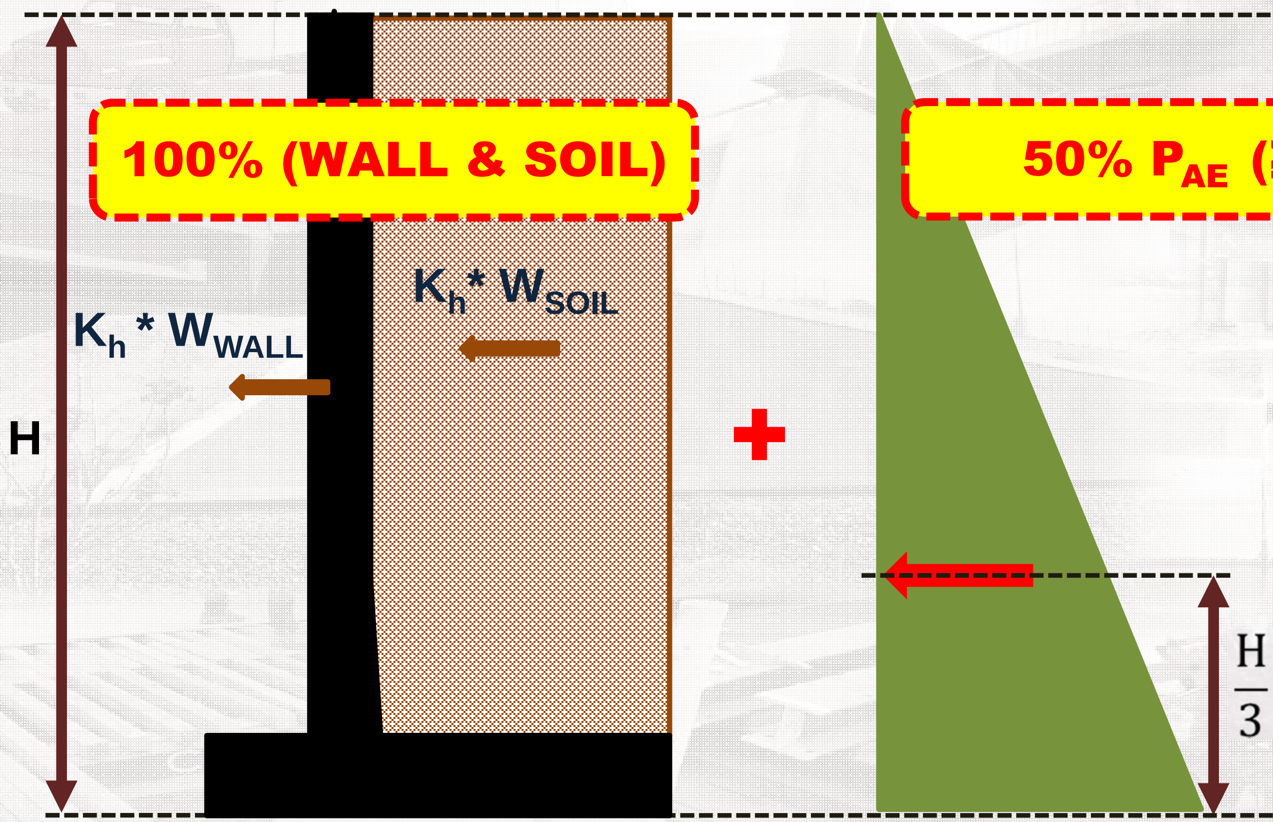
K_{ae}

Western Bridge Engineers' Seminar

T·Y·LIN INTERNATIONAL

SEISMIC INERTIA LOAD COMBINATIONS

2012 & 2014 AASHTO



TOTAL SEISMIC SOILPRESSURE

INERTIA LOAD CASE-2

K_{ae}

AASHTO 2014: 11.6.5

$$P_{IR} = k_h(W_w + W_s) \quad (11.6.5.1\text{-}1)$$

and where:

P_{AE} = dynamic lateral earth pressure force

P_{IR} = horizontal inertial force due to seismic loading of the wall mass

k_h = seismic horizontal acceleration coefficient

W_w = the weight of the wall

W_s = the weight of soil that is immediately above the wall, including the wall heel

