OPERE DI PROTEZIONE CONTRO LA CADUTA MASSI: ASPETTI PROGETTUALI

Rilevati paramassi

Daniele PEILA

ROCK FALL PROTECTION USING REINFORCED EMBANKMENTS

Lecture index

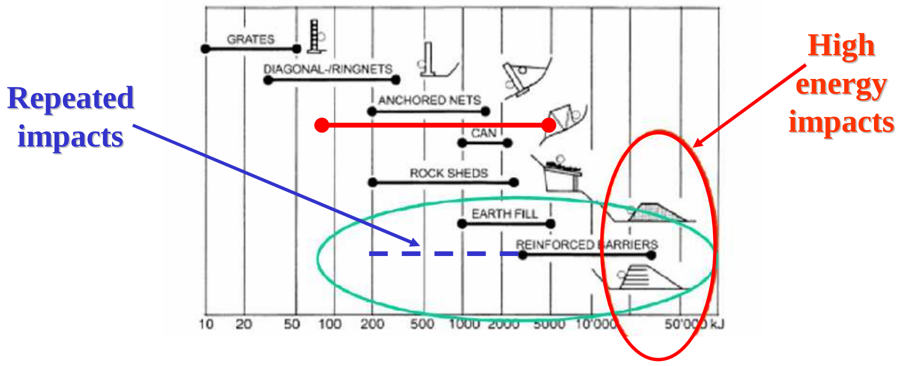
- general concepts;
- analysis of relevant examples;
- tests on enbankments;
- numerical back analysis of full scale tests;
- parametric numerical modelling;
- proposal of a design method;
- discussion on a new type of rockfall embankment.

ROCK FALL PROTECTION USING REINFORCED EMBANKMENTS

General concepts

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 Descoeudres, 1997

GENERAL CONCEPTS

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.

ROCK FALL PROTECTION USING REINFORCED EMBANKMENTS

Examples

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;

EXAMPLES

Different types of rockfall protection embankments (2/2)

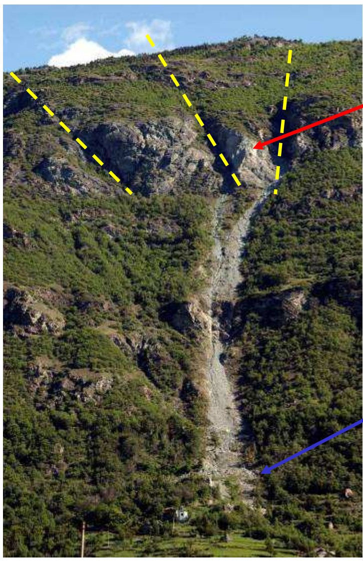
• embankment reinforced with wood elements;

• 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°.

EXAMPLES

Embankments built with huge rock blocks



Detachment point

1100 m asl

Serpentinite rock mass

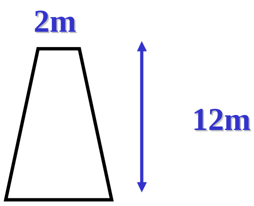
Embankment position

500m asl

Grand Frayan, Aosta Valley

EXAMPLES

Embankments built with huge rock blocks

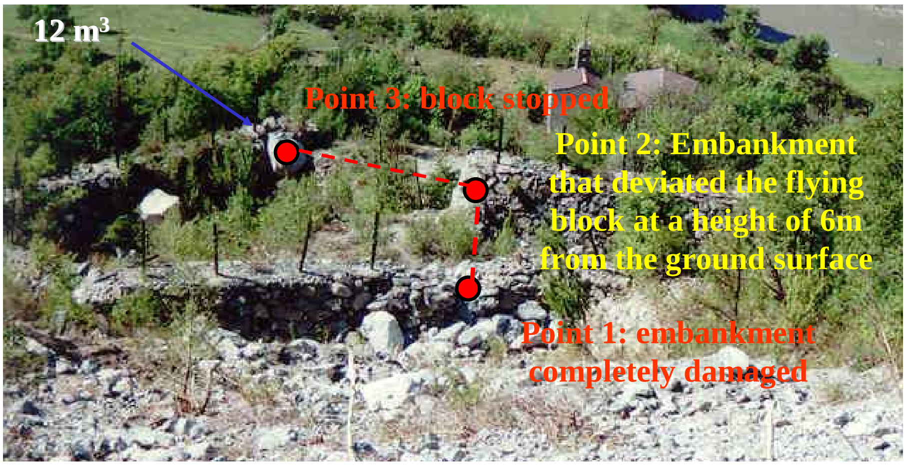


Problem: the embankment worked as a dam to superficial water since it was not designed any by water by-pass below it

Grand Frayan, Aosta Valley

EXAMPLES

Embankments built with huge rock blocks



Grand Frayan, Aosta Valley

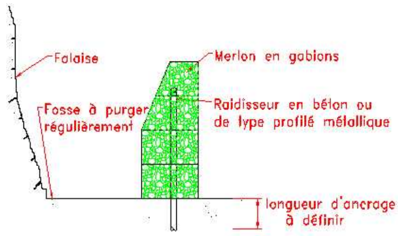
EXAMPLES

Ground embankment with a face made of wire mesh gabions

Timau (Paluzza, UD)

EXAMPLES

Embankment made of gabbions



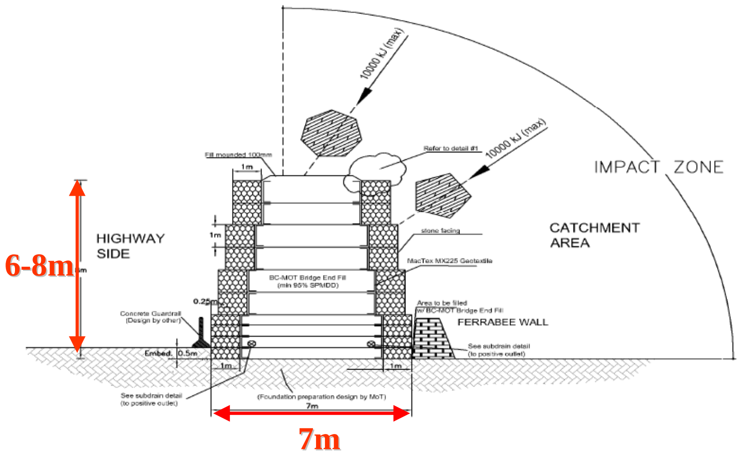
(courtesy Officine Maccaferri SpA)

EXAMPLES

Ground embankment with faces made of wire mesh gabions

British Columbia. (Simons, Pollak e Peirone, 2009)

EXAMPLES



Max impact energy 10000kJ

Block size ranging between 0.6m to 3m

British Columbia. (Simons, Pollak e Peirone, 2009)

EXAMPLES

Embankment made of gabbions

France examples

EXAMPLES

Ground embankment with a face made of reinforcing elements


Gorges de l'Arly (73)

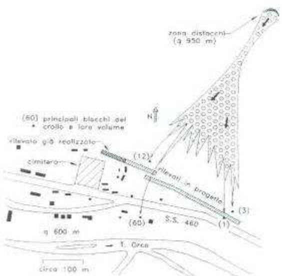

Lumbin (38)


St Etienne de Cuynes (73)

