ALUMINUM STRUCTURAL PLATE HEADWALLS
AASHTO LRFD BASIS OF DESIGN

LANE ENTERPRISES, INC.
www.lane-enterprises.com

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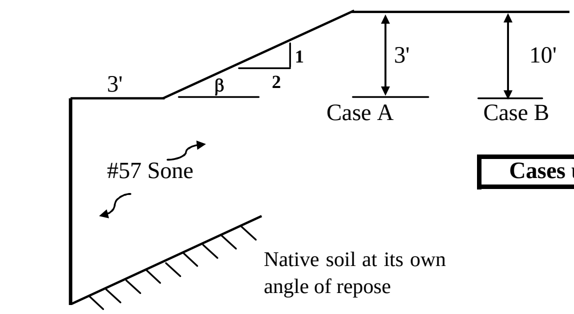
BASIS OF DESIGN

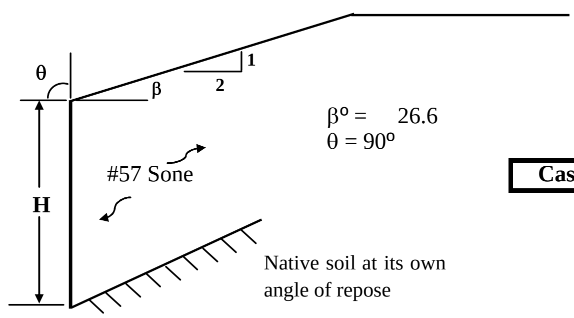
Required Backfill and Load Cases:

Backfill in the strucutral zone shall be AASHTO No. 57 stone ($\phi = 40^o$).



Cases used to calculate deadmen locations

The above cases as directed by SCDOT for the development of the tables. However, these cases could easily develop into a constant slope from the top of the wall over time. Therefore the following case will be used to calculate the loads on the wall, and the above cases will be used to calculate the depth of bury and rod lenghts for the deadmen anchors.



Case used to calculate loads on the wall

AASHTO LRFD Bridge Design Specifications 3.11.5.3 - Active Lateral Earth Pressure Coefficient, k_a

$$k_a = \frac{\sin^2(\theta + \phi'_f)}{\Gamma[\sin^2\theta \sin(\theta - \delta)]} = \mathbf{0.28} \qquad \Gamma = \left\{1 + \left[\frac{\sin(\phi'_f + \delta)\sin(\phi'_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}\right]^{1/2}\right\}^2 = \mathbf{2.242}$$

Where,	deg	rad	
ϕ'_f, effective angle of internal friciton	40.0	0.698	
β, angle of fill to the horizontal	26.6	0.464	
θ, angle of back face to horizontal	90.0	1.571	
δ, friction angle between fill and wall	22.0	0.384	(Table 3.11.5.3-1)

Compare [1]Coulomb Theory results for k_a and k_p:

$$k_a = \frac{\cos^2\phi}{\cos\delta\left\{1 + \left[\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos\delta\cos\beta}\right]^{1/2}\right\}^2} = \mathbf{0.28}$$