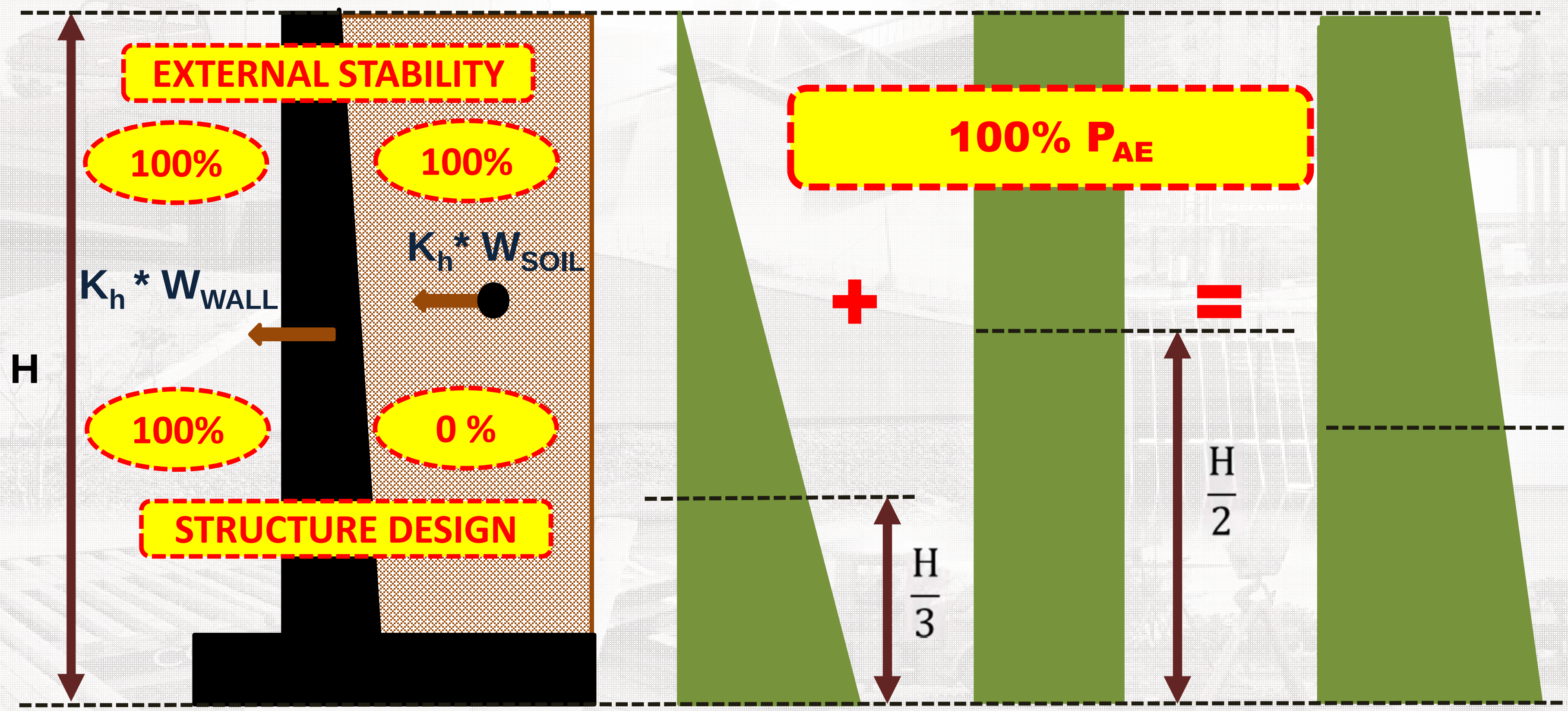
- Combine 100 percent of the seismic earth pressure P_{AE} with 50 percent of the wall inertial force P_{IR} and
- Combine 50 percent of P_{AE} but no less than the static active earth pressure force (i.e., F_1 in Figure 11.10.5.2-1), with 100 percent of the wall inertial force P_{IR}.

SEISMIC INERTIA LOAD

Western Bridge Engineers' Seminar

T.Y. LIN INTERNATIONAL

CALTRANS



INERTIA FORCE COMPONENT	EXTERNAL STABILITY	STRUCT DESIGN
WALL & FOOTING	YES	YES
SOIL MASS OVER HEEL	YES	NO

STATIC SOILPRESSURE	SEISMIC ONLY SOILPRESSURE	TOTAL SOILPRESSURE
K_a	$K_{ae}-K_a$	K_{ae}

REASONS FOR CHANGES

- LRFD/CALTRANS Eccentricity Limit Requirements
- LRFD/CALTRANS Live Load Surcharge Load Application Requirements
- LRFD/CALTRANS Seismic Load Application Requirements
- LRFD/CALTRANS Seismic Inertia Force Application Requirements
- **LRFD/CALTRANS Strength/Service/Extreme Load Combination with Max-Min Factor Design Requirements**
- LRFD/CALTRANS Rectangular Bearing Pressure

AASHTO LOAD COMINATIONS

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	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						
EXTREME EVENT I	γ_p	γEQ	1.00	—	—	1.00	—						
EXTREME EVENT II	γ_p	0.50	1.00	—	—	1.00	—						
SERVICE I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20						
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						
SERVICE IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20						
FATIGUE—*LL*, *IM* & *CE* ONLY	—	0.75	—	—	—	—	—						

EH-Max + EV-Max

EH-Max + EV-Min

EH-Min + EV-Max

EH-Min + EV-Min

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 Erection Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		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

