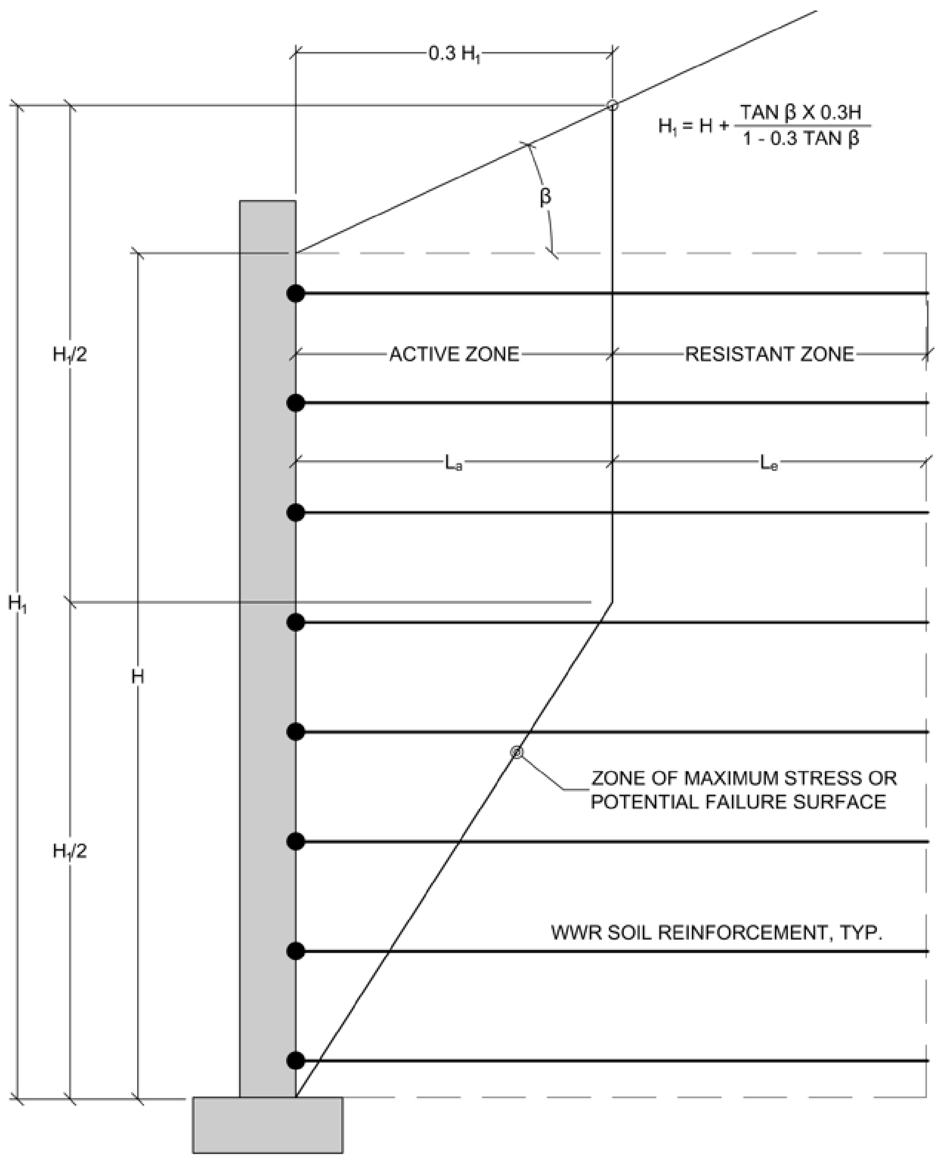
Welded Wire Reinforcement in Mechanically Stabilized Earth (MSE) Walls – Part II

In WRI’s March 2024 Technical Blog Part I, we introduced the use of WWR in mechanically stablized earth (MSE) walls per Article 11.10 of the 2020 AASHTO LRFD Bridge Design Specifications, Ninth Edition (AASHTO LRFD). Here in Part II we will look more closely at the AASHTO LRFD provisions and how WWR fits into the design procedure.

WWR mats as an inextensible soil reinforcement in MSE walls are checked for three primary attributes:

1. Steel design life considerations due to embedment in corrosive backfill material
2. Connection rupture strength and pullout length at zone of maximum stress
3. Connection rupture strength at wall facing

For a given wall we first need to define the zone of maximum stress (potential failure surface).



AASHTO LRFD Figure 11.10.6.3.1-1 for inextensible reinforcement

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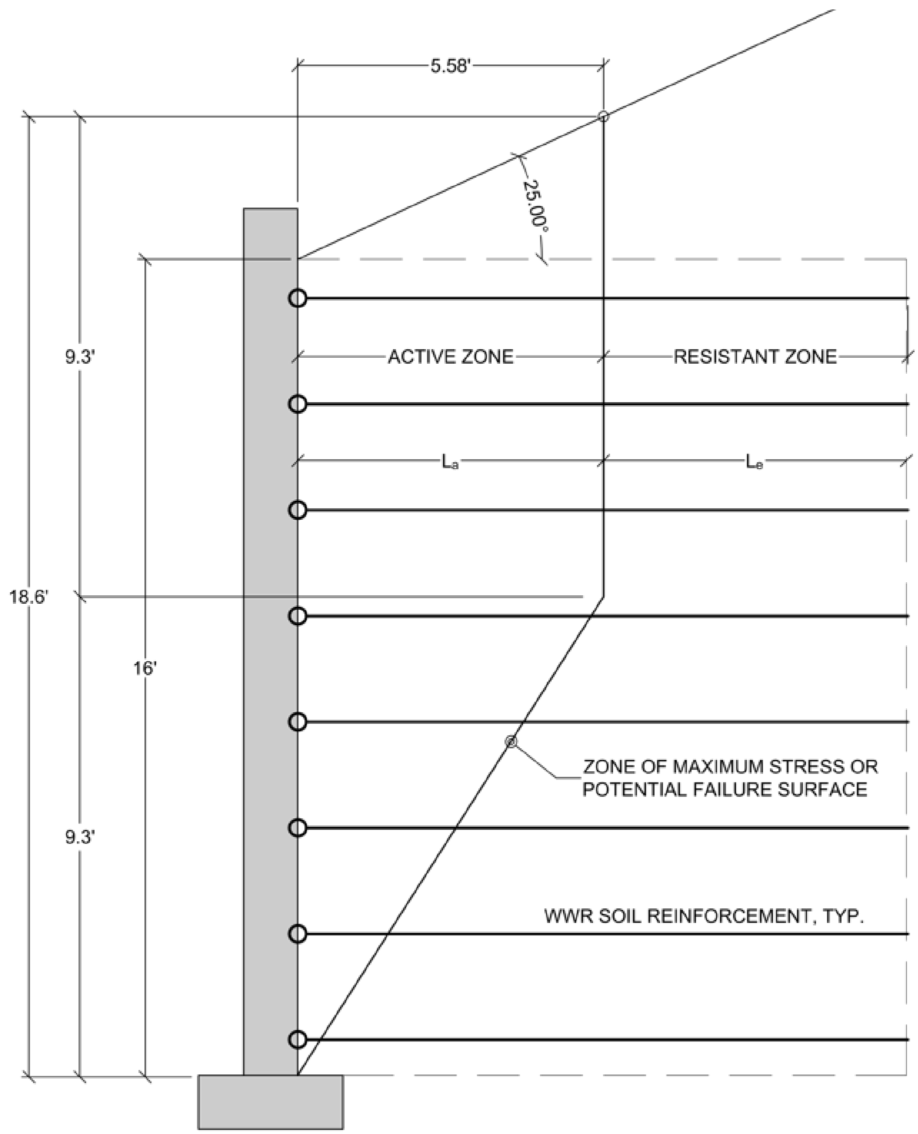
For the purposes of this technical blog and to illustrate the WWR-related design considerations, we will work through a selective design example. *Note that this example is targeted to WWR soil reinforcement considerations only. There are numerous overall wall design attributes not captured or calculated here, including but not limited to wall performance criteria, global stability, settlement, and external stability.*