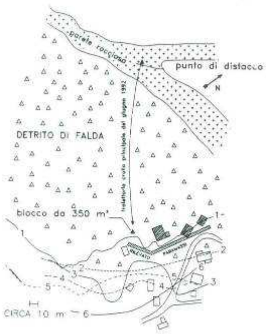
France examples

EXAMPLES

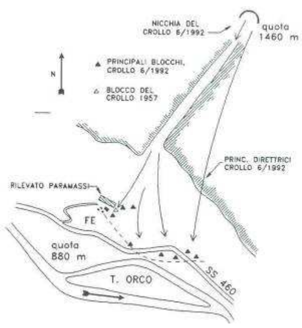
Reinforced ground embankment



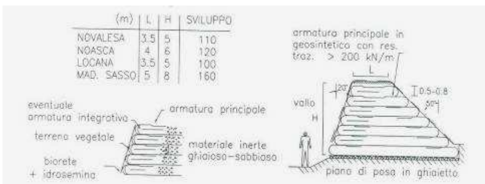
Locana



Novalesa



Noasca



Piedmont examples

EXAMPLES

Reinforced ground embankment

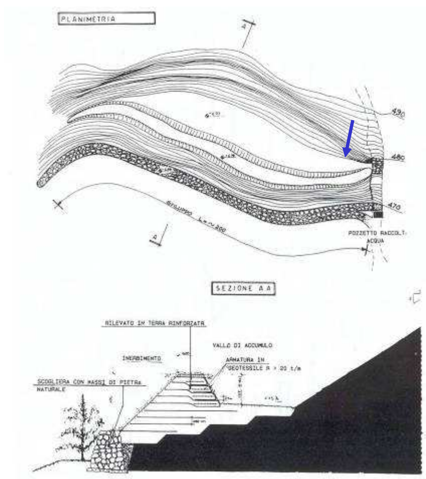
Chienes, Val Pusteria (BZ)

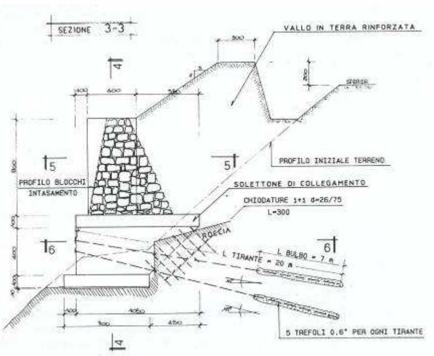
Rhêmes Saint-Georges (Aosta Valley)

OPERE DI PROTEZIONE CONTRO LA CADUTA MASSI: ASPETTI PROGETTUALI

EXAMPLES

Reinforced ground embankment





Problem of foundations and slope stability

Carema (TO)

EXAMPLES

Reinforced ground embankment

Cogne Aosta Valley

EXAMPLES

Reinforced ground embankment

Cogne Aosta Valley

EXAMPLES

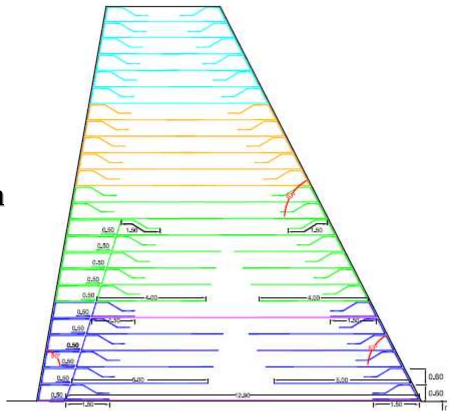
Reinforced ground embankment used to control of both local rock falls and a potential collapse of 200.000 m^3

Assisi (Italy)

EXAMPLES

Reinforced ground embankment used to control of both local rock falls and a potential collapse of 200.000 m^3

14.5m



Assisi (Italy)

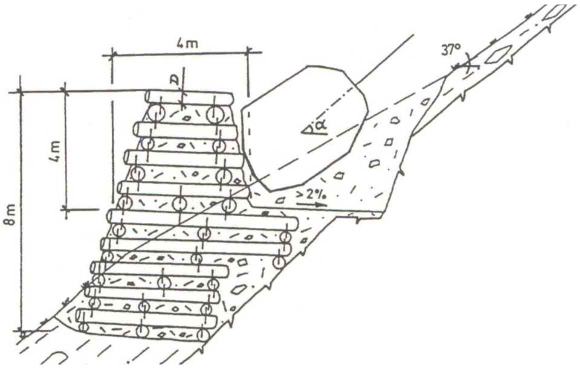
EXAMPLES

Reinforced ground embankment used to control of both local rock falls and a potential collapse of 200.000 m^3

Assisi (Italy)

EXAMPLES

Ground embankment reinforced with wood



Dorénaz (Vallis - Switzerland)

ROCK FALL PROTECTION
USING REINFORCED EMBANKMENTS

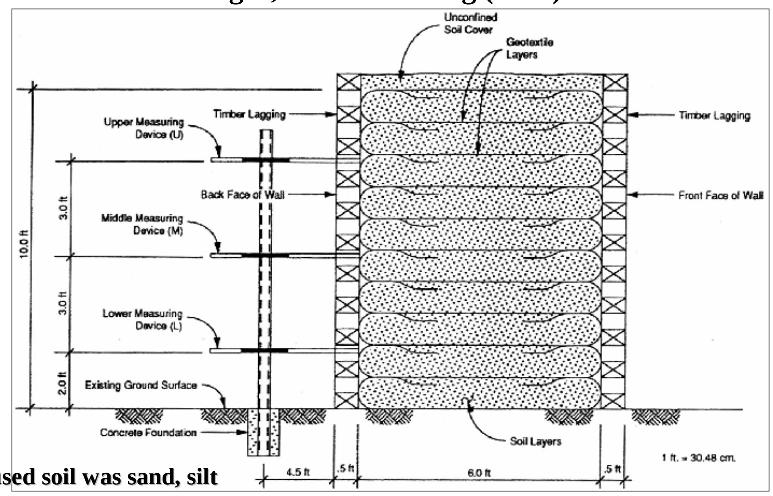
Full scale tests

Prof. Daniele PEILA
Politecnico di Torino

CARRIED OUT TESTS

Burroughs, Henson e Jiang (1993).

3m



The used soil was sand, silt and gravel

1.8 m

Burroughs, Henson e Jiang (1993): main results

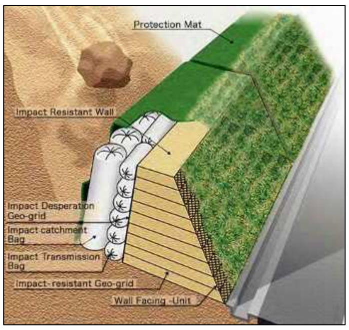
18 tests up to 1500kJ

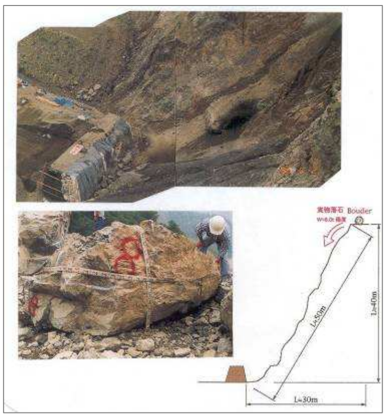
The blocks were rolled on a slope against the embankment

