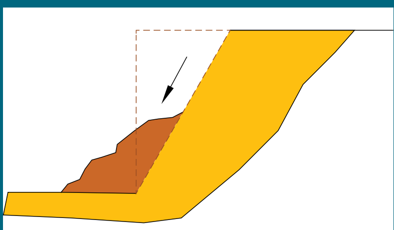
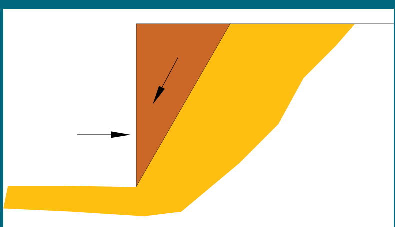
SRW HISTORY
ARTICLE SERIES

SRW DESIGN

This is the third article in a series of ten articles on the history of segmental retaining walls developed under a grant from the NCMA Education and Research Foundation.



a. Soils Fail When Cut Vertically



b. Driving and Resisiting Forces on a Retaining Wall

Figure 1. Retaining Wall Function

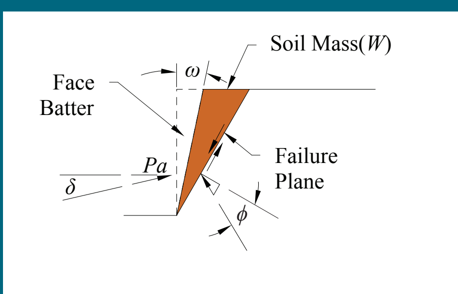


Figure 2. Earth Pressure for a Battered Wall

INTRODUCTION

In previous articles, we featured over 100 years of proven service from zero-slump concrete products, reviewed the history of Segmental Retaining Walls (SRWs) introduced into the market in the 1980s, and the work to address the durability concerns in freeze-thaw/de-icing salt environments in northern climates. SRW research has shown, and good field performance has since proven, that zero-slump concrete products can be made as durable as wet-cast concrete when tested in ASTM C666 (the freeze-thaw test methods used by the DOTs for wet-cast concrete) and in ASTM C1262, the test method NCMA developed to verify freeze-thaw durability for zero-slump concrete, *a test that could be much more severe than ASTM C666.*

With concerns for durability and material performance resolved, the next step is to look at the design of SRWs. Today, about 50% of SRW walls are sold through retail outlets for smaller, gravity walls—walls that don't rely on internal soil reinforcement—applications . The increasing variety of SRW applications prompted an evaluation of the many possible design options available today. This article will discuss the characteristics and design of some of the most common segmental retaining walls used by the construction industry.

When designing a segmental retaining wall, designers can follow established National Concrete Masonry Association (NCMA) or American Association of State Highway and Transportation Officials' (AASHTO) methodologies. Instrumented walls for both design methods have been researched and laboratory- and field-tested for stiffness, stresses (including seismic), sustained load (creep) and predicted vs. measured loading. Each component of a design is important to wall stability.

EARTH PRESSURE

Soil with a vertical face will cause the front edge to slide down into a pile at the base. Soil has a natural angle of repose and anything beyond that angle will eventually roll down (see Fig. 1a).

Retaining walls are designed to support the soil mass and loads placed behind the wall. Designing retaining walls involves balancing the resisting forces with the driving forces to create a stable mass with a margin of safety against failure (see Fig. 1b). Two design theories developed in the 18th and 19th centuries are used for wall design: Coulomb Earth Pressure and Rankine Earth Pressure theory.

Coulomb Theory

In 1776, Coulomb calculated the state of stress in soil against a rough surface, assuming the wall is free to move (for active earth pressure considerations) and that the water contained in the voids does not exert any seepage pressure. The equation for the earth pressure coefficient used for active earth pressure design is:

$$K_a = \frac{\cos^2(\phi+\omega)}{\cos^2\omega\cos(\omega-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\omega-\delta)\cos(\omega+\beta)}}\right]^2} \qquad \text{Eq. 1}$$

Where:
ϕ – angle of internal friction of the soil
δ – friction angle between the soil and the wall facing ($\delta = {}^2/_3\ \phi$)
ω – wall batter
β – slope angle above the wall.

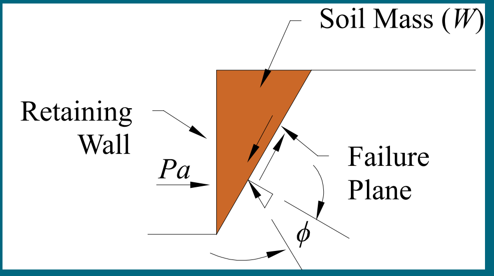


Figure 3. Earth Pressure for a Vertical Wall

This is the equation shown in the NCMA Design Manual (Ref. 10 - Equation 5-3) and used for most wall design applications. The advantages of the Coulomb equation for SRW design is that the equation accounts for wall batter (ω) used in construction, and wall friction that develops between the soil and SRW units. The original NCMA design methodology (1993 and 1997) conservatively assumed this friction was used in the calculation of earth pressure, but was not used as a resisting force in the design of SRW.

Rankine Theory

The Rankine Theory was developed in 1886 to describe the state of stress within a soil mass. The method assumes a vertical soil interface and assumes no friction between the soil and the retaining wall structure. Without considering wall friction or wall batter the method is conservative. The equation for a simple Rankine solution is:

$$K_a = \tan^2\left(45 - \frac{\phi}{2}\right) \qquad \text{Eq. 2}$$

Figure 4. Gravity Retaining Wall (Courtesy of Keystone Retaining Walls)

AASHTO uses a simplified design method assuming a Rankine type of earth pressure for internal stability of Mechanically Stabilized Earth (MSE) designs; however, it still evaluates external stability using Coulomb earth pressure theory.

Regardless of the methodology, if the assumptions used in both equations are the same, the resulting answers will be the same.

DESIGNING SEGMENTAL RETAINING WALLS

There are two types of segmental retaining walls. Geosynthetically reinforced soil systems, or mechanically stabilized earth walls (MSE wall), and gravity (conventional) walls. The conventional SRW is a gravity wall: a structure where there is sufficient mass and resisting moments to support the earth loads and surcharge loads applied by the retained soil mass. These are typically dry stacked interlocking units in a column or in multi-depth configurations.