Wall height = 16 feet
Sloping backfill angle = β = 25°

$$H_1 = H + \frac{Tan\,\beta \times 0.3H}{1 - 0.3Tan\,\beta} = 16' + \frac{Tan\,25° \times 0.3 \times 16'}{1 - 0.3Tan\,25°} = 18.6\,ft$$

$$0.3H_1 = 0.3 \times 18.6' = 5.58'$$



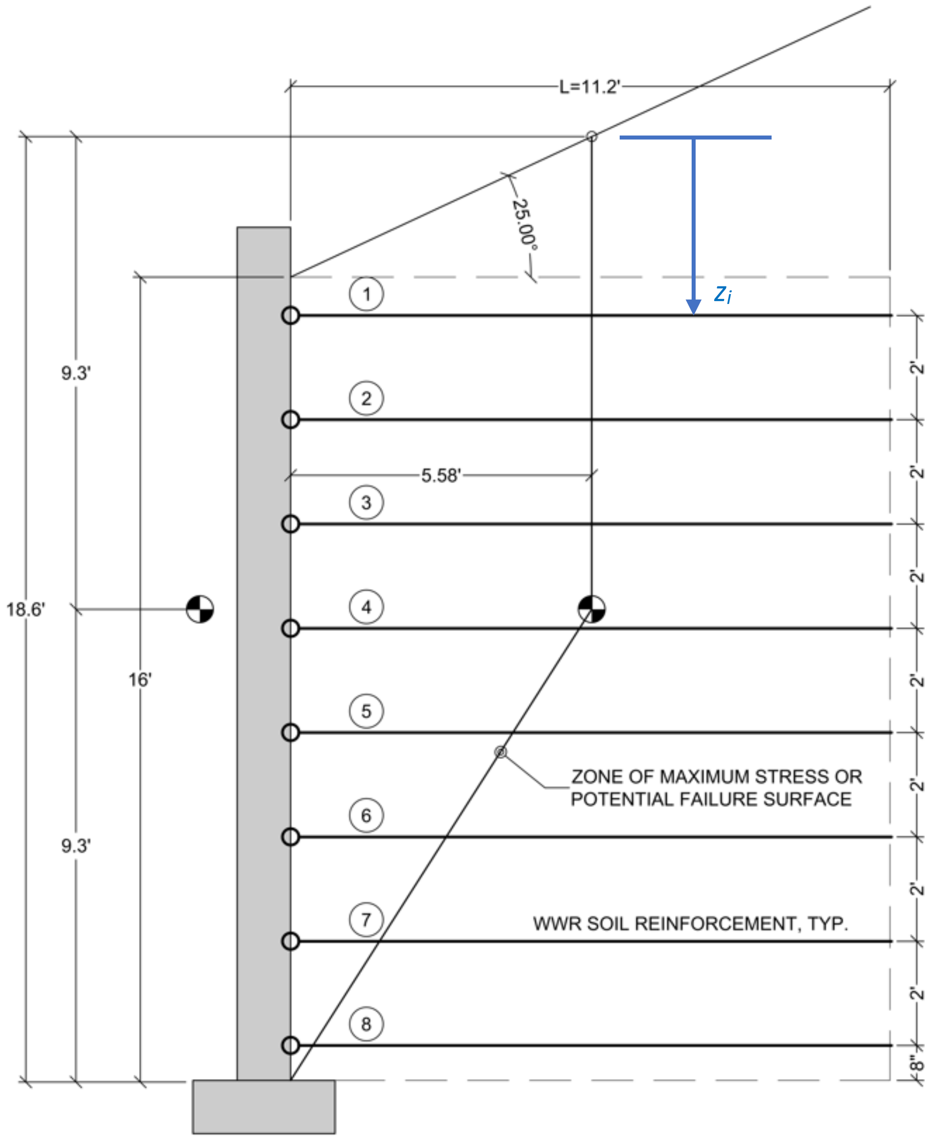
Example Figure 1 – location of potential failure surface

WWR can be used in MSE walls as both a wall facing element and as the tensile reinforcement relied upon to engage the soil mass behind the wall (some proprietary systems even combine these two wall features into common element). Our focus here is on the latter application only.

AASHTO LRFD Article 11.10.2.1 states that the minimum soil reinforcement length shall be 70% of the wall height measured from the leveling pad, with increases in length as required for surcharges, other external loads, or for soft foundation soils. AASHTO LRFD C11.10.2.1 states that a minimum reinforcement length of 8 feet is recommended based on historical practice.

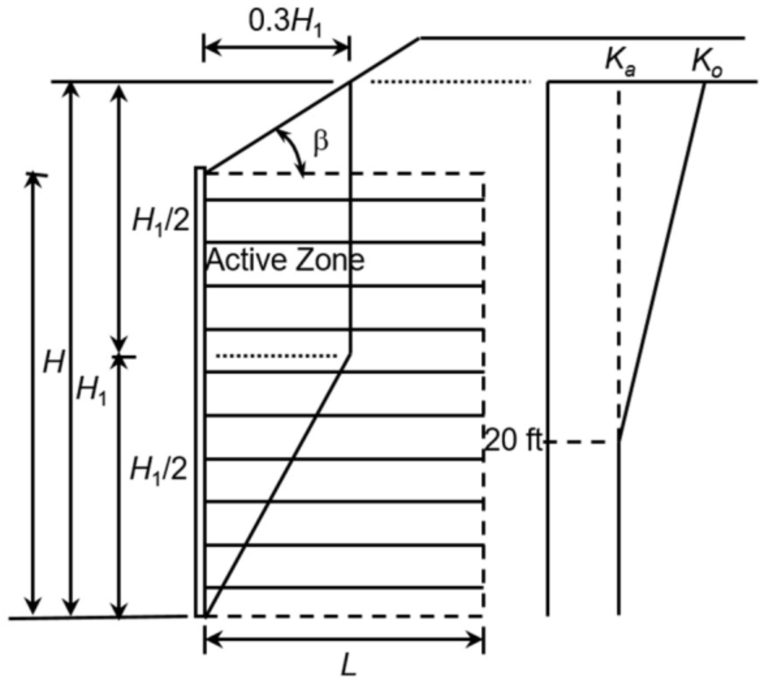
For the wall shown in Example Figure 1, 70% of 16 feet is 11.2 feet, which exceeds the prescriptive 8-foot minimum. We will start with a **trial length L = 11.2 feet.**

Trial length of WWR soil reinforcement layers, as well as layer vertical spacing is shown in Example Figure 2.