Different in ASD & LRFD

Big Envelope with Max-Min Factors

Resulted Bigger Geometry

Western Bridge Engineers' Seminar

AASHTO LOAD COMINATIONS

T·Y·LIN INTERNATIONAL

Table 3.4.1-1 Load Combinatio

Load Combination Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS					
STRENGTH I (unless noted)	γ_p	1.7					
STRENGTH II	γ_p	1.3					
STRENGTH III	γ_p	—					
STRENGTH IV	γ_p	—					
STRENGTH V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20
EXTREME EVENT I	γ_p	γ_{EQ}	1.00	—	—	1.00	—
EXTREME EVENT II	γ_p	0.50	1.00	—	—	1.00	—
SERVICE I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20
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
SERVICE IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20
FATIGUE— LL, IM & CE ONLY	—	0.75	—	—	—	—	—

...ese at a Time

X

100 % P_{AE} + 50% Inertia

50% P_{AE} *+ 100% Inertia

(* - 50% $P_{AE} \geq P_{A\ (STATIC)}$)

EH-Max + EV-Max

EH-Max + EV-Min

EH-Min + EV-Max

EH-Min + EV-Min

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

Type of Load ...on Type, and Method Used ...te Downdrag		Load Factor Maximum	Load Factor Minimum
DC: Component and Attachme...		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, ...d	1.4	0.25
	Piles, λ M...	1.05	0.30
	Drilled shaft... ...eill and Reese (1999) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
•		1.50	0.90
•		1.35	0.90
•		1.35	N/A
EL: Loc...		1.00	1.00
EV: Ve...			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		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

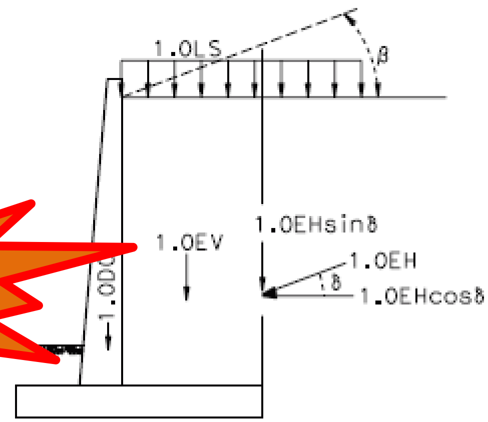
8 COMBINATIONS FOR EXTREME EVENT

CALTRANS LOAD COMINATIONS

Memo to Designers 5-5 April 2014

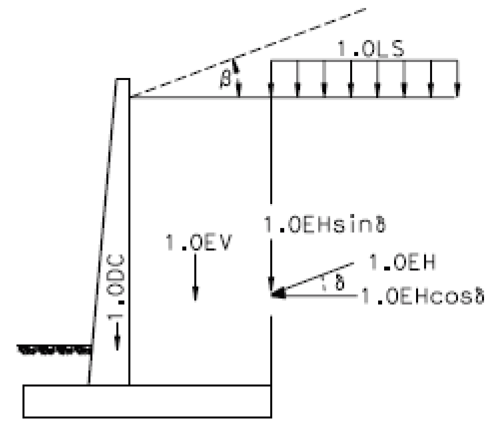
Service-Ia

SIMPLIFIED

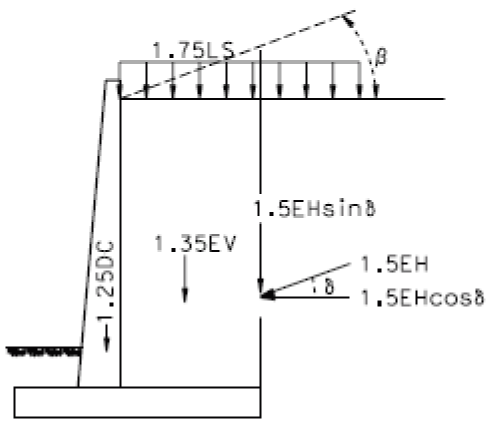


Service Ia - for settlement analysis

Service – 1b

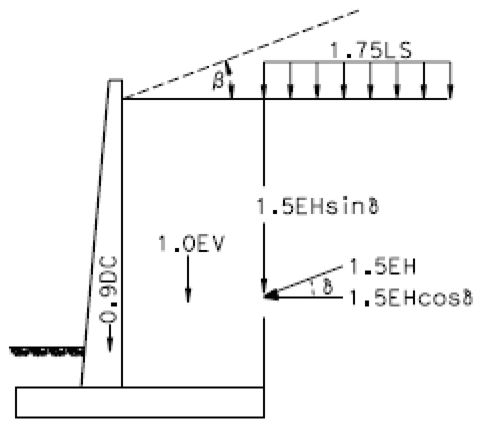


Service Ib - for eccentricity and deformation analysis



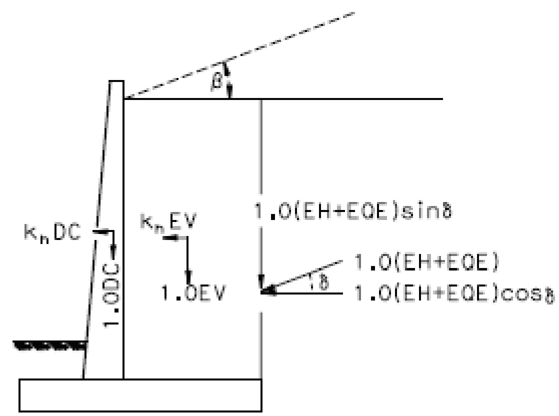
Strength Ia - for bearing and structrual capacity analysis