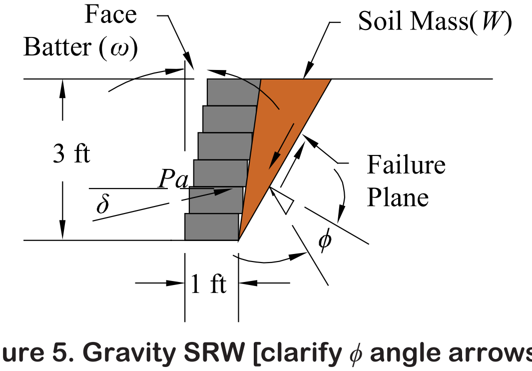


Figure 5. Gravity SRW [clarify ϕ angle arrows]

GRAVITY RETAINING WALLS

Gravity retaining walls are structures where there is sufficient mass and resisting moments to support the earth loads and surcharge loads applied by the retained soil mass. These structures are typically the SRW units dry-stacked in a column, or in multi-depth configurations, supporting the earth loads.

Single Depth

Single depth gravity walls are popular in residential settings for planter walls, small rises in grade with embedded stairs or as architectural features on commercial sites, adding color and variety. A crushed stone leveling pad is used, below the column on SRW units. Typical design heights for gravity walls are 2.5 to 3 times the depth of the unit, or 3 to 4 ft (0.9 to 1.2 m) for most SRW units

Gravity wall analysis requires verifying the external and the internal stability of the wall. The external stability assumes the SRW units perform as a solid block and analyzes the potential for base sliding, column overturning, and bearing capacity for the soil and surcharges behind the SRW units. The internal stability analyzes the shear capacity of the SRW units to avoid sliding between units.

In designing a 3 ft (0.9 m), single depth gravity wall, begin with a 6 in. (228.6 mm) high solid SRW unit with a unit weight of 120 lb/ft^3 (1922 kg/m^3). Each unit has a setback of 1 in. (25.4 mm) ($\omega = 9.5°$). A well compacted silty sand backfill (assume $\phi = 28°$, $\gamma = 115$ pcf (1842 kg/m^3)) should be placed behind the units with a level top surface ($\beta = 0°$). The active Coulomb earth pressure coefficient, K_a, is calculated as 0.259

According to earth theory, earth pressure is calculated based on the wall height and the assumed soil weight. It forms a triangular distribution (see Fig. 6). The resulting pressure is applied at H/3 of the wall height and is applied at an angle to the rough surface ($\delta - \omega$).

$$P_a = 0.5\,\gamma\,H^2\,K_a \cos(\delta - \omega) \qquad \text{Eq. 3}$$

For a 3 ft (0.9 m) tall retaining wall (H = 3 ft (0.9m)) the earth pressure (P_a) is only 132 lb/ft (1.92 kN/m) of wall. The weight of the retaining wall, calculated as W_W = 3 ft x 120 pcf x 1 ft deep = 360 lb/ft (W_W = 0.9 m x 18.8 kN/m^3 x 1 m deep = 5.25 kN/m). Sliding along the base of the wall is calculated based on equation 6-19 from the NCMA Design Manual (Fig 7).

$$R_{Sc} = \mu_b\,W_W \tan\phi \qquad \text{Eq. 4}$$

Assuming a crushed stone leveling pad ($\mu_b = 0.7$), the friction at the concrete on the crushed stone leveling pad R_{sc} = 0.7 x 360 lb/ft x tan (40°) = 211.5 lb/ft (R_{sc} = 0.7 x 5.25 kN/m x tan (40°) = 3.09 kN/m). The factor of safety is calculated at 211.5 lb/ft resisting/132 lb/ft driving (3.09kN/m/1.92kN/m) = 1.60. The accepted factor of safety is 1.5. In this case, the wall will meet the suggested factors of safety.

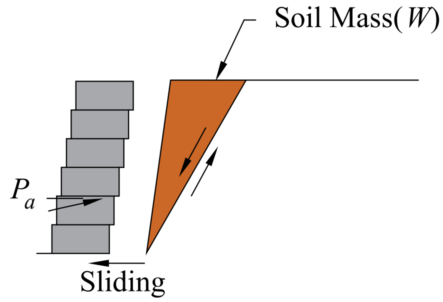


Figure 7. SRW Sliding Failure Mode

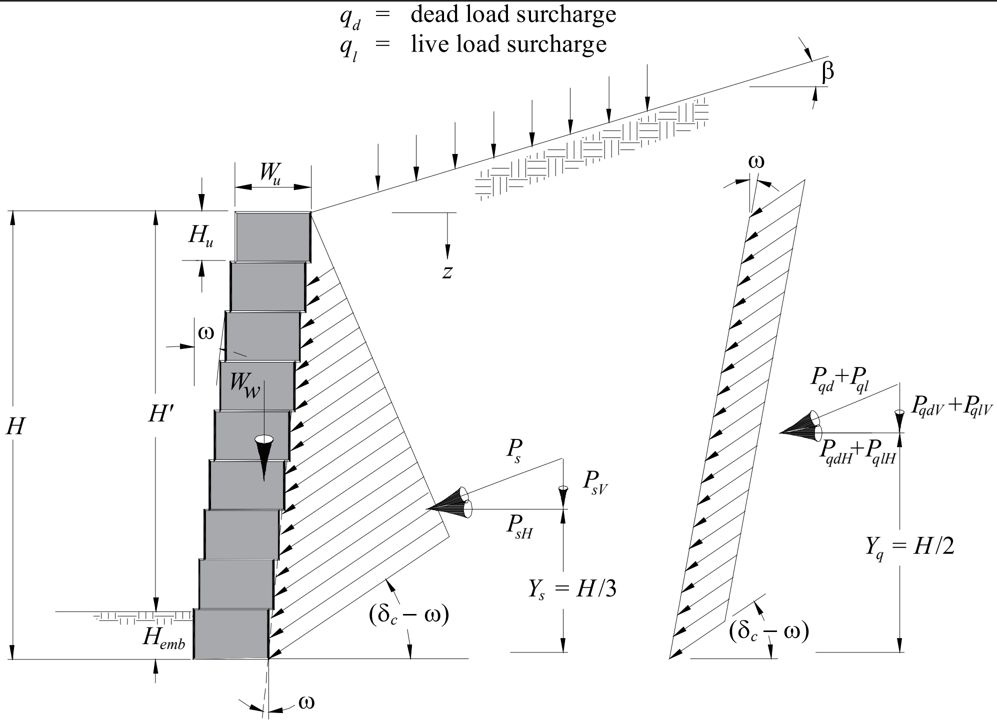


Figure 6. Forces and Geometry for External Stability Analysis of Conventional SRWs (Ref. 10 Figure 6-2)

