Multicomponent Geo-synthetic Slope Protection and Erosion Control with Stabilizing Features

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Abstract

Shotcrete has long been a standard method for providing surface erosion control and stabilization for non-vegetative steep mountain cut slopes. It involves spraying a concrete layer, often reinforced with wire mesh, onto the exposed soil or rock face. However, this rigid system presents significant challenges from both sustainability and performance perspectives. The production of cement for shotcrete is carbon-intensive, and the impermeable nature of the facing can lead to the buildup of hydrostatic pressure behind the wall if drainage is not perfectly managed, potentially causing bulging, cracking, and even catastrophic failure. This paper proposes a shift towards geosynthetic alternatives for slope facing. These systems, composed of permeable, high-strength polymer materials, not only prevent surface erosion but also offer integrated drainage capabilities, greater flexibility to accommodate minor ground movements, and a significantly lower carbon footprint. This change is necessary to create more resilient, environmentally friendly, and versatile slope stabilization solutions that can better adapt to the practical challenges of water management in soil structures.

Definitions and Referred Documents

Definitions

- **Shotcrete**: A method of applying concrete projected at high velocity onto a surface to create a hard, protective facing.
- **Geosynthetics**: A category of polymeric materials used in geotechnical engineering, including geotextiles, geogrids, geonets, and geomembranes, which perform functions such as reinforcement, filtration, drainage, and separation.
- **Geogrid**: A geosynthetic material formed into a grid-like structure, used primarily for soil reinforcement.
- **Geotextile**: A permeable geosynthetic fabric used for filtration, separation, and drainage.
- **Geocomposite**: A manufactured product using a combination of geosynthetic components (e.g., a geonet heat-bonded with two geotextiles) to provide a specific function, such as drainage.
- **Soil Nailing**: A ground reinforcement technique where passive steel bars are installed into a slope to increase its shear strength and stability.

- **Global Stability**: The overall stability of a slope against large-scale failure, typically analyzed by assessing a potential critical failure surface (slip circle).
- **Slip Circle (Failure Surface)**: The hypothetical curved surface within a soil mass along which shear failure is expected to occur.
- **Limit Equilibrium Method (LEM)**: An analytical technique used to evaluate slope stability by analyzing the equilibrium of forces and moments on soil slices.
- **Hydrostatic Pressure**: The pressure exerted by impounded water behind a facing or within soil, which reduces effective stress.
- **Pore Water Pressure (u)**: The pressure of groundwater within soil pores, which decreases the soil's shear strength by reducing effective stress between particles.
- **Factor of Safety (FoS)**: A ratio of the soil's available shear strength (resisting forces) to the shear stress acting along a potential failure surface (driving forces). An FoS greater than 1.0 indicates stability.

- **ConstruckSlope Plus** – A multicomponent geosynthetic based erosion control product system that has been trialled and installed by Construck Systems & Technology for non-vegetative mountain cut slopes.

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Introduction

In geotechnical engineering, the decision to recommend shotcrete as a slope protection system is primarily driven by the need to prevent surface erosion and weathering on a slope that has been analyzed for overall, or **global**, stability. The design process begins with a geotechnical investigation to determine the soil's engineering properties. A stability analysis is then conducted, often using Limit Equilibrium Methods (LEM). This analysis models a potential **slip circle**—the most likely path of a rotational failure—especially under worst-case scenarios, such as fully saturated soil conditions following intense rainfall.

The fundamental goal is to ensure the **Factor of Safety (FoS)** is adequate. The FoS for a simple slope can be conceptually understood using the Infinite Slope Method for a cohesionless soil, where the FoS is given by:

$$FoS = \tan(\phi') / \tan(\beta)$$

Where:

- ϕ' is the effective angle of internal friction of the soil.
- β is the slope angle.

For more complex analyses involving cohesive soils and pore water pressure, methods like Bishop's Modified Method are used. The general principle involves balancing the resisting forces (soil shear strength) against the driving forces (the weight of the potential sliding mass). The shear strength (τ) of soil is described by the Mohr-Coulomb failure criterion:

$$\tau = c' + (\sigma n - u)\tan(\phi')$$

Where:

- c' is the effective cohesion.
- σn is the total normal stress on the failure plane.
- u is the pore water pressure.

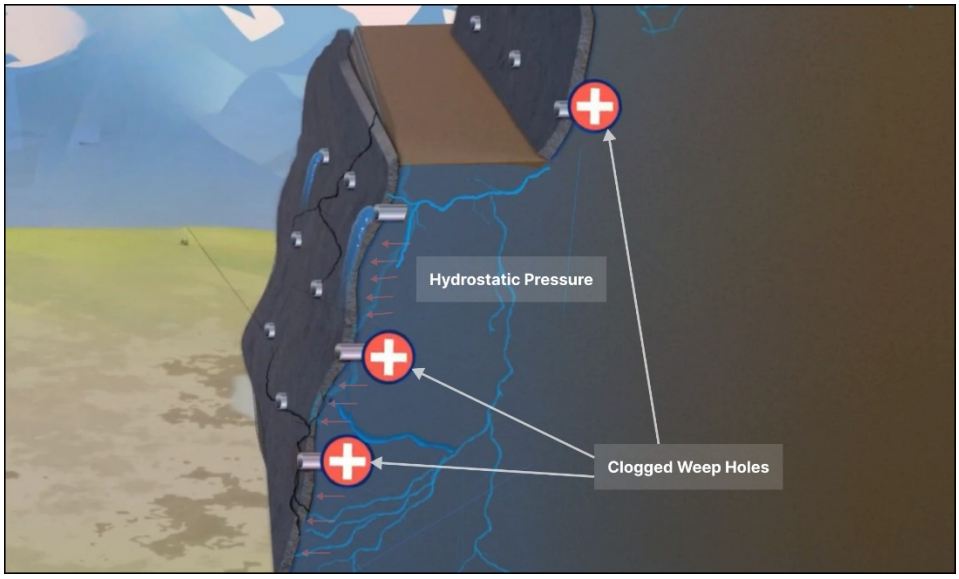
The geotechnical engineer's rationale is that if the slope's geometry is stable against a deep-seated failure, the remaining vulnerability is surface degradation from rain and wind. Shotcrete is chosen as a "hard armor" solution to seal and protect this surface. It is to be noted that this is mostly used in non-vegetative conditions where a grass layer is impossible or not recommended.

Common Issues with the Conventional Approach

The conventional approach, while sound in its initial stability assessment, harbors critical flaws in its assumptions about long-term performance, particularly concerning water management.

The analysis assumes that once a saturated condition is reached, water will be effectively drained away by features like weep holes or strip drains. However, this assumption is often not met in practice. Weep holes can clog, and drainage paths can become blocked over time. When drainage is compromised, the system behaves like an un-drained water tank (see Figure 1). The trapped water exerts significant **hydrostatic pressure** directly onto the back of the impermeable shotcrete facing, a force that is often not fully accounted for in the initial design. This pressure acts in addition to the **internal pore water pressure** within the soil mass.

Figure 1. Hydrostatic Pressure Build-up Behind Impermeable Shotcrete Facing with Clogged Weep holes



This combined water pressure can lead to a **bulging** effect, pushing the shotcrete shell outwards. Standard LEM or Finite Element Method (FEM) analyses do not typically model the flexural stress this bulging induces on a thin (e.g., 10 cm) shotcrete layer, especially one with only light wire mesh reinforcement and no deep anchoring like rock bolts. The system's stability becomes reliant on the shotcrete's limited tensile strength and the pullout resistance of short fixing pins (e.g., 50 cm), which provide minimal resistance against such powerful hydrostatic forces. Furthermore, the connection at the top of the slope often lacks a structural element to resist the vertical and outward stresses, creating a point of weakness. Essentially, the model

fails to account for the very real possibility that the drainage system will fail, turning a protective layer into a dam that facilitates a buildup of destabilizing pressure.

Comparative Analysis and Calculations

To demonstrate the dramatic effect of poor drainage on slope stability, a comparative analysis is presented below.

Assumed Soil Properties (Weakly Cemented Conglomeratic Soil)

- **Unit Weight (γ)**: 19 kN/m3 (unsaturated reference)
- **Effective Cohesion (c′)**: 12 kPa
- **Effective Angle of Internal Friction (ϕ′)**: 30∘
- **Slope Height (H)**: 10 m
- **Target Factor of Safety (FoS)**: 1.3
- **Saturated Unit Weight (γsat)**: 21 kN/m3 (Gs = 2.65 and e = 0.4465 give 21 kN/m3; a degree of saturation of about 0.34 gives the 19 kN/m3 reference)
- **Unit Weight of Water (γw)**: 9.81 kN/m3
- **Wetted-Layer Depth (z)**: 1.457 m measured vertically (1.03 m normal to the slope): the depth to which prolonged rainfall saturates the cut face; the soil below remains unsaturated
- **Pore Pressure and Strength**: u = 0 in the unsaturated case, with suction strength conservatively omitted; c′ and ϕ′ are retained on wetting

This is one plausible hypothetical material, not a tested sample, chosen deliberately modest so that each stage of behaviour can be followed. Production cuts are normally made at 75–80° in ground strong enough to stand at those angles on each bench (rock or strongly cemented conglomerate); the same reasoning applies there, with higher strengths and anchor capacities. This soil itself would not stand at those angles (whole-slope FoS 1.03 at 60°, 0.84 at 75°).

The comparison follows the slope through five stages: **Stage 1**, the cut as excavated; **Stage 2**, prolonged rain with the drainage working; **Stage 3**, water held behind the facing because its drainage has failed; **Stage 4**, movement of the wetted layer with loss of its cementation, where only a facing connected to stable ground can hold it; and **Stage 5**, earthquake. Shotcrete and the geosynthetic system meet these stages differently, and each is assessed against the conditions it actually faces.

The wetted layer is analysed as a shallow translational slip parallel to the face, by effective-stress limit equilibrium per metre width. With slip length L = H / sin β:

$$W = \gamma\, z\, H / \tan\beta \qquad D = W \sin\beta \qquad N = W \cos\beta \qquad U = u\, L$$

$$R = c'L + (N - U)\tan\phi' \qquad FoS = (R + T) / D$$

W is the weight of the sliding layer, D and R are the driving and resisting forces, U is the pore-water force on the slip plane and T is any resisting force supplied by the facing parallel to the slip plane (T = 0 unless stated); by convention the facing force is added to the resisting side. For a given depth, H cancels:

$$FoS = c' / (\gamma\, z \sin\beta \cos\beta) + [1 - u / (\gamma\, z \cos^2\beta)] \cdot \tan\phi' / \tan\beta$$

A circular-slip check of the whole slope (Bishop's simplified method) gives FoS = 1.32 for the as-cut 45° slope. During rain only the surface layer is wetted, and the deep circle remains at FoS = 1.28, so the wetted layer governs the comparison that follows.

We use this force balance to compare the slope angle (β) that gives the target FoS under different drainage conditions (Table 1).

Case 1: Unsaturated and Saturated Conditions (with effective drainage)

1. **Slope Angle (β)**: **45∘** gives FoS = 282.7 / 195.7 = 1.44 for the surface layer and 1.32 for the whole slope (Stage 1, u = 0): the cut stands with the required margin.

2. Saturated Condition (with full drainage):

Here, the soil is saturated, but drainage is efficient, so no excess hydrostatic pressure builds up. The pore water pressure (u) reduces the effective stress, which in turn reduces the frictional resistance of the soil. The analysis accounts for this reduction in shear strength.

Pore pressure on the slip plane is taken as u = γw z cos²β (seepage parallel to the slope within the wetted layer). At 45° (Stage 2): u = 7.14 kPa, U = 101.0 kN/m, D = 216.3 kN/m, R = 236.2 kN/m and FoS = 1.09. The slope stands, but below the 1.3 target. Effective drainage means no water held behind the facing, not zero pore pressure in the saturated layer.

- **Slope Angle for Target FoS (β)**: **34.08∘** gives FoS = 1.30.

Case 2: Saturated Condition with Poor Drainage (2x Hydrostatic Pressure)

In this scenario, we assume the drainage system has failed. Water becomes trapped behind the shotcrete, and the average pore water pressure within the potential failure mass is taken as

double the drained saturated case ($u = 2\gamma_w z \cos^2\beta$). This represents water held behind an impermeable facing whose drains have blocked (Stage 3). This severely reduces the soil's effective stress and shear strength.

The effective stress (σ') is calculated as $\sigma' = \sigma n - u$. If u doubles, σ' is significantly reduced, causing a drastic drop in the frictional component of soil strength, $(\sigma n - u)\tan(\phi')$.

Water held behind the facing is deepest at the toe. A mean slip-plane pressure of twice the drained value (u = 14.29 kPa) corresponds to water standing about 3.1 m above the toe, which presses on the facing with about 30 kPa at the toe. At 45°: U = 202.1 kN/m and FoS = 177.9 / 216.3 = 0.82; the layer slides. Near the toe the water pressure exceeds the weight of the soil above once water rises only about 0.1 m behind the facing, so the lowest 4 m of the layer is lifted; the FoS based on the mean pressure is therefore the representative, conservative value.

- **Slope Angle for Target FoS (β)**: **20.45◦** gives FoS = 1.30.

Table 1. Slope angle giving target FoS = 1.3 vs. drainage condition

Condition	**Pore Water Pressure (Assumed)**	**Slope Angle (β) for FoS = 1.3**
1. Unsaturated (Dry)	u=0	45.00◦
2. Saturated with Effective Drainage	Normal (u = uref = $\gamma_w z \cos^2\beta$)	34.08◦
3. Saturated with Poor Drainage (Pressure x2)	Elevated (u = 2uref)	20.45◦

For the unsaturated case the wetted-layer model gives FoS above 1.3 up to 60°, so the angle is governed by the whole-slope check (FoS = 1.32 at 45°). The saturated angles are flatter-branch solutions of the wetted-layer model, which is not used above about 60°.

The calculations clearly show that while a slope may be designed to be stable at a steep angle (e.g., 34.08◦) for a "worst-case" saturated and drained scenario, a failure of the drainage system dramatically compromises its stability. The safe angle plummets from **34.08◦ down to 20.45◦** (see Table 1), a reduction of 13.63 degrees. A slope cut to 34.08◦ and faced with shotcrete would be at high risk of failure if its weep holes were to clog. This highlights the fundamental weakness of relying on an impermeable facing with a drainage system that is prone to failure over time. A permeable geosynthetic facing, by its very nature, would prevent the buildup of such excessive hydrostatic pressures and provide a far more resilient and reliable solution.

Shotcrete without rock bolts is employed as an erosion control system, primarily to regulate hydrostatic pressure by facilitating or inhibiting water ingress and egress from the rock mass.

The application of shotcrete, reinforced with a 6mm BRC mesh, directly onto a slope face provides inherent resistance to surficial displacement. This facing resistance is a local structural action: it adds slope-parallel resistance only where a connection transfers force to stable ground. Its local strength limits can be quantitatively assessed.

Let's plug in a shotcrete system on the above mentioned slope

- **Shotcrete Panel:** 10 cm thick, C30 concrete, with A142 mesh (6mm bars @ 200mm spacing).
- **Supports:** The panel is held by 50 cm long fixing pins arranged on a **1.5m x 1.5m grid**. These pins are the only points of resistance against the outward pressure. Typical pins are 16 mm ribbed bars driven 0.5 m into the ground; for the strip calculation they are idealized as simple line supports, since short pins do not provide rotational fixity.
- **Loading:** A uniform hydrostatic pressure (P) acts on the back of the panel due to trapped water.

Failure Mode 1: Flexural (Bending) Failure

We analyze a 1m wide strip of the shotcrete panel as a beam spanning 1.5m between fixing pins. The water pressure creates a bending moment, and the panel fails if this moment exceeds its capacity.

- **Effective Depth (d)**: With 30mm cover, d=100mm−30mm (cover)−3mm (half bar dia)=67mm.
- **Steel Area (As)**: For A142 mesh, As=141 mm2 per meter width, placed 30 mm from the outer face. Steel yield strength fyk = 500 MPa (partial factor 1.15) and C30 concrete, fck = 30 MPa, are assumed.
- **Moment Capacity (MRd)**: Using the design resistance of the section with a lever arm of 0.95d:

$$\mathrm{fyd} = 500 \,/\, 1.15 = 434.8\ \mathrm{MPa}$$

$$\mathrm{MRd} = \mathrm{As}\ \mathrm{fyd}\ (0.95\ \mathrm{d}) = 141.4\ \mathrm{mm}^2 \times 434.8\ \mathrm{MPa} \times 63.65\ \mathrm{mm} \approx 3.91\ \mathrm{kN{\cdot}m/m}$$

The nominal capacity As fy (d − As fy / 1.7 f'c b) = 4.64 kN·m/m is an unfactored value and is not used as the design limit.

- **Maximum Applied Moment (Mmax)**: The maximum moment from a uniform pressure (P) on a simply supported 1.5 m span is:

$$\mathrm{Mmax} = \mathrm{P} \times \mathrm{L}^2 / 8 = \mathrm{P} \times (1.5\ \mathrm{m})^2 / 8 = 0.28125 \times \mathrm{P}$$

- **Failure Pressure for Bending (Pflexural)**: The design limit is reached when Mmax = MRd.

$$0.28125 \times \mathrm{Pflexural} = 3.91\ \mathrm{kN{\cdot}m/m}$$

$$\mathrm{Pflexural} = 3.91 / 0.28125 \approx 13.9\ \mathrm{kPa}$$

This means the shotcrete panel reaches its design bending limit once the water pressure behind it exceeds just **13.9 kPa**.

Failure Mode 2: Punching Shear Failure

Here, the water pressure pushes a square section of the panel (1.5m x 1.5m) directly off the fixing pin. The concrete fails in shear in a cone shape around the fixing pin head.

- **Force on a single pin (Fpin)**: Fpin = P × (1.5 m × 1.5 m − Acont) = P × (2.25 − 0.12) = 2.13 × P, where Acont = 0.120 m² is the area inside the control perimeter.

- **Punching Shear Capacity (VRd,c)**: EN 1992-1-1 expression, assuming a 100 mm × 100 mm washer plate, γc = 1.5, ρ = 0.0021 and k = 2:

$$\mathrm{vRd,c} = \max[0.12\ \mathrm{k}\ (100\ \rho\ \mathrm{fck})\text{^}(1/3)\ ;\ 0.035\ \mathrm{k}\text{^}(3/2)\ \mathrm{fck}\text{^}(1/2)] = 0.542\ \mathrm{MPa}$$

$$\mathrm{u1} = 4\mathrm{a} + 4\pi\mathrm{d} = 1.242\ \mathrm{m} \qquad \mathrm{VRd,c} = \mathrm{vRd,c}\ \mathrm{u1}\ \mathrm{d} \approx 45.1\ \mathrm{kN}$$

- **Failure Pressure for Shear (Pshear)**: Failure occurs when Fpin = VRd,c.

$$2.13 \times \mathrm{Pshear} = 45.1\ \mathrm{kN}$$

$$\mathrm{Pshear} = 45.1 / 2.13 \approx 21.2\ \mathrm{kPa}$$

For a 100 mm plate, EN 1992-1-1 and ACI 318 expressions give about 45–81 kN; the EN value is used. Bearing at the plate is not checked; pin pull-out is checked below.

Failure Mode 3: Pin Pull-out

Each pin carries the water pressure on its 1.5 m × 1.5 m tributary area (2.25 m²). Its pull-out resistance is the soil shear strength mobilised along its embedded length:

$$\mathrm{Qpin} = \pi\ \mathrm{d}\ \mathrm{L}\ \mathrm{qs}, \qquad \mathrm{qs} = \mathrm{a} + \sigma'\mathrm{n}\ \tan\ \delta$$

- d = 0.016 m (ribbed bar; failure through the soil), L = 0.5 m; adhesion a = 0.5–1.0 c′ = 6–12 kPa; σ′n ≈ 1.0 × 19 × 0.35 ≈ 6.7 kPa at mid-length; tan δ = tan 20° to tan 30° = 0.36–0.58

$$\mathrm{qs} \approx 8.4\text{–}15.9\ \mathrm{kPa} \qquad \mathrm{Qpin} = \pi \times 0.016 \times 0.5 \times \mathrm{qs} \approx 0.21\text{–}0.40\ \mathrm{kN}$$

$$\mathrm{Ppullout} = \mathrm{Qpin} / 2.25 \approx 0.1\text{–}0.2\ \mathrm{kPa}$$

Even if the pins were grouted into 75 mm holes at a high bond stress of 150 kPa, Qpin = 17.7 kN and Ppullout ≈ 7.9 kPa, still below the bending limit. The 16 mm bar itself (≈ 87 kN) is not critical. Adhesion of the shotcrete to the soil is not credited, because water held at the interface means the shell has already separated from the ground there.

Conclusion: Panel Capacity Limits

The analysis reveals the following capacities for the isolated shotcrete panel:

- **Flexural (Bending) Capacity**: **13.9 kPa**
- **Punching Shear Capacity**: **21.2 kPa**
- **Pin Pull-out Capacity**: 0.1–0.2 kPa (driven pins; up to 7.9 kPa if grouted)

Pin pull-out governs: with typical driven pins, the panel is held essentially by its own weight and its contact with the slope, and any water held behind it exceeds this limit.

Now that the shotcrete inherent shear strength/resistance capacity is calculated, let's use that to calculate the improved FoS if any on the same slope.

