SRW COMMERCIAL VS. TRANSPORTATION MARKETS

SRW HISTORY ARTICLE SERIES

This is the sixth 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.

Segmental Retaining Walls (SRWs) are used in both private and public development projects. Private development projects include commercial, industrial and residential markets. Public development markets include federal, state and municipal projects, from parks and recreational areas to heavy construction. SRWs utilized in the private market are generally designed in accordance with the NCMA design specifications and the design methodology presented in the *NCMA Design Manual for Segmental Retaining Walls* (NCMA 2009, Ref. 18). In the transportation market the design specifications follow the *AASHTO LRFD Bridge Design Specifications* (AASHTO LRFD, Ref. 1) published by the American Association of State Highway Officials for the local state transportation markets. Cities and counties may adopt either the State specifications (AASHTO) or the commercial specifications (NCMA).

Figure 1. Transportation SRW (Courtesy of Tensar)

Figure 2. Commercial SRW (Courtesy of Allan Block)

While the specifications, design methodologies and material requirements have some similarities the two markets their differences. Regardless of design methods chosen, walls properly designed with one of the two methodologies will perform well.

In this paper we will compare the design methodologies, identify the differences in design requirements, and look at the material requirements. Transportation markets are accustomed to wet-cast concrete products and steel reinforced soils and have consequently developed their requirements around these products (e.g. concrete leveling pad, non-corrosive backfills). Another difference is the design and construction specifications, particularly the gradations of the aggregate fill materials.

DESIGN METHOLOGIES

NCMA Methodology

The NCMA design methodology is based around the Coulomb earth pressure design (Coulomb 1776). Earth pressures are based on a homogeneous, infinite soil mass in a state of plastic equilibrium. The soil mass has a frictional component (ϕ), a cohesion (c) and a unit weight (γ). The internal stability design assumes cohesion in the soil is zero, however, it can be used in design if the designer thinks it is applicable. Coulomb's method assumes there is wall friction (δ) between the soil mass and the back of the retaining wall, in this case, the back of the SRW units. Coulomb's method also accounts for wall batter (wall facing inclination from vertical) in design. Both wall friction and wall batter reduce the amount of earth pressure acting on the wall.

In taller walls, where the mass of the SRW units is not sufficient to resist the earth pressures, soil reinforcement (polymer geosynthetics or steel grids/bars in some systems) is used to reinforce the soil mass, forming a mechanically stabilized earth (MSE) structure. The SRW units in an MSE design provide a form to retain the soils and a facing to the soil mass. In the current design methods, there is very little structural capacity attributed to the facing. Connection between the geogrids and the SRW facing

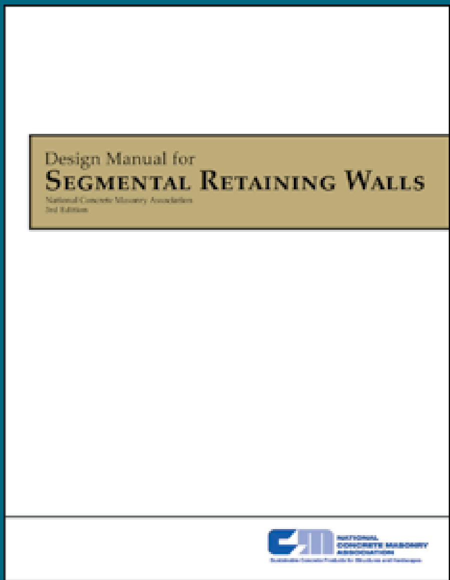

Figure 3. NCMA Design Manual for Segmental Retaining Walls 3rd Ed.

can be frictional (normal load dependent) or mechanical. From laboratory testing of the units and geogrids (ASTM D6638, Ref. 10), a connection curve is developed for use in design. The allowable connection capacity (*Tac*) is the peak connection strength divided by a factor of safety, generally 1.5.

$$Tac = \frac{T_{ultconn}}{FS}$$

Pullout capacity of the reinforcement is also developed from laboratory testing of the geogrid reinforcement and typical soil types (ASTM D6706, Ref. 11). A factor of safety of 1.5 is used in calculating the required length of the reinforcement beyond the Coulomb failure plane. NCMA recommends a minimum of one foot (approx. 300 mm) of the soil reinforcement be embedded beyond the theoretical failure plane.

NCMA recommends an allowable stress design (ASD) methodology, when calculating the driving forces and moments, the resisting forces and moments, and designs to a minimum factor of safety of resisting forces / driving forces. The recommended factors of safety are shown in Table 1.

As a final design note, NCMA requires a minimum geogrid length reinforcement of 60 percent of the height of the wall or 4 ft (1.2 m), whichever is greater.