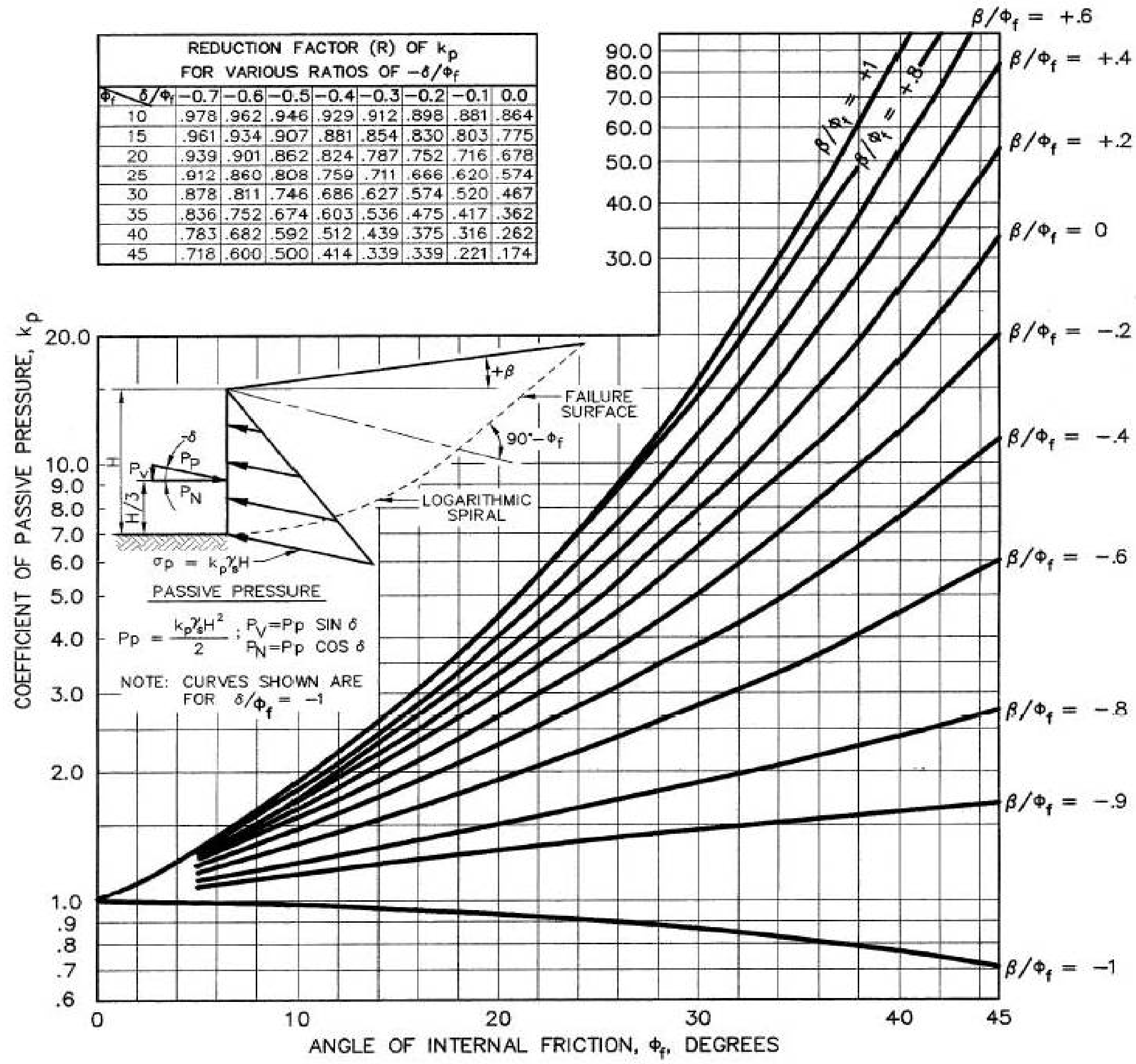
δ taken as positive for the active case

$$k_p = \frac{\cos^2\phi}{\cos\delta\left\{1 - \left[\frac{\sin(\phi+\delta)\sin(\phi+\beta)}{\cos\delta\cos\beta}\right]^{1/2}\right\}^2} = \mathbf{3.67}$$

δ taken as negative for the passive case

[1] Trigonometric expressions of Coulomb's theory taken from Page 7 of USS Steel Sheet Piling Design Manual (dated July 1984).

AASHTO LRFD PASSIVE LATERAL EARTH PRESSURE COEFFICIENT ANALYSIS



REDUCTION FACTOR (R) OF k_p FOR VARIOUS RATIOS OF $-\delta/\phi_f$

ϕ_f \ δ/ϕ_f	−0.7	−0.6	−0.5	−0.4	−0.3	−0.2	−0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.339	.221	.174

Figure 3.11.5.4-2—Computational Procedures for Passive Earth Pressures for Vertical Wall with Sloping Backfill (U.S. Department of the Navy, 1982a)

AASHTO LRFD Bridge Design Specifications 3.11.5.4 - Passive Lateral Earth Pressure Coefficient, k_p

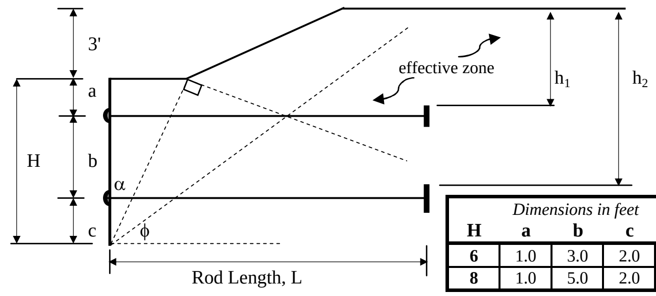
For non-cohesive soils, values of the coefficient of passive earth pressure may be taken from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill.

Where,	deg	rad					
ϕ_f, effective angle of internal friciton	40.0	0.698	β/ϕ_f =	0.66			
β, angle of fill to the horizontal	26.6	0.464	$-\delta/\phi_f$ =	-0.6	R =	0.682	
δ, friction angle between fill and wall	22.0	0.384	$k_p = R[k_p\ (\delta/\phi_f = -1)]$ =			(0.682)(58) =	39.6

A value of 36.9 is too large a k_p for deadmen application (note that the above chart is for vertical walls) in that it will not produce a conservative pullout resistance of the anchors (i.e. the lower the value the more conservative the result). Therefore, use the more traditional result from Coulomb's theory calculated on the previous page (k_p = 3.67).

ROD LENGTHS AND DMA BURIAL DEPTHS

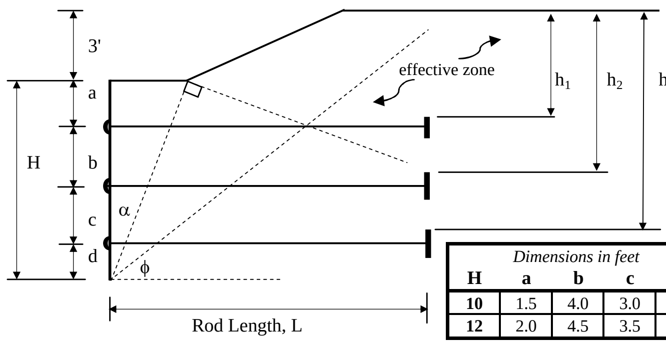
Rod lengths are determined by measuring from the wall to the effective zone (along the top row), adding an additional three feet, and then rounding up to the nearest whole foot. All rods will be the same length as the top row rods.



