Design methods for drainage of reinforced soil walls

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ABSTRACT: Drainage is an essential feature in the design and construction of reinforced walls and slopes. Cohesive and marginal fills are more and more utilized and the low permeability of these materials requires special attention in the design of the drainage system, both along the perimeter of the reinforced block and inside the fill itself. But even granular soil fills with relatively high permeability may be subject to hydraulic conditions which may cause saturation either at the bottom, at the top, or behind the reinforced soil block, hence requiring again special attention in the design of the drainage system. Various types of draining geocomposites and even a peculiar type of draining geogrids are presently available on the market, in addition to the traditional method of using coarse granular soil as drainage medium. The paper analyzes the different drainage conditions that can be encountered and set the design principles for each case. Then the selection procedure for the suitable draining geosynthetic in each situation is introduced.

Keywords: Reinforced soil walls, drainage

1 INTRODUCTION

Reinforced soil walls and steep slopes are usually designed to be stable under static and seismic conditions by assuming a defined pore pressure parameter r_u or a defined position of the phreatic surface. Under this assumption, pore water pressure increases with depth throughout the soil profile.

However there are situations where the phreatic surface raises very quickly compared to its "at rest" position: this is typically the case when the reinforced soil structure lays in the proximity of a river; if the river is subject to flooding, then its level will raise very quickly and the water table will raise as well, up to a certain distance, which may include the reinforced soil structure; eventually the flooding will extinguish and the water level in the river will lower rather quickly, sometimes in few hours; but it may happen that the water table cannot lower so quickly, because the water flow and velocity are limited by the permeability of the soil; hence it may happens that in the reinforced soil structure the water table is high, while in front of it the water level has already returned to the usual level; such difference in water level will produce increased water thrust in the reinforced soil structure; this situation is addressed by the design engineers through stability analyses in rapid draw down conditions, which usually lead to two solutions: a) increase the tensile strength and length of geogrids compared to the drained conditions; b) provide a drainage system for removing the excess water pressure.

In other cases the water table may convey high water flow at the back of the reinforced soil structure, due to outlets from ponds, lateral discharge from roads, spills from pipes, etc.; in such situation the permeability of the fill may be insufficient to assume that the fill itself will be in free draining conditions; again this may lead the design engineer to two solutions: a) increase the tensile strength and length of geogrids compared to the free drained conditions; b) provide a drainage system for removing the excess water pressure at the back of the reinforced soil structure.

Moreover experience has shown that reinforced soil structures may get saturated from the top, due to snow melt, rain or runoff water percolation, only down to a limited depth from the top. Soil below that limit may be unsaturated or not be subjected to pore water pressure. When the top part of a reinforced soil structure get saturated, the local increase in pore water pressure generates horizontal thrusts and decrease in effective stress which, if not considered in design, may lead to a top – down hydraulic induced failure mechanism. This type of failure has been experienced in particular both with segmental concrete block walls and green faced reinforced soil structures. As a result, the spacing, strength, main length and wrapping / connection length of geogrid layers, required to sustain the additional horizontal thrust generated by the hydraulic pressure at the top of the structure, need to be defined; and / or a drainage system has to be designed for removing the excess pore water pressure at the top of the structure.

The three situation of water flow from the bottom and consequent rapid draw down, flow from the back and flow from the top of the reinforced soil structure will now be analyzed, and the suggested design method for the required drainage systems will be indicated.

2 WATER FLOW FROM THE BOTTOM AND RAPID DRAW DOWN

In terms of drainage, saturation from the bottom of the reinforced soil structure (RSS) becomes a problem when the water, after reaching the maximum level, is subject to a rapid draw down; other situations, where both the raise and lowering of the water table occurs slowly, present much lower problems in terms of drainage and stability of the RSS.

Let's introduce now the most typical problem associated with rapid draw down of water level; all other cases can be approached in a similar manner.

Let's consider the case when the RSS lays in the proximity of a river; after a river flooding the water level in the river will decrease rather quickly, while in the reinforced soil structure the water table may remain high for hours or days, that is an excess pore pressure may remain in the fill and the water table will be similar to the one shown in Fig. 1.

Stability analyses in rapid draw down conditions will require increased tensile strength and length of geogrids compared to the drained conditions, as shown in Fig. 2. In fact it can be seen that, due to the increased thrust produced by the unbalanced water pressure, the geogrid length need to be increased: for the example in Fig. 2, related to a wall height of 10.0 m with ΔH_w = 6.5 m, the geogrid length / wall height ratio changes from L / H = 0.70 without rapid draw down conditions to L / H = 1.17 with rapid draw down conditions; the required geogrid ultimate tensile strength (for typical extruded HDPE uniaxial geogrids) changes from 90 kN/m to 150 kN/m.

The design engineer may try to solve the problem by providing geocomposites for drainage (GCD) at the back of the retaining structure, and / or GCD strips internally to the fill (Fig. 3), in order to remove the excess pore pressure.

The internal drainage system (Fig. 3) can consist of strips of draining geocomposites (GCD strips), or of draining geogrids (Fig. 4). The GCD strips will have width B and lateral spacing S_h, as shown in Fig. 5.