Strength – 1a



Strength Ib - for bearing, sliding and structural capacity analysis

Strength – 1b



Extreme Event I - for eccentricity, bearing, sliding and structural capacity analysis

Extreme-1

Western Bridge Engineers' Seminar

CALTRANS REQUIRED CHECKS

T·Y·LIN INTERNATIONAL

Minimum Analyses for Earth Retaining System Design

Memo to Designers 5-5 April 2014

Table 1 Analysis for ERS Design

Limit State	Service I	Strength Ia	Strength Ib	Extreme Event I (Seismic)
Bearing Stresses*	X	X	X	X
Eccentricity	X			X
Sliding			X	X
Structural Service Performance	X			
Structural Capacity		X	X	X

* To be checked against actual project conditions before use of the Standard

AASHTO LRFD vs CALTRANS

CHECK		AASHTO	CALTRANS
BEARING	SERVICE	YES-SETTLEMENT	YES-SETTLEMENT
	STRENGTH	YES	YES
	EXTREME	YES	YES
ECCENTRICITY	SERVICE	NO	YES
	STRENGTH	YES	NO
	EXTREME	YES	NO
SLIDING	SERVICE	N/A	N/A
	STRENGTH	YES	YES
	EXTREME	YES	YES
STRUCTURAL DESIGN	SERVICE	YES - CRACKING	YES - CRACKING
	STRENGTH	YES	YES
	EXTREME	YES	YES

Western Bridge Engineers' Seminar

REASONS FOR CHANGES

T·Y·LIN INTERNATIONAL

- LRFD/CALTRANS Eccentricity Limit Requirements
- LRFD/CALTRANS Live Load Surcharge Load Application Requirements
- LRFD/CALTRANS Seismic Load Application Requirements
- LRFD/CALTRANS Seismic Inertia Force Application Requirements
- LRFD/CALTRANS Strength/Service/Extreme Load Combination with Max-Min Factor Design Requirements
- **LRFD/CALTRANS Rectangular Bearing Pressure**

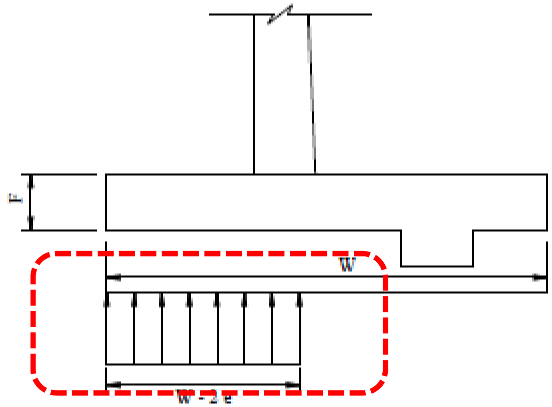
Western Bridge Engineers' Seminar

BEARING PRESSURE

T·Y·LIN INTERNATIONAL

A. GEOTECHNICAL BEARING PRESSURE:

For geotechnical soil bearing pressures evaluation, soil pressure is distributed as uniformly distributed pressure below the footing for a length of (W-2e) (where e-eccentricity). Following figure shows soil pressure distribution.



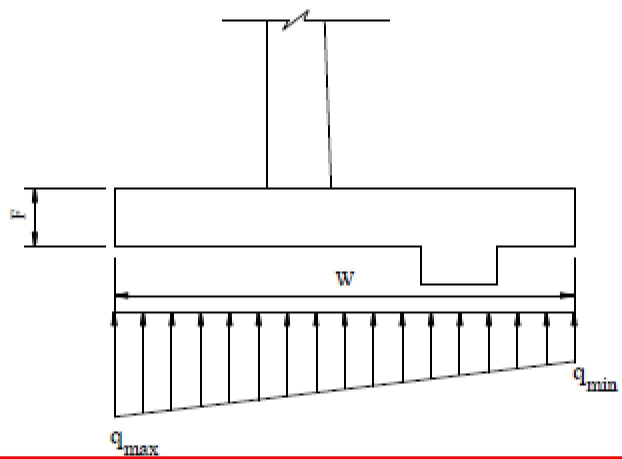
Plastic Soil – For Geotechnical Pressure Evaluation

$$\sigma_v = \frac{\Sigma V}{B - 2e} \qquad (11.6.3.2\text{-}1)$$

$$\sigma_{vmax} = \frac{\Sigma V}{B}\left(1 + 6\frac{e}{B}\right) \qquad (11.6.3.2\text{-}2)$$

$$\sigma_{vmin} = \frac{\Sigma V}{B}\left(1 - 6\frac{e}{B}\right) \qquad (11.6.3.2\text{-}3)$$