Case A
Two levels of wale beams
$\phi = 40°$
$\alpha = 45 - \phi/2$

Dimensions in feet

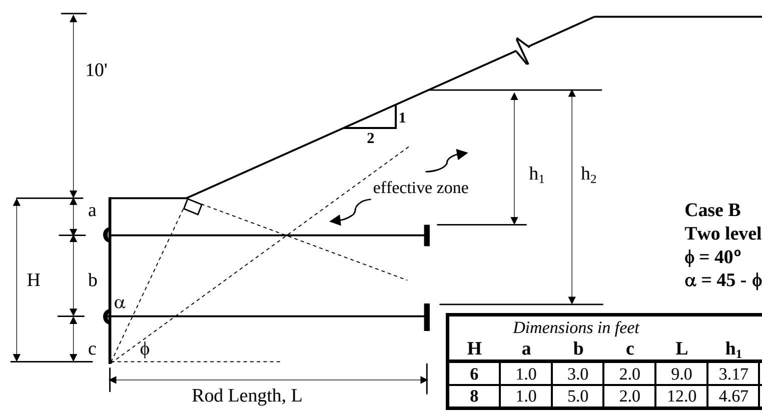
H	a	b	c	L	h_1	h_2
6	1.0	3.0	2.0	9.0	3.17	5.83
8	1.0	5.0	2.0	12.0	3.17	7.83



Case A
Three levels of wale beams
$\phi = 40°$
$\alpha = 45 - \phi/2$

Dimensions in feet

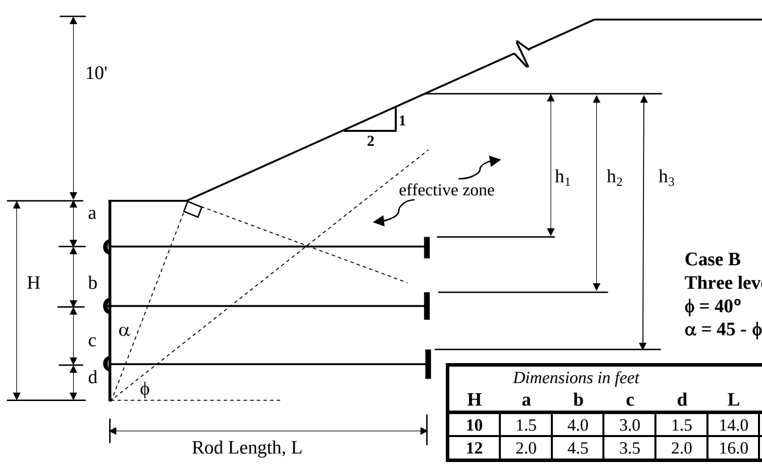
H	a	b	c	d	L	h_1	h_2	h_3
10	1.5	4.0	3.0	1.5	14.0	3.67	7.33	10.33
12	2.0	4.5	3.5	2.0	16.0	4.17	8.33	11.83



Case B
Two levels of wale beams
$\phi = 40°$
$\alpha = 45 - \phi/2$

Dimensions in feet

H	a	b	c	L	h_1	h_2
6	1.0	3.0	2.0	9.0	3.17	5.83
8	1.0	5.0	2.0	12.0	4.67	9.33



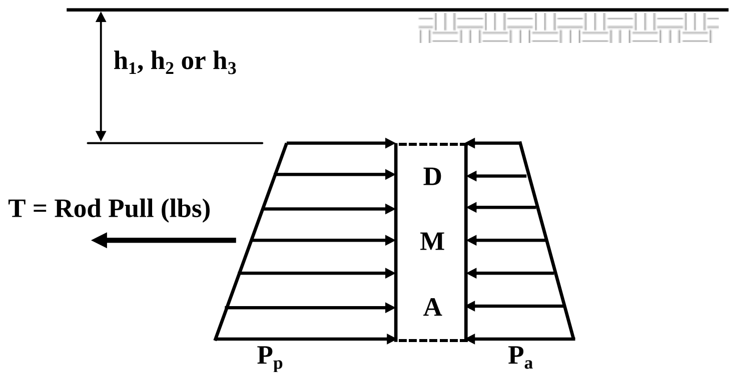
Case B
Three levels of wale beams
$\phi = 40°$
$\alpha = 45 - \phi/2$

Dimensions in feet

H	a	b	c	d	L	h_1	h_2	h_3
10	1.5	4.0	3.0	1.5	14.0	6.17	9.83	12.83
12	2.0	4.5	3.5	2.0	16.0	7.67	11.83	15.33

DMA PULLOUT CAPACITY - BACKFILL/EMBEDMENT RESISTANCE

Anchor capacities will be based on the difference in passive and active soil pressures at the anchor elevation, and based solely on the area of the anchors (methods that account for the shear capacity of the soil above the anchor are more appropriate for a single anchor near the surface).



Dead Man Anchor (DMA) Pullout Force

$P_p - P_a = (k_p - k_a)\gamma h = 3.39\gamma h$

$\gamma = 125$ pcf

$k_p = 3.67$

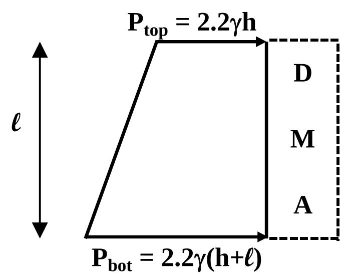
$k_a = 0.28$

T_u = Factored Pullout Resistance = $\phi(P_p - P_a) = \phi(3.39\gamma h)$

ϕ = resistance factor for anchor pullout

ϕ = 0.65 for anchors in granular soils, Table 11.5.6-1

$T_u = 2.2\gamma h$



ℓ = 1'-8" (top row), 2'-4" (intermediate and bottom rows)

w = width of DMA (into the page) = 2'-4"

A = Area of DMA = ℓ x w (planar area)

$$T_u \text{ (lbs)} = \left(\frac{P_{top} + P_{bot}}{2}\right) A$$

See "Rod Lengths and DMA Burial Depths" sheet for dimensions H, h_1, h_2 and h_3.