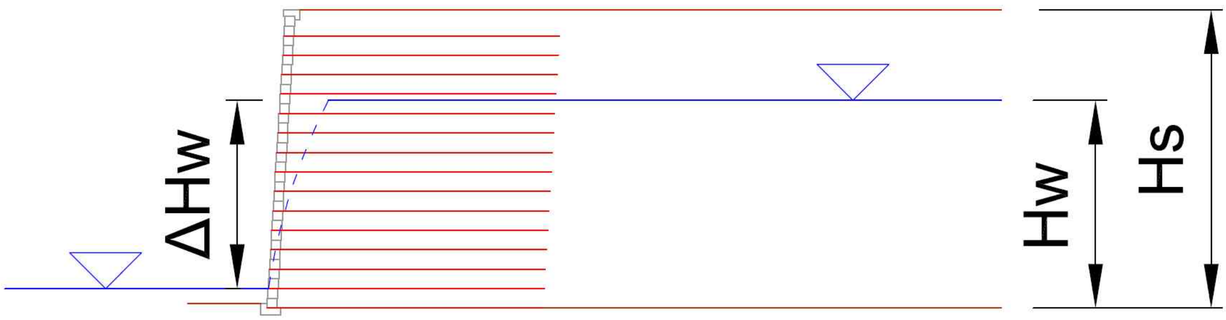


Fig. 1. Typical position of the water table in rapid draw down conditions

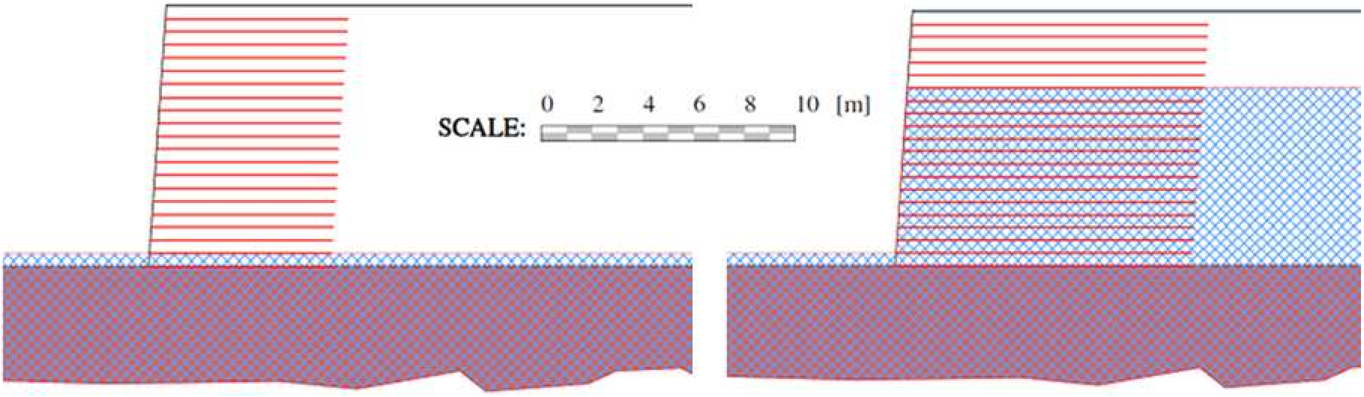


Fig. 2. Example of design layout for a 10 m high wall without and with rapid draw down conditions

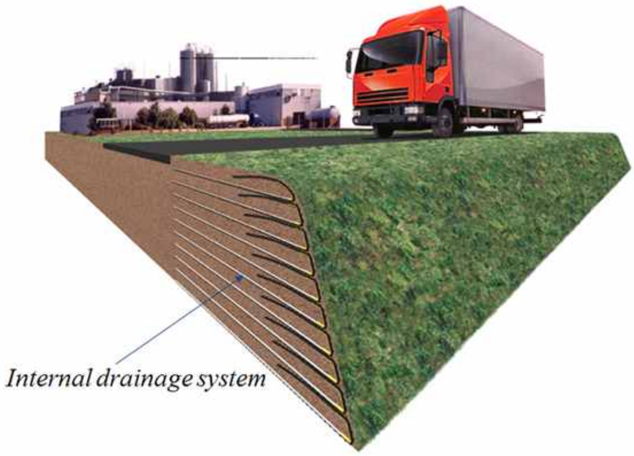
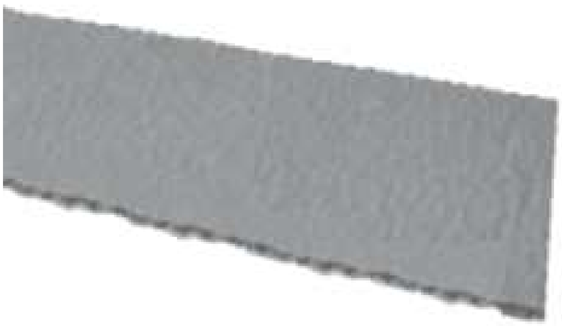


Fig. 3. Example of drainage at the back of the RSS and of internal drainage system