No structural collapses were registered

block mass [kg]	block speed [m/s]	contact time [s]	upstream displacements [m]	downstream displacements [m]
196	9.0	0.20	0	0.008
672	15.0	0.25	0.155	0.027
664	9.0	0.23	0.076	0.015
3388	13.5	0.50	0.305	0.199
2600	16.8	0.34	0.305	0.102
5212	19.2	0.43	0.609	0.207
8167	18.0	0.80	0.914	0.728

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

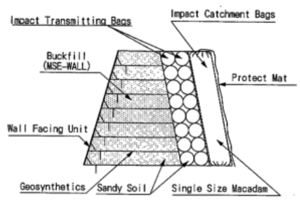




CARRIED OUT TESTS

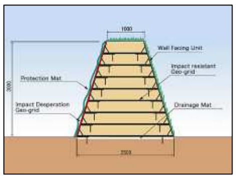
Test No.	Weight (kN)	Speed (m/sec)	Energy (kJ)	Height of Collision (m)	Max. Deformation I.S. (mm)	Max. Deformation B.S. (mm)
1	10.8	14.1	109.5	0.2	26	Ng
2	11.3	17.7	180.6	3.0	295	Ng
3	33.0	24.0	969.8	2.0	222	Ng
4	28.0	10.0	142.9	0.5	99	Ng
5	20.0	24.0	58.8	0.5～1.0	Ng	Ng
6	23.0	17.7	367.6	0～3.0	Ng	Ng
7	60.0	20.8	1324.4	0～4.0	161	Ng
8	77.0	24.0	2262.9	2.0～3.0	N.M.	91
9	170.0	17.7	2717.3	3.0～4.0	N.M.	500

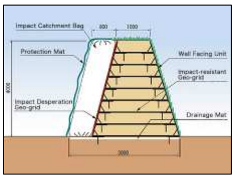
Note: I.S.=Impact surface; B.S.=Buck surface;
Ng=Negligible; N.M.=Can not be measured

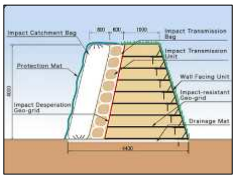


CARRIED OUT TESTS

Fukui tests – Japan (2001-2002)
Different types of reinforced strata;
Blocks with a weigth of 30 kN dropped from 4m.



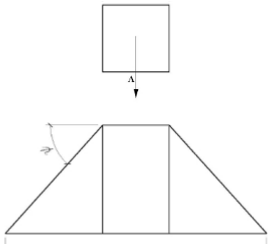


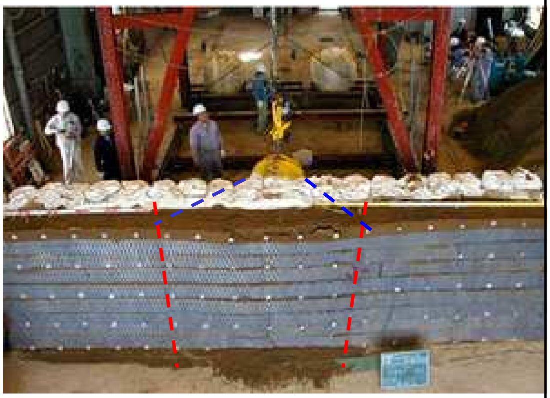


CARRIED OUT TESTS

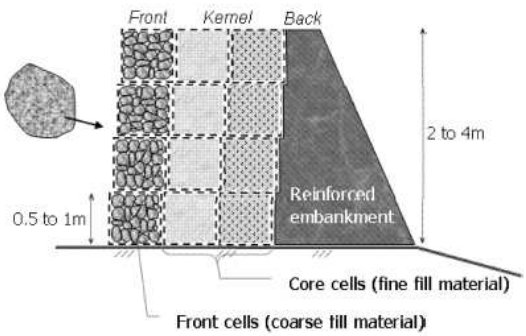
2001-2002 Fukui Japan

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

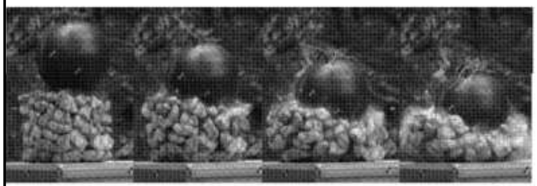
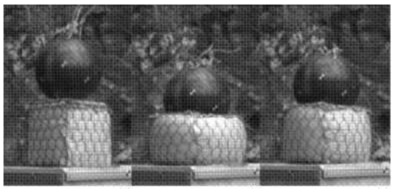


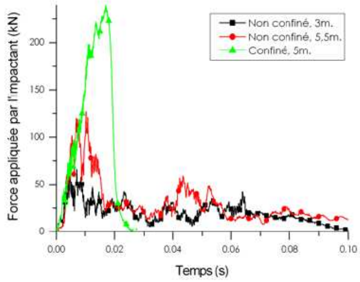


Lambert, Gotteland and Nicot (2009)
Cellular structure



Lambert, Gotteland and Nicot (2009)
Cellular structure

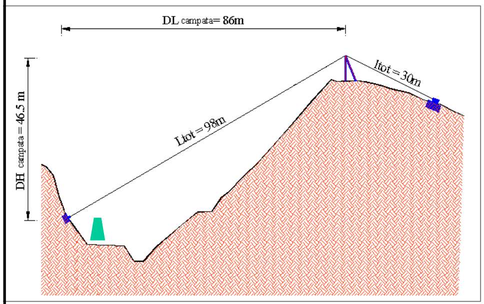





Lambert, Gotteland and Nicot (2009)
Cellular structure

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.



Peila D., Oggeri C., Castiglia C. (2007), *Ground reinforced embankments for rockfall protection: design and evaluation of full scale tests*, Landslides, vol. 4(3), pp. 255-265

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

Politecnico di Torino tests

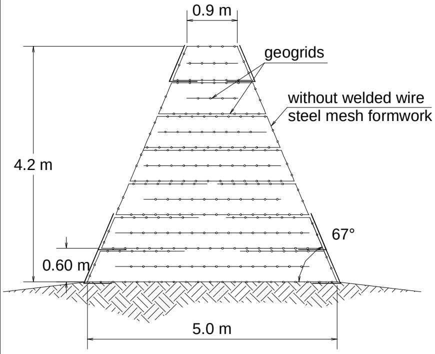
Carried out test on reinforced embankments

Test number	block speed [m/s]	block mass [kg]	block energy [kJ]	number of impacts	face surface steel mesh	Geogrid types	soil type	geometry type (Fig. 4)
1	31	5000	2402	1	Yes	tensile strength 50kN/m	1	a)
2	31	8700	4180	3	Yes	tensile strength 45kN/m	1	a)
3	31	8700	4180	1	No	Absent	1	a)
4	31	8700	4180	1	Yes	tensile strength 45kN/m	2	b)

Soil 1 c' [kPa]=9 ϕ' [°] = 34 γ' [kN/m^3]=21 sand and gravel

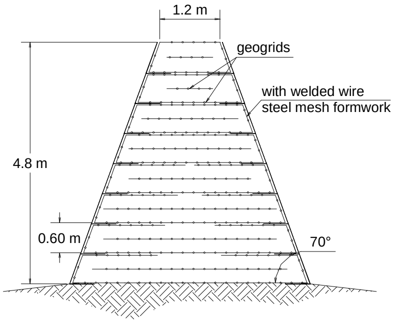
Soil 2 c' [kPa]= 50 ϕ' [°] = 30 γ' [kN/m^3]=17 silt and clay