H (ft)	Case	h_1 (ft)	T_u (lbs)	h_2 (ft)	T_u (lbs)	h_3 (ft)	T_u (lbs)
6	A	3.17	4,286	5.83	7,132	*n/a*	*n/a*
6	B	3.17	4,286	5.83	7,132	*n/a*	*n/a*
8	A	3.17	4,286	7.83	9,272	*n/a*	*n/a*
8	B	4.67	5,891	9.33	10,877	*n/a*	*n/a*
10	A	3.67	4,821	7.33	8,737	10.33	11,947
10	B	6.17	7,496	9.83	11,412	12.83	14,622
12	A	4.17	5,356	8.33	9,807	11.83	13,552
12	B	7.67	9,101	11.83	13,552	15.33	17,297

DESIGN LOADS

Live Load Surcharge, LS

The increase in horizontal pressure (Δ_p) due to live load surcharge may be estimated as $k_a \gamma_s h_{eq}$ (3.11.6.4-1).

Δ_p = constant horizontal earth pressure due to live load surcharge, LS

$k_a = 0.28$ $\gamma_s = 125$ pcf

h_{eq} = equivalent height of soil for vehicular load per Table 3.11.6.4-1

The load factor for both vertical and horizontal components of live load surcharge (LS) shall be 1.75 (Table 3.4.1-1).

Horizontal componenet of LS = Δ_p

Use h_{eq} values for LS located 1.0 ft or further from the backface of the wall.

$1.75\Delta_p$ = (1.75)(0.28)(125 lbs/ft^3)(2 ft) = 122.5 lbs/ft^2

Table 3.11.6.4-1 — Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic

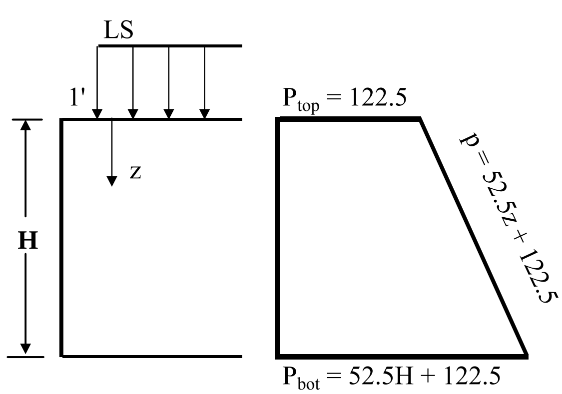
Retaining Wall Height (ft)	h_{eq} (ft) Distance from wall backface to edge of traffic	
	0.0 ft	1.0 ft or Further
5.0	5.0	2.0
6.0	4.7	2.0
7.0	4.4	2.0
8.0	4.1	2.0
9.0	3.8	2.0
10.0	3.5	2.0
11.0	3.4	2.0
12.0	3.2	2.0
13.0	3.1	2.0
14.0	2.9	2.0
15.0	2.8	2.0
16.0	2.6	2.0
17.0	2.5	2.0
18.0	2.3	2.0
19.0	2.2	2.0
≥20.0	2.0	2.0

Horizontal Earth Pressure, EH

The load factor for the active horizontal earth pressure shall be taken as 1.50 (Table 3.4.1-2).

$p = k_a \gamma_s z$ $k_a = 0.28$ $\gamma_s = 125$ pcf

$p = (1.50)(0.28)(125)z = 52.5z$



Factored Wall Loads		
H (ft)	**P_{top} (psf)**	**P_{bot} (psf)**
6	122.5	437.5
8	122.5	542.5
10	122.5	647.5
12	122.5	752.5

LOAD CASES

Four load cases shall demonstrate the various loadings that can occur during construction and over the service life of the completed structure:

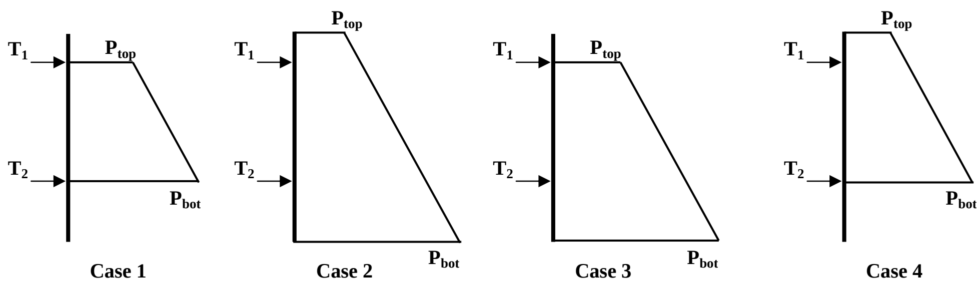
Case 1 Backfill only between top and bottom anchors. This case produces the maximum moment in a two anchor wall and sometimes the maximum tension on the middle anchor on a three anchor wall

Case 2 Completley backfilled with no passive pressure at the toe. This case shall ensure stabiltiy in the event of total scour at the base of the wall.

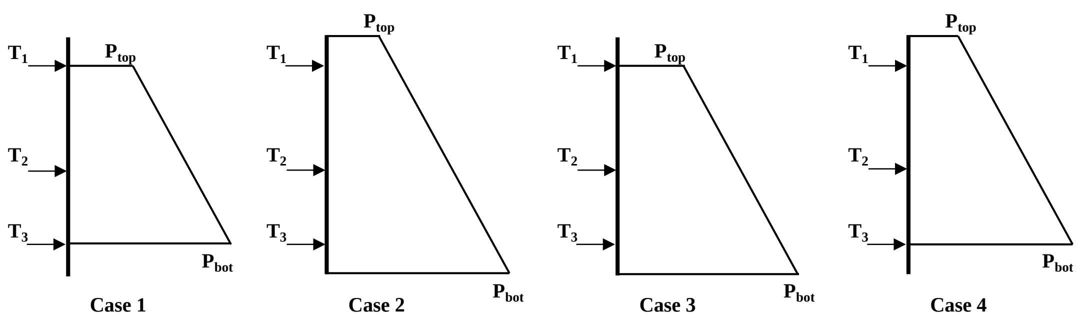
Case 3 Backfill to the centerline of the top anchor with no passive pressure at the toe. This case shall represent the worst case construction loads.