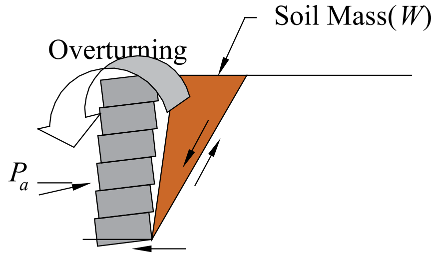


Figure 8. SRW Overturning Failure Mode

The designs also look at overturning (or eccentricity) as a controlling factor (Fig. 8). Assuming the wall rotates about the front toe, the driving moment is M_o = 132 lb/ft x 3ft/3 = 132 lb-ft (M_o = 1.92 kN/m x 0.9 m/3 = 0.58 kN-m) and the resisting moment is M_R = W_W* *lever arm (horiz. distance between the toe and the centroid of the facing)* = 255 lb-ft (1.13 kN-m), resulting in a factor of safety of 1.93. The suggested factor of safety for overturning is 1.5, so this also is acceptable.

This example assumes no surcharge loads above the wall (traffic or dead loads) and assumes no water pressures behind the wall face. This example demonstrates the rule of thumb of 2.5 to 3 times the block depth for single depth walls is a valid concept.

Note: A factor of safety is the resisting forces divided by the driving forces where a factor of 1.0 indicates the structure is at a failure condition. A factor of 1.5 means we have 50% more resisting forces than driving forces, a safer design. Acceptable factors of safety are set based on the uncertainty of the loads and the results of a failure; the more risk the higher the margin of safety, the more severe damage that can be done as a result of failure, the higher the margin of safety is set. In the AASHTO LRFD design method, the factors are based on a Load Reduction Factor Design (LRFD). The driving forces are factored up based on their uncertainty (Live load factor LL = 1.75; Dead load factor = 1.35) and the resisting forces are factored down ($RF_{sliding}$ = 0.9). The result of the equations yields a Capacity Demand Ratio (CDR). A CDR of 1.0 or greater is the target of the design. Ideally a design done by Allowable Stress Design (ASD) and a design done by LRFD (Load and Resistance Factor Design) should yield similar results.

Bearing and Global Stability

Design for gravity retaining walls and MSE walls require a check of the capacity of the foundation soils to support the load of the applied structure. Equation 12-1 in the NCMA Design Manual is the classic equation for bearing capacity:

$$Q_{ult} = c_f N_c + 0.5\,\gamma_f B'_f N_\gamma + \gamma_f H_{emb} N_q \qquad \text{Eq. 5}$$

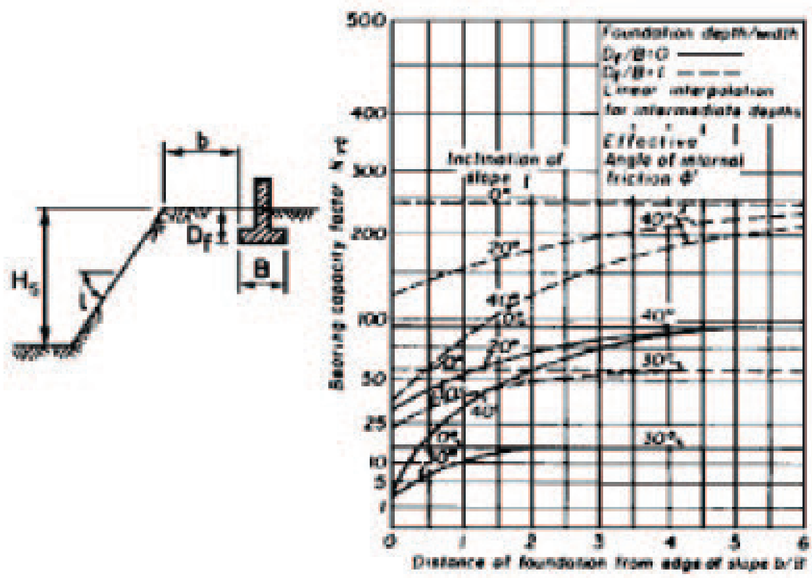


Figure 9. Modified Bearing Capacity Factors for a Footing in Cohesionless Soils and on or adjacent to Sloping Ground after Meyerhof (1957). (Ref. 15)

Bearing capacity is based on the width of the base, the depth of embedment (depth below finished grade on the front of the wall) and the soil parameters below the wall. In most cases bearing for single depth retaining walls is acceptable due to the low wall heights and low bearing pressures. However, in cases where the soil in front of the wall slopes away from the wall, the amount of embedment (resisting forces of the soils in front of the wall) reduces quickly. The NCMA Design Manual does not cover this topic; however, conventional bearing capacity equations can be modified to account for a sloping toe condition, as developed by Meyerhof (1957) and illustrated in Fig. 9. As can be seen from the chart, walls founded on slope can lose 30% to 50% of the bearing capacity compared with the same wall founded on a level base.

The last design check for wall design is a check of overall stability or global stability. For walls on a level base this may not be a concern but for walls founded on slopes, tiered walls, weak foundations, and high surcharges, the design should be checked. A factor of safety of 1.3 to 1.5 is generally acceptable. As can be seen in Fig. 10, the 3 ft (0.9 m) retaining wall designed above would not meet industry standards of design, nor would the design have met the requirements for bearing capacity or global stability on a 18° front slope.

