

THE IMPORTANCE OF DRAINAGE CONTROL FOR GEOSYNTHETIC REINFORCED MECHANICALLY STABILIZED EARTH WALLS

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ABSTRACT

Mechanically stabilized earth (MSE) walls reinforced by geogrids and geotextiles have seen a tremendous growth over the past thirty years. However, along with this growth has come numerous failures consisting of excessive deformation and, in some cases, actual collapse. Of the 82-cases in the authors data base, improper drainage control was the cause in 68% of them. As a result, this paper is focused on both internal drainage issues within the reinforced soil mass within the reinforced soil mass (46%) and external drainage issues around the soil mass (22%). After a brief introduction of the technology some elements of traditional design will be presented. The issue of proper versus improper methods of drainage control will then form the core of the paper. A summary and recommendations section aimed at preventing drainage problems in the future will conclude the paper.

Key words: MSE walls, wall drainage, wall failures, internal wall drainage, external wall drainage, hydraulic pressures.

1. INTRODUCTION

1.1 Background

A monumental change from rigid concrete retaining walls to flexible mechanically stabilized walls (MSE) was made by H. Vidal in 1965. This is not to say that reinforced soil masses were not used for many years beforehand, see Hausmann (1990) and Jones (1996) for a historical review, it is just that Vidal formalized the procedure and produced products and designs to accomplish the final system. Vidal first used smooth steel strips (60 to 120 mm wide and 4 mm thick) and then changed to 50 mm wide steel strips ribbed on top and bottom. These strips were welded to galvanized sheet steel formed into half cylinders as the wall facing. Competing systems of welded wire mesh (2 to 6 W7.5 to W24) at either 150 or 200 mm spacings were introduced in the 1980's and steel gabions shortly thereafter. Note that all of these metallic inclusions (steel, and also aluminum) are presently referred to as inextensible reinforcements.

Converse to metallic inclusions are polymer geotextiles and geogrids which are sometimes referred to as extensible reinforcements. The first geotextile reinforced wrap-around wall was constructed in France in 1971 (Schlosser 1976) and the first in America by the U.S. Forest Service in 1974 (Bell and Steward 1977). Geogrids for soil reinforcement of walls were first used in England in 1978 (BE3/78). Extensive growth then occurred in Europe and America beginning about 1983. Today, geogrids are used much more frequently than geotextiles for MSE wall reinforcement. In this regard, all types of geogrids are used and the competition is intense to say the least. By groupings, current geogrids fall into the following categories:

- homogeneous (punched and drawn) HDPE and PP geogrids
- coated woven or knit PET or PA geogrids
- welded strap (or bar) PET or PP geogrids

Regarding MSE wall facings, there are many type and styles. The progression over the years is approximately as follows:

- wrap around walls
- timber faced walls
- welded wire mesh/geogrid faced walls
- gabion faced walls
- precast full-height concrete faced walls
- precast panel concrete faced walls
- modular block faced walls (also called segmental retaining walls, or SRW's)

Figures 1(a) and 1(b) show welded wire mesh/geogrid walls and modular block walls respectively. These two facing types are by far the most common MSE wall facing presently used.

Behind the above illustrated facings is, of course, horizontal layers of geotextile or geogrid reinforcement which defines the reinforced soil zone. Figure 2 shows this as Zone "2" which is typically behind a gravel drainage layer (Zone "1") and either masonry blocks (as shown) or a vegetated facing of welded wire mesh and biaxial geogrids. If the masonry blocks are hollow, the gravel also fills in the empty spaces. Behind the reinforced soil zone is the retained soil zone (Zone "3") and beneath it is the foundation soil or rock (Zone "4").

1.2 Applications

There are an estimated 40,000 MSE walls in America, of which approximately 75% are modular block faced. Note that as the front slope decreases toward the horizontal (called "wall batter" as indicated by an increasing " ω " in Fig. 2), the system eventually becomes a steep soil slope. Facings for steep soil slopes are either welded wire mesh/geogrid or geosynthetic wrap-around types as described previously.

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(a) Vegetated welded wire mesh wall with biaxial geogrid face