Politecnico di Torino tests



Embankment geometry a)

test n. 1, 2 , 3



Embankment geometry b)

test n. 4

Politecnico di Torino tests

Test 2 – first impact

Politecnico di Torino tests

Test 2 – second impact

Politecnico di Torino tests

Test 2 – third impact

CARRIED OUT TESTS

Politecnico di Torino tests

Test 1

CARRIED OUT TESTS

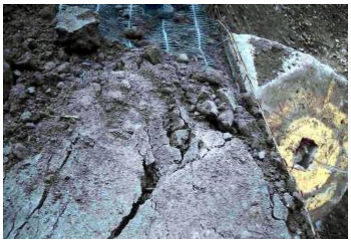
Politecnico di Torino tests

Test 3

CARRIED OUT TESTS

Politecnico di Torino tests

Test 4



ROCK FALL PROTECTION USING REINFORCED EMBANKMENTS

Numerical back analysis of full scale tests

Prof. Daniele PEILA
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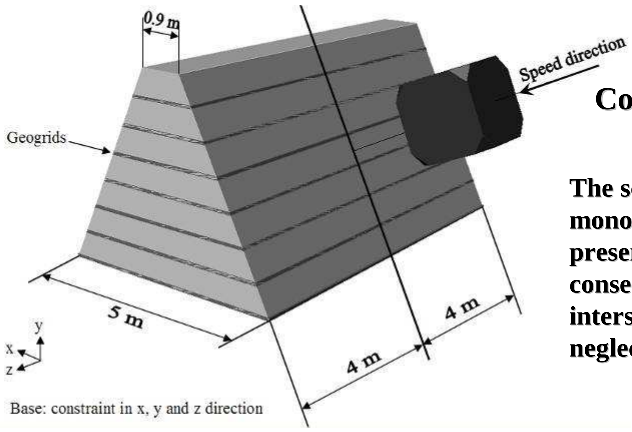
BACK ANALYSIS OF CARRIED OUT TESTS



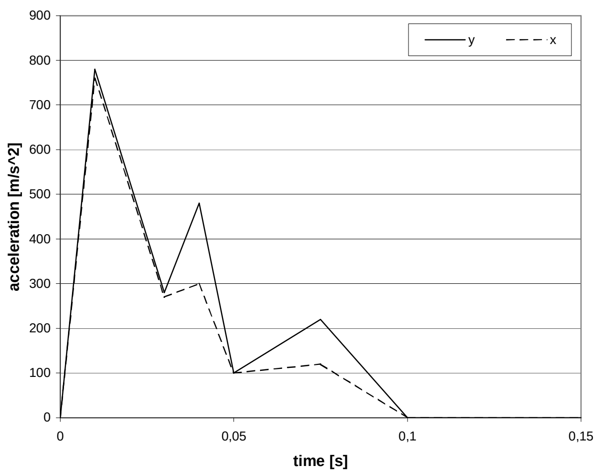
Code: Abacus / Esplicit 3D FEM

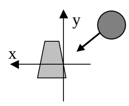
The soil was a homogeneous and mono-phase material and the presence of water and the consequent consolidation and interstitial stresses were neglected

Peila D., Oggeri C., Castiglia C. (2007), *Ground reinforced embankments for rockfall protection: design and evaluation of full scale tests*, Landslides, vol. 4(3), pp. 255-265

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

BACK ANALYSIS OF CARRIED OUT TESTS





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/s2 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.

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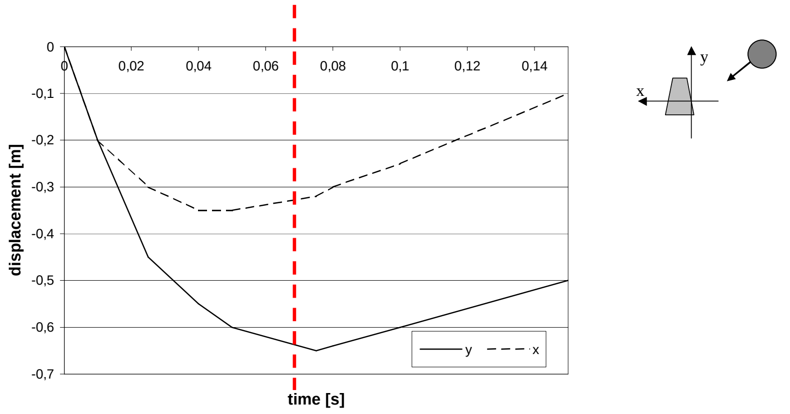
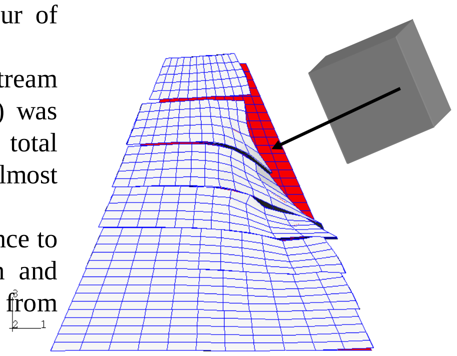


Displacement of the barycentre of the block vs. time during the impact in the back-analysis of test n. 1.

back analysis test 1

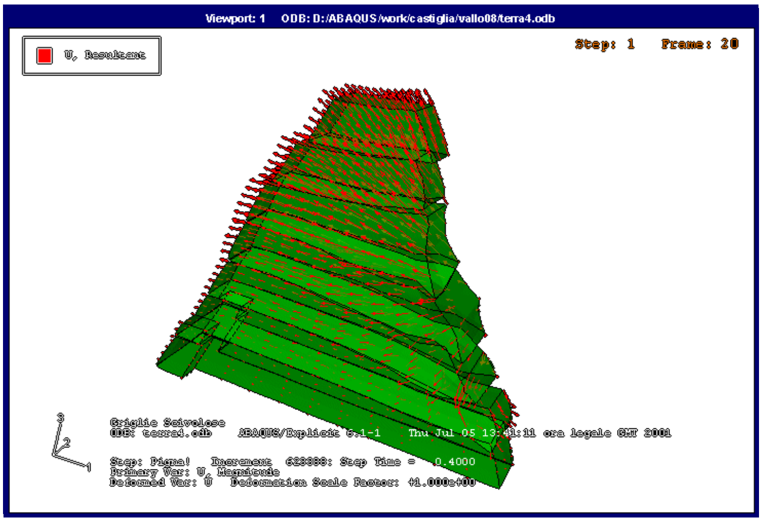
An analysis of the displacements showed that:
- only the layers directly impacted by the block translate parallel to the direction of the geogrids due to the anisotropic behaviour of the ground reinforced embankment;
- the penetration of the block on the upstream side (measured in a horizontal direction) 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.

