Yellowstone Άνοιξη των Κοραλλιών (Shell Spring), Η.Π.Α.

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& ΓΕΩΤΕΧΝΙΚΗΣ
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Τα Νέα της Ε Ε Ε Ε Γ Μ

41

Αρ. 41 – ΝΟΕΜΒΡΙΟΣ 2011

SIMSG ISSMGE

Η Εκτελεστική Επιτροπή της ΕΕΕΕΓΜ
εύχεται σε όλους Καλά Χριστούγεννα

C O N T E N T S

Fire Falls in Yosemite National Park

Yosemite National Park is located in California, and the falls were found in 1890 by Canyon National Park rangers.

Fire Falls look this way due to a specific angle of the sun's rays falling on the water and the formation of the optical effect, which make it look like flame.

This effect can only be witnessed 2 times and for two weeks, usually in December and January.

Many photographers often wait take several years to shoot pictures of the fire falls, because it needs to meet several conditions to get the right effect.

Unfortunately, during December to February the weather is very harsh and volatile in the National Park and the park is closed. Based on these harsh conditions, the successes of shooting fire waterfall photos are not many.

EUROPEAN YOUNG GEOTECHNICAL ENGINEERS CONFERENCE 2012

EYGEC 2012 Gothenburg
22nd
European Young Geotechnical Engineers Conference
Gothenburg, Sweden, August 26th to 29th, 2012
www.sgf.net

Invitation

On behalf of the ISSMGE and the Swedish Geotechnical Society the organizing committee have the pleasure of inviting you to Gothenburg, Sweden where the 22nd European Young Geotechnical Engineering Conference will be held from Sunday 26th of August to Wednesday the 29th of August.

All sessions will be held at Chalmers University of Technology which is located in the centre of Gothenburg. The city of Gothenburg is Sweden's second largest city and is situated on the West coast. For more information about the city refer to www.goteborg.com.

Conference Topics

Papers covering the following topics will be accepted:

* Site investigations and laboratory testing
* Design parameters and modelling
* Shallow and deep foundations
* Deep excavations and retaining structures
* Tunnelling and underground structures
* Slope stability and landslides
* Infrastructure projects
* Ground improvement
* Environmental geotechnics

Local Conference Organizing Committee

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Henrik Möller, henrik.moller@tyrens.se

Η ΕΕΕΕΓΜ θα εκπροσωπηθή στο συνέδριο από δύο νέους συναδέλφους μέλη της (ηλικίας μέχρι 35 ετών την 31.12.2012) μετά από επιλογή βάσει των περιλήψεων των άρθρων τους που προτίθενται να παρουσιάσουν.

Οι ενδιαφερόμενοι θα πρέπει να αποστείλουν τις περιλήψεις στην γραμματεία της ΕΕΕΕΓΜ (geotech@central.ntua.gr) μέχρι τις 10 Ιανουαρίου 2012. Η προθεσμία για την αποστολή στους διοργανωτές των δύο περιλήψεων που θα επιλεγούν λήγει στις 16 Ιανουαρίου 2012.

Geosynthetics application for the mitigation of natural disasters

Heinz Brandl (*)

ABSTRACT

The paper first describes the versatile application of geosynthetics for the mitigation of floods, landslides, rockfalls, debris flows and avalanches. It focuses on dykes or flood protective dams respectively, on geosynthetic reinforced stabilizing fills (up to 130 m height) and barrier dams. Geosynthetic reinforced floating embankments (up to 70 m height) in creeping slopes and seismic areas show clear advantages over rigid structures (e.g. bridges) not only from a geotechnical point of view but also regarding economy, maintenance and environmental aspects. Environmental protection is predominantly considered by gaining renewable energy from the ground via “energy-geosynthetics". Several other applications are also mentioned. Compaction optimization and control of geosynthetic-soil structures is recommended by roller-integrated CCC (Continuous Compaction Control), thus improving their behaviour significantly.