(b) Masonry block, also called segmental retaining walls, faced wall

Fig. 1 Most common types of MSE wall facings currently used

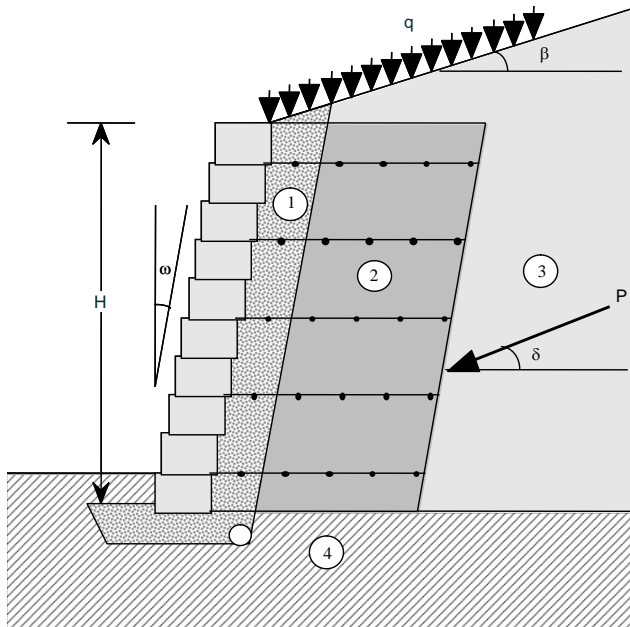
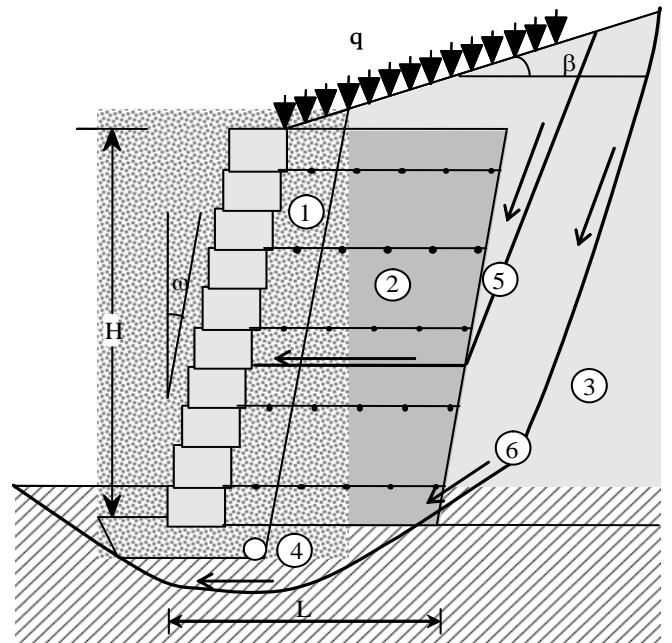


Fig. 2 MSE wall cross section showing various zones behind the facing

Within vertical or near vertical walls and steep soil slopes there are an enormous number of applications for MSE systems. Almost all sectors of civil engineering are involved. For example, within each of the following groups there are hundreds of specific applications.

- highway and transportation engineering
- geoenvironmental and landfill engineering
- hydraulic and impoundment engineering
- industrial private development
- residential private development

Interestingly, the transition of terminology from a wall to a slope is largely subjective. One discriminating value could be the traditional Rankine plane of plastic equilibrium which is at an angle of $45 + \phi/2$ from the horizontal. In the U.S., however, the Federal Highway Administration (Elias *et al.* 2001) uses a transition value of 70° . This is equivalent to a soil with a shear strength friction angle of 50° which is extremely high. That said, the issue is not particularly important since design must investigate all possible modes of failure. Figure 3 shows a cross section in which Curve “5” is a piecewise linear failure plane which has multiple possible trajectories, and Curve “6” is a global curvilinear arc which can also take many trajectories. All possibilities must be investigated.



Legend	
1	– front drainage
2	– reinforced soil
3	– retained soil
4	– foundation soil
5	– internal failure
6	– external failure

Fig. 3 Potential failure surfaces superimposed onto a MSE wall cross section

1.3 Cost

There have been several surveys of wall costs over the years, the most recent one being conducted by the Geosynthetic Institute, Koerner *et al.* (1998). The mean costs of publicly financed walls for four different types are shown in Fig. 4(a). Based on area of wall facing, it is seen that MSE walls with geosynthetic reinforcement are the least expensive of all wall types. Even further, the classical gravity type of walls are more than twice as expensive as another type. Figure 4(b) goes further in that the MSE geosynthetic wall costs of public versus private financed walls are compared to one another. Here it is seen that privately financed walls are from one-third to one-half the cost of publicly financed walls. The reason for such dramatically lower costs of privately financed walls is uncertain but several possible reasons are given in Table 1. The actual reason may be a combination of these items as well as other possibilities.

2. MSE GEOSYNTHETIC REINFORCED WALL PERFORMANCE

A recent report by Koerner and Koerner (2009) has evaluated eighty-two failures of geosynthetic reinforced MSE wall. These failures are subdivided into two categories;

- excessive deformation (23 cases), and
- total collapse (59 cases).

Examples of each category are given in Figs. 5 and 6. The major statistical findings from this study were as follows:

- (a) 100% were private walls
- (b) 80% were North American
- (c) 75% were masonry block faced (*i.e.*, SRWs)
- (d) 75% were 4 to 8 m high
- (e) 83% were geogrid reinforced (others GT)
- (f) 86% failed in less than four years
- (g) 76% used silt and clay backfill soils

- (h) only 20% had good compaction
- (i) 95% of the failures were caused by improper design or construction
- (j) 68% of the failures were caused by internal or external water

