OPERE DI PROTEZIONE CONTRO LA CADUTA MASSI: ASPETTI PROGETTUALI

Rilevati paramassi

Daniele PEILA

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ROCK FALL PROTECTION USING REINFORCED EMBANKMENTS

Lecture index
- general concepts;
- analysis of relevant examples;
- tests on embankments;
- numerical back analysis of full scale tests;
- parametric numerical modelling;
- proposal of a design method;
- discussion on a new type of rockfall embankment.
GENERAL CONCEPTS

Rockfall protection embankments have been used to stop high kinetic energy rockfall or debris both in civil and mining applications to protect the roads, inhabited areas, quarries areas, etc.

Scheme proposed by Descoedres, 1997
Embankments allow
- control of high or very high energy rock falls;
- control of repeated impacts both at high energy and at low energy;
- ability to control repeated impacts;
- reduced or nil maintenance after impacts;
- high durability in time also in areas where corrosion can be high;
- reduced environmental impact with reference to other devices;
- easy and rapid installation if the slope geometry is adequate.

Examples
Different types of rockfall protection embankments (1/2)

• ground embankment, made of selected and compacted natural ground, with trapezium shape and faces inclination (on both size) at about 35-40°;

• embankments built with huge rock blocks; the faces inclination is usually equal to 65÷70°;

• unreinforced ground embankment with a face made of wire mesh gabions; the gabion side inclination is about 90°, the other side inclination is of about 35°;

• embankment totally made of gabions;

Different types of rockfall protection embankments (2/2)

• embankment reinforced with wood elements;

• reinforced ground embankment, made of compacted soil with reinforcement and adsorbing mattress on the mountain side face;

• reinforced ground embankment, made of compacted soil with reinforcements (i.e. geotextiles, geogrids, metallic wire nets), with trapezium shape in the cross section and faces inclination of about 70°.
Embankments built with huge rock blocks

Detachment point
1100 m asl

Serpentinite rock mass

Embankment position
500m asl

Problem: the embankment worked as a dam to superficial water since it was not designed any by water by-pass below it

Grand Frayan, Aosta Valley
Embankments built with huge rock blocks

Point 1: embankment completely damaged

Point 2: Embankment that deviated the flying block at a height of 6m from the ground surface

Point 3: block stopped 12 m

Ground embankment with a face made of wire mesh gabions

Timau (Paluzza, Ud)
**Embarkment made of gabbions**

(courtesy Officine Maccaferri SpA)

**Ground embankment with faces made of wire mesh gabions**

British Columbia. (Simons, Pollak e Peirone, 2009)
Max impact energy 10000kJ
Block size ranging between 0.6m to 3m
British Columbia. (Simons, Pollak e Peirone, 2009)

Embankment made of gabbions
France examples
Ground embankment with a face made of reinforcing elements

France examples

Reinforced ground embankment

Piedmont examples
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**EXAMPLES**

**Reinforced ground embankment**

- Chienes, Val Pusteria (BZ)
- Rhêmes Saint-Georges (Aosta Valley)

**EXAMPLES**

**Reinforced ground embankment**

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**Problem of foundations and slope stability**

- Carema (TO)
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EXAMPLES

Reinforced ground embankment

Cogne Aosta Valley

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EXAMPLES

Reinforced ground embankment

Cogne Aosta Valley

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Reinforced ground embankment used to control of both local rock falls and a potential collapse of 200,000 m³

Assisi (Italy)
Reinforced ground embankment used to control of both local rock falls and a potential collapse of 200,000 m$^3$

EXAMPLES

Assisi (Italy)

Ground embankment reinforced with wood

Dorènaz (Vallis - Switzerland)
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Full scale tests

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Burroughs, Henson e Jiang (1993).