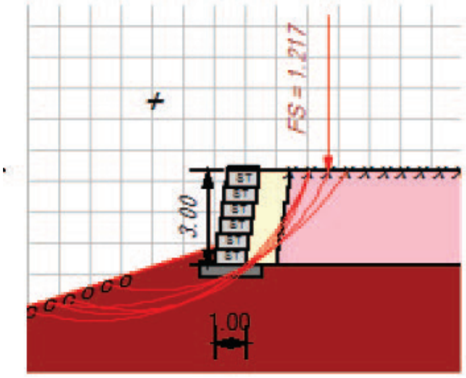


Figure 10. Gravity SRW Global Stability Analysis

NOTE: For walls under 4 ft (1.2 m), a design professional may not be involved but as it was shown in Fig. 10 ignoring the analysis of bearing and global stability could be a problem even for small walls.

Multi-Depth

Multi-depth walls are made of SRW units with anchors and trunks attached to the facing units to give more depth to the wall structure (see Fig. 12). They are used in applications where a taller gravity wall is required and areas with space restrictions, such as an application where pipes or utilities need to be installed close to the wall or where long-term maintenance may be required behind the wall. SRW MSE walls, with wide base requirements, aren't viable options in such situations. The design height of this type of wall is basically unlimited, the taller the wall the more anchors that are attached. The base-to-height ratio is about 40% to 50%, where an MSE wall is 60% [NCMA] or 70% [AASHTO]. While multi-depth wall heights may be unlimited in design, MSE walls with geogrid become more cost competitive in heights over 15 ft (4.5 m).

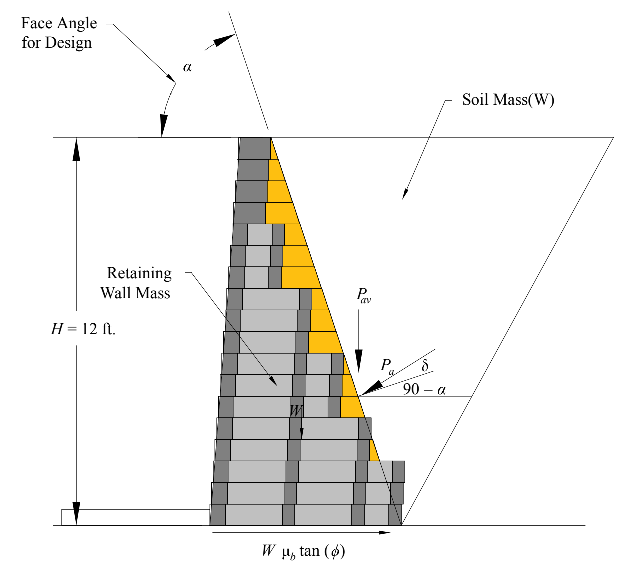


Figure 11. Multi-Depth Gravity SRWs

While not covered in the NCMA design manual or design software, designing multi-depth walls follows NCMA's gravity wall methodology with a few modifications and is detailed in the AASHTO LRFD Bridge Design Specification, Article 3, Lateral Earth Pressures for Prefabricated Modular Walls and 11.11 Prefabricated Modular Walls.

Earth pressures for multi-depth walls are calculated using the Coulomb earth pressure equation shown before. However, the face batter (ω) used for design is measured as the angle from the tail of the base unit to the back of the cap unit (Fig. 11). The design assumes the soil fill under the assumed failure plane is also used as a resisting force. In multi-depth walls, the angle of wall friction between the retained fill and the wall structure is $\delta = {}^{3}/{}_{4}\ \phi_f$ (instead of the $\delta = {}^{2}/{}_{3}\ \phi_f$ for soils against a vertical concrete face) because the wall fill is resting over the resisting mass, not sliding along a concrete/soil interface.

Figure 12. Multi-Depth SRWs (Courtesy of WestBlock Systems, Inc.)

Sliding, overturning (eccentricity), and bearing stability analysis follow the established methods. The analyses include the SRW units' weight, the gravel fill contained within the units, and the fill below the assumed failure line except that for overturning a maximum of 80% of the gravel fill can be considered and for bearing capacity a minimum of 80% of that gravel fill weight has to be considered.

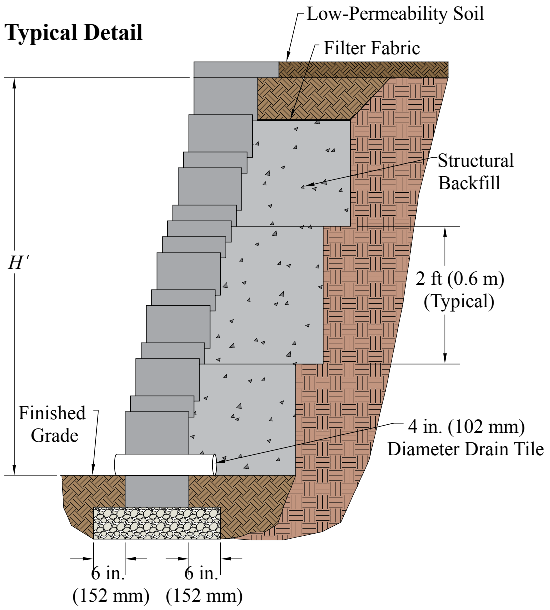


Figure 13. Permeable Concrete Fill Placement (Courtesy of Anchor Wall Systems)

Low Strength Permeable Concrete Fill

Low Strength Permeable concrete is a no-fines concrete that cures with void spaces sufficient to allow water to drain through the concrete mass. This type of wall system uses the modular SRW facing units and then the area behind them is filled with a permeable concrete mix (see Fig. 13) mostly in residential or light commercial applications where there is no room for geogrid reinforcement behind the wall. The concrete adds the structural mass sufficient for a larger concrete retaining wall, while the SRW facing adds the aesthetics desired.

Applying the same design methodology used for the single depth wall, a design section for a 12 ft (3.7 m) retaining wall would be as shown in Fig. 13. The calculated factors of safety were:

- $FS_{sl} > 1.7$
- $FS_{ot} > 1.8$
- $FS_{bc} > 2$

Figure 14. SRW with Permeable Concrete Backfill (Courtesy of Anchor Wall Systems)

This is another design option for designers to consider when designing SRW walls in locations where space is limited.

MECHANICALLY STABILIZED EARTH WALLS

For taller wall applications, engineers often default to the mechanically stabilized earth (MSE) wall section using geogrid reinforcement with an SRW facing unit. SRW MSE walls have been built all over the world and have been constructed to heights over 70 ft (21 m) tall.