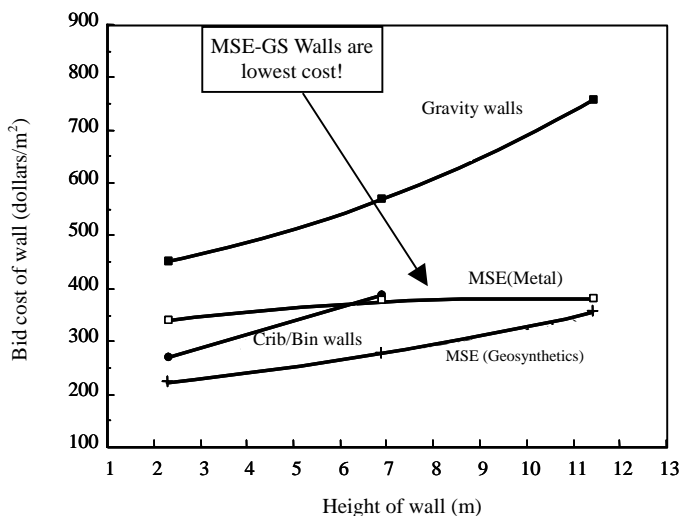
This paper has been essentially generated because of the last item which concerns the improper design and/or construction of wall drainage systems.

3. ELEMENTS OF DESIGN

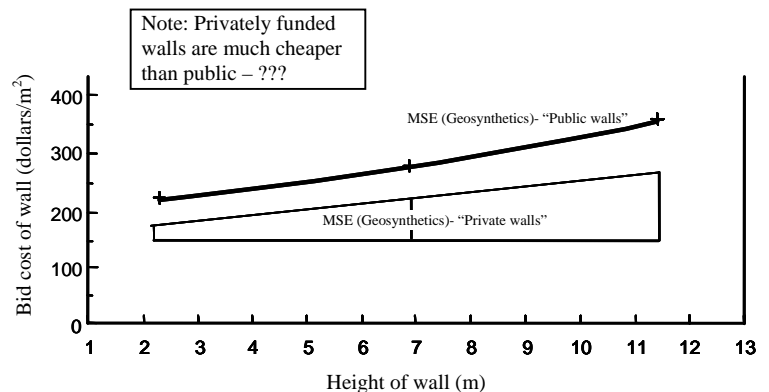
The design of MSE geosynthetic reinforced walls follows the basic principles of all MSE walls irregardless of the type of reinforcement or facing; see for example, Lee *et al.* 1973, Jones 1996, and Elias *et al.* (2001). Whatever the method used, there are three necessary design issues insofar as internal stability is concerned so as to form a coherent reinforced soil mass. They are as follows:

Table 1 Possible reasons for much lower costs of private versus public geosynthetic reinforced walls

Item	Private	Public	Comment
Design computer code	NCMA	FHWA	FHWA is a slightly more conservative code
Reinforcement length (L/H ratio)	0.5 to 0.7	≥ 0.7	Reinforcement lengths are generally longer for public walls
Soil backfill type	many fines, and med-to-high plasticity	few fines, and low plasticity	Soil backfill can represent 50% to 75% of the total wall cost
Compaction oversight	rare	common	CQA represents major implications for both wall cost and performance



(a) Mean costs of publicly funded retaining walls



(b) Mean costs of public vs. private financed MSE-geosynthetic retaining walls

Fig. 4 Mean costs of retaining walls (Koerner *et al.* 1998)



Fig. 5 Cases of excessive wall deformations (various contributors including authors)



Fig. 6 Cases of wall collapse (various contributors including authors)

- reinforcement tensile strength leading to reinforcement spacing intervals
- resistance to soil pullout leading to reinforcement lengths
- connection strength so as to assure facing connection to the reinforced soil mass

Once the coherent reinforced soil mass and facing system is designed, focus shifts to external stability where another three necessary design issues are involved. They are as follows:

- resistance to mass sliding at all possible elevations including the base layer
- overturning of the coherent mass about all possible locations including the wall toe
- bearing capacity of the foundation soil beneath the coherent mass

The above mentioned design issues can be approached on the basis of traditional geotechnical practice, *e.g.*, use of a modified Rankine analysis for lateral earth pressures (see Koerner 2005). Alternatively, there are two computer codes developed specifically for design and analysis of MSE reinforced walls. These are available from ADAMA Engineering, Inc. and from NCMA (2009). A generalized comparison of the three approaches toward design is given in Table 2. In order to numeri-

cally compare these three design approaches to one another a 7.0 m high retaining wall with a 15 kN/m² surcharge was designed and the numeric comparison shown in Table 3 was generated.

The assumption in the numeric comparisons in Table 3 is that the FHWA code (ADAMA, Inc.) results in the appropriate factor-of-safety values, *i.e.*, it is assigned 100% of each design element. By comparison in this analysis mode of investigation, the modified Rankine approach is the most conservative (highest values), and the NCMA computer code is the least conservative (lowest values). This is not to say that the NCMA code is incorrect, it is only that the inherent assumptions are more favorable so as to generate lower FS-values.