1 FLOOD PROTECTION

1.1 General

Climate change requires a close cooperation between hydrology, hydro engineering and geotechnical engineering. The increasing frequency and magnitude of floods indicate that the hitherto prognoses of such events should be revised for many regions. Consequently, the improvement of old dykes or flood protection dams respectively and the construction of new ones has become essential since the 1990ies. Environmentally optimal solutions are achieved by (re-)creating retention areas, combined with dams – that can be overflowed, thus flattening the flood wave. Besides these permanent flood protection measures temporary ones are increasingly required for emergency situations. Geosynthetics have proved very suitable also for mobile systems.

The topic of coastal protection should be also mentioned in the list of possible applications of geosynthetics in the field of natural disaster mitigation. However, it will not be discussed in this paper because it was dealt with already in the 2006 Giroud Lecture delivered by C. Lawson.

Quality assessment of existing structures, design aspects for new ones, and the required safety factors depend on purpose, risk potential and monitoring intensity. Therefore dykes (permanently loaded by water) and flood protection dams (only temporarily loaded by water) should be distinguished. Most geotechnical aspects, however, are relevant for both.

1.2 Future trends and residual risks

Floods have affected worldwide millions of people over the past decades. In several regions the magnitude and frequency of flood waves have increased dramatically since long-term measurements and historical reports have existed. Figure 1 shows an example from Austria, where a 2000 to 10 000 – year flood event was back calculated from the flood disaster in the year 2002. Such hitherto singular values cannot be taken as design values for flood protective dams; but they underline the necessity of local overflow crests or spillway sections respectively. Moreover, they clearly demonstrate that a residual risk is inevitable – despite most costly protective measures. The so called “absolute safety” as frequently demanded by the public, by politicians, media or jurists cannot be achieved in reality.

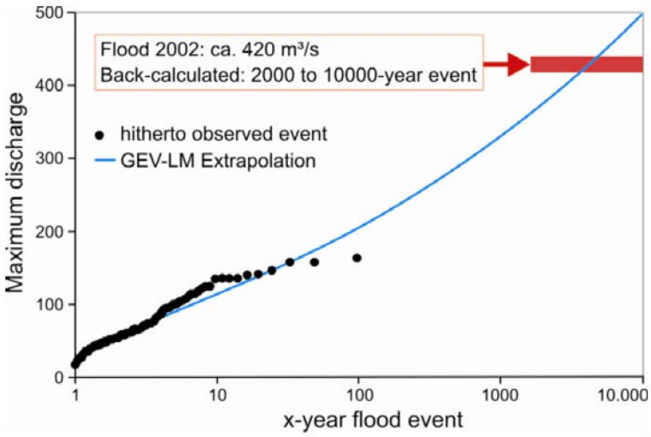


Figure 1. Statistics of the annual maximum discharge values of the river Kamp in Austria (Gutknecht et al. 2002).

1.3 Failure modes of dykes/dams

The knowledge of possible failure modes is an essential prerequisite for a reliable quality assessment of existing dykes/dams and for an optimized design of new ones, both in connection with the application of geosynthetics. Figure 2 summarizes the dominating failure modes for typical ground conditions along rivers (near-surface low permeability sandy to clayey silts underlain by high permeability sand, gravel):

- Slope failure due to excessive pore-water pressures, seepage or inner erosion;
- Overtopping of the dyke/dam crest;
- Slope failure due to a quick drop of the flood water level;
- Hydraulic fracture;
- Surface erosion and failure of the water-side slope due to wave action;
- Piping due to animal activities, especially from beavers and rats;
- Unsuitable planting of dykes (especially trees with flat roots).

Hydraulic failure is frequently underestimated but may occur in different forms (e.g. Eurocode 7):

- By uplift (buoyancy).

  The pore-water pressure under the low permeability soil layer exceeds the overburden pressure

- By heave.

  Upward seepage forces act against the weight of the soil, reducing the vertical effective stress to zero, soil particles are then lifted away by the vertical water flow. This “boiling” dominates in silty-sandy soil and is combined with internal erosion (Figs. 3a, 3b).

- By internal erosion.