SRW MSE walls have the advantage of the SRW units acting as a permanent form allowing compaction and maintaining the desired wall batter that cannot be attained with systems that only use geogrid or geofabrics requiring temporary forms. Research (Holtz, Ref.12) has found that the loads at the face are relatively small and, in some cases, the face has been removed without compromising the wall.

Figure 15. Mechanically Stabilized Earth Walls (Courtesy of Versa-Lok Retaining Walls Systems)

MSE design is based on Bell's Tie-Back Wedge (1975, Ref. 4) method of design first developed for the U.S. Forest Service. In this method, the internal earth pressure is modeled as an active earth pressure force and the geogrid reinforcement holds that active wedge of soil in place (See Fig. 16).

SRW MSE wall analysis requires verifying the external stability to determine the reinforcement length (similar to the gravity wall analysis), internal stability to determine the grid strength, facial stability to determine the reinforcement spacing, and the new Internal Compound

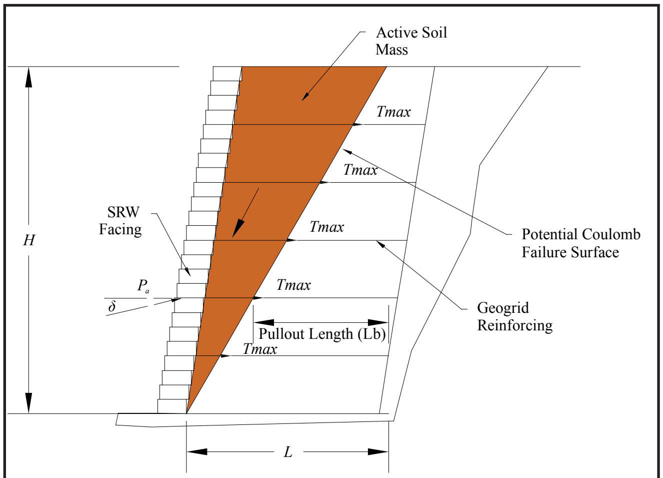


Figure 16. Completed SRW Wall

Stability Analysis introduced by NCMA Design Manual 3rd edition (Ref. 10), which analyzes the coherence of the block-geosynthetic-soil system.

For external stability, the SRW MSE mass of soil is assumed to be a rigid block. The resisting forces are based on the size and mass of the reinforced soil block, and the driving forces are the active earth pressures (Coulomb Earth Pressure Theory) and surcharges calculated at the back of that mass. The potential sliding, overturning and bearing capacity failures on the wall are analyzed.

For internal stability, the SRW MSE analysis assumes that the stresses (soil and surcharges) behind the SRW units are distributed on the geosynthetic layers. The geosynthetic strength and distribution are selected to ensure structural integrity of the reinforced zone is preserved with respect to reinforcement over-stressing, pullout of geosynthetic reinforcement layers from the reinforced soil mass, and internal sliding along the reinforcement layers.

For facial stability, the reinforcement spacing is determined to avoid rupture of the connection between the geosynthetic reinforcement and the SRW units for the loads in the reinforcement, and the maximum unreinforced height of SRW units at the top of the wall to avoid overturning.

NCMA Design Manual

The NCMA Design Manual is based on the work by Bell (1975), design methodologies developed by FHWA 1990 (TF-27, Ref. 11), design manuals developed by the geogrid manufacturers in 1990 (Mirafi Ref. 16, Tensar Ref. 18), and field experience. The AASHTO bridge specifications used a Rankine and gravity earth design approach based on concrete panels with steel reinforcement designs, where the geogrid manufacturers used the Coulomb theory to account for the wall batter and wall friction. In the 1980s when geogrid reinforcement was introduced, AASHTO was predominantly a steel reinforcement market following the Reinforced Earth Company design model. Geogrid reinforcement was economical, worked well with the SRW units and offered the market a good solution to earth structures in residential and commercial settings where panels were not feasible.

NCMA initially specified a minimum geogrid length (L) of 50% of the design height of the wall (0.5H) (Ref. 9) where the AASHTO (Section 11.10.2.1, Ref. 1) design methods have always specified a minimum length of 70% of the design wall height. In the second NCMA design manual, the minimum reinforcement length was increased to 60% of the design height. Although design calculations can be shown to work with geogrid lengths as short as 40% of the wall height, the walls and soil mass are flexible and performance was potentially compromised with shorter geogrid lengths. The move to 60% of the wall height provided an empirical answer to what can be evaluated when performing Internal Compound Stability analysis (discussed later).

1980s Wall Design

Initial wall designs were based on spacing the geogrid reinforcement at two times the block depth (e.g. 2 ft (0.6 m) for a 1 ft (0.3 m) deep unit, 4 ft (1.2 m) for a 2 ft (0.6 m) deep unit). This was based on overturning calculations on the SRW units assuming soil and compaction stresses along with experience that noted the smaller units tended to rotate more during construction (based on the compactions stresses). Geogrid connection strength was based on a factor of safety of 1.5 on peak connection and 1.0 for serviceability concerns. Industry worked with FHWA and AASHTO and established industries recommendation: maximum geogrid spacing two times the unit depth or 2.7 ft (32 in. (813 mm)) maximum (Section 11.10.2.3.1, Ref. 1).

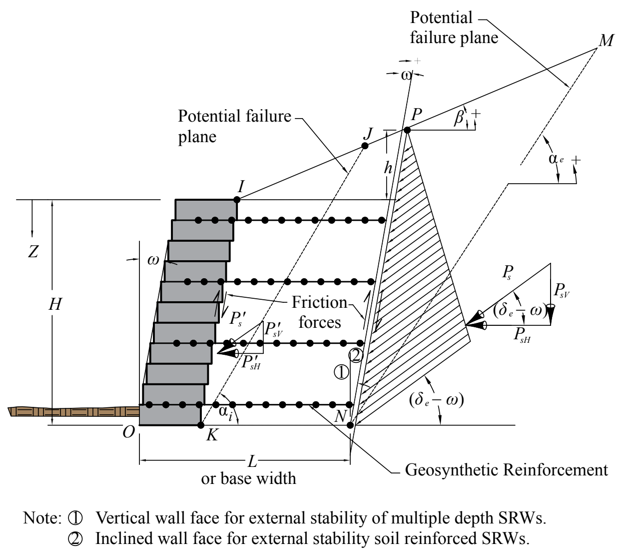


Figure 17. Earth Pressure Distribution and Force Resolution SRW System (Ref. 10, Fig. 5-5)