4. DRAINAGE CONTROL FOR MSE WALLS

In the previous design section and illustration for MSE geosynthetic reinforced walls, the tacit assumption was made that hydrostatic pressures were nonexistent. Stated differently, the usual MSE wall system designs are assumed to contain “free-drainage” components such that water will be readily discharged around and/or through the reinforced soil mass and the wall facing. This is indeed to be expected but only if free-draining sands and gravels are used in the reinforced soil zone. As found by

Table 2 Design methods in common use

Item	mod.-Rankine	FHWA	NCMA
K_a – basis	Rankine	Coulomb	Coulomb
K_p – basis	neglect	neglect	neglect
Earth pressure angle	horizontal	inclined	inclined
Surcharge	applicable	applicable	applicable
Wall batter	not applicable	applicable	applicable
Inclined backslope	not applicable	applicable	applicable

Table 3 Comparison of example problem results “assuming that FHWA has it right at 100%”!

Design issues	Mod. Rankine	FHWA	NCMA
“Internal stability”			
Tensile overstress	99%	100%	98%
Soil pullout	123	100	85
Facing connection	n/a	100	83
“External stability”			
Mass sliding	111%	100%	80%
Bearing capacity	100	100	67
Overturning	100	n/a	70

Thus: mod. Rankine = most conservative
 FHWA = intermediate values
 NCMA = least conservative

Koerner and Koerner (2009), however, this is clearly not the case. In 62 of the 82 wall failures (76%) silts and/or clays were used in the reinforced soil zones and, even further, only 16 of these (20%) had good compaction control. Clearly, these situations were not “free draining” and hydrostatic pressures should have been considered in the design stage when using such low permeability soils. This comment is not meant to imply that silts and/or

clays cannot be used in the reinforced soil zone. It is meant to imply that if these poorly draining soils are used, they must be used with proper drainage components which will be described in the next section. Of course, if sands and/or gravels are used throughout the reinforced soil system then such drainage controls are essentially not necessary.

This section of the paper addresses five specific drainage control situations which must be considered in the design of MSE walls with geosynthetic reinforcement when using silt and/or clay soil backfills in the reinforced soil zone. These five situations were associated with the root cause of 56 of the 82 failures investigated, *i.e.*, 68% of them.

4.1 High Phreatic Surface

Whenever MSE walls are built adjacent to, or even near to, standing or flowing water concern over a rising water surface must be considered. Obviously, streams and rivers are the most suspect but other situations are also possible. A flood is defined as a temporary condition of partial or complete inundation of water. This inundation may include the overflow of inland or tidal waters, rapid accumulation of runoff, mudflow, or the collapse of land along a shore due to water that has exceeded anticipated cyclical levels. Whether the flooding at a wall results from storm surge, riverine flooding, or urban flooding, the physical forces of the floodwaters which act on the structure are generally divided into three load cases. These load cases are hydrostatic loads, hydrodynamic loads, and impact loads. These load cases can often be exacerbated by the effects of water scouring soil from around and below the foundation.

The hydrostatic loads are both lateral (pressures) and vertical (buoyant) in nature. The lateral forces result from differences in interior and exterior water surface elevations. As the floodwaters rise, the higher water on the exterior of the wall acts inward. Similarly though less common, a rapid drawdown of exterior floodwaters may result in outward pressures on the wall as the retained indoor floodwater tried to escape. Sufficient lateral pressures may cause permanent deflections or collapse to the wall as shown in Fig. 7. The lateral pressures associated with the saturated soils can possibly exceed the capacity of the wall or liquefy the soil behind or beneath the wall.

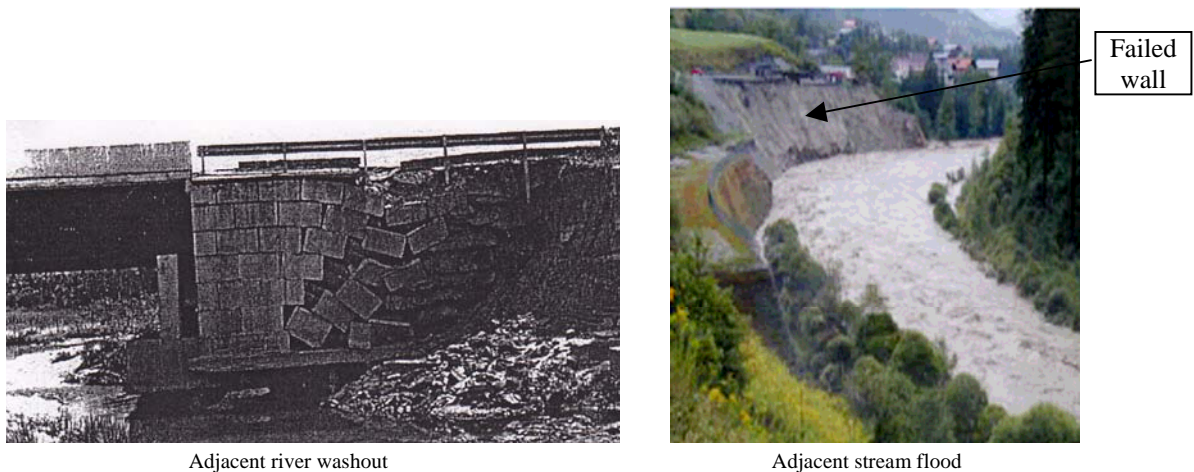


Fig 7 Wall collapse when adjacent to streams and rivers (various contributors including authors)

The buoyant forces are the vertical uplift of the wall due to the displacement of water. These uplift forces may be due to air within the soil matrix not being able to escape quickly enough. When the buoyant forces associated with the flood exceed the weight of the wall components, it may “float.” Occurrences of this situation are rare but possible when soil is encapsulated, sealed let down structures are specified and or lightweight fill strategies are utilized.

