

Design of geosynthetic-reinforced veneer slopes

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ABSTRACT: This paper provides a framework for the design of steep reinforced veneer slopes such as soil covers in landfill facilities. Instead of using geosynthetic reinforcements along the veneer slope, an approach that becomes unsuitable for high slopes, the proposed framework analyzes the use of horizontally placed inclusions within the veneer slope. Analytical expressions are derived, which are useful for preliminary parametric evaluations and final design. A reinforced veneer was designed using the approach proposed herein for the cover of steep slopes at the Operating Industries, Inc. (OII) Landfill. Design criteria include requirements that the final cover should control percolation, resist erosion, and it should remain stable for both static and seismic conditions. The cover selected and constructed includes a 1.8-m thick layer of selected soil that, in order to satisfy stability criteria, was reinforced using horizontally placed geogrids. The geogrid reinforcements were embedded into the underlying solid waste mass in order to provide adequate pullout resistance. Construction of the reinforced veneer at this site was completed recently, and involved stripping the in place cover soil, screening the soil for reuse, and placing an engineered evapotranspirative cover reinforced with geogrid layers.

1 INTRODUCTION

The design of veneer slopes (e.g. steep cover systems for waste containment facilities) poses significant challenges to designers. The use of uniaxial reinforcements placed along the slope (under the veneer and above a typically strong mass of soil or solid waste) and anchored on the top of the slope has been a common design approach. However, this alternative may not be feasible for steep, long veneer slopes. If the veneer slope rests on top of a comparatively stronger mass of soil, rock, or solid waste, an innovative, alternative approach consists of using uniaxial reinforcements placed horizontally (rather than along the slope) and anchored into the underlying mass. A framework for analysis and design of veneers reinforced using horizontally placed inclusions is presented in this paper. The framework is useful not only for final design, but also for parametric evaluations that may be performed to define the reinforcement requirements for this system. This approach is particularly suitable for reinforcement of landfill soil cover systems, constructed on top of comparatively strong solid waste, and for stabilization of protective soil layers (colluvium) on top of more resistant bedrock or residual soils.

A reinforced veneer approach was used to stabilize the final cover system for the North Slope of the Operating Industries, Inc. (OII) Landfill in southern

California. The final cover system of this hazardous waste landfill is an evapotranspirative cover. The design criteria for the North Slope were that the cover should control percolation, resist erosion, have a static factor of safety of at least 1.5, and lead to calculated deformations in the event of the design earthquake of less than 150 mm. Compliance with the stability criteria was achieved by including geogrids at 1.5-m vertical spacing within the cover and by embedding the geogrids into exposed refuse. The reinforced veneer was constructed on slopes as steep as 1.5 horizontal to 1.0 vertical to reinforce landfill slope sections as high as 65 m.

2 ANALYSIS OF GEOSYNTHETIC-REINFORCED VENEER SLOPES

2.1 *General considerations*

This section presents an analytical framework for quantification of the reinforcement requirements for reinforced veneers where reinforcements are placed horizontally and embedded into a comparatively strong underlying mass. An infinite slope configuration is considered for evaluation of stability.

Although different definitions for the factor of safety have been reported for the design of reinforced soil slopes, the definition used in this study is relative to the shear strength of the soil:

$$FS = \frac{\text{Available soil shear strength}}{\text{Soil shear stress required for equilibrium}} \quad (1)$$

This definition is consistent with conventional limit equilibrium analysis, for which extensive experience has evolved for the analysis of unreinforced slopes. Current design practices for reinforced soil slopes often consider approaches that decouple the soil reinforcement interaction and do not strictly consider the factor of safety defined by Equation (1). Such analyses neglect the influence of reinforcement forces on the soil stresses along the potential failure surface and may result in factors of safety significantly different than those calculated using more rigorous approaches.

Considering the normal and shear forces acting in a control volume along the veneer slope (or infinite slope), and assuming a Mohr-Coulomb shear strength envelope, Equation (1) can be expressed as:

$$FS = \frac{c + (N/L) \tan \phi}{S/L} \quad (2)$$

where N = normal force acting on the control volume; S = shear force acting on the control volume; L = length of the control volume; c = soil cohesion; and ϕ = soil friction angle.

