Design and construction of a reinforced soil avalanche barrier at Neskaupstaður

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ABSTRACT
Reinforced soil structures offer a number of advantages for the construction of avalanche barriers. This paper describes the design and construction of a 14m high, reinforced soil avalanche barrier constructed at Neskaupstaður in Iceland. It explains the basic design procedure and describes how on-site installation damage testing was employed to assess the effects of using a non-standard fill material on the soil reinforcement.

Keywords: Reinforced soil, avalanche barrier

1 INTRODUCTION

As part of a programme to protect towns and villages in Iceland from the effects of avalanches, a range of protection schemes have been constructed. In certain locations protection is provided by large earth barriers. Two phases of barrier construction have been implemented at Neskaupstaður in north east Iceland. The first phase, completed in 1999, related to the snow path named Drangagil. It consists of a 12m high main catching barrier and smaller breaking barrier protecting the eastern part of the town. The second phase of protection for the western side of the town was constructed in 2012 and related to the avalanche path named Tröllagil. This paper explains the design and construction of the main catching barrier constructed as part of the second phase. The barrier is a Reinforced Earth® structure constructed using the GeoTrel® system supplied by Reinforced Earth Company in the United Kingdom.

The overall scheme is further summarised in Annex D of the European Commision report "The design of avalanche protection dams".

2 STRUCTURE GEOMETRY

The main catching barrier is approximately 650m long and has a facing height of up to 14.2m. The front face has an inclination of 1h:4v. There is a 5m wide horizontal section at the top of the barrier and the rear face of the structure has a slope of 2h:1v. The geometry is shown in Figure 1.

The structure is a reinforced soil steepened slope with structural stability being provided by the interaction between the soil and layers of polymeric strips placed within the compacted soil. The front face is retained with a galvanized steel mesh panel connected to the soil reinforcement by galvanized steel hooks and a curved steel plate.

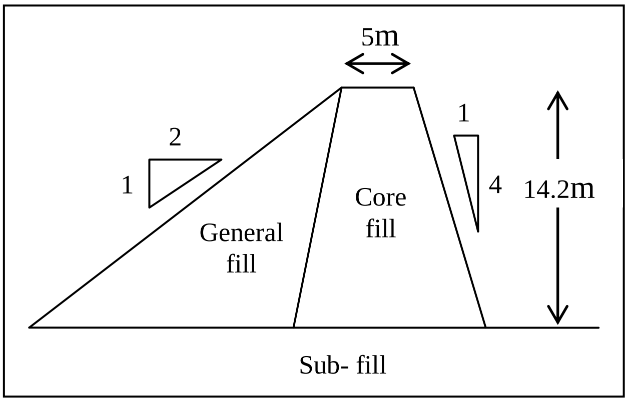

Figure 1 Basic structure geometry.

3 CONSTRUCTION MATERIALS

3.1 Fill materials

The core fill within the reinforced zone is a blasted and crushed igneous rock having a bulk density of 21.5kN/m^3 and a characteristic angle of internal friction of 45°. The general fill forming the rear face has a bulk density of 20kN/m^3 and a characteristic angle of internal friction of 35°. The barrier is constructed above a 3m deep sub-fill layer having the same properties as the core fill within the reinforced zone. All of the fill materials were excavated from local sources.

3.2 Facing panels

The facing panels forming the front of the structure are galvanised carbon steel panels manufactured from 8mm welded steel bars with a 100mm aperture spacing in the horizontal and vertical directions. The bars have a yield strength of 500N/mm^2. The facing panels are hot-dip galvanised with a protection thickness of 140µm. The galvanising provides sacrificial corrosion protection to the facing. The nominal facing panel size is 3.0m x 1.3m with two rows of soil reinforcement connected to each facing, giving a 650mm vertical spacing between layers of soil reinforcement.