In addition to these hydrostatic loads, the water flowing around the wall during a flood event creates hydrodynamic loads on the structure. These loads are the frontal impact loads from the upstream flow, the drag on the sides of the wall, and the suction on the rear face of the wall as the floodwaters flow around the structure. The magnitude of the hydrodynamic loads is dependent upon the velocity of the floodwaters and the shape of the wall. Like the hydrostatic pressures discussed earlier, these lateral pressures associated with the flowing water may be capable of collapsing the wall or shift it from its foundation.

Further exacerbating the physical forces applied directly to the wall, rapidly flowing water may also scour the soils which supports the wall. While the rate and ease with which a soil will scour depends upon many factors, it must be stable over time to support the wall. If the soils are eroded from around or below a wall’s foundation system, the capacity of the wall is reduced. Ultimately this may lead to a shifting of the wall, a partial collapse of the system, or even a complete collapse of the structure.

Impact loads during flood events may be the direct forces associated with waves, as typically encountered during coastal flooding, or the impact of floating debris within the floodwaters. Impact loads can be especially destructive because the forces associated with them may be an order of magnitude higher than the hydrostatic and hydrodynamic forces during the flood event. In addition, as debris travels downstream during a flood event, it exerts impact loads on walls it may encounter.

The solution to such situations requires a base drain beneath the entire reinforced soil zone and full wall length to the discharge outlets. Furthermore, if the water level exceeds the top of the base drain, then all of the backfill in the reinforced soil zone must be free draining up the maximum elevation. Depending on the rapidity of water level rise one would use either sand or

gravel. Clearly, this is not a situation for fine-grained soil backfills. It is also a situation where soil encapsulation and light weight fill should not be used. Furthermore the facing elements of the wall must be extremely robust.

4.2 Retained Soil Drainage

Groundwater drainage from the retained soil zone can be remarkably large and if hindered by a low permeability backfill soil will cause the mobilization of hydrostatic pressures. This is particularly a concern in cut-sections as the photographs of Fig. 8 indicate.

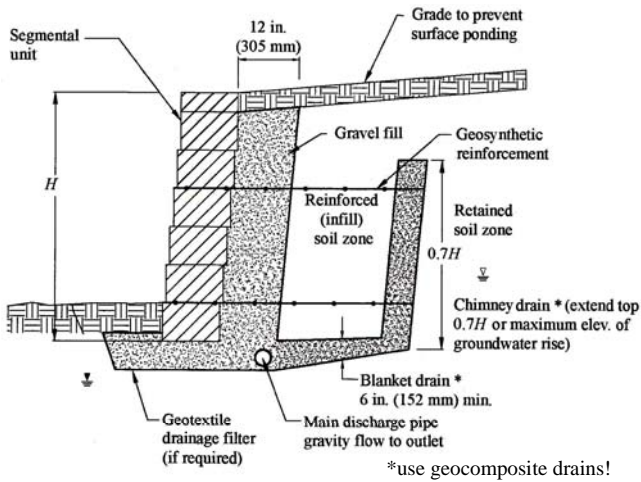
Obviously, the possibility of this occurrence must be investigated by upgradient water observation wells. If such water is identified or even anticipated, it must be intercepted by a back drain between the reinforced and retained soil zones as shown in Fig. 9(a). Note that it forms a vertical continuation of the base drain recommended in the previous section. Due to the difficulty in constructing vertical layers of soil, however, this back drain will invariably be a geocomposite drain of which many are available (Koerner and Soong 2000 and 2005). Figure 9(b) shows the geocomposite alternative with pipe outlets. The geocomposite back drain alternative is shown in Fig. 9(c) as being continuous or intermittent in their coverage. The latter are called “chimney drains”.

4.3 Drainage from Paved Surfaces and Adjacent Structures

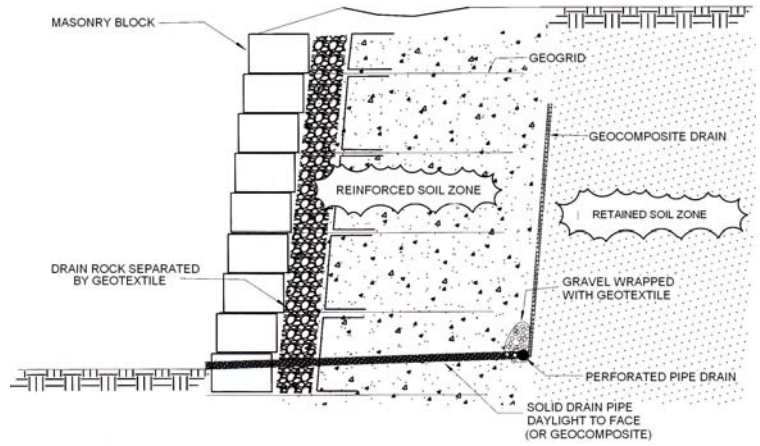
The customary reason for constructing a wall is to gain horizontal space along the upper surface. Most commonly, space is required for parking, roadways, storage areas, and homes and buildings. By so doing, rainwater and snowmelt coming from these surfaces must be properly accommodated. The accumulated flow is commonly oriented toward the wall and to prevent it from flowing over the top of the wall, it is collected in a catch basin, inlet, or manhole located within the reinforced soil zone. There the water is often transmitted parallel to the wall until it can be daylighted at lower elevations. Figure 10 shows common situations in this regard.