B. SOIL PRESSURE FOR STRUCTURAL DESIGN:



If resultant force is outside the middle one third of footing width,

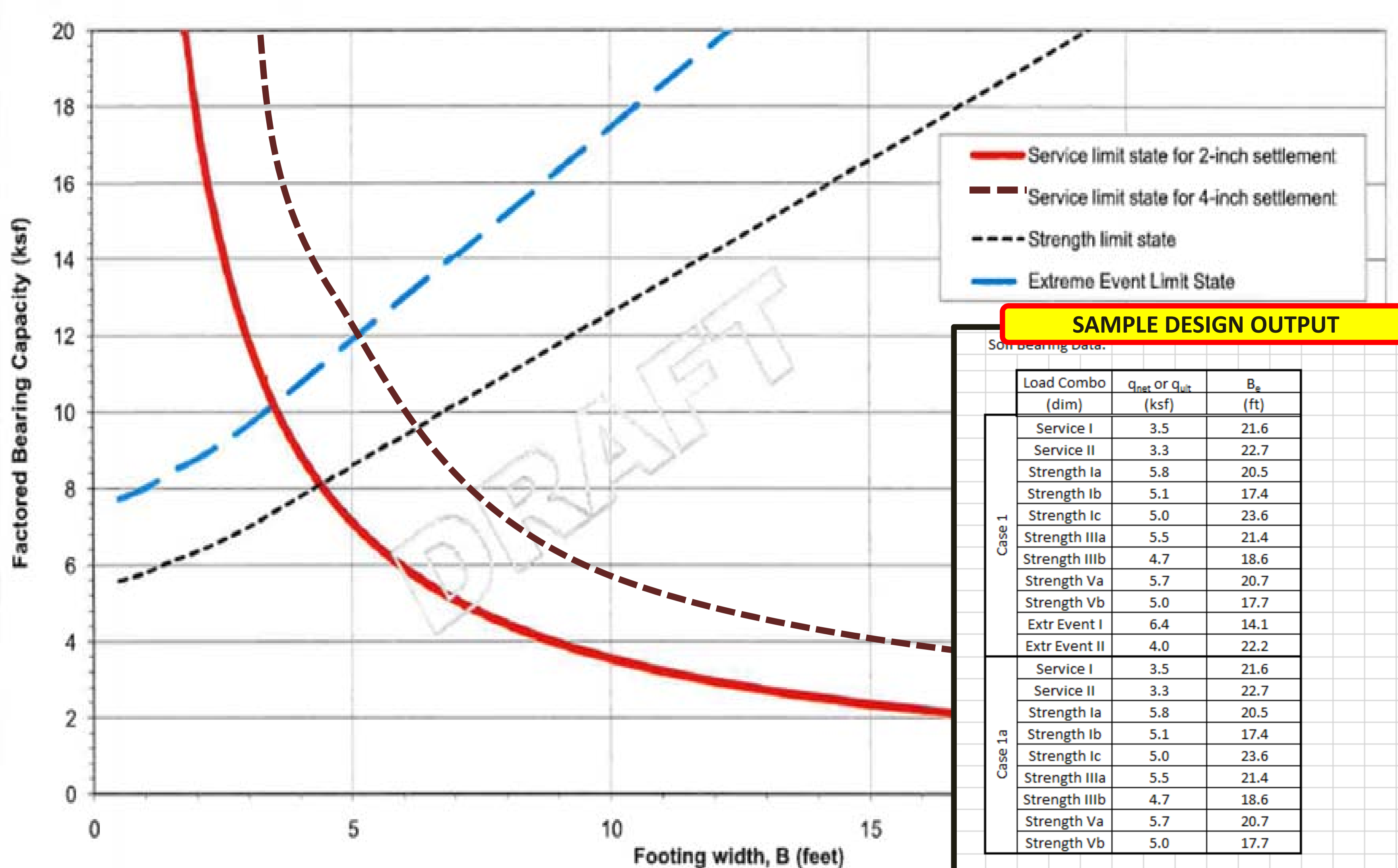
$$\sigma_{vmax} = \frac{2\Sigma V}{3[(B/2) - e)]} \qquad (11.6.3.2\text{-}4)$$

$$\sigma_{vmin} = 0 \qquad (11.6.3.2\text{-}5)$$

Triangular Pressure Distribution Critical for Structural Design

Soil Pressure Reporting: (q_u,B')

