DESIGN OF EXTERNAL DRAINAGE SYSTEMS FOR SEGMENTAL RETAINING WALLS

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ABSTRACT: This paper on geosynthetic reinforced segmental retaining walls (SRW's) focuses on drainage pressures, quantification and design beneath and behind the reinforced soil zone of SRW's. This is an important consideration because the majority of problems with SRW's (in 20 out of 26 case histories) were brought about by low permeability backfill soils in the reinforced zone. Without proper drainage, hydrostatic pressures can result in forces which can deform or even fail the entire wall system. A design method based on finite differences is presented. It results in the determination of the seepage pressures, as well as quantification of the required flow rate for a given set of conditions. When this value is compared to the allowable flow rate for a candidate drainage product, a flow rate factor of safety is obtained. The entire design process is numerically illustrated for ten different geosynthetic drainage materials. The conclusion reached is that if low permeability soils are used for backfill in the reinforced soil zone, adequate drainage behind the zone must be provided. There are numerous geosynthetic drainage materials that are readily designable and available. While there certainly will be an increase in cost, the increase is not felt to be prohibitive in light of the currently low cost of SRW's. Conversely, the cost of a failure is certainly prohibitive and must be avoided.

1 INTRODUCTION

The advent of mechanically stabilized wall systems with geosynthetic reinforcement has ushered into common use a particular type of retaining wall known as segmental retaining walls (SRWs). Also called *modular block walls*, these systems are dry stacked masonry blocks generally requiring geosynthetic reinforcement between individual block layers. Usually geogrids, but also geotextiles and geostraps, have been used.

Several instances of excessive deformation and actual collapse, however, have been reported. The largest subset of these problems is the situation where water enters behind the soil of the reinforced zone and exerts hydrostatic pressure. The basic problem is the use of low permeability backfill soil in the reinforced zone and its accompanying lack of drainage capability.

The focus of this paper is as follows; (i) the calculation of seepage pressures/forces, (ii) methods to avoid these seepage pressures/forces from occurring, (iii) design of geosynthetic drainage systems, and (iv) illustration of the entire process by means of a numeric example.

2 BACKGROUND

This background section is an abbreviated form of an earlier paper by the authors which presented an overview of SRWs and their field performance, Koerner and Soong (1999). It should be mentioned that there are many complimentary papers in the open literature. In this regard, see Bathurst and Simac (1994), Leschinsky (1977), Meyers, et al. (1998), and others.

2.1 *Aesthetics*

The aesthetics of SRWs can only be described as being outstanding. In addition to the texture and color uniqueness of the facing, SRWs have successfully resulted in walls up to 38 m in height, under high surcharge loads, and in seismically active areas.

2.2 *Cost*

A survey of retaining wall costs was conducted for retaining walls in the USA, Koerner, J. et al. (1998). Geosynthetic reinforced MSE walls are the least expensive of all wall types and for all wall heights.

2.3 *Design*

The design of SRWs utilizes a typical cross section which includes a thin gravel layer which separates the blocks from the reinforced soil zone. The reinforced soil zone follows which contains the geosynthetic reinforcement layers and constitutes the bulk of the mechanically stabilized earth zone. The retained soil zone which is either the in-situ soil (in cut situations), or locally available backfill soil (in fill situations), is the final zone.

Design, per se, consists of separate calculations which are common to most MSE retaining wall situations. Both external and internal stability issues must be assessed. SRWs are designable by a number of credible computer codes. Implicit in all of these designs is that there are no seepage pressures acting on the system. As will be seen in the next section, this is a poor assumption when dealing with low permeability backfill soils in the reinforced zone.

2.4 *Field Performance*

The vast majority of SRWs have been successful insofar as their performance is concerned. Yet, there have been cases of unacceptable field performance of SRWs with respect to both serviceability (excessive deformation) and failure (actual collapse), Koerner and Soong (1999).

In the serviceability case histories, large scale excessive deformation can be at the top, bottom or throughout the wall, and can even be rotational. Within the group of seven design related case histories, six had fine-grained backfill soils in the reinforced zone and only one had granular soils (high surcharge problem) throughout.