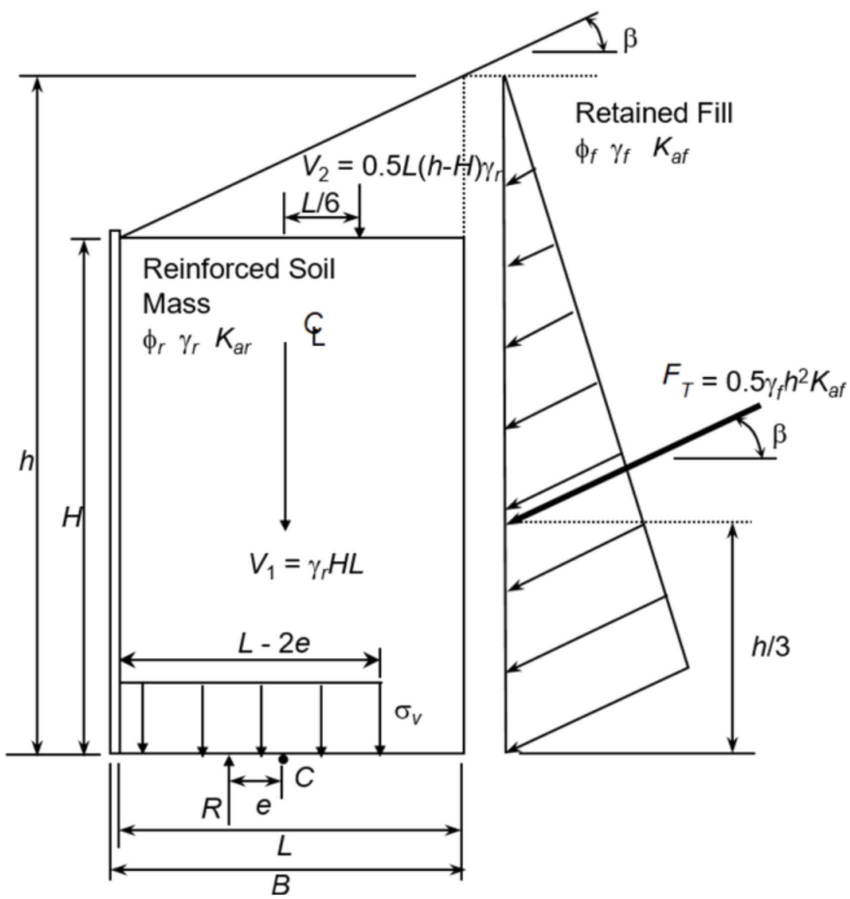
Example Figure 2 – trial soil reinforcement configuration

Per AASHTO LRFD Article 11.10.6.2.1d on the Coherent Gravity Method (CGM), for steel-reinforced wall systems, the lateral earth pressure coefficient (k_r) used shall be equal to k_o at the point of intersection of the theoretical failure surface with the ground surface at or above the wall top, transitioning to k_a at a depth of 20.0 feet below the intersection point, and constant at k_a at depths greater than 20.0 feet. This is shown in AASHTO LRFD Figure 11.10.6.2.1d-1.



AASHTO LRFD Figure 11.10.6.2.1d-1: determination of lateral earth pressure coefficients for internal stability design of steel-reinforced MSE walls using the Coherent Gravity Method

The calculation of T_{max} using the CGM is predicated on a Meyerhof Vertical Stress Distribution as shown in AASHTO LRFD Figure 11.10.6.2.1d-2.



AASHTO LRFD Figure 11.10.6.2.1d-2: Forces and stresses for calculating Meyerhof Vertical Stress Distribution in MSE Walls

We will calculate T_{max} at each level of soil reinforcement. The following equations apply.

AASHTO LRFD Equation 11.10.6.2.1d-1:

$$T_{max} = S_v k_r \sigma_v$$

AASHTO LRFD Equation 11.10.6.2.1d-2:

$$\sigma_v = \frac{V_1 + V_2 + F_T sin\, \beta}{L - 2e}$$

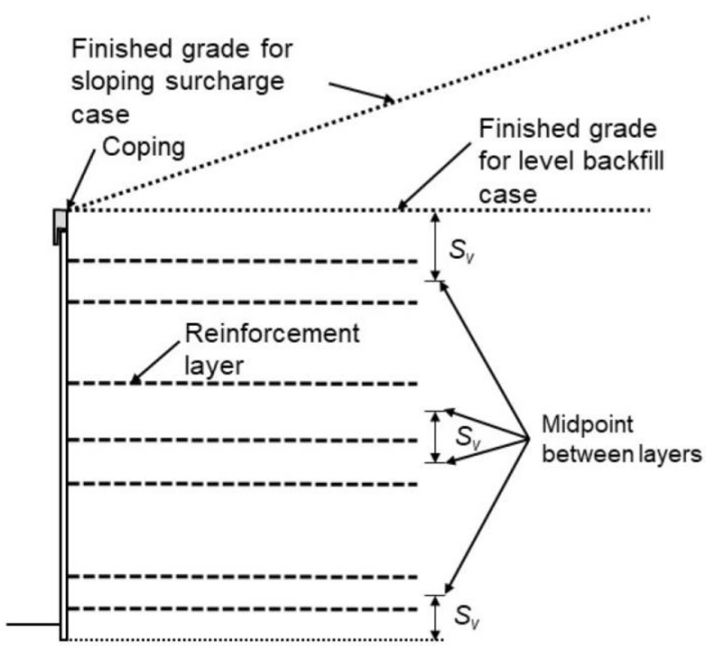
AASHTO LRFD Equation 11.10.6.2.1d-3:

$$e = \frac{F_T(\cos\beta)(h/3) - F_T(\sin\beta)(L/2) - V_2(L/6)}{V_1 + V_2 + F_T(\sin\beta)}$$

Define the individual WWR soil reinforcement levels. See Example Figure 2 for point from which vertical dimension "z" is referenced.

WWR level,i	Z_i
1	3.93'
2	5.93'
3	7.93'
4	9.93'
5	11.93'
6	13.93'
7	15.93'
8	17.93'

Determine the tributary layer thickness, S_v, per AASHTO LRFD 11.10.6.2.1b and Figure 11.10.6.2.1b-1.



AASHTO LRFD Figure 11.10.6.2.1b: Determination of tributary layer thickness, S_v

WWR level	z	S_v
1	3.93’	2.33’
2	5.93’	2’
3	7.93’	2’
4	9.93’	2’
5	11.93’	2’
6	13.93’	2’
7	15.93’	2’
8	17.93’	1.67’

For this example, the following soil properties apply. Note that separate sets of pressure coefficients apply for the reinforced soil mass and the retained backfill.