LRFD BEARING CAPACITY -SAMPLE



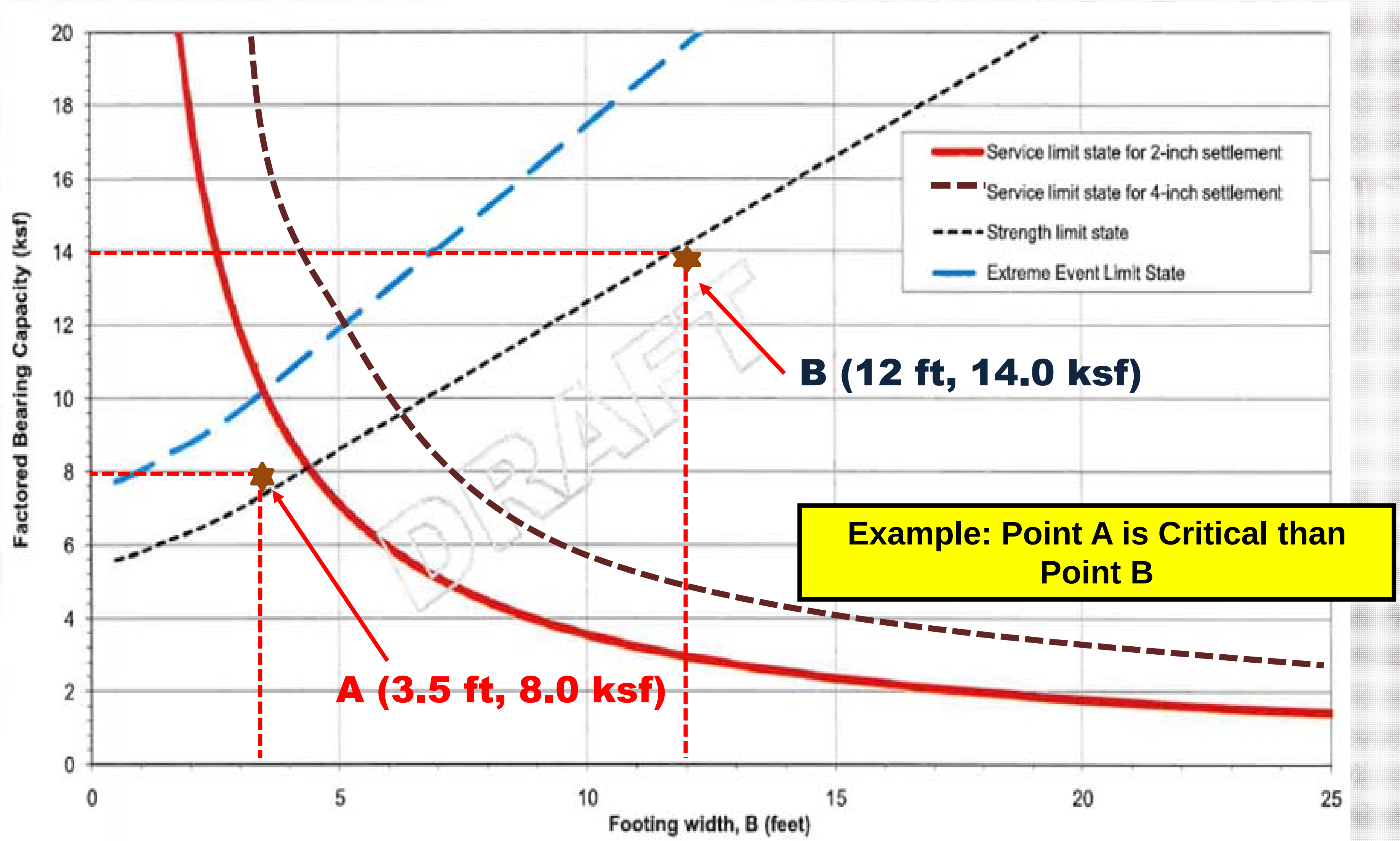
SAMPLE DESIGN OUTPUT

Soil Bearing Data:

	Load Combo (dim)	q_{net} or q_{ult} (ksf)	B_e (ft)
Case 1	Service I	3.5	21.6
	Service II	3.3	22.7
	Strength Ia	5.8	20.5
	Strength Ib	5.1	17.4
	Strength Ic	5.0	23.6
	Strength IIIa	5.5	21.4
	Strength IIIb	4.7	18.6
	Strength Va	5.7	20.7
	Strength Vb	5.0	17.7
	Extr Event I	6.4	14.1
	Extr Event II	4.0	22.2
Case 1a	Service I	3.5	21.6
	Service II	3.3	22.7
	Strength Ia	5.8	20.5
	Strength Ib	5.1	17.4
	Strength Ic	5.0	23.6
	Strength IIIa	5.5	21.4
	Strength IIIb	4.7	18.6
	Strength Va	5.7	20.7
	Strength Vb	5.0	17.7