The used soil was sand, silt and gravel

3m

1.8 m
Burroughs, Henson e Jiang (1993): main results

18 tests up to 1500kJ
The blocks were rolled on a slope against the embankment
No structural collapses were registered

<table>
<thead>
<tr>
<th>block mass [kg]</th>
<th>block speed [m/s]</th>
<th>contact time [s]</th>
<th>upstream displacements [m]</th>
<th>downstream displacements [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>196</td>
<td>9.0</td>
<td>0.20</td>
<td>0</td>
<td>0.008</td>
</tr>
<tr>
<td>672</td>
<td>13.0</td>
<td>0.23</td>
<td>0.155</td>
<td>0.007</td>
</tr>
<tr>
<td>664</td>
<td>9.0</td>
<td>0.23</td>
<td>0.076</td>
<td>0.015</td>
</tr>
<tr>
<td>3388</td>
<td>13.5</td>
<td>0.50</td>
<td>0.305</td>
<td>0.199</td>
</tr>
<tr>
<td>2620</td>
<td>16.8</td>
<td>0.34</td>
<td>0.305</td>
<td>0.102</td>
</tr>
<tr>
<td>5214</td>
<td>19.2</td>
<td>0.48</td>
<td>0.609</td>
<td>0.207</td>
</tr>
<tr>
<td>8167</td>
<td>18.0</td>
<td>0.80</td>
<td>0.914</td>
<td>0.728</td>
</tr>
</tbody>
</table>

Test of Yoshida e Nomura (1998)
9 tests - Impact energy ranging between 58÷2700 kJ
CARRIED OUT TESTS

Different types of reinforced strata;
Blocks with a weight of 30 kN dropped from 4m.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Weight (kN)</th>
<th>Speed (m/sec)</th>
<th>Energy (kJ)</th>
<th>Height of Collision (m)</th>
<th>Max. Deformation I.S. (mm)</th>
<th>B.S. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>18.3</td>
<td>14.3</td>
<td>189.5</td>
<td>0.2</td>
<td>26</td>
<td>Ng</td>
</tr>
<tr>
<td>2</td>
<td>11.3</td>
<td>17.7</td>
<td>180.6</td>
<td>3.0</td>
<td>205</td>
<td>Ng</td>
</tr>
<tr>
<td>3</td>
<td>33.0</td>
<td>24.0</td>
<td>969.8</td>
<td>2.0</td>
<td>227</td>
<td>Ng</td>
</tr>
<tr>
<td>4</td>
<td>28.0</td>
<td>10.6</td>
<td>141.9</td>
<td>0.5</td>
<td>59</td>
<td>Ng</td>
</tr>
<tr>
<td>5</td>
<td>20.0</td>
<td>24.6</td>
<td>58.8</td>
<td>0.5~1.0</td>
<td>Ng</td>
<td>Ng</td>
</tr>
<tr>
<td>6</td>
<td>23.0</td>
<td>17.7</td>
<td>367.6</td>
<td>0~3.0</td>
<td>Ng</td>
<td>Ng</td>
</tr>
<tr>
<td>7</td>
<td>60.0</td>
<td>20.8</td>
<td>1554.4</td>
<td>0~4.0</td>
<td>181</td>
<td>Ng</td>
</tr>
<tr>
<td>8</td>
<td>77.0</td>
<td>24.0</td>
<td>2561.9</td>
<td>2.0~3.0</td>
<td>N.M.</td>
<td>93</td>
</tr>
<tr>
<td>9</td>
<td>170.0</td>
<td>17.7</td>
<td>2717.3</td>
<td>3.0~4.0</td>
<td>N.M.</td>
<td>500</td>
</tr>
</tbody>
</table>

Note: I.S.=Impact surface; B.S.=back surface;
Ng=Negligible; N.M.=Can not be measured
2001-2002 Fukui Japan

Test Method: block weight: 30kN, falling height: 4m
Energy: 120 kJ

Lambert, Gotteland and Nicot (2009)
Cellular structure
Lambert, Gotteland and Nicot (2009)
Cellular structure
Politecnico di Torino tests: 2001-2003
The tests were carried out in Meano near Trento (North-East of Italy) where a hauling device that is able to launch reinforced concrete blocks of up to 10000 kN with a speed of 32m/s against passive protection structures such as net fences and embankments was constructed.


