

## LimitState:GEO Case Study

### Geogrid Reinforced Embankment Stability Analysis

#### Summary

The stability of a reinforced embankment of summit width 14m and side slopes 1:1.75 was analysed using LimitState:GEO. The embankment was constructed from sand ( $\phi'=30^\circ$ ,  $\gamma=18.1 \text{ kN/m}^3$ ) and reinforced with 6 horizontal layers of geogrid at 0.6m vertical spacing. The embankment was founded on a 2.65m stratum of silt ( $c' = 7 \text{ kN/m}^2$ ,  $\phi'=15^\circ$ ,  $\gamma=18 \text{ kN/m}^3$ ) overlying a 1.2m stratum of clay ( $c' = 11 \text{ kN/m}^2$ ,  $\phi'=10^\circ$ ,  $\gamma=18 \text{ kN/m}^3$ ) overlying a rigid base. The surface of the embankment was constructed of road superstructure material having assumed properties of  $c' = 0 \text{ kN/m}^2$ ,  $\phi'=45^\circ$ ,  $\gamma=20 \text{ kN/m}^3$ .

Assuming an interface friction coefficient for the geogrid of  $\mu=0.99$ , the embankment was found to have an additional factor of safety on soil strength of approximately 2.4 over and above the pre-applied Eurocode 7 Design Approach 1, Combination 2 partial factors, with the failure mechanism depicted in Figure 1. In this mechanism the embankment behaves essentially as a single entity, extruding the soil beneath it.

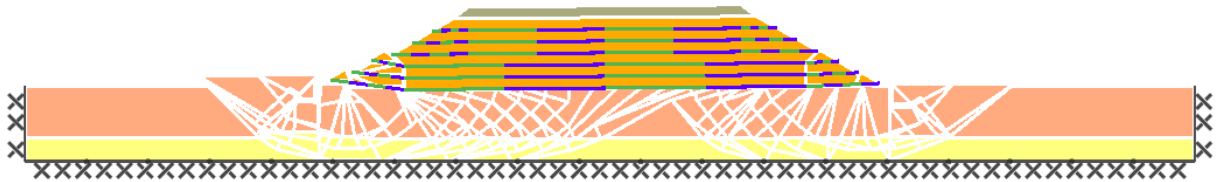


Figure 1: Failure mechanism for friction coefficient for the geogrid of  $\mu=0.99$

#### Problem formulation

##### *Factor of safety*

In a conventional method of slices type analysis, slip surfaces are guessed and the ratio of restoring to driving moments/forces calculated for each slip surface. This ratio is normally equivalent to a factor on soil shear strength, and the lowest value is sought. This approach is valid if the set of assumed slip surfaces includes or closely approximates the critical one.

For a general purpose computational limit analysis approach such as is used in LimitState:GEO, the critical failure mechanism is found from millions of possibilities and is unrestricted in shape. In order to determine the critical mechanism, it is necessary to drive the system to the Ultimate Limit State (ULS) by some additional loading and use the magnitude of this additional loading to assess safety. If additional

loading is required to drive the system from its working state to an Ultimate Limit State then the system can be considered safe (against ULS). However the margin of safety would normally need to be determined.

If negligible additional loading is required to generate an Ultimate Limit State, then the system is just on the point of failure. Thus if soil strength is factored by a factor  $F$  prior to an analysis which is subsequently found to be just on the point of failure then this factor  $F$  is the safety factor on strength for the problem. In general this will correspond to the factor that is obtained from a slip-circle analysis (assuming the slip circle analysis has found the critical mechanism).

In order to assess the modeled embankment at its Ultimate Limit State, the additional loading was applied via additional horizontal body forces (equivalent to applying a pseudo-static seismic horizontal acceleration  $A$ ). This is the generally recommended approach for slope stability type problems. Essentially this means that any value of  $A$  returned above 0 indicates that the system is stable, i.e. additional body forces are required to cause failure. Otherwise a result of 'unstable' is returned by the software.

We thus seek a factor on strength  $F$  that generates a value of  $A$  of just zero.

Note that for a Eurocode Design Approach 1, Combination 2 (DA1/2) check, it is only necessary to pre-apply the Eurocode partial factors and solve the problem. Any value of  $A$  greater than zero indicates stability and that design actions are less than design resistances. Thus only one calculation is required.

Due to the symmetry of the problem, it is arbitrary as to whether the horizontal acceleration is applied to the left or right. Here it is applied to the left.

### ***Soil reinforcement***

The horizontal geogrid reinforcement layers were modeled using the Engineered Element feature in the software which allows general purpose longitudinal and lateral pullout factors to be defined.