Case 1: No live load surcharge placed over heel
Case 1a: With live load surcharge placed over heel, as applicable

LRFD BEARING CAPACITY EVALUATION – STRENGTH LIMIT STATE EXAMPLE

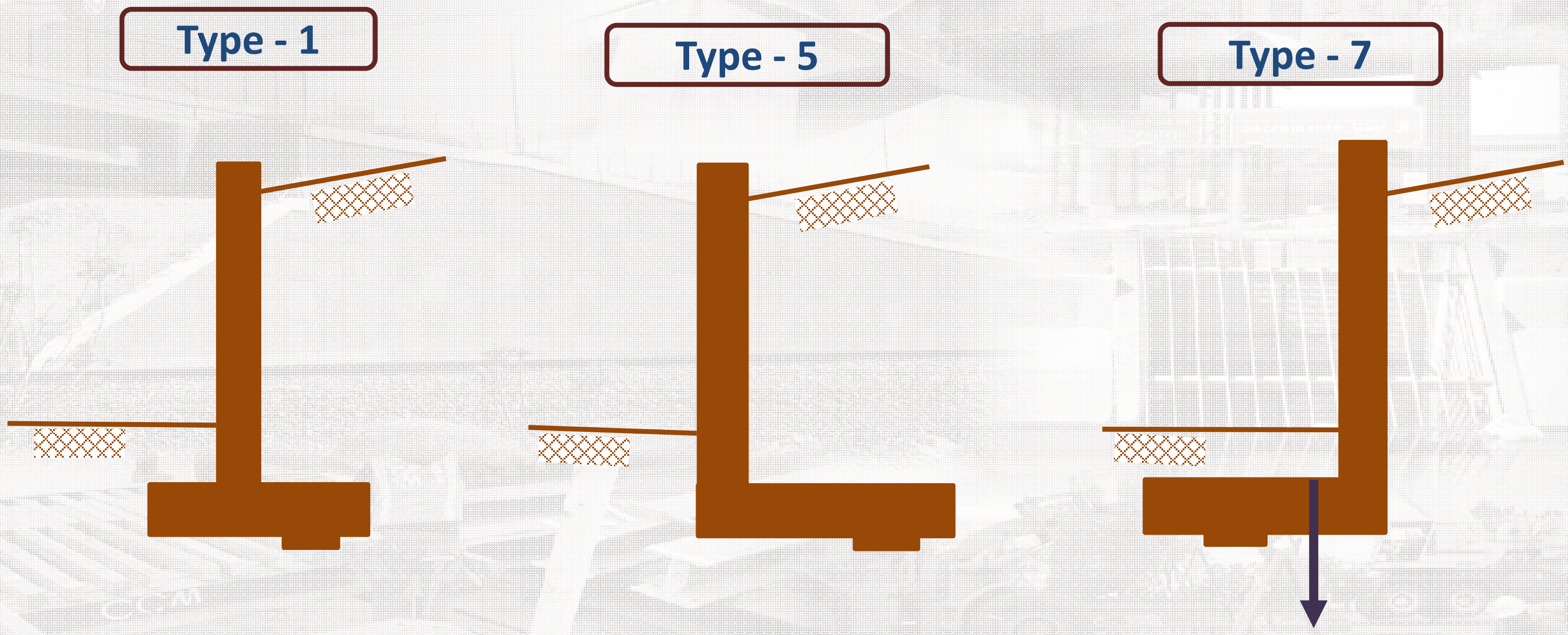


OUTLINE

- Geometry & Cost Comparison ASD/LRFD
- Standard Walls vs Special Design Walls
- LRFD Retaining Wall Design Background
- Summary of Reasons For Changes
- **Other Type of Walls**
- Conclusion