2.2 Unreinforced veneer

In the case of an unreinforced veneer (Figure 1), the shear and normal forces required for equilibrium of a control volume can be defined as a function of the weight of this control volume. That is:

$$S = W \sin \beta \quad (3)$$

$$N = W \cos \beta \quad (4)$$

$$W = \gamma L T \quad (5)$$

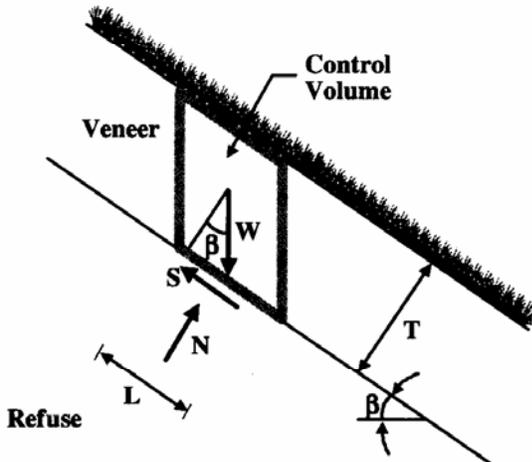


Figure 1. Unreinforced veneer.

where W = weight of the control volume; β = slope inclination; T = veneer thickness; and γ = soil total unit weight.

From Equations (2), (3), (4), and (5), the classic expression for the factor of safety FS_u of an unreinforced veneer can be obtained:

$$FS_u = \frac{c}{\gamma L T \sin \beta} + \frac{\tan \phi}{\tan \beta} \quad (6)$$

2.3 Reinforced veneer

In the case of a reinforced veneer (Figure 2), the shear and normal forces acting on the control volume are defined not only as a function of the weight of the control volume, but also as a function of the tensile forces that develop within the reinforcements. For the purpose of the analyses presented herein, the reinforcement tensile forces are assumed horizontal and represented by a distributed reinforcement tensile stress t , which corresponds to a uniformly distributed tensile force per unit height. In this case, the shear and normal forces needed for equilibrium of a control volume are defined by:

$$S = W \sin \beta - t H \cos \beta \quad (7)$$

$$N = W \cos \beta + t H \sin \beta \quad (8)$$

$$H = L \sin \beta \quad (9)$$

where H = vertical component of the length of the control volume.

From Equations (2), (5), (7), (8), and (9), the following expression can be obtained for the factor of safety FS_r of a reinforced veneer:

$$FS_r = \frac{\frac{c}{\gamma L T \sin \beta} + \frac{\tan \phi}{\tan \beta} + \frac{t}{\gamma T} \sin \beta \tan \phi}{1 - \frac{t}{\gamma T} \cos \beta} \quad (10)$$

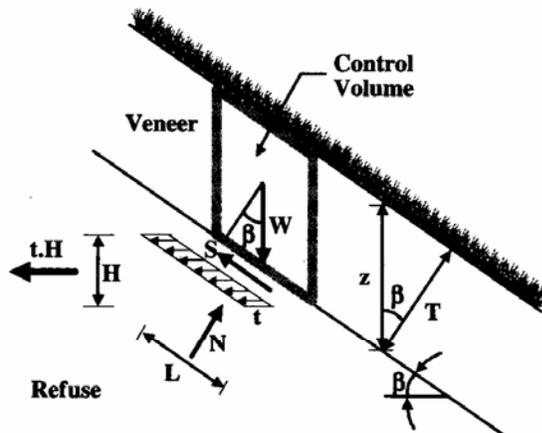


Figure 2. Reinforced veneer.

The equation above can be simplified by defining the normalized distributed reinforcement tensile stress t^* (dimensionless), as follows:

$$t^* = \frac{t}{\gamma T} \cos \beta \quad (11)$$

Using Equations (6) and (11) into Equation (10) leads to:

$$FS_r = \frac{FS_u + t^* \tan \beta \tan \phi}{1 - t^*} \quad (12)$$

Equation (12) provides a convenient expression for stability evaluation of reinforced veneer slopes. It should be noted that if the distributed reinforcement tensile stress t equals zero (i.e. in the case of unreinforced veneers), Equation (12) leads to $FS_r = FS_u$. When the soil cohesion c equals zero, Equation (10) can be simplified as follows:

$$FS_r = \frac{\tan \phi \left(\frac{1 + t^* \tan^2 \beta}{1 - t^*} \right)}{\tan \beta} \quad (13)$$

