LESSONS LEARNED FROM A SEGMENTAL RETAINING WALL FAILURE

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1.0 INTRODUCTION

In North America, the use of segmental retaining walls (SRWs) has gained wide acceptance as an economical alternative to both conventional cast-in-place concrete retaining walls and mechanically stabilized earth (MSE) walls using metallic reinforcements. The design procedures for determining the external, internal, and facing stability of SRWs, and for determining the long-term allowable strengths of the reinforcement have become well established state-of-the practice procedures as specified in the NCMA Design Manual for Segmental Retaining Walls, Second Edition; AASHTO 1999 Interim Bridge Specifications; and FHWA Demonstration Project 82 – Mechanically Stabilized Earth Walls and Reinforced Soil Slopes).

Research conducted across the world, both in laboratory and full-scale test walls, has demonstrated that the calculated design loads, using any of the procedures listed above, over-estimate the actual loads with respect to the internal stability of an SRW. These design procedures, therefore, appear to be conservative. However, there are still a large number of SRWs that do not perform as intended. Failures are occurring across the country. What are the reasons for the abundance of poorly performing SRWs when the design approach has been proven to be conservative? The following is a list, based on the author's experience, of many of the causes of these SRW failures:

- Poor construction
- Poor engineering
- Inferior quality materials
- Unexpected conditions (Acts of God)
- Lack of coordination of responsibilities between the Owner, the Design Consultants, and the Contractor

This paper will focus on a failure of an SRW that can be attributed to poor engineering and the lessons that may be learned from critically reviewing the causes of the failure. A case history will be presented which involves the facing stability of a 28.5 foot high SRW.

2.0 FACING STABILITY CASE HISTORY

A commercial development in New England utilized an SRW to optimize the available space on the site. The maximum height of the wall was 28.5 feet and was over 400 feet in length. The average wall height was approximately 20 feet. The wall was designed by a consultant under contract to the material supplier. The wall was constructed by a specialty contractor under a separate contract with the developer. The SRW system that was utilized on this project consisted of a segmental concrete unit that uses fiberglass pins for alignment. Steel mesh was used as the primary reinforcement, and polyester geogrid was used as the secondary reinforcement. The published properties on the SRW unit and reinforcement are provided in Table 1.

Table 1. SRW System Material Properties

SRW Unit	Width (inches)	Length (inches)	Height (inches)	Batter (°)
Unit A	12	16	6	7
Reinforcement Type	**Aperture Size (inches)**	**T_{ult} (lbs/ft)**	**LTDS (lbs/ft)**	
PVC Coated Steel Mesh	3.25 x 4.5	na	2800	
PCV Coated Polyester Geogrid	1.0 x 1.2	1418	738	

2.1 SRW Design

The wall was designed using a vendor developed software program which generally followed the National Concrete Masonry Design Methodology for internal and external stability. However, the software did not directly address facing stability. The program was also developed specifically for one type of reinforcement which was not specified nor utilized on this project.

Geologically the site consists of glacial till. The SRW design called for the reinforced and retained fill to be the site soils. The soil parameters used in the design of the SRW were provided by the geotechnical engineer and are listed in Table 2. The wall was designed assuming no water or hydrostatic pressure would be present within the reinforced fill zone.

Table 2. Soil Parameters

Soil	$\varphi°$	Cohesion	γ (pcf)
Reinforced Fill	32	0	120
Retained Fill	32	0	120
Foundation Soil	32	0	120

A typical cross-section of the wall consisted of primary reinforcement (steel mesh) placed 4 foot on center vertically. The steel mesh, because of its stiffness and thickness (over ¼ inch) was not attached directly to the SRW units. The steel mesh was brought to the back of the SRW units and was terminated before being connected to the SRW units. One layer of secondary reinforcement (PVC coated polyester geogrid) was placed at the mid-height of the primary reinforcement. The secondary reinforcement was attached to the SRW units and extended 3 feet into the reinforced soil mass behind the back of the SRW unit. A typical cross-section of the wall is shown in Figure 1.