Step 1: Calculate the Baseline Unstable FoS

This step remains the same. The underlying soil slope, without any effective structural support, is fundamentally unstable.

$$\mathrm{FoSsoil} = \mathrm{R} / \mathrm{D} = 177.9 / 216.3 \approx 0.82 \qquad (45°, \text{drains blocked, Stage 3})$$

With an FoS of 0.82, the slope is theoretically failing on its own.

Step 2: Calculate the Effective Resisting Force from Shotcrete

The shotcrete panel can only add resistance if it remains intact. First lets find out the effective hydrostatic pressure. It can be determined the below formula. Here γw is the unit weight of water (9.81 kN/m3) and h is the depth below the level of the held water.

$$\text{Upper bound, water standing to the crest: } P = \gamma w\, h = 9.81 \times 10 = 98.1 \text{ kPa at the toe}$$

$$\text{Representative blocked case, water 3.09 m above the toe: } P = 9.81 \times 3.09 \approx 30.3 \text{ kPa at the toe}$$

Even the representative 30.3 kPa exceeds every panel limit: pin pull-out (0.1–0.2 kPa), bending (13.9 kPa), first cracking (17.2 kPa) and punching (21.2 kPa). In addition, the 0.5 m pins lie within the 1.03 m wetted layer and do not reach stable ground, so no slope-parallel resistance is credited to the shotcrete, whether intact or failed.

- Effective Resisting Force from Shotcrete (Teffective) = 0 kN/m

Step 3: Calculate the New Factor of Safety

We use the same driving and resisting forces for the soil as in the original calculation but add the effective resisting force from the failed shotcrete, and include the facing self-weight carried by the sliding layer (24 kN/m3 × 0.10 m × 14.14 m = 33.9 kN/m).

- **Driving Force (FD)** ≈ 240.3 kN/m (soil 216.3 + facing 24.0)
- **Soil Resisting Force (FR)** = c′L + (N − U) tan ϕ′ = 169.7 + (240.3 − 202.1) × 0.577 ≈ 191.8 kN/m
- **New Resisting Force (FR,new)** = FR + Teffective = 191.8 + 0 = 191.8 kN/m
- **New Factor of Safety (FoSnew)**:

$$\text{FoSnew} = \text{FR,new} / \text{FD} = 191.8 / 240.3 \approx 0.80$$

Step 4: Quantify the Improvement

The application of the isolated shotcrete layer did not increase the Factor of Safety; its self-weight reduced it slightly.

- **Percentage Change in FoS**:

$$\%\ \text{Change} = ((\text{FoSnew} - \text{FoSunstable}) / \text{FoSunstable}) \times 100$$

$$\%\ \text{Change} = ((0.798 - 0.823) / 0.823) \times 100 \approx -3.0\%$$

Conclusion of the Reworked Analysis

This more realistic calculation demonstrates that an isolated 10cm reinforced shotcrete layer provides no increase in the Factor of Safety of the wetted layer under adverse saturation conditions.

The shotcrete shell is structurally too weak to withstand the hydrostatic pressure that its own impermeable nature creates. It will fail locally long before it can contribute to resisting a larger, deep-seated slide. This creates a dangerous condition where a seemingly protected slope is, in fact, critically unstable and prone to sudden, catastrophic failure once the brittle shotcrete facing snaps. This reinforces the central argument that preventing pressure buildup with a permeable geosynthetic system is an inherently more resilient and safer strategy.

Consider the influence of shotcrete application on the stability of a slope, exhibiting a 45-degree inclination under unsaturated conditions and a 34.08-degree inclination when saturated. This study will explore the geotechnical implications of this protective measure.

When the 45° slope becomes saturated, the increased soil weight and pore pressure reduce its stability. The additional resistance needed along the slip plane is the **force deficit**: with drainage working it is **44.9 kN/m** to restore FoS = 1.3; with drainage blocked it is **38.4 kN/m** just to prevent sliding (FoS = 1.0) and **103.3 kN/m** for FoS = 1.3.

Here is the step-by-step calculation explaining how we arrive at these values.

Load Transfer Analysis

This scenario assumes the slope is maintained at 45° and becomes fully saturated. This process increases the soil's weight and reduces its internal strength, creating an unstable condition. Any facing must then act as a structural element, connected to stable ground, to resist the impending failure.

Step 1: Baseline State (Unsaturated 45° Slope)

First, we define the forces at play when the 45° slope is dry and stable, with a Factor of Safety (FoS) of 1.44 for the surface layer.

- **Unsaturated Unit Weight (γ_{dry})**: 19 kN/m³
- **Driving Force ($F_{D,dry}$)**: The force pulling the soil wedge down the slope is **195.7 kN/m**.

- **Resisting Force (FR,dry)**: The soil's natural shear strength resisting this movement is **282.7 kN/m**.

- **Factor of Safety**: FoS = 282.7 / 195.7 = 1.44 (u = 0)

Since the resisting force is greater than the driving force, the slope is stable.

Step 2: The Change (Saturated 45° Slope)

Now, the soil becomes saturated. This has two critical effects:

1. **Increased Weight:** The unit weight of the soil increases. We'll assume a saturated unit weight (γsat) of **21 kN/m³**.
2. **Reduced Strength:** Pore water pressure reduces the soil's effective stress, causing a significant loss of frictional strength (u = γw z cos²45° = 7.14 kPa; U = 101.0 kN/m).

Based on our earlier analysis, a saturated slope is only stable at 34.08°. At 45°, its margin falls below the target. The new Factor of Safety of the **soil alone** drops to 1.09.

- **New Driving Force (FD,sat)**: Due to the heavier, saturated soil, the driving force increases to **216.3 kN/m**.
- **New Soil Resisting Force (FR,sat)**: The weakened, saturated soil can now only provide **236.2 kN/m** of resistance.
- **New Factor of Safety (Soil Alone)**: FoS = (Resisting Force / Driving Force) = 236.2 / 216.3 = **1.09**.

The slope still stands (FoS > 1.0), but without the required margin. If the drainage behind the facing then blocks (Stage 3, u = 2uref), R falls to 177.9 kN/m and FoS = 0.82: the layer slides.

Step 3: Calculating the Required Additional Resistance

Any facing or reinforcement would have to provide, along the slip plane and through a connection to stable ground, enough force to cover the deficit between the driving force and the soil's weakened resisting force just to stop the movement (i.e., to achieve an FoS of 1.0) or to restore the target FoS of 1.3.

- **Force Deficit** = Driving Force - Soil Resisting Force
- **Drainage working, to FoS = 1.3** = 1.3 × 216.3 − 236.2 = **44.9 kN/m**
- **Drainage blocked, to FoS = 1.0** = 216.3 − 177.9 = **38.4 kN/m**
- **Drainage blocked, to FoS = 1.3** = 1.3 × 216.3 − 177.9 = **103.3 kN/m**

Conclusion

When the 45° slope becomes saturated, a slip-plane resistance deficit of 44.9 kN/m (drainage working) to 103.3 kN/m (drainage blocked) must be supplied to keep FoS = 1.3. This is not a tensile load in the shotcrete facing: a facing can supply it only through a connection to stable ground beyond the slip plane, which pinned shotcrete does not have.

As we established from before

- **Failure Pressure for Shear (Pshear)**: Failure occurs when Fpin=VRd,c.
  (2.25 – 0.12) × Pshear = 45.1 kN
  Pshear = 45.1 / 2.13 ≈ 21.2 kPa; pin pull-out (0.1–0.2 kPa) governs

The panel limits are pressures normal to the facing, whereas the deficit above is a force along the slip plane. Saturation therefore leads to panel failure through the water held behind it (Stage 3): with typical driven pins, the pull-out limit is exceeded as soon as water stands a few centimetres behind the facing.

The accompanying graph (see Table 2) illustrates the gradual increase in hydrostatic pressure, as water rises behind a shotcrete facing whose drains have blocked, with the facing status and the FoS of the shotcrete-faced layer.

Table 2. Effect of increasing hydrostatic pressure on a shotcrete-faced slope: facing status, resisting/driving forces, and FoS.

Water Pressure at Toe (kPa)	Status of Shotcrete Facing	Resisting Force (kN/m)	Driving Force (kN/m)	Factor of Safety (FoS)	Outcome
0	✅ No water held	248.0	240.3	1.03	Stands, below target
4.9	❌ Pins pulled out	244.1	240.3	1.02	Stands, below target
9.8	❌ Pins pulled out	238.1	240.3	0.99	Layer slides
13.9	💥 Bending limit reached	231.7	240.3	0.96	Layer slides

Water Pressure at Toe (kPa)	Status of Shotcrete Facing	Resisting Force (kN/m)	Driving Force (kN/m)	Factor of Safety (FoS)	Outcome
17.2	❌ Cracked	225.5	240.3	0.94	Layer slides
21.2	❌ Punching limit reached	216.7	240.3	0.90	Layer slides
29.4	❌ Failed & broken	194.4	240.3	0.81	Layer slides
30.3	❌ Failed & broken	191.8	240.3	0.80	Representative blocked case

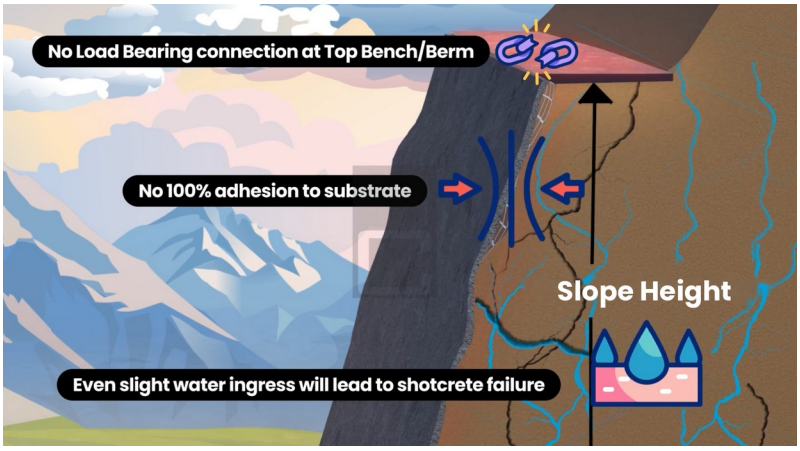
The water level behind the facing rises from 0 to 3.09 m above the toe (pressure at the toe = 9.81 × water level). The FoS uses the mean slip-plane pore pressure for each water level, the full facing weight (33.9 kN/m) and no shotcrete resistance (T = 0). Pin pull-out (0.1–0.2 kPa) is exceeded from the first centimetres of water; bending, cracking and punching limits follow at 13.9, 17.2 and 21.2 kPa. At zero water level the slip plane below the toe still carries a small head, so the FoS is 1.03 rather than the 1.04 of free drainage.

Other issues to consider

The quantitative assessment of the shotcrete system's flexural strength, rigidity, or shear strength could serve as a stability parameter. Nevertheless, the intrinsic rigidity of the system renders it susceptible to internal soil expansion, whether induced by hydrostatic pressure or soil swelling, which invariably results in structural fissuring and compromised integrity. Furthermore, even in the presence of rigidity, should the system be anchored in a zone predisposed to collapse, its rigidity provides no compensatory support due to the absence of a structural nexus to the slope.

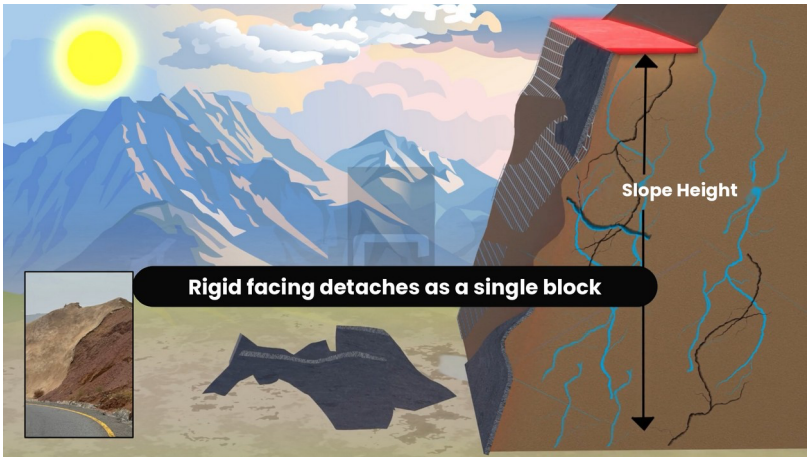
When shotcrete is applied to an unstandardized slope morphology, it tends to aggregate into a cohesive matrix that adheres to the extant slope surface, potentially exhibiting superior cohesive properties relative to the intrinsic cohesion of the facing pedological strata(see Figure 2). Consequently, in the event of a minor displacement, this shotcrete functions as a continuous, destabilizing surcharge upon the slope's planar aspect, precipitating its monolithic detachment rather than providing stabilization.

Figure 2. Behavior of shotcrete on unstandardized slopes: rigid facing acting as a surcharge and detaching as a single block during deep-seated failure.



When a failure plane develops in the soil *behind* the shotcrete layer, the shotcrete's high cohesion and weight transform it from a protective skin into a single, heavy, unstable block (see Figure 3). Instead of preventing a failure, it contributes to a larger, more catastrophic collapse.

Figure 3. Deep-seated failure mechanism in shotcrete-faced slopes: rigid facing detaches as a monolithic block when a slip plane develops behind it.



The Two Failure Modes: Surface vs. Deep-Seated

The key to understanding this is to distinguish between the two types of slope failure.

- **Surface Failure:** This is what shotcrete is designed to prevent. It's the slow raveling and erosion of the outer few centimeters of soil due to rain and wind. Here, the shotcrete acts like a strong, cohesive skin, and it works well.

- **Deep-Seated Failure:** This is the dangerous scenario we described. The failure happens not on the surface, but on a slip plane deep within the soil mass, behind the shotcrete.

Falling Block Failure.

1. **Water Infiltration:** Water seeps behind the shotcrete layer, either through small cracks or from the top and bottom edges.
2. **Soil Weakening:** This trapped water saturates the soil, increasing its weight and pore water pressure, which significantly reduces the soil's internal shear strength.
3. **Failure Initiates:** A "slight slip" begins along a failure plane in the now-weakened soil. At this point, the shotcrete facing is still intact.
4. **A Change of Role:** The shotcrete is no longer reinforcing the slope. It has become a single, rigid, and very heavy slab that is being carried along by the failing soil behind it.
5. **Unfavorable Weight:** The weight of the shotcrete itself now adds to the **driving forces** pushing the slope to fail. It's like putting a heavy concrete backpack on a person who is already falling over—it only makes the fall faster and more severe.
6. **The Collapse:** Because the shotcrete is strong and internally cohesive, it doesn't just crumble. The entire mass—the shotcrete slab and the soil attached to it—collapses together as large, dangerous blocks of rock-like debris.

While shotcrete's cohesion is beneficial against surface erosion, it becomes a liability in a deep-seated failure. By holding the failing mass together as a single block and adding significant weight, it can create a more sudden and hazardous collapse than if the soil were allowed to fail on its own.

This brittle failure is a key reason why modern engineering often prefers permeable geosynthetic systems that prevent water buildup in the first place, thereby avoiding the conditions that lead to this type of failure.

Quality Control issues of Shotcrete.

Here are the usual quality control issues faced with shotcrete systems.

Material & Batching Issues

- **Expired Mix:** To avoid waste, concrete that has passed its usable time limit is sometimes illegally "re-tempered" (water added) and used, severely compromising its final strength.
- **Inconsistent Batching:** Incorrect proportions of cement, water, aggregate, and admixtures result in a product that fails to meet design specifications.
- **Temperature Control:** Mix that is too hot or cold can lead to flash setting, poor bonding, or significantly delayed strength gain.

Application & Workmanship Issues

- **Uneven Spraying:** Inconsistent nozzle technique causes variable thickness, voids (especially behind reinforcement), and weak, unbonded layers called laminations.
- **Poor Reinforcement Encapsulation:** Incorrectly placed wire mesh or rebar can create "shadows" where shotcrete doesn't fill, leaving voids and exposing steel to corrosion.
- **Rebound Mismanagement:** Rebound is material that bounces off the surface during application. Illegally incorporating this inferior, cement-poor material back into the work is a common malpractice.
- **Inadequate Surface Preparation:** Applying shotcrete to unprepared, loose, or dirty surfaces results in poor bonding and delamination.

Post-Application & Testing Issues

- **Improper Curing:** Failing to properly cure the shotcrete by maintaining moisture and temperature leads to a weak surface with high shrinkage cracking.
- **Omission of Standard Tests:** A frequent and critical issue is the failure to perform required ASTM tests on-site. This includes skipping pre-construction test panels to certify the mix and operator, and neglecting to take core samples from the finished wall to verify actual in-place strength and bond.

We need to design a highly adaptive system with robust structural integrity to handle and distribute loads effectively. It must also accommodate irregular volumetric expansions and dynamically adjust to fluctuating pressures without losing function. A key feature will be a "physical and mechanical properties memory and recall" mechanism to store and retrieve data on its deformation history and stress-strain relationships. This enables predictive modeling and self-optimization for sustained performance.

Other attempts at replacing shotcrete

Shotcrete, while effective, has seen limited fundamental advancement and remains a resource-intensive solution. The core process still relies heavily on cement, a major contributor to global CO_2 emissions, and requires significant water and energy for production and application.

The most notable improvement has been the integration of **steel or synthetic fibers** (e.g., polypropylene) into the mix. This enhances ductility and crack control, but it doesn't change the basic fact that shotcrete is a rigid, impermeable, and carbon-heavy system. The core need is for alternatives that offer a longer service life with less environmental impact.

Several technologies have been developed to replace shotcrete, each with its own set of strengths and weaknesses.

Geosynthetic Cementitious Composite Mats (GCCMs)

GCCMs, widely known by the brand name **Concrete Canvas**, are a significant innovation. They consist of a 3D fiber matrix impregnated with a dry concrete mix, all sealed within a waterproof backing and a permeable top layer. When hydrated with water, the mat cures to form a thin, durable, and hard layer of concrete.

While excellent for erosion control in ditches, channels, and on less severe slopes, GCCMs have notable deficiencies when considered as a direct replacement for structural shotcrete on steep slopes:

- **Limited Structural Capacity:** A thin GCCM layer (typically 5-13 mm) does not provide the structural reinforcement needed to resist deep-seated slope failures. It's a surface cover, not a stabilization system.
- **Anchoring Challenges:** While they can be pegged or nailed, they lack a robust system to transfer significant loads from the soil to deep anchors, which is essential for stabilizing a potentially unstable soil mass.
- **Flexibility Issues on Complex Terrain:** While flexible before hydration, draping a GCCM over highly irregular or rocky surfaces without creating voids behind it can be difficult.

Cambridge, Ontario, condominium slope reinforcement project -

A proven platform for our model. The below project executed 35 years ago was a precursor to the idea and has so far been a successful project in this new approach. This was detailed by Colin Alston - *TRANSPORTATION RESEARCH RECORD 1330*

The project involved stabilizing a steep, 18-meter-high hillside of glacial till to allow for the construction of a high-rise condominium. The success of the project can be measured in several key areas:

- **Performance**: The reinforced slope, which used soil nails with a geogrid and geocomposite facing, proved to be stable and satisfactory through two winters and spring thaws.
- **Cost**: The soil nail system resulted in considerable savings, costing **40% less** than the originally proposed reinforced concrete and soldier pile retaining walls.
- **Logistical Solution**: The top-down construction method enabled the project to proceed without needing to encroach on neighboring properties, which had made the original plan impractical.
- **Aesthetics**: The final vegetated slope was considered less visually obtrusive than a concrete wall, satisfying the owner's aesthetic requirements

Introduction to Geosynthetic Facing Reinforced Slope Control

Based on the preceding analysis and more case studies to follow, this section introduces an advanced, multi-component geosynthetic system engineered as a direct replacement for conventional shotcrete.

The design philosophy is fundamentally based on the principles demonstrated by the successful Cambridge, Ontario project, which has served as a precursor for this approach for over 35 years. That case study underscored the critical importance of integrated drainage and the superior performance of flexible geosynthetics over rigid facings, particularly in challenging climates.

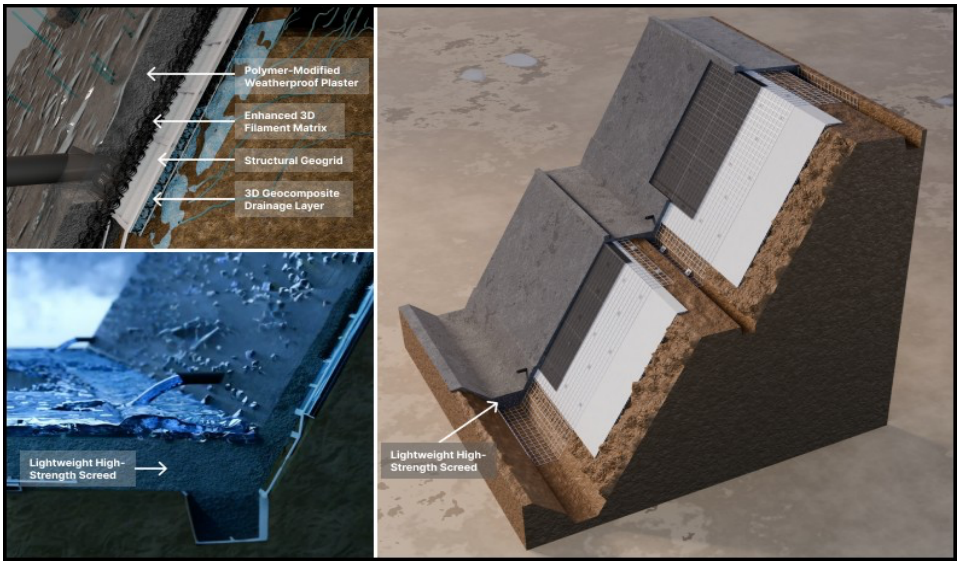
The proposed solution is an engineered composite system that synergistically combines several key functions. It integrates an inbuilt drainage layer to relieve water pressure behind the facing, a high-strength tensile element for structural reinforcement, a durable UV-protective cover, and

a robust mechanical locking mechanism for effective load transfer at the crest and toe of the slope. Collectively, these components form a comprehensive solution for **Geosynthetic Facing Reinforced Slope Control**, designed for superior resilience and long-termperformance.