Most of the early SRW units were hollow and the voids needed to be filled with a material that was easy to place, would drain, was easy to compact, and would not spill out through gaps between the units. Pea gravel ($^3/_8$ in. (9.5 mm) rounded aggregate) was tried, but with a $^1/_2$ in. (13 mm) gap between units, the whole column of stone could drain out. Therefore a $^3/_4$ in. (19 mm) stone was tried, designing the aggregate size based on a filter criteria for a slot drain ($^1/_2$ in. (13 mm) gap) that worked well. This was the basis for the current $^3/_4$ in. (19 mm) gravel fill specification. With the one foot (0.3 m) deep units, compaction equipment needed to be kept away from the units to reduce tilting or rolling forward of the units during construction. A one foot (0.3 m) zone of rock worked well for this, therefore the next specification item was added: 12 in. (305 mm) of gravel fill behind the unit tails. The larger units (18 in.+ (457 mm +) deep units), did not need the additional gravel fill since they were more stable and there was already 24 in. (610 mm) of drainage at the face by just filling the units.

A 1990 SRW design specification would have read:

- *Facing units: SRW units with a minimum compressive strength of 3,000 psi (20.7 MPa).*
- *Geogrid reinforcement: HDPE or high tenacity PET soil reinforcement geogrid.*
- *Max. vertical spacing of geogrid reinforcement: two times unit depth or 32 in. (813 mm) maximum.*
- *Min. geogrid length: 60% H (8 ft (2.4 m) min. and 70% H for AASHTO), or as required for external stability.*
- *Gravel fill: 24 in. (610 mm) of gravel fill from the wall face.*
- *Connection capacity shall be based on the peak connection strength with a factor of safety of 1.5.*
- *The connection design load shall be the maximum calculated stress in the geogrid layer (T_{max}).*

Many walls, several in the 40 ft (12 m) range, were constructed to this specification and are still performing well today in both the highway market (State Department of Transportation) and in the commercial market.

Internal Compound Stability

In the most recent Design Manual (Ref. 10), NCMA introduced the Internal Compound Stability (ICS) that evaluates the coherence of the block-geosynthetic-soil system. The analysis checks potential compound slip circles that originate behind a soil-reinforced SRW and exit at the face of the wall (slip arcs) (see Fig. 17). It considers the three parts in the system: the unreinforced soil forces analyzed through slope stability methods; the reinforcement with resisting forces; and the facing contributing with resisting shear or connection forces. ICS is a special case of global stability analysis and does not replace it.

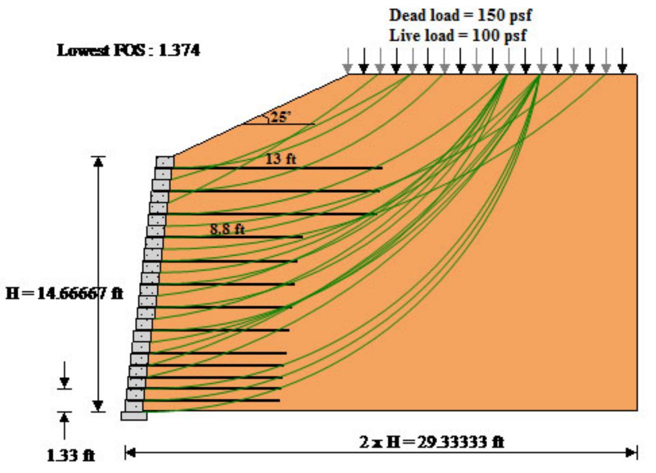


Figure 18. Internal Compound Stability Analysis Example

Compound failure surfaces will not generally be critical for simple structures with rectangular geometry, relatively uniform reinforcement spacing, and a near vertical face. If, however, complex conditions exist (high surcharge loads, significant slopes at the toe or above the wall, or tiered structures) compound failures may be a design limit state (see Figure 18). The Third Edition of the DMSRW recommends that the responsibility for ensuring adequate compound stability rests with the retaining wall designer and a minimum saftey factor of 1.3 (FS $\geq$ 1.3).

NCMA versus AASHTO

Comparing the requirements by NCMA with AASHTO, they both are very similar, with the AASHTO requirements more conservative than NCMA because of two primary reasons. AASHTO utilizes Rankine Earth Pressure theory for internal stability, and AASHTO considers a Load-Resistance Factor Design Method that has proven to generate more conservative designs compared to the allowable stress methods of NCMA and previous AASHTO specifications. The key differences are:

Minimum Geogrid Length

NCMA 4 ft (1.2 m) or 0.6 H whichever is greater
AASHTO 8 ft (2.4 m) or 0.7 H whichever is greater from the back of the block

AASHTO (Section C11.10.2.1, Ref. 1) based the minimum length (8ft (2.4 m)) on experience and the ability to get construction equipment in behind the panels to compact the soil. The 0.7H ratio is also based on experience with steel reinforcement, while research has shown with short walls (i.e. 10ft (3.0 m)) 0.8H works and for taller walls (40 ft (12 m)), 0.63H is adequate. NCMA design recommendations are based on geogrid reinforcement with 100% coverage (better pullout capacity), versus steel reinforcement with less than 25% coverage, thus the 0.6H has worked well for all geogrid wall heights. AASHTO ignores the length of reinforcement embedded in the blocks while NCMA considers it in the total length of the reinforcement.

T_{max}

While NCMA always uses the Coulomb approach, factoring the slope into the equation, along with wall friction and the effective wall batter, thus reducing the load (T_{max}) in the reinforcement layers compared with an AASHTO design that only uses Coulomb Earth Pressures for walls above 10°, face batter and Rankine Earth Pressures below 10°. AASHTO uses a "simplified" method of design. This assumes a level top surface and treats the slope as an equivalent surcharge.

Reinforced Fill Soil Types

NCMA allows for the use of materials having a maximum 35% passing the #200 (75 μm) sieve, and suggests that materials with greater percentage of fines may be used when a geotechnical engineer is involved in the design. AASHTO restricts the use of materials for the reinforced fill zone and requires granular materials with less than 15% passing the #200 (75 μm) sieve.

