New Horizons in Earth Reinforcement – Otani, Miyata & Mukunoki (eds)

Case study of a MSE wall supporting a multi-story building

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ABSTRACT: The proposed construction of a multi-story building supported by a Mechanically Stabilized Earth (MSE) retaining wall provided many unique engineering challenges. Foremost was that the building's shallow strip and spread footings were founded directly on top of and behind the reinforced MSE retaining wall mass. These footings imparted maximum loads of 120kPa onto the reinforced volume. The design was complicated by the presence of a permanent lake adjacent to the MSE wall, which meant evaluating rapid draw-down and dam breach conditions as well as increased metal loss issues in submerged wall applications. Further design complexities included a required one horizontal to eight vertical (1 H: 8 V) front face batter and continuous stone fascia.

This paper presents the design methods and modeling used for evaluating internal stability (bond and strip rupture), external stability (overturning and sliding) and global stability. All designs were checked for normal (steady state) conditions, as well as rapid draw-downs. The engineering design also included global stability analysis for normal conditions as well as rapid draw-down and the unlikely event of a catastrophic dam failure. This discussion will also include the design modifications necessary to alter a standard vertical face MSE wall system in order to meet the required batter with continuous stone fascia. Special considerations given for drainage, select backfill and geotextile selection for joint cover in submerged applications will be presented.

1 INTRODUCTION

1.1 *Preliminary assumptions*

Bid plans for the site work for *The Tom Harkin Global Communications Center at the Centers for Disease Control and Prevention* in Atlanta, DeKalb County, Georgia indicated that a Mechanically Stabilized Earth (MSE) retaining wall was required adjacent to the building plaza. The wall was to provide a grade separation between the plaza and a permanent lake as well as providing a scenic overlook. Structural plans were not part of the site work and it was assumed by The Reinforced Earth Company® (RECo), when preparing their bid, that the building included a basement. The wall being in close proximity to the building meant that the discrete metallic strips used to reinforce the wall mass extended to the assumed basement in some areas. The benefit of this was that there is not a lateral load due to active earth pressure against the reinforced mass in that case.

1.2 *Final reality*

RECo was successful in their bid and final retaining wall plans were prepared from the site package. The wall construction plans were forwarded to the wall installer, MC Inc., and to the General Contractor, Turner Construction. These plans were in turn submitted to the Architect, Thompson, Ventulett, Stainback & Associates, and to U.S. Government reviewing agencies for review and comment. Plans were returned not approved with annotations concerning building loads on top of the wall. Building plans and loading were then provided and re-engineering of the retaining wall was initiated.

2 DESIGN

2.1 *Design requirements*

Normally RECo is responsible only for the internal stability of the MSE mass, including pullout resistance

Figure 1. The Tom Harkin global communications center.

and rupture of the soil reinforcement. The external stability of the structure; sliding, overturning and slope stability (i.e. compound global stability) is usually the domain of the Geotechnical Engineer. On this project, however, the responsibility was placed on Construction Manager and the Retaining Wall Engineer for the overall wall stability.

Although the required design life was seventy five (75) years, calculations for stresses and factors of safety were based on one hundred (100) years. This was due to critical nature of the project and a portion of the wall being in water. It should be pointed out that the steel design is based on 0.55 of the yield stress of steel (f_y) at the end of the design life. Design stresses and factors of safety were calculated for working stress in accordance with 1996 AASHTO Specifications for Highway Bridges.

2.2 *Loading conditions*

Initially the preliminary design of the wall only considered the case of a level backfill with 6kPa pressure from pedestrian surcharge with no regard to the building. Final design considered in addition to the active earth pressure loads, that the MSE wall was to support footing loads from a four-story building bearing directly on the reinforced mass. The loads imposed by the building were concentrated on shallow spread footings and strip footings. The vertical pressures imposed by the spread footings and the strip footings reached 135 kPa and 125 kN/m respectively. These

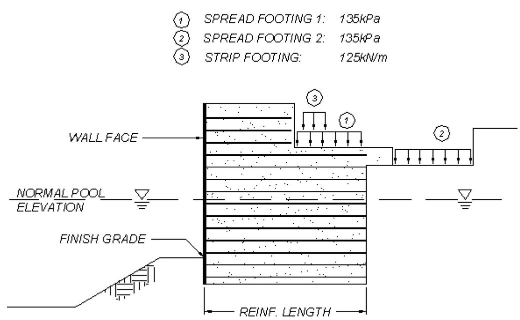

Figure 2. Cross section showing the loading conditions with building loads and normal pool elevation.

values were converted to horizontal pressures using empirical Boussinesq's formulae.