AASHTO Methodology

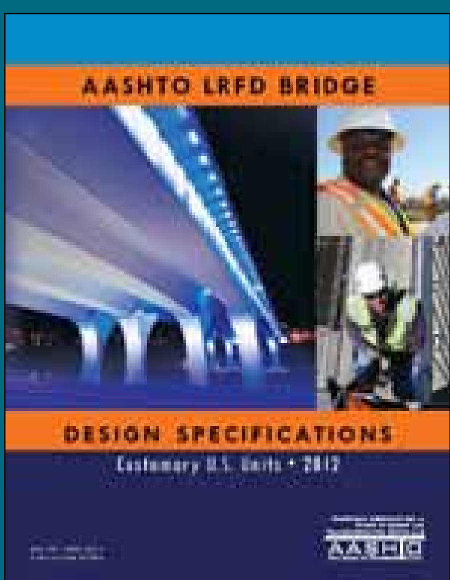

Figure 4. AASHTO LRFD Bridge Design Specification 2012

The design methods used in the transportation market have changed over the years. In the 1990s design was based on a Rankine design approach. In the early 2000s the method was simplified using a Rankine approach for walls with less than 10 degrees face batter and assuming the slope above the wall is an equivalent surcharge (level surface half the height of the slope above the wall at 70% depth). In 2007, a Load and Resistance Factor Design (LRFD) method was adopted. It is interesting how the AASHTO and NCMA methods have converged and then diverged but the results remain similar. In this article, we will only address the LRFD design approach currently in use by the transportation markets (see the SRW History Article III, *SRW Design* in this series for more details regarding SRW advancements).

In LRFD, "load factors" are applied to the driving forces and moments and "resistance factors" are applied to the resisting forces. The results from allowable stress design (ASD) and current LRFD design are very similar due to the methodology used to determine the load and resistance factors. For example, ASD design uses a factor of safety of 1.5 on the resisting forces for tension divided by the driving forces. In LRFD design, a load factor of γEV of 1.35 is applied on the calculated earth pressure and a resistance factor (RF) of 0.9 is applied in the calculation. The equivalent factor of safety would be 1.35/0.9 = 1.5, the same as an ASD design. In LRFD, different load factors and resistance factors can be applied to different loads, the same way the factors of safety can be changed by the designer. In Strength I Load combination (see Table 2) design (maximum forces):

- Internal earth loads are factored by 1.35
- Live loads are factored by 1.75 (more uncertainty)
- Dead loads are factored by 1.5 (less uncertainty)
- External driving earth loads are factored by 1.5

Table 1. NCMA Table 5-2: Recommended Minimum Factors of Safety and Design Criteria for Conventional/ Reinforced SRWs (1, 2, 3, and 4)			
Failure Modes		**Static**	**Seismic**
Wall Design			
Base Sliding	FS_{sl}	1.5	1.1
Overturning	FS_{ot}	1.5/2.0	1.1/1.5
Internal Sliding	$FS_{sc}/FS_{sl(i)}$	1.5	1.1
Tensile Overstress	FS_{to}	1.5	1.1
Pullout	FS_{po}	1.5	1.1
Connection	FS_{co}	1.5	1.1
Internal Compound Stability	FS_{com}	1.3	1.1
Geotechnical Concerns			
Bearing Capacity	FS_{bc}	2.0	1.5
Global Capacity	FS_{gl}	1.3-1.5	1.1
Other Wall Design Criteria			
Minimum Reinforced Zone Width	L	0.6H or 4ft (1.2 m)	
Minimum Wall Embedment	H_{emb}	0.5 ft (152 mm)	
Minimum Anchorage	L_a	1.0 ft (304 mm)	
Maximum Wall Batter	ω	20 degrees	

1. The minimum factors of safety given in this table assume that stability calculations are based on measured site-specific soil/wall data. Measured data are defined as the results of tests carried out on actual samples of soils and geosynthetic products at the proposed structure and actual samples of masonry concrete units (i.e., the same molds, forms, mix designs and infill material or same broad soil classification (e.g. G, S, if applicable).
2. When estimated data is used, the designer may need to use larger factors of safety than those shown in this table or conservative estimates of parameter values. Estimated data includes bulk unit weight and shear strength properties take from the results of ASTM methods of testing (or similar protocols) carried out on sample of soil having the same USCS classification as the project soil and the same geosynthetic product.
3. Estimated data for facing shear capacity and connection capacity analyses shall be based on laboratory tests carried out on the same masonry concrete unit type under representative surcharge pressures for the project structure (and the same broad soil classification type, e.g. G.S. if applicable).
4. To determine maximum unreinforced wall height, determine height to which factors of safety for conventional SRWs are satisfied.
5. Minimum reinforcement length is 0.6H and must meet minimum requirements above.
6. Wall embedment to be determined as per Table 5-1 and must meet minimum requirements above.

In serviceability design (ASD), all of the load factors are set to 1.0. In seismic design, the load factors again are set to 1.0. (See Tables 2 and 3.) AASHTO continuous to work to calibrate the load factors to better meet the design requirements, so factors may change with each update to the AASHTO bridge manual.

As shown on the tables, LRFD design requires more design checks, analyzing several different design cases, and then taking the most critical one for the final design.

AASHTO uses a Rankine earth pressure approach to calculate the internal earth pressures when the wall batter is less than 10 degrees. The top surface is assumed to be level with any slope modeled as an equivalent earth surcharge load. Therefore, with this assumption, the wall friction angle (δ) is equal to the slope angle (β), which is zero degrees, and the wall batter is assumed to be vertical. For wall batters greater than 10 degrees, the Coulomb approach is used in AASHTO; however, most transportation walls are near vertical or less than eight degrees in wall batter. The resulting Rankine earth pressure is a little more than the Coulomb approach NCMA utilizes.