Case 4 Completely backfilled with the toe secured with passive pressure. This case shall model the actual installation of the structure as designed.

Cases for Two Levels of Wale Beams (see "Rod Lengths and DMA Burial Depths" sheet for dimensions H, a, b, and c)



Cases for Three Levels of Wale Beams (see "Rod Lengths and DMA Burial Depths" sheet for dimensions H, a, b, c and d)



H = 6ft

Case	P_{top} (psf)	P_{bot} (psf)	T_1 (lb/ft)	T_2 (lb/ft)	M_{max} (lb-ft/ft)
1	175.0	332.5	341.2	420.0	286.2
2	122.5	437.5	245.0	1435.0	**804.8**
3	175.0	437.5	72.9	**1458.0**	804.8
4	122.5	332.5	**513.3**	396.7	254.1

H = 8ft

Case	P_{top} (psf)	P_{bot} (psf)	T_1 (lb/ft)	T_2 (lb/ft)	M_{max} (lb-ft/ft)
1	175.0	437.5	656.2	875.0	961.9
2	122.5	542.5	616.0	2044.0	**1015.0**
3	175.0	542.5	453.2	**2058.0**	1015.0
4	122.5	437.5	**819.0**	861.0	929.6

H = 10ft

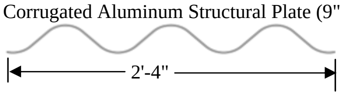
Case	P_{top} (psf)	P_{bot} (psf)	T_1 (lb/ft)	T_2 (lb/ft)	T_3 (lb/ft)	M_{max} (lb-ft/ft)
1	201.3	568.8	394.6	**1724.0**	577.2	591.8
2	122.5	647.5	**728.6**	1333.0	**1788.0**	698.9
3	201.3	647.5	432.1	1403.0	1772.0	**698.9**
4	122.5	568.8	691.2	1654.0	593.1	543.9

H = 12ft

Case	P_{top} (psf)	P_{bot} (psf)	T_1 (lb/ft)	T_2 (lb/ft)	T_3 (lb/ft)	M_{max} (lb-ft/ft)
1	227.5	647.5	495.4	**2228.0**	777.0	871.0
2	122.5	752.5	**1005.0**	1543.0	**2702.0**	1434.0
3	227.5	752.5	565.2	1658.0	2677.0	**1434.0**
4	122.5	647.5	935.1	2113.0	802.3	782.4

DMA PULLOUT CAPACITY - ANCHOR SYSTEM RESISTANCE

Corrugated Aluminum Structural Plate (9" x 2½")



0.150" ASP (Top Row DMA's)

I = 0.1249 in^4/in (28 in) = 3.4972 in^4

c = (2.50 + 0.150)/2 = 1.325 in

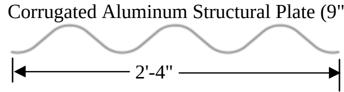
S = I/c = 2.639 in^3

$M_y = F_yS$ = 24,000(2.639)(1/12) = 5,278 lb-ft

M_p = 3.18 k-ft/ft (published)

M_p = 3180 ft-lbs/ft (2.33 ft) = 7,409 lb-ft

Corrugated Aluminum Structural Plate (9" x 2½")



0.250" ASP (Intermediate and Bottom Row DMA's)

I = 0.2094 in^4/in (28 in) = 5.8632 in^4

c = (2.50 + 0.250)/2 = 1.375 in

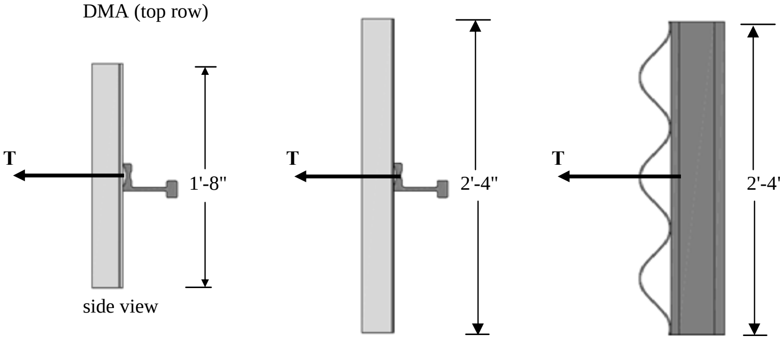
S = I/c = 4.264 in^3

$M_y = F_yS$ = 24,000(4.264)(1/12) = 8,528 lb-ft

M_p = 5.30 k-ft/ft (published)

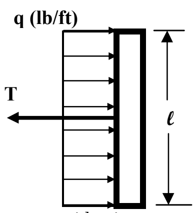
M_p = 5300 ft-lbs/ft (2.33 ft) = 12,349 lb-ft