The MSE wall was also designed for permanently submerged conditions due to the presence of a manmade lake in front of the wall. Figure 2 shows an example of loading conditions with building loads and permanent water elevation (normal pool). Because of a potential of high water elevation, the wall was designed to withstand 0.90 m of rapid drawdown from the 100-year flood elevation. The 100-year flood elevation was approximately 3.20 m higher than the permanent water elevation. The wall was also modeled with hydrostatic loads corresponding to a rapid drawdown from the 100-year elevation to the toe elevation on the downstream side of the lake dam (up to 6.25 m of rapid drawdown).

Table 1. Design parameters.

	Structural backfill*	Retained backfill	Residual foundation
Internal friction angle (degree)	45	30	30
Cohesion	0	0	0
In-place unit weight (kN/m^3)	15.1	18.1	18.1
Saturated unit weight (kN/m^3)	17.3	18.9	18.9

* Structural backfill is open graded stone.

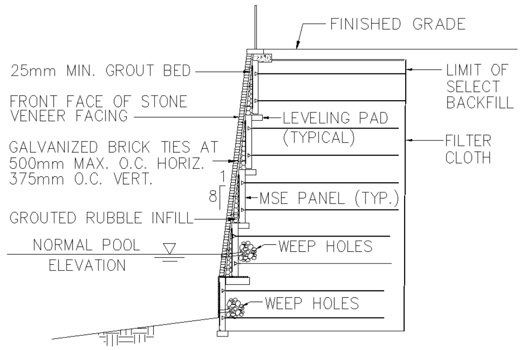

Figure 3. Cross section showing weep holes, filter fabric and stone fascia.

2.3 *Subsurface conditions*

Based on the Geotechnical Engineer's (MACTEC) recommendations, the walls were to be founded on residual soils or structural fills. In selected areas, between 1.00 m and 1.50 m of existing fills were excavated underneath the wall and replaced with structural fill (compacted open graded stones). This provided a solid foundation for the MSE wall and thus controlled the amount of differential settlement between the shallow spread footings of the four-story building.

Based on in-situ and laboratory tests, geotechnical parameters were determined to use for the design of the wall. These design parameters are presented in Table 1. Please note that the internal friction angle for the structural backfill tested at 49 degrees maximum, but 45 degrees was used based on a corresponding 12 mm of movement in the direct shear test.

2.4 *Design considerations*

Several design considerations were addressed to satisfy the unique conditions of this MSE wall. Special considerations were given to submerged conditions, the complexity of the wall and the importance of the building structure that it supports.

First, fluctuation of the water in the reinforced mass induces temporary hydrostatic pressures, which need to dissipate. To make this possible, the backfill consisted of a free draining open graded stone. On a typical MSE wall a 20 mm open joint between the panels would allow dissipation of the water. However, because of the specified stone masonry veneer to be installed in front of the wall, the MSE panel joints were sealed and waterproofed. Therefore, to mitigate potential hydrostatic pressure, weep holes (refer to Figure 3) were installed every 1.50 m just above the finish grade at the bottom of the wall and also provided at 1.50 m centers at 150 mm below normal pool.

Second, to prevent the retained backfill, which contains materials passing the #200 U.S. sieve, from migrating through the reinforced mass, it was recommended to install a non woven geotextile between the open graded stone backfill and the retained backfill.

Finally, compound global stability calculations were performed using the STABL program modified by RECo to model the metal reinforcing strips shearing resistance. Compound global stability calculations do not restrict the failure surface from crossing through the reinforced mass. These calculations were necessary because of the complexity of the wall and the critical nature of the building it supports. The slope at the toe of the wall affected the factor of safety for compound global stability as well. Therefore, it was prudent to provide more embedment at the bottom of the walls (between 1.00 m and 1.50 m).

2.5 *Factors of safety*

The coherent gravity method (Meyerhoff) was utilized to calculate the horizontal pressures against the wall for the internal stability of the wall. Each reinforcing strip tension was designed not to exceed 32kN for permanent conditions. The maximum allowed tension was increased by 25%, in the rare case of a high water elevation or a dam breach. Each reinforcing strip was designed to provide a minimum factor of safety of 1.50 for pullout. This factor of safety was reduced to 1.20 for the temporary condition of high water case and the unlikely case of dam breach. It was noted that the closer to the wall face the footing of the building were, the more strips per unit area were necessary in the upper most layers.

Factor of safety against sliding and overturning was computed. For sliding, a minimum of 1.50 was maintained for the permanent conditions and 1.20 for the high water case and the dam breach case. For overturning, a minimum of 2.00 was maintained for the permanent conditions and 1.50 for the high water case and the dam breach case. These conditions did not govern the design of the soil reinforcement length.

Table 2. Compound global stability results.