The system meets the stages of the comparison differently from shotcrete. Its continuous drainage mat, with outlets at 3 m centres, keeps water from being held behind the facing, so Stage 3 reduces to rare local clogging. Its geogrid is pre-tensioned and locked in the bench screed beyond the slip zone, so it can catch a moving layer (Stage 4). And the facing is flexible: it deforms and gives warning rather than cracking.

System Components and Rationale for Selection

Figure 4. Schematic layout of ConstruckSlope™ Plus system components: geogrid reinforcement, drainage layer, 3D filament matrix, plaster coating, and anchorage configuration.



1. High-Strength Structural Geogrid

The primary structural reinforcement is a composite geogrid engineered from high-tenacity, low-creep polyester (PET) yarns. These yarns are bundled into robust bars and co-extruded with a protective polyethylene (PE) coating. The longitudinal and transverse bars are then joined at

their intersections by high-strength integral welding to form a dimensionally stable grid with a consistent, near-square aperture of up to 8 cm.

- **Reason for Selection**: This manufacturing process yields a geogrid with a high tensile modulus, meaning it exhibits very low elongation under working loads, which is critical for immediate slope reinforcement. The grade used in the worked example, ConstruckGrid HS 200/100, has an ultimate tensile strength of 200 kN/m in the machine direction and 100 kN/m across it, with 34 kN/m at 1% and 48 kN/m at 2% strain (EN ISO 10319, product data sheet); grades up to 400 kN/m are available. The stable, welded-junction aperture geometry provides superior interlock with temp fixing pins and concrete compared to strap-based geogrids (e.g., Paralink, Armalynk) and offers a different structural behavior than punched-and-drawn geogrids (e.g., Tensar). The selection was based not only on tensile strength but on the grid's specific anatomy, which ensures efficient load transfer and dimensional stability.

2. 3D Geocomposite Drainage Layer

This component is a geocomposite drain, defined as a high-flow geonet core thermally bonded between two layers of a non-woven, needle-punched geotextile filter. The core is composed of extruded polymer filaments that create a high-void structure capable of transmitting significant volumes of water.

- **Reason for Selection**: This layer provides a continuous and reliable drainage path across the entire slope face, preventing the buildup of hydrostatic pressure—a primary cause of slope failure. The geotextile layers act as filters to prevent soil particles from clogging the drainage core. Thermal bonding of the layers is specified over adhesive bonding to ensure the composite remains intact without delaminating under the shear stresses and environmental pressures experienced on a slope.

3. Enhanced 3D Filament Matrix (Turf Reinforcement Mat)

This component is a three-dimensional Turf Reinforcement Mat (TRM) manufactured from a matrix of entangled and thermally fused polymer monofilaments. This process creates a high-void (typically >90%), open structure. The mat is further reinforced with an integrated, open-mesh uniaxial woven backing for enhanced dimensional stability.

- **Reason for Selection**: The unique, high-void structure provides an exceptional mechanical interlock for the weatherproof plaster, which embeds itself deeply into the

matrix. This creates a strong composite bond and prevents the plaster from slipping or delaminating. The woven backing ensures the mat has a low strain response under load, providing immediate surface stability during installation.

4. Polymer-Modified Weatherproof Plaster

A polymer-modified, fiber-reinforced cementitious plaster is spray-applied in a thin layer (2-3 cm) onto the 3D filament matrix.

- **Reason for Selection**: This layer serves two primary functions. First, it provides long-term UV and mechanical protection for the underlying geosynthetic components. Second, by embedding into the 3D matrix, it forms a durable, semi-flexible, and continuous protective shell that is mechanically locked to the slope face.

5. Lightweight High-Strength Screed

This is a semi-dry, fiber-reinforced screed with a low water-cement ratio, designed for high compressive and flexural strength.

- **Reason for Selection**: Its semi-dry consistency allows for easy pumping and placement, while its high strength-to-weight ratio is ideal for slope applications. This material is used to create mechanical anchor trenches and locking blocks on the benches and at the crest and toe of the slope. The structural geogrid is embedded into this screed, ensuring that the tensile forces generated in the grid are effectively transferred into the stable ground, providing a secure and reliable anchoring mechanism.

Now let's plug this system into the earlier mentioned slope and see the increase in performance.

With the new system's capabilities, the pre-tensioned geogrid anchored in the bench screed provides a tensile reserve of **44 kN/m** (conservative design value; up to **81 kN/m** expected). With drainage working, this raises the saturated slope's Factor of Safety from **1.08** to **1.28** (**1.44** with the expected reserve).
The new system succeeds because it adds significant structural resistance while preventing water from being held behind the facing.

Calculation of the Final Factor of Safety

Here is the step-by-step analysis of the 45° slope as it transitions from a stable unsaturated state to a saturated state, now protected by the new geogrid system.

Step 1: Define the Failing Condition (Without New System)
First, we establish the forces at play the moment the 45° slope becomes fully saturated. As calculated previously, the soil weakens and the driving forces increase, reducing its margin of safety.

- **Driving Force (FD,sat)**: The downward force of the heavy, saturated soil is **216.3 kN/m**.
- **Soil's Resisting Force (FR,sat)**: The weakened, saturated soil can only resist with **236.2 kN/m**.
- **Factor of Safety (Soil Alone)**: The resulting FoS is **1.09**, which means the slope stands but below the 1.3 target.
- **System Self-Weight**: the drainage mat, geogrid, 3D mat and 20–30 mm plaster weigh about 0.62 kN/m² of slope face (8.8 kN/m over the 14.14 m slip length), giving D = 222.5 kN/m, R = 239.8 kN/m and FoS = 1.08 before the geogrid acts.

Step 2: Add the New Protection System's Resistance

The new system introduces a constant and reliable additional resisting force. Crucially, its integrated drainage blanket ensures that no water is held behind the facing, so the soil's resisting force stays at the saturated value with drainage working (drainage does not reduce the slip-plane pore pressure to zero).

- **Additional Resisting Force from Geogrid (bench anchor reserve)**: **44.0 kN/m**

- **Load path**: geogrid → bench screed (grid pull-out from the screed ≥ 93 kN/m, from test) → bench ground beyond the slip zone. The 4 m bench screed with one shear lock gives 44 kN/m as a conservative design value (half the soil cohesion as bond under the screed and a 0.3 m key) and up to 81 kN/m with full bond and a 0.5 m key. The bench’s inward fall of 1–2° aids drainage but is not counted.

- **Geogrid strength**: HS 200/100, ultimate 200 kN/m (machine direction, EN ISO 10319, product data sheet); with an estimated combined reduction factor of 2.0 for creep,

installation damage and durability (an assumption, as no product-specific factors are available), the design strength is 100 kN/m, above the anchor reserve.

- **Pre-tension**: the grid is locked in the bench screed at the top, hung free of wrinkles under a weighted belt along its lower edge (about 0.8 t per metre for 8 kN/m where the belt hangs freely on a steep cut; on a 45° face a heavier belt or a mechanical tensioner is used), and locked by casting the toe screed. The pre-tension is estimated at 8–10 kN/m (8 kN/m used) until it is measured in the full-scale chamber test, so the grid responds from the first millimetres of movement.

- **Tie rods**: used only as temporary fixings during installation; not relied upon.

- **Load transfer from the soil**: the toe trench and toe ground prevent the wetted layer from exiting at the slope foot, so a sliding layer can only fail by bulging through the face. The geogrid, restrained at the bench and the toe trench, resists this bulging by membrane tension transmitted through the drainage mat, so the layer is held by the passive resistance of the toe, which the facing forces it to mobilise (about 121 kN/m with drainage working, about 61 kN/m if water is held at the toe). Interlock between soil and geogrid is not relied upon, and toe drainage is essential.

- **New Total Resisting Force**: the soil's saturated strength plus the bench anchor reserve.

$$\mathrm{FR,new} = \mathrm{FR,sat} + \mathrm{Fgeogrid} = 239.8\ \mathrm{kN/m} + 44.0\ \mathrm{kN/m} = 283.8\ \mathrm{kN/m}$$

Step 3: Calculate the Final Factor of Safety

With the new total resisting force, we can now determine the final Factor of Safety for the saturated 45° slope with drainage working (Stage 2).

- **Final Factor of Safety (FoS)**:

$$\mathrm{FoS} = 283.8\ \mathrm{kN/m}\ /\ 222.5\ \mathrm{kN/m} \approx 1.28 \quad (\text{conservative reserve})$$

$$\mathrm{FoS} = (239.8 + 81.0)\ \mathrm{kN/m}\ /\ 222.5\ \mathrm{kN/m} \approx 1.44 \quad (\text{expected reserve})$$

Exactly 1.3 requires 49.4 kN/m: the conservative reserve gives 1.28, just below, and the expected range covers it. For any wetting narrower than the full slope, the surrounding soil adds resistance on the sides of the wet zone (below), which lifts the result above 1.3.

Stage 3 for this system: local clogging of the drainage mat

With a continuous mat and outlets at 3 m centres, a blocked outlet diverts water to its neighbours, so a full-width blockage is not a realistic condition for this system. As a robustness check, a local patch of clogged mat is considered with water held as in the shotcrete case (u = 2uref); the demand for FoS = 1.3 is then 107.8 kN/m per metre. A wet patch of limited width is also held by the shear strength of the drier soil on its two sides, about 2 × c′ × L × 1.03 m ≈ 350 kN in total. In the first few centimetres of movement, while that side resistance is at its peak, the pre-tensioned grid adds about 10 kN/m. Together they hold a clogged patch up to about 3.6 m wide at FoS = 1.3, wider than the 3 m outlet spacing, against about 2.9 m for a shotcrete facing, which has only the side resistance.

Stage 4: progressive check – the grid catching a moving layer

Passive reinforcement acts only once the soil begins to move. To test the reserve beyond the limit-equilibrium values, the wetted layer is assumed to slide and to lose all cementation along its base, keeping only friction. As it moves, the grid tension rises from the 8 kN/m pre-tension (stiffness from 48 kN/m at 2% strain over the 14.14 m face; the data sheet gives 34 kN/m already at 1%, so the grid is stiffer at small strain and this assumption is conservative) and the toe resistance builds up over the first 5 cm.

- **Drainage working**: friction (70.1 kN/m) and toe resistance (121 kN/m) leave 31.4 kN/m for the grid, which it reaches after about 14 cm of movement, within the 44 kN/m anchor reserve and at one third of the grid's design strength. The layer is caught.

- **Full-width blocked drainage**: the grid would have to carry about 150 kN/m, within its ultimate strength but above the single-lock bench anchor. This case is excluded by the drainage design rather than by the grid.

The movement needed to engage the grid gives visible warning, and the plaster should be inspected and repaired after such an event. The full-scale chamber test is planned to measure the movement at arrest, the grid tension and the bench and toe reactions.

The result is a slope that stands with drainage working, holds local wetting at FoS ≥ 1.3, and is caught by the grid if its cementation breaks. This demonstrates the system's effectiveness in both reinforcing the soil and managing water, which is the key to long-term stability.

Other perceived issues that can cause problems to shotcrete is Bulging effect due to soil saturation.

The bulging effect is governed by the water pressure held behind the facing (Stage 3) and the stiffness of the shotcrete panel:

- **Water pressure at the toe**: about 30.3 kPa in the representative blocked case (98.1 kPa if water stood to the crest)
- **Panel**: 100 mm C30 shotcrete, 1.5 m span between pins, mean tensile strength fctm = 2.9 MPa, Ecm = 33 GPa

Simplified Calculation

The panel strip is treated as a 1 m wide beam simply supported between pins, consistent with the bending check.

1. **Cracking Moment (Mcr)**: $M_{cr} = f_{ctm} \times I / (h/2) = 2.9\ \text{MPa} \times 8.33 \times 10^{-5}\ \text{m}^4 / 0.05\ \text{m} = 4.83\ \text{kN·m/m}$
2. **Cracking Pressure (Pcr)**: $P_{cr} = 8 M_{cr} / L^2 = 8 \times 4.83 / 1.5^2 \approx 17.2\ \text{kPa}$
3. **Deflection at First Cracking (δcr)**: $\delta_{cr} = 5 P_{cr} L^4 / (384 E_{cm} I) \approx 0.41\ \text{mm}$

Interpretation

The representative water pressure of 30.3 kPa at the toe is almost twice the cracking pressure, and the panel cracks after only about 0.4 mm of outward movement. For a rigid, brittle material like shotcrete, the outward push of held water is therefore more than sufficient to cause cracking and the structural failure previously calculated.
Shotcrete offers **almost no flexibility** to withstand this type of movement. As a brittle material, it will crack and fail under very small deformations.
The failure of the 10 cm thick shotcrete panel therefore begins at a bulge of approximately **0.4 mm**, long before any bulge becomes visible.

Connection Strengths: Bench and Trench Anchorage

A critical aspect of ensuring the integrity of the geosynthetic-facing slope protection system is the anchorage of the geogrid into structural elements such as berm concrete and trench locks. The geogrid (HS 200/100, 200 kN/m ultimate) is not the weak link; the anchorage must transfer the reserve force into stable ground without slip.

Bench Interlock Anchorage

To validate the grid-to-screed connection, a pull-out test was conducted on geogrid embedded in fiber-reinforced concrete screed. A single geogrid aperture (67 mm × 61 mm) was embedded in a Diamond/Habe Polyfiber Screed block and loaded in tension until slip occurred. The test was performed on one specimen by an in-house method and certified by Lonestar Alpha Laboratories (report LMMET-1903198, 21 October 2019).

- Observed Pull-Out Strength (Single Aperture, approx. 67–75 mm Embedment):
  6.25 kN before slippage occurred
  Refer to Appendix H: Bench Screed Interlock Test Report.

- Normalized Capacity:
  Scaling the single aperture across a 1 m width only, with no extrapolation in length (67 mm aperture pitch):

$$\mathrm{R_{grid-screed}} \approx 6.25\ \mathrm{kN} \times (1000\ /\ 67) \approx 93\ \mathrm{kN/m}$$

- Comparison with the bench anchor:

$$93\ \mathrm{kN/m\ (grid\ into\ screed)} > 44\ \mathrm{kN/m\ (screed\ into\ bench,\ design\ value)}$$

The grid therefore pulls out of the screed only at more than twice the bench anchor's design capacity, even at the short tested embedment; in the bench, the grid runs across the full 4 m width. The governing link is the screed bearing on the bench ground (44 kN/m design, up to 81 kN/m). This paper relies on the single specimen; further specimens are listed under Research Gaps.

Trench Anchorage

At the toe of the slope, the geogrid is anchored into a 50 cm x 50 cm trench lock filled with high-strength fiber screed or C25 concrete. The geogrid is inserted in an L-shape, providing a total embedment length of 1.0 m (0.5 m on each side). The trench locks the pre-tension and forms the toe lock of the facing; its resistance is part of the toe resistance described above.

Table 3. Anchorage design summary: capacity of each link in the load path (4 m bench, one shear lock)

Parameter	Value
Geogrid ultimate strength (HS 200/100)	200 kN/m (design 100 kN/m)
Grid into screed (test, width-scaled)	≥ 93 kN/m
Screed into bench (calculated)	44 kN/m design (up to 81 kN/m)
Governing reserve	44 kN/m (bench anchor)
Pre-tension locked in	8–10 kN/m, estimated (8 kN/m used)

The bench and trench anchorage system, using the 4 m bench screed with one shear lock, provides a quantifiable load path with a conservative design capacity of 44 kN/m, above the 31.4 kN/m needed to catch a moving layer with drainage working (Stage 4 check above). Full-scale tests are required to confirm the capacity (see Research Gaps).

Refer to Appendix H for detailed pull-out test reports and Appendix I for anchorage design calculations.

Primary Effects of Traffic Vibration

A precise numerical calculation of the vibrations' effect isn't possible without highly specific data on traffic loads, distance from the slope, and the soil's dynamic properties. However, a robust engineering assessment can be made based on the system we've designed.

The result of vibrations from heavy traffic would be a **minor, temporary reduction in the Factor of Safety (FoS)**. With the static FoS of 1.28 calculated above (drainage working, conservative reserve), the protected layer remains well above 1.0 for the acceleration below. Traffic vibrations impact a slope in two main ways:

1. **Increased Shear Stress**: The ground shaking temporarily adds a horizontal force to the soil mass. This momentarily increases the "driving forces" that promote failure.
2. **Liquefaction Potential**: In specific soils (typically loose, uniform, saturated sands), cyclic vibrations can cause a rapid increase in pore water pressure, leading to a sudden loss of strength known as liquefaction.

Effects of Vibrations on the slope due to traffic

We can assess both effects for our 45-degree slope, which is composed of **the assumed weakly cemented conglomeratic soil** and protected by the new geosynthetic system.

1. Reduction in Factor of Safety

We can estimate the impact of the increased shear stress using a **pseudostatic analysis**. This method converts the dynamic shaking into a simple, static horizontal force. A horizontal acceleration coefficient (kh) of 0.05 is assumed as a sensitivity input. Traffic accelerations at a slope are normally well below the design earthquake, so the earthquake check below governs. The analysis shows this horizontal force would cause a temporary reduction in the Factor of Safety:

$$\mathrm{Dh} = \mathrm{W}\,(\sin\beta + \mathrm{kh}\cos\beta) = 314.7 \times (0.7071 + 0.05 \times 0.7071) = 233.6\ \mathrm{kN/m}$$

$$\mathrm{Nh} = \mathrm{W}\,(\cos\beta - \mathrm{kh}\sin\beta) = 314.7 \times (0.7071 - 0.05 \times 0.7071) = 211.4\ \mathrm{kN/m}$$

$$\mathrm{FoSdyn} = [\mathrm{c'L} + (\mathrm{Nh} - \mathrm{U})\tan\phi' + \mathrm{T}] \,/\, \mathrm{Dh} = (169.7 + 63.7 + 44.0) \,/\, 233.6 \approx 1.19$$

- **Static FoS (Saturated, no vibrations)**: **1.28**
- **Dynamic FoS (With traffic vibrations)**: **≈ 1.19**

Even during heavy traffic events, the slope maintains a robust FoS well above the failure threshold of 1.0. The reduction is about 7% and only occurs for the brief moment a heavy vehicle passes.

2. Liquefaction Risk

The risk of liquefaction for this slope is expected to be low, but the adopted c′, ϕ′ and unit weights do not establish density, grading or cyclic resistance, so no quantitative conclusion is drawn here. This phenomenon is a concern for loose, saturated, uniform sands; liquefaction susceptibility requires appropriate soil characterization and cyclic-loading assessment.
The vibrations from adjacent heavy traffic would not compromise the stability of the protected slope. The primary effect is a minor, transient dip in the Factor of Safety, which remains well within safe limits. Furthermore, the flexible, interlocking nature of the geosynthetic-reinforced soil mass is expected to tolerate these vibrations better compared to a rigid shotcrete shell, which could be prone to fatigue cracking from long-term cyclic loading.

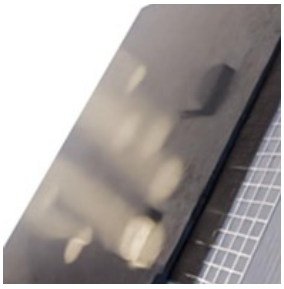
Earthquake Check (Stage 5)

The project sites in Appendix G have design peak ground accelerations of 0.08–0.12 g. Taking half of these as the horizontal pseudostatic coefficient (Hynes-Griffin and Franklin, 1984) gives kh = 0.04–0.06. FoS of the wetted layer (static → kh = 0.04 → kh = 0.06):

- **As-cut slope, dry (Stage 1)**: 1.44 → 1.37 → 1.33
- **Rain, drainage working, ConstruckSlope with 44 kN/m reserve**: 1.28 → 1.20 → 1.17
- **Same, pre-tension only (8 kN/m)**: 1.11 → 1.05 → 1.02
- **Rain, drainage working, shotcrete**: 1.04 → 0.98 → 0.95

The as-cut slope remains stable in the design earthquake. If an earthquake occurs during rain, the geosynthetic-faced layer stays above 1.0 even before its reserve is mobilised, while the shotcrete-faced layer falls below 1.0 even with its drains working. Earthquake and prolonged rain together are a severe combination, shown here for comparison.

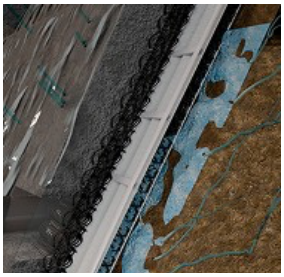
Other benefits/features of the system that helps it function properly or to address the deficiency of the system



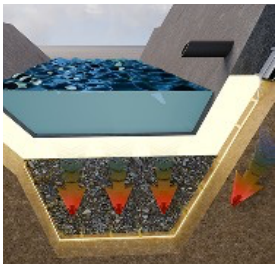
UV protection - The weatherproof plaster coating shields the geosynthetic materials from harmful sun rays, preventing degradation and maintaining system strength over decades.