Oggeri C., Peila D., Recalcati P. (2004), Rilevati paramassi, Convegno Bonifica di versanti rocciosi per la protezione del territorio, Trento, Peila Ed., GEAM

Carried out test on reinforced embankments

<table>
<thead>
<tr>
<th>Test number</th>
<th>block speed [m/s]</th>
<th>block mass [kg]</th>
<th>block energy [MJ]</th>
<th>number of impacts</th>
<th>face surface steel mesh</th>
<th>Geogrid types</th>
<th>soil type</th>
<th>geometry type (Fig. 4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>31</td>
<td>5000</td>
<td>2402</td>
<td>1</td>
<td>Yes</td>
<td>tensile strength 53MPa</td>
<td>1</td>
<td>a)</td>
</tr>
<tr>
<td>2</td>
<td>31</td>
<td>3700</td>
<td>4180</td>
<td>3</td>
<td>Yes</td>
<td>tensile strength 45MPa</td>
<td>1</td>
<td>a)</td>
</tr>
<tr>
<td>3</td>
<td>31</td>
<td>6700</td>
<td>4180</td>
<td>Yes</td>
<td>No</td>
<td>Absent tensile strength 45MPa</td>
<td>1</td>
<td>a)</td>
</tr>
<tr>
<td>4</td>
<td>31</td>
<td>6700</td>
<td>4180</td>
<td>Yes</td>
<td>Absent tensile strength 45MPa</td>
<td>2</td>
<td>b)</td>
<td></td>
</tr>
</tbody>
</table>

Soil 1 \(c' [kPa]=9\) \(\phi' [^\circ]=34\) \(\gamma' [kN/m^3]=21\) sand and gravel

Soil 2 \(c' [kPa]=50\) \(\phi' [^\circ]=30\) \(\gamma' [kN/m^3]=17\) silt and clay
CARRIED OUT TESTS

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Politecnico di Torino tests

Test 2 – first impact

Embarkment geometry a)

- test n. 1, 2, 3

Embarkment geometry b)

- test n. 4

CARRIED OUT TESTS
CARRIED OUT TESTS

Politecnico di Torino tests

Test 2 – second impact

CARRIED OUT TESTS

Politecnico di Torino tests

Test 2 – third impact
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CARRIED OUT TESTS

Politecnico di Torino tests

Test 1

Politecnico di Torino tests

Test 3
CARRIED OUT TESTS

Politecnico di Torino tests

Test 4

ROCK FALL PROTECTION USING REINFORCED EMBANKMENTS

Numerical back analysis of full scale tests

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Politecnico di Torino
The soil was a homogeneous and mono-phase material and the presence of water and the consequent consolidation and interstitial stresses were neglected.


Oggeri C., Peila D., Recalcati P. (2004), Rilevati paramassi, Convegno Bonifica di versanti rocciosi per la protezione del territorio, Trento, Peila Ed., GEAM

The rock mass is stopped by the embankment in about 0.1s and that deceleration has a triangular type trend with a maximum equal to about 750m/s^2 in both the x and y directions.

Acceleration of the barycentre of the block vs. time during the impact in the back-analysis of test n. 1.
Displacement of the barycentre of the block vs. time during the impact in the back-analysis of test n. 1.

back analysis test 1

An analysis of the displacements showed that:
- only the layers directly impacted by the block translate parallel to the direction of the geogrids due to the anisotropic behaviour of the ground reinforced embankment;
- the penetration of the block on the upstream side (measured in a horizontal direction) was of the order of 0.38m while the total displacement (which had a trend almost orthogonal to the face) was 0.71m;
- the maximum extrusion, in correspondence to the downstream side, was about 0.20m and was located in the 3rd, 4th and 5th layers from the base of the embankment.