DMA (top row) | DMA (intermediate and bottom rows) | All DMA's



side view | top view

DMA PLATE CAPACITIES



side view

$$M_{max} = q\left(\frac{(\ell/2)^2}{2}\right)$$

$$q\ (\text{lb/ft}) = \frac{2M_{max}}{(\ell/2)^2}$$

T (lbs) = q x ℓ

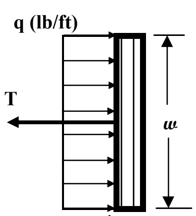
$T_u = \phi T$ (ϕ = 0.65) see above

DMA Plate Capacities ($M_{max} = M_p$)

Top row	Intermed/bottom rows
q = 21,509 lbs/ft	**q = 18,145 lbs/ft**
T = 35,920 lbs	**T = 42,338 lbs**
T_u = 23,348 lbs	**T_u = 27,519 lbs**

DMA RIB CAPACITIES

w/o composite action (distributed load)



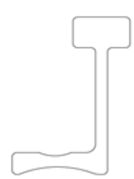
top view

$$M_{max} = q\left(\frac{(\omega/2)^2}{2}\right)$$

$$q\ (\text{lb/ft}) = \frac{2M_{max}}{(\omega/2)^2}$$

T (lbs) = q x ω

$T_u = \phi T$ (ϕ = 0.65) see above



Type VI Rib

F_y = 35 ksi

M_p = 16.18 k-ft

DMA Rib Capacities ($M_{max} = M_p$)

q = 23,774 lbs/ft

T_{allow} = 55,393 lbs

T_u = 36,005 lbs

CAPACITY OF ¾″ THREADED TIE RODS
(use ASTM A36 Threaded Rods, F_y = 36 ksi)

Area based on nominal diameter = 0.4418 in^2

$T_u = \phi F_yA = (0.90)F_yA$ = 14.3 kips

CAPACITY OF 1″ THREADED TIE RODS
(use ASTM A36 Threaded Rods, F_y = 36 ksi)

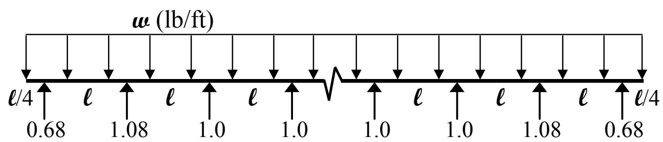
Area based on nominal diameter = 0.7854 in^2

$T_u = \phi F_yA = (0.90)F_yA$ = 25.4 kips

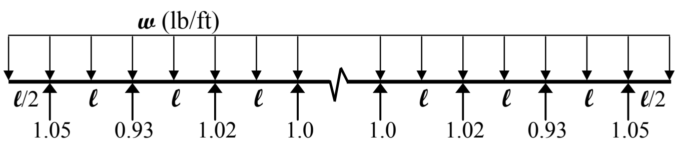
T_u = factored tensile resistance of the anchor tendon. For mild steel the resistance factor (ϕ =0.90) shall be applied to F_y (Table 11.5.6-1)

ANCHOR SPACING

There will be an increase in one or more of the end reactions due to the overhang of the wale beam at the end of the wall. The increase is minimal but a factor will be used to increase reactions accordingly. The increased reaction will be used to determine the anchor spacing across the entire level. The diagrams below model an infinitely long wale beam with deadmen anchors spaced a distance ℓ apart with a maximum overhangs of $\ell/4$ and $\ell/2$.



Reaction coefficients for span ℓ and span overhang $\ell/4$
Reaction = (Reaction coefficient) x $w\ell$



Reaction coefficients for span ℓ and span overhang $\ell/2$
Reaction = (Reaction coefficient) x $w\ell$

The beam analysis for an overhang of $\ell/4$ shows a maximum reaction coefficient of 1.08 (second reaction from the end).

The analysis for the $\ell/2$ overhang shows a maximum reaction coefficient of 1.05.

Based on the beam analysis an end factor of **1.08** will be used as a multiplier to accommodate the worst case scenario, while the maximum overhang will be established as $\ell/2$.

Maximum Reactions	**H = 6 ft**	**H = 8 ft**	**H = 10 ft**	**H = 12 ft**	**Increased Reactions**	**H = 6 ft**	**H = 8 ft**	**H = 10 ft**	**H = 12 ft**
T_1 (lb/ft)	513.3	819.0	728.6	1005.0	$1.08T_1$ (lb/ft)	554.4	884.5	786.9	1085.4
T_2 (lb/ft)	1458.0	2058.0	1724.0	2228.0	$1.08T_2$ (lb/ft)	1574.6	2222.6	1861.9	2406.2
T_3 (lb/ft)	n/a	n/a	1788.0	2702.0	$1.08T_3$ (lb/ft)	n/a	n/a	1931.0	2918.2

	Anchor Tendon Resistance (lbs)		DMA Structural Resistance (lbs)		[1]Minimum DMA Pullout Resistance (lbs)		
	¾″ Tie-Rods	1″ Tie-Rods	Top Row	Intrmd/Bottom	Top	Intrmd	Bottom
H = 6 ft	14,300	25,400	23,348	27,519	4,286	n/a	7,132
H = 8 ft	14,300	25,400	23,348	27,519	4,286	n/a	9,272
H = 10 ft	14,300	25,400	23,348	27,519	4,821	8,737	11,947
H = 12 ft	14,300	25,400	23,348	27,519	5,356	9,807	13,552

[1]Pullout resistance of the anchors provides the least resistance and will therefore be used to determine anchor spacing.