In the failure (or collapse) case histories, hydrostatic pressure arising from lack of drainage from fine-grained soil backfill in the reinforced zone was the overriding reason for the failures. This occurred in 10 of the 14 case histories evaluated.

Interestingly, in 17 of the 26-case histories that were evaluated the problems were associated with fine-grained backfill in the reinforced soil zone.

2.5 *Focus of Paper*

The concern over low permeability backfill soil in the reinforced zone is a major issue leading to the overwhelming proportion of SRW problems. Obviously, the use of low permeability backfill soil contributes greatly toward an inexpensive wall system, but it can also lead to buildup of hydrostatic pressures leading to excessive deformations and/or failure. Thus, the situation must be addressed.

3 DETERMINATION OF SEEPAGE PRESSURE AND/OR SEEPAGE FORCE

This section presents various seepage pressure scenarios on low permeability reinforced zone backfill soils when adequate drainage is not available.

3.1 *Significance*

It is readily shown in all geotechnical engineering texts that poorly drained backfill soil behind a retaining wall with full hydrostatic head results in approximately twice the lateral earth pressure/force than a similar wall with proper drainage. With reinforced SRW backfilled with low permeability soils, the necessary drainage zone moves to the back of the reinforcement zone. The water coming from the retained zone, plus any water coming from the ground surface or artesian conditions in the foundation soil or rock, must be collected and removed. Focus in this section is on water coming from the retained soil zone.

3.2 *Fundamentals*

Under standard assumptions it can then be shown (Cedergren, 1989) that the quantity of water entering an element of soil must equal that leaving, and the equation of continuity takes the following form:

$$\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}=0 \qquad (1)$$

The terms, u, v and w are discharge velocity components in the, x, y, and z directions, in cartesian coordinates.

According to Darcy's equation (v_d = ki), the components of the discharge velocity are the following:

$$u=-k\frac{\partial h}{\partial x},\quad v=-k\frac{\partial h}{\partial y},\quad w=-k\frac{\partial h}{\partial z} \qquad (2)$$

Substituting the above in Eq. 1 we obtain the following:

$$\partial\frac{-k(\partial h/\partial x)}{\partial x}+\partial\frac{-k(\partial h/\partial y)}{\partial y}+\partial\frac{-k(\partial h/\partial z)}{\partial z}=0 \qquad (3)$$

If the soil mass is isotropic in its permeability, i.e., if $k_x = k_y = k_z = k$, the value "k" can be factored out of the left side of the equation and cancelled, resulting in the following equation:

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}+\frac{\partial^2 h}{\partial z^2}=0 \qquad (4)$$

The above equation is the common form of Laplace's equation for three dimensional flow of water through isotropic homogeneous soils. In two dimensions, the equation has the form:

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}=0 \qquad (5)$$

If the soil mass is anisotropic in its permeability, i.e., if $k_x \neq k_y \neq k_z$, the equations become somewhat unwieldy. They result in the following equations in three dimensional and two dimensional forms, respectively.

$$k_x\frac{\partial^2 h}{\partial x^2}+k_y\frac{\partial^2 h}{\partial y^2}+k_z\frac{\partial^2 h}{\partial z^2}=0 \qquad (6)$$

and

$$k_x\frac{\partial^2 h}{\partial x^2}+k_Y\frac{\partial^2 h}{\partial y^2}=0 \qquad (7)$$

The above equations can be solved by one of the following methods:

- Analytic solutions which are precise but greatly complicated when boundary conditions are not simple.
- A graphical method using the technique known as flow net construction which is, at best, approximate.
- Computer methods based on either finite differences or finite elements which are the way of the future.

This paper uses the finite difference technique to develop computer spread sheets as presented by Bardet (1997).

3.3 *Finite Difference Method*

The finite difference method is a numerical approach for solving partial differential equations such as those governing steady-state seepage flow in soils. As shown in Figure 1, the two dimensional space behind a retaining wall (the back of the wall being represented by the y-axis) is discretized with a grid of points (called nodes or nodal points); the coordinates of which are denoted by i and j.