Reinforced Soil Mass

$\gamma_r = 125\ pcf\ unit\ weight\ of\ soil\ in\ reinforcement\ zone$

$\varphi'_{f,reinforced} = 35°\ internal\ active\ earth\ pressure\ coefficient$

Per AASHTO LRFD Article 11.10.6.2.1c, k_a shall be determined assuming no wall interface friction and level backfill slope. AASHTO LRFD Equation 3.11.5.3-1 simplifies to Equation C11.10.6.2.1c-1 for active pressure, while at-rest pressure is calculated using AASHTO LRFD Equation 3.11.5.2-1:

$$active: k_{a,reinf} = tan^2\left(45 - \frac{\varphi'_{f,reinforced}}{2}\right) = 0.270$$

$$at\ rest: k_{o,reinf} = 1 - \sin\left(\varphi'_{f,reinforced}\right) = 0.426$$

Retained Backfill

$\gamma_f = 120\ pcf\ unit\ weight\ of\ soil\ backfill\ behind\ and\ above\ wall$

$\varphi'_{f,backfill} = 30°\ internal\ active\ earth\ pressure\ coefficient$

$\theta = angle\ of\ back\ face\ of\ wall\ relative\ to\ horizontal = 90°$

$\delta = friction\ angle\ between\ soil\ zones = 0.67 \times \varphi'_{f,backfill} = 20.1°$

$\beta = angle\ of\ fill\ relative\ to\ horizontal = 25°$

Per AASHTO LRFD Article 11.10.5.2, external stability design of MSE walls shall be taken as specified in Article 3.11.5.8. Per Article 3.11.5.8, k_a shall account for backfill slope as well as the interface friction angle between soil zones.

$$\Gamma = \left[1 + \sqrt{\frac{\sin\left(\varphi'_{f,backfill} + \delta\right) \times \sin\left(\varphi'_{f,backfill} - \beta\right)}{\sin\left(\theta - \delta\right) \times \sin\left(\theta + \beta\right)}}\right]^2$$

$$\Gamma = 1.639$$

$$k_{a,backfill} = \frac{sin^2(\theta - \varphi'_{f,backfill})}{\Gamma \times [sin^2\theta \times \sin\left(\theta - \delta\right)]}$$

$$k_{a,backfill} = 0.487$$

As mentioned previously, for steel-reinforced wall systems, the reinforced soil mass lateral earth pressure coefficient used shall be equal to k_o at the point of intersection of the theoretical failure surface with the ground surface at or above the wall top, transitioning to k_a at a depth of 20.0 feet below the intersection point, and constant at k_a at depths greater than 20.0 feet. By linear interpolation, this gives us the following kr values at each reinforcement level.

WWR level	z	S_v	k_r
1	3.93’	2.33’	(0.426-0.270)/20 x (20-3.93) + 0.270 = 0.395
2	5.93’	2’	(0.426-0.270)/20 x (20-5.93) + 0.270 = 0.380
3	7.93’	2’	(0.426-0.270)/20 x (20-7.93) + 0.270 = 0.364
4	9.93’	2’	(0.426-0.270)/20 x (20-9.93) + 0.270 = 0.349
5	11.93’	2’	(0.426-0.270)/20 x (20-11.93) + 0.270 = 0.333
6	13.93’	2’	(0.426-0.270)/20 x (20-13.93) + 0.270 = 0.317
7	15.93’	2’	(0.426-0.270)/20 x (20-15.93) + 0.270 = 0.302
8	17.93’	1.67’	(0.426-0.270)/20 x (20-17.93) + 0.270 = 0.286

Length of soil reinforcement layers, L, is a constant 11.2 feet as previously defined. V1 and V2 are calculated per equations as illustrated previously in AASHTO LRFD Figure 11.10.6.2.1d-2.

WWR level	h	S_v	k_r	L	V_1
1	3.93’	2.33’	0.395	11.2’	125 pcf x 1.33’ x 11.2’= 1,862 lb/ft
2	5.93’	2’	0.380	11.2’	125 pcf x 3.33’ x 11.2’= 4,662 lb/ft
3	7.93’	2’	0.364	11.2’	125 pcf x 5.33’ x 11.2’= 7,462 lb/ft
4	9.93’	2’	0.349	11.2’	125 pcf x 7.33’ x 11.2’= 10,262 lb/ft
5	11.93’	2’	0.333	11.2’	125 pcf x 9.33’ x 11.2’= 13,062 lb/ft
6	13.93’	2’	0.317	11.2’	125 pcf x 11.33’ x 11.2’= 15,862 lb/ft
7	15.93’	2’	0.302	11.2’	125 pcf x 13.33’ x 11.2’= 18,662 lb/ft
8	17.93’	1.67’	0.286	11.2’	125 pcf x 15.33’ x 11.2’= 21,462 lb/ft

WWR level	h	S_v	k_r	L	V_1	V_2
1	3.93’	2.33’	0.395	11.2’	1,862 lb/ft	0.5 x 11.2’ x 2.6’ x 125 pcf = 1,820 lb/ft
2	5.93’	2’	0.380	11.2’	4,662 lb/ft	1,820 lb/ft
3	7.93’	2’	0.364	11.2’	7,462 lb/ft	1,820 lb/ft
4	9.93’	2’	0.349	11.2’	10,262 lb/ft	1,820 lb/ft
5	11.93’	2’	0.333	11.2’	13,062 lb/ft	1,820 lb/ft
6	13.93’	2’	0.317	11.2’	15,862 lb/ft	1,820 lb/ft
7	15.93’	2’	0.302	11.2’	18,662 lb/ft	1,820 lb/ft
8	17.93’	1.67’	0.286	11.2’	21,462 lb/ft	1,820 lb/ft

The lateral earth force at the back of the MSE wall mass, F_T, is calculated at each reinforcement level per the equation form illustrated previously in AASHTO LRFD Figure 11.10.6.2.1d-2, with “z” substituted for h and $k_{a,backfill}$ representing K_{af}.

WWR level	z	F_T
1	3.93’	0.5 x 120 pcf x 3.93^2 x 0.487 = 451 lb/ft
2	5.93’	0.5 x 120 pcf x 5.93^2 x 0.487 = 1,028 lb/ft
3	7.93’	0.5 x 120 pcf x 7.93^2 x 0.487 = 1,837 lb/ft
4	9.93’	0.5 x 120 pcf x 9.93^2 x 0.487 = 2,881 lb/ft
5	11.93’	0.5 x 120 pcf x 11.93^2 x 0.487 = 4,159 lb/ft
6	13.93’	0.5 x 120 pcf x 13.93^2 x 0.487 = 5,670 lb/ft
7	15.93’	0.5 x 120 pcf x 15.93^2 x 0.487 = 7,415 lb/ft
8	17.93’	0.5 x 120 pcf x 17.93^2 x 0.487 = 9,394 lb/ft

Eccentricity, e, is calculated as previously noted on Page 5 of this Technical Blog, with “z” substituted for h.

$$e = \frac{F_T(\cos\beta)(z/3) - F_T(\sin\beta)(L/2) - V_2(L/6)}{V_1 + V_2 + F_T(\sin\beta)}$$