Connection

NCMA uses the maximum geogrid load (T_{max}) with factor of safety of 1.5 and compares this to the Peak Connection Capacity between the geosynthetic and SRW unit. AASHTO also uses a factor of safety of 1.5 on T_{max}; however, AASHTO also requires an additional reduction factor to be applied to the Peak Connection Capacity that is derived from long-term, sustained load connection testing. [*In 2000, at the request of FHWA, industry suggested a long-term sustained load test method that made a clear delineation between units with mechanical connections (normal load independent) and friction connections (normal load dependent). Only systems that were comprised of mechanical connections required consideration of the additional reduction factor. That was adopted by AASHTO 2002 (ASD design, for frictional vs. mechanical connections). In the AASHTO LRFD specifications the same criteria was adopted; however, a comment added that long-term sustained connection testing should be considered for all connections to be conservative (AASHTO C11.10.6.4.4b, Ref. 1).*] This is a very conservative assumption and not justified by performance or data from instrumented structures that indicates the conservatism inherent in current SRW design methods (Ref. 2 and 11). It is probable that there will be changes in this portion of the design in the future because the design loads are extremely different from what the instrumented walls are showing.

Overturning

NCMA has suggested a factor of safety of overturning (ability of the structure to resist rotating forward) of 1.5 for gravity walls and 2.0 for MSE structures. AASHTO ASD (AASHTO 2002) suggested a factor of safety of 2.0 for all structures. In AASHTO LRFD (AASHTO 2012), eccentricity is specified (the location of the load resultant with respect to the center of the footing, L) as $e/L < 0.33$. The resultant should be within the middle $1/3$ of the base. e/L yields about the same design as a factor of safety of 2 would in NCMA.

Design Example

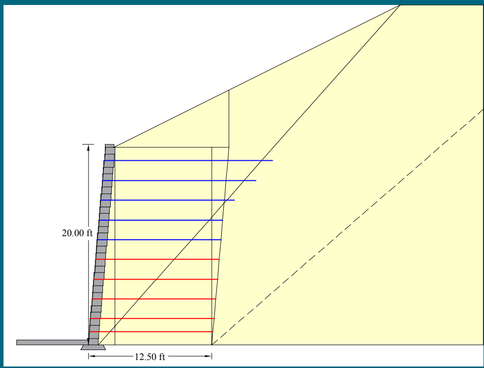
Although this is not a comparison of AASHTO to NCMA, both designs are comparable and the design exercise will demonstrate the design procedures for comparison.

Example:

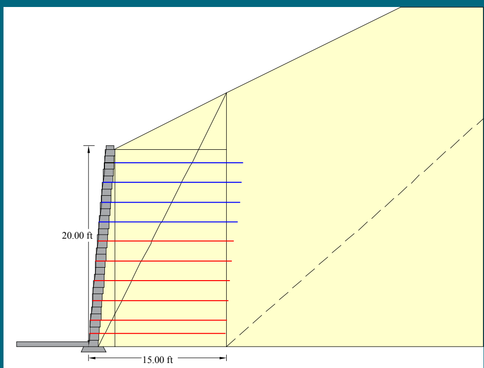
$$H = 20 \text{ ft } (6 \text{ m}), \beta = 26°$$

Embedment:
- NCMA: 6 in. (15 cm) or $H'/20$ where H' is the exposed wall height (NCMA Table 5-1, Ref. 10).
- AASHTO: Frost depth or 1 ft (0.3 m) or $H/20$ (AASHTO 11.10.2.2, Ref. 1).

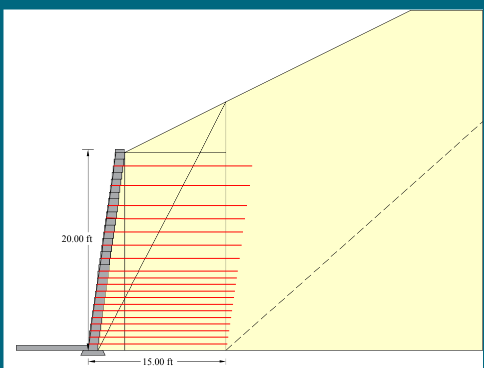
Minimum Reinforcement Length
- NCMA: $L \geq 0.6H$ = 12 ft (3.6 m)
- AASHTO: $L \geq 0.7H$ = 14 ft (4.3 m), $L_b = L$ + the facing depth = 15 ft (4.6 m).



NCMA Design



AASHTO without Long Term Connection



AASHTO with Long Term Connection

Figure 20. Segmental Retaining Wall Design Comparison

Soils:

Reinforced fill:	Foundation:	Retained Fill:
$\phi = 34°$	$\phi = 30°$	$\phi = 30°$
$c = 0$ psf	$c = 0$ psf	$c = 0$ psf
$\gamma = 120$ pcf (18.8 kN/m^3)	$\gamma = 120$ pcf (18.8 kN/m^3)	$\gamma = 120$ pcf (18.8 kN/m^3)

The AASHTO section requires 3 ft (0.9 m) more excavation than the NCMA design since AASHTO measures the reinforcement from the back of the wall face and requires 0.7H vs. 0.6H. Because of this, the external design height (H_n) for AASHTO is taller than the NCMA external design height (Fig. 20).

External Stability

K_a based on the Coulomb Equation

- NCMA: $K_a = 0.471$ ($\phi = 30°$, $\omega = 5°$, $\beta = 26.6°$)
- AASHTO: $K_a = 0.538$ ($\phi = 30°$, $\omega = 0°$, $\beta = 26.6°$, $\delta = 26.6°$)

NOTE: For wall batters less than 10°, AASHTO assumes a vertical interface ($\omega = 0°$) and in the Rankine approach, $\delta = \beta$; for Coulomb NCMA uses $\delta = \phi$.

NCMA: Pa = 18,772 lb/ft (Pa_h= 17,013 lb/ft, Pa_v = 7,933 lb/ft)
AASHTO: Pa = 23,542 lb/ft (Pa_h = 21,050 lb/ft, Pa_v = 10,541 lb/ft)

Resisting Forces = Soil Weight + Pa_v
NCMA: (20ft + 25.75ft)/2*12ft * 120 pcf + Pa_v = 40,873 lb/ft
AASHTO: (20ft + 27.5ft)/2*15ft * 120 pcf + Pa_v = 53,291 lb/ft

NOTE: Pa_v is the vertical component of the load and is used here to make the designs comparable.