Plaster as UV Protection Only – The polymer-modified plaster layer serves solely as a UV and surface protection coating. It is **not relied upon for tensile strength** to resist driving forces. Therefore, minor surface cracks do not compromise structural performance, and any water ingress is harmless due to the geosynthetic core. This characteristic makes the system particularly well-suited for **marine and coastal environments**, where durability against moisture and salt exposure is critical.

External erosion control - The interlocked erosion control mat with weatherproof plaster forms a protective barrier that resists rainfall and runoff, preserving slope integrity even in extreme weather.



Internal erosion control - The integrated geotextile filter (opening size O90 = 90 µm) is selected to retain the soil while allowing water to pass, preserving slope strength and long-term stability; filter compatibility is to be confirmed for each site's grading.



Scour protection benefits - Strategic concrete anchoring positioned below potential scour depths prevents water from undermining the system foundation, ensuring long-term stability.

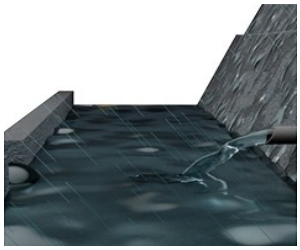


Long life due to no corrosion - Geosynthetic materials resist rust and deterioration, giving a long design life compared to steel-based systems (75–120 years targeted, to be confirmed by durability testing).

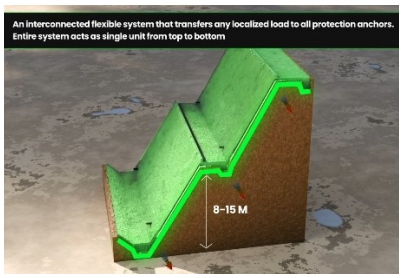
Longer creep time due to less time for geogrid in active state - Efficient load distribution reduces constant stress on geogrid components ensuring long-term slope stability.

Easy maintenance - Pre-engineered modular components enables targeted inspection and component replacement without dismantling entire sections, reducing maintenance costs and system downtime.

Environment friendly due to low use of resources - The system is estimated to use about 50% less concrete, water and energy than shotcrete (see Appendix F), reducing environmental impact.

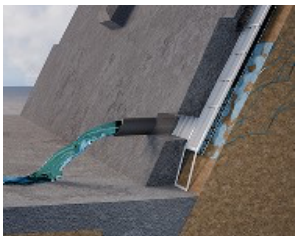


Water drainage on berm due to use of semi-dry mix with fibers - Fiber-reinforced concrete berms create efficient drainage channels that actively direct water away from critical areas, preventing water buildup and ensuring long-term stability.



Benefits of continuous interconnected system - Only a flexible layer with rigid connectors can effectively transfer loads over uneven slope surfaces. This is crucial as the loads at any part of the slope is transferred to the entire system including the berm anchors and allows to increase mobilized resistance to support extra failure loads while reducing localized overstress.

How the anchor trenches work to protect the slope in a simple way with help of gravity. - Gravity-based anchor systems transfer slope loads to stable ground areas, providing reliable restraint without requiring expensive deep foundation or rock bolting solutions.



Efficient connection between drainage mat and drainage pipes - Engineered connection interfaces ensure continuous water flow from drainage mats to collection systems, maintaining optimal water management and preventing pressure buildup that could compromise slope stability.

High temp viability to ensure that the product used can be used in arid and hot conditions. -The system is UV protected by a special plaster coat, which protects the geosynthetic materials from sun exposure and high temperatures. This unique feature ensures the product can be used in arid and hot conditions without compromising performance.

Why a factory tested component system helps a better site control giving birth to the idea of prepackaged pre-tested modular slope protection system. - Each element is pre-tested before installation, giving consistent site quality and reliable results—removing risks from field variability.

Presence of any chemical in the soil and how it affects the system - The geosynthetics and protective layers resist damage from typical soil chemicals, maintaining strength and durability in varied ground conditions.

Vibration imparted by the heavy vehicles on the road transferring it to the system and its effect - The flexible geosynthetic design is expected to tolerate traffic-induced vibrations without structural damage, maintaining system integrity under repeated dynamic loads.

Why component selection is important in the geosynthetic system

It’s a common mistake that any products can be selected as long as the technical data sheet values match. However in this unique application its not the case.

We encountered various site complications and interaction of joints issues when tried with multiple various of the same components. Several tests and sampling was done before a final production selection that can ensure a system without weak links.

Geogrids

The geogrid in this system does more than carry tension down the slope. It is wrapped around the rebar and cast into the bench and toe screeds, it holds the drainage mat and the 3D mat against the face, and it carries load across the slope as well as down it. For this it must have near-square (proportional) apertures, with ribs in both directions at similar spacing, and strength in both directions (ConstruckGrid HS 200/100: 200 kN/m down the slope, 100 kN/m across it).

Long-strip uniaxial geogrids, made to be buried in layers of reinforced soil, have long, narrow apertures and cannot be used as a facing:

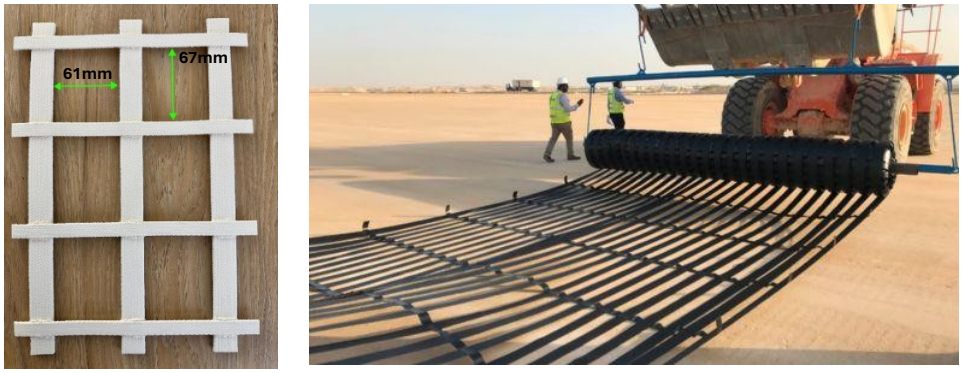
- **Pull-out from the screed:** the grid is held in the screed by its transverse ribs bearing on the concrete and on the rebar loop. Near-square apertures place many ribs within the embedded length; long apertures place only one or two, so the required pull-out strength cannot be developed.

- **Support of the facing:** between widely spaced bars, the drainage mat and the 3D mat are unsupported and bulge under soil and water pressure.

- **Strength across the slope:** the strength runs along the strips only, so a local wet or moving patch cannot draw on the surrounding facing.

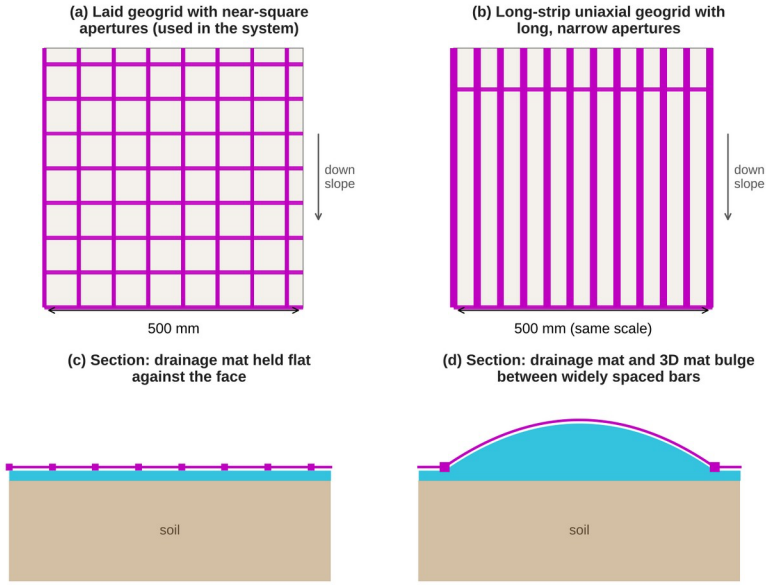
The two types are compared in Figure 5.

Figure 5. Comparison of geogrid aperture sizes



Left: laid geogrid with near-square apertures, as used in the system. Right: long-strip uniaxial geogrid with long, narrow apertures, which is not suitable as a facing.

The two types drawn at the same scale, in plan and in section:



Geogrid shown magenta, drainage mat cyan. Schematic, not to scale except where marked.

It is also noted that the Geogrid must be low elongation type and hence only joints that doesn't cause elongation due to lateral load is considered.

Drainage Mat

The permeability (K value) of the drainage element is important to avoid an impermeable barrier. Fine particles carried by seepage can form a low-permeability layer on a geotextile whose openings are too small, while openings that are too large let the soil wash out. The opening size (O90 = 90 µm) is therefore chosen to satisfy both retention and permeability criteria for the soil being protected.

We also explain what kind of 3D filament matrix gives the best UV protection and crack resistance and method to install it properly and some installation values to other 3d filament matrix(spaghetti filaments punched and modelled not suitable)

The Enhanced 3D Filament Matrix

The manufacturing of a reinforced erosion control mat is a continuous, integrated process. It begins with the extrusion of polypropylene monofilaments, which are then randomly deposited while hot onto a moving conveyor. As these filaments entangle and cool, they thermally fuse together to form a thick, three-dimensional matrix.

To create the reinforced version, a high-strength element like a geogrid is fed into the production line, and the hot filaments are deposited directly onto it. This causes the matrix to thermally bond to the reinforcement as it cools, creating a single, inseparable composite material that is then trimmed and rolled.
However the below manufacturing method of the product is to be avoided –
The manufacturing of a reinforced erosion control mat is a continuous process that begins with extruding polypropylene monofilaments. These hot filaments are randomly deposited onto a moving conveyor, entangling and thermally fusing as they cool to create a thick, three-dimensional matrix.
The selection of the manufacturing method for the 3D matrix is critical to the system's performance, as it directly impacts the bond with the plaster layer and the system's dimensional stability.

Entangled Monofilament Matrix (Selected Method)

The superior performance of the first manufacturing method is attributed to the **high mechanical interlock** achieved between the plaster and the three-dimensional, entangled monofilament matrix. While the component's primary function is not traditional turf reinforcement, its manufacturing process yields a structure with ideal properties for this specific application as a high-bond-strength substrate. A key feature of this manufacturing method, particularly when integrated with reinforcement, is its **high dimensional stability**, resulting in a low strain response under load. This stability provides a static substrate, allowing the plaster to cure monolithically without being subjected to damaging differential movement, thus ensuring a crack-free, durable finish.

Thermoformed Matrix (To be avoided)

Conversely, the thermoformed (second) method produces a structure that exhibits **higher initial elongation** under low loads. This occurs because the three-dimensional dimples or cuspates tend to flatten or deform before the material's full tensile strength is engaged. When a cementitious plaster is applied to such a thermoformed matrix, its self-weight can induce this initial elongation. This movement during the critical hydration and curing phase of the plaster can lead to the formation of **microcracks**, compromising the integrity and weatherproof function of the finished surface.

Research Gaps

Here are the primary research gaps that would need to be addressed for its widespread adoption and inclusion in engineering design codes.

Long-Term Durability and Degradation

- **UV Degradation of the System:** While the plaster provides a UV shield, there is a lack of data on the long-term performance of the underlying 3D filament matrix and structural geogrid if the plaster develops cracks or spalls over time. Accelerated aging tests on the complete composite system are needed.
- **Long-Term Creep and Fatigue:** The paper assumes low creep for the geogrid, but the long-term creep and fatigue behavior of the *entire system* under sustained and cyclic loads (like traffic) is unknown. This includes the load transfer mechanism between the geogrid and the screed anchor.
- **Biological and Chemical Degradation:** Research is needed to understand the effects of long-term exposure to soil microbes, root intrusion, and various chemical compositions in groundwater on the polymer components.

Component Interaction and Performance Limits

- **Plaster-Matrix Bond Strength:** The mechanical interlock between the weatherproof plaster and the 3D filament matrix is a core concept. This interface needs to be quantified through laboratory direct shear testing to understand its bond strength, especially under wet/dry and freeze-thaw cycles.
- **Drainage System Clogging Potential:** While the system is designed to be free-draining, long-term field studies are needed to assess the potential for clogging of the geocomposite drain from fine soil particles (fines) or biological growth (biofilm), particularly in challenging soil types.
- **Full-Scale Anchorage Performance:** The theoretical calculations for the "trenchlock" and bench-screed anchors should be validated with full-scale pullout tests to confirm the actual anchorage capacity and failure mechanisms in different soil conditions.
- **Grid-to-Screed Pull-out Test Series:** The grid-to-screed value (6.25 kN per aperture, 93 kN/m) is based on a single specimen with a 67 mm × 61 mm aperture. A series of at least three specimens of ConstruckGrid HS 200/100 at the actual bench embedment is required to confirm it statistically.

- **Full-Scale Chamber Test of the System:** A full-scale chamber test is required to measure the values that this paper estimates or calculates: the pre-tension retained after the toe screed is cast (estimated at 8–10 kN/m), the movement at which the grid arrests a sliding layer (calculated at about 14 cm), the grid tension at arrest (calculated at 31.4 kN/m), and the reactions at the bench anchor (44 kN/m design, up to 81 kN/m) and at the toe (about 121 kN/m with drainage working).

Proposed Numerical Modeling and Further Studies

To rigorously validate the performance of the proposed multi--component system, advanced numerical modeling is required. This goes beyond conventional Limit Equilibrium analysis and necessitates a coupled hydro-mechanical Finite Element Method (FEM) approach, utilizing platforms capable of simulating complex soil-structure-water interactions.

Factors for Numerical Investigation

The following critical questions must be addressed through the FEM analysis to quantify the system's performance under operational stresses:

For each question, the basis used in this paper is stated below it. These are limit-equilibrium values or stated assumptions; the FEM analysis and the full-scale chamber test are to replace them with calculated and measured values.

- **Transient Hydraulic Response:** What is the **transient pore water pressure regime** within the soil mass during a design rainfall event, considering the high-capacity drainage provided by the geocomposite? This will quantify the *residual* hydraulic load on the system and the degree of saturation.
  Current basis in this paper: not yet modelled. Steady seepage parallel to the slope is assumed within a 1.03 m wetted layer (u = 7.1 kPa with drainage working) and, for blocked drainage, twice that value (water about 3.1 m above the toe, 30 kPa at the toe).
- **Soil Volumetric Strain:** What are the **volumetric strains and associated stresses** induced by saturation-desaturation cycles within the soil, and how are these stresses transferred to the flexible facing?
  Current basis in this paper: not assessed. c' and ϕ' are assumed to be retained on wetting, with no loss of strength from wetting and drying cycles.
- **Filtration Stability:** What is the potential for **internal erosion or suffusion** (loss of fine particles) at the soil-geotextile filter interface under high hydraulic gradients, and does this affect the long-term stability of the retained soil?

Current basis in this paper: the geotextile opening size O90 = 90 µm is selected for retention and permeability; its compatibility with the grading of each site soil is still to be confirmed.

- **Effects of Partial Saturation:** Does the drainage system maintain the soil in a **partially saturated state**, and can the resulting **soil suction (negative pore water pressure)** be reliably quantified to account for an increase in apparent cohesion and overall slope stability?
  Current basis in this paper: suction is conservatively ignored (u = 0 in the as-cut case, no apparent cohesion from suction). Quantifying it would increase the calculated safety factors.
- **System Stress Distribution:** What is the **distribution of mechanical stress** throughout the system's components (geogrid, plaster facing, anchors), and where are the primary **stress concentration points** located?
  Current basis in this paper: the bench anchor governs at 44 kN/m design value (up to 81 kN/m); the grid into the screed carries at least 93 kN/m and the grid has a design strength of 100 kN/m; the toe resists about 121 kN/m with drainage working; a grid force of about 31.4 kN/m arrests a sliding layer. The highest demands are therefore at the bench anchor and the toe lock. The distribution along the grid and around the anchors is still to be modelled.
- **Resultant System Strain:** What are the **resultant strains and deformations** in the flexible geosynthetic elements under the combined hydraulic and mechanical loads, and do they remain within acceptable long-term serviceability limits?
  Current basis in this paper: a sliding layer is arrested after about 14 cm of movement, at a grid strain of about 1% (31.4 kN/m against 48 kN/m at 2%); the estimated pre-tension of 8 kN/m corresponds to about 0.2% strain. Serviceability limits for the facing are still to be defined.

Answering these questions requires a sophisticated **coupled hydro-mechanical Finite Element model**. This numerical approach must integrate: 1) transient groundwater flow, 2) non-linear soil mechanics, and 3) the structural behavior of the multi-component geosynthetic system. Such a model will allow for a holistic evaluation of both the global slope stability and the serviceability of the facing system under realistic operational loads.

Required Outputs from Modeling

The modeling should produce the following quantifiable results, expressed in standard geotechnical units:

This paper already gives limit-equilibrium values for each output (shown in brackets); the FEM model is to confirm them.

- **Unprotected Slope Failure Load:** The shear load along the critical failure surface required to cause collapse of the unprotected slope, expressed in **kN per linear meter** (this paper: driving force 216.3 kN/m for the saturated 45° example; soil resistance 236.2 kN/m with drainage working and 177.9 kN/m with drainage blocked).
- **System Resisting Force:** The mobilized tensile resistance provided by the geogrid reinforcement system to counteract the failure load, expressed in **kN per linear meter** (this paper: 44 kN/m design value, up to 81 kN/m).
- **Final System Performance:** The final **Factor of Safety** of the protected slope, accounting for the full interaction of the soil, hydraulic forces, and geosynthetic components (this paper: 1.28 with drainage working, 1.44 with the maximum reserve).
- **Design Safety Margin:** A quantitative comparison of the mobilized resisting force versus the capacity of each link in the load path (grid 100 kN/m design, 200 kN/m ultimate; bench anchor 44–81 kN/m) to evaluate the system's **reserve strength**.

Final Conclusion

The analyses presented herein show that a shotcrete facing is vulnerable to the one condition its impermeable surface creates: water held behind it. In the illustrative 45° slope, which stands as cut (whole-slope FoS 1.32) and in rain with its drains working (FoS 1.04 with a shotcrete facing), blocked drains raise the water pressure at the toe to about 30 kPa, far above the panel’s pin pull-out, bending and punching limits, and reduce the FoS to 0.80. Pinned shotcrete has no load path to stable ground, so it offers no reserve once the soil begins to move, and in an earthquake during rain it falls to 0.95.

A multi-component, geosynthetic-faced system meets each stage differently. Its continuous drainage mat with outlets at 3 m centres keeps water from being held behind the facing, and its pre-tensioned geogrid, anchored in the bench screed beyond the slip zone, provides a tensile reserve of 44 kN/m (conservative design value; up to 81 kN/m expected). With drainage working it raises the slope from FoS 1.08 to 1.28 (1.44 with the expected reserve); a clogged patch up to about 3.6 m wide is held at FoS 1.3 with the help of the surrounding soil; if the wetted layer

loses its cementation and slides, the grid catches it after about 14 cm of movement; and in the design earthquake during rain it remains above 1.0 before its reserve is mobilised. Full-width blocked drainage would require a larger bench anchor and is excluded by the drainage design. Production cuts at 75–80° are made in stronger ground, where the same reasoning applies with higher capacities, and the full-scale chamber test is planned to confirm the engagement of the grid.