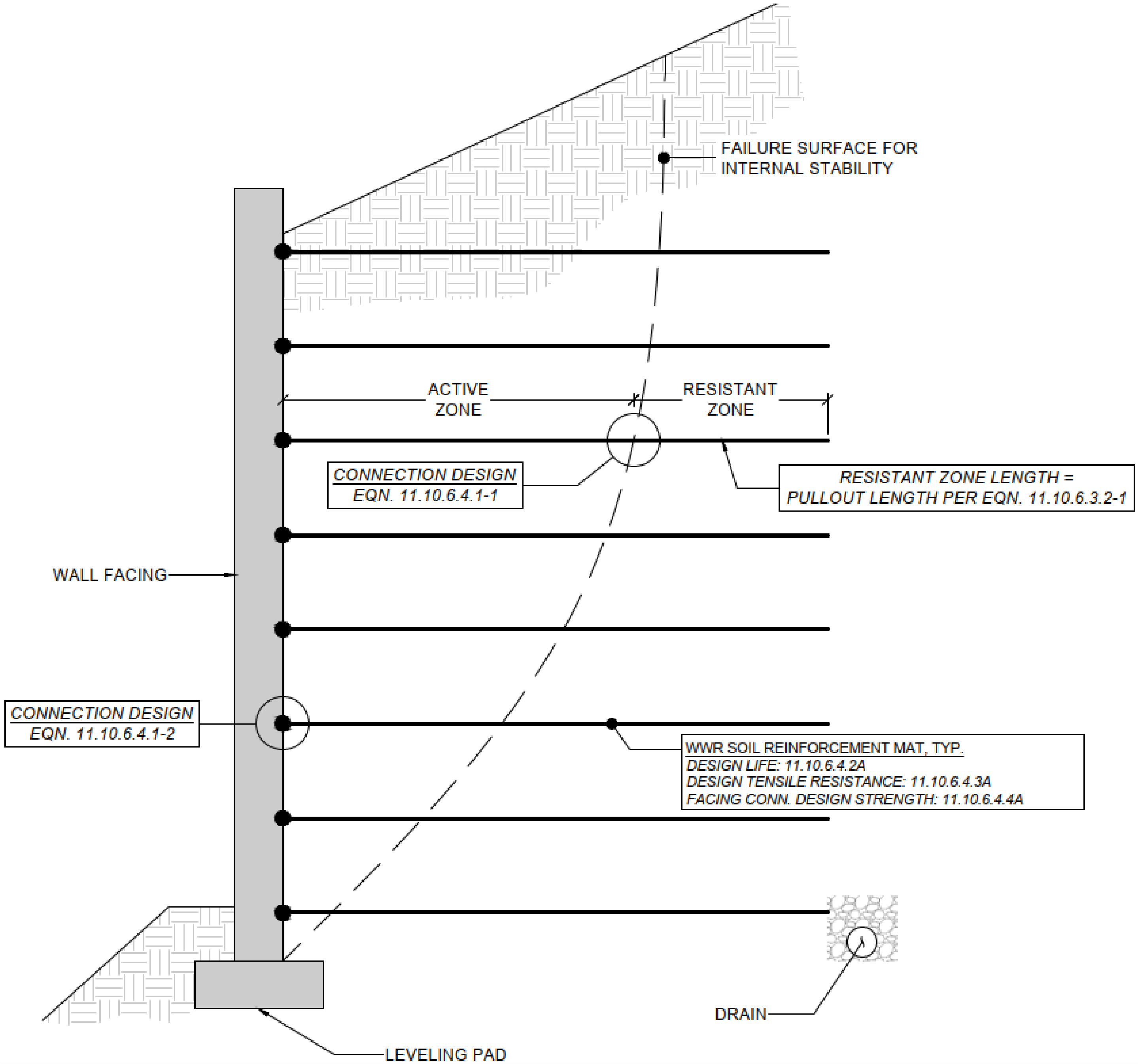
WWR level	z	F_T	V_1	V_2	e
1	3.93’	451 lb/ft	1,862 lb/ft	1,820 lb/ft	-1.01 ft, use 0’
2	5.93’	1,028 lb/ft	4,662 lb/ft	1,820 lb/ft	-0.58 ft, use 0’
3	7.93’	1,837 lb/ft	7,462 lb/ft	1,820 lb/ft	-0.32 ft, use 0’
4	9.93’	2,881 lb/ft	10,262 lb/ft	1,820 lb/ft	-0.12 ft, use 0’
5	11.93’	4,159 lb/ft	13,062 lb/ft	1,820 lb/ft	0.105 ft
6	13.93’	5,670 lb/ft	15,862 lb/ft	1,820 lb/ft	0.351 ft
7	15.93’	7,415 lb/ft	18,662 lb/ft	1,820 lb/ft	0.624 ft
8	17.93’	9,394 lb/ft	21,462 lb/ft	1,820 lb/ft	0.927 ft

We can now summarize the vertical pressure at each soil reinforcement elevation, σ_v, as well as the service-level soil load applied to reinforcement, T_{max}, at each level.

WWR level	F_T	V_1	V_2	e	σ_v
1	451 lb/ft	1,862 lb/ft	1,820 lb/ft	0.00 ft	346 psf
2	1,028 lb/ft	4,662 lb/ft	1,820 lb/ft	0.00 ft	618 psf
3	1,837 lb/ft	7,462 lb/ft	1,820 lb/ft	0.00 ft	898 psf
4	2,881 lb/ft	10,262 lb/ft	1,820 lb/ft	0.00 ft	1,148 psf
5	4,159 lb/ft	13,062 lb/ft	1,820 lb/ft	0.105 ft	1,514 psf
6	5,670 lb/ft	15,862 lb/ft	1,820 lb/ft	0.351 ft	1,913 psf
7	7,415 lb/ft	18,662 lb/ft	1,820 lb/ft	0.624 ft	2,373 psf
8	9,394 lb/ft	21,462 lb/ft	1,820 lb/ft	0.927 ft	2,916 psf

WWR level	S_v	k_r	σ_v	T_{max}
1	2.33’	0.395	346 psf	318 lb/ft
2	2’	0.380	618 psf	470 lb/ft
3	2’	0.364	898 psf	654 lb/ft
4	2’	0.349	1,148 psf	801 lb/ft
5	2’	0.333	1,514 psf	1,008 lb/ft
6	2’	0.317	1,913 psf	1,213 lb/ft
7	2’	0.302	2,373 psf	1,433 lb/ft
8	1.67’	0.286	2,916 psf	1,393 lb/ft

From Part I of this Technical Blog (March 2024), the diagram below illustrates important components of the WWR design.



Example Figure 3 – WWR design components

As mentioned previously, WWR mats as an inextensible soil reinforcement in MSE walls are checked for three primary attributes:

1. Steel design life considerations due to embedment in corrosive backfill material (Article 11.10.6.4.2a)
2. Connection rupture strength and pullout length at zone of maximum stress (Eqns. 11.10.6.4.1-1 and 11.10.6.3.2-1)
3. Connection rupture strength at wall facing (Eqn. 11.10.6.4.1-2)

Steel Design Life Considerations

For this example the WWR mats are assumed to be hot-dip galvanized in accordance with ASTM A1060, ASTM A123, and AASHTO M111.

We must select a trial WWR style for the purposes of evaluating design life, connection strengths, and pullout length requirements. Assume a WWR mat comprised of W5.0 longitudinal wires (perpendicular to wall facing) and W5.0 transverse wires (parallel to wall facing). W5.0 wires have a a diameter of 0.252 inches. AASHTO LRFD states that transverse wire diameter shall be less than or equal to the longitudinal wire diameter, and that galvanized coatings shall be applied after WWR fabrication to a minimum thickness of 3.4 mils (2.0 ounces per square foot). Note that this exceeds the reference ASTM A1060 Coating Thickness Grade 80 that corresponds to 3.1 Mils (1.90 ounces per square foot) of zinc coating after fabrication.