Fig. 8 Groundwater exiting from retained soil zone in cut-situation (authors)



(a) Back drain using sand (Elias *et al.* 2001)



(b) Back drain using drainage geocomposite (compl., TenCate Geosynthetic, Inc.)



(c) Use of continuous and intermittent geocomposite back drains when using fine grained soils in the reinforced soil zone (compl., TenCate Geosynthetics, Inc.)

Fig. 9 Various approaches to providing back drainage behind MSE walls

Whenever, the backfill soil in the reinforced zone consists of fine-grained silts and/or clays, it is felt to be a very dangerous practice to bring the surface water into the reinforced soil zone.

Furthermore, if compaction control and inspection is not rigorous, such design should be forbidden. Settlement of the drainage system, exacerbated by outward deformation of the wall itself, commonly leads to leakage and even pipe breakage as shown in the photographs of Fig. 10.

The solution to this situation is shown in Fig. 11. Here is seen that the wall itself has to be made sufficiently high so that drainage flows away from the face of the wall to the end of the reinforced soil zone. At this location, the inlet and pipe transmission system is constructed. Thus the reinforcement is not interrupted in any way. Furthermore, if leakage occurs at this location, it can be accumulated and transmitted into the back drain and eventually out of the system via the base drain.

4.4 Waterproofing Backfilled Surface

When using low permeability soils (silts and/or clays) in the reinforced zone, water and snowmelt will often accumulate on the ground surface. While such water should be directed to the front or back drainage layer, it often infiltrates into the backfill soil. Here it can generate hydrostatic pressure against the wall facing causing excessive deformation or actual collapse. Needed in this regard is a geomembrane covering the surface as shown in Fig. 12.

4.5 Tension Crack Sealing

Tension cracks commonly occur in the backfilled soil exactly at the end of the reinforcement particularly when dealing with silt and/or clay backfills, see Fig. 13. This occurs primarily due to volume decrease of the reinforced soil mass, but also due to the outward deformation of the wall facing.

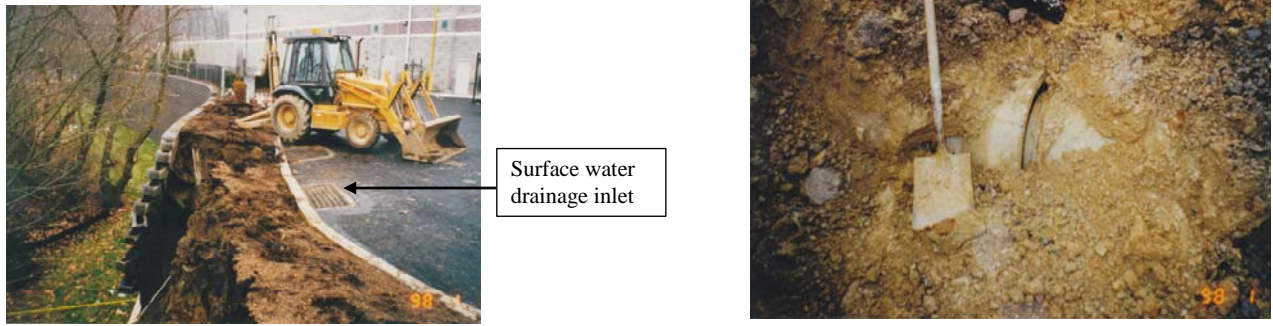


Fig. 10 Various internal drainage failures (various contributors including authors)