For calculating the geogrid connection strength, AASHTO has a more conservative approach than NCMA, which takes into account the ratio of the ultimate tension (T_{ult}) of the geogrid (published: Minimum Average Roll Value (MARV)) divided by the lot strength (T_{lot}), strength of any individual lot, and includes a factor for long-term connection strength of the connection (CR_{cr}). AASHTO also includes a resistance factor of 0.9 for pullout of the geogrid

Table 2. Load Combinations and Load Factors (Source: AASHTO 2012 Table 3.4.1-1														
Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	*LL* *IM* *CE* *BR* *PL* *LS*	*WA*	WS	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time *EQ*	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	--	--	1.00	0.50/1.20	γ_{TG}	γ_{SE}	--	--	--	--	--
Strength II	γ_p	1.35	1.00	--	--	1.00	0.50/1.20	γ_{TG}	γ_{SE}	--	--	--	--	
Strength III	γ_p	--	1.00	1.40	--	1.00	0.50/1.20	γ_{TG}	γ_{SE}	--	--	--	--	--
Strength IV	γ_p	--	1.00	--	--	1.00	0.50/1.20	--	--	--	--			
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}					
Extreme Event Event I	γ_p	γEQ	1.00	--	--	1.00	--	--	--	1.00	--	--	--	--
Extreme Event Event II	γ_p	0.50	1.00	--	--	1.00	--	--	--	--	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	--	--	--	--	--
Service II	1.00	1.30	1.00	--	--	1.00	1.00/1.20			--	--	--	--	--
Service III	1.00	0.80	1.00	--	--	1.00	1.00/1.20	γ_{TG}	γ_{SE}	--	--	--	--	--
Service IV	1.00	--	1.00	0.70	--	1.00	1.00/1.20	--	1.0	--	--	--	--	--
Fatgue I - *LL*, *IM*, & *CE* only	--	1.50	--	--	--	--	--	--	--	--	--	--	--	--
Fatgue I II- *LL*, *IM*, & *CE* only	--	0.75	--	--	--	--	--	--	--	--	--	--	--	--

from the facing elements. (T_{marv} is a conservative, published minimum average roll value, generally not the actual MARV value. T_{marv}/T_{lot} is generally about 3% to 5%, but the conservative published values tend to be toward 7% to 9%).

For those systems that actively participate in the transportation market and that have completed the long-term testing, the range of values appears to be between 1.0 and 1.3 for frictional systems and equal to the geogrid creep reduction factor for mechanically connected systems. (See the discussion under "The Future" to see the changes coming.)

Resistance factors for AASHTO LRFD are shown in Table 3 and 4.

PROJECT SPECIFICATIONS

An area having the most differences is in the project specifications. NCMA designs were established for commercial and residential projects providing safe, economic designs. AASHTO specifications are written for transportation structures following the established requirements for panel/steel reinforced mechanically stabilized earth (MSE) walls.

Table 3. Load Factors for Permanent Loads, γ_p (Source: AASHTO 2012 Table 3.4.1-2)			
Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DC: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
• Metal Box Culverts and Structural Plate Culverts with Deep Corrugations		1.5	0.9
• Themoplastic Culverts		1.3	0.9
• All Others		1.95	0.9
ES: Earth Surcharge		1.50	0.75

Table 4. Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State (Source: AASHTO 2012 Table 10.5.5.2.2-1)			
Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical Method (Munfakh et al., 2001), in clay	0.50
		Theoretical Method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical Method (Munfakh et al., 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_t	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

Soil Recommendations

Designs using geosynthetic reinforcements do not require non-corrosive backfills, so using granular on-site soils is generally preferred. NCMA's recommendation on backfill soils are:

> *The reinforced backfill shall be free of debris and consist of one of the following inorganic USCS soil types: GP, GW, SW, SP, SM, meeting the following gradation as determined in accordance with ASTM D422 [Ref. 9]. (See Table 5.)*

Table 5. NCMA Reinforced Backfill Requirements

Sieve Size	*Percent Passing*
1 in.	*100*
No. 4	*100 - 20*
No. 40	*0 - 60*
No. 200	*0 - 35*[1]

(NCMA 2009, Ref. 18)

AASHTO specifications were originally developed for steel reinforcing systems that required non-corrosive fill soils, granular, and free-draining fills. A typical transportation specification for reinforced fill would be:

Table 6. AASHTO Granular Fill Requirements

NHI Table 3-1. MSE Wall Select Granular Reinforced Fill Requirements

Gradation:	U.S. Sieve Size	Percent Passing[a]
(AASHTO T-27)	4 in. (102 mm)[a,b]	100
	No. 40 (0.425 mm)	0 - 60
	No. 20 (0.075 mm)	0 - 15
Plasticity Index, PI (AASHTO T-90)	PI < 6	
Soundness: (AASHTO T-104)	The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles (or a sodium sulfate value less than 15 percent after five cycles).	