The soil backfill in this example is assumed to be nonaggressive as outlined in AASHTO LRFD. For structural design, sacrificial thicknesses shall be computed for each exposed surface as follows:

Loss of galvanizing: 0.58 mil per year, for first 2 years
0.16 mil per year, for subsequent years

Loss of carbon steel: 0.47 mil per year after zinc depletion

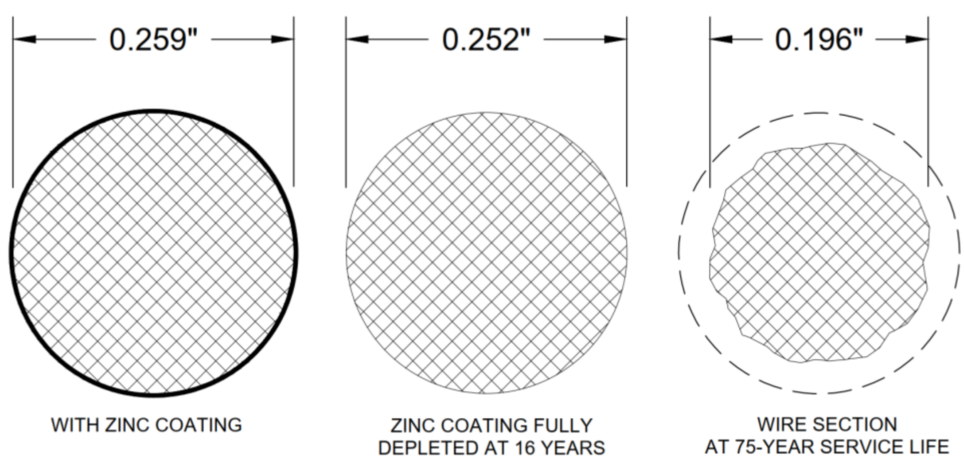
Since this is a permanent retaining wall, the wall should be designed for a minimum service life of 75 years per AASHTO LRFD Article 11.5.1.

Zinc coating depletion is determined as follows:

3.4 mil = (2 years)(0.58 mil/yr) + (x subsequent years)(0.16 mil/yr); x = 14 years

Remaining exposure time for carbon steel loss = 75 years minus 16 years = 59 years

Loss of carbon steel = (59 years)(0.47 mil/yr) = 28 mils = 0.028 inches



Example Figure 4 – Illustration of zinc depletion and carbon steel loss

It is worth noting that per AASHTO LRFD, epoxy coatings can be used. Currently, however, there is insufficient evidence regarding their long-term performance to be considered equivalent to galvanizing in an MSE wall application. If epoxy-type coatings are to be used, they should meet the requirements of ASTM A884 and have a minimum thickness of 16 mils.

Our trial WWR style is 3x3 W5.0/W5.0, i.e., W5.0 in both directions as previously noted, with wires at 3" on center in both directions. Corrected for corrosion loss, the cross-sectional area of each wire is reduced from 0.050 in^2 to π x 0.196^2/4 = 0.030 in^2.

Steel design life considerations have been completed.

Calculation status:

1. **Steel design life considerations due to embedment in corrosive backfill material**
2. Connection rupture strength and pullout length at zone of maximum stress
3. Connection rupture strength at wall facing

Pullout Length at Zone of Maximum Stress

Per AASHTO LRFD Equation 11.10.6.3.2-1, the effective pullout length shall be determined as follows:

$$L_e \geq \frac{\gamma_{p-EV} \times T_{max}}{\varphi \times F^* \times \alpha \times \sigma_v \times C \times R_c}$$

Values for L_e at each reinforcement level are easily determined by linear interpolation from Example Figures 1 and 2.

WWR level	T_{max}	L_e
1	318 lb/ft	5.62'
2	470 lb/ft	5.62'
3	654 lb/ft	5.62'
4	801 lb/ft	6.00'
5	1,008 lb/ft	7.19'
6	1,213 lb/ft	8.39'
7	1,433 lb/ft	9.59'
8	1,393 lb/ft	10.79'

$\gamma_{p-EV} = 1.35 = load\ factor\ for\ vertical\ earth\ pressure\ per\ Table\ 3.4.1(2)$

$\varphi = 0.90 = resistance\ factor\ for\ reinforcement\ pullout\ from\ AASHTO\ LRFD\ Table\ 11.5.7(1)$

$\alpha = 1.0 = scale\ effect\ correction\ factor$

$C = 2.0 = reinforcement\ surface\ area\ geometry\ factor$

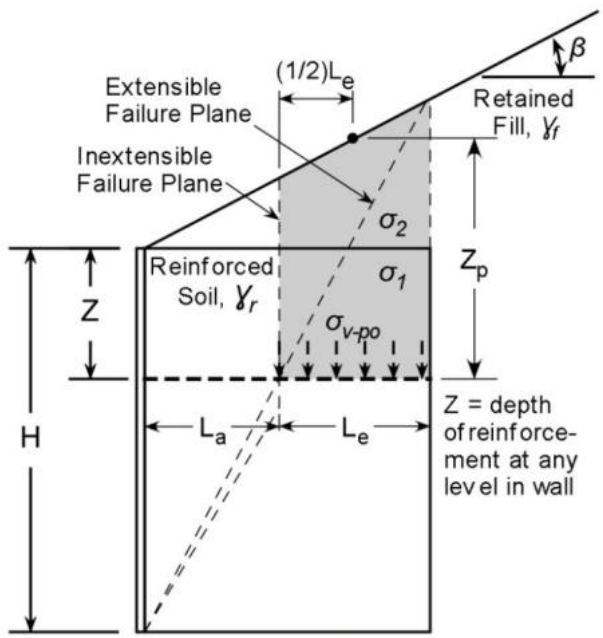
$R_c = 1.0 = reinforcement\ coverage\ ratio\ (WWR\ mats\ are\ \text{continuous}\ in\ direction\ parallel\ to\ wall\ facing)$

F^* is the pullout friction factor, and is contingent upon a vertical distance Z_p measured from the top of the sloped backfill.

For pullout resistance, the vertical stress at the reinforcement level in the resistant zone, σ_v, is determined from AASHTO Figure 11.10.6.3.2-1. Note that this vertical stress calculation is <u>not</u> the same as that which was used in our previous calculations of T_{max}. to differentiate, then, we will refer to it as σ_{v-po}.

For γ_r = 125 pcf and γ_f = 120 pcf:

WWR level	T_{max}	L_e	Z	Z_p	σ_1	σ_2	$\sigma_{v\text{-}po}$
1	318 lb/ft	5.62’	1.33’	5.24’	166 psf	469 psf	635 psf
2	470 lb/ft	5.62’	3.33’	7.24’	416 psf	469 psf	885 psf
3	654 lb/ft	5.62’	5.33’	9.24’	666 psf	469 psf	1135 psf
4	801 lb/ft	6.00’	7.33’	11.15’	916 psf	458 psf	1374 psf
5	1,008 lb/ft	7.19’	9.33’	12.88’	1166 psf	426 psf	1592 psf
6	1,213 lb/ft	8.39’	11.33’	14.60’	1416 psf	392 psf	1808 psf
7	1,433 lb/ft	9.59’	13.33’	16.32’	1666 psf	359 psf	2025 psf
8	1,393 lb/ft	10.79’	15.33’	18.04’	1916 psf	325 psf	2241 psf



Nominal Vertical Confining Pressure :