2.2 SRW Construction

Construction of the wall began in early November 1998 and was substantially complete by the end of that year. Construction observation was performed by the Developer's geotechnical engineer. A review of the field reports during construction indicates that no deviation from the construction drawings was observed and that the reinforced fill was generally compacted to project specifications.

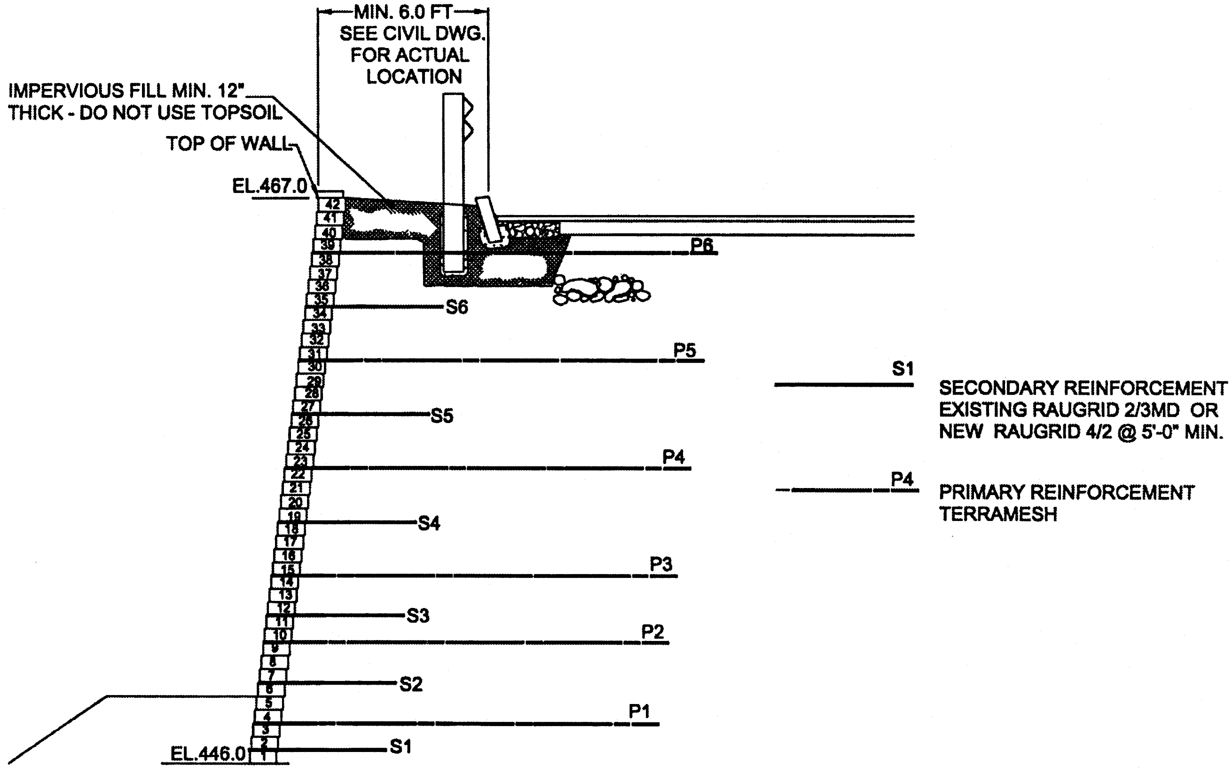


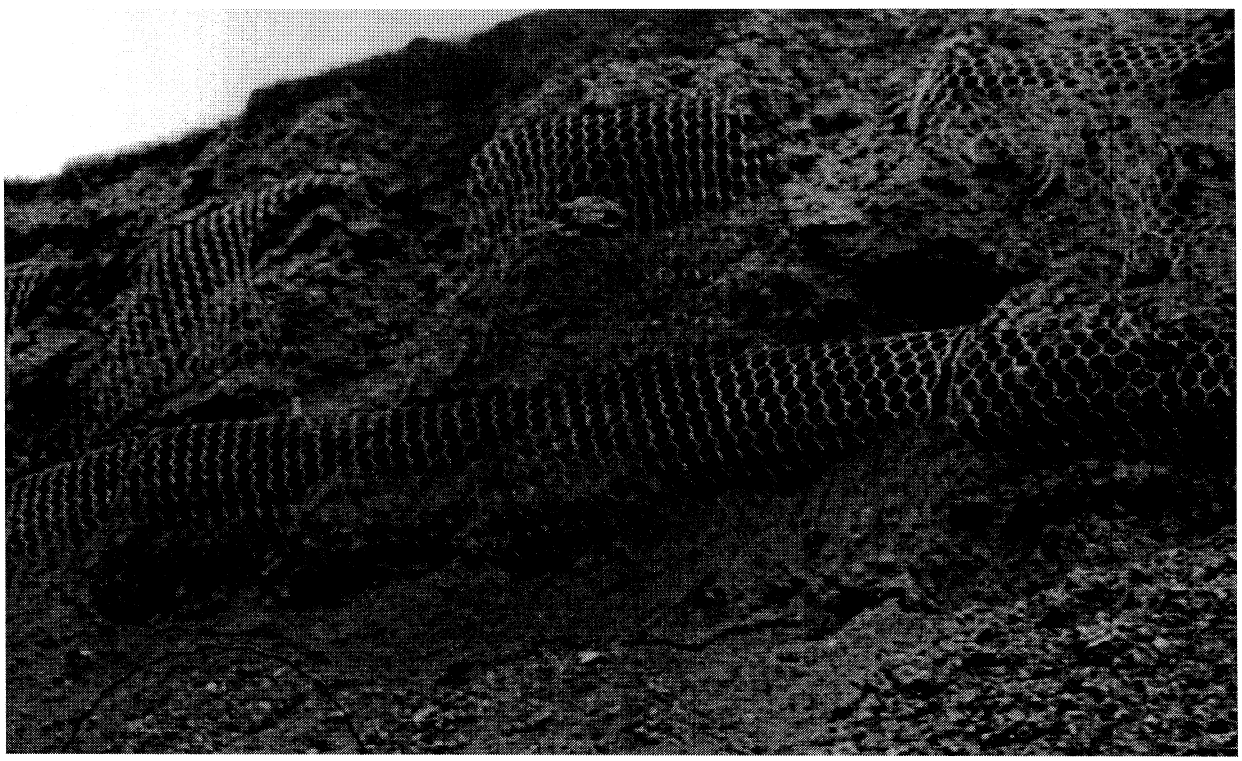
Figure 1. Typical SRW Design Cross Section

2.3 SRW Performance

The first failure occurred at the end of May 1999. The highest section of wall (28.5 ft) failed during a heavy rain storm. Water drained to a catch basin located behind the wall. At the time of the failure, the parking lot above the wall had not been paved, leaving the catch basin entrance above existing ground. Thus, the site water directed to that area could not enter the catch basin and drained into the reinforced fill, subsequently saturating it. Photograph 1 shows the wall after failure. The face of the wall (SRW units and secondary reinforcement) peeled away from the reinforced soil mass. It is evident from the photograph that the reinforced soil mass supported by the primary reinforcement was still performing. The failure was confined to the wall face.

A second failure occurred in September of 1999 during Hurricane Floyd. This failure appeared similar to the first, in that the SRW units, secondary reinforcement and reinforced fill within about 3 to 4 feet behind the face of the wall fell away from the soil mass reinforced by the primary reinforcement.

Both failures occurred during or shortly after significant rainfall events. Water appeared to be the trigger for both failures. Observations the day after the second failure revealed no evidence of seepage, erosion or water movement at the reinforced soil face. The near vertical face appeared to consist of uniformly damp, relatively undisturbed glacial till backfill. During a heavy rain a few days after the failure water was observed flowing out of the pavement base course at the collapsed section of wall.



Photograph 1. Wall Failure - Primary Reinforcement Visible

It appears that for the second failure, the primary water source is near surface infiltration laterally from the adjacent pavement base course and vertically through the grass area between the pavement and the wall face.