Very good agreement with the real scale test
Displacements in the central cross section after 0.1s – 0.2s – 0.3s – 0.4s

Energy distribution block vs. time in the back-analysis of test n. 1.
The impact phenomenon inside the embankment can be subdivided into different stages: a first stage of local soil compression, with a partial upward displacement of the soil (creation of the crater stage) and dissipates most of the kinetic energy of the block. This stage is followed by a translation stage in which the soil layers dissipate the remaining kinetic energy in friction by sliding on the geogrid layers.

ROCK FALL PROTECTION USING REINFORCED EMBANKMENTS

Numerical back analysis of a real event

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The embankment in Cogne (AO, Italy) was built with Green Terramesh Maccaferri elements with reinforcing geogrids with a strength of 100 kN/m each 3 layers of Terramesh (interax of 2.19 m).

Considered impact: block of 6 m$^3$

Modelled block sizes: 2*2*1.5 m
Block mass of 15000 kg
**Geotechnical parameters used in the numerical modelling**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil density [kg/m³]</td>
<td>1900</td>
</tr>
<tr>
<td>Elasticity modulus [kPa]</td>
<td>75000, 90000, 110000</td>
</tr>
<tr>
<td>Poisson modulus [-]</td>
<td>0.30</td>
</tr>
<tr>
<td>Drained friction angle [°]</td>
<td>35</td>
</tr>
</tbody>
</table>

**Impact speed:**

- Vh = 20 m/s
- Vv = 0 m/s

**Friction coefficient between the layers:** 0.40

**Impact with a corner of the block**

**Used code:** Abacus Explicit 3D
The mountain side penetration depends on the soil elasticity modulus:

<table>
<thead>
<tr>
<th>Elasticity modulus [kPa]</th>
<th>Penetration mountain side [m]</th>
<th>Sliding valley side [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>75000</td>
<td>0.74</td>
<td>0.17</td>
</tr>
<tr>
<td>90000</td>
<td>0.71</td>
<td>0.07</td>
</tr>
<tr>
<td>110000</td>
<td>0.62</td>
<td>0.17</td>
</tr>
</tbody>
</table>

With higher elasticity modulus of the soil that is to say with a better compactation, the penetration is reduced and there is a higher percentage of elastic rebound of the block.

Distribution of displacements with soil with elastic modulus of 75000 kPa for t: 0.00, 0.05, 0.10 e 0.15s (displacement scale ranging between 0 and 0.8 m).

No physically measurable displacement on the valley side.
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Parametrical numerical analysis

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Code: Abaqus/Explicit 3D FEM

90 different simulation

Impact on different embankments reinforced with Maccaferri mesh.
4 different standard geometries were considered.
The impact cubic element had different sizes 1.5m and 2m side.
Different impact speed were considered to get different impact energy.
Maccaferri: MAC.RO System
Impact height for all the models

<table>
<thead>
<tr>
<th>Soil properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Densità $\rho$ [kg/m$^3$]</td>
<td>2100</td>
</tr>
<tr>
<td>Modulo di Young $E$ [kPa]</td>
<td>110000</td>
</tr>
<tr>
<td>Coefficiente di Poisson $\nu$ [-]</td>
<td>0.25</td>
</tr>
<tr>
<td>Angolo d'attrito Mohr-Coulomb $\Phi$ [°]</td>
<td>34</td>
</tr>
<tr>
<td>Angolo d'attrito Drucker-Prager $\beta$ [°]</td>
<td>54</td>
</tr>
<tr>
<td>Flow stress ratio $k$ [-]</td>
<td>0.78</td>
</tr>
<tr>
<td>Dilatanza $\psi$ [°]</td>
<td>0</td>
</tr>
<tr>
<td>Yield stress [kPa]</td>
<td>540</td>
</tr>
</tbody>
</table>

Druker-Prager plasticity envelope

Block speed: horizontal (only)

Cubic block size: 1m and 2m
Block mass: 8700kg – 20000kg

The impacting block has been modelled as a perfectly rigid body.
Due to numerical calculation problems the steel mesh, usually used in the embankment faces, was not modelled, also because the full scale tests carried out in the past have shown that this element is not significant as far as concerns the dynamic effects.