The upper layer must be considered as safety factor

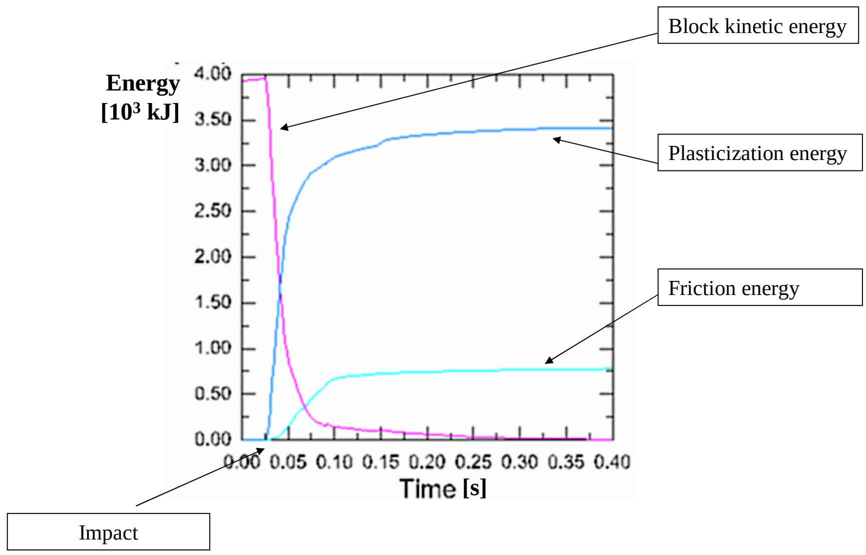


Very good agreement with the real scale test

back analysis test 1



Displacements in the central cross section after 0.1s – 0.2s – 0.3s – 0.4s

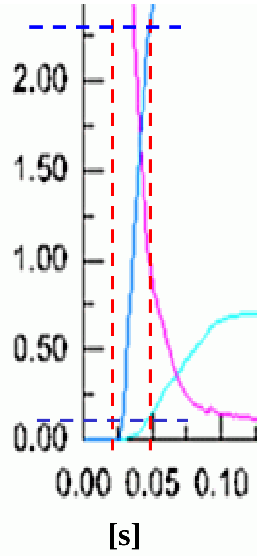


Energy distribution block vs. time in the back-analysis of test n. 1.

BACK ANALYSIS OF CARRIED OUT TESTS

61% of the energy

5% of the energy



The impact phenomenon inside the embankment can be sub-divided 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 o n the geogrid layers.

ROCK FALL PROTECTION USING REINFORCED EMBANKMENTS

Numerical back analysis of a real event

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Politecnico di Torino

BACK ANALYSIS OF A REAL EVENT

Valley side after impact – no displacements can be observed

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).

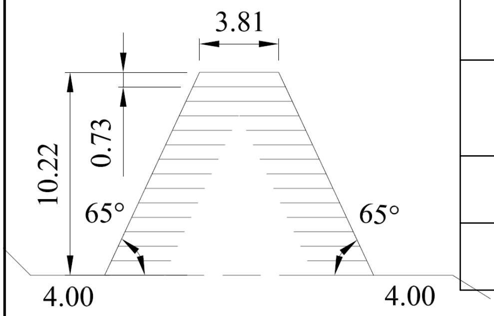
BACK ANALYSIS OF A REAL EVENT

Considered impact : block of 6 m^3

Modelled block sizes: 2*2*1.5 m
Block mass of 15000 kg

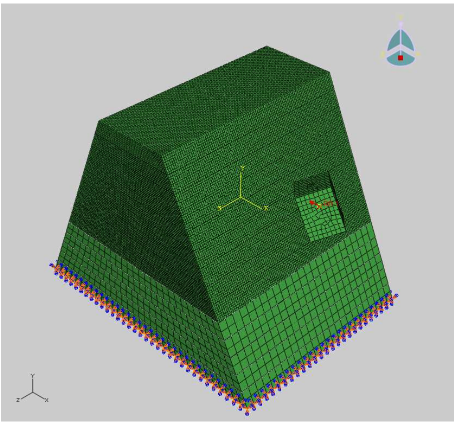
BACK ANALYSIS OF A REAL EVENT

Geotechnical parameters used in the numerical modelling



Soil density [kg/m³]	1900
Elasticity modulus [kPa]	75000 90000 110000
Poisson modulus [-]	0.30
Drained friction angle [°]	35

BACK ANALYSIS OF A REAL EVENT



Impact speed:

Vh= 20 m/s

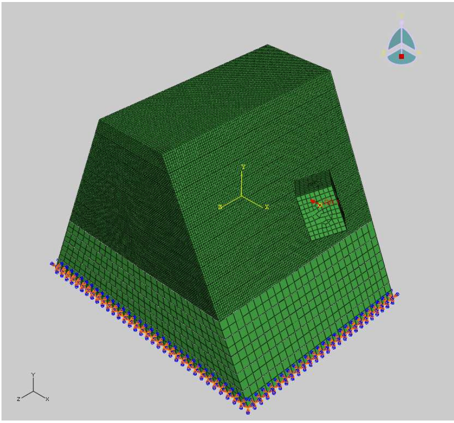
Vv = 0 m/s

Friction coefficient beween the layers: 0.40

Impact with a corner of the block

Used code: Abacus Explicit 3D

BACK ANALYSIS OF A REAL EVENT

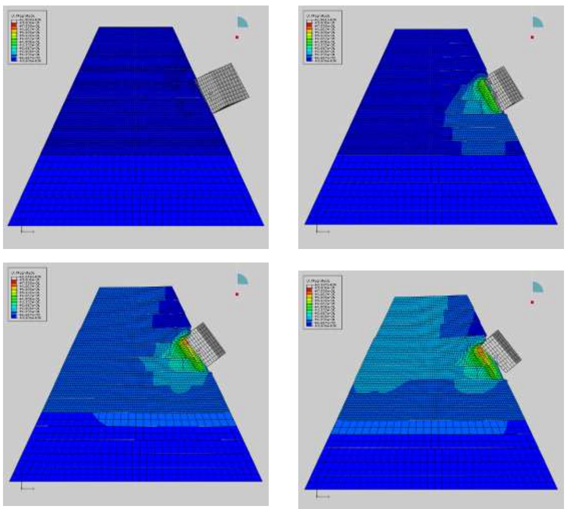


The mountain side penetration depends on the soil elasticity modulus

Elasticity modulus [kPa]	Penetration mountain side [m]	Sliding valley side [m]
75000	0.74	0.17
90000	0.71	0.07
110000	0.62	0.17

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

BACK ANALYSIS OF A REAL EVENT



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).

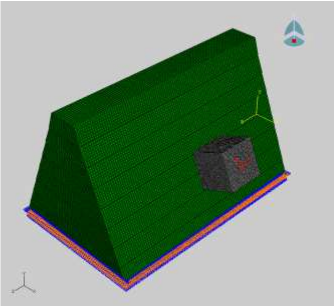
No physically measurable displacement on the valley side

ROCK FALL PROTECTION USING REINFORCED EMBANKMENTS

Parametrical numerical analysis

Prof. Daniele PEILA
Politecnico di Torino

PARAMETRICAL NUMERICAL ANALYSIS



Code: Abaqus/Explicit 3D FEM

90 different simulation

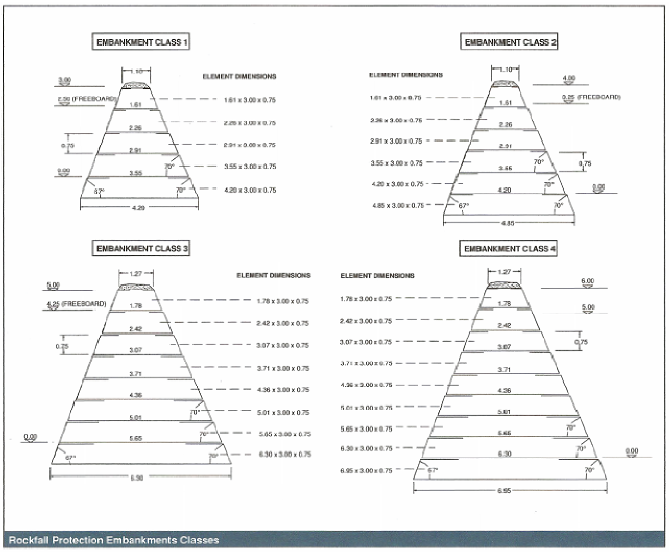
Impact on different embankments reinforced with Maccaferri mesh.