2.4 Failure Investigation

A review of the original design was performed using the National Concrete Masonry Association Design Guidelines. Internal, external, facing stability and global stability were evaluated. The original design was based of the assumption that no hydrostatic forces would be acting on the SRW. A check of the original design assuming no hydrostatic pressure was performed. This design check used the NCMA procedure in its entirety. Connection strength properties between the SRW units and secondary reinforcement were based on laboratory tests provided by the manufacturer. The design section presented here is for the tallest section of wall measuring 28.5 ft in height from leveling pad to top of wall. The primary reinforcement length for this section was 18 ft. The secondary reinforcement length was 4 ft. The calculated factors of safety based on the original design assumptions for external stability are listed in Table 3. The original design satisfies external stability with the calculated factors of safety well above the recommended minimum values per NCMA.

Table 3. Original Design - External Factors of Safety – No Water

External Stability	Factor Safety	NCMA Recommended Minimum Factor Safety
FS Sliding	3.50	1.5
FS Overturning	6.02	2.0
FS Bearing Capacity	4.34	2.0

Table 4 lists the factors of safety for internal stability. The internal stability was evaluated considering only the primary reinforcement. Any benefits from the secondary reinforcement were conservatively ignored. The minimum recommended factor of safety for tensile overstress (per NCMA) is 1.0 and the minimum factor of safety for pullout of the reinforcement is 1.5. The factor of safety for tensile overstress is met for each layer of primary reinforcement. The factor of safety for pullout is also met for every layer except the top layer. The top layer of reinforcement must be lengthened to satisfy the pullout requirement.

Table 4 Original Design – Internal Stability - Primary Reinforcement – No Water

Layer #	Elevation	$FS_{\text{tensile overstress}}$	FS_{pullout}
1	26.5	5.92	inadequate
2	22.5	2.99	3.80
3	18.5	2.00	6.43
4	14.5	1.89	11.44
5	12.0	1.88	15.52
6	9.0	1.81	20.62
7	7.0	2.05	28.13
8	5.0	2.17	35.37
9	3.5	2.03	37.31
10	1.5	2.24	47.69
11	0.5	3.21	73.59

Table 5 lists the factors of safety for facing stability of the wall and considers only the secondary reinforcement. Only the secondary reinforcement was considered because the primary reinforcement was not connected to the SRW units but was terminated behind the SRW unit. The design did not meet the recommended factor of safety for connections of 1.5 (per NCMA). In fact, the connection was substantially under-designed. The average factor of safety for the connection was approximately 0.50. The connection between the face of the wall and the reinforced soil mass was capable of carrying only 50% of the design load with no factor of safety. However, the wall did not fail until 6 months after it was complete and not until it was subjected to hydrostatic forces. A review of the original design would suggest that water was the trigger to the failures, but was not the cause of the failures. The cause of the failures was the low overall connection strength of the wall face to the reinforced soil mass through the secondary reinforcement.

Table 5. Original Design - Facing Stability – Secondary Reinforcement – No Water

Layer #	Elevation	FS $_{connection}$
1	24.5	0.61
2	20.5	0.56
3	16.5	0.46
4	13.5	0.44
5	10.5	0.41
6	8.0	0.44
7	6.0	0.52
8	4.5	0.49
9	2.5	0.54
10	1.5	0.38

2.5 Remedial Design

Both wall failures occurred during heavy rain storms. It was obvious that the remedial designs must consider the impact of water on the stability of the structure. As part of the failure analysis the original civil drawings were reviewed. These drawings revealed that a storm water detention structure was located below the parking lot within 25 feet of the face of the SRW. The potential impact of water from this structure loading the reinforced soil mass was therefore included in the design for the fix for the wall. Figure 2 shows the phreatic surface that was used in the redesign of the wall.