The contact between the soil layers which are obtained with the reinforcement elements has been modelled using a “master-slave weighted penalty method” (which checks for possible mesh collision between the given surfaces or nodes during every time step and calculates a surface reaction force applied in the next time step) assuming a friction angle between the various layers determined on the basis by shear tests on reinforcement elements back analyzed to verify the feasibility of the adopted numerical model.

The choice of this parameter is a key point for a CORRECT MODEL.

The simulations have been carried out with the same type of block by rising up the impact speed till collapse of the structure occurs.

Definition of ULS collapse

Example of the collapse condition ULS: the structure is not any more statically stable and the upper part of the porting that was moved by the block can collapse.
Example of the contours plot of the displacement $t = 0.03-0.06-0.09-0.12-0.15-0.17\text{s}$ (displacement scale ranging between 0 and 0.7 m).

MA.CRO – Class 3 block of 12000kg (1.25m size) and speed impact direction normal to the impacted face Impact enegry of 5400kJ

Reinforced enbankment
Impact energy
5400kJ
Unreinforced embankment

Impact Energy

2400kJ

Example of induced displacements on MACRO 3 - Block size 2 m

Mountain side displacement

Hole depth

Valley side displacement

Sliding
The angular coefficient of the interpolation line of the penetration data \( k \) depends on the reinforcement type (strength and friction between the layers), type of soil and its compaction and type of embankment.

\[
\text{Penetration} \times \text{block area} = k \times E
\]
Example of an impact at low energy

Contour displacement (scale from 0 to 0.25 m).

- Height of the embankment = 4.27 m
- Walls inclination = 70°
- Top thickness = 1.39 m
- Kinetic energy = 150 kJ
- Diameter = 1.3 m; mass = 2875 kg

ROCK FALL PROTECTION USING REINFORCED EMBANKMENTS

Design approach

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The development of three-dimensional numerical models in the dynamic field is usually difficult to be calibrated:
- it requires a specialized engineer
- large computational time

\textit{it cannot be considered as a usual design tool}

It is therefore useful for design engineers and geologists to have a feasible simplified design scheme that can permit a simple design evaluation of the embankment impacted at the design energy.

\textbf{Design of a rockfall reinforced ground embankment}

\textbf{Static computation (usual design)}

a. equilibrium of the embankment and the slope (bearing capacity of the foundation, sliding and tilting)
b. the internal stability of the embankment (tensile and pull-out strength of the reinforcing nets)

\textbf{Dynamic computation}

a. the launch of fragments over the embankment during the impact
b. the overcoming of the embankment because of the block rolling on the up face
c. the collapse of the embankment due to block penetration and sliding of the soil layers
dynamic computation

a. The condition that the embankment should not launch rock fragments during the impact toward valley is normally easily respected in reinforced soil embankment since usually the structure is made of small elements, if compared with the size of the collapsing block.

the plastic deformation of the soil normally prevents the launching of fragments.

b. The risk of the overcoming of the embankment depends on the rolling speed of the falling block. It was verified by real tests that the rotational kinetic energy may be usually only of about the 10÷15% of the total kinetic energy. Since soil reinforced embankments have a mountain side face inclined of 67°-80° with reference to the horizontal, the block has usually not enough rotational energy to overcome the embankment after it has been impacted and the crater has been created on the mountain face.
The stability of the structure during the impact should verify that the sliding of the soil layers interested by the impact and plasticization of soil on the mountain side face with the formation of a crater don’t trigger the collapse of the structure for the computation at the Ultimate Limit State and permit an easy rehabilitation and repair at the Service Limit State.