TYPE OF RETAINING WALLS

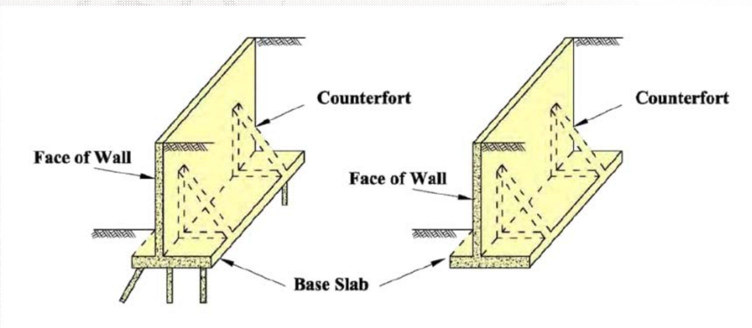
Cantilever Type Walls



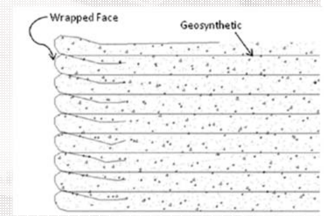
Foundation Type: Footing or Piles

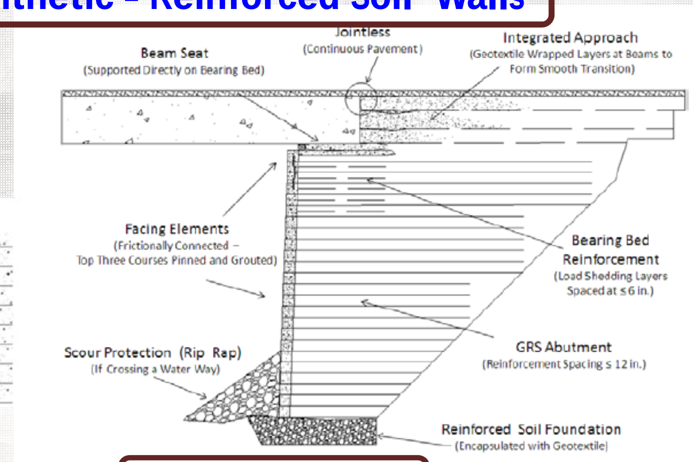
TYPE OF RETAINING WALLS

Counterfort Retaining Walls

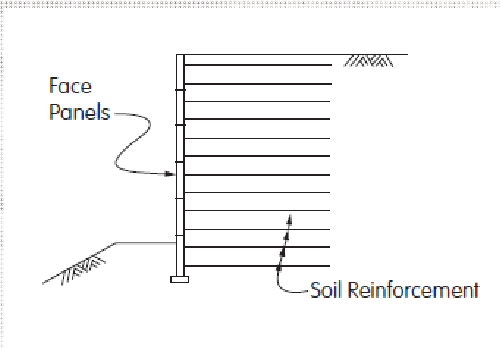


Geosynthetic - Reinforced Soil Walls



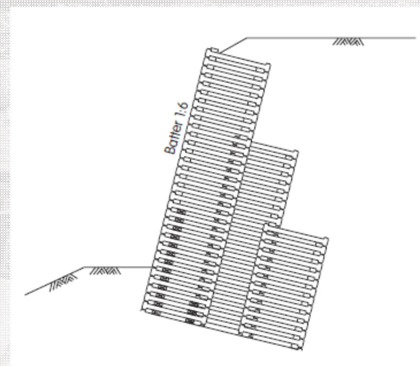


MSE Walls



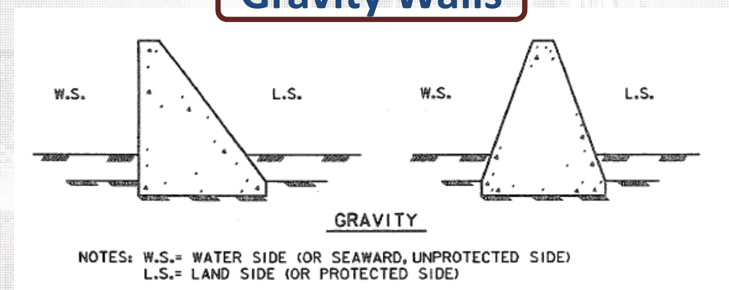
MSE Wall with Precast Concrete Face Panels

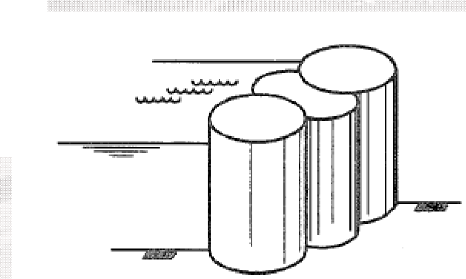
Crib Walls



Precast Concrete Crib Wall

Gravity Walls





- CSM (Concrete-Soil-Mix) Wall
- DSM (Deep-Soil-Mix)
- Secant Pile Walls

(US ARMY CORP EM 1110-2-2504) (FHWA NHI-11-032 AUG 2011 (REV-1)

OUTLINE

- Geometry & Cost Comparison ASD/LRFD
- Standard Walls vs Special Design Walls
- LRFD Retaining Wall Design Background
- Summary of Reasons For Changes
- Other Type of Walls
- **Conclusion**

CONCLUSION

- Retaining Wall Geometry has significantly changed because of LRFD Design Criteria compare to ASD
- LRFD Design Criteria Constantly changed from 2007 to 2014.
- AASHTO significantly reduced Design Requirements in Recent LRFD Codes compare to 2007 LRFD Code.
- DOTs' Issues Interim Memorandums to Amend AASHTO LRFD Requirements.

WAITING FOR UPDATED STANDARD PLANS & XS SHEETS

QUESTIONS & ANSWERS

Contact Details:

Ahilan Selladurai,

Phone: 916-349-4250 x 4266

Direct: 916-349-4266

Mobile: 916-662-3592

E-Mail: ahilan.selladurai@tylin.com