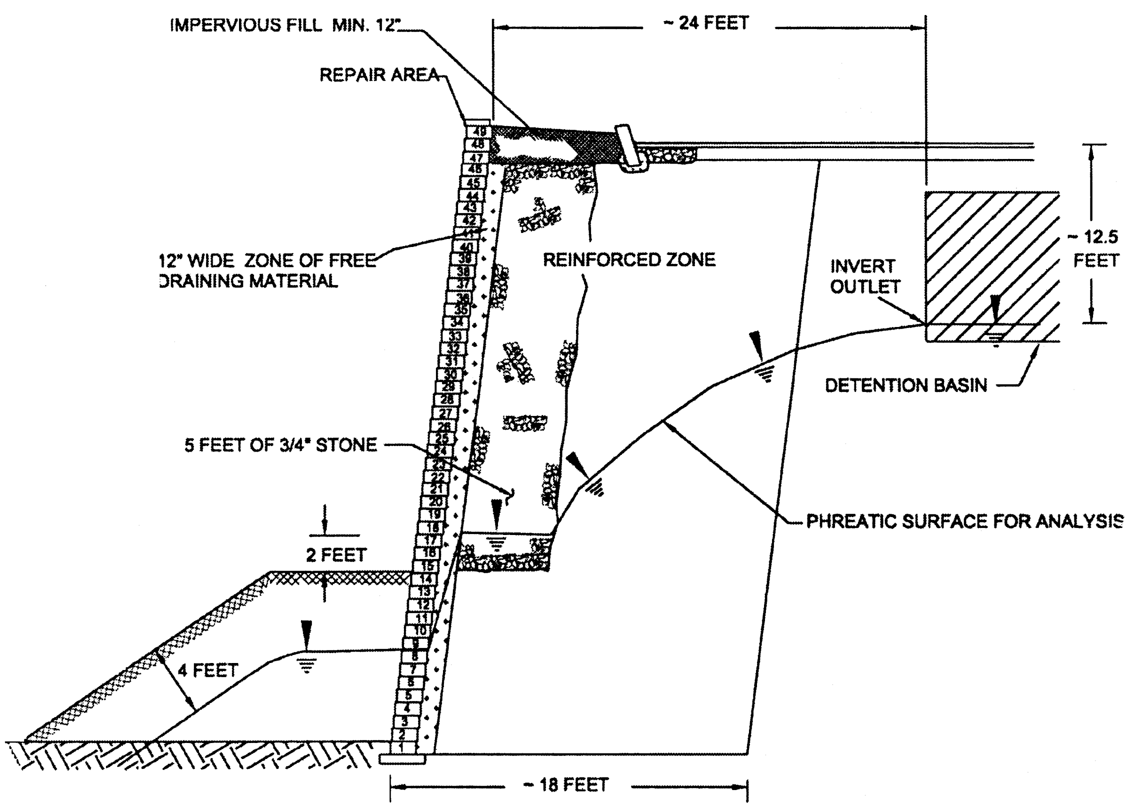
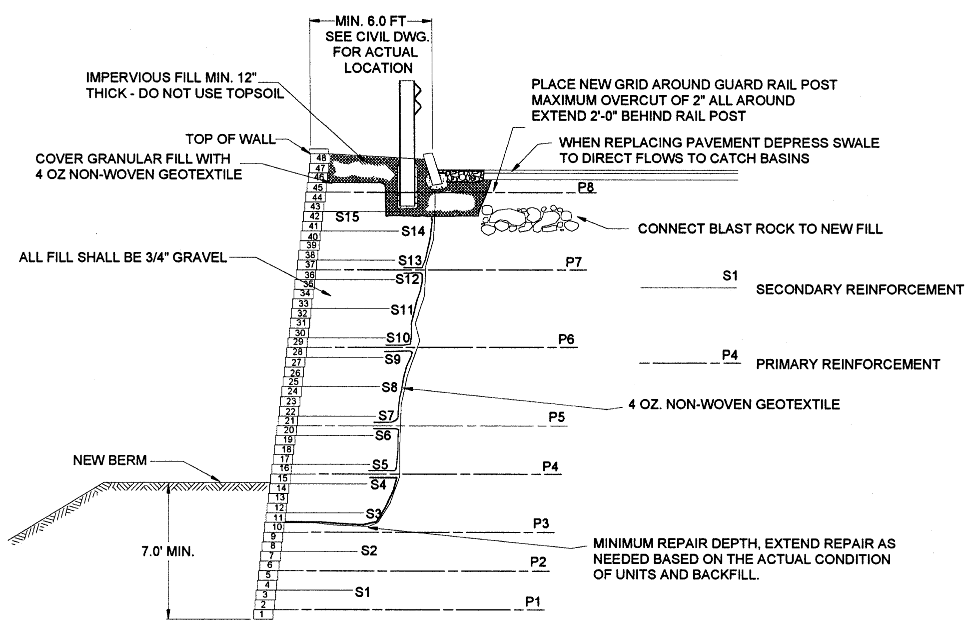