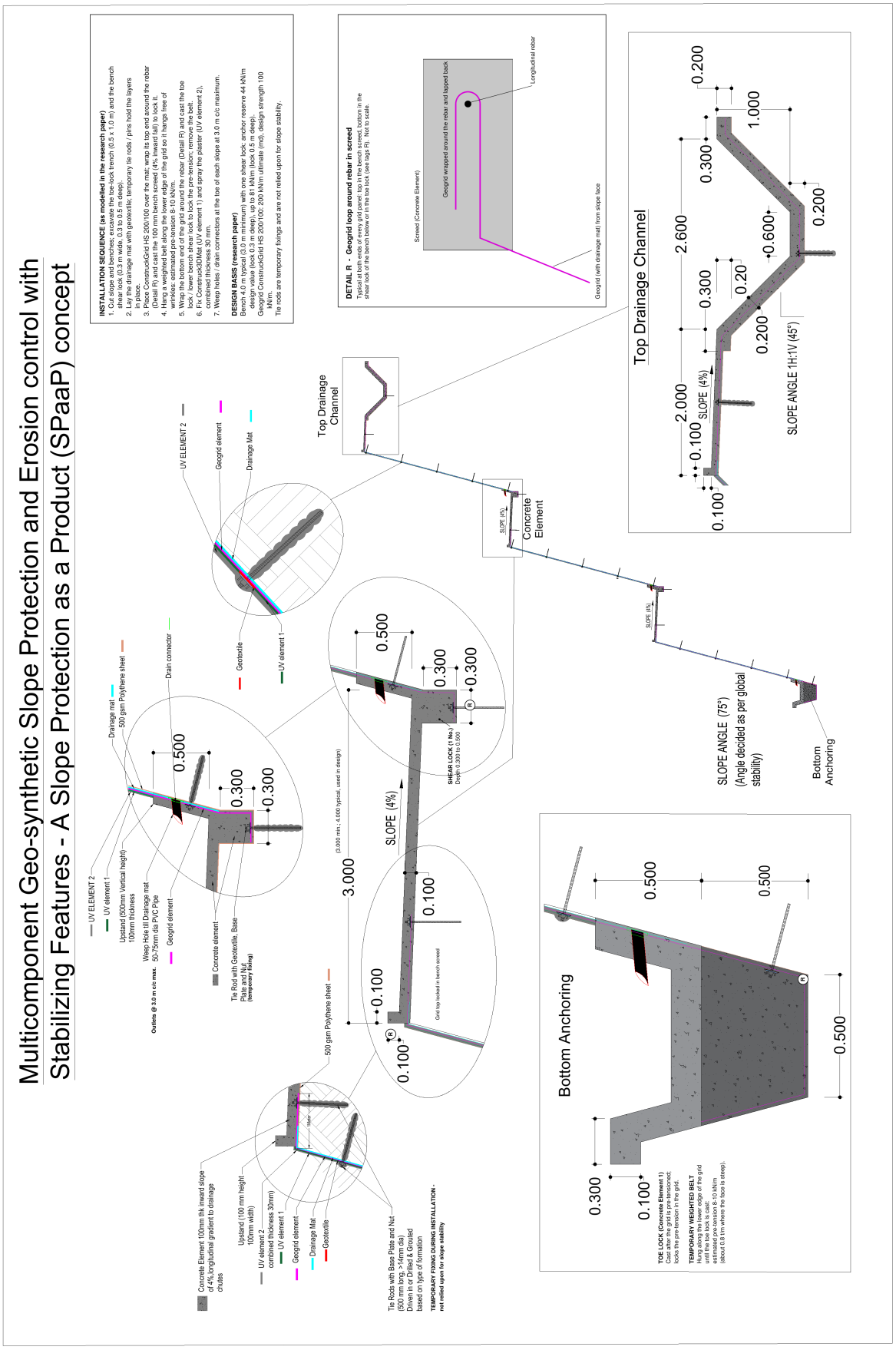
The successful implementation of such an advanced system is, however, contingent upon meticulous design, appropriate material selection, and adherence to proven installation methodologies as outlined in this paper.

These engineering principles and analyses have culminated in the development of a geosynthetic-faced erosion control system designed to these principles, named **ConstruckSlope**, by Construck Systems & Technology. The extensive project history associated with ConstruckSlope serves as a valuable repository of performance data, providing historical validation for the technical success of this advanced approach to slope stabilization.

Appendices

- **Appendix A:** Standard Drawing of ConstruckSlope Plus
- **Appendix B:** Data Sheet of ConstruckSlope Components
- **Appendix C:** Method Statement including site preparation
- **Appendix D:** Case Studies of Existing Project
- **Appendix E:** More Photographs of Project
- **Appendix F:** Environmental Benefits Comparison
- **Appendix G:** Rainfall Data/Seismic information of the existing project locations
- **Appendix H:** Bench-Geogrid Interlock Detailed Pull-Out Strength Test
- **Appendix I:** Geogrid Anchorage Capacity at Berm Concrete Bench
- **Appendix J:** Historical Use Case of Geosynthetic-Based Slope Reinforcement

Appendix A - Standard Drawing of ConstruckSlope Plus



Appendix B - Technical Data Sheet: ConstruckSlope™ Plus 200

Product Description: The ConstruckSlope™ Plus is a multi-component, engineered slope protection system designed for high-demand applications. It integrates a laid geogrid of high-tenacity polyester filaments with extruded polyethylene coating and welded junctions, providing exceptional tensile reinforcement across the slope. In addition to a pre-tensioned tensile reserve, the system incorporates integral drainage to relieve water pressure behind the facing and a durable, UV-resistant facing for long-term durability. The system is mechanically anchored at the crest, toe, and on berms to ensure effective load transfer and long-term performance.

System Performance Properties

The following table lists the key technical values of the ConstruckSlope™ Plus system as a composite. These values are derived from the specified individual components.

Property	Component	Test Method	Unit	System Value
STRUCTURAL REINFORCEMENT	ConstruckGrid™ HS			
Mass per unit area	Geogrid Element	EN ISO 9864/ ASTM D5261	g/m²	1,010
Max. Tensile Strength (md/cmd)	Geogrid Element	EN ISO 10319 / ASTM D6637	kN/m	≥200 / ≥100
Elongation at Max. Strength (md/cmd)	Geogrid Element	EN ISO 10319 / ASTM D6637	%	≤10.5 / ≤10.5
Tensile Strength at 1% Elongation (md)	Geogrid Element	EN ISO 10319 / ASTM D6637	kN/m	34
Tensile Strength at 2% Elongation (md)	Geogrid Element	EN ISO 10319 / ASTM D6637	kN/m	48
Aperture size (md / cmd)	Geogrid Element	-	mm x mm	67 x 61
UV resistance (remaining tensile strength)	Geogrid Element	EN 12224	%	≥ 85

Property	Component	Test Method	Unit	System Value
Weather resistance	Geogrid Element	FGSV / ASTM D4355	Class	High
Production specific elongation	Geogrid Element	-	%	0
Roll width	Geogrid Element	-	m	4.75
Raw Material	Geogrid Element	-	-	Polyester (PET) w/ Polyethylene (PE) Coating
HYDRAULIC & FILTRATION	ConstruckDrain™			
In-Plane Water Flow Rate (i=1.0, 20 kPa)	Drainage Element	EN ISO 12958 / ASTM D4716	l/(m·s)	1
Geotextile Opening Size (O^{90})	Drainage Element	EN ISO 12956 / ASTM D4751	µm	90
Raw Material	Drainage Element	-	-	Polypropylene (PP)
Berm Drainage Dispersion			m^3/Lm	0.3
SURFACE PROTECTION & INTERLOCK	Construck3DMat™ & Plaster			
Matrix Thickness for Plaster Interlock	UV Protection Element 1	EN ISO 9863-1 / ASTM D5199	mm	20
Max. tensile strength, md/cmd	UV Protection Element 1	EN ISO 10319 / ASTM D6818	kN/m	≥ 2.0 / ≥ 0.4
Elongation at max. strength, md / cmd	UV Protection Element 1	EN ISO 10319 / ASTM D6818	%	≥ 15 / ≥ 10
Mass per unit area	UV Protection Element 1	EN ISO 9864 / ASTM D6566	g/m^2	400
Plaster Facing Compressive Strength	UV Protection Element 2	ASTM C109	MPa	>3

Property	Component	Test Method	Unit	System Value
Plaster Facing Adhesion Strength	UV Protection Element 2	BS EN 1015-12 / ASTM C952	MPa	>0.3
Raw Material	UV Protection Element 1	-	-	Polypropylene (PP) / Polyethylene (PE)
ANCHORAGE SYSTEM	ConstruckScreed™			
Anchor Compressive Strength (28 days)	Concrete Element 1 & 2	EN 12390-3 / ASTM C39	N/mm^2 (MPa)	25 to 31
Anchor Material Density	Concrete Element 1 & 2	EN 12390-7 / ASTM C642	kg/m^3	~2,000 - 2,200
Bottom TrenchLock mass per unit length	Concrete Element 1	-	Kg/Lm	500-550
Upper Bench Lock and Lower Bench Shearlock mass per unit length	Concrete Element 2	-	Kg/Lm	680 – 748
Concrete Class	Concrete Element 1 & 2	EN 206 / ASTM C94	-	C25/30
Pull-out Strength (per aperture)	Concrete Element 1 & 2 interlock with Geo-grid	ASTM D6706	kN/apert.	6.25 (one specimen, 67–75 mm)
Minimum Embedment Length (bench/trench)	Concrete Element 1 & 2 interlock with Geo-grid	-	m	4.0 (bench width)
Overall System Thickness	-	-	mm	30-33
Length / Height of Slope	-	-	m	9-10

Component Rationale and Function within the System

- **Geogrid Element (ConstruckGrid™ HS 200/100):** Provides the primary tensile reinforcement and the pre-tensioned tensile reserve of the facing, anchored in the bench screed. Its low elongation at working loads prevents deformation of the slope face.
- **Drainage Element (ConstruckDrain™):** Its high in-plane flow capacity ensures rapid drainage of infiltrating water, preventing the buildup of hydrostatic pressure, which is a primary cause of slope failure. The integrated geotextile is selected to retain the soil; filter compatibility is confirmed for each site.
- **UV Protection Element 1 (Construck3DMat™):** A thick, three-dimensional mat that creates a high-void structure for the plaster to mechanically interlock with, forming a strong, composite bond that prevents the facing from slipping or delaminating.
- **UV Protection Element 2 (Plaster):** A durable, fiber-reinforced plaster that provides long-term UV and mechanical protection for the underlying geosynthetic components.
- **Concrete Elements 1 & 2 (ConstruckScreed™):** A high-strength, low-shrinkage screed used to create robust mechanical anchors (shear locks, crest/toe trenches) that ensure the tensile forces from the geogrid are effectively transferred into the stable ground.

- The values presented in this data sheet are based on the individual material specifications and are to be used for design and quality assurance purposes for the ConstruckSlope™ Plus 200 system.

Appendix C - METHOD STATEMENT FOR LAYING CONSTRUCKSLOPE PLUS

SLOPE PROTECTION SYSTEM ON MOUNTAIN SLOPES

1. <u>INTRODUCTION</u>

This method statement describes the typical methodology for laying **ConstruckSlope® Plus** system. **ConstruckSlope® Plus** is a slope protection and erosion control system which makes use of anchor pins and a three-layered system with its own purposes in protecting the slope of the mountain.

In the three-layered system, the bottom most layer known as **ConstruckDrain®** is designed to take water out from the mountain to reduce the pore pressure built up. The second layer applied, known as **ConstruckGrid® HS**, is a high-strength geogrid layer with capacities

mentioned in the design scope. The third layer, known as **Construck3DMat®**, is meant to hold the cement plaster onto the slope, to protect the first two layers from UV light and weather conditions.

2. SCOPE OF WORK

The scope of work is to apply the ConstruckSlope® Plus Slope Protection System at the select location. The primary objective is to address erosion concerns as identified in the drawings.

The area is suggested to be protected with the ConstruckSlope® Plus system, which consists of three layers. The first layer, ConstruckDrain, is a drainage mat designed to effectively remove water from the slope. This is followed by the ConstruckGrid® HS geogrid layer, which provides the necessary stability to the slope, utilizing a high-strength geogrid with a capacity of over 200 kN. The top layer, Construck3DMat, holds the cement plaster in place while offering additional protection from UV light and weather conditions. On top of this, a cement plaster layer is applied to provide an extra protective coating.

To secure the system to the slope, 16mm diameter tie rods are installed in a grid pattern as temporary fixings during installation, each with a base plate and nut, as per the slope cut and protection design recommendations. The system is anchored at both the top and bottom of the slope using anchor trenches filled with Construck Polyfibre Screed. These trenches will also serve as drainage channels in certain areas. At the berms, the system will be anchored into ConstruckSlope Berm, which will be bermlining with 10 cm thick Construck Polyfibre Screed, including upstands on both sides. The berms will be designed with transverse and longitudinal gradients to direct runoff water. Finally, weep holes will be provided at the toe of each slope, spaced at a maximum interval of 3 meters center to center, to effectively drain water from the drainage layer.

3. MATERIALS

ConstruckSlope® Plus System consists of:

1. ConstruckDrain® Drainage Mat – 3D Drainage Mat with Geotextile on both sides
2. ConstruckGrid® HS - High Strength Geogrid 200/100 kN/m (ConstruckGrid HS 200/100)
3. Construck3DMat® - Erosion Control Mat – 3D Filament Matrix
4. Cement Plaster

5. Anchor pins – 50 cm long 16 mm steel bars/Tie rods/threaded rods with base plate and nut (temporary fixing).
6. Temporary Anchor Pins - J/L Shaped 8mm rebar up to 20cm length used as per site requirements to temporarily fix the various layers to the slope.
7. Construck Polyfibre Screed to be used in Anchor Trenches and berms - 21-25 N/mm^2 after 28 days.
    a. Special Poly Propylene Fiber
    b. OPC Cement
    c. Sand – size up to 5 mm
8. Potable Water
9. Readymix Concrete (grade C25 or above) to be used in anchor trenches if required.
10. 200-gauge Polythene sheet or Hessian cloth for curing

4. EQUIPMENT & MANPOWER

4.1 Equipment & Tools

4.1.1 Various lifting equipment (Crains, Boom Loader and Hiab) to lift various equipment and machinery.

4.1.2 Spoons

4.1.3 Shovels

4.1.4 Spray Plaster Machine

4.1.5 Screed Mix Machine

4.1.6 Drilling Machine

4.1.7 Cutting equipment

4.1.8 Finishing tools

4.1.9 Measurement Boxes

4.2 Manpower

4.2.1 Management Team

HSE – 1

QA/QC – 1

Project Manager – 1

Supervisor – 1

Foreman / Leading Hand – 1

4.2.2 Labour Group (Number of groups can be required as per site requirements):

Screed Pump Operator/Plaster Machine Operator	01
Drilling Operator	01
Mat Laying Helpers	03
Drilling and Anchor Fixing Labors	04
Concrete Specialist Mason	02

Note: Number of groups can be increased for the required performance and pace of work.

5. WORK SEQUENCE

1. Approvals
2. Material Procurement, Transportation and Delivery
3. Surface Preparation
4. Trench Preparation
5. Material Storage at site as per Instruction and Plan
6. Inspection of delivered material by client
7. Preparation of area for drilling anchor pins and pumping of screed/plaster
8. Marking, Drilling and Fixing of Anchor Pins
9. Placing the Construck Drainage Mat on the slope
10. Placing and pre-tensioning the Construck HS Grid on the slope
11. Pouring of the Construck Polyfiber Screed at berms, trenches and drainage channels.
12. Locking the system with Construck Polyfiber Screed at the anchor trenches or berms
13. Fixing of the Construck3DMat
14. Fixing and tightening of base plates and nuts of Anchor Pins
15. Spraying of Cement Plaster/Finishing of the surface
16. Protection and curing with water for minimum 7 days under 200 gauge Polythene Sheet or Hessian cloth
17. Hand Over

6. PROCEDURE

6.1 Methodology of Surface Preparation:

The surface of the slope is scaled, and loose materials are removed using garden rakes, shovels and other necessary equipment. The removed material is transported to a previously identified disposal area.

6.1.1 General Arrangements & Cleaning:

The scaling of the slopes consists of the removal of loose fragments, flakes and boulders in unstable equilibrium from the slope. The activities are executed by specialized climbers with appropriate manual equipment (rakes, crow bars, shovels, picks, brushes), excluding the use of heavy machinery or explosives.

6.1.1.1 Delimitation of Working Area

The boundary of the working area shall be clearly marked with safety barriers, warning tapes and appropriate warning signals. The placing of fencing devices shall be decided by the site supervisor.

6.1.1.2 Drilling Of Anchorage Points

In order to provide safe anchorage points to the climbers, provisional pins Ø 15.0-24.0 mm shall be drilled 50 cm inside (variable) the crest of the slope. The pins shall be driven into rock by means of sledgehammer; in case of weak rock, they shall be grouted with a water/cement mix. The anchorage point locations shall be evaluated and checked by the foreman climber and supervisor, who shall verify their integrity considering that every anchor point shall bear the loads of:

- Climbers
- Equipment
- Possible rescuer
- Dynamic stress due to fall

6.1.1.3 Scaling

The scaling should start from the top portion of a slope. If there are multiple levels in this work area, people can work at different levels at different heights provided they are not in the direct line of fire from above.

For each level, the climbers shall descent from the anchorage points downwards towards the toe of the slope. Every climber shall clean the sector of the slope assigned to him without interference with other authorized persons. In any case it must be avoided to work directly above others.

The flakes, fragments and cobble (V <0.30 cu.m) in unstable equilibrium shall be removed with manual tools like shovels, rakes, crow bars, brushes, picks etc. If the unstable material of considerable volume cannot be scaled by means of manual tools, the site supervisor shall evaluate the site condition and inform the geologist. In case of a volume V > 0.50 cu.m boulder cannot be scaled, in order to avoid risk of falling down, the piece shall be anchored during the execution of the slope protection using rock dowels.

6.1.1.4 Mucking

Once the scaling activities are over, the site supervisor shall use the mucking team to remove the scaled materials from the bottom of the slope. The machinery shall be kept ready to muck the scaled materials once the scaling operation in a particular area is done. Wheelbarrows shall be used to transport scaled materials in areas where access is not possible for heavy machinery.

6.2 Marking, Drilling, Inserting Anchor Pins

The anchor pins (temporary fixings) shall be 16mm dia tie rods of 50cm length with base plate and nut. The spacing between the anchor pins shall be between 2-4 m. Pilot holes shall be made into the formation in the specified interval and the tie rods shall be driven into these pilot holes.

6.3 Installation of ConstruckSlope Layers on the slope (Layers 1 & 2)

6.3.1 ConstruckDrain drainage layer shall be cut into panels of the same slope length with minimum 20cm extra to allow for top and bottom anchors. The drainage layer shall be

unrolled full length into the slope and held in place temporarily by any anchor pins. Cuts shall be made in places of anchor pins to allow them to pass through.

6.3.2 Cut the ConstruckGrid HS with minimum 1m additional length of the slope on both sides and place it over the drainage mat on all slopes and anchor them with the same anchor pins used earlier. After the top of the grid is locked in the bench screed, a weighted belt is attached along the bottom edge so that the grid hangs free of wrinkles under an estimated pre-tension of 8–10 kN/m; the toe screed is then cast to lock the pre-tension. ConstruckGrid HS should not have joints in the middle of a slope, it should be one continuous sheet from top to bottom. Lateral overlaps between adjacent sheets are provided in such a way that each anchor pin will lock in 2 adjacent ConstruckGrid HD panels within the maximum allowable spacing between anchor pins.

6.4 Methodology of Applying Construck Polyfiber Screed on Berms

Construck Polyfibre Screed for berms with upstands on either side of the slope shall be applied to berms to allow proper drainage of water along the berms and to prevent overflowing of runoff water. Detailed methodology is provided separately.

Construck Polyfibre Screed is applied at a thickness of 10 cm, with upstands on both sides and gradients (transverse and longitudinal) to ensure drainage of runoff water from the slope. The required materials include sharp sand, cement, additives, and admixtures. The operatives loading the pump will be fully trained in the correct mixing procedures (as per the approved mix design) to ensure compliance with the specification. All operators will be knowledgeable about the mix ratio (as per the mix design in Annexure I) to meet the required specifications.

Flexible rubber hoses, connected by a series of metal coupling clamps, will be attached to the pump to transport the screed from the pumping machine to the berm. Once a batch of Construck Polyfibre Screed is mixed and pumped to the berm on-site, the labourers will spread the screed to a height of 10 cm using trowels and levelling bars. Control joints shall be provided at a spacing of 2m along the center of the berm.

6.5 Construction Methodology for Anchoring ConstruckSlope Plus System

6.5.1 Top of the Slope

At the top of the slope, the ConstruckSlope Plus Slope Protection and Erosion Control System is anchored either to the formation using tie rods or an anchor trench filled with Construck Polyfibre Screed. When Interception ditches are required, ConstruckSlope System can be anchored under the ditch as well.

6.5.2 Toe of the Slope

At the toe of the slope, ConstruckSlope Plus Slope Protection and Erosion Control System shall be anchored using one of the following approved methods:

6.5.2.1 Anchor Trench Method:

An anchor trench with dimensions of 50 cm x 50 cm shall be excavated along the toe of the slope. The bottom edge of the system shall be laid into the trench and secured by filling the trench with either polyfiber screed or ready-mix concrete (grade C25 or higher), ensuring complete embedment and fixation.

6.5.2.2 Drainage Channel Integration:

Where a bottom drainage channel is to be constructed as part of the site works, the system shall be anchored within the channel itself. The installation shall ensure that the system is securely fixed and integrated into the concrete base of the drainage channel.

6.5.3 Temporary Fixing on Slope

Temporary fixing shall be done using tie rods of 16 mm diameter and 50 cm length, drilled and inserted or driven into the substrate through the aperture of the Geogrid. First, a hole of suitable diameter should be drilled, and the tie rod should be inserted/driven through the aperture. Care must be taken to ensure enough length protrudes to accommodate layer 3 and subsequent fastening. The anchor pin installed should be tightened to the grid with a base plate and nut in the next step. These rods hold the layers in place during installation and are not relied upon for slope stability.

6.5.4 Anchoring at Berm

If the toe of the slope ends in a berm, inserting the toe end into the berm lining in a tongue-in-cheek arrangement, along with anchor pins, will be sufficient. If the top of a slope ends in a berm, placing the high strength geogrid at least 1m under the berm lining and fixed to the berm using anchor pins shall be sufficient. The toe detailing can be carried out before or after the slope plastering is completed. However, no plastering on the slope should take place before the top-of-slope anchoring is completed.

6.6 Applying Layer 3 – Construck3DMat

Construck3DMat shall be cut or joined as needed and placed on the slope profile over the geogrid, starting from the top and extending to the berm screed area (or continuously where the berm width is less than 2.5 m). It shall be fastened to the geogrid using cable ties. These cable ties provide temporary fastening until the cement plaster cures and becomes a monolithic product with the geogrid. Once placed, the anchor pin protrusions can be fastened with base plates and nuts over the Construck3DMat, ensuring that all three layers are fastened together.