  Soil particles are transported within a soil stratum or at the interface of soil strata (Figs. 4, 5). This may finally result in regressive erosion, leading to ground failure of the dyke/dam.

- By piping.

  Failure by piping is a particular form of internal erosion, where erosion begins at the surface, then regresses until a pipe-shaped discharge tunnel is formed (Fig. 6). Failure occurs as soon as the water-side end of the eroded tunnel reaches the river bed or bottom of the reservoir. This process may be induced or significantly promoted by animal activities, as field observations over many years have disclosed (Fig. 2).

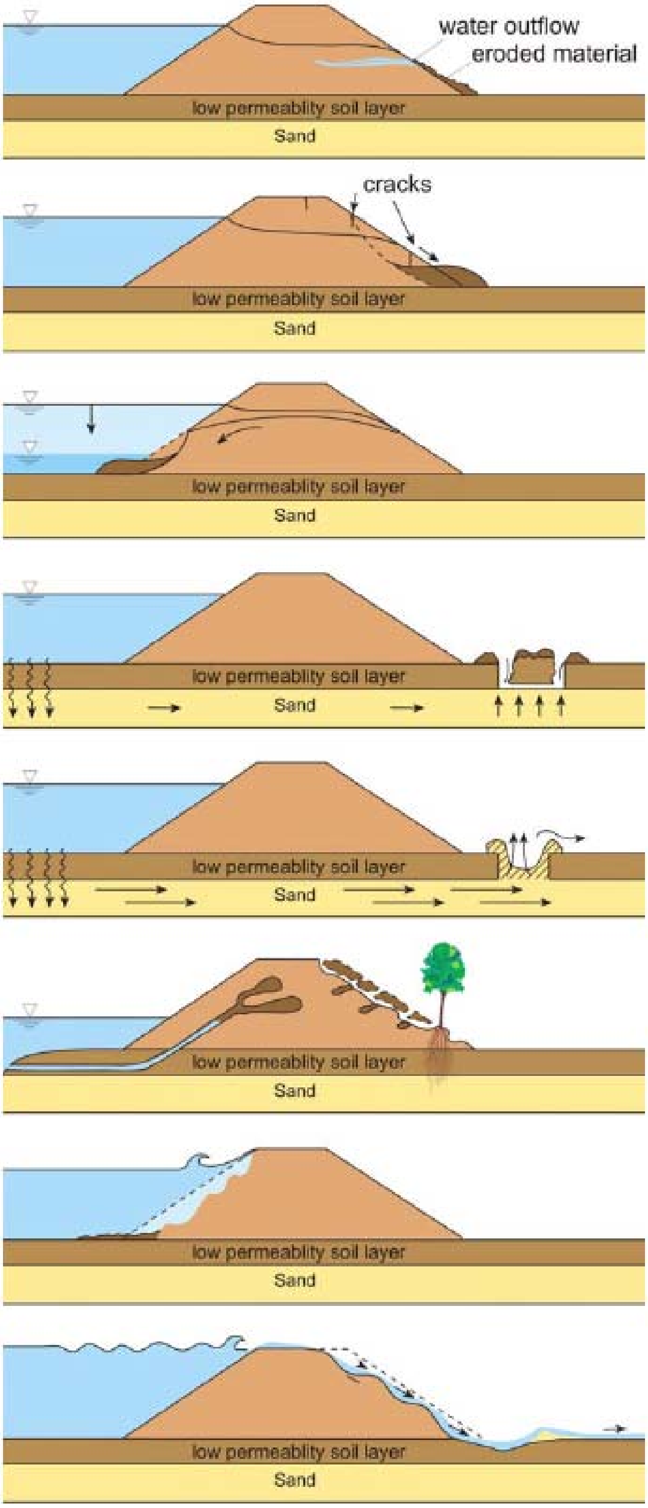


Figure 2. Failure modes of dykes/dams. Near-surface stratum of low permeability (possibly with local "windows"). After VDZ 2002.

Figure 3. "Boiling" behind dykes indicates inner erosion and beginning of hydraulic failure. a) randomly distributed multiple boiling; b) detail of an erosion spot.