3.3 Soil reinforcement

The polymeric soil reinforcement comprises closely packed high tenacity polyethylene terephthalate (PET) fibres encased in a low density polyethylene (LDPE) sheath. A knurled finish is provided to the sheath. The finish helps to provide frictional resistance between the soil and the soil reinforcement, which, in addition to its tensile capacity, enables the structure to resist the applied loads. This type of soil reinforcement has been extensively used for reinforced soil structures since the 1980s; mostly with precast concrete facing panels but more recently also with wire mesh facing panels.

Three different grades of soil reinforcement are used in the construction of the dam. These being 37.5kN, 50kN and 65kN corresponding to the initial strength of the soil reinforcement. Each grade has a width of 50mm. The soil reinforcement is CE marked to demonstrate conformity with the requirements of the harmonised European Standard for soil reinforcement. Representative samples of the soil reinforcement are tested by the manufacturer for tensile strength, elongation, cone puncture resistance, static puncture resistance and durability. The test results allow the manufacturer to provide a Declaration of Performance and a Certificate of Conformity with each batch of materials.

3.4 Connections

Each of the soil reinforcement strips is connected to the facing panel by a galvanised steel hook with a horseshoe shaped steel plate. The hooks and horseshoes are protected against corrosion by hot-dipped galvanisation. The dimensions of the system ensure a non-damaging bending diameter of 60mm for the soil reinforcement as it passes around the connection. It has been demonstrated that diameters smaller than 20mm can have a detrimental effect on the soil reinforcement capacity (Sankey & Lozano 2015).

4 DESIGN

The structure was designed using the principles described in BS8006:2010. This is a limit state procedure considering both ultimate and serviceability limit states. Partial load factors greater than unity are applied to the nominal loads having disturbing effects. Design material properties are calculated by dividing the characteristic properties by the appropriate partial resistance factor.

The ultimate limit state considers the factor of safety against collapse and the serviceability limit state considers the magnitude of deformation that will occur during the service life of the structure. The ultimate limit state assessment considers both the external and internal stability of the structure.

Three load combinations are considered. For Load Combination A, a partial load factor of 1.5 is applied to the permanent and variable loads. This combination usually generates the maxium tension in the soil reinforcement and the maximum foundation bearing pressure.

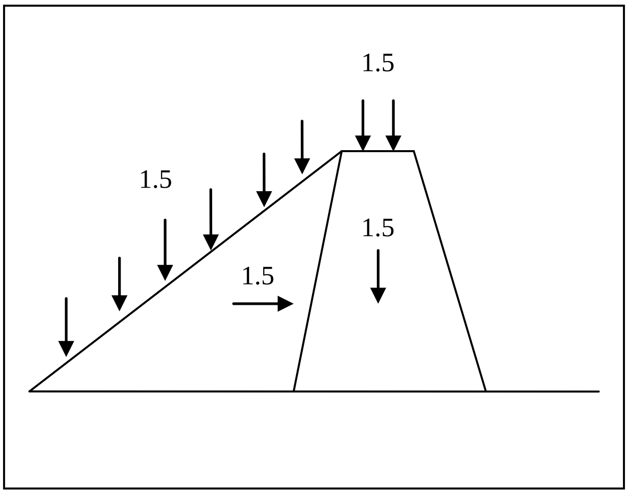

Figure 2 Load Combination A.

For Load Combination B partial load factors of 1.0 are applied to the the mass of the soil within the body of the structure and the variable load behind the structure. No variable load is applied above the reinforced soil mass. This combination usually generates the critical case for overturning and sliding of the structure and normally determines the soil reinforcement requirements for pull-out resistance.

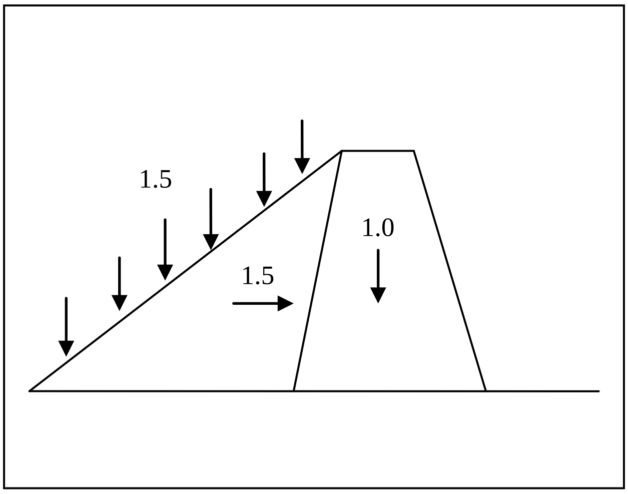

Figure 3 Load Combination B.