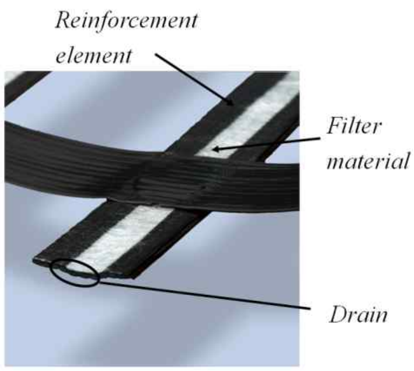


Fig. 4. Example of draining geocomposite strips and of draining geogrid

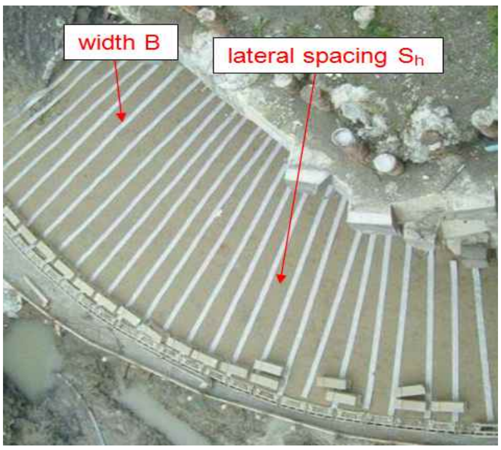


Fig. 5. Internal drainage system made up of geocomposite strips

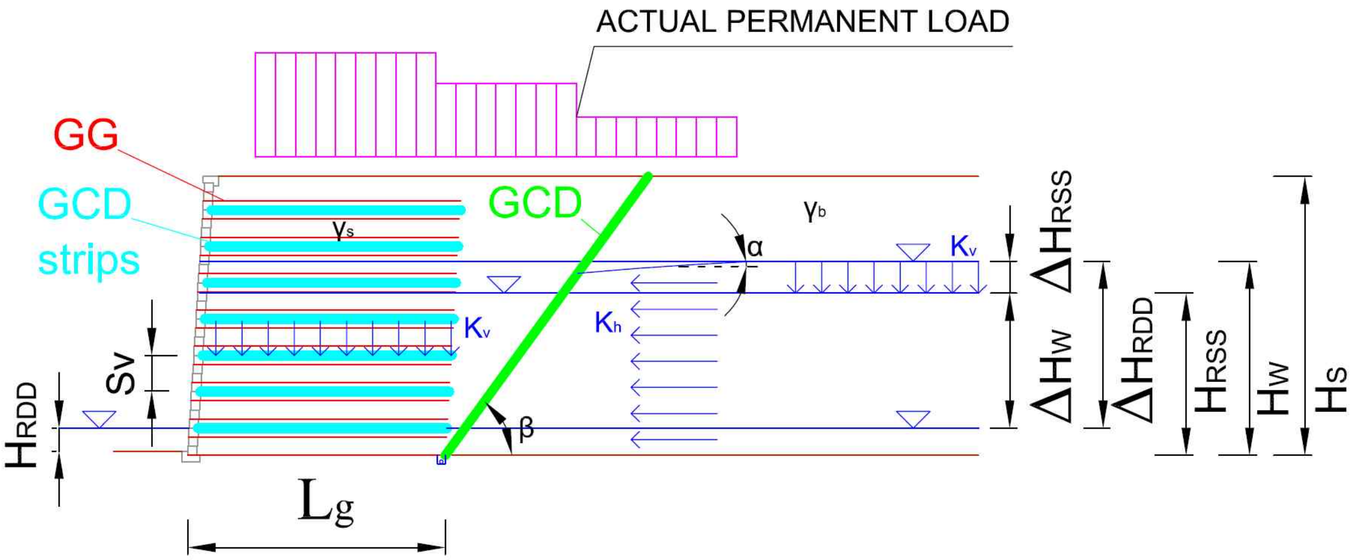


Fig. 6. Scheme for rapid draw down conditions when geocomposites for drainage (GCD) at the back of the RSS and/or GCD strips inside the reinforced body are provided

While the draining layer at the back should have the function of moving the increase in water table level away from the face, the internal drainage system should have the function of putting the fill in equivalent free draining conditions.

But in rapid draw down conditions both solutions have negligible effect in lowering the pore pressure inside the RSS block.

In fact, to be considered as self draining, in rapid draw down conditions the fill shall have a permeability larger than the velocity at which the water level decreases in the river.

With reference to Fig. 6, if the water level increased to a level H_w (m), and during rapid draw down it decreased to a level H_{RDD}, by estimating the time t_d required for draw down (seconds), the velocity of water level decrease in the river V_r (m/s) will be:

$$V_r = (H_w - H_{RDD}) / t_d = \Delta H_{RDD} / t_d \quad (1)$$

If the vertical permeability of the soil K_v is higher than V_t, then the fill will actually be in self draining conditions, and no drainage system is required.

Instead, if the vertical permeability of the soil K_v is lower than V_r, during rapid draw down an excess water pressure will remain in the fill, possibly for a time long enough to produce a sudden decrease in stability conditions.

In such conditions the velocity at which the water table lowers inside the RSS is limited by the vertical permeability K_v of the fill: whichever is the vertical spacing S_v of the internal drainage system (where the geocomposite strips or the draining geogrids are placed horizontally) there will always be a delay between the decrease in water table level and the decrease in river level (or the decrease in water level externally to the RSS), since the difference in water lowering velocity cannot be influenced by the internal drainage system. In fact below the decreasing external water level there will be the same water pressure inside and outside the RSS, hence no hydraulic gradient can be established along the geocomposite strips; while above the decreasing external water level the velocity of water is limited by soil permeability; therefore in these conditions the high horizontal flow rate of the geocomposites is useless to decrease the water level inside the RSS.

The drainage system at the back of the RSS will have absolutely no influence on the velocity of water level decrease in the RSS block: hence if only this drainage layer is provided, the internal stability of the RSS shall be carried out considering the residual pore pressure at the end of draw down (see hereafter); while the global stability analyses should be carried out with the water table positioned like in Fig. 1.

Hence in this conditions the only way to increase the velocity at which the water level decreases in the RSS would be to provide vertical draining elements, such as perforated pipes or prefabricated vertical drains (PVD) at close lateral spacing; but such drainage system would be extremely complicated to build during RSS construction; it could be done by perforating from the top at the end of RSS construction, but all reinforcing layers would be seriously damaged.

Hence we are left with two only possibilities:

a) Substitute the fill with a soil having higher permeability, such that $K_v > Vr$;
b) Evaluate the maximum pore pressure left by the rapid draw down and design the RSS for such pore pressure.
For case b), with reference to Fig. 6, the procedure is the following.
During the time of draw down t_d the water level inside the RSS will decrease of a height ΔH_{RSS} (m):

$$\Delta H_{RSS} = t_d \cdot K_v \tag{2}$$

Hence the net drop in water level during rapid draw down ΔH_W (m) will be:

$$\Delta H_W = H_{RSS} - H_{RDD} = t_d \cdot (V_r - K_v) \tag{3}$$

And the net drop in pore water pressure will be:

$$\Delta U = \Delta H_W \cdot \gamma_W \tag{4}$$

The residual pore pressure in the RSS, U_{RSS} (kPa), and the residual pore pressure parameter, r_u, will be:

$$U_{RSS} = H_{RSS} \cdot \gamma_W \tag{5}$$

$$r_u = U_{RSS} / (\gamma_s \cdot H_s) \tag{6}$$

where:
γ_s = unit weight of fill in saturated conditions (kN/m^3)
H_s = height of the RSS (m).
Then the stability of the reinforced wall at the end of the rapid draw down shall be evaluated by performing stability analyses on the RSS: reasonably, by similarity with other type of rapid transient loads (e.g. seismic conditions), during water draw down there will be no variable surcharge applied nor seismic load, hence only the self weight of fill and permanently carried loads shall be considered, with Amplification Factors for loads set at unity (γ_f = 1.0 in Eurocode 7, EN 1997-1); moreover for the relatively short time of water draw down the Reduction Factors for materials and resistances can be set at unity (γ_M = 1.0 in Eurocode 7). For reinforcement the Reduction Factor for creep RF_{CR} can be set equal to 1.0 as well.
This means that "traditional" stability analyses can be performed (all amplification and reduction factors equal to unity), just including the residual pore pressure as U_{RSS} or r_u or ΔH_W (according to the stability analysis method or software used); the length and tensile strength of reinforcing geosynthetics shall be varied, by trials and errors, until a minimum Factor of Safety FS_{DD} (γ_R in Eurocode 7) in draw down conditions is achieved:

$$FS_{DD} \geq 1{,}10 \tag{7}$$

Obviously the starting configuration of reinforcement will be the layout resulting from stability analyses in static conditions; if condition (7) is already reached there is no need to proceed further; if condition (7) is not respected, then the layout shall be modified with longer and / or stronger reinforcements.

The internal drainage system may be provided anyway: if the vertical permeability of the soil is far lower than V_r, then ΔH_W, ΔU and r_u may result to be very high; hence to lower the water table inside the RSS in short time, which is beneficial for stability anyway, geocomposite strips or draining geogrids can be provided, at least up to H_W.