Figure 2. Remedial Design – Location of Phreatic Surface

Current SRW design software does not allow for the modeling of hydrostatic pressures within the reinforced soil mass. Therefore, a spread sheet was developed to analyze the stability of the SRW for external, internal, and facing stability considering hydrostatic forces. The remedial design was developed to meet industry standards for external, internal, facing and global stability considering hydrostatic forces.

Several options were considered to repair the failed sections of the wall and stabilize the sections of the wall that had not yet failed. The owner elected to rebuild the face of the wall in the failed sections with adequate connection capacity. It was also decided that all sections of the wall that did not meet industry standards with respect to facing stability would be rebuilt. This would involve removing the face of the wall and rebuilding it with adequate connection capacity.

The general scheme for repairing the wall was to dismantle the wall face down to within 5 – 8 ft. of the bottom of the wall. Next, the drainage stone and reinforced fill was removed to a depth of 5 ft. behind the face of the wall. The wall face was then rebuilt adding approximately three times the amount of secondary reinforcement than was in the original wall. A berm was added to provide support to the wall face at the bottom of the wall. Several modifications were also made to the site drainage to minimize the potential for water to enter the reinforced soil mass. Figure 3 shows a typical cross-section of the remedial design. The wall was reconstructed in the fall of 2000.



TYPICAL REPAIR SECTION

Figure 3. Typical Cross-Section Remedial Design

3.0 LESSONS LEARNED

There has been much debate within industry about connection strength and connection load requirements for SRWs (Collin 1997). Laboratory research on large-scale model tests suggests that the load on the connection is approximately 50% of the load calculated using the NCMA Design Methodology (Bathurst et al. 2000). Because of the unique design of the SRW reviewed in this case history the connection load was isolated (secondary reinforcement). The failure of this SRW allowed a rare opportunity to evaluate the load at the connection between the SRW units and the reinforced soil mass. At failure, the actual factor of safety of the connection was one. Stability analysis demonstrated that using the loads calculated with the conventional design approach the factor of safety was 0.5. Revising the connection loads to obtain a factor of safety of one reduces the load by a factor of two. This correlates to actual load at the connection of approximately 50% of the design load.

Engineers familiar with the design and performance of SRWs have long suspected that the design loads used in the analysis of SRWs overestimate the actual loads in the structure. This was confirmed in the failure of this SRW. This conservatism may have lead to the original designer ignoring the design of the connection. The software that was used for the original design did not specifically address connections. No calculations were ever provided by the original designer that addressed connections. Had the original design addressed connections in accordance with industry practice (NCMA Design Manual) the wall would have performed as intended even with the addition of hydrostatic pressures.

The software that was used by the original designer was based on a different SRW system than the one that he designed. Inherent in the software was the fact that for the system with which it was intended to be used, the connection capacity was equivalent to the strength of the reinforcement. Therefore, for that system, connections did not control the design. The engineer who elected to use this vendor supplied software without fully understanding it, applied it to an SRW system that was not compatible with the program's inherent assumptions. However, had the designer used a generic program that allows the user to enter the connection capacity of the system, the unique nature of the composite SRW system used on this project could have been considered in the design. The connection between the SRW units and the reinforced soil mass is an important component in the design of SRWs. The unique characteristics of any SRW system must be considered in the design. Only by doing this will we eliminate design related connection failures of SRWs.

References

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