For Load Combination C, no variable loading is applied and the load factors for permanent loads are unity. This combination is used to check the serviceability limit state.

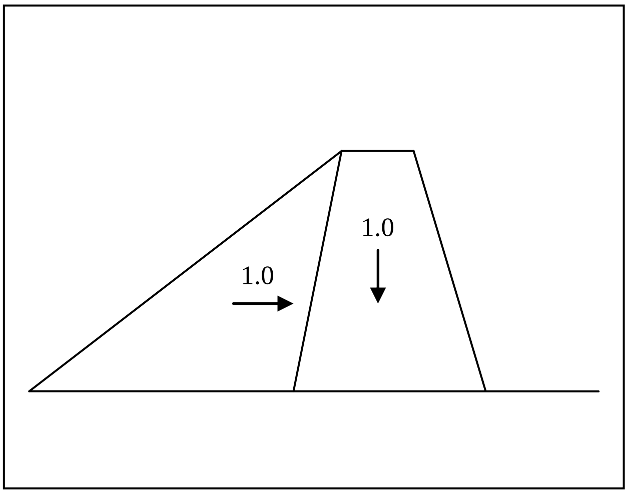

Figure 4 Load Combination C.

4.1 External stability assessment

The external stability assessment considers the possibility of failure by forward sliding, overturning and a bearing capacity or slip circle failure in the supporting soil.

The stability against forward sliding is assessed using the following expression:

$$f_s R_h \leq R_v \frac{\tan \phi'_p}{f_{ms}} \qquad (1)$$

Where,
f_s is the partial resistance factor against base sliding;
R_h is the design horizontal disturbing force;
R_v is the design resultant vertical force;
ϕ'_p is the peak angle of shearing resistance under effective stress conditions; and
f_{ms} is the partial material factor applied to tan ϕ'_p

As the materials used in the structure and the foundation have very little fines content, the analysis considers only drained effects and the term relating to effective cohesion is omitted from equation 1.

An assessment of the potential for a global or circular slip failure was undertaken using the procedure described in BS EN 1997-1:2004. The stability was assessed using the Bishop method of slices. Two load combinations were considered with the following partial factors being applied to the actions and the soil parameters.

Table 1 Partial factors on actions and for soil parameters.

Effects	Comb. 1	Comb. 2
Soil unit weight	1.35	1.0
Variable load	1.5	1.3
Shearing resistance tan ϕ	1.0	1.25
Undrained shear strength Cu	1.0	1.4
Effective cohesion c'	1.0	1.25

The commercially available geotechnical software Talren v5 was used to perform the slip circle analysis. The results for the two load combinations are presented in figures 5 and 6.

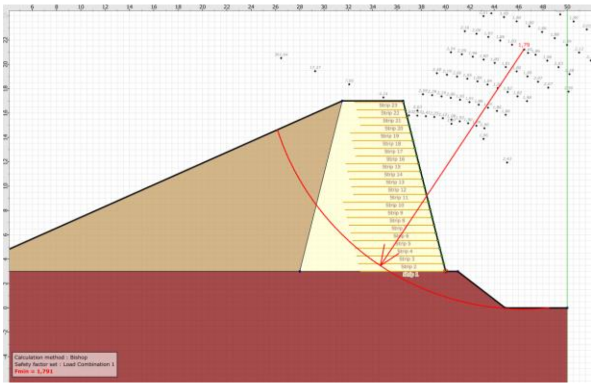

Figure 5 Stability assessment for Load Combination 1.