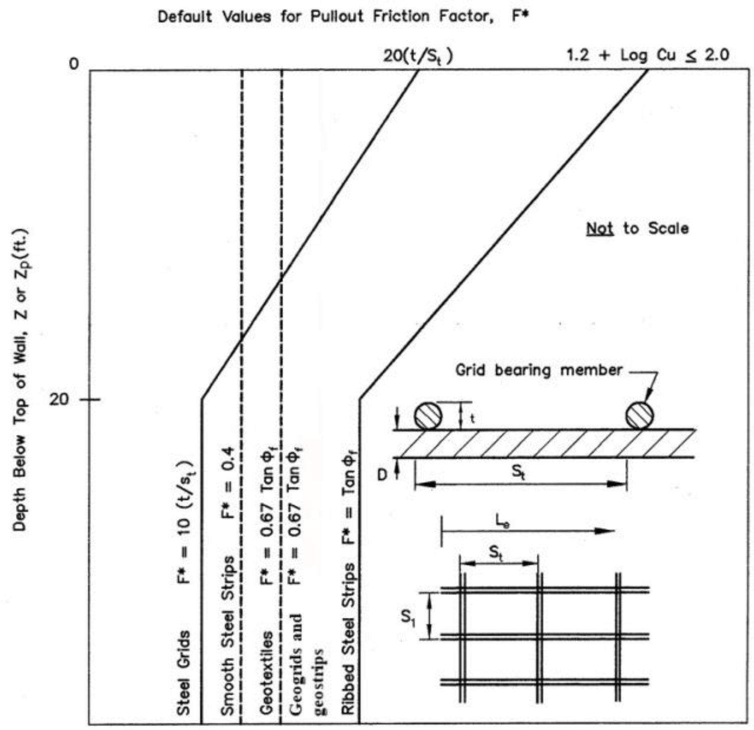
$\sigma_1 = \gamma_r Z$

$\sigma_2 = \gamma_f (Z_p - Z)$ for sloping backfill

$\sigma_{v-po} = \gamma_r Z + \gamma_f \left(Z_p - Z\right)$

$Z_p = Z + \left(L_a + \left(\frac{1}{2}\right) L_e\right) \tan\beta$ for sloping backfill

AASHTO LRFD Figure 11.10.6.3.2-1: Vertical confining pressure and Zp depth in resistant zone beneath sloping backfill



*AASHTO LRFD Figure 11.10.6.3.2-2: Default values for the Pullout Friction Factor, F**

Using AASHTO LRFD Figure 11.10.6.3.2-2 for steel grids, we see that the Pullout Friction Factor is a function of both the diameter and spacing of the wires oriented parallel to the wall. For the wire diameter we will use the the 75-year service life diameter of 0.196”. with a spacing of 3”, this gives t/S_t= 0.0653. F* is a maximum of 1.307 at the ground surface and a minimum of 0.653 at a depth of 20 feet (and deeper) measured vertically down from the ground surface.

WWR level	Z_p	F*
1	5.24’	1.136
2	7.24’	1.070
3	9.24’	1.005
4	11.15’	0.9423
5	12.88’	0.8860
6	14.60’	0.8296
7	16.32’	0.7733
8	18.04’	0.7171

We can now check to see of our effective pullout lengths are satisfactory.

From earlier, we need:

$$L_e \geq \frac{\gamma_{p-EV} \times T_{max}}{\varphi \times F^* \times \alpha \times \sigma_{v-po} \times C \times R_c}$$

$\gamma_{p-EV} = 1.35 = load\ factor\ for\ vertical\ earth\ pressure\ per\ Table\ 3.4.1(2)$

$\varphi = 0.90 = resistance\ factor\ for\ reinforcement\ pullout\ from\ AASHTO\ LRFD\ Table\ 11.5.7(1)$

$\alpha = 1.0 = scale\ effect\ correction\ factor$

$C = 2.0 = reinforcement\ surface\ area\ geometry\ factor$

$R_c = 1.0 = reinforcement\ coverage\ ratio\ (WWR\ mats\ are$ continuous $in\ direction\ parallel\ to\ wall\ facing)$

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WWR level	L_e	T_{max}	F*	σ_{v-po}	Equation value
1	5.62’	0.318 kip/ft	1.136	0.635 ksf	0.34’
2	5.62’	0.470 kip/ft	1.070	0.885 ksf	0.38’
3	5.62’	0.654 kip/ft	1.005	1.135 ksf	0.44’
4	6.00’	0.801 kip/ft	0.9423	1.374 ksf	0.47’
5	7.19’	1.008 kip/ft	0.8860	1.592 ksf	0.54’
6	8.39’	1.213 kip/ft	0.8296	1.808 ksf	0.61’
7	9.59’	1.433 kip/ft	0.7733	2.025 ksf	0.69’
8	10.79’	1.393 kip/ft	0.7171	2.241 ksf	0.66’

At all WWR levels, the available length of reinforcement in the resisting zone L_e, back-calculated from the previously defined constant trial length of 11.2 feet, far exceeds that which would be mathematically required to develop T_{max}.

If we were to isolate a single wire at a particular WWR level, the above relationship is further illustrated. For example, a single wire at WWR Level 7 that is part of a continuous run of WWR with wires spaced at 3” oc:

- Is subjected to a tensile force T_{max} = 1.433 kip/ft x 0.25 ft = 0.358 kips
- Receives 2.025 kips/ft^2 x 0.25 ft = 0.506 kips per foot length of resistance due to the soil stack above.

$$L_{e,min}\ per\ wire\ @\ WWR\ Level\ 7 = \frac{\gamma_{p-EV} \times T_{max}}{\varphi \times F^* \times \alpha \times \sigma_{v-po} \times C} = \frac{1.35 \times 0.358\ kip}{0.9 \times 0.7733 \times 1.0 \times 0.506\ kip/ft \times 2.0} = 0.69\ ft$$

$$L_{e,available} = 9.59' > L_{e,min} = 0.69', \therefore\ pullout\ length\ is\ adequate$$

Pullout lengths at zone of maximum stress are satisfactory.

Calculation status:

1. **Steel design life considerations due to embedment in corrosive backfill material**
2. Connection rupture strength and **pullout length at zone of maximum stress**
3. Connection rupture strength at wall facing

Connection rupture strength at the zone of maximum stress is calculated per Equation 11.10.6.4.1-1, with T_{al} from Equation 11.10.6.4.3a-1:

$$\gamma_{p-EV} \times T_{max} \leq \varphi \times T_{al} \times R_c$$

This equation modified to check an individual representative wire is illustrated below:

$$\gamma_{p-EV} \times T_{max,per\ wire} \leq \varphi \times \left(A_c \times f_y\right)$$

where:

$\gamma_{p-EV} = 1.35 = load\ factor\ for\ vertical\ earth\ pressure\ per\ Table\ 3.4.1(2)$

$\varphi = 0.65 = resistance\ factor\ for\ reinforcement\ tension\ from\ AASHTO\ LRFD\ Table\ 11.5.7(1)$

$A_c = area\ of\ reinforcement\ corrected\ for\ corrosion\ loss, in^2$

$f_y = minimum\ yield\ strength\ of\ steel = 70\ ksi\ for\ this\ example$

We are using the same WWR throughout the MSE wall's soil mass, so it is accceptable to check the worst-case scenario at Level 7 where the magnitude of Tmax is the highest:

$$Demand\ per\ wire = 1.35 \times 1.433 \frac{kip}{ft} \times 0.25ft = 0.484\ kips$$

$$Capacity\ per\ wire = 0.65 \times 0.030\ in^2 \times 70\ ksi = 1.365\ kips$$

$$Demand\ to\ Capacity\ Ratio, DCR = \frac{0.484}{1.365} = 0.355\ < 1.0\ \therefore selected\ WWR\ is\ adequate$$

Connection rupture strength at zone of maximum stress is satisfactory.