Case no.	D	Fore Slope	H	B	B/H	Factor of safety (a)	(b)	(c)
1	N/A	Flat	7.0	5.8	0.83	1.71	1.50	1.41
2	4.9	Flat	9.5	8.5	0.90	1.60	1.52	1.46
3	6.4	4:1	7.3	10.0	1.38	1.65	1.57	1.35
4	N/A	3:1	5.2	5.8	1.29	1.66	1.53	1.10
5	1.4	3:1	5.0	11.0	2.18	1.52	1.50	1.29

D = Distance from edge of building footing to face of wall (m)
H = Wall height from top of leveling pad to top of coping (m)
B = Reinforcement length (m)
(a) Permanent submerged conditions
(b) 0.90 m rapid drawdown from 100-year water elevation
(c) Dam breach at 100-year water elevation.

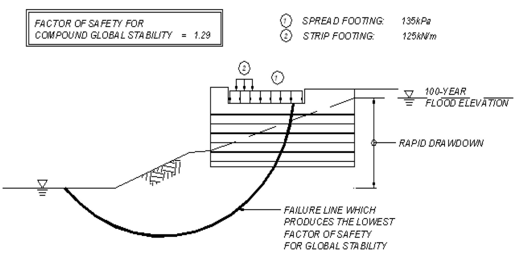

Figure 4. Example of calculated failure mode for compound global stability – Case 5, dam breach.

In the calculations of the compound global stability, the geotechnical parameters for the residual materials or the structural fill were conservative to allow for any unknowns in the subsurface. A minimum factor of safety of 1.50 was required for the permanently submerged conditions and also for the 100-year water elevation with 0.90 m of rapid drawdown. A minimum factor of safety of 1.1 was used in the highly unlikely event of a dam breach at the 100-year water elevation.

This 100-year water elevation with 0.90 m of rapid drawdown criterion governed the length of the soil reinforcement. It is noted that the reinforcement length over wall height ratio rapidly increased for a design case with a slope at the toe of the wall or when the building loads to the wall face.

Table 2 shows the compound global stability results at five locations with three hydrostatic conditions each.

Figure 4 illustrates an example of calculated failure mode for compound global stability in the event of a dam breach. In this example, it is noted that the failure line is predicted in the middle of the reinforced mass. This is due to the significant building loads being located very close from the wall face. The calculated factor of safety of 1.29 is conservative for a temporary and unlikely loading condition.

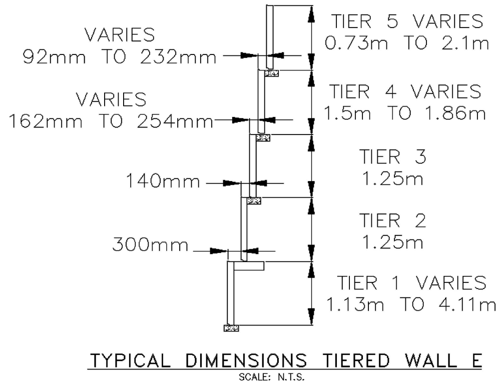

Figure 5. Typical MSE wall section.

3 AESTHETICS

3.1 *Requirements*

A crab orchard stone masonry veneer was specified to cover and embellish the precast MSE panels. The masonry veneer was specified to have a permanent batter of 1H : 8V. Tree wells were also specified close to the wall.

3.2 *Resolutions*

Attaching the veneer was a simple matter of casting dovetail insert slots into the front face of the MSE panels. Dovetail anchors were designed to attach into the slots and support the loads of the veneer.

Achieving the required batter was a more difficult task, due to MSE walls are designed to be plumb. The first proposed solution was to construct a plumb full-height MSE wall with a 0.15 m offset from the horizontal alignment of the final course of stone. This idea was dismissed because of the 1.20 m width of stone required at the bottom of a 9.50 m wall. This would mean excess veneer in excess of 1.00 m as the minimum fascia thickness was 0.15 m.

The resolution arrived at was to split the wall into tiers. The tiers were designed to minimize the stone fascia by reducing the width of stone required at the bottom of each tier. Now a 1.13 m wall height only required a 0.30 m width at the bottom (0.15 m + 1.13 m /8). Figure 5 shows the final typical section.

4 CONSTRUCTION

Construction of five almost parallel walls on reversing curves was a challenging task. It was made more difficult due to the upper two walls had a variable wall offset depending on wall height. Mr. Rod Kindoll of MC Inc. must be commended for his expert wall construction.

5 CONCLUSION

Though MSE walls are routinely used to support spread footing abutments with higher loads (192 kPa) than exerted by the building, the higher tolerances of the building come in to play. Of great concern to RECo was the potential for differential settlement of the footings on top of the reinforced mass and those of the footings behind the mass.

The use of a MSE wall supporting a multi-story building is a highly unusual application, but it is a viable one.

REFERENCE

Gathany, James (CDC/CCHIS/NCHM) Figure 1. Photograph of *The Tom Harkin Global Communications Center,* 2007.