4 different standard geometries were considered.

The impact cubic element had different sizes 1.5m and 2m side

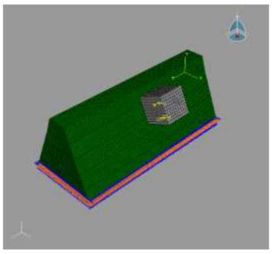
Different impact speed were considered to get different impact energy

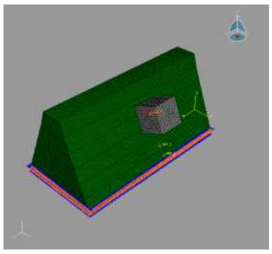
PARAMETRICAL NUMERICAL ANALYSIS

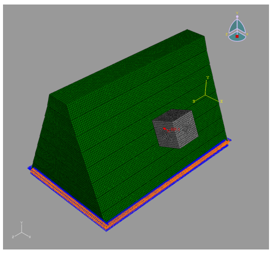


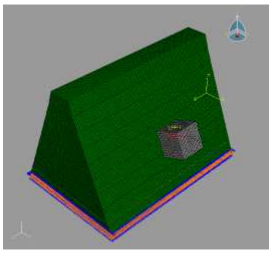
Maccaferri: MAC.RO System

PARAMETRICAL NUMERICAL ANALYSIS

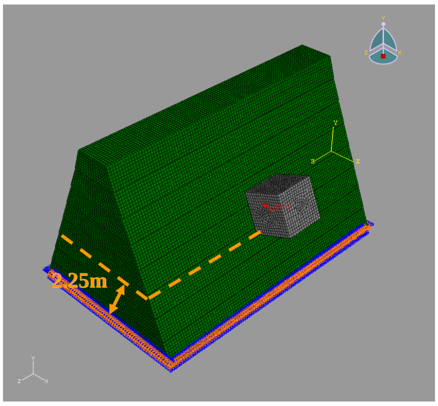








PARAMETRICAL NUMERICAL ANALYSIS



Impact heigth for all the models

PARAMETRICAL NUMERICAL ANALYSIS

Soil properties

Densità ρ [kg/m³]	2100
Modulo di Young E [kPa]	110000
Coefficiente di Poisson υ [-]	0.25
Angolo d'attrito Mohr-Coulomb Φ [°]	34
Angolo d'attrito Drucker-Prager β [°]	54
Flow stress ratio k [-]	0.78
Dilatanza ψ [°]	0
Yield stress [kPa]	540

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

PARAMETRICAL NUMERICAL ANALYSIS

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.

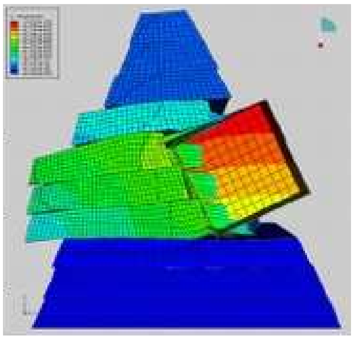
The contact between the soil layers which are obtained with the reinforcement elements has been modelled using a "*master-slave weighted penalty method*" (which checks for possible mesh collision between the given surfaces or nodes during every time step and calculates a surface reaction force applied 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

PARAMETRICAL NUMERICAL ANALYSIS

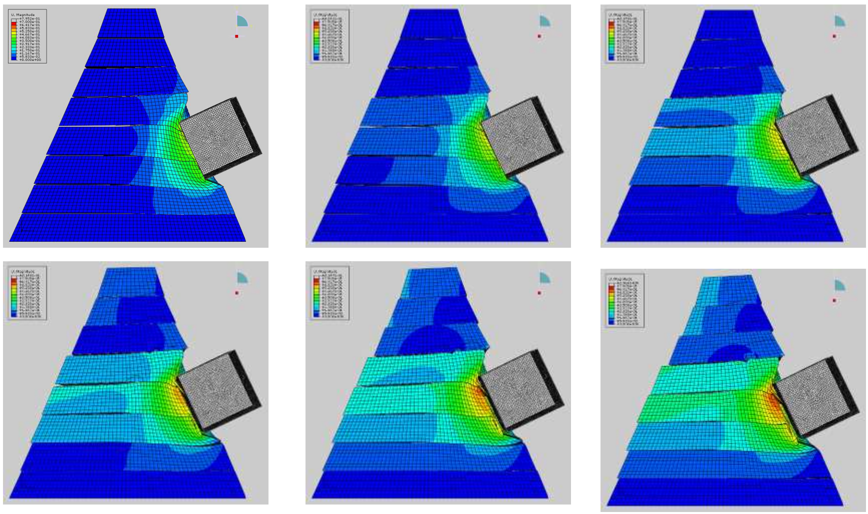
The simulations have been carried out with the same type of block by rising up the impact speed till collapse of the structure occurs.

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

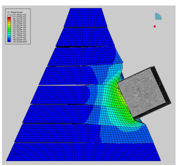
PARAMETRICAL NUMERICAL ANALYSIS



Example of the contours plot of the displacement t = 0.03-0.06-0.09-0.12-0.15-0.17s (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