Notes:
(a)To apply default F* values, C_u, should be greater than or equal to 4.
(b)As a result of recent research on construction survivability of geosynthetics and epoxy coated reinforcements, it is recommended that the maximum particle size for these materials be reduced to 3/4-in. (19 mm) for geosynthetics, and epoxy and PVC coated steel reinforcements unless construction damage assessment tests are or have been performed on the reinforcement combination with the specific or similarly graded large size granular fill. Prequalification tests on reinforcements using standard agency fill materials should be considered.

(FHWA-NHI 2009, vol.1, Ref. 13)

NOTE: a free-draining fill has generally less than 5 percent fines. Fills up to 15 percent silty fines will allow for good drainage. Clayey fills are not recommended.

Segmental Retaining Wall (SRW) Units

Commercial projects follow ASTM C1372 (Ref. 8) which specifies a minimum concrete compressive strength of 3,000 psi (20.5 MPa) for normal weight aggregates. AASHTO is more conservative requiring minimum concrete strengths of 4,000 psi (27.5 MPa).

In areas of repeated freezing and thawing, ASTM C1372 allows to demonstrate durability with proven field performance or testing the units for freeze-thaw durability following the ASTM C1262 (Ref. 7) method. To meet the ASTM C1372 generally accepted criteria five specimens shall have less than 1% weight loss after 100 cycles of repeated freezing and thawing in water or if the before criteria is not met, four out five specimens shall have less than 1.5% weight loss after 150 cycles in water.

The transportation market is subjected to use of deicing chemicals on roads, bridges and walkways. Deicing chemicals (calcium chloride, sodium chloride, potassium chloride, or magnesium chloride) are very aggressive and can be damaging to concrete; the salts may cause scaling and breakdown of the concrete. This damage applies to SRWs as well as cast-in-place walls that are subject to road spray during winter months. Many transportation specifications may require higher compressive strengths for better durability and a more stringent requirement than ASTM C1372. For example, some State Department of Transportation specifications require less than 1% of weight loss after 40 cycles of repeated freezing and thawing in a 3% saline solution for freeze thaw durability.

Inspection

Commercial projects recommend construction inspection as the SRWs are constructed, however this may not be a requirement, and if that is the case, construction inspection can be neglected. Residential walls may or may not be professionally designed and often do not have on-site testing included with the construction specifications. The lack of construction inspection in the field can result in poor quality construction and could result in performance issues in the walls.

Transportation markets generally have full-time site inspection in place for all projects. The testing frequency and acceptance criteria are set in the construction specifications. The fill soils are generally imported and checked regularly for gradation and plasticity. The contractors for transportation projects are usually well-qualified and most likely have constructed several similar projects. In these cases, the quality assurance on the site is typically good and walls perform as intended.

SUMMARY

Summarizing the commercial markets versus the transportation markets is simple:

1. The designs are fairly similar regarding the amount of reinforcing and the facing areas required.
2. Transportation markets are more conservative on material selection, requiring select fill materials for backfill where the commercial markets may use poorer quality (greater fines content), on-site fill material.
3. Transportation markets are more conservative and specify higher concrete compressive strengths for the SRW units. Transportation markets typically require a minimum of 4,000 psi (27.5 MPa).
4. Depending on the state conditions, transportation project may often require freeze-thaw testing in water or saline solutions in areas of repeated freezing and thawing. Commercial markets often do not specify freeze-thaw testing but when they do, a water solution is used instead of the more aggressive saline tests.
5. Commercial project may or may not have on-site inspection where the transportation markets generally require it.

As noted above, the designs for both systems have similarties. The possible advantages of the transportation market are full-time inspection during construction and the use of high quality fills for the reinforced zone. Both of these design and construction items have significant impacts on the long-term performance of the wall work. If everything is properly considered in design and properly constructed, walls will perform well (see comments from Geocomp research below).

SRW APPLICATIONS

The following are some commercial and transportation projects with reinforced SRWs that have been built in the last decade.

Montaño de El Dorado Complex, Scramento, California

The Montaño de El Dorado is a retail/restaurant complex with a challenging topograpy. The site features a high end that re-

Figure 5. Montaño de El Dorado SRW Project (Courtesy of VERSA-LOK)