2.4 Determination of Reinforcement Requirements

Reinforcement requirements needed to achieve a target factor of safety FS_r of the reinforced veneer, expressed in terms of the normalized required distributed tensile stress t_{req}^* , which can be derived from Equation (12):

$$t_{req}^* = \frac{FS_r - FS_u}{FS_r + \tan \beta \tan \phi} \quad (14)$$

Similarly, reinforcement requirements that are needed to achieve a target factor of safety FS_r , expressed in terms of the required tensile stress t_{req} can be obtained from Equations (11) and (14) as follows:

$$t_{req} = \frac{FS_r - \frac{c}{\gamma L T \sin \beta} \frac{\tan \phi}{\tan \beta} \frac{\gamma T}{\cos \beta}}{FS_r + \tan \beta \tan \phi} \quad (15)$$

Equation (15) can be used to assess the reinforcement requirements for a given soil shear strength and veneer configuration (slope inclination and veneer thickness). For example, Figure 3 shows the reinforcement tensile stress required to achieve a factor of safety $FS_r = 1.5$ in a veneer slope where the soil shear strength is characterized by a cohesion $c = 5$ kPa and a friction angle $\phi = 30^\circ$. The figure shows the various combinations of veneer thickness and veneer slope inclination that satisfy the design criterion.

Additional aspects that should be accounted for in the design of reinforced veneer slopes include the evaluation of the pullout resistance (i.e. embedment length into the underlying mass), assessment of the factor of safety for surfaces that get partially into the

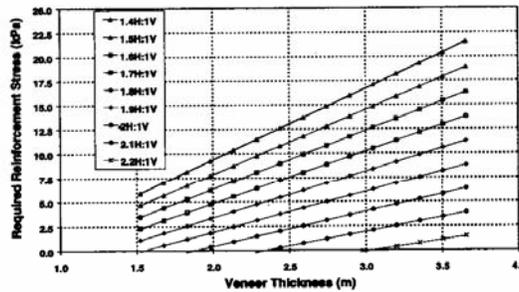


Figure 3. Reinforcement requirements to achieve a factor of safety of 1.5 (soil cohesion = 5 kPa; soil friction angle $\phi = 30^\circ$).

underlying mass, evaluation of reinforcement vertical spacing, and analysis of seismic stability of the reinforced veneer. These important considerations are beyond the scope of this paper but should be accounted for in the design of reinforced veneer slopes.

3 CASE HISTORY: GEOSYNTHETIC-REINFORCED VENEER SLOPE FOR A HAZARDOUS WASTE LANDFILL

A reinforced veneer was constructed as part of the final closure of the Operating Industries, Inc. (OII) landfill. This case history highlights the final closure of a hazardous waste landfill where the severe site constraints were overcome by designing and constructing an alternative final cover incorporating horizontal geosynthetic veneer reinforcement.

3.1 Site description

The 60-hectare south parcel of the OII landfill was operated from 1948 to 1984, receiving approximately 30-million cubic meters of municipal, industrial, liquid and hazardous wastes. In 1986, the landfill was placed on the National Priorities List of Superfund sites. Beginning in 1996, the design of a final cover system consisting of an alternative evapotranspirative soil cover was initiated, and subsequent construction was carried out from 1997 to 2000. The refuse prism, which occupies an area of about 50 hectares, rises approximately 35 m to 65 m above the surrounding terrain. Slopes of varying steepness surround a relatively flat top deck of about 15 hectares. Slopes on the north and east are generally the steepest with considerable portions of the North Slope as steep as 1.5:1 (horizontal:vertical).

3.2 Design criteria

The final cover design criteria mandated by the U.S. Environmental Protection Agency (EPA) primarily deal with the percolation performance of the cover,

static and seismic stability of the steep sideslopes of the landfill, and erosion control. The percolation design criteria required that the performance of the final cover system be hydraulically equivalent to or better than a layered regulatory cover (prescriptive cover) that includes a 300-mm thick barrier layer with a saturated hydraulic conductivity of 1×10^{-8} m/sec or less. The stability criteria were a static factor of safety of 1.5, and acceptable permanent seismically induced deformations less than 150 mm under the maximum credible earthquake. The basis of the seismic stability criteria is that some limited deformation or damage may result from the design earthquake, and that interim and permanent repairs would be implemented within a defined period.

3.3 Final design

One of the most challenging design and construction features of the project was related to the north slope of the landfill. The north slope is located immediately adjacent to the heavily traveled Pomona freeway (over a distance of about 1400 meters), rises up to 65 meters above the freeway, and consists of slope segments as steep as 1.5:1 and up to 30 m high separated by narrow benches. The toe of the North Slope and the edge of refuse extends all the way up to the freeway. The pre-existing cover on the North Slope consisted of varying thickness (a few centimeters to several meters) of non-engineered fill. The cover included several areas of sloughing instability, chronic cracking and high levels of gas emissions. The slope was too steep to accommodate any kind of a layered final cover system, particularly a cover incorporating geosynthetic components (geomembranes or GCL). Because of the height of the slope and lack of space at the toe, it was not feasible to flatten the slope by pushing out the toe, removing refuse at the top, or constructing a retaining / buttress structure at the toe of slope.