6.7 Laying Cement Plaster & Finishing of Slopes

The premixed cement plaster powder shall be added into the spraying machine as per the manufacturer's recommendations, including the appropriate water content, and sprayed onto the Construck3DMat at a sufficient distance to ensure penetration and minimize loss. The layer is applied in two spray finishes. However, care must be taken to ensure that the combined thickness of the cement plaster layer does not exceed 30mm and that the Construck3DMat is completely covered. A slight color difference may be observed when plastering different areas due to varying setting and curing times, as well as natural color variations in the cementitious product, but these variations should be maintained consistently.
The plaster will take the shape of the substrate below, and as such, the finish will be rough due to the mountain profile.

6.8 Weep Holes

Weep holes are placed at the center of each grid, with the interval between two adjacent weep holes horizontally not exceeding 3 m, at the bottom of the slope. The weep holes (50mm diameter PVC pipes) will be placed firmly touching the drainage mat layer but never penetrating it.

6.9 Curing

Cure the Construck Polyfibre Screed under 200g polythene/hessian cloth for a period of 5-7 days. The polythene sheets/hessian cloth must be well lapped to cover the entire screed, including the exposed edges. Failure to prevent moisture loss from the edges significantly increases the risk of curling of the screed. Premature drying also increases the risk of cracking. Upon completion of curing, the polythene should be removed. Depending on site conditions, alternative curing methods can be adopted.

Plaster should be cured by hydrating three times over 7 days or as per the manufacturer's recommendations.

6.10 Control/Release Joints

Within 24 hours after casting, it is necessary to place release joints in the Construck Polyfiber Screed. These joints take care of the shrinking and curing cracks and are applicable for fiber screed.

Release joints should also be applied to the second coat of cement plaster on slopes to reduce the formation of shrinkage cracks, with an interval of 3-4m, depending on the aesthetic requirements.

6.11 Day Joints

Day joints are placed wherever a bay of daily production ends. These day joints will also act as release joints (refer to Release Joints section).

6.12 Trafficking

Access to the screed should be restricted for at least 24 hours to prevent damage to the surface. After this period, light foot traffic is permitted. Normal site traffic on the screed is allowed only after the completion of the 14-day drying period. There shall be no kind of trafficking area above the plaster layer.

6.13 Drying

Screed/Plaster drying times are based on the technical data sheets and depend on ambient temperatures both inside and outside. If the screed gets wet due to leaks or rainfall, the drying times will be extended. Hairline cracks are expected during or after the curing process and do not require any further treatment.

6.14 Methodology of Crack Repair (only required for wide open cracks bigger than 3-4 mm wide)

The cracks which may form in Construck Polyfiber Screed or cement plaster do not affect the strength of ConstruckSlope Plus Slope Protection System. However, wider cracks are

aesthetically unpleasing and hence cracks wider than 3-4 mm in both cement plaster and Construck Polyfiber Screed shall be repaired using crack filling material.

6.14.1 Materials and Equipment

- ✓ Crack Repair Material
- ✓ Grinding machine or saw cutting machine
- ✓ Hand mixer (drilling machine)
- ✓ Spoon
- ✓ Brush
- ✓ Bucket
- ✓ Steel travel or knife

6.14.2 Preparing the area

Prepare the cracks by cleaning them with a soft brush or blower, removing all loose materials and dust as thoroughly as possible. Wider cracks should be reopened using a grinding machine or saw-cutting machine. The crack or joint should be opened up to a depth of 1 cm and an additional 2-3 mm in width. After reopening, clean out the crack again with a soft brush or blower to remove any loose materials and dust.

6.14.3 Preparing the mix

Prepare the crack-filling mix according to the manufacturer's recommendations and mix at a slow speed for 2-3 minutes, ensuring the entire contents are thoroughly blended. For screed, add sand to bulk out the product until it forms a paste suitable for deep, wide cracks or holes. For small cracks, use a small amount of sand, and for wider cracks, add more sand (C).

6.14.4 Applying the mix

Pour the mixed product directly into the crack or hole, and smooth it off using a metal straightedge, ensuring the entire crack or hole is completely filled. For small, thin cracks, a filling knife or paintbrush can be used to fill and smooth the surface, using the back of the knife for an even finish.

7. ROLES AND RESPONSIBILITIES

Project Manager shall be responsible for the overall implementation of this method statement, and to ensure that Section/Site engineer, Procurement/Material Engineer, Quality/HSE Engineer and general foremen are well informed of its requirements. He is responsible for:

- Providing all required resources in terms of Manpower, Equipment and Material
- Controlling the coordination between all involved parties to ensure Safety, Quality and schedule requirements are met.
- Controlling the progress of work and ensuring final approval is obtained for all related shop drawings, materials and methods etc. well in advance and before the casting scheduled date.
- Ensuring all works have been carried out as per relevant approved drawings.
- Planning of works to be conducted well ahead of time and ensure there is no delay at site due to non-availability of materials, machinery or equipment.
- Addressing any issues pointed out either by the team or from the client side and resolving them as fast as possible.

Site Engineer shall be responsible for:

- Controlling and ensuring the construction work is carried out to the Method Statement, Inspection & Test Plan, Contract specification and approved construction drawings.
- Notifying QC engineer and issuing the IR (Inspection Request) at least 24 hours before as detailed in the relevant ITP.
- Accompanying the consultant for ongoing inspection and obtaining his approval to proceed.
- Rectifying comments recorded in IR by Client as agreed and ensuring re-inspection done for approval.
- Giving all Foremen and leading hands precise instructions on their responsibilities for proper working methods; seeing that they do not require or permit men to take unnecessary risk.
- Organizing site so that work is carried out to the required standard with minimum risk to men, equipment and materials.
- Making certain that all plant operators and maintenance personnel are only employed for that which they have been thoroughly trained.

- Knowing the requirements of the project HSE Plan and Regulations and other relevant legislation and how it applies to the project.

Site HSE shall be responsible for

- Ensuring all activities at site shall be conducted under proper supervision.
- Making everyone working aware of all the risks involved in the work and at site in general.
- Conducting toolbox meetings and explaining the safety measures to be taken while conducting the day's work.
- Monitoring the activities at all times and ensuring proper safety precautions are always undertaken before doing any activity.
- Liaison with client HSE and rectifying his concerns.
- Preparing and submitting weekly HSE reports.

Quality Representative shall be responsible for monitoring the effectiveness of the Quality System and providing feedback to appropriate personnel. As the Quality Representative he shall ensure that:

- All products are checked and verified during the delivery of the product.
- The system is maintained and remains applicable in all operations.
- Provide monthly report reflecting the status of works inspected, Non-Conformances raised, samples submitted and approved, etc.
- Liaison with the Client's Quality Administrator about quality related matters.
- Prepare weekly reports of progress of works and activities at site.

Site Supervisor shall oversee all work carried out by the assigned workforce of Marmol Integrated LLC. It is the responsibility of the Site Foreman to ensure that the requirements of the project are always met. He shall be responsible for delimiting the work area, and ensuring proper rigging plans and precautions are taken for lifting equipment and movement of heavy machinery. Site supervisor shall also make sure proper anchor points are identified and executed for climbers to anchor the ropes.

Foremen/Leading Hand shall ensure the labourers assigned to him are well aware of the works to be conducted and the HSE requirements to be fulfilled to carry out the job assigned to his team. He must assist the site supervisor and ensure proper communication between his team and site management.

Labourers shall make sure the work assigned to them are completed in the safest manner in the shortest possible time and achieve the best quality possible. They must listen to the site HSE and supervisors and adhere to all safety precautions laid out for them. They must raise any concerns regarding work or safety to their immediate supervisor get proper clarifications before and during the execution of any work. They must maintain the decorum of site at all times.

8. RIGGING PLAN AND HEAVY EQUIPMENT MOVEMENT

The general purpose is to ensure that a Safe System of Work (SSOW) applies to the risks associated with rigging, lifting, moving, and lower ConstruckGrid, ConstruckDrain, Construck3DMat, Rock Bolts, other accessories, machinery, etc. Each activity has been carefully considered and appropriate controls implemented. The specific purpose of this Method Statement procedure is to describe the method of rigging lifting, moving, and lower ConstruckGrid, ConstruckDrain, Construck3DMat, Rock Bolts, other accessories, machinery, etc. Marmol Integrated LLC will be using a Crane/Heavy Lifting Machinery/Man Lift to lift, move, and lower ConstruckGrid, ConstruckDrain, Construck3DMat, Rock Bolts, other accessories, machinery, etc.

For the construction of Rockfall Protection systems, the safe operating procedure includes:

8.1 Task Risk Assessment

Before proceedings to any lifting/rigging work, Risk Hazard Identification to be conducted for the particular rigging/lifting operation. This is required to be done for every rigging/lifting operation activity/work. If a series of similar operations are to be undertaken, one RA may be sufficient. But check carefully that something critical has not changed, for example, if the size and/or weight of the load is much greater than for the previous lift, then a new RA will need to be undertaken. This method statement provides a generic safe system of work for rigging/lifting operations.

8.2 Rigging / Lifting plan

The rigging plan is intended to ensure that the proper procedures, equipment and rigging is used for each operation and to ensure that the supporting structure can supporting the rigging loads.

Lifting plan provides a systematic approach towards planning a loft and ensures that the lifting operation is properly managed.

8.3 Supervision

The Site supervisor supervises the entire lifting operations. For every lifting/rigging task, one competent person must assigned/allocated the role of Lifting/rigging Supervisor having valid and approved certification and take accountability for the Risk Assessment/Hazard Identification and gives instructions for the lifting operation. Only qualified workers will be allowed to use mechanical tools and equipment. After Risk Assessment/Hazard Identification has been accomplished and details added to this method statement, all workers involved in the lifting operation will be inducted on the work; given details of the Risk Assessment & Method Statement (RAMS), and any other information they require.

CONSTRUCK POLYFIBER SCREED MIX DESIGN

CONSTRUCK POLYFIBER SCREED MIX SPECIFICATION

The screed mix shall have the following properties:

- Cement Sand Ratio = 1:6
- Sand size up to 5 mm
- Additive = 250 ml / 50 kg of cement
- Polypropylene Fiber – 80g for 50Kg of cement
- Cement for screed mix shall be Ordinary Portland Cement 42.5N minimum and comply with requirements specified in BS EN 197-1
- Water for screed mix shall be clean fresh water
- The screed mix shall have a minimum compressive strength of 25 N/mm^2 at 28 days.

Components for 1:6 Mix Ratio

Cement & Sand	1:6 mix
Water	20 L to 25 L
Polypropylene Fibre	80g
Additive	250 ml

Appendix D: Case Studies of Existing Project

Slope Stabilization and Erosion Control at DUQM Gas Supply Station using ConstruckSlope Systems

Project

EPC Works for the Ad Duqm Gas Supply Station (Al Turki)

Scope of Work

Rip rap Alternative Works - Supply and apply of ConstruckSlope System and Berm Protection

Quantity

15,000 m²







Sustainability Development Goals (SDGs) Alignment



Slope Stabilization and Erosion Control at Musandam Gas Plant using ConstruckSlope Systems over an area of 45,500 m²

Project

Slope Stabilization, Erosion Control and Rockfall Protection at Musandam Gas Plant

Scope of Work

Design, Supply and Apply of Slope Stabilization, Erosion Control and Rockfall Protection Systems

Quantity

45,500 m²







Sustainability Development Goals (SDGs) Alignment



Project Sustainability Report

C02 Reduction

682,875 Kg C02-eq

Reduction in Water Usage

1.13Million L

Fuel Reduction

4,318 L

Slope Protection at Al Khad Sham, Sohar using ConstruckSlope Plus over an area of 74,100 m²

Project

Slope Protection at Al Khad Sham Road, in Sohar

Scope of Work

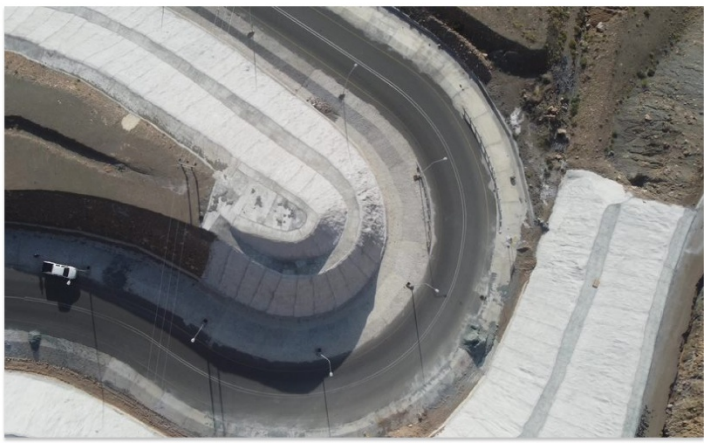
Design and installation of slope protection system to provide long-term slope stabilization, erosion control, and structural reinforcement.

Quantity

74,100 m²







Sustainability Development Goals (SDGs) Alignment



Project Sustainability Report

C02 Reduction

1.11M Kg C02-eq

Reduction in Water Usage

1.85Million L

Fuel Reduction

7,039.5 L

Appendix E: More Photographs of Project














Appendix F: Environmental Benefits Comparison

Environmental Benefits of ConstruckSlope™ Plus vs. Conventional Shotcrete Systems

1. Embodied Carbon and Material Use

Criterion	Conventional Shotcrete	ConstruckSlope™ Plus 400	Environmental Advantage
Cement usage	400–500 kg/m³ shotcrete; heavy dependence on cement (major CO_2 source)	Thin polymer-modified plaster (~30 mm) + lightweight screed anchor blocks	**>70% reduction in cement consumption**
Steel reinforcement	Requires mesh/rebar (corrosion-prone, high embodied energy)	No structural steel; geogrid reinforcement via PET/PE	**Eliminates steel production impacts**
Overall system mass	2400–2500 kg/m³ applied volume	<600 kg/m³ equivalent system composite	**~75% lighter → reduced transport & handling emissions**

2. Water & Energy Demand

Criterion	Shotcrete	ConstruckSlope™ Plus	Advantage
Mixing water demand	High — 150–200 L/m³ shotcrete	Minimal — hydration only for plaster layer and screed anchors	**>50% less water**
Energy during batching/application	Requires batching plant + compressors/pumps	Factory-prepared geosynthetics; only thin plaster layer mixed	**Lower operational energy demand**

3. Durability & Service Life

Criterion	Shotcrete	ConstruckSlope™ Plus	Advantage
Corrosion	Steel mesh + rebar prone to corrosion, spalling	Geosynthetics inert to corrosion	**Longer life, fewer replacements**
Cracking	Brittle, prone to fissures under swelling/pressure	Flexible, accommodates soil movements	**Reduced repair cycles**
Service life	30–40 years typical (with repairs)	75–120 years (no corrosion; UV-tested)	**2–3× service life**

4. Land & Ecosystem Impact

Criterion	Shotcrete	ConstruckSlope™ Plus	Advantage
Drainage	Impermeable, traps water → hydrostatic pressure buildup	Integrated drainage (ConstruckDrain™)	**Restores natural hydrology**
Vegetation	Hard concrete surface, prevents revegetation	Compatible with soil cover/vegetation regrowth	**Improves biodiversity**
Site disruption	Heavy equipment, batching, rebar work	Prefabricated rolls, minimal site footprint	**Lower disturbance to terrain & habitat**

5. Lifecycle Sustainability

- **Carbon footprint:** Reduced by >50% due to lower cement and steel inputs.
- **Transport emissions:** Lighter materials → fewer truckloads.
- **Waste reduction:** No rebound waste (common in shotcrete), geosynthetics cut-to-fit.
- **Circularity:** System can integrate recyclable PET/PP materials.

6. Overall Summary

ConstruckSlope™ Plus is a **low-carbon, resource-efficient alternative** to shotcrete. By minimizing cement, water, and steel use while delivering longer service life, it directly supports **global sustainability goals (SDG 9, 11, 12, 13)** and aligns with **green infrastructure policies**.

Appendix G: Rainfall Data/Seismic information of the existing project locations

This appendix summarizes the climatic and seismic boundary conditions considered in the design of ConstruckSlope™ project sites in Oman. The following values were adopted during engineering design and system validation, based on national meteorological and seismic hazard data.

Project	Location / Region	Rainfall Data (Design Basis)	Seismic / Tectonic Data (Design Basis)
Slope Protection at Al Khad Sham Road	Sohar, Al Batinah	Mean annual rainfall: ~120 mm; Max daily intensity used in design: 100 mm/24h	Design PGA (475 yr): 0.08 g
Musandam Gas Plant Slope Protection	Musandam Peninsula	Annual rainfall: ~180 mm; Design storm: 300 mm/24h (cyclone scenario)	Design PGA (475 yr): 0.12 g
Government of Oman Slope Stabilization	Interior/Coastal Oman	Annual rainfall: 150–200 mm; Local peak storm event: 220 mm/24h	Design PGA: 0.10 g
Ad Duqm Gas Supply Station (Al Turki)	Duqm region	Baseline <100 mm/year; Cyclone design storm: 350 mm/24h	Design PGA: 0.10 g
Sultan Haitham Bin Tariq Villa	Boushar (Muscat region)	Muscat mean rainfall: ~100 mm/year; Design daily intensity: 85 mm/24h	Design PGA: 0.11 g
Oman Shooting Range	Azaiba (Muscat coast)	Annual rainfall: ~100 mm; Design storm event: 150 mm/24h	Design PGA: 0.11 g

Load Bearing Capacity Test of Benches-Geogrid Interlock (Pull-Out Strength Test)

Purpose

To determine the pull-out strength of a single aperture high tensile geo-grid from a diamond/Habe Poly-fiber screed.

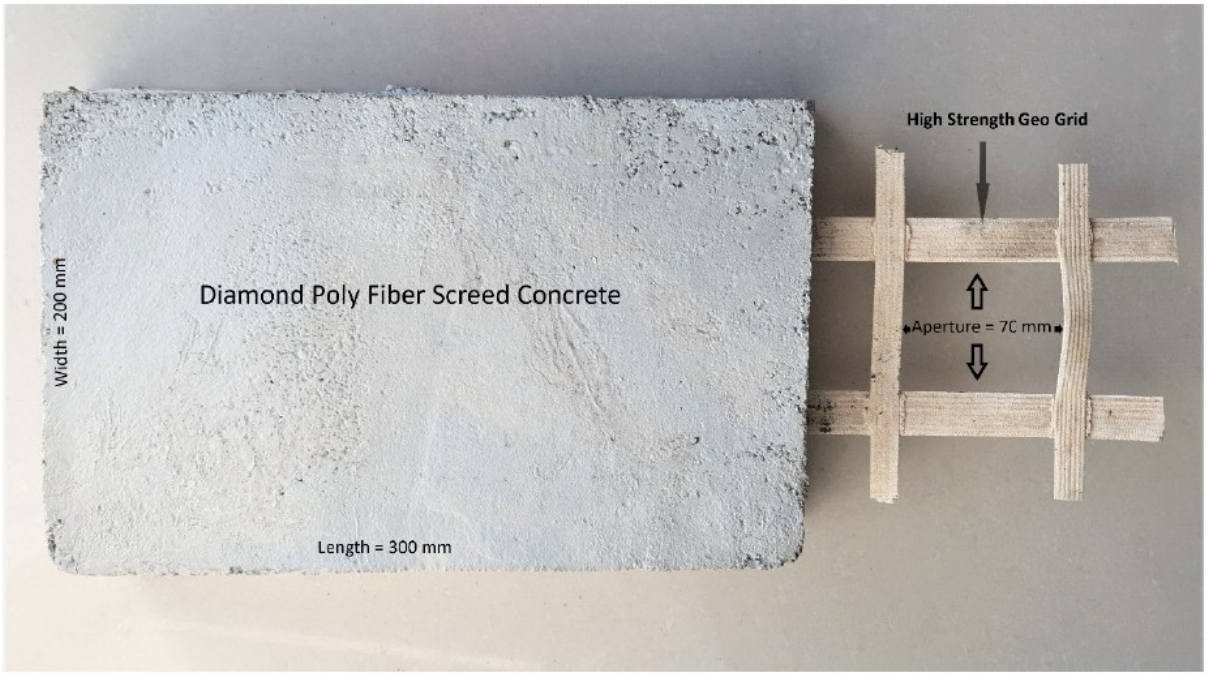
Introduction

In a regular slope protection system, The BRC Mesh/6MM/8MM rebar mesh are not flexible and forms only a facing reinforcement for the shotcrete. Such reinforcements are basically not transferring the load on the facing to any structure and hence serves only as a facing erosion control and may collapse under localized failures.

Whereas, Construck Slope Protection system not only serves as a facing erosion control with its High strength and flexible geogrid, it also contains localized failures with the Secugrid-Screed Interlock with Concrete Screed (Berm Concrete) on the benches.

The aim of this test is to investigate the effect of loading rate on the ultimate pullout resistance of Geo grids set into the concrete

Test specimens

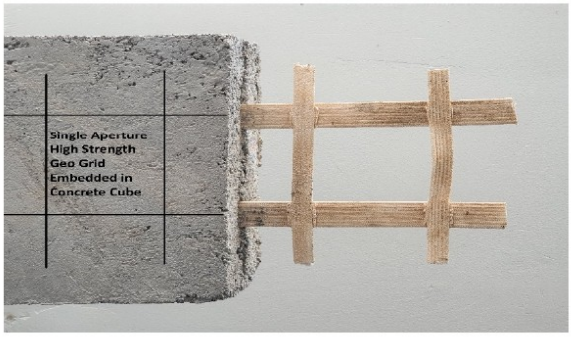


Test specimens shown in Fig. 1, 1.1 and 1.2. The test specimen consisted of a Diamond Poly Fiber Screed Concrete block [W200×L300×H80 (mm)] in which a High Strength Geogrid single aperture [Size -67 x 61 mm] was installed with a 100mm embedment depth.