At the end of draw down the hydraulic conditions will be different inside and outside the RSS (outside the water level will be at H_{RDD}, inside it will be at H_{RSS}, see Fig. 6)), hence now a hydraulic gradient can be established along the geocomposite strips: therefore in these conditions the high horizontal permeability of the geocomposites will allow relatively rapid removal of water from the RSS.

In such conditions the velocity at which the water table lowers inside the RSS is still limited by the vertical permeability K_v of the soil, but, with reference to Fig. 6, all the draining strips will catch water from the soil above at the same time: hence the drainage length will be decreased from ΔH_W to S_v, and the time for dissipating the pore water pressure inside the RSS will be decreased from ($\Delta H_W / K_v$) to (S_v / K_v).

3 DESIGN OF THE DRAINAGE SYSTEM INSIDE THE RSS

When draining strips are placed horizontally inside the RSS at regular vertical spacing (Fig. 3 right), the water will typically move downward vertically toward the closest draining strips.
If the draining strips are placed at vertical spacing S_v (m), since they all work together the time t_{RSS} required to remove the excess water pressure will be equal to the time required to drain the water between each draining layer, that is:

$$t_{RSS} = S_v / K_v \tag{8}$$

On the other hand, if the time t_{RSS} is set, then the vertical spacing of draining strips will be:

$$S_v = t_{RSS} \cdot K_v \tag{9}$$

Now we need to evaluate the flow rate required for the GCD strips or the draining geogrid.
The volume of water V_W (m^3/m) to be drained by each draining layer is:

$$V_W = n_s \cdot S_v \cdot L_G \cdot 1 \tag{10}$$

where:
L_G = length of the draining geosynthetic strips (m)
n_s = porosity of fill.
The required flow rate per unit width will then be:

$$Q_s = V_W / t_{RSS} = (n_s \cdot S_v \cdot L_G) / t_{RSS} \tag{11}$$

By applying a Factor of Safety on drainage capacity FS_D, the design flow rate becomes:

$$Q_D = Q_s \cdot FS_D \tag{12}$$

A rational value would be: $FS_D = 1.30$ (13)

If GCD strips or Geogrid draining strips have width B_{strip} and horizontal spacing S_h, the design flow rate for the strips will be:

$$Q_{D,\,strip} = Q_D \cdot S_h / B_{strip} \tag{14}$$

The hydraulic gradient will be provided by the difference in water pressure between the back and the front of the GCD (which will be at atmospheric pressure at the face of the RSS) divided by the vertical pressure in the fill.
The worst conditions for drainage usually occur at the bottom of the RSS, anyway in general both the top and bottom drainage layers should be checked.
For the top drainage layer (the first below the water level H_{RSS}) the hydraulic gradient i_{top} and applied pressure p_{top} (kPa) will be:

$$i_{top} = S_v / L_g \quad ; \quad p_{top} = \gamma_s \cdot (H_S - H_W) + (\gamma_s - \gamma_w) \cdot S_v \tag{15}$$

while for the bottom drainage layer the hydraulic gradient i_b and applied pressure p_b (kPa) will be:

$$i_b = H_W / L_g \quad ; \quad p_b = \gamma_s \cdot (H_S - H_W) + (\gamma_s - \gamma_w) \cdot H_w \tag{16}$$

Hence the condition for selecting the appropriate GCD or draining geogrid is:

$$Q_{GCD}\,(i_{top} \text{ or } i_b,\ p_{top} \text{ or } p_b,\ \text{contacts S / S, T°}) \geq Q_{D,\,strip} \tag{17}$$

where:
T° = design temperature for the water flowing in the GCD.

In fact Q_{GCD} is highly dependent on water viscosity, which in turn depends on the temperature of the water; T° can be assumed as the average annual temperature in the area of the project.
The contacts shall be Soft – Soft (S / S) because there is soil on both sides of the GCD strips.

4 DESIGN OF THE DRAINAGE SYSTEM AT THE BACK OF THE RSS

When there is high water flow at the back of the reinforced soil structure (Fig. 6), due to rapid draw down, outlets from ponds, lateral discharge from roads, spills from pipes, etc., the permeability of the fill may be insufficient to assume that the fill itself will be in free draining conditions; then the design engineer has two possibilities:

a) increase the tensile strength and length of geogrids compared to the free drained conditions (Fig. 2);
b) provide a drainage system for removing the excess water pressure at the back of the reinforced soil structure.

For case b) the design of the drainage system at the back of the RSS, with reference to Fig. 6, shall follow this procedure.
In this condition the water flows almost horizontally, with the piezometric surface of the water table having a slope α (usually α = 1 – 3 deg)
Hence the input flow rate into the drainage system at the back of the RSS, according to Darcy's law, is:

$$Q_i = K_h \cdot A \cdot i = K_h \cdot H_w \cdot 1 \cdot \sin\alpha \tag{18}$$

where:
K_h = horizontal permeability of the soil at the back of the RSS
H_w = undisturbed water table level (m)
Note that K_h is usually much larger than K_v for any type of soil.
The hydraulic gradient in the GCD and the applied pressure p_{back} (kPa) will be in this case:

$$i_{back} = \sin\beta \quad ; \quad p_{back} = \gamma_s \cdot (H_S - H_W) + (\gamma_s - \gamma_w) \cdot H_w \tag{19}$$

where:
β = inclination of the GCD at the back face of the reinforced soil block (deg).
If a coarse granular soil is used for the drainage layer, according to Darcy's law it shall have thickness T_G given by:

$$T_G = Q_D / (K_G \cdot i_{back}) \tag{20}$$

where:
K_G = permeability of the coarse granular soil (m/s)
In case a GCD is used as drainage layer, by applying a Factor of Safety (Eq. 13) on drainage capacity FS_D, the design flow rate becomes:

$$Q_D = Q_i \cdot FS_D \tag{21}$$

The GCD shall have a minimum flow rate respecting the condition:

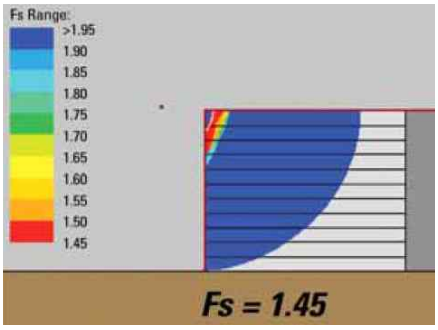
$$Q_{GCD}\,(i_{back}, p_{back}, \text{contacts S / S}, T°) \geq Q_D \tag{22}$$

5 DESIGN OF THE DRAINAGE SYSTEM IN CASE OF SATURATION FROM TOP

Reinforced soil structures may get saturated from the top, due to snow melt, rain or runoff water percolation, usually only down to a limited depth from the top. Soil below that limit may be unsaturated or not be subjected to pore water pressure. When the top part of a reinforced soil structure get saturated, the local increase in pore water pressure generates horizontal thrusts and a decrease in effective stress that, if not considered in design, may lead to a top – down hydraulic induced failure mechanism. This type of failure has been experienced in particular both with segmental concrete block walls and green faced structures. As a result, the spacing, strength, main length and wrapping / connection length of geogrid layers, required to sustain the additional horizontal thrust generated by the hydraulic pressure at the top of the structure, need to be defined; and / or a drainage system has to be designed for removing the excess pore water pressure at the top of the structure.