Montaño de El Dorado Complex
Sacramento California

DEVELOPER

Sacramento Commercial Properties, Folsom, California

SRW LICENSOR

VERSA-LOK

SRW PRODUCER

McNear Brick & Block, San Rafael, California

SRW CONTRACTOR

Retaining Walls Company, Tracy,California

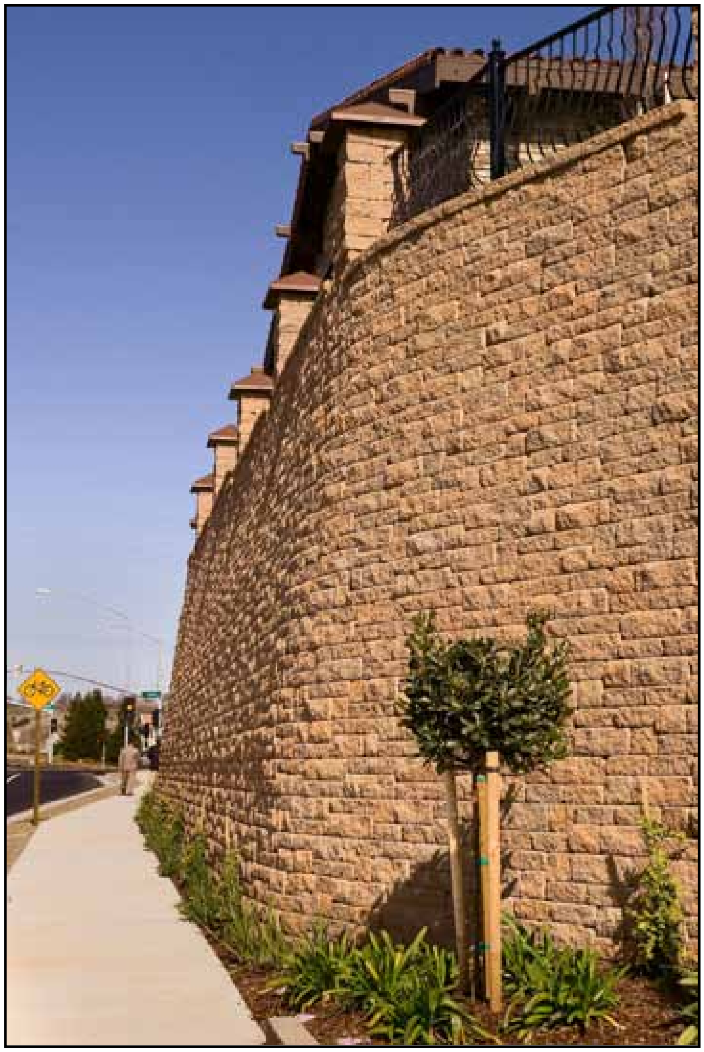

Figure 6. Montaño de El Dorado Complex, Sacramento, California — Commercial Project (Courtesy of VERSA-LOK)

Figure 7. Montaño de El Dorado SRW Project

quired a substantial retaining wall in order to facilitate construction on top and prevent erosion.

The original plan called for a cast-in-place concrete wall approximately 20 ft (6 m) tall and 900 ft (274 m) long with a stucco finish. However, the price of the wall approached $1.5 million which threatened the completion of the project.

The designer proposed a redesign of the wall using SRWs that resulted in saving close to $1 million on the project. The developer was delighted with the wall, accented with columns, decorative lighting, and wrought iron fencing, which gave a higher-end appearance than the original cast-in-place stucco finish wall.

About 20,000 ft^2 (1860 m^2) of SRW units in a Mojave color were used in the construction of the wall, which stretches the length of the development along Latrobe Road and White Rock Road. Stepped and tiered sections with inset plantings, along with a mosaic random pattern, break up the tall wall's size. Atop the wall, freestanding columns adorned with ornamental caps provide additional visual interest required in the area.

Cattlemen Road Extention, Sarasota, Florida

The North Cattlemen Road Extension in Sarasota County, FL, is a $20 Million transportation improvement project that connects University Parkway at its north end with Fruitville Road to the south. With north and south ends of the roadway now joined, local traffic can bypass Interstate 75, and have easier access to some of the area's most desirable commercial, recreational and sporting facilities.

An existing two-lane road was transformed into a four-lane divided roadway, and the four divided lanes were extended further north for another 1.75 miles (2.8 km). Funds supplied by the federal government were administered through the Florida Department of Transportation (FDOT) on behalf of Sarasota County. The project encompasses the road, a 96 ft (29.3 m) bridge on Cattlemen Road to Center Island on a 400 acre (162 hectares) lake, excavation and reconfiguration of a 50 acre (20 hectares) lake, and two additional bridges to a new island built as part of the road's program.