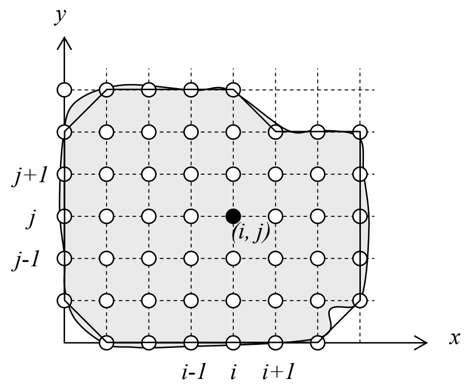


Figure 1 Discrete representation of a two dimensional region

If Δx and Δy are the nodal intervals in the x and y directions, respectively, the discretized form of Equation (7) at point (i,j) can be written as:

$$\frac{k_x}{\Delta x^2}\left(h_{i+1,j}+h_{i-1,j}-2h_{i,j}\right)+\frac{k_y}{\Delta y^2}\left(h_{i,j+1}+h_{i,j-1}-2h_{i,j}\right)=0 \qquad (8)$$

As shown in Eq. (8) and Figure 1, only the values of the head in the immediate vicinity of the selected node "i,j" contribute to the solution of the head of the selected node.

When $\Delta x = \Delta y$, Eq. (8) becomes the following:

$$h_{i,j} = \frac{1}{2(1+\alpha)}\left(\alpha h_{i+1,j} + \alpha h_{i-1,j} + h_{i,j+1} + h_{i,j-1}\right) \qquad (9)$$

where $\alpha = k_x/k_y$. Furthermore, when $\Delta x = \Delta y$ and $k_x = k_y$ (i.e., $\alpha = 1$), Eq. (8) reduces to the following:

$$h_{i,j} = \frac{1}{4}\left(h_{i+1,j} + h_{i-1,j} + h_{i,j+1} + h_{i,j-1}\right) \qquad (10)$$

Eqs. (9) and (10), along with the prescribed boundary conditions, are used to determine the total head distribution over the considered domain.

Figure 2 presents a seepage problem associated with a reinforced SRW. The reinforced zone soil is assumed to be of low permeability soil and is saturated with a water level maintained stationary at the ground surface, i.e., a worst case assumption. The retained zone soil is also low permeability and assumed to be of the same permeability as the reinforced zone soil. The soil mass of consideration (i.e., the influence domain of calculation) extends twice the wall height behind the wall drain. .

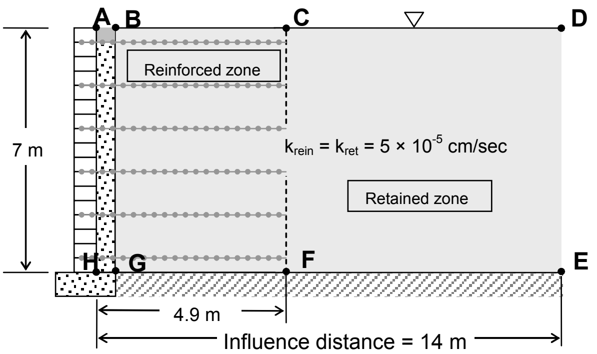


Figure 2 Node identification for example seepage problem

The entire soil mass, including both reinforced and retained soil zones, is divided into 10 divisions in height and 20 division in width. Thus, the domain consists of 231 (11×21) nodal points regardless of the actual wall height.

The spreadsheet program of MS Excel 97® was used in this study. The iterative process is initiated by choosing Calculations in the Options box of the Tools menu. Excel® has a two dimensional capability to represent the distribution of the calculated total head values. Figure 3 shows the results of a two-dimensional contour plot for the example problem. Similar results can be used to determine seepage pressure distributions and the corresponding seepage force along any line (or curve) of interest behind the wall for either external or internal stability considerations.