After evaluating various alternatives, an evapotranspirative cover constructed in a monolithic fashion (monocover), and incorporating geogrid reinforcement for veneer stability was selected as the appropriate cover for the North Slope. The evapotranspirative cover had additional advantages over traditional layered cover systems, including superior long-term percolation performance in arid climates, ability to accommodate long-term settlements, construct ability, and ease of long-term operations and maintenance. The selected cover system included the following components, from the top down: 1) vegetation to promote evapotranspiration and provide erosion protection; 2) a 1.2 m – thick evapotranspirative soil layer to provide moisture retention, minimize downward migration of moisture, and provide a viable zone for root growth; and 3) a foundation layer consisting of soil and refuse of variable thickness to provide a firm foundation for the soil cover system.

The detailed design of the cover system was preceded by an extensive laboratory test program to characterize the shear strength, hydraulic characteristics (moisture retention properties and unsaturated hydraulic conductivity), and shrinkage (desiccation cracking) potential of on-site and imported cover soils. The hydraulic equivalence of the evapotranspirative cover to the prescriptive cover was demonstrated by modeling the percolation through both covers. Modeling was performed under simulated rainfall conditions for 30- and 100- year periods, included parametric studies to evaluate the effects of cover thickness and degradation, vegetation, and irrigation.

The potential critical modes of slope movement for the North Slope were relatively shallow failures through the cover soils and/or soil/refuse interface. Due to the relatively high strength of the refuse mass, the 1.5:1 slopes had a sufficiently high factor of safety against deep-seated movement. Static stability analyses of the cover soil veneer were based on effective stress parameters from backpressure saturated consolidated undrained (C-U) triaxial tests conducted at relatively low confining pressures (24, 48, and 96 kPa) to simulate cover conditions. Pseudostatic stability analyses to support seismic deformation analyses were based on total stress parameters from C-U triaxial tests performed on soaked samples. Unsaturated flow modeling of the evapotranspirative cover on the North Slope indicated that saturation of the cover and seepage parallel to the slope face, which is a usually assumed design condition for surficial stability analyses of slopes, was unlikely to occur. Therefore, stability analyses for the North Slope veneer were performed assuming saturated conditions (back-pressure saturated shear strength parameters and saturated unit weight) for the cover soils, but no perched water table or seepage.

Stability analyses showed that for most available monocover materials, compacted to practically achievable levels of relative compaction on a 1.5:1 slope (90% of modified Proctor or 95% of Standard Proctor), the minimum static and seismic stability criteria were not met. Veneer geogrid reinforcement with horizontally placed geogrids was then selected as the most appropriate and cost-effective method for stabilizing the North Slope cover. The analytical framework discussed in Section 2 was used in the design. For the given cover veneer configuration (veneer thickness, slope inclination and shear strength of cover soils) the minimum reinforcement required to achieve a static factor of safety of 1.5 was evaluated from charts such as that shown in Figure 3, developed from Equation (15) for a soil shear strength characterized by a cohesion of 5 kPa and a friction angle of 30°. The type of geogrid, vertical spacing and minimum embedment that are required to provide the minimum reinforcement stress were then adopted. Figure 4 shows the typical ve-

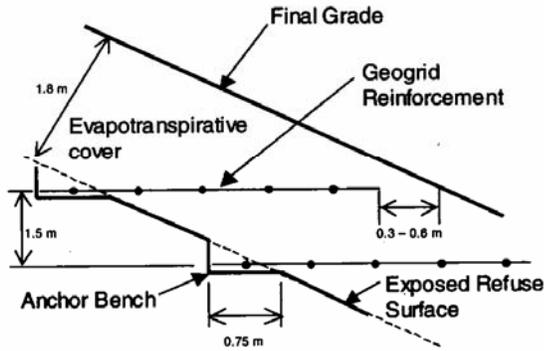


Figure 4. Typical veneer reinforcement detail.

neer reinforcement detail selected based on the shear strength of the soils used in construction.