Calculation status:

1. **Steel design life considerations due to embedment in corrosive backfill material**
2. **Connection rupture strength and pullout length at zone of maximum stress**
3. Connection rupture strength at wall facing

The connection rupture strength at the wall facing is calculated per Equation 11.10.6.4.1-2:

$$\gamma_{p-EV} \times T_o \leq \varphi \times T_{ac} \times R_c$$

Simplified to a "per wire" basis:

$$\gamma_{p-EV} \times T_{max,per\ wire} \leq \varphi \times T_{ac,per\ wire}$$

where:

$\gamma_{p-EV} = 1.35 = load\ factor\ for\ vertical\ earth\ pressure\ per\ Table\ 3.4.1(2)$

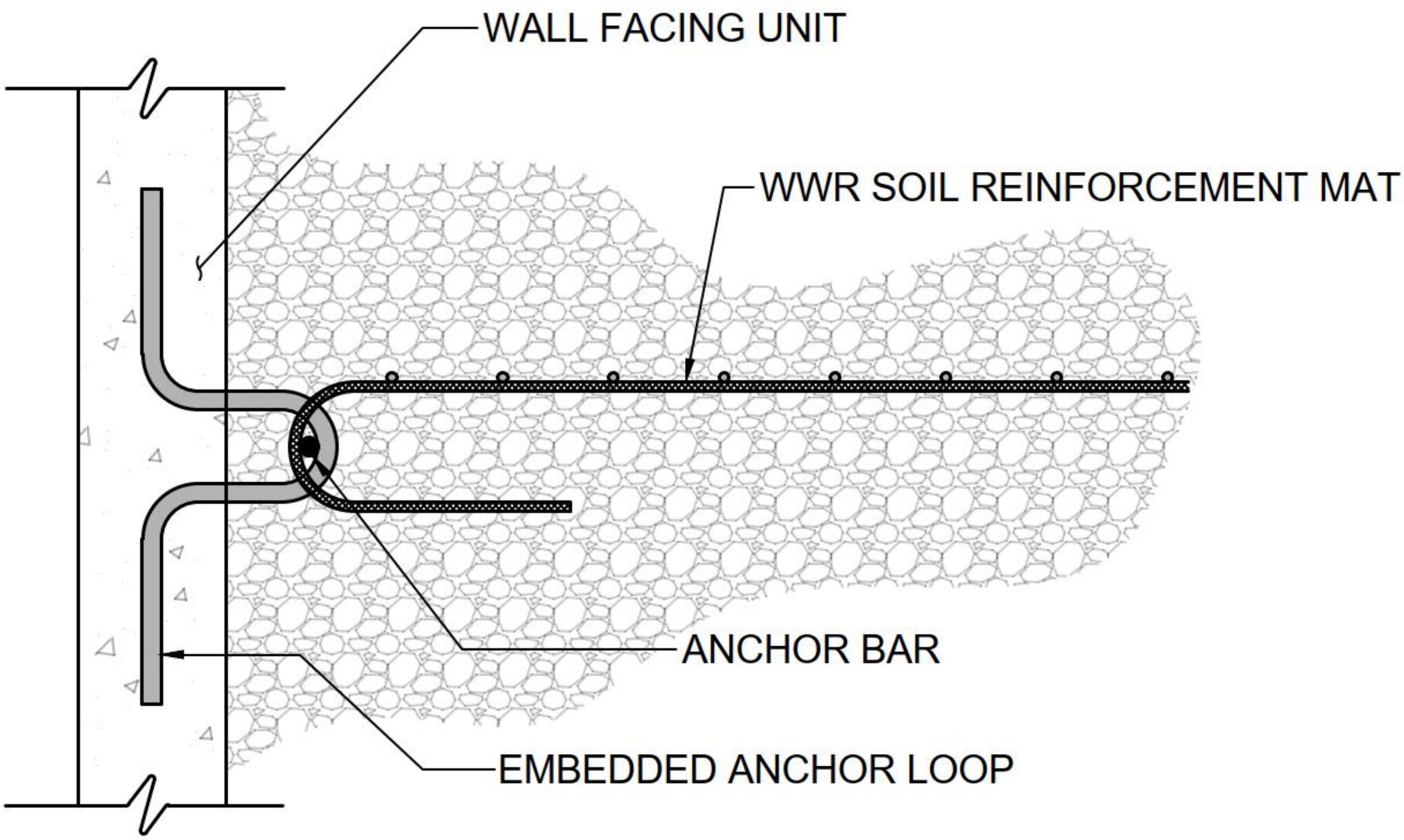
$T_o = T_{max} = applied\ load\ at\ reinforcement\ facing\ interface\ connection\ specified\ in\ Article\ 11.10.6.2.2, kip$

$\varphi = 0.65 = resistance\ factor\ for\ reinforcement\ tension\ in\ connectors\ from\ AASHTO\ LRFD\ Table\ 11.5.7(1)$

$T_{ac} = per\ Article\ 11.10.6.4.4a, nominal\ long\ term\ reinforcement/facing\ connection\ strength, kips$

The connection demand is comprised of a factored applied force (135% of T_{max}) and is compared to the connection capacity comprised of a reduced connection strength (65% of T_{ac}).

Tac is function of the interface connection composition and geometry itself, and is typically based on proprietary arrangements that vary from one MSE wall manufacturer to another. An example wall facing connection is shown below.



Example Figure 5 – Illustration of example reinforcement to wall facing connection

In the above simplified example connection, the WWR soil reinforcement is hooked around a field-place anchor bar. The anchor bar is sleeved through an embedded anchor loop. As the soil mass pushes the wall facing unit to the left, and the WWR soil reinforcement remains stationary as a result of having a sufficient pullout length beyond the point of maximum stress (calculated previously, not shown in this illustration), the hooked end of the WWR mat bears against the anchor bar, and the anchor bar bears against the embedded anchor loop, resulting in static equilibrium. Limit states associated with this arrangement that may need to be checked include but are not limited to:

- WWR wire bearing strength
- WWR wire shear strength at anchor bar bearing
- WWR weld shear
- Flexure in the WWR wire arising from eccentricity between bearing point at hook and the wire longitudinal axis above
- WWR deformation
- Anchor bar shear
- Anchor bar flexure
- Anchor bar deflection
- Anchor loop bearing and embedment within facing unit
- Anchor loop tensile rupture
- Anchor loop shear

So while the form of the connection rupture strength equation itself is quite simple, characterized by a comparison of a factored demand to a reduced capacity/strength, there are numerous variations and aspects of the connection that must be checked to ensure that all demand-to-capacity ratios and differential movement criteria are satisfactory. A detailed check of these limit states is beyond the scope of this WRI technical article given that the variation from one MSE system to the next is so significant.

For this example, then, the trial WWR style of 3x3 W5.0/W5.0 is satisfactory presuming connection rupture stength at the wall facing is confirmed to be adequate in light of proprietary system geometry and composition.

Calculation status:

1. **Steel design life considerations due to embedment in corrosive backfill material**
2. **Connection rupture strength and pullout length at zone of maximum stress**
3. **Connection rupture strength at wall facing (manufacturer-specific)**

The information presented herein is intended to serve as a technical introduction to WWR usage as an inextensible soil reinforcement in MSE walls, guided by the requirements of the AASHTO LRFD Bridge Design Specifications. Project-specific designs are the responsibility of qualified registered design professionals.

For more information visit www.wirereinforcementinstitute.org.

References:

1. *"LRFD Bridge Design Specifications, 9th Edition", American Association of State Highway Transportation Officials, Washington, DC, 2020.*

2. *"Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete (ASTM A1064/A1064M-22)", ASTM International, West Conshohocken, PA, 2022.*

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