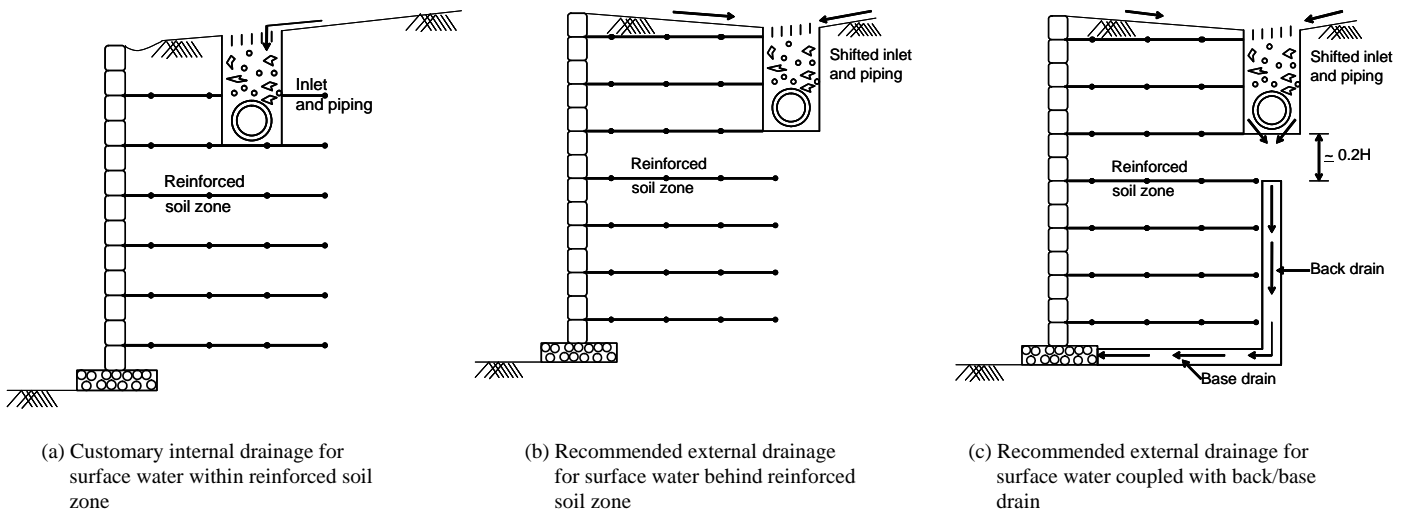


Fig. 11 Recommended backgrading from wall face and shifting of internal drainage systems from within to behind the reinforced soil zone

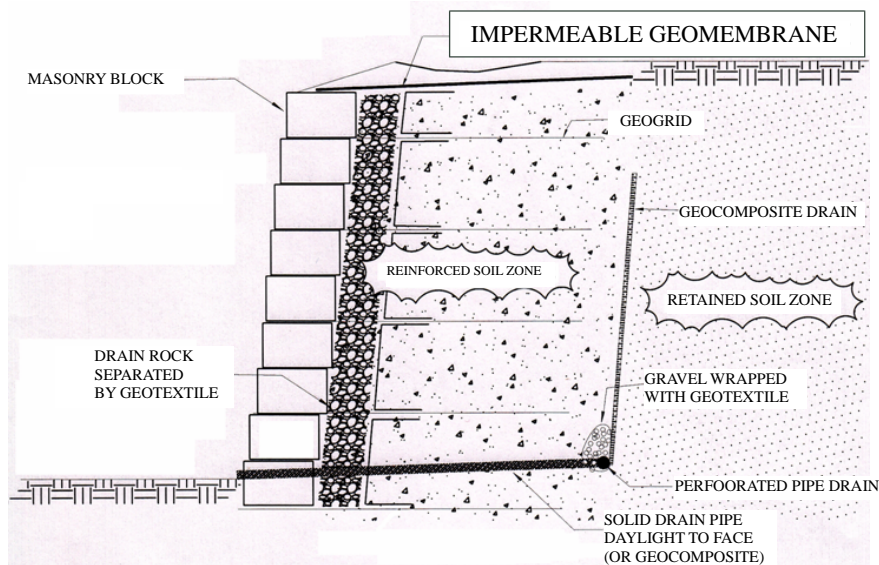


Fig. 12 Use of a geomembrane waterproofing layer above the reinforced soil zone



Fig. 13 Tension cracks occurring exactly at the end of the wall reinforcement (authors)