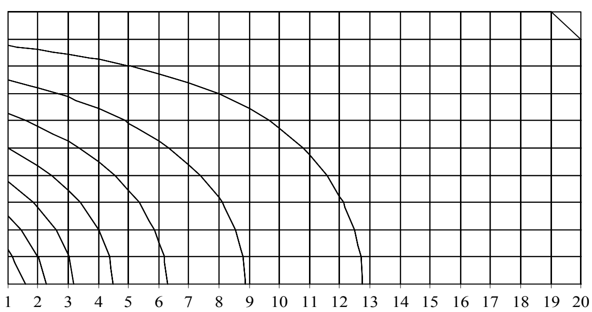


Figure 3 Total head distribution for the example problem

3.4 *Selected Seepage Studies*

The finite difference procedure described in the previous section allows for innumerable studies to be conducted.

Using the wall cross section shown in Figure 2, soil and seepage pressures can be generated for various wall geometries and soil permeabilities. The resulting values are then integrated over the wall height to result in a seepage force "P_w". Thus, the relative influence of the seepage force to the soil force can be assessed.

Such studies are available in Soong and Koerner (1999) for the following situations:

- effect of backslope inclination
- effect of surface cracking
- effect of different soil permeability ratios
- effect of anisotropic permeability
- effect of tapered reinforcement length
- effect of reinforcement inclination

4 DRAINAGE METHOD TO AVOID SEEPAGE PRESSURES/FORCES

The recommended strategy for SRWs is to provide for proper drainage of the seepage arriving at the back of the reinfored soil zone for avoiding seepage pressures. This strategy is developed in this section insofar as behavior is concerned, and in the next section as far as a flow rate design is concerned. There are many design schemes that are available.

4.1 *Combined Base Drain and Back Drain (External)*

By far the simplest method for collecting and transmitting seepage is to extend the SRW footing (which is usually gravel) in the form of a full base drain as shown in Figure 4. Instead of gravel, the base drain can also be a geocomposite drain. The drainage core will be either a geonet or other configuration (cuspations, column, flutes, 3-D meshes, etc.). Whatever its configuration, it must be protected with geotextiles on the upper surface and usually on the lower surface as well.

This base drain is then extended upward against the retained soil slope in the form of a back (or chimney) drain. This back drain is almost always a geocomposite drain due to its ease of placement in a near vertical orientation. In fact, this back drain can be used by itself with pipe outlets thereby replacing the gravel or geocomposite base drain.

The finite difference analysis presented earlier has been utilized to illustrate the effectiveness of a base drain combined with a back drain of different heights, "h".

Shown by the equipotential lines in Figure 4 (left side) is that the base drain draws the equipotentials to the back of its length. With a back drain (right side) they are raised to the extent of the back drain. This truncates the seepage pressure diagram significantly and greatly reduces the seepage pressure.

The nondimensionalized seepage force curves, in terms of $P_w/(\gamma_w H^2)$, are also presented in Figure 4 for varying back drain heights "h". Also varied was the permeability ratio, "k_{rein}/k_{ret}". Regarding the height of the back drain, it is seen that the seepage force decreases steadily with increasing height "h", At a full height of h = 1.0 H, the seepage force is zero. Regarding the permeability ratio, it is seen that increasing the permeability of the reinforced zone soil, hence increasing k_{rein}/k_{ret} ratio, decreases the seepage force in a nonlinear manner. The major decrease is in the range of k_{rein}/k_{ret} of 0.1 to 10.

Most important to note in the curves of Figure 4, however, is that the seepage force is zero under all conditions of permeability for a full width base and a full height back drain. It is precisely the desired effect in utilizing a back drain with outlet capability.