	[1]DMA Spacings, (ft)		
	Top	**Intrmd**	**Bottom**
H = 6 ft	4.50	n/a	4.50
H = 8 ft	4.50	n/a	4.17
H = 10 ft	4.50	4.50	4.50
H = 12 ft	4.50	4.07	4.50

Minimum anchor resistance at each level, R_A (lbs)

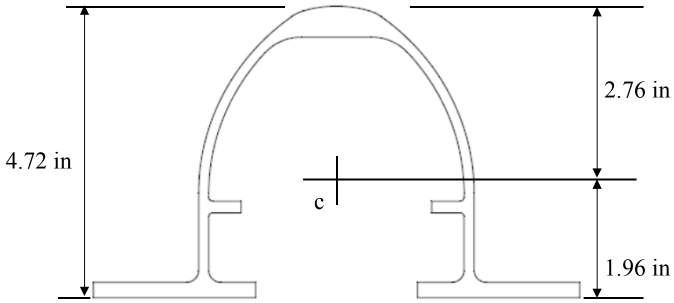
Increased maximum reaction at each level, T_{max} (lb/ft)

DMA Spacing, S (ft) = R_A/T_{max} [S ≤ 4'-6"]

[1]Maximum DMA spacings of 4'-6" are established to enhance constructability, improve safety, and minimize deflection.

MOMENT CAPACITY CHECKS

Wale Beam (6061-T6 Alloy, F_y = 35,000 psi)



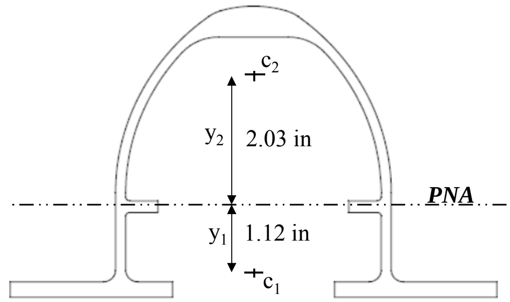
$A = 3.5881\ in^2$

$I = 11.104\ in^4$

$S = I/c = 11.104/2.76 = 4.02\ in^3$

M_y (ft-lbs) = $F_y S/12$ = 11,725 lb-ft

Plastic Analysis



$$M_p = \frac{F_y A(y_1 + y_2)}{2} = 197{,}794 \text{ in-lbs } (16{,}483 \text{ lb-ft})$$

The design flexural strength for plastic analysis = $\phi_b M_p$

ϕM_p = 0.8(16,483) = 13,186 lb-ft

AASHTO LRFD Bridge Design Specifications Table 7.5.4-1 Resistance Factors (Aluminum Structures)

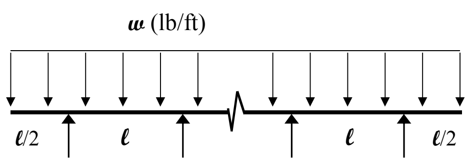
Tension in Extreme Fibers of Beams, $\phi_u = 0.8$

Compression in Extreme Fibers of Beams, $\phi_b = 0.8$

Compression in Components of Beams, $\phi_c = 0.8$

$0.8F_y$ = 0.8(35,000 psi) = 28,000 psi

Check the wale beam moment capacity assuming the maximum moment on the wale beam occurrs at the overhang reaction:



$$\mathbf{M_{max}\ (lb\text{-}ft/ft) = \frac{w(\ell/2)^2}{2}}$$

w_{max} = 2918.2 lb/ft

ℓ_{max} = 4.5 ft

M_{max} = 7,386.7 lb-ft (88,640.4 lb-in)

σ_{max} = Mc/I = (88640.4)(2.76)/11.104 = 22,032.4 psi

less than 28,000 psi therefore okay

Increased Max Reactions	H = 6 ft	H = 8 ft	H = 10 ft	H = 12 ft
$1.08T_1$ (lb/ft)	554.4	884.5	786.9	1085.4
$1.08T_2$ (lb/ft)	1574.6	2222.6	1861.9	2406.2
$1.08T_3$ (lb/ft)	n/a	n/a	1931.0	2918.2

Corrugated (9" x 2½") Aluminum Structural Plate, ASP

0.150" ASP

I (in^4/ft) = (0.1249 in^4/in)(12 in/ft) = 1.4988 ≈ 1.50 in^4/ft

c = (2.50 + 0.150)/2 = 1.325 in

S (in^3/ft) = I/c = 1.131 in^3/ft

M_y (lb-ft/ft) = $F_y S$ = 24,000(1.131)(1/12) = 2,262 lb-ft/ft

M_p = 3.18 k-ft/ft (published) (M_p = 3,180 lb-ft/ft)

Flexural capacity of vertical elements $\phi M_p = 0.9M_p$ (Table 11.5.6-1)

ϕM_p = 0.90(3,180) = 2,862 lb-ft/ft

M_{max} (lb-ft/ft) from sheet entitled "Load Cases"			
H = 6 ft	**H = 8 ft**	**H = 10 ft**	**H = 12 ft**
804.8	1015.0	698.9	1434.0

The factored moment resistance of the 0.150" aluminum structural plate exceeds the maximum moment in each case above.

Aluminum Structural Plate Headwall - General Notes:

1. Aluminum structural plate headwalls shall conform to the latest requirements of AASHTO M219 or ASTM B746 with a minimum thickness of 0.150".

2. Headwalls may incorporate the full variety of shapes and sizes available in corrugated metal pipe and structural plate culverts (arch pipe, arch, box culvert, et al). Additionally, headwalls may be equipped with wingwalls of the same design and material. However, it shall be incumbent upon the project engineer to ensure constructability and structural adequacy through the implementation of submittal requirements (shop drawings, calculations, etc).