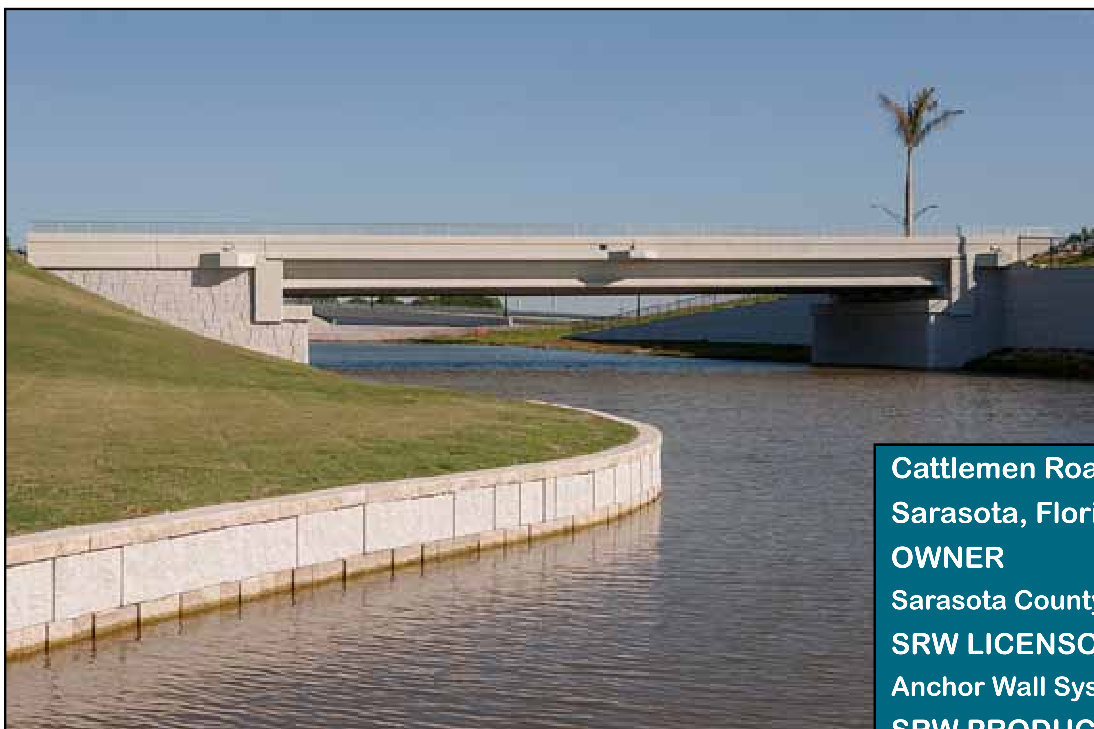

Figure 8. North Cattleman Road Extension Sarasota County, FL — Transportation Project (Courtesy of Anchor Wall Systems/Oldcastle)

Cattlemen Road Extention
Sarasota, Florida
OWNER
Sarasota County/Benderson Development
SRW LICENSOR
Anchor Wall Systems
SRW PRODUCER
Coastal an Oldcastle Company
GEOSYNTHETIC MANUFACTURER
TenCate Geosynthetics
DESIGN/BUILD CONTRACTOR:
Associated Construction Products (ACP), Inc.
Prince contracting Co.

Figure 9. North Cattleman Road Extension Project (Courtesy of Anchor Wall Systems/Oldcastle)

The Cattlemen Road Extension project included realigning a land bridge about 300 ft (91.4 m) away from its original location. The move was necessary to expand the rowing area of the park's main lake and accommodate 6,561 ft (2,000 m) Olympic-class rowing events, with Sarasota's sights set on hosting the 2017 World Rowing Championships.

The masonry retaining walls were less costly upfront including installation and created a beautiful wall that offered flexibility compared to the precast MSE wall originally intended in the project. The SRWs gave all three bridges a cohesive look and allow for a speedy construction without the delay of ording precast panels. Also, the shop drawing review process normally required with other materials was shortened, the daily installation production of the contractor was higher than panel MSE walls and the maintenance of this system is less than others. Finally the switch to SRW units was also a cost savings for the owner thanks to the ability to use on site fill material as opposed to importing expensive select fill. The material available was a well-graded, coarse aggregate with less than 15% of fines (material finer than the #200 sieve) with a plasticity index less than 6%, but it didn't meet the stringent electrochemical requirements to be used with metallic reinforcement.

The project totaled 50,000 ft^2 (4645 m^2) of facing in four bridges with a maximum height of 18 ft (5.5 m). Once the walls were completed, erosion protection was added to the front of the walls to avoid scour due to the water moving in front.

THE FUTURE

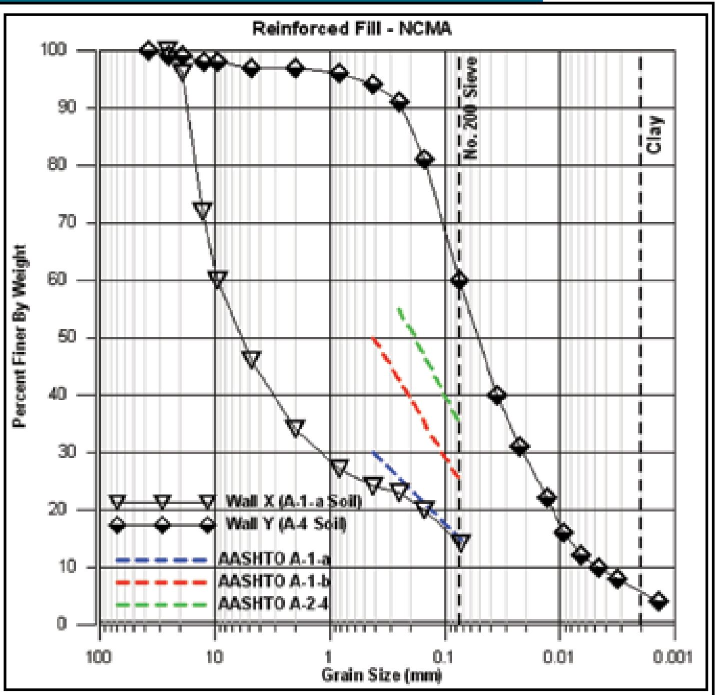

Figure 10. NCMA Test Wall Graduations (Geocomp 2009, Ref. 15)

In the late 90s, Mr. Tony Allen, State Geotechnical Engineer of Washington Department of Transportation initiated an investigation of wall performance reviewing all available data (Allen 2003, Ref. 4). Funding was provided by pooled funds from the State of Washington transportation group and other state departments of transportations. The research was completed by Dr. Richard Bathurst, Royal Military College, Kinston, Ontario and Dr. Robert Holtz, University of Washington. The results of this research and modeling showed that the actual loads in the reinforcing of geosynthetically reinforced MSE walls were only 30% to 50% of the projected design loads. The AASHTO T15 committee is now looking at revising the design methodology to more accurately predict internal stresses on the reinforcement.