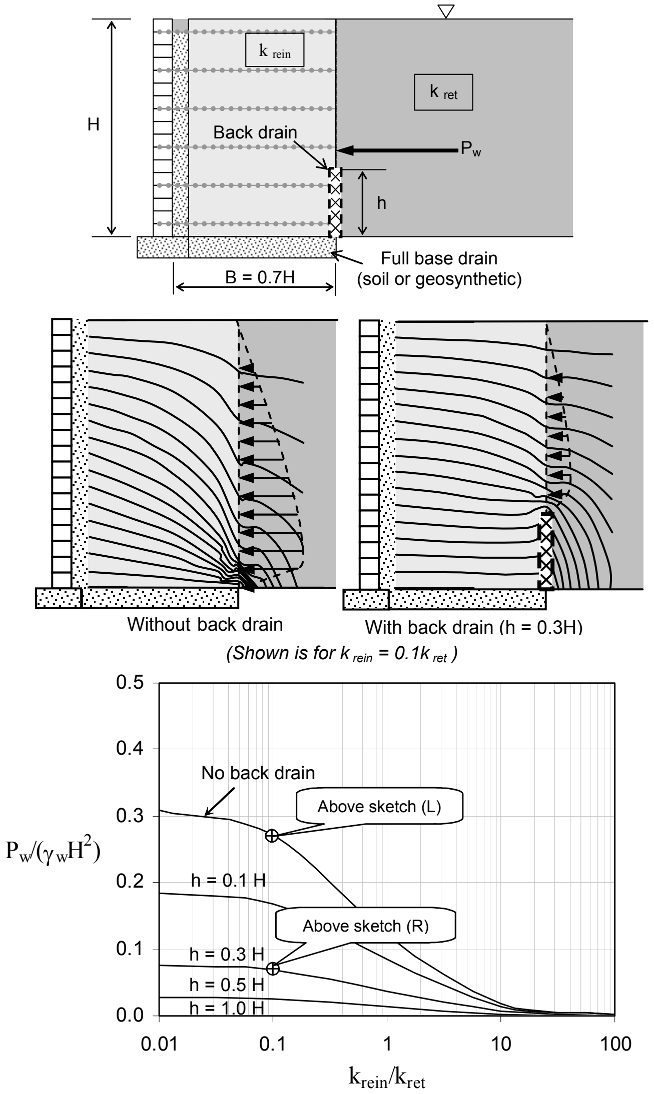


Figure 4 Effect of base drain and vertical back drain on seepage force

4.2 *Drainage* ***Within*** *the Reinforced Zone (Internal)*

It is certainly possible to transmit water coming to the back of the reinforced zone by means of suitable drainage within it. This is best accomplished by layers of a thick needle-punched nonwoven geotextile or a drainage geocomposite, see Koerner and Soong (1999) for this drainage strategy.

5 FLOW RATE FACTOR-OF-SAFETY-DESIGN FOR SRW DRAINAGE SYSTEMS

The candidate materials for collecting and transmitting site specific seepage must now be designed with respect to the required flow rate or transmissivity. Emphasis will be on geocomposite drains, but the procedure is exactly the same for sand or gravel drainage layers.

5.1 *Flow Rate Factor-of-Safety Concept*

The essence of design-by-function is to formulate a factor-of-safety (FS) value. For flow rate, or transmissivity, the equivalent equations are as follows:

$$FS = \frac{q_{allow}}{q_{reqd}} = \frac{\theta_{allow}}{\theta_{reqd}} \tag{11}$$

where

FS = factor-of-safety
q_{allow} = allowable flow rate
q_{reqd} = required flow rate
θ_{allow} = allowable transmissivity
θ_{reqd} = required transmissivity

The relationship between "q" and "θ" is as follows:

$$\frac{q}{W} = i\theta \tag{12}$$

where
q = flow rate (m^3/s)
W = unit width (m)
i = hydraulic gradient (dimensionless)
θ = transmissivity (m^2/s)

5.2 *Allowable Flow Rate on Transmissivity*

Various permeability tests are utilized for the testing of drainage soils wherein isotropic flow is usually assumed. For granular soils (sand or gravels), the constant head permeability test is utilized. Procedures are covered in all experimental soil mechanics texts.