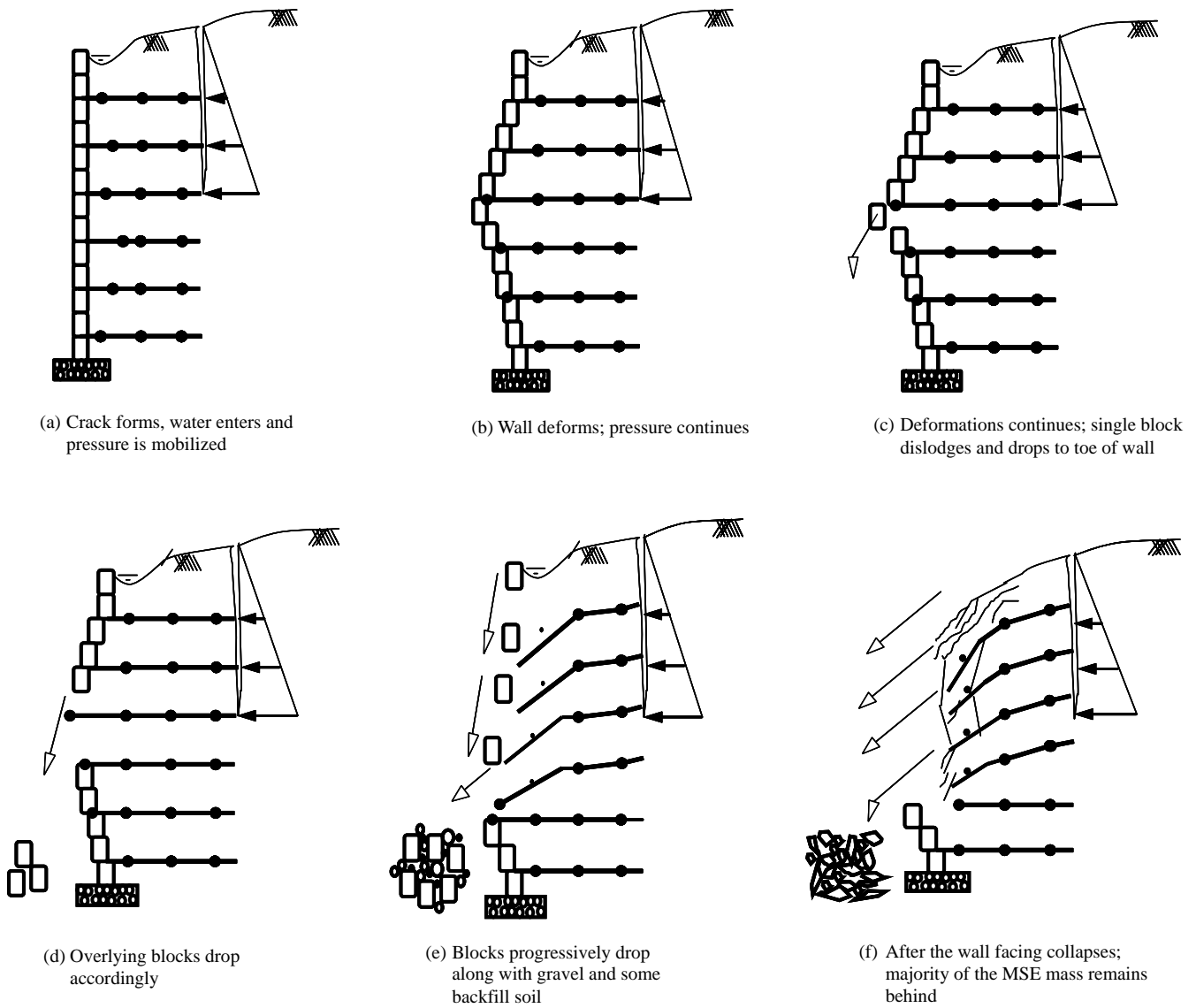


Fig. 14 Modular block wall collapse progression due to hydrostatic pressure in tension cracks

These ground surface cracks readily fill with water thereby exerting hydrostatic pressure against the reinforced soil mass. With silts and/or clay backfills, the permeability is so low that the reinforced soil mass is translated, *i.e.*, actually moved outward along with the facing system. As movement progresses, a set of masonry blocks eventually falls off of its supporting layer. A cascading effect of higher rows of blocks sequentially fails until remnants of the reinforced soil mass is all that remains. The multiple-stage process is illustrated in Fig. 14.

The solution of this type of external drainage issue is to extend the geomembrane waterproofing beyond the reinforced soil zone and onto the retained soil zone. Of course, the selection of the geomembrane is very important and extensibility, flexibility, and durability are all important factors in the selection process, Koerner (2005) and others.

5. SUMMARY AND RECOMMENDATIONS

This paper has focused on the importance of considering drainage control at mechanically stabilized earth (MSE) walls with geosynthetic reinforcement when using silt and/or clay backfill soils. The incidence of failures (both excessive deformation and actual collapse) is alarming and in 68% of the 82-failures cases evaluated improper drainage control was the cause. The hydrostatic pressure arising from improper drainage control can be either internal or external with respect to the reinforced soil mass. Internal drainage failures are caused by the following:

- perched water pressure within the soil mass
- leakage from catch basins, inlets, and manholes located in reinforced soil zone
- leakage from transmission pipes located in the reinforced soil zone

External drainage failures are caused by the following:

- pressures from elevated phreatic surface in groundwater or surface water
- pressures from water exiting the retained soil zone
- pressure from water in tension cracks behind the reinforced soil zone

The paper addressed each of the above groupings and made the following recommendations.

Base drainage using granular soils (sand and/or gravels) must extend beneath the reinforced soil zones for cases where high water in adjacent streams and rivers are anticipated. Furthermore, this base drain must be extended upward in the form of a back drain using granular soils or geocomposite drains for cases where water is emerging from the retained zone.

When using fine-grained soil backfills, such as silts and/or clays, it is critical to keep surface water drainage out of the reinforced soil zone. The required catch basins, inlets, manholes, and transmission pipes must be located behind the reinforced soil zone. In turn, this requires the ground surface of the reinforced soil zone to be back-graded away from the face of the wall.

Even further, the upper surface of the wall must be waterproofed with a suitable geomembrane. The geomembrane must

cover the entire reinforced soil zone and even extend onto an adequate portion of the retained soil zone. This will prevent surface water from entering the reinforced soil zone as well as water in tension cracks that might be formed behind it. The geomembrane must be designed for its extensibility, flexibility, and durability.

Thus it is seen that the entire reinforced soil zone must be encapsulated by waterproofing from above the drainage from beneath and behind wherever the backfill consists of fine-grained soils. These precautions are felt to be absolutely necessary to prevent wall drainage failures from occurring in the future.

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