In 2005, NCMA contracted with Geocomp to build an instrument SRW test walls parallel to a study done by National Cooperative Highway Research Project (NCHRP) Project 24-22, Selecting Backfill Materials for MSE walls (Geocomp 2009). The purpose of the study was to evaluate the use of lower quality fills in the construction of MSE with geosynthetic reinforcement wall structures. The NCMA test walls were 20 ft (6.1 m) tall, one backfilled with an AASHTO type highway fill (less than 15% fines) and the other designed with a fine grained fill with 60% passing the No. 200 sieve. The walls were designed by conventional methods and were designed to fail under the applied loads.

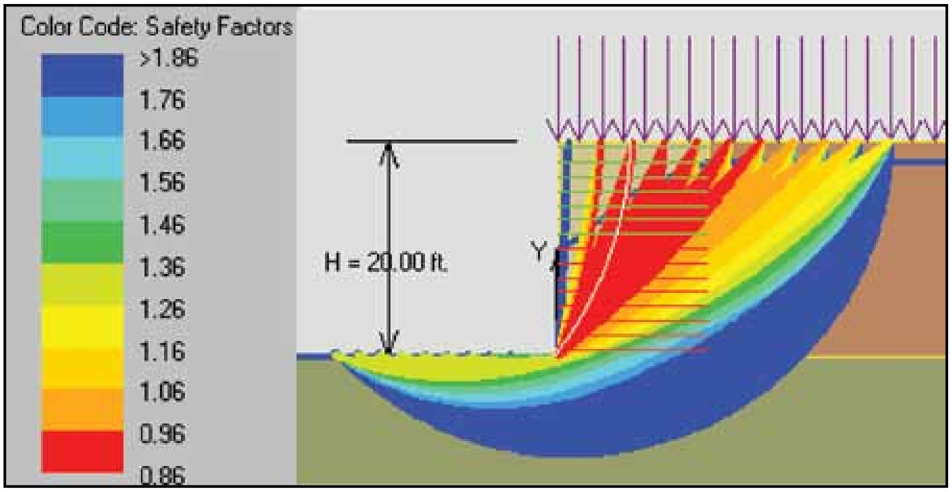

Figure 11. Example "Safety Map" from Limit Equilibrium Analysis (ReSSA, Ref. 2)

Figure 11 shows a 'safety map' of the structure. The zones in red have a factor of safety of less than 1 (failure). Figure 12 shows wall deflections under the different fill conditions. The fine-grained soils moved more than the select materials but did not impact the wall performance. Results of the study showed the walls deflected but none failed, in spite of being designed to do so.

The Geocomp report showed the design loads are much less than the predicted loads (confirming assumptions derived from the K-Stiffness development). It was also confirmed that the use of soil with up to 25% fines can be successfully used as backfill materials. The research showed that walls with up to 60% of fines in the backfill could be built when the necessary changes are made to the design to account for the long-term changes these materials could have. This is especially important to the transportation markets where select fills are more difficult to locate and more expensive to process and more option are necessary. It also confirms NCMA's recommendation of 35 % maximum fines for commercial projects that walls will perform as intended. The transportation and commercial markets are moving in the same direction on researching other types of wall backfills.

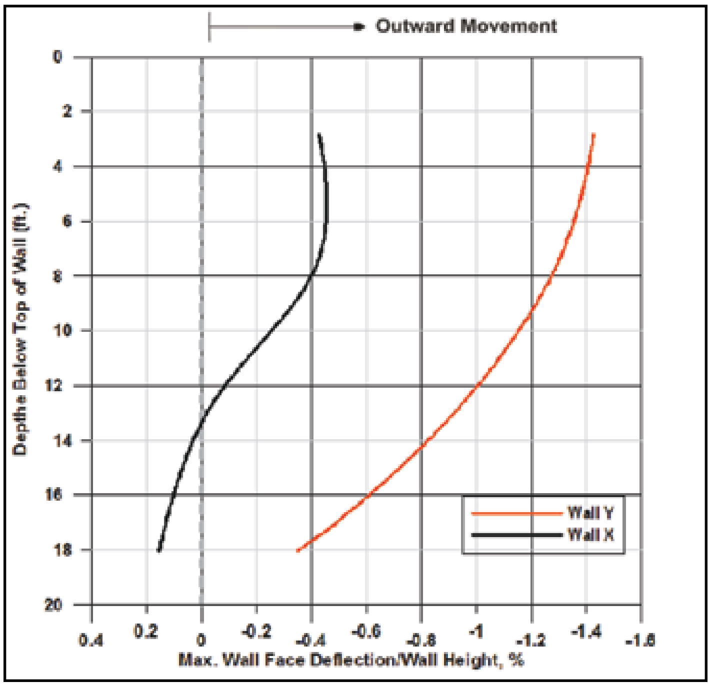

Figure 12. Maximum Wall Deflections as Percent of Wall Height (Ref. 15)

FHWA GRS Wall

Although not falling under the transportation (AASHTO) markets, the Federal Highway Administration (FHWA) developed a Geosynthetic Reinforced System (GRS) using a zero-slump concrete facing and geosynthetic (fabrics) reinforcing. These walls have been used for bridge abutments and are performing well (Adams 2011, Ref. 3).