The concrete block was cast using the following materials in the given ratio:

- Cement/Sand ratio = 1:6
- Sand Size upto 4mm
- Water Ratio = 100 ltrs/m3
- Diamond Water reducing Admixture = 250ml per 50 kg
- Diamond Special Polypropylene Fibers = 400 gms/m3
- The block was cured in laboratory air. All tests were executed within a period of 1 day after 28 days of curing.



The embedded High Strength Geo Grid has the following specifications:

- Raw material (filament) = Polyester/PET
- Raw material (coating) = polyethylene
- Mass per unit area = 1,300/m2
- Aperture Size = 67mm x 61 mm
- Max. tensile strength, md/cmd2 >=400 / >=100

Test Method/ procedure

In house Test method followed

Testing apparatus

Universal Testing Machine, Capacity 1200 kN

Test Results and discussions

A concrete structure along with the High Strength Geogrid was fixed into the UTM. The Geogrid end was fixed into the lower Jaw and Concrete cube (tightened with belt) into the upper jaw of the machine. After Fixing, the geogrid was pulled from the Concrete cube and Maximum load withstood by Geogrid is 6.25 kN before slippage occurred.






Conclusion

The following conclusions may be drawn from this study:

1. One aperture, i.e. approx. 6.7 cm of Geogrid HS inserted into our fiber screed concrete gives a pull out strength of 6.25kn

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TEST CERTIFICATE

Report No.	LMMET-1903198 Rev 0	**Date Reported**	21 Oct 2019
LAL Sample No.	MET / 3198	**Date Received**	20-Oct-19

TEST REPORT

1. Information Provided by the Customer

Customer for LAL : Construck Systems & Technology
Customer Ref. : Request Date on 20 Oct 2019
Sample Description : High Strength Geogrid with 7.5cm Poly Fibre Screed Embedded in Cocrete Cube

2. Information Provided by the Laboratory

Nature of Test :	**Load Bearing Capacity Test**	**Test Method :**	In House Method
Date Tested :	20 Oct 2019 - 20 Oct 2019	**Test References :**	-

ID	Maximum Load Withstood (kN)	Observation
LT - 1	6.25	Slippage from Concrete Cube

LT- Load Bearing Capacity Test

LONESTAR ALPHA LABORATORIES LLC ★ P.O. Box:1197,P.C:130,Sultanate of Oman ★ C.R.No:3245330 17

Pratik Raval
Metallurgy Supervisor

QC CHECKED

For and On behalf of Lonestar Alpha Laboratories

Unless otherwise indicated, latest editions of Standards/Specifications have been used for the Testing.
This Test Report shall not be reproduced either in part or fully, without approval of the Laboratory.

Geogrid Anchorage Capacity at Berm Concrete Bench

Objective

To verify that the pre-tensioned ConstruckGrid HS 200/100 facing, anchored in the bench screed, can deliver the tensile reserve used in the main report (44 kN/m design value, up to 81 kN/m) without the grid or its connection to the screed governing, and that this reserve covers the demand on the facing.

Data & Assumptions

- Geogrid: ConstruckGrid HS 200/100, max. tensile strength ≥ 200 / ≥ 100 kN/m (md / cmd, EN ISO 10319, product data sheet)
- Strength at 1% / 2% elongation (md): 34 / 48 kN/m; aperture approx. 67 × 61 mm (near square)
- Estimated combined reduction factor (creep, installation damage, durability): 2.0 → design strength 100 kN/m (assumption)
- Bench width B = 4.0 m; screed thickness t = 0.10 m; screed unit weight 20–22 kN/m³; one shear lock
- Bench ground: $c' = 12$ kPa, $\phi' = 30°$, $\gamma = 19$ kN/m³; passive coefficient on the shear-lock key $K_p = 3.0$
- Bond under the screed: 0.5 c' (design) to 1.0 c' (maximum); shear-lock key depth h = 0.3 m (design) to 0.5 m (maximum)
- Estimated pre-tension locked in: 8–10 kN/m (8 kN/m used)

Calculation

Screed weight: $W_s = t \times B \times \gamma_{screed} = 0.10 \times 4.0 \times 20 = 8.0$ kN/m (22 kN/m³: 8.8 kN/m)
Base resistance: $R_{base} = W_s \tan \phi' + \alpha\, c'\, B$
design: $8.0 \times 0.577 + 0.5 \times 12 \times 4.0 = 28.6$ kN/m; maximum: $8.8 \times 0.577 + 1.0 \times 12 \times 4.0 = 53.1$ kN/m
Shear-lock key: $R_{key} = \tfrac{1}{2} K_p \gamma h^2 + 2\, c' \sqrt{K_p}\, h$
design (h = 0.3 m): 2.6 + 12.5 = 15.0 kN/m; maximum (h = 0.5 m): 7.1 + 20.8 = 27.9 kN/m
Bench anchor capacity: $R_{anchor} = R_{base} + R_{key} = 28.6 + 15.0 \approx 44$ kN/m (design); $53.1 + 27.9 \approx 81$ kN/m (maximum)
The inward fall of the bench (1–2°) aids drainage and is not counted.

Empirical Validation

- Pull-out test, grid into fibre-reinforced screed (one specimen, approx. 67–75 mm embedment, ConstruckGrid HS 200/100, 1 aperture of 67 × 61 mm; Lonestar Alpha Laboratories, LMMET-1903198) → 6.25 kN at slip.
- Width-scaled: 6.25 × (1000 / 67) ≈ 93 kN/m, with no extrapolation in length.
- 93 kN/m (grid into screed) > 44 kN/m (screed into bench): the screed bearing on the bench governs.
- One specimen only: a series of at least three HS 200/100 specimens at the bench embedment is required (Research Gaps).

Summary Table

Parameter	Value
Geogrid ultimate strength (HS 200/100, md)	200 kN/m (design 100 kN/m)
Grid into screed (test, width-scaled)	≥ 93 kN/m
Screed into bench (calculated, 4 m bench, one shear lock)	44 kN/m design (up to 81 kN/m)
Governing anchorage reserve	44 kN/m

Demand: grid force to arrest a sliding layer (drainage working)	31.4 kN/m
Demand: reserve for FoS = 1.3 (drainage working)	49.4 kN/m (FoS 1.28 with 44 kN/m; 1.44 with 81 kN/m)
Estimated pre-tension	8–10 kN/m (8 kN/m used)
Result	Grid and connection do not govern; bench anchor 44 kN/m governs

Conclusion

The grid's strength (100 kN/m design) and its connection to the screed (≥ 93 kN/m) both exceed the bench anchor, so the screed bearing on the 4 m bench with one shear lock governs at 44 kN/m (design), up to 81 kN/m. This exceeds the 31.4 kN/m needed to arrest a sliding layer with drainage working, and gives FoS 1.28 (1.44 with the maximum) for the saturated 45° example slope. The values are to be confirmed by further pull-out specimens and the full-scale chamber test.

TRANSPORTATION RESEARCH RECORD 1330

Construction of a Geogrid- and Geocomposite-Faced Soil-Nailed Slope Reinforcement Project in Eastern Canada

COLIN ALSTON

A case history of a project involving the installation of soil nails with a connected protective membrane facing as a slope-reinforcement technique is presented. The project involved the reshaping of a naturally stable hillside in Cambridge, Ontario. This slope is formed of very dense glacial till material that was resting at a gradient of 1v:2h, before construction; project requirements necessitated developing the slope at a grade of about 3v:1h. The construction involved the installation of a soil nail reinforcement system to permit the development of an 18-m-high slope. At the highest point, the upper 12 m of this slope is permanent; the lower 6 m was a temporary slope excavated to permit basement construction to proceed. Three slope facing systems were used. In one system a facing of nonwoven geotextile restrained by geogrids tied to the soil nails was applied; a sand, topsoil, and water slurry was injected behind the facing to serve as a void filler and to tension the membrane. In another portion of the slope a geocomposite wall was constructed in front of the soil nail-reinforced slope, and this was tied to the soil nails using geogrid. The third system, which was applied to the temporary slope, consisted of a single membrane of woven geotextile tied to the soil nails with anchor blocks. The reinforced slope has proved to be satisfactory through two winters and two spring thaws. Selection of the soil nail support system for the project was based on economic considerations, and the selection resulted in considerable cost savings for the owner.

A case history of soil nail design and installation is presented for a site in Cambridge, Ontario, that has been developed to permit construction of a highrise condominium. The site is on the side of a relatively steeply sloping river valley, such that the development required considerable encroachment into the slope of the valley side.

The original concept for supporting the excavation was to construct two separate retaining structures: a soldier pile and lagging wall was to have served as temporary support for the basement levels; the upper slope, which extends from the main floor level to the crest of the slope, was to have been supported permanently by a reinforced concrete gravity retaining wall. The maximum supported slope height was to be about 18 m, to be reduced to 12 m after completion of the basement structure. However, it was found that this original earth retention concept could not be constructed without temporary mechanical support because it would have required encroaching into land not owned by the developer, and the adjacent land owners were not willing to permit any construction within their properties. The original concept was therefore impractical.

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At this stage, the author reviewed the geotechnical design of the project, and as a result of this review an alternative was proposed in which a system of soil nailing would be installed to support the reprofiled slope. The soil nails were to be designed to replace both the upper permanent reinforced concrete retaining wall and the lower temporary soldier pile and lagging wall. The principle virtues of the new (soil nail) concept to the developer were that (a) no negotiations were required with the adjacent property owners for excavation easements because the reprofiled slope could be constructed from the top down (starting at the intersection of the cut slope with the crest) rather than from the more conventional bottom up, and (b) there would be a substantial cost saving to the project.

SITE AND SUBSURFACE CONDITIONS

The Cambridge condominium site is located on the north bank of the Eramosa River valley. At this location, the slope has developed naturally to a gradient of about 1v:2h. The subsurface conditions at the site consist of a compact, rapidly becoming very dense till stratum overlying bedrock. Although principally silty in character, the composition of the till is somewhat variable, ranging in gradation from silty clay to silty fine sand. Much of the soil deposit is highly susceptible to frost, and it is also susceptible to piping in the event that seepage from the slope were to be unrestrained. The compact condition of the soil is represented by standard penetration test *N*-values ranging from 30 to more than 100 and typically more than 60 below a depth of 3 m. The more clayey zones of the deposit are concentrated in lenses, and the soil immediately above most of the clayey lenses was found to be saturated; thus, several perched water tables are present within the slope. Another consideration of the variable moisture condition of the soils exposed in the slope is that free water is available as a source for ice lensing under freezing conditions in the highly frost-susceptible silt soils that predominate in the slope.

Observations of cuts made into the hillside at an early stage in construction indicated that the native soil was able to stand unsupported at a nearly vertical inclination for 1 to 2 weeks before deterioration began to affect the stability of the cut face.

DESIGN CONSIDERATIONS

The project required the design of the slope stabilization system to take into account that the upper portion of the reprofiled slope would be permanent and exposed, whereas the lower portion of the slope was to be temporary, required only for the duration of basement wall construction.

At the design stage, it was expected that the temporary slope would be required only to provide support for a maximum of 2 months in fall and that all the elements of the substructure would be complete before the onset of winter. There was, therefore, no perceived need to design the facing to the temporary slope to withstand freezing forces or heavy lateral pressures resulting from temporary loss of soil strength during the thaw. The (permanent) retained slope carried a trunk water main at its crest that had to remain in service and could not be interrupted or relocated. However, no superimposed loads were expected to be applied to the tableland at the crest.

As noted, the native soils at the site are susceptible to frost. Hence, it was required that the external sheeting membrane to be applied to the permanently exposed slopes be designed to accommodate continuing cyclic soil volume changes and corresponding load variations without rupturing. The membrane would also be required to facilitate drainage from saturated zones in the slope face and prevent the occurrence of piping.

The other consideration at the site was aesthetic: the condominium development was geared to the upper end of the market, so engineering design was to make allowance for the wishes of the project's landscape architect. This was achieved by constructing the slope in a series of terraces, each being about 3 m high, and with a bench about 1.2 m wide. These terraces were designed to extend over the entire height of the permanent slope (see Figure 1). Planters were to be placed on each of the terraces.

SLOPE DESIGN

General

In plan, the excavation entailed removing from the hillside a prism of ground that had a triangular footprint. The height of the excavation was a maximum of about 18 m at the intersection of the two faces of the wedge, and this tapered out with distance away from this intersection. Typical cross sections that illustrate high and low areas of reinforced slope are given in Figure 1, which also shows the relationship of the permanent to temporary slope heights. The west and north excavation faces were approximately 90 and 80 m long, respectively.

Soil Nail Design

The design of the soil nail system was based on the method given in the *NCHRP Report 290* (*1*), and it relied on tensile forces in the soil nails only; no benefit for the shear resistance only a nominal surcharge loading was allowed on the tableland at the crest of the slope. In the absence of a Canadian code of practice, the design was prepared to conform with safety standards recommended by Stocker et al. (*2*). The specified nail was a "Dywidag" threaded steel bar with a guaranteed minimum yield stress of 415 MPa and diameters of 22 or 25 mm (depending on slope height); they were installed in 100-mm-diameter drilled holes. The bars were grouted into the drill holes with a cement–fly ash grout with a water/cement ratio of 0.4 and a compressive strength of 25 MPa. The design of the nail system was carried out using an angle of internal friction of the very dense till of 38 degrees and a unit weight of 22.5 kN/m^3.

The nails were installed on a 1.8- × 1.5-m grid, in all sections of the slope. The nail lengths for various sections of slope are given in Table 1.

As described, the project requirements dictated two slope conditions: a permanent slope of variable height and a lower temporary slope that increased the total height of retained soil by 6 m. Thus, the stresses in the soil nails in the short term were significantly larger than in the long term. Taking into account the lower long-term stresses, the tensile strength of the installed soil nails is not fully used. Thus, by comparing the forces applied over a reduced area to allow for corrosion, an approach now confirmed by Schlosser and Unterreiner in a paper in this Record, there was found to be no need to encapsulate the steel tendon with a plastic sheath. The outer 2 m of the nail was given two coatings of zinc paint as corrosion protection.

The prepared soil nail design was compared with published case histories of successful installations and design charts before being made final (*3–5*).

Design of Facing Membranes

Considerable attention was given to the selection of the slope face membranes. As noted, it was believed necessary to provide a surface membrane that would retain its strength properties through cyclic loading caused by soil expansion due to seasonal freezing in the permanent section of the slope. Furthermore, in view of the several perched water tables found in the slope and the frost-susceptible character of the native soils, it would be a significant advantage to apply a facing membrane with good drainage characteristics. This would then limit the supply of free water to fuel the growth of ice lenses, in turn reducing tension loads on the soil nails under winter conditions. It was also necessary for the membrane to conform to the terraced slope profile design that was acceptable to the landscape architect. Because of aesthetics, the owner was unwilling to accept a shotcrete facing membrane; a shotcrete facing was also expected to be inefficient at permitting drainage through the membrane as well as to be intolerant to soil movement caused by the formation of ice lenses in the retained soil.

The initially selected membrane system was a combination of geogrid and geotextile. The geotextile was used to permit the exit of groundwater from the slope but at the same time retain soil, thereby eliminating the risk of piping. The grid

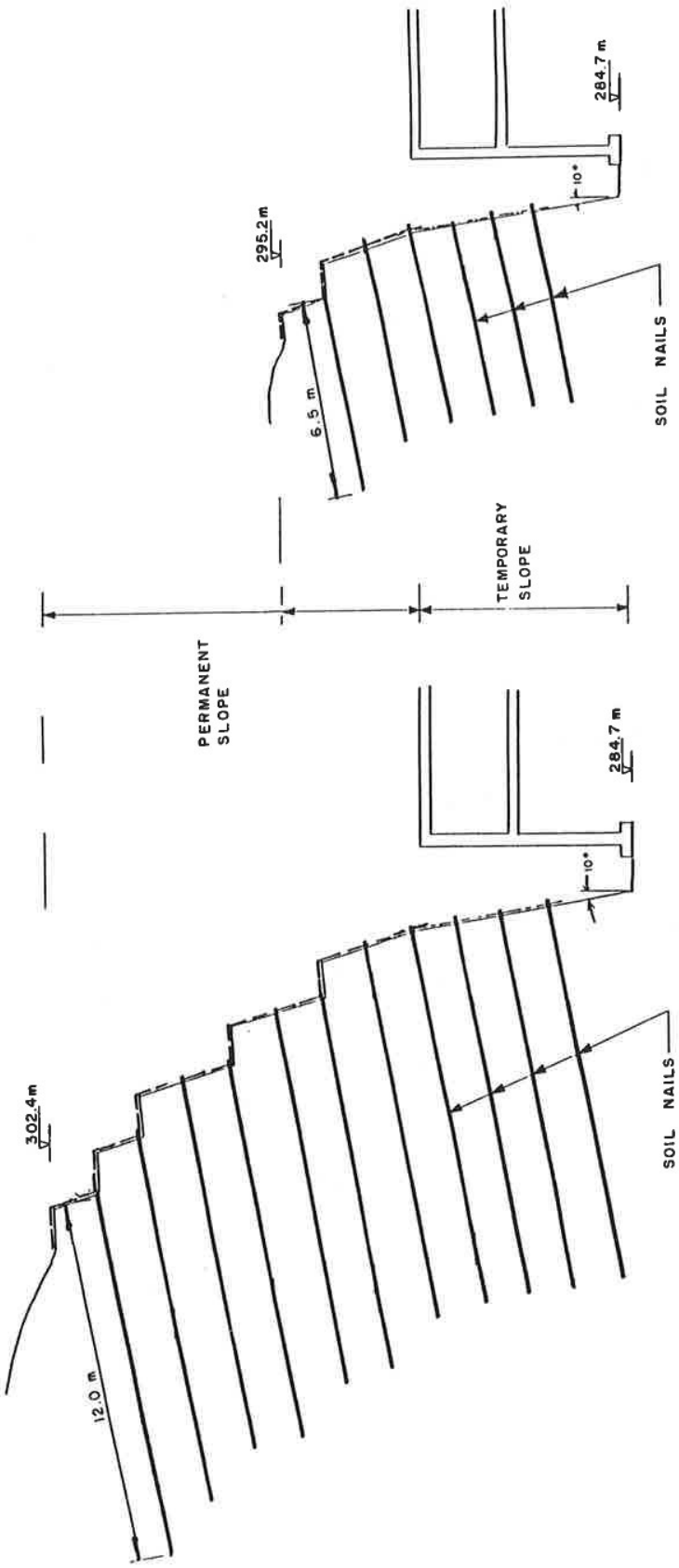


FIGURE 1 Typical cross sections through soil nail reinforced slope.

TABLE 1 SOIL NAIL LENGTHS FOR VARIOUS SLOPE HEIGHTS

Height of Slope		Length of Soil Nail (m)
Permanent (m)	Temporary & Permanent (m)	
4.5	10.5	6.5
8	14	8.8
12	18	12.0

angle bars was used to pick up the earth pressure loads from the geogrid membrane and to span between adjacent soil nails in the horizontal direction (Figure 2). The membrane chosen to provide the strength properties was an unequal bidirectional geogrid in which the prime reinforcement was aligned vertically and the secondary reinforcement horizontally. To accommodate the expected cyclic movement of soil in the face and to protect the reinforcing strands from atmospheric degradation, a grid with multiple polyester strands enclosed in polyolefin with a long-term allowable design load (LTADL) (i.e., design tensile capacity) of 40 kN/m width was selected. Beneath this geogrid, the nonwoven geotextile was used to keep the soil fines from escaping and to prevent surface erosion.

The geotextile was constructed of polyester fibers; it had a mass of 375 g/m^2 and an EOS of 75 μm. The geotextile-geogrid membrane was connected to the soil nails with a system of angle bars that span horizontally between adjacent nails. The angle bars were coated with zinc primer paint and compatible overcoatings of corrosion protection. Application of the membrane consisted of draping the geotextile and then the geogrid over the profiled face of the slope. The angle bars were then placed on top of the membrane and connected to the nails.