For geocomposite drains, the test to be utilized is ISO 12958 or ASTM D4716. It is a relatively large (300 mm × 300 mm) in-plane flow test. Flow is maintained at a constant head and the test results in either flow rate per unit width (q/W) or transmissivity (θ). The subsequent value is obviously product specific and depends on normal stress and hydraulic gradient. Figure 5 presents ultimate flow rates for an array of geosynthetic material responses.

Regarding the allowable value of flow rate, the laboratory test configuration must be viewed in light of the actual situation. If, as is usually the case, the laboratory test is not modeled completely, reduction factors must be applied. One way of accomplishing this adjustment is to ascribe reduction factors on each of the items not directly simulated in the laboratory test. The usual approach is as follows:

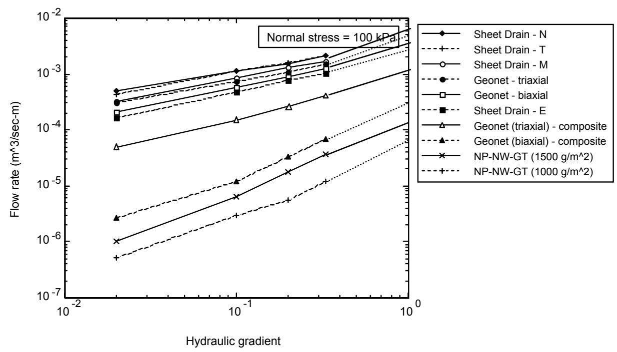


Figure 5 Index flow rate behaviour of various geosynthetic drainage materials (after Koerner, 1998)

$$q_{allow} = q_{ult}\left[\frac{1}{RF_{IN} \times RF_{CR} \times RF_{CC} \times RF_{BC}}\right] = \frac{q_{ult}}{\Pi RF} \qquad (13)$$

where

q_{ult} = ultimate (or maximum) flow rate from short-term tests between solid platens,

q_{allow} = allowable flow rate to be used in Eq. 11 for final design purposes,

RF_{IN} = reduction factor for elastic deformation, or intrusion, of the attached geotextiles into the geocomposite's core space,

RF_{CR} = reduction factor for chemical clogging and/or precipitation of chemicals in the geocomposite core space, and

RF_{BC} = reduction factor for biological clogging in the geocomposite's core space.

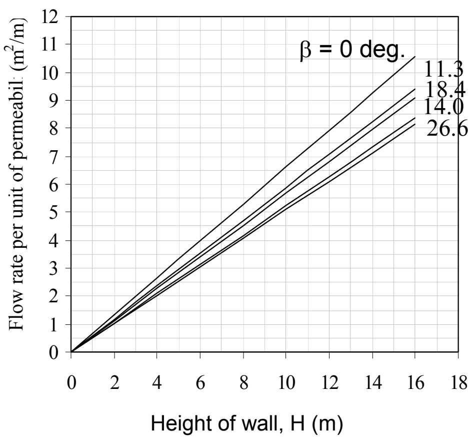
Other reduction factors, such as installation damage, temperature effects, and liquid turbidity, could also be included. If needed, they can be included on a site-specific basis. On the other hand, if the actual laboratory test procedure has included a particular item, it would appear in the above formulation as a value of unity.

5.3 *Required Flow Rate or Transmissivity*

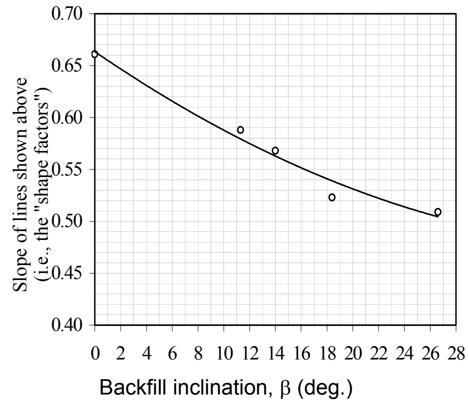
To obtain the required flow rate or transmissivity coming to the drainage system from the retained soil zone, the seepage pressures from Section 3 are utilized. The following finite difference equation is used from which the flow rate is obtained directly, see Bardet (1997).