The Engineered Element model allows soil-reinforcement to be modeled very flexibly and allows complex failure mechanisms to be identified that act through and around the reinforcement, while retaining the equivalent fundamental mechanics adopted in conventional (e.g. the tie- back wedge) methods of analysis. The model is thus not restricted to e.g. representing the action of reinforcement as a single equivalent force acting on a slip circle. The longitudinal pullout factor  $T_q$  was evaluated as  $2 \cdot \mu \cdot \tan \phi / \gamma_{Tq}$  where  $\mu$  is a friction modifier coefficient,  $\phi$  is soil friction angle and  $\gamma_{Tq}$  is the partial factor applied on the pullout factor. The latter was increased whenever partial factors on soil strength changed, by the same factor  $f$ , later described. The software then computes the pullout resistance of the geogrid as  $T_q \sigma'_v$  per unit width, where  $\sigma'_v$  is the nominal vertical effective stress acting on the centre of the geogrid.

In addition to pullout, it is also necessary to allow for the possibility for a body of soil to slide along the surface of a geogrid. Therefore the geogrid was also assigned an interface shear strength given by  $\phi = \tan^{-1}(\mu * \tan \phi)$ .

The lateral factor  $N_c$  was modeled as essentially zero (a value of 0.01 kPa was used to prevent reinforcement 'floating' out of the soil). This is equivalent to allowing the geogrid to deform or flex laterally with any soil movement.

### ***Problem geometry and soil properties***

Analyses were performed as long term analyses and at this stage, no water table has been modelled. The nodal density used was that with a target number of 300. The problem is to be checked against Eurocode DA1/2.

If no reinforcement in the embankment is modeled then the embankment side slopes are just on the point of failure since the slope angle is close to the angle of friction of the embankment soil.

However if only the horizontal reinforcement layers are modeled, the embankment is also found to collapse, this time with a localized slope failure between the reinforcing layers. To prevent this local failure a thin surface layer of concrete was modeled on the slope surface that extended into the base layer (with similar geometry to that indicated for the geosynthetic concrete framework in the provided drawings). This concrete layer was not connected to the horizontal reinforcement element. It was found that the presence of the concrete framework and the horizontal reinforcement generally led to the embankment behaving as a single structure.

### ***Application of partial factors***

The problem was analysed using Eurocode DA1/2 partial factors on soil strength. Initial factors of 1.25 were applied to the drained soil strength  $c'$  and  $\tan \phi'$ , and a factor of 1.4 to the pullout strength (based on an unfactored value of  $\tan \phi'$ )

In order to determine the additional margin of safety present in the construction, the problem was studied by multiplying the original partial factors by an additional coefficient  $f$  (greater than 1). 12 steps of factoring are considered; each step considers the same factor  $f$  on soil strength and pullout resistance.

So, for example, in STEP3 (as shown in Table 1),  $f = 1.2$  and the partial factors applied are:

$$\gamma_{c'} = f * 1.25 = 1.2 * 1.25$$

$$\gamma_{\tan \phi'} = f * 1.25 = 1.2 * 1.25$$

$$\gamma_{T_q} = f * 1.4 = 1.2 * 1.4$$

Analyses are carried out until the problem becomes unstable (equivalent to  $A < 0$ ).

## Results

	$\mu=0.99$					
	$f$	$\gamma_{c'}$	$\gamma_{\phi'}$	$\gamma_{Tq}$	$T_q$	$A$
STEP 1	1	1.25	1.25	1.4	0.816538	0.2044
STEP 2	1.1	1.375	1.375	1.54	0.742307	0.1785
STEP 3	1.2	1.5	1.5	1.68	0.680448	0.1566
STEP 4	1.5	1.875	1.875	2.1	0.544359	0.1046
STEP 5	1.6	2	2	2.24	0.510336	0.091
STEP 6	1.7	2.125	2.125	2.38	0.480316	0.07903
STEP 7	1.8	2.25	2.25	2.52	0.453632	0.06845
STEP 8	2	2.5	2.5	2.8	0.408269	0.0487
STEP 9	2.2	2.75	2.75	3.08	0.371154	0.03003
STEP 10	2.4	3	3	3.36	0.340224	0.01076
STEP 11	2.5	3.125	3.125	3.5	0.326615	unstable
STEP 12	2.6	3.25	3.25	3.64	0.314053	unstable

Table 1: Analysis of embankment stability, varying additional factor on strength  $f$  (geogrid interface shear strength parameter  $\mu=0.99$ ).