Figure 13. Geosynthetic Reinforced Soil — Integrated Bridge System (GRS-IBC) Wall

In the GRS design there is a layer of reinforcement on every course of units (every 8 inches (200 mm)). With the tight spacing of reinforcement the influence zone of one layer interacts with the adjacent layer and the result is very little design loading applied to the facing elements. Therefore, creep on the connection is not an issue and connection strength is not a design issue (Nicks 2013, Ref. 20).

NCMA is investigating the GRS approach to design and it is very feasible that geogrid spacing on a two-unit spacing (16 in. (400 mm)) will have a very similar design impact at every unit. This improved performance was shown in full scale seismic testing done by Ling (Ling 2002, Ref.16 and Ling 2006, Ref. 17) with two unit spacing instead of the more traditional three unit spacing (24 in. (600 mm)).

IMPROVEMENTS

The transportation market (AASHTO) is working on new design methodology to reduce the calculated design loads; this may come to the market in four to five years (the State of Washington already has a K-Stiffness design in their project specifications). FHWA has data published on well-performing GRS bridge abutments; this design installation method is completely different on connection and connection creep performance requirements. The commercial market is investigating improvements to its design methodology to reflect the research that has been conducted, good performance of its test walls in the Geocomp study, and published research. Moving forward the Transportation and commercial markets will likely improve design methodologies to more competitive and reliable designs.

REFERENCES

1. AASHTO, "AASHTO LRFD Bridge Design Specification", AASHTO, Washington, D.C. 7th edition, 2014.
2. ReSSA software, ADAMA Engineering.
3. Adams, M.T., Nicks. J, Stable, T., Wu, J., Schlatter, W., and Hartman, J., Geosynthetic Reinforced Soil Integrated Bridge System, Interim Implementation Guide, Federal Highway Administration report FHWA-HRT-11-026, January 2011.
4. Allen, T.M. and Bathurst, R.J., "Prediction of Reinforcement Loads in Reinforced Soil Walls," Washington Department of Transportation, 2003.
5. ASTM C33, "Specification for Concrete Aggregates, American Society for Testing and Materials," American Society for Testing and Materials, 2013.
6. ASTM C90, "Standard Specification for Loadbearing Concrete Masonry Unit," American Society for Testing and Materials, 2013.
7. ASTM C1262, "Standard Test Method for Evaluating the Freeze-Thaw Durability of Manufactured Concrete Masonry Units and Related Concrete Units,", Annual Book of ASTM Standards 2003, Volume 04.05, ASTM International, West Conshohocken, PA, pp. 829–832, 2013.
8. ASTM C1372, "Standard Specification for Dry-Cast Segmental Retaining Wall Units," American Society for Testing and Materials, 2013.
9. ASTM D422, "Standard Test Method for Particle-Size Analysis of Soils," American Society for Testing and Materials, 2013.

10. ASTM D6638, "Standard Test Method for Determining Conncection Strenght Between Geosynthetic Reinforcement and Segmental Concrete Units (Modular Concrete Blocks), American Society for Testing and Materials, 2011
11. ASTM D6706, "Standard Test Method for Measuing Geosynthetic Pullout Resistance in Soil", American Society for Testing and Materials, 2013
12. Bathurst, R. J. and Huang, B., "A Geosynthetic Modular Block Connection Creep Test Apparatus, Methodology, and Interpretation", Geotechnical Testing Journal, Vol. 33, No. 2.
13. Berg, R., Christopher B. R., and Samtani, N., "FHWA-NHI-10-024: Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes", Washington DC, 2009
14. Coulomb, C.A. (1776), "Essai sure une Application des Regles des Maximis at Minimus a quelques Problemes de Statique Relatifs a l'Architecture" (An attempt to apply the rules of maxima and minima to several problems of stability related to architecture), Mem. Acad. Ro. Des Sciences, Paris, 3, p. 38.
15. Geocomp Corporation, Report on Full-Scale Test Walls, Leominster, MA, Geocomp Corporation, Boxborough, MA, 2009.
16. Ling, H., Leshchinsky, D. and Bott, T., "Executive Summary, Seismic Testing, Geogrid Reinforced Soil Structures Faced with Segmental Retaining Wall Block", Columbia University, 2002.
17. Ling, H., Mohir, Y., Leshchinsky, D., Burke, C., Matsushima, K., and Liu, Huabei, Large-Scale Shaking Table Tests on Modular-Block Reinforced Soil Retaining Walls, Journal of Geotechnical and Geoenvironmental Engineering, April 2005, pp 465-476.
18. National Concrete Masonry Association, "Design Manual for Segmental Retaining Walls, 3rd ed.," TR 127B, Herndon, VA. 2009.
19. NCMA-ICPI, Quality Control Manual for Dry-Cast Manufactured Concrete Products Production Plants, BETA Edition No. 9, February 2012.
20. Nicks, J.E., Adams, M.T. and Alzamora, D.E., Frictional Connection Testing of Geosynthetics and Implications on Design, University of Minnesota, 61st Annual Geosynthetic Engineering Conference, 2013).