Dynamic computation

The DESIGN SCHEME therefore requires that:

a) the energy \( E_{\text{embankment}} \) that can be dissipated in safe condition by the embankment is greater (with an adequate safety factor) than the design energy \( E_{\text{design}} \), that is linked with the size and speed of the falling block and that is computed with the classical physics formulations and on the basis of the trajectory evaluation:

\[
E_{\text{design}} - \frac{E_{\text{embankment}}}{\gamma_{ER}} \leq 0
\]

b) the interception height \( h_i \), that is the embankment height minus the upper soil layer is greater than the height of the 95% computed trajectories \( h_{\text{design}} \) of the falling block with an adequate safety factor:

\[
h_{\text{design}} - \frac{h_i}{\gamma_h} \leq 0
\]
The maximum impact energy the embankment ($E_{embankment}$) can fulfil is computed by verifying the global static stability of the structure after the impact, in the deformed shape and taking into account both the maximum sliding toward valley of the layers interested by the impact ($\xi$) and the penetration ($\delta_p$) due to soil compaction.

**The A and B bodies must remain statically stable**

The evaluation of this two parameters can be done by both with numerical or with analytical computations.

$$\delta = \delta_p + \xi$$

Deformed shape after the impact used for the analytical computation
dynamic computation

Evaluation of the deformed parameters

The $\delta_p$ value (plastic penetration in the mountain side) can be obtained:

a. from the peak force acting during the stopping phase ($F_{\text{max}}$)
   Balancing the reduced percentage (Pr %) of the kinetic energy of the block that was numerically shown that is the one that creates the crater and the plastic deformation work done by the stopping force
   \[
   \delta_p = \frac{(\text{Pr} \%) \cdot mv^2}{F_{\text{max}}}
   \]

b. by interpreting the numerical modelling results.
dynamic computation

Evaluation of the deformed parameters: option a)

It is possible to use the formulation proposed by Montani et al. (1996):

$$ F_{\text{max}} = 1.765 M_E^{2/5} R^{1/5} \left((0.80 / 0.85)E_{\text{kin}}\right)^{3/5} \text{[kN]} $$

where: $M_E$: the soil elasticity coefficient (generally computed from the first load curve of a plate loading test) [kN/m²], $R$: the impacting block radius [m], $E_{\text{kin}}$: the block kinetic energy [kJ].

This approach has some limitation due to the fact that was derived from studies on rockfall shelter cover thickness therefore with a limited thickness and under different geometrical conditions of the impact.

The impact penetration can be evaluated using the abacus prepared by Calvetti e Di Prisco (2007) for the filling material layers above rock sheds.

This approach has some limitation due to the fact that was derived from studies on rockfall shelter cover thickness therefore with a limited thickness and under different geometrical conditions of the impact.
dynamic computation

Evaluation of the deformed parameters: option b)

Interpreting the parametrical numerical modelling results

This is the best approach since it takes into account the real geometry of the embankment, the compaction of the soil, the type of the soil, the presence of the reinforcement layers and its friction property.

Example of interpretation of numerical modeling models
dynamic computation

Evaluation of the deformed parameters

The \( \xi \) value can be computed by balancing the 100-\((Pr)-(Ec)\) (%) (where Ec is the elastic percentage) of the kinetic energy of the block and the work done by the friction forces on the interface between the sliding layers, taking into account the real sliding surface geometry (value that can be evaluated only from the results of numerical models), the number of sliding layers and the friction between the layers that depends on the type of embankment and of the reinforcement type.

Evaluation of the deformed shape

When the deformed shape is determined it is possible to evaluate both the ULS and the SLS conditions

ULS condition

is determined by evaluating the static stability of the structure after the deformation by simple equilibrium evaluation
**dynamic computation**

**SLS condition**

conditions that permit an easy maintenance.

The suggested values are:

mountainside displacement lower than 20% of the thickness of the embankment at the impact height and not larger than 0.5-0.7m since for larger displacement it is difficult to repair the structure.

Valley-side displacement should not be larger than 0.3-0.4m since the movement are induced by sliding.