$$q_{reqd} = \frac{k}{4}\left[h_{i+1,m} - h_{i-1,m} + 2\sum_{j=m+1}^{n-1}\left(h_{i+1,j} - hi_{i-1,j}\right) + h_{i+1,n} - h_{i-l,n}\right]$$

This equation was used to develop required flow rate values as a function of the following variables: retained soil zone permeability (k), height of wall (H), backfill inclination behind the wall (β), and shape factor (F/N). This information is presented in Figure 6 in the form of two alternative design graphs. Figure 6(a) presents ordinate values which must be multiplied by the permeability of the retained soil zone (in units of m/sec) to obtain "q_{reqd}". Figure 6(b) presents ordinate values which must be multiplied by both the permeability (in units of m/sec) and height (in units of m) of wall to obtain "q_{reqd}".



(a) Graph for "q_{reqd}" to be multiplied by "k"



(b) Graph for "q_{reqd}" to be multiplied by "k" and "H"

Figure 6 Nomographs to obtain the required flow rate for geonet and geocomposite drain design

5.4 *Seepage From Additional Sources*

There are other sources of seepage in addition to that coming from the retained soil zone, which might be considered on a site-specific basis, for example,

- high permeability sand lenses,
- fractured rock,
- artesian conditions within the foundation soil, and/or
- surface water moving across the backfilled slope.

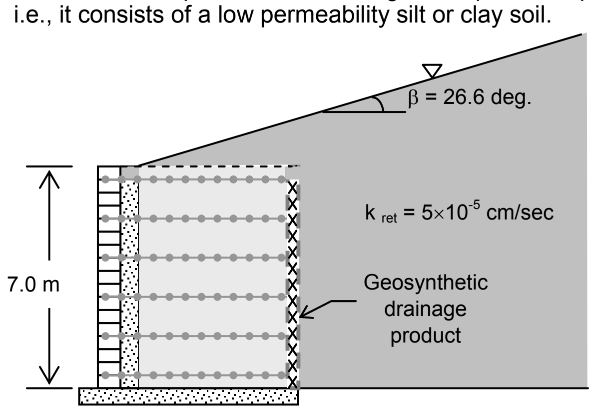
These are difficult issues to quantify and the approach in this paper is to include a (seepage) load factor to the calculated value of required flow rate or transmissivity. Table 1 gives some recommended values.

Table 1 Seepage Load Factors to Adjust "q_{reqd}" to Site Conditions

Additional Water Source	Permeability of Retained Soil (cm/sec)		
	Low $< 10^{-6}$ cm/sec	Medium 10^{-6} to 10^{-4} cm/sec	High $> 10^{-4}$ cm/sec
Sand Lenses	2	1.5	1
Fractured Rock	2	1.5	1
Artesian Base Condition	←	1 to 2	→
Ground Surface Inflow	←	1 to 3	→

5.5 *Numeric Example*

Determine the flow rate factor-of-safety of all ten of the geosynthetic drainage products illustrated in Figure 5 for the wall shown below. Assume that the reinforced soil zone backfill is not capable of transmitting the required seepage, i.e., it consists of a low permeability silt or clay soil.



Step 1 - From laboratory testing, obtain q_{ult}. See Figure 5 for data on 10-different geosynthetic materials.

Step 2 - Apply site-specific reduction factors to obtain q_{allow}. Using Eq. 13, we will arbitrarily assume ΠRF = 5.0. Thus, all of the data shown in Figure 5 will be divided by 5.0 to obtain "q_{allow}"

Step 3 - Determine q_{reqd} of the example for a retained soil permeability of 5×10^{-5} cm/sec. From Figure 6(a) for H = 7.0 m and β = 26.6 deg, we have

$$\frac{q}{k} = 3.6\, m^2/\text{min.}$$

$$q_{reqd} = 3.6\,(0.00005/100)$$

$$= 1.80 \times 10^{-6} m^3/\sec - m$$

Step 4 - Apply a seepage load factor for additional possible sources of seepage. From Table 1 we will arbitrarily use a value of 1.5. Thus,

$$q_{reqd(\text{mod})} = 1.80 \times 10^{-6}(1.5)$$

$$= 2.70 \times 10^{-6} m^3/\sec - m$$

Step 5 - Arrange the data from the previous four steps in a table form and calculate the resulting FS-values. This results in Table 2 which is in units of m^3/sec-m. Products are listed in the same order as illustrated in Figure 5. The resulting FS-values in the far right column are self explanatory.