Let's imagine that a green faced RSS, with geogrids wrapped around at the face, get saturated from the top, down to 2.0 m from the crest, and that in this space there are 3 geogrid layers, with 1.50 m anchorage length for the wrapped-around part: the increase in pore water pressure will produce an increased thrust on the face; the top geogrid layers are subject to the minimum vertical pressure, hence to the minimum pull-out resistance; then the increased thrust on the face may produce pull-out of the top geogrid, firstly; then the second geogrid from the top will have to bear the thrust of the top two layers; if the pull-out resistance of the second geogrid is not adequate for such increased thrust, even the second geogrid wrapped-around part will fail in pull-out; and so on; hence a progressive failure, like a progressively opening zipper, may occur.

Fig. 7 shows the different stability conditions when the top portion of the RSS is unsaturated and saturated (from Leschchinsky, 2008): it is clear that saturation from the top produces local unstable conditions, which would produce the failure of the RSS at top, and would eventually propagate top – down with the resulting failure of the whole RSS. Or the failure would be confined in the top portion, if the geogrid layers below are able to stop the progressive failure; anyway a catastrophic failure of the top portion would occur.

a)


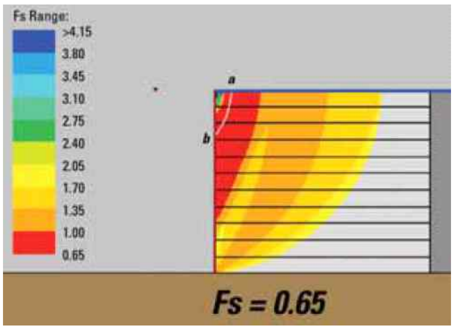
b)


Fig. 7. Safety map corresponding to dry conditions (a) and to saturated conditions of the top portion of the RSS (b) (from Leschchinsky, 2008)

In such situation there are two possibilities:

a) Design the geogrids, in terms of length, tensile strength, and wrap-around length, based on stability analyses carried out considering the saturated portion at the top (see Fig. 9.b);
b) Design the geogrids in dry conditions (see Fig. 9.a) and provide a drainage system inside the top part of the RSS.

In the latter case the design of the drainage system, with reference to Fig. 8, should be carried out as follows:

1) Evaluate the maximum water level H_t on top of the RSS
2) Provide GCD or GCD strips between the crest and the top geogrid layer, at depth Z_1 below the crest
3) Design the GCD or GCD strips:

   Saturation from top will occur vertically at velocity $V_t = K_v$, where K_v is the vertical permeability of the fill;

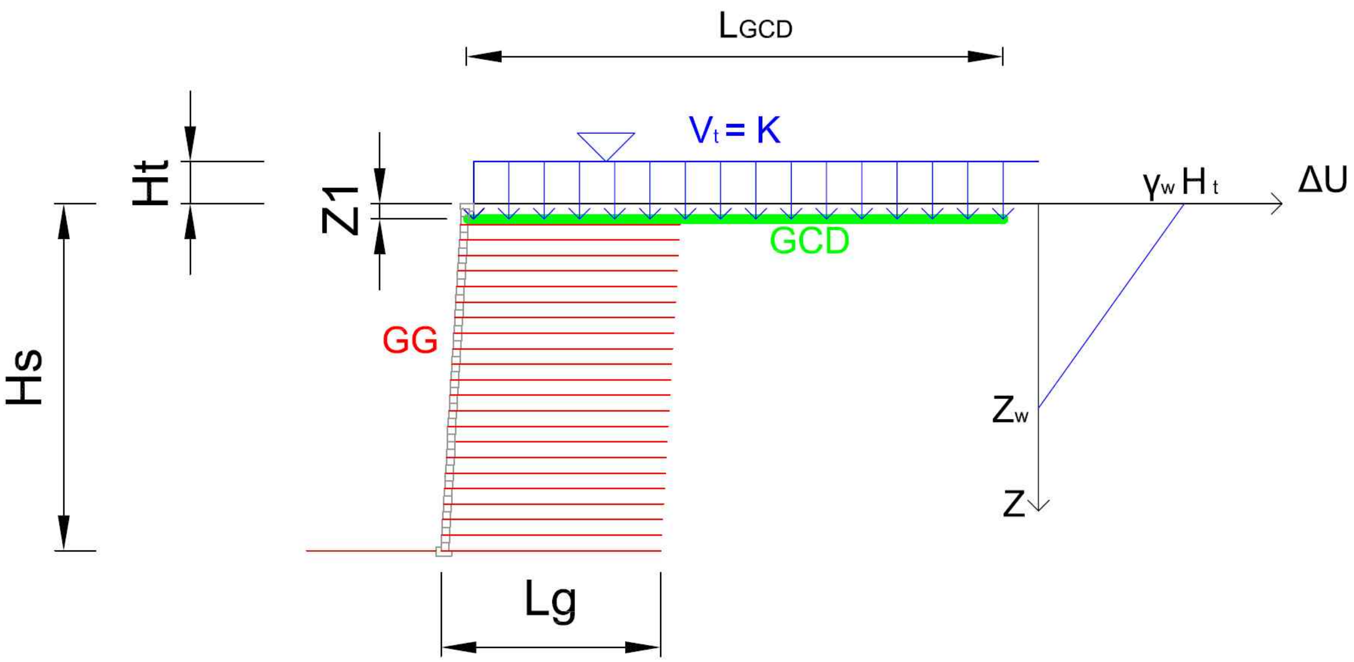


Figure 8. Scheme for water flow from top of the RSS

Hence the unit input flow rate q ($m^3/s/m^2$) in the GCD, according to Darcy's law, will be:

$$q = K_v \cdot A \cdot i / A = K_v \tag{23}$$

where:
A = unit cross-sectional area of vertical water flow (m^2)
i = hydraulic gradient (= 1.0 for vertical flow)
The input flow in the GCD is:

$$Q_i = q \cdot L_{GCD} \tag{23}$$

The design input flow in the GCD is:

$$Q_D = Q_i \cdot FS_D \tag{24}$$

where:
FS_D = Factor of Safety for drainage = 1.30 (default value)
If GCD strips are provided, rather than contiguous GCD sheets, having width B_{strip} and horizontal spacing S_h, the design flow rate for the strips will be given by Eq. 14.
Then set the length L_{GCD} of the GCD by performing stability analyses with increasing GCD length (starting from the length L_g of geogrid reinforcement at top), with a constant water level H_t on the length L_{GCD}, or with a triangular water pressure distribution down to the depth of water saturation Z_w, as shown in Fig. 8, until the resulting Factor of Safety is satisfying.
As a first indication:

$$L_{GCD} \geq 2\ L_g \tag{25}$$

$$Z_W = H_S / 2 \tag{26}$$

In this case, the hydraulic gradient in the GCD is:

$$i_{GCD} = (H_t + Z_1) / L_{GCD} \tag{27}$$

The pressure on the GCD is:

$$p_{GCD} = Z_1 \cdot (\gamma_S - \gamma_w) + p_{PL} \tag{28}$$

where:
p_{PL} = uniform vertical pressure $Q_T = Q_{20} \cdot \left(\frac{\mu_{20}}{\mu_T} \right) = Q_{20} \cdot CF_T$ produced by permanent loads on top of the wall (kPa)

The proper GCD shall be selected with the condition:

$$Q(i_{GCD}, p_{GCD}, T°) \geq Q_D \text{ or } Q_{D,\,strip} \tag{29}$$

6 SELECTION OF THE GCD

The water flow capacity in the plane of draining geocomposites is measured according to ISO 12958:2010 test standard. Testing results are usually summarised in charts giving the specific flow rate Q (l/s/m or m²/s) vs the applied compressive stress (in terms of uniform applied pressure) p (kPa) and the hydraulic gradient i. Typical charts are shown in Figure 9. Each chart is referred to a specific combination of the type of contacts on the two faces of the GCD: contact may be either Rigid (that is, practically no intrusion of the geotextile filter into the draining core will occur) or Soft (that is, intrusion of the geotextile filter into the draining core will occur).

In this case there is soil in contact with both faces of the GCD, hence the considered contacts shall be Soft – Soft (S / S in Fig. 9).

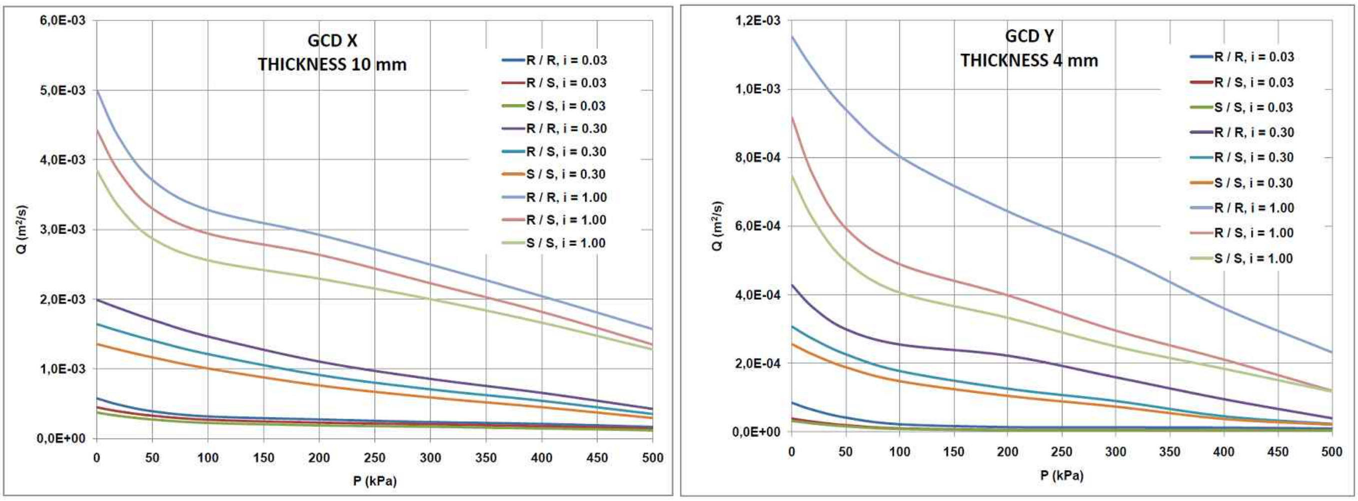


Fig. 9. Typical charts providing the specific flow rate of two GCD of different thickness, as function of the hydraulic gradient and the applied pressure, for the three possible contact combinations

If the flow rate has to be evaluated for a gradient i_2 different from the immediately higher value i_1 used for testing and shown in the charts, it is possible to evaluate the specific flow rate for the actual hydraulic gradient i_2 by the following formula (Cancelli and Rimoldi, 1989):

$$Q_{i2} = Q_{i1} \cdot \sqrt{\frac{i_2}{i_1}} \tag{30}$$

with:
Q_{i1}= specific flow-rate from the chart (l/s/m or m²/s);
Q_{i2}= specific flow-rate for the i_2 gradient (l/s/m or m²/s);
i_1 = hydraulic gradient on the chart, immediately higher than the actual hydraulic gradient;
i_2 = actual hydraulic gradient.
Hence the correction factor for hydraulic gradient, CF_i, is:

$$CF_i = (i_1 / i_2)^{0.5} \geq 1.0 \tag{31}$$

Moreover, standard tests are performed using water at 20°C temperature. It is possible to calculate the specific flow rate for another temperature or liquid viscosity with the equation :

(32)