The veneer reinforcement consisted of polypropylene uniaxial geogrids, installed at 1.5-meter vertical intervals for slopes steeper than 1.8:1, and at 3-meter vertical intervals for slopes between 2:1 and 1.8:1. The geogrid panels are embedded a minimum of 0.75 meters into the exposed refuse slope face from which the pre-existing cover had been stripped. The geogrid panels were curtailed approximately 0.3 to 0.6 meters away from the finished surface of the slope cover. This was done to permit surface construction, operation and maintenance activities on the slope face without the risk of exposing or snagging the geogrid.

3.4 Construction

The pre-existing non-engineered cover on the North Slope was generally unsuitable to be left in place and was completely removed prior to construction of the new reinforced veneer. The original cover was stripped by a fleet of scrapers and dozers, starting at the top of the slope. Stripping was generally extended to the refuse. The stripped slope face was generally excavated to a 1.5:1 slope with local areas as steep as 1:1. The stripped soil was screened through a rotary (Trommel) screen to remove refuse, vegetation and oversize particles, and reused as engineered evapotranspirative cover. Refuse exposed on the slope face was covered with a sprayed-on temporary cover (Posi-shell) to control erosion, refuse migration and odors.

Once the entire North Slope was stripped, construction of the geogrid-reinforced veneer commenced from the base of the landfill. To create the geogrid embedment bench into the exposed stripped slope face, further excavation of the refuse and soil had to be undertaken. The excavated refuse and soil from the embedment bench was generally incorporated and compacted into the foundation zone of the cover. The cover fill was constructed in horizontal

lifts utilizing scrapers, dozers and sheepsfoot compactors. Since the entire existing cover was stripped off the North Slope surface, the fill included the 1.2-meter thick evapotranspirative cover and the 0.6-meter thick foundation layer, placed as a monolithic 1.8-meter thick veneer. The fill was generally placed along narrow working benches, approximately 4.5 meters in width. When the specified geogrid placement elevation (every 1.5 m vertical intervals on slopes steeper than 1.8:1, and 3m vertical intervals on flatter slopes) was reached, the embedment bench was created as described above and the geogrid was placed over the compacted fill. Geogrid panels were pre-cut to the required length and placed adjacent to each other with a 150 mm overlap. The overlap was in the direction of scraper traffic and the geogrid panels were not attached to the subgrade. This placement pattern was found to be the least disruptive to the geogrid panels when scraper traffic and fill placement on top of the geogrid was undertaken. The geogrid panels were free to slide over one another rather than create 'waves' and/or 'buckle' when the scraper train was driven over the geogrid layer.

The surface on which the geogrid was placed was intentionally kept rough, typically by scarifying prior to placement. This was done to encourage bonding between geogrid and soil and to avoid the formation of horizontal laminations caused by placing geogrids on a smooth surface. To verify the adequacy of the placement method, test trenches were excavated into completed portions of the reinforced cover. Observations showed an intimate contact between the soil and geogrid, and an absence of horizontal laminations or voids adjacent to the geogrid.

Construction of the North Slope was accomplished in 12 months. Approximately 500,000 cubic meters of soil and 170,000 square meters of geogrid were placed. Total area of geogrid placement exceeded 9.3 hectares. The maximum height of reinforced portion of the landfill slopes was 55 m (the maximum height of the total landfill slope was 65 m).

4 CONCLUSIONS

A framework is provided for the design of steep reinforced veneer slopes such as soil covers in landfill facilities. Instead of using geosynthetic reinforcements along the veneer slope, the proposed framework analyzes the use of horizontally placed inclusions within the veneer slope. Analytical expressions provide the reinforcement requirements for veneer slopes as a function of the soil shear strength and the veneer configuration (thickness and slope inclination).

A reinforced veneer was designed using the approach proposed herein for the cover of steep slopes

at the Operating Industries, Inc. (OII) Landfill. Design criteria include requirements that the final cover should control percolation, resist erosion, and it should remain stable for both static and seismic conditions. The cover selected and constructed includes a 1.8-m thick layer of selected soil that, in order to satisfy stability criteria, was reinforced using horizontally placed geogrids. The geogrid reinforcements were embedded into the underlying solid waste mass in order to provide adequate pullout resistance. The use of horizontally placed geosynthetic reinforcements led to a technically sound and economically feasible approach for stabilization of an

up to 55 m high soil cover placed over typically 1.5:1 (H:V) landfill slopes.

5 ACKNOWLEDGEMENTS

The authors are indebted to New Cure, Inc., responsible for the site activities described herein, Foster Wheeler Environmental Corporation, general contractor for design and construction, and GeoSyntec Consultants and Advanced Earth Sciences, for contributions to the design and quality control programs of the case history described as part of this paper.