Table 2 FS-results from numeric example

Product	q_{ult}[1]	q_{allow}[2]	q_{reqd}[3]	q_{reqd}[4] (mod.)	FS[5]
sheet drain - N	4.0×10^{-3}	8.0×10^{-4}	1.80×10^{-6}	2.70×10^{-6}	300
sheet drain – T	3.5×10^{-3}	7.0×10^{-4}	1.80×10^{-6}	2.70×10^{-6}	260
sheet drain – M	3.0×10^{-3}	6.0×10^{-4}	1.80×10^{-6}	2.70×10^{-6}	220
geonet – trivial	2.5×10^{-3}	5.0×10^{-4}	1.80×10^{-6}	2.70×10^{-6}	190
geonet - biaxial	2.2×10^{-3}	4.4×10^{-4}	1.80×10^{-6}	2.70×10^{-6}	160
sheet drain – E	2.0×10^{-3}	4.0×10^{-4}	1.80×10^{-6}	2.70×10^{-6}	150
GN T – composite	1.0×10^{-3}	2.0×10^{-4}	1.80×10^{-6}	2.70×10^{-6}	75
GN B – composite	2.3×10^{-4}	4.6×10^{-5}	1.80×10^{-6}	2.70×10^{-6}	17
GT 1500 g/m^2	1.4×10^{-4}	2.8×10^{-5}	1.80×10^{-6}	2.70×10^{-6}	10
GT 1000 g/m^2	4.0×10^{-5}	8.0×10^{-6}	1.80×10^{-6}	2.70×10^{-6}	3.0

Notes 1 – q_{ult} is obtained using Figure 5 at i = 1.0
2 – $q_{allow} = q_{ult}/\Pi RF = q_{ult}/5.0$
3 – q_{reqd} is obtained from Figure 6(a)
4 – q_{reqd} (mod) = $q_{reqd} \times 1.5$; per Table 1
5 – FS = q_{allow}/q_{reqd} (mod.)

6 SUMMARY AND CONCLUSIONS

A finite difference design procedure was presented to determine the seepage pressures that can arise against the reinforced soil zone of SRWs when using low permeability backfill soils. Seepage forces are essentially doubled (over geostatic forces) when drainage is not accommodated. A number of drainage alternatives are possible. The most common is a soil or geocomposite base drain coupled with a geocomposite back drain.

A design approach leading to a flow rate factor of safety was presented. This involves obtaining an allowable flow rate for a candidate drainage material. Laboratory test data for ten different geosynthetics were presented. Also needed to complete the design is a required flow rate value based upon site specific conditions. The finite difference method was again used. Having both allowable flow rate (decreased using reduction factors) and required flow rate (increased using load factors), the FS-values were obtained for 10-products using a specific design example. The FS-values were seen to vary from 300 to 3.0.

Clearly, the use of drainage systems behind the reinforced soil zone of SRW's is possible via a number of commercially available drainage products. Essentially all of the problems cited evaluated to date could have been avoided if such drainage systems were designed, specified and installed. Unfortunately, none of the 26-case histories investigated had such drainage provisions.

The conclusion reached is obvious. The low cost of SRW's is due in part to the use of locally available (or actual site) soils. Typically these soils have a low permeability. They can, and should, be used for backfill in the reinforced soil zone but *only if drainage behind and beneath this zone is designed, specified and constructed accordingly*. This paper has attempted to show this design methodology which is felt to be within the state-of-the-practice and should be utilized accordingly.

7 ACKNOWLEDGEMENTS

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