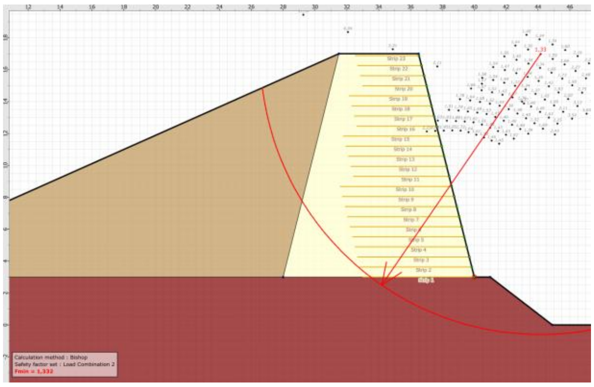

Figure 6 Stability assessment for Load Combination 2.

4.2 Internal stability assessment

The internal stability assessment considers the tensile and adherence capacity of the soil reinforcement, the capacity of the connections and the design of the facing.

4.3 Tensile load in the soil reinforcement

The tension in each layer of reinforcement was determined using the coherent gravity method described in BS8006. The tensile force at each layer is a function of the coefficient of earth pressure (K), the effective vertical stress (σ) and the spacing of the reinforcement (s).

$$T = K\sigma s \tag{2}$$

The coefficient of earth pressure (K) is considered to vary linearly; from K_o at the surface to K_a at a depth of 6m and beyond.

The effective vertical stress takes into account the eccentricity of the vertical force due to the horizontal earth pressure and variable loading behind the structure. The stress is considered to be uniform at each layer and is determined using a Meyerhoff pressure distribution.

4.4 Tensile capacity of the soil reinforcement

The long-term design strength of the soil reinforcement for the ultimate limit state T_D (kN) is calculated using the following expression from BS8006:

$$T_D = \frac{T_{CR}}{f_m} \tag{3}$$

Where T_{CR} is the reduction factor for creep and f_m is the material safety factor calculated as shown below.

$$f_m = RF_{ID} \times RF_W \times RF_{CH} \times f_s \tag{4}$$

Where,

RD_{ID} is the reduction factor for installation damage;
RF_W is the reduction factor for weathering;
RF_{CH} is the reduction factor for chemical / environmental effects; and
f_s is the factor of safety for the extrapolation of data.

The soil reinforcement has been tested for use in a range of different granular fill materials. Installation damage factors have been determined for fill materials with grain sizes of 0 – 5mm, 0 – 32mm, and 0 – 125mm. The fill material used in the construction of the avalanche barrier was a well-graded gravel with sand and cobbles. The material was excavated from a quarry at the Neskaupstaður site and crushed to the required size. The grain size varied from 0.1mm to 200mm. As this was larger than the material used in previous installation damage testing for the soil reinforcement, additional installation damage testing was undertaken to determine the appropriate installation damage factors.

4.5 Adherence capacity of the soil reinforcement

The adherence capacity of the soil reinforcement was determined by the supplier using pull out testing, both extensively in the laboratory and on full-scale structures. The soil reinforcements exhibit an increased friction capacity in granular soil due to the arching effects between adjacent strips from the same layer. This phenomena, though slightly more pronounced, was originally observed in structures using high adherence (i.e. ribbed) steel soil reinforcement.

The tensile capacity of the soil reinforcement is calculated using the following expression.

$$T = \frac{2B\mu}{f_p f_n} \int_{L-Laj}^{L} f_{fs} \sigma_v(x) dx \qquad (5)$$

Where,

- B is the reinforcement width;
- μ is the coefficient of friction between the soil and the soil reinforcement at the appropriate vertical stress level;
- L is the total soil reinforcement length;
- L_{aj} is the length of soil reinforcement beyond the maximum tension line at the appropriate level;
- $\sigma_v(x)$ is the vertical stress along length x of the reinforcement;
- f_n is the partial factor for the economic ramifications of failure;
- f_p is the partial factor for reinforcement pull-out resistance;
- f_{fs} is the partial load factor and
- 2 is for two sides of the soil reinforcement.

4.6 Design of the connections

The capacity of the connections was assessed using full-scale destructive testing. The tests were repeated several times and a statistical approach was used to determine the characteristic strength to be considered, following the requirements of Annex D of Eurocode 0 (EN 1990).