Without getting into all the details of the calculations, the results are:

Method	Sliding FS_{sl} (CDR)	Overturning FS_{ot}	Bearing FS_{bc} (CDR)	Connection FS_{cn} (CDR)
NCMA	1.51	2.87	4.24	1.62 (layer 2)
AASHTO	1.03	$^e/_L = 0.10$	3.17	1.62 (layer 2)
AASHTO w/conn. creep	1.03	$^e/_L = 0.10$	3.17	1.4 (layer 8)

FS – Factor of Safety
CDR – Capacity Demand Ratio

The top reinforcement layers have design strength of 1,900 lb/ft (27.7 kN/m); the lower (red) layers have design strength of 3,400 lb/ft (49.6 kN/m).

What do we notice about the designs?

1. The NCMA and AASHTO (no long term connection) designs have about the same number of low and high strength geogrid layers.
2. The AASHTO design considering long term connection has all high strength layers, and 60% more layers of reinforcement.
3. The NCMA design has longer geogrid lengths at the top, where the AASHTO lengths are constant. (NCMA used the Coulomb failure plane and 1 ft (0.3 m) of reinforcement is required beyond the failure plane. AASHTO used the Rankine failure plane ($45 + {}^{\phi}/_2$) and requires 3 ft (0.9 m) beyond the failure plane).

If we assume the same design conditions that we had in 1990, the NCMA design and the AASHTO designs are very close AASHTO had slightly longer reinforcement lengths. If we take the long term connection requirement and apply it to all connections, as suggested in AASTHO LRFD, then the cost of the resulting retaining wall is now significantly more than it was before, both in construction and material costs a change that is not justified by *the millions of 20 ft +(6.1 m+) walls built prior to 2007 (AASHTO LRFD specification) that are still performing well.*

Seismic Design

Both the NCMA and AASHTO methods use the Mononobe-Okabe methods for pseudo static design. Both also assume the total dynamic stress is evenly distributed over all the layers of reinforcement (P_{ae}/N). The Mononobe-Okabe method used to assume an inverted trapezoid of force (maximum dynamic force near the top of the structure) is recommended for the design for flexible anchored sheet pile walls under seismic conditions. The earlier NCMA manuals assumed this distribution thus making the top layers very long and requiring more design strength. Early AASHTO methods (Ref. 1) on the other hand, assumed the dynamic load was distributed based on the amount of embedment length beyond the failure plane, an assumption used for steel reinforcement design. Thus the dynamic loading was more at the base of the structure. Full scale testing has been performed to levels of 0.8g horizontal and 0.4g vertical that were modeled after the Kobe earthquake that indicated that the initial assumptions were incorrect and justified the change in load distribution. Both methods are now using the same approach, making the design more consistent.

NCMA, however, promotes the use of seismic design methods in seismically active zones where AASHTO (AASHTO 11.5.4.2, Ref. 1) does not consider seismic design mandatory in zones 1-3 unless liquefaction induces lateral spreading or slope failure, or seismically induced slope failure, due to the presence of sensitive clays that lose strength during seismic shaking, which may impact the stability of the wall. Also, if the wall supports another structure that is required by code or specification to be designed for seismic loading, poor seismic performance could impact the structure.

The performance of MSE walls has been very good, even in the largest, most damaging earthquakes, and cases where either the wall collapsed or severe wall displacements have occurred are rare. Even in cases where the walls were not designed for seismic events, the performance was very good (AASHTO C11.5.4.2, Ref. 1).

Other Design Items

In comparing MSE design using geosynthetic reinforcement using the NCMA design method versus the AASHTO design method, both are similar design methods and can yield similar results. There are other special applications that have not been discussed that with enough care can be successfully designed and built. Some examples are of other applications are:

- Terraced walls
- Steel reinforced SRW walls
- Composite multi-depth and MSE walls
- SRW structural wall (internal steel reinforced cells)
- Back-to-back SRW gravity walls
- Quarry support walls (Fig. 21)
- SRW veneer walls

SUMMARY

In closing, we have briefly reviewed the design basis of gravity walls and MSE SRW structures. Gravity structures may seem like a minor item in wall design since as engineers and designers we concentrate on the 20 ft to 50 ft (6.1 to 15.2 m) walls, "the big stuff". However, the volume of gravity walls sold is significant. The introduction of multi-depth SRW walls and low strength permeable concrete makes gravity walls a more significant player in the market.

We also reviewed MSE design used in the 1980s with a sample specification from the 1990s time period compared with designs today. Not much has changed: design for Tmax based on a Coulomb Earth Pressure, uses a factor of safety of 1.5 on Peak Connection, and uses a minimum length of 0.6H (AASHTO 0.7H).

We also compared an NCMA design with a public transportation (AASHTO LRFD) design. Without sustained load reduction on the connection, the designs were very similar: similar tensions on the reinforcement, similar geogrid spacing, and the same number of layers of reinforcement. AASHTO LRFD added a comment to include sustained load testing (creep) on the connection,

Figure 21. Quarry Support Wall (Courtesy of Keystone Retaining Walls)

initially required for mechanical connection systems only, to all systems which can double the reinforcement in SRW walls and severely limits the allowable design heights for frictional systems. Research has shown that the loads at the connection (Ref. 2 and 11) are low but the current design methodology is making the connection design the controlling condition. Professor Holtz has shown there is very little stress at the facing; if a face is removed, a little gravel is lost but the structure does not fail.

Design methods for MSE structures are conservative as research and performance has shown; the stresses are less than theory predicts. However, maybe a more granular (permeable) fill could be used to reduce hydrostatic loading and reduce post construction settlements (loads on the connection) and could prove to be an advantage (Koerner 2011, Ref. 14). Industry could follow AASHTO and reduce the requirements for seismic designs for peak ground accelerations of less than 0.4g, where reinforced fill materials have less than 15% fines. AASHTO could review the design model currently used to avoid over predicting loads and overdesigning.

There is a lot more to design than these simple details, but if the design meets these simple rules, the project should be successful.

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