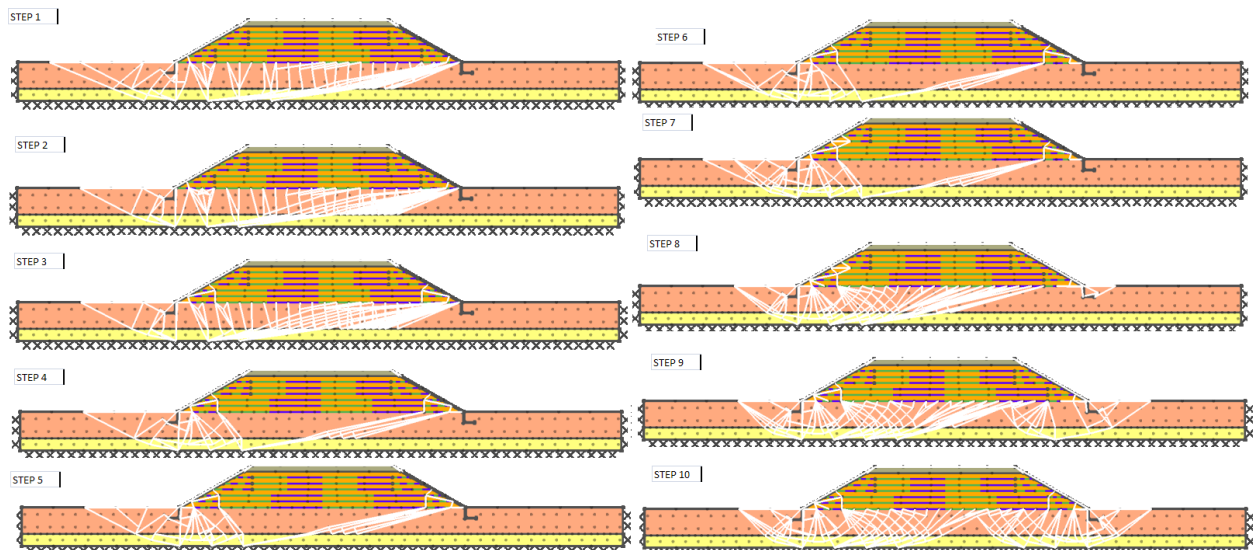


Figure 2: Change in embankment failure mechanisms, with change in additional factor on strength  $f$ , and corresponding horizontal acceleration  $A$  (geogrid interface shear strength parameter  $\mu=0.99$ ).

	$\mu=0.99$	$\mu=0.5$	$\mu=0.25$
	A	A	A
STEP 1	0.2044	0.2032	0.1973
STEP 2	0.1785	0.1765	0.1704
STEP 3	0.1566	0.1534	0.1467
STEP 4	0.1046	0.1012	0.09431
STEP 5	0.091	0.08804	0.08128
STEP 6	0.07903	0.07641	0.06944
STEP 7	0.06845	0.0661	0.05816
STEP 8	0.0487	0.04615	0.03864
STEP 9	0.03003	0.02719	0.01796
STEP 10	0.01076	0.005697	unstable
STEP 11	unstable	unstable	unstable
STEP 12	unstable	unstable	unstable

Table 2: Analysis of embankment stability, varying additional factor on strength  $f$ , for different geogrid interface shear strength parameters  $\mu$ .