5 INSTALLATION DAMAGE TESTING

The procedure for testing the samples is explained in Appendix D of BS8006:2010. Seven samples of each grade of soil reinforcement (twenty one in total) were placed on a 650mm deep base of fill material that had been previously levelled and compacted. The test area was divided into three zones. In Zone 1, 650mm of crushed rock was placed above seven samples and the material was then compacted using a self-propelled vibratory roller having a mass per metre of 4,690kg. Twenty one samples were placed in Zone 2. This zone was again backfilled with 650mm of crushed rock but was subjected to twice the number of passes with the roller as Zone 1. The same number of samples was placed in Zone 3. This zone had two 650mm layers of crushed rock placed above the samples with compaction taking place after each 650mm layer had been placed. Seven samples of each grade of soil reinforcement were retained as control samples.

Figure 7 Installation damage trial.

After the backfilling and compaction had been completed, the backfill was carefully removed and the samples were visually inspected for damage.

A record of cutting, splitting, bruising and general abrasion was made. On retrieval the samples exhibited very little damage.

Figure 8 Minor damage to reinforcement.

To correctly assess the effects of the installation damage, all samples were subjected to tensile testing to failure. The results of the testing are presented in Tables 2 to 5.

Table 2 Control samples.

Sample	T_D(kN)	T_D(kN)	T_D(kN)
1	41.3	53.8	68.2
2	41.6	53.8	68.8
3	41.6	53.9	69.1
4	41.5	54.0	68.8
5	42.0	53.4	69.2
6	42.1	53.8	69.4
7	40.5	54.4	69.2
Mean	41.5	53.9	69.0

Table 3 Samples subjected to single compaction.

Sample	T_D(kN)	T_D(kN)	T_D(kN)
1	41.1	51.9	64.3
2	41.0	54.3	68.2
3	40.5	50.8	64.0
4	40.0	51.7	64.4
5	41.2	52.6	64.1
6	40.6	46.7	66.1
7	39.4	51.5	63.8
Mean	40.5	51.4	65.0

Table 4 Samples subjected to double compaction.

Sample	T_D(kN)	T_D(kN)	T_D(kN)
1	38.9	48.3	60.7
2	38.2	50.3	65.4
3	38.6	45.1	62.6
4	41.0	51.4	64.6
5	39.3	51.4	64.1
6	38.9	48.9	64.3
7	41.3	51.1	63.6
Mean	39.5	49.5	63.6

Table 5 Samples subjected to double compaction and double backfilling.

Sample	T_D(kN)	T_D(kN)	T_D(kN)
1	40.8	42.8	63.1
2	41.2	52.8	63.5
3	37.3	52.2	63.3
4	39.3	51.3	63.4
5	40.9	52.0	64.4
6	39.2	45.4	63.5
7	40.3	44.6	56.1
Mean	39.9	48.7	62.5

The installation damage factor for each grade of reinforcement was calculated by dividing the lowest mean tensile strength for samples in each test zone by the mean control strength. The limiting case for the 35.5kN grade was from Zone 2 and for grades 50kN and 65kN from Zone 3. In spite of the very coarse nature of the backfill, the reduction factor for installation damage determined through the site specific testing ranged from 1.05 to 1.11 depending on the grade of soil reinforcement.

6 CONSTRUCTION

Construction of the main dam commenced in July 2012. The location of the project site near to a deep water port made delivery of the manufactured materials by sea freight possible.

Following the typical construction procedure, the first row of facing panels were installed directly onto the prepared sub-fill layer. Temporary timber bracing was used to set the first row of panels at the correct inclination. A half-height facing panel was placed at the base of alternative columns of panels to provide a staggered horizontal joint. This technique allowed subsequent rows of facing panels to be connected to the exposed part of the facing panel in the row below. This helped to provide fall protection to the operatives constructing the structure.

Figure 9 Installation of the first row of facing panels.