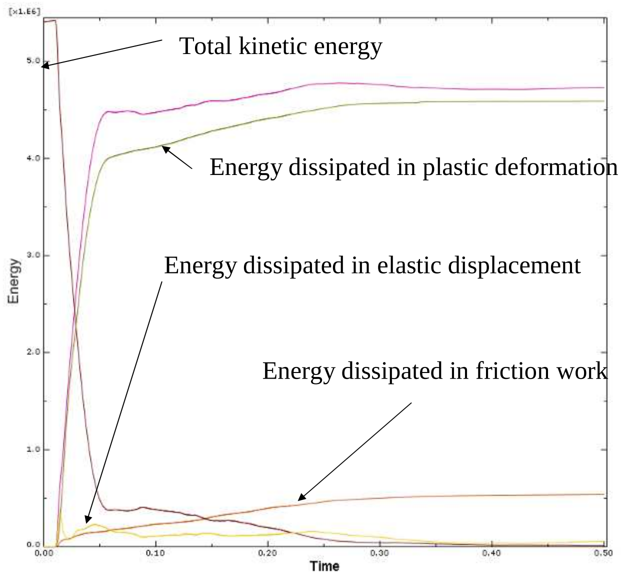
PARAMETRICAL NUMERICAL ANALYSIS



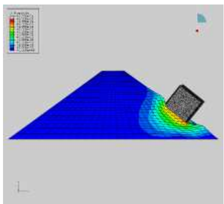
Reinforced enbankment

Impact energy

5400kJ



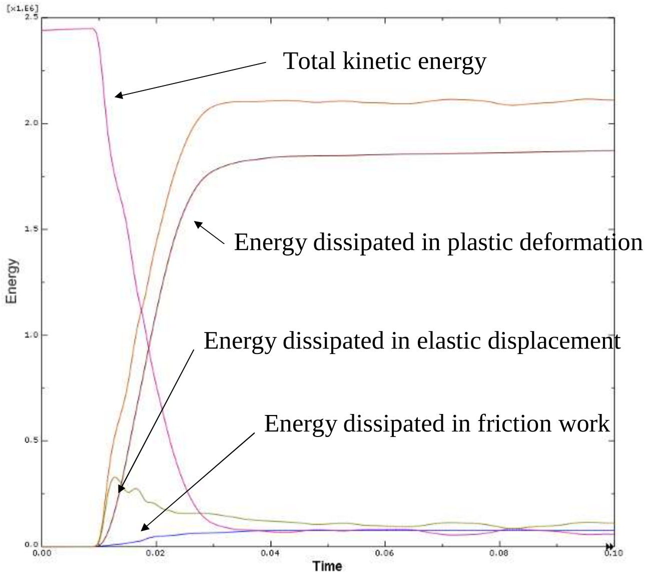
PARAMETRICAL NUMERICAL ANALYSIS



Unreinforced enbankment

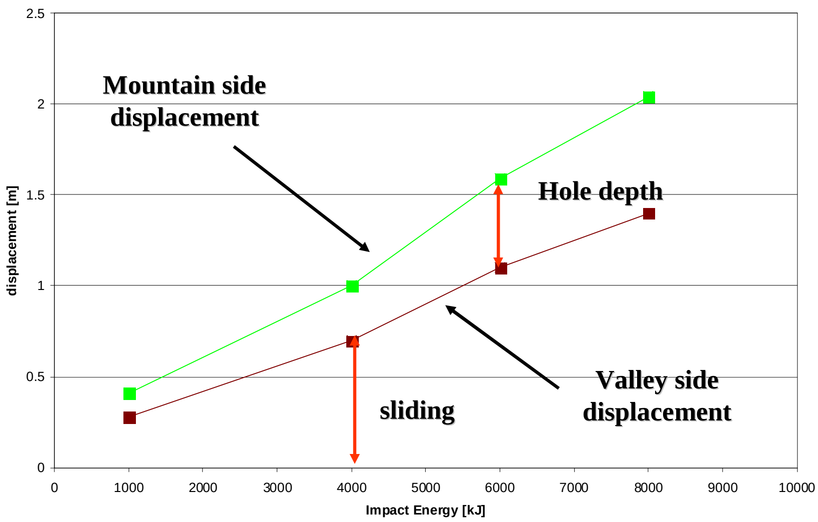
Impact Energy

2400kJ



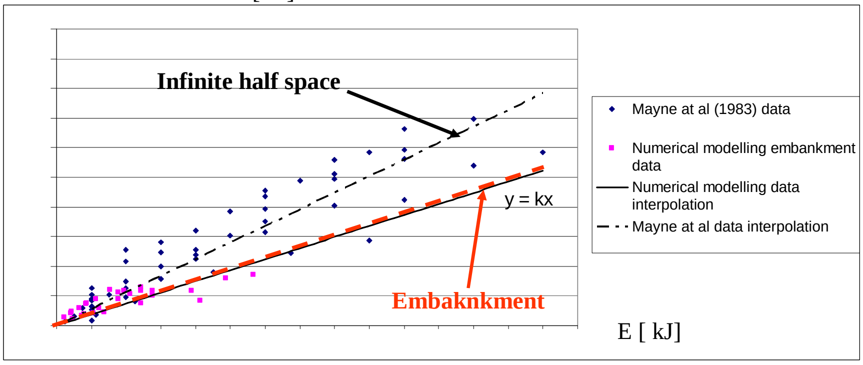
PARAMETRICAL NUMERICAL ANALYSIS

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



PARAMETRICAL NUMERICAL ANALYSIS

Penetration * block area [m^3]



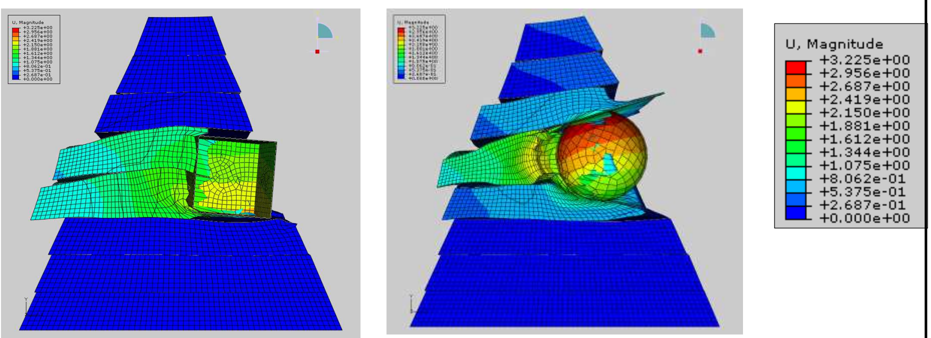
The angular coefficient of the interpolation line of the penetration data (k) depends on the reinforcement type (strength and friction between the layers), type of soil and its compaction and type of embankment.

Penetration * block area = k * E

PARAMETRICAL NUMERICAL ANALYSIS

Influence of block shape

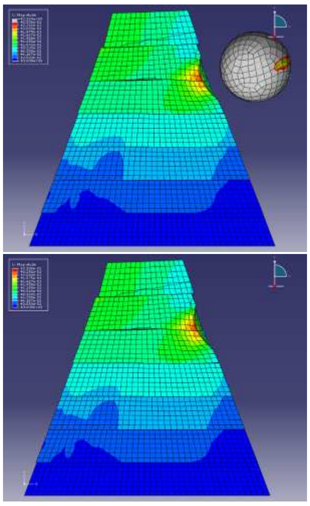
Comparison between a cubic block and a sphere block

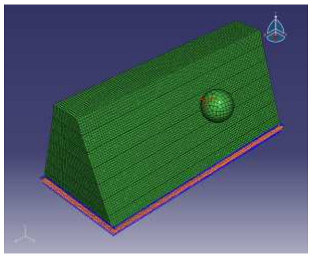


Contour plots of displacements when the block speed is nil when modelling Macro Embankments of class 3 with two blocks of a mass of 8700kg and an impact energy of 8000 kJ.

PARAMETRICAL NUMERICAL ANALYSIS

Example of an impact at low energy





heigth 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

Contour displacement (scale from 0 to 0.25 m).

ROCK FALL PROTECTION USING REINFORCED EMBANKMENTS

Design approach

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Politecnico di Torino

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

it cannot be considered as a usual design tool

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

Design of a rockfall reinforced ground embankment

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)

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

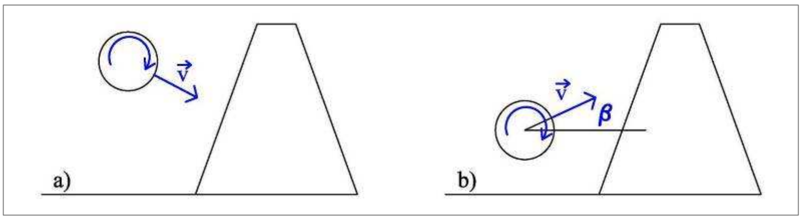
dynamic computation

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.

dynamic computation

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.