Hence: $CF_T = \mu_{20} / \mu_T$ (33)
where:
Q_{20}, Q_T = specific flow rate at 20°C and T °C;
μ_{20}, μ_T = viscosity of water at 20°C and T °C;
CF_T = correction factor for temperature and viscosity.
For all applications, according to present ISO WD TR 18198 and ISO WD TR 18228-4, the general procedure for evaluating the available flow rate of the geocomposites consists in applying a set of Reduction Factors which take into account all the phenomena that may decrease the flow rate over the entire design life compared to the short term flow rate measured in the tests according to ISO 12958:2010 standard:

$$Q_a = \frac{\mathrm{Q_L}}{\mathrm{RF_{in}} \cdot \mathrm{RF_{cr\text{-}Q}} \cdot \mathrm{Rf_{cc}} \cdot \mathrm{RF_{bc}} \cdot \mathrm{RF_L}} \tag{34}$$

where:
Q_a = available long term flow rate for the geocomposite;
Q_L = short term flow rate obtained from laboratory tests;
RF_{in} = Reduction Factor for the intrusion of filter geotextiles into the draining core;
$RF_{cr\text{-}Q}$ = Reduction Factor for the compressive creep of the geocomposite;
RF_{cc} = Reduction Factor for chemical clogging of the draining core
RF_{bc} = Reduction Factor for biological clogging of the draining core
RF_L = Reduction Factor for overall uncertainties on laboratory data and field conditions.
The Reduction Factors shall be set considering the specific conditions of each project, taking into consideration the experience and/or research on similar conditions of use.
Eq. (34) can be put in the form:

$$Q_L = \mathrm{Q_a} \cdot \mathrm{RF_{in}} \cdot \mathrm{RF_{cr\text{-}Q}} \cdot \mathrm{RF_{cc}} \cdot \mathrm{RF_{bc}} \cdot RF_L \tag{35}$$

Introducing the correction factors for hydraulic gradient and water temperature in Eq. 35, the design short term flow rate of the GCD becomes:

$$Q_{LD} = Q_L \cdot CF_i / CF_T \tag{36}$$

where:
Q_{LD} = design short term flow rate of the GCD (l/s/m or m²/s).
The condition for selecting the GCD is:

$$Q_{LD} \geq Q_D \quad \text{or} \quad Q_{D,\,strip} \tag{37}$$

From Eqs. 35 ÷ 37, the suitable GCD can be selected by using directly the flow rate charts, like in Fig. 9: for the gradient i_1 used in Eq. 30 (that is the immediately higher gradient of the actual gradient) and applied pressure p, the value Q on the curve shall respect the condition (37).

7 EXAMPLE

The maximum water level for a river in flooding conditions is H_W = 5.30 m on the base of an adjacent RSS. From historical records it can be estimated that the flooding level will return to the level H_{RDD} = 0.50 m in a time t_d = 12 hours = 43,200 s. The project is located in a region with temperate weather, with average daily temperature T° = 22°C.
Hence it is: $V_r = (H_W - H_{RDD}) / t_D = 0.39$ m/h $= 1.1 \times 10^{-4}$ m/s
The permeability of the fill, made up of sand and gravel with 20 % silt, is: $K_v = 1 \times 10^{-5}$ m/s
Since $K_v < V_t$ then a residual water level will remain inside the RSS at the end of draw down.
The RSS has height H_s = 10 m, with the face at inclination of 85°.
The fill has unit weight γ_s = 20 kN/m³.
Hence it results:

$\Delta H_{RSS} = K_v \cdot t_d = 0.43$ m
$\Delta H_{RDD} = t_d \cdot (V_r - K) = 4.32$ m
$\Delta U = \Delta H_W \cdot \gamma_W = 43.2$ kPa

$U_{RSS} = H_{RSS} \cdot \gamma_W = 48.0$ kPa
$r_u = U_{RSS} / (\gamma_s \cdot H_s) = 0.24$
The stability of the wall shall be checked for such pore pressure conditions.

Now, let's design the internal drainage system, made up of GCD strips, for the following input data:
$n_s = 0.40$
$L_G = 8.0$ m
$B_{strip} = 0.30$ m
$S_h = 1.50$ m
$t_{RSS} = 12$ hours = 43,200 s
Then it results:
$S_v = t_{RSS} \cdot K_v = 0.43$ m
$V_W = n_s \cdot S_v \cdot L_G \cdot 1 = 1{,}38\ m^3/m$
$Q_s = V_W / t_{RSS} = (n_s \cdot S_v \cdot L_G) / t_{RSS} = 3.2 \times 10^{-5}\ m^2/s$
$Q_D = Q_s \cdot FS_D = 3.2 \times 10^{-5}\ m^2/s \cdot 1.30 = 4.1 \times 10^{-5}\ m^2/s$
$Q_{D,\ strip} = Q_D \cdot S_h / B_{strip} = 2.1 \times 10^{-4}\ m^2/s = 0.21$ l/s/m
For the top GCD strips, just below the maximum water level H_w, it results:
$i_{top} = S_v / L_G = 0.054$
$p_{top} = \gamma_s \cdot (H_S - H_W) + (\gamma_s - \gamma_w) \cdot S_v = 98.3$ kPa
If the hydraulic gradient on the flow rate chart, immediately higher than i_{top}, is $i_1 = 0.30$, the correction factor for hydraulic gradient is:
$CF_i = (0.30 / 0.054)^{0.5} = 2.36$
While the correction factor for water temperature (since $\mu_{20°C} = 1.005$ cP and $\mu_{22°C} = 0.957$ cP) is:
$CF_T = (1.005 / 0.957) = 1.050$
Let's select the GCD among a family of products for which we can assume (for $p = p_{top} = 98.3$ kPa and contact S / S):
$RF_{in} = 1.284$
$RF_{cr\text{-}Q} = 1.099$
$RF_{cc} = RF_{bc} = 1.0$
$RF_L = 1.30$
Then from Equations (35) and (37) it results:
$Q_{LD} = 0.21 \cdot (1.284 \cdot 1.099 \cdot 1.0 \cdot 1.0 \cdot 1.3) \cdot (2{,}36 / 1.050) = 0.87$ l/s/m
Let's check if GCD X, that is a product with W shaped geomat drainage core of 10 mm thickness, affording the flow rate chart shown in Fig. 9, is suitable: it is easy to see that for ($i = 0.30$, $p = 98.3$ kPa, contact S / S) the flow rate of GCD X is $Q = 1.0 \times 10^{-3}\ m^2/s = 1.0$ l/s/m $> Q_{LD}$.
Hence GCD X is suitable at top.
For the bottom GCD strips it results:
$i_b = H_W / L_g = 0.66$
$p_b = \gamma_s \cdot (H_S - H_W) + (\gamma_s - \gamma_w) \cdot H_w = 147$ kPa
If the hydraulic gradient on the chart, immediately higher than i_b, is $i_1 = 1.0$, the correction factor for hydraulic gradient is:
$CF_i = (1.0 / 0.66)^{0.5} = 1.23$
While the correction factor for water temperature is still:
$CF_T = (1.005 / 0.957) = 1.050$
Let's check the same GCD X; this time (for $p = p_b = 147$ kPa and contact S / S) we can assume:
$RF_{in} = 1.332$
$RF_{cr\text{-}Q} = 1.142$
$RF_{cc} = RF_{bc} = 1.0$
$RF_L = 1.30$
Then from Equations (35) and (37) it results:
$Q_{LD} = 0.21 \cdot (1.332 \cdot 1.142 \cdot 1.0 \cdot 1.0 \cdot 1.3) \cdot (1.23 / 1.050) = 0.48$ l/s/m
Let's check if GCD X, affording the flow rate chart shown in Fig. 9, is suitable: it is easy to see that for ($i = 1.0$, $p = 147$ kPa, contact S / S) the flow rate of GCD X is $Q = 8.0 \times 10^{-4}\ m^2/s = 0.80$ l/s/m $> Q_{LD}$.
Hence GCD X is suitable also at bottom.