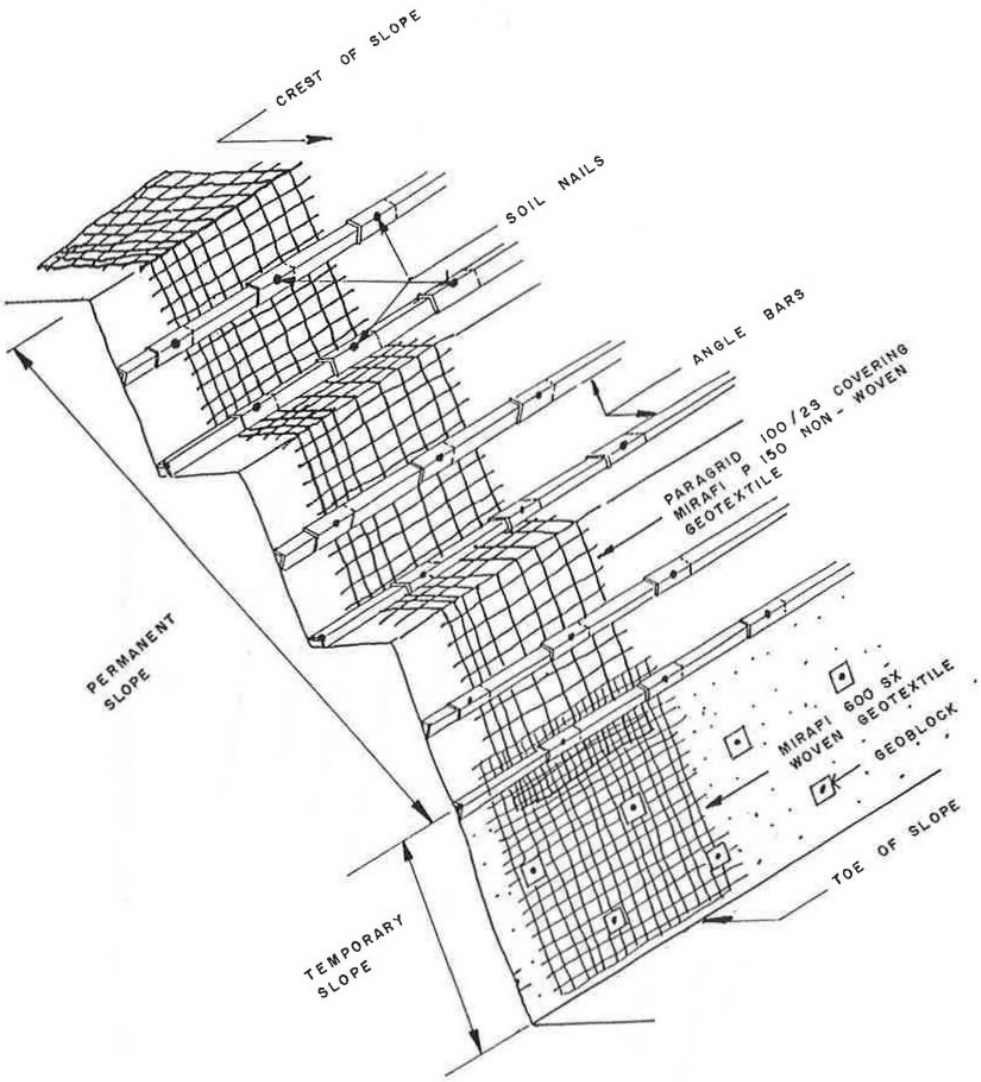


FIGURE 2 Schematic illustration of geotextile-geogrid surface covering of permanent and temporary slopes.

The action of drawing the angle bars to the face of the slope was used to tension the membrane and bring it into contact with the profiled slope. The loads in the membrane and angle bars were determined using the method given in *NCHRP Report 290* (*1*).

After the geogrid-geotextile complex was installed and tensioned, a sand, topsoil, and water slurry mixed in approximately 1.5:1:1 proportions was injected between the geotextile and the native soil as a void filler. The mixture was injected using a gravity-fed funnel and tubing arrangement with pressure being applied with a 1.5-m head of slurry. This method was adopted after trials using a slurry pump to inject the mixture found the pump to be much less effective than was hoped.

Circumstances necessitated a change in facing membrane design over part of the length of the retained slope. A wet area in the slope face (later found to be a consequence of a leak in the water main buried in the crest of the slope) collapsed continually and progressively, and winter came before the area could be stabilized. The face of this portion of the slope remained unrestrained throughout the winter. The winter of 1988–1989 was severe on the incomplete structure in that there were many freeze-thaw cycles and heavy rains alternated with snow. As a consequence, the completely unprotected portion of the "terraced" slope deteriorated and the face slumped to expose a succession of steep scarp slopes. To reduce additional deterioration of the partly slumped slope through the main spring thaw, a restraining membrane of nonwoven geotextile held in place by a timber grillage connected to the soil nails through the angle bars was applied to the soil face. In the spring, a new face had to be constructed and connected to the soil nail reinforcing system.

The face design selected for reconstruction of the deteriorated area of slope featured a facing of geocomposite (Geoweb infilled with sand) with a geogrid-reinforced granular infill placed between the geocomposite and the face of the scarp slope. The design of this section of face was developed on the presumptions that soil nails stabilized the retained slope and that the geocomposite would be required to support the forces developed in the active wedge of soil; these forces were transferred from the face into a reinforced backfill, and to the nails, with horizontally laid sheets of geogrid. The tensile forces in the geogrid and the active pressure were calculated using the method described by Jones for a sloping reinforced structure (*6*).

The Geoweb material was manufactured from 200-mm-wide strips of high-density polyethylene ultransonically welded together to give an open-cell construction having a cell diameter of 200 mm. The concept behind the use of this material in earth works is that when the Geoweb is expanded and infilled with granular material, the cells of the web laterally confine the soil to provide a stable soil-geosynthetic composite. For slope works, the cell structure effectively confines the infill soils so that raveling or sloughing of the slope face is prevented. The Geoweb fascia was also chosen because it is easy to form, it facilitates longitudinal wall contours, and it can be placed rapidly by unskilled labor with a minimum of training; in addition, the exposed cells in the structure can be vegetated (*7*). The geogrid reinforcement for the reconstructed portion of the slope was provided using an uniaxial geogrid, also of multistrand polyester construction, with an LTADL of 15 kN/m. The grid was installed at 0.6-m vertical spacings, and every other grid sheet (or every third, depending on vertical positioning) was tied back to the angle bars, which were in turn attached to the soil nails, thereby connecting the geocomposite facing to the soil nail reinforcement system. Pullout resistance of the geogrid at the geocomposite facing was attained by frictional forces (Figure 3).

In the temporary portion of the slope, the purposes of the surface membrane were to control surface erosion and to retain any minor dislodgements that might occur in the face of the slope. This slope was constructed without any architectural features, at an inclination of 10 degrees to the vertical. For the stated purposes a woven geotextile constructed of polypropylene material was thought to be acceptable for the 2-month design lifespan of the surfacing membrane, and a woven geotextile with a tensile strength of 1350 N was applied (Figure 2). This geotextile was secured to the soil nails with 300-mm^2 blocks ("Geoblocks") attached to the nails by nuts threaded onto the soil nails. Void filler was not injected behind this temporary membrane because of the expected limited design lifetime of this section and the presumption that it would not be exposed to freezing conditions.

CONSTRUCTION OF SOIL NAIL SYSTEM

The threaded steel bar soil nails were installed in 100-mm-diameter predrilled holes at the design locations in the slope. It was found that in the "dry" areas of the face, the drill holes would stay open without collapse for a few hours. This was long enough to allow several holes to be drilled successively, the threaded bars to be installed in the drilled holes (positions maintained with centralizers), and the bars to be grouted into position by pumping the grout through a 25-mm-diameter tube taped to the bars. The tubes fed the grout to the in-ground end of the nail. The grout was pumped in at low pressure and filled the annulus between the bar and the enclosing soil from the in-ground end outward. Grouting of the annulus was judged complete when uncontaminated grout exited freely at the face of the slope; then the tube was withdrawn.

Where the drill holes intercepted one (or more) of the perched water tables, the unsupported hole collapsed when conventional air drilling methods were used, and soil and groundwater would flow out of the face. At these locations, the drill hole would remain temporarily stable when the drilling was carried out using a venturi bit that injected a flushing air and water mixture both backward and forward. These drill holes were grouted immediately on completion, before the next installation was begun. Several times it was necessary to redrill a hole because of collapse, but never more than once.

At the design stage, the adhesion between the soil nail and the enclosing soil was evaluated using an adhesion factor applied to the overburden pressure (*8*). To verify the tensile capacity of the installed nails, cyclic loading tests were carried out on four specially installed test nails. The results for these tests are presented in Figure 4. Tests were carried out more than 7 days after nail installation. The ultimate tensile load capacity of these nails was estimated using the method developed by Chin (*9*) for evaluating the ultimate load-carrying capacity of a piled foundation that has not been taken to

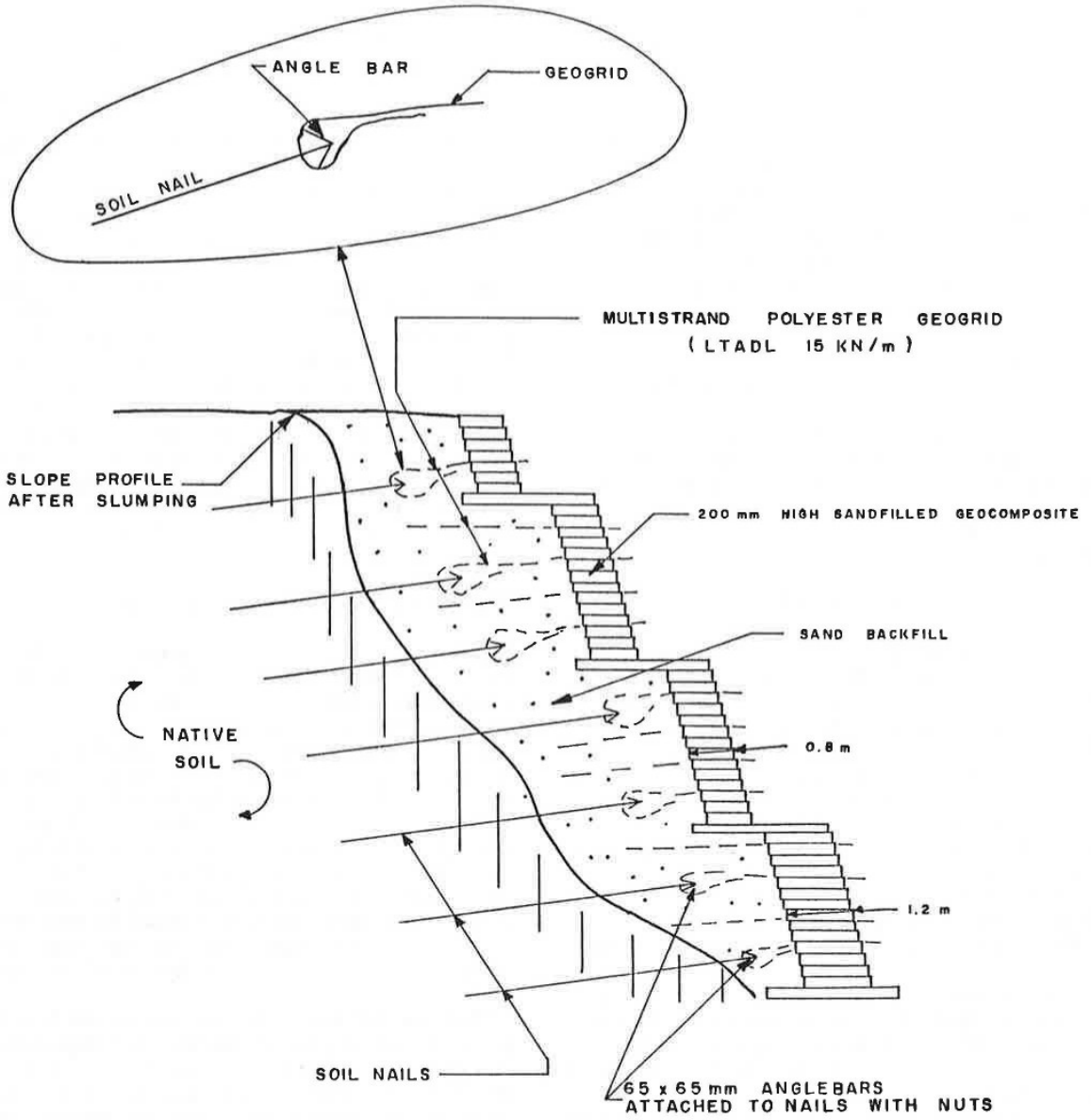


FIGURE 3 Schematic illustration of slope facing construction using geocomposite.

failure. For this analysis, the behavior of the nail was plotted using deflection normalized against tensile load versus deflection of the nail head (Figure 5). The failure load was evaluated from the inverse slope of the plot, and the test results were interpreted to give ultimate adhesions between the grout and the enclosing soil ranging from 225 to 700 kPa. The depth of each of the test nails below the crest and a qualitative estimate of the moisture condition of the soil are given in Table 2. These results illustrate the influence of the moisture condition of the soil on the available adhesion. They also indicate that for the heavily overconsolidated soils found at the subject site, this factor was of greater influence than the overburden pressure.

PERFORMANCE OF STRUCTURES

The performance of the soil nail retaining structure at the site has been generally satisfactory through two winters. The unforeseen major delay that occurred during construction of the building substructure resulted in the temporary slope's having to remain operational well beyond its design lifespan. The

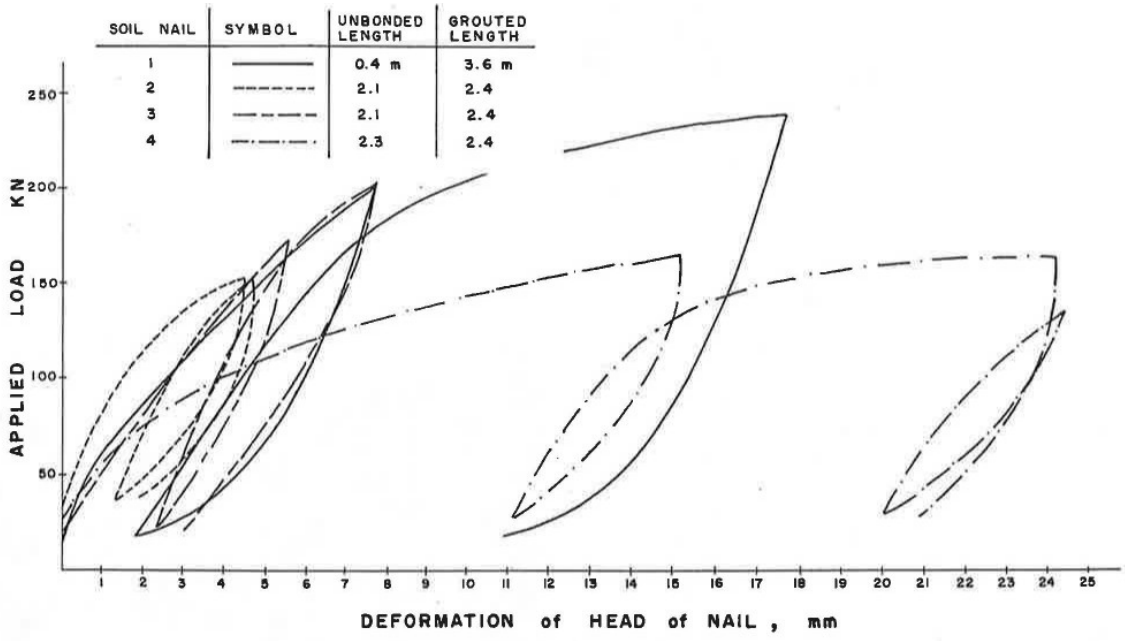


FIGURE 4 Tensile test results for soil nails.

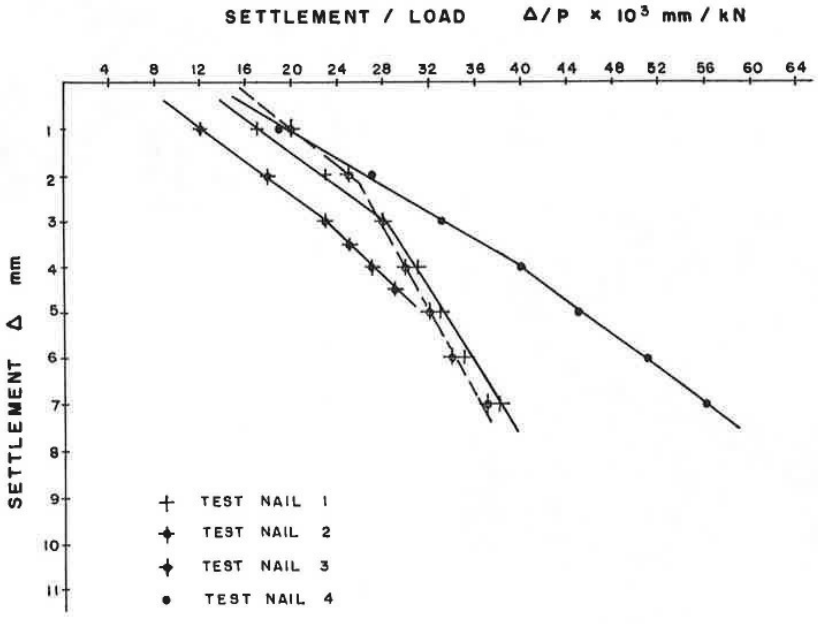


FIGURE 5 Summary of tensile test results for soil nails.

TABLE 2 TENSILE TEST RESULTS FOR TEST NAILS

Test Nail No.	Depth Below Crest of Slope (m)	Moisture Condition of Soil	Representative 'N'-value (blows/0.3 m)	Unbonded Length (m)	Grouted Length (m)	Measured Ultimate Adhesion (kPa)
1	3.0	moist	50	0.4	3.6	700
2	9.0	moist	100	2.1	2.4	475
3	12.0	moist	100	2.1	2.4	600
4	12.0	wet	100	2.3	2.4	255

consequence of this delay was that the temporary slope, including the relatively weak woven geotextile surface membrane, was required to remain in service throughout the entire winter season and spring thaw. During this time, some relatively large slabs of soil dislodged from the soil face, primarily as a consequence of freezing and thawing, and these slabs were too heavy for the woven geotextile surface retention membrane; this resulted in tearing of the geotextile and detachment of the membrane from the restraining blocks that had secured it to the soil nails. Additional support to the temporary slope covering membrane was supplied with a timber grillage in areas where the service life of this slope support system had to be prolonged (Figure 6), and this application was successful.

The unfaced portion of the permanent slope suffered surface deterioration and slumping, and the surface profiling was lost through a succession of minor slumps in the face. This necessitated the rebuilding of the face using the geocomposite wall described previously. There was some loss of the design shape in the portion of the terraced slope that had been covered with geogrid and geotextile, but which had not had the void-filling slurry injected beneath the surfacing membrane before the onset of winter.

The permanent slope at the Cambridge site is now structurally complete; it is being vegetated with creeping vines as visual screening. Photographs of the geogrid-surfaced and geocomposite-faced areas of the slope are presented in Figures 7 and 8. Since the void filler was injected under the geotextile-geogrid surface area of the slope, there has been no further deterioration, and the slope support is satisfactory.

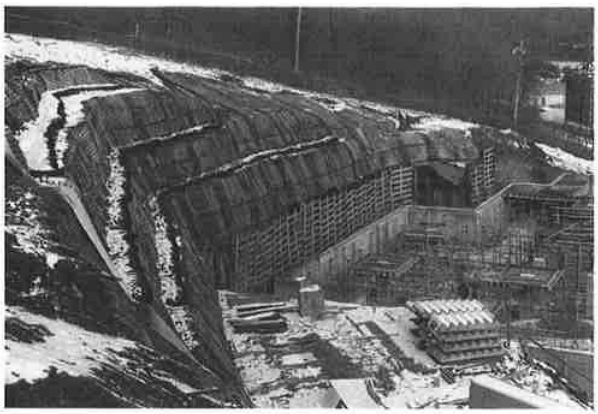


FIGURE 6 Geotextile-geogrid–faced permanent slope and geotextile-timber-grillage–faced temporary slope.

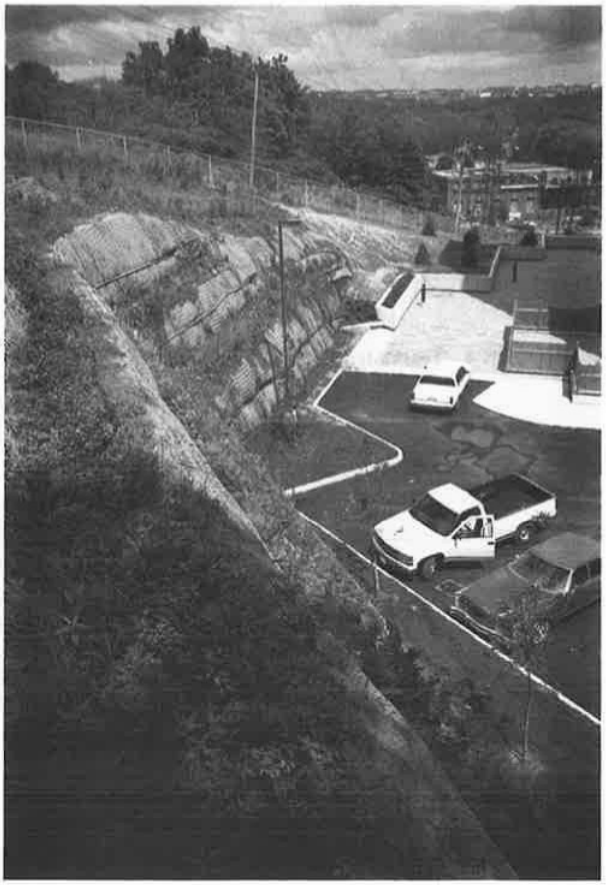


FIGURE 7 Completed slope in area of Figure 6.

The geocomposite facing has not experienced any problems. Both facing systems permit free drainage of wet areas of the face.

The postconstruction monitoring program for the nailed wall consists of two detailed visual inspections of the slope each year: after the spring thaw and before the onset of winter.

CONCLUDING REMARKS

At the subject site, the adoption of a soil nail solution enabled the project to proceed without surface intrusion into neighboring lands. The cost of the soil nail structure was 40 percent