Direct flying impact **Impact just before the impact on the embankment**

dynamic computation

c. 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 reabilitation and repair at the Service Limit State.

dynamic computation

The DESIGN SCHEME therefore requires that:

a) the energy ($E_{embankment}$) that can be dissipated in safe condition by the embankment is greater (with an adequate safety factor) than the design energy (E_{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:

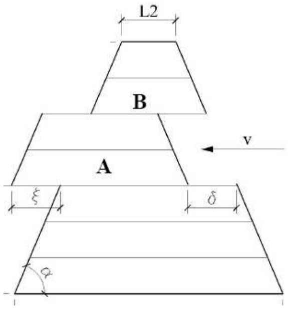
$$E_{design} - \frac{E_{embankment}}{\gamma_{ER}} \leq 0$$

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

$$h_{design} - \frac{h_i}{\gamma_h} \leq 0$$

dynamic computation

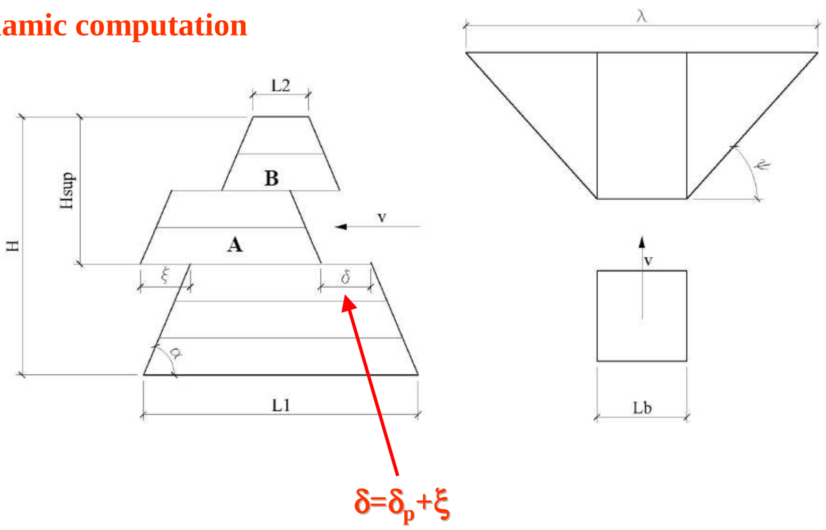
The maximum impact energy the embankment ($E_{embankmnet}$) can fulfil is computed by verifying the global static stability of the structure after the impact, in the deformed shape and taking into account both the maximum sliding toward valley of the layers interested by the impact (ξ) and the penetration (δ_p) due to soil compaction.



The A and B bodies must remain statically stable

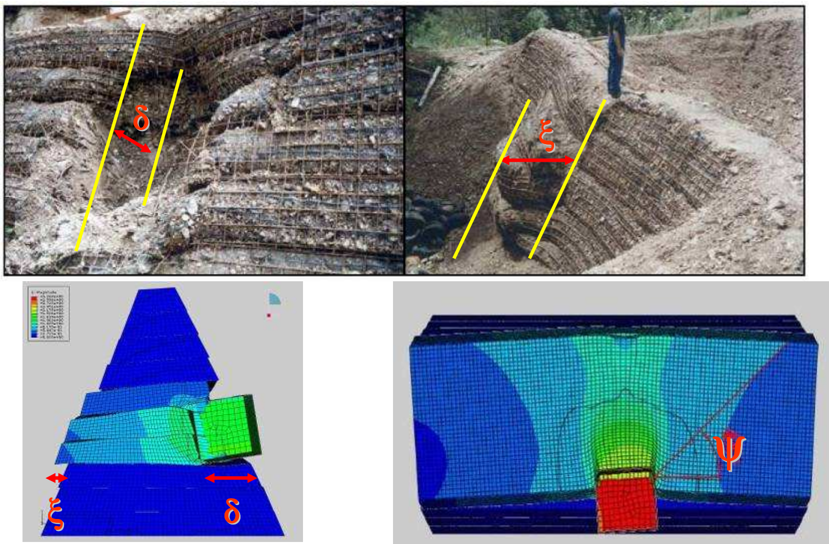
The evaluation of this two parameters can be done by both with numerical or with analytical computations.

dynamic computation



Deformed shape after the impact used for the analytical computation

dynamic computation



dynamic computation

Evaluation of the deformed parameters

The δ_p value (plastic penetration in the mountain side) can be obtained:

a. from the peak force acting during the stopping phase (F_{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

$$\delta_p = \frac{(\text{Pr}\,\%) \cdot mv^2}{F_{\max}}$$

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):

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

where: M_E: the soil elasticity coefficient (generally computed from the first load curve of a plate loading test) [kN/m^2], R: the impacting block radius [m], E_{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 geometical conditions of the impact.

dynamic computation

Evaluation of the deformed parameters: option a)

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

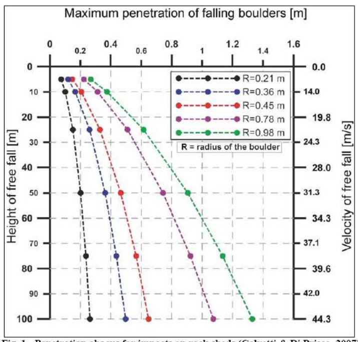


Fig. 1 - Penetration abacus for impacts on rock sheds (Calvetti & Di Prisco, 2007)

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 geometical conditions of the impact.

dynamic computation

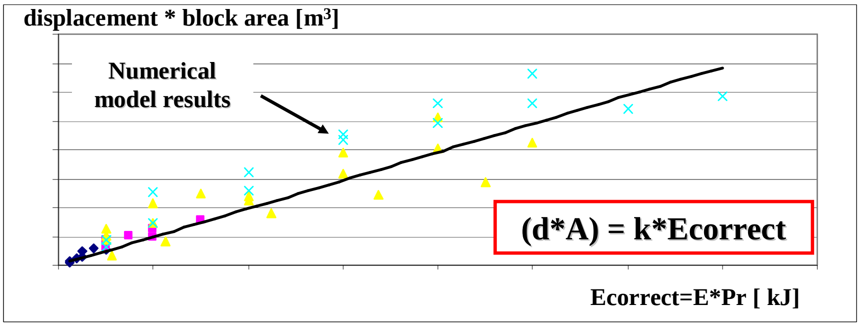
Evaluation of the deformed parameters: option b)

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.

dynamic computation

Evaluation of the deformed parameters: option b)

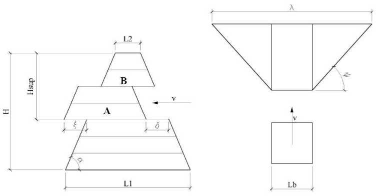


Example of interpretation of numerical modeling models

dynamic computation

Evaluation of the deformed parameters

The ξ 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.



OPERE DI PROTEZIONE CONTRO LA CADUTA MASSI: ASPETTI PROGETTUALI
ANALYTICAL DESIGN METHOD

dynamic computation

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 suggeste values are:

mountainside displacement lower than 20% of the thickness of the embanlment at the impact heigth 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 that 0.3-0.4m since the movement are induced by sliding.