Let's design now the drainage layer at the back of the RSS, made up of contiguous GCD sheets, for the following input data:

$K_h = 1 \times 10^{-4}$ m/s

$\alpha = 3°$

$\beta = 60°$

Then it results:

$Q_i = K_h \cdot H_w \cdot \sin\alpha = 2.8 \times 10^{-5}$ m²/s = 0.028 l/s/m

$Q_D = Q_i \cdot FS_D = 0.028 \cdot 1.30 = 0.036$ l/s/m

$i_{back} = \sin\beta = 0.866$

$p_{back} = \gamma_s \cdot (H_S - H_W) + (\gamma_s - \gamma_w) \cdot H_w = 147$ kPa

The condition for selecting the proper GCD is:

Q_{GCD} ($i_{back} = 0.866$, $p_{back} = 147$ kPa, T° = 22°C) ≥ 0.036 l/s/m

If the hydraulic gradient on the chart, immediately higher than i_{back}, is $i_1 = 1.0$, the correction factor for hydraulic gradient is:

$CF_i = (1.0 / 0.866)^{0.5} = 1.075$

While the correction factor for water temperature is still:

$CF_T = (1.005 / 0.957) = 1.050$

Being Q_{LD} relatively small, let's select GCD Y (see Fig. 9), that is a GCD from the same family of GCD X but with just 4 mm thickness; being of the same family, let's assume that for GCD Y we can apply the same RF values assumed for GCD X used for the bottom GCD strips:

$RF_{in} = 1.332$

$RF_{cr\text{-}Q} = 1.142$

$RF_{cc} = RF_{bc} = 1.0$

$RF_L = 1.30$

Therefore from Equations (35) and (36) it results:

$Q_{LD} = 0.036 \cdot (1.332 \cdot 1.142 \cdot 1.0 \cdot 1.0 \cdot 1.3) \cdot (1.075 / 1.050) = 0.073$ l/s/m

From Fig. 9 it is easy to find that GCD Y, for (i = 1.0, p = 147 kPa, contact S / S) would afford a flow rate $Q = 3.6 \times 10^{-4}$ m²/s = 0.36 l/s/m > Q_{LD}.

Hence GCD Y is suitable for drainage at the back of the RSS.

It is evident that a drainage layer made up of contiguous GCD sheets, and not of discrete strips, requires much less thickness to provide the required flow rate.

Finally, let's design the drainage system in case of saturation from the top.

Let's suppose that heavy snow melting produces an equivalent water level $H_t = 0.50$ m at the top of the wall, for sufficiently long time to saturate the top portion of the wall, and let's design the GCD according to the scheme in Fig. 8.

First of all, let's set the length and vertical position of the GCD:

$L_{GCD} = 2\,L_g = 16$ m

$Z_1 = 0.40$ m

Then, according to Eq. 23:

$q = K_v = 1 \times 10^{-5}$ m/s

The input flow in the GCD is:

$Q_i = q \cdot L_{GCD} = 1.6 \times 10^{-4}$ m²/s

The design input flow in the GCD is:

$Q_D = Q_i \cdot FS_D = 2.08 \times 10^{-4}$ m²/s = 0.208 l/s/m

If the permament load on top of the wall is $p_{PL} = 10$ kPa, the hydraulic gradient in the GCD and the pressure on the GCD are:

$i_{GCD} = (H_t + Z_1) / L_{GCD} = 0.056$

$p_{GCD} = Z_1 \cdot (\gamma_S - \gamma_w) + p_{PL} = 14$ kPa

If the hydraulic gradient on the flow rate chart, immediately higher than i_{top}, is $i_1 = 0.30$, the correction factor for hydraulic gradient is:

$CF_i = (0.30 / 0.056)^{0.5} = 2.31$

Since the water comes from snow melting, we can assume that the temperature of the infiltrating water is 5°C.
Therefore the correction factor for water temperature (since $\mu_{20°C} = 1.005$ cP and $\mu_{5°C} = 1.516$ cP) is:
$CF_T = (1.005 / 1.516) = 0.663$
Let's select again GCD X (see Fig. 9), for which we can assume (for p = 14 kPa and contact S / S):
$RF_{in} = 1.11$
$RF_{cr-Q} = 1.03$
$RF_{cc} = RF_{bc} = 1.0$
$RF_L = 1.30$
Then from Equations (35) and (36) it results:
$Q_{LD} = 0.208 \cdot (1.11 \cdot 1.03 \cdot 1.0 \cdot 1.0 \cdot 1.3) \cdot (2{,}31 / 0.663) = 1.08$ l/s/m
From Fig. 9 it is easy to see that for (i = 0.30, p = 14 kPa, contact S / S) the flow rate of GCD X is $Q = 1.3 \times 10^{-3}$ m^2/s = 1.3 l/s/m > Q_{LD}.
Hence GCD X is suitable for draining the water infiltrating from the top of the wall.

8 CONCLUSIONS

The design of drainage systems for RSS requires a preliminary examination of the hydraulic conditions, where water flow from the bottom and consequent rapid draw down, flow from top, and flow from the back of the RSS shall be identified. Each type of water flow requires a different approach for the design of the drainage system.
The present paper provides the criteria and design equations needed to carry out the design of the drainage system for RSS in the three flow conditions above identified.

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