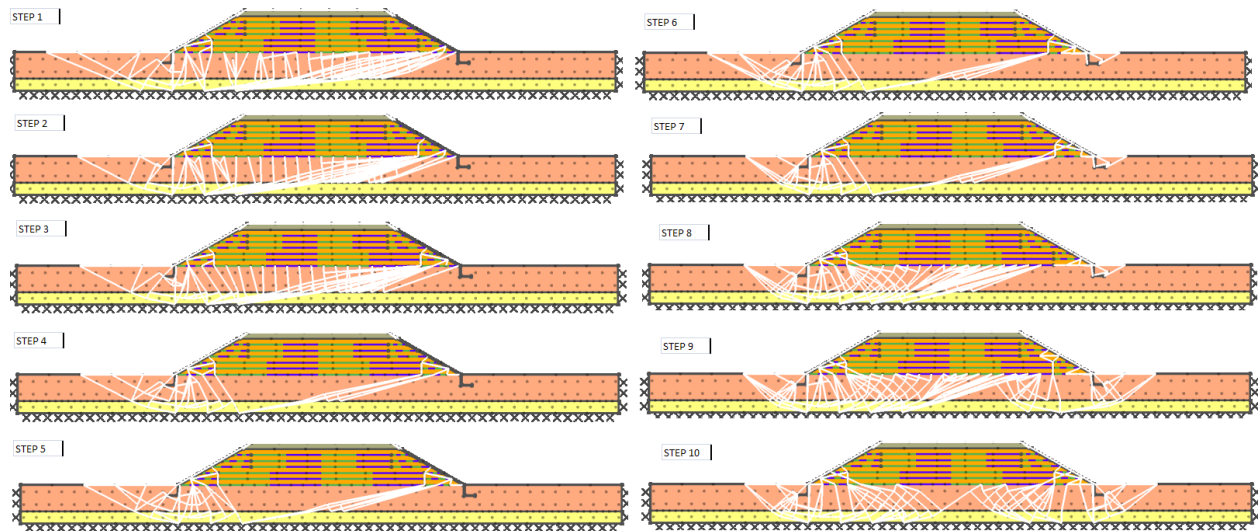


Figure 3: Change in embankment failure mechanisms, with change in additional factor on strength  $f$ , and corresponding horizontal acceleration  $A$  (geogrid interface shear strength parameter  $\mu=0.5$ ).

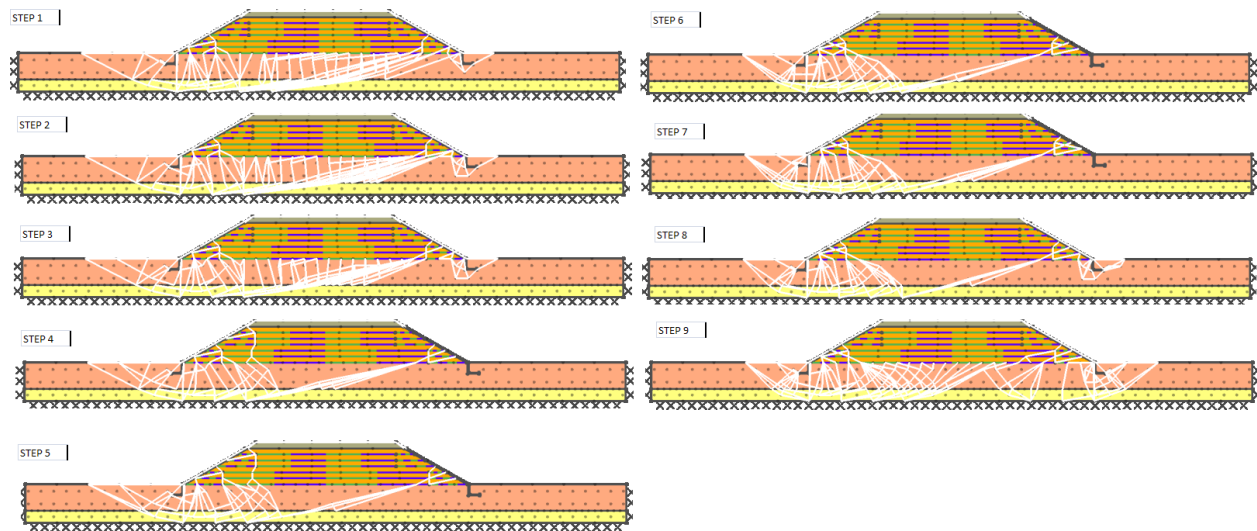


Figure 4: Change in embankment failure mechanisms, with change in additional factor on strength  $f$ , and corresponding horizontal acceleration  $A$  (geogrid interface shear strength parameter  $\mu=0.25$ ).

## Discussion

The modeled failure mechanisms and their dependence on the factor on strength were little influenced by the geogrid friction modifier coefficient  $\mu$  and generally have a similar shape and size. At the larger factors on strength (from around Step 8) onwards, the failure mechanism tends towards a symmetrical bearing capacity type failure mechanism as would be expected (at this level of strength factoring, the horizontal accelerations required to generate failure are becoming negligible).

The additional factor of safety on strength  $f$  (over and above the Eurocode DA1/2 factors) against ULS for the embankment corresponds to the value adopted when the returned result from the analysis transitions from a small positive value of  $A$  to 'unstable'. From Table 1, this can be seen to be approximately 2.4 for an interface friction coefficient for the geogrid of  $\mu=0.99$ .

## Conclusions

- LimitState:GEO was able to identify the critical mechanism which was not restricted to a simple shape (e.g. a slip circle).
- Assuming an interface friction coefficient for the geogrid of  $\mu=0.99$ , the embankment was found to have an additional factor of safety on soil strength of approximately 2.4 over and above the pre-applied Eurocode 7 Design Approach 1, Combination 2 partial factors, with the failure mechanism depicted in Figure 1. In this mechanism the embankment behaves essentially as a single entity, extruding the soil beneath it.
- Modifying the value of the friction modifier coefficient  $\mu$  to 0.5 or 0.25, made little significant difference to the factor on safety on strength for the problem.

For more information: [www.limitstate.com/geo](http://www.limitstate.com/geo)

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