The first layer of core fill material was placed using a 360° hydraulic excavator and compacted using a self-propelled vibratory roller, as used in the installation damage trials. The first layer of soil reinforcement was then connected to the facing panels and laid on to the compacted fill material. A small mound was constructed approximately half way along the soil reinforcement length to enable a small amount of tension to be applied to the soil reinforcement. This was particularly important to ensure good alignment of the facing panels in the completed structure.

Figure 10 Laying out the soil reinforcement.

The construction sequence was repeated until the maximum structure height was reached.

Figure 11 The completed structure.

7 GENERAL DESIGN CONSIDERATIONS

Avalanche protection dams using reinforced soil techniques exhibit a number of advantages:

7.1 Limited environmental impact

The structures are predominantly made of locally available fill materials. The manufactured materials delivered from remote locations represent a very small proportion of the total weight of the structure. The construction technique offers a number of environmetal benefits. As there are no concrete components in the system and only a small amount of steel components, the CO_2 emissions are small and mostly due to the extraction and placing of the earthworks.

7.2 Limited environmental impact

The technique allows a variety of possible layouts, shapes and face inclinations. This gives designers of avalanche protections structures a high degree of freedom when choosing the layout of their schemes. Smaller breaking mounds and deflecting dams can be constructed using the same technique.

In the event of an avalanche impact, it is possible that some damage may occur as rocks and debris come into contact with the facing. Procedures have been developed to allow localised repair of the facing without the need to dismantle the structure. New facings can be placed in front of the damaged ones and a connection made between the soil reinforcement and the new facing panel.

7.3 A highly resilient structure

Most importantly, reinforced soil structures offer, at low relative cost, an ideal material for sustaining dynamic loads. This is well known for example in Japan, where reinforced soil structures have been widely used based on the outstanding seismic performance, even under the most severe earthquake motions (Otani, 2013). But it is also the case for dynamic impacts, distributed like avalanche or explosion blasts, or localised in the case of rock falls. The last point is also important for avalanche loads since the snow flow is likely to bring other debris which will hit the dam or the braking mounds, like rocks or trees. Recent research on resistance to localised impacts has been initiated in France, in partnership between the Terre Armée group and IFSTTAR (Joffrin, 2016). The test structure was 3.5m high and 3.0m thick. The results are promising and show that the structure could sustain very large impact forces. See Figure 12.

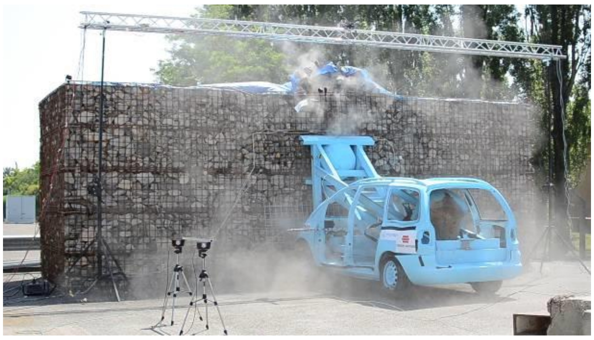

Figure 12 Impact on a reinforced fill protective bund with vertical facing, with an energy of 800 kJ (mass: 2700 kg, impacting speed: 88 km/h).

The localised impacting energy is absorbed by moderate localised internal deformation of the granular fill. Localised damage is likely to be observed in those cases, but procedures have been developed which enable the integrity of the wire mesh facing (and the visual aspect of the structure) to be restored without the need for dismantling part of it. The repair basis consists of simply placing new wire panels in front on the damaged ones and restoring connections between the reinforcing strips and these new panels. The damaged panels are left in place and the volume between both panels is filled with stones.

8 CONCLUSIONS

Reinforced soil structures with steep or vertical faces provide a number of advantages for the construction of avalanche protective barriers of all kinds: braking mounds, deflecting dams and catching dams. The structure at Neskaupstaður, presented in this paper, proved to be a very good example of the versatility of the solution and of its integration within the local environment. A number of other projects are currently under development in Iceland. Involving a reinforced soil specialist designer at the preliminary stage of a project can be beneficial for the optimization of such schemes.

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