3. It shall be the responsibility of the installation crew to implement sound installation practices consistent with AASHTO LRFD Bridge Construction Practices. As necessary and at the discretion of the project engineer, the headwall manufacturer or other expertise may be enacted to supervise construction when a bid item for such activity has been included in the contract documents or project specifications.

4. A culvert stub shall be integral with the headwall by means of a full periphery weld on both the interior and exterior of their junction. The headwall is properly placed at the design elevation by ensuring the stub is placed at grade for the culvert crossing.

5. Backfill placement and compaction shall be consistent with Section 26 of the AASHTO LRFD Bridge Construction Specifications. All backfill in the structural zone shall be #57 washed stone or other as approved by the engineer of record.

6. The headwall shall be properly shored through the backfilling process. In general, the wall should be braced at the wale line located above the fill line until the corresponding anchor is completely embedded. The wall shall also be braced at the top anchor location until completely backfilled.

7. All steel components (nuts, bolts, tie back rods) shall have a hot-dipped galvanized coating.

8. As a matter of expedience and to the extent practical, the headwall-culvert system may be completely or partially assembled and lifted as a unit to facilitate placement of the unit in a prepared excavation complete with bedding to grade.

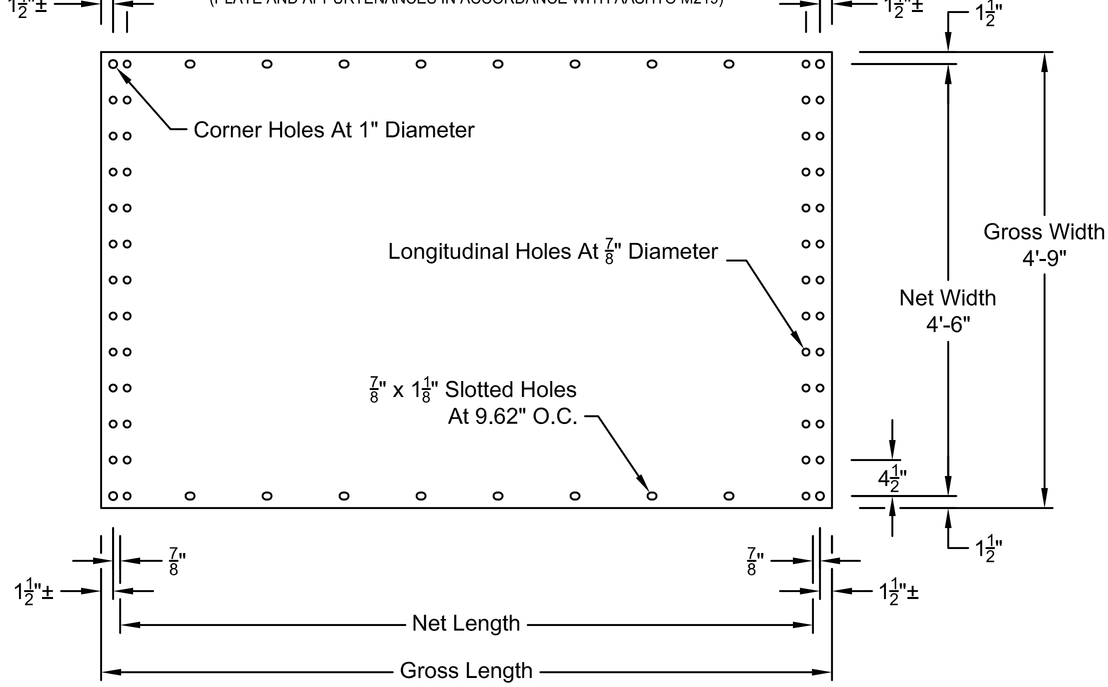
EXHIBIT 1

Standard Sizes and Weights									
Length	**Net Length**	**Gross Length**	**Weight Per Plate**						
***N**	**in.**	**in.**	**.100**	**.125**	**.150**	**.175**	**.200**	**.225**	**.250**
8N	76.96	81.71	52.7	65.9	79.1	92.2	104.7	117.8	130.9
9N	86.58	91.33	58.9	73.6	88.4	103.1	117.8	132.6	147.3
10N	96.20	100.95	65.1	81.4	97.7	114.0	130.2	146.5	162.8
11N	105.82	110.57	71.3	89.2	107.0	124.8	142.6	160.5	178.3
12N	115.44	120.19	77.5	96.9	116.3	135.7	155.1	174.4	193.8
13N	125.06	129.81	83.7	104.7	125.6	146.5	167.5	188.4	209.3
14N	134.68	139.43	89.9	112.4	134.9	157.4	179.9	202.4	224.8
15N	144.30	149.05	96.1	120.2	144.2	168.3	192.3	216.3	240.4
16N	153.92	158.67	102.4	127.9	153.5	179.1	204.7	230.3	257.8
17N	163.54	168.29	108.6	135.7	162.8	190.0	217.1	244.2	271.4
18N	173.16	177.91	114.8	143.5	172.1	200.8	229.5	258.2	286.9

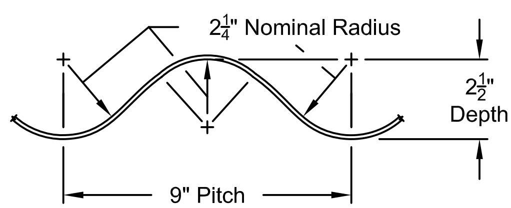
Notes: (1) Weights based on nominal thickness. (2) Bolt holes have not been deducted.
*N = 9.62"

Standard Plate Detail

(PLATE AND APPURTENANCES IN ACCORDANCE WITH AASHTO M219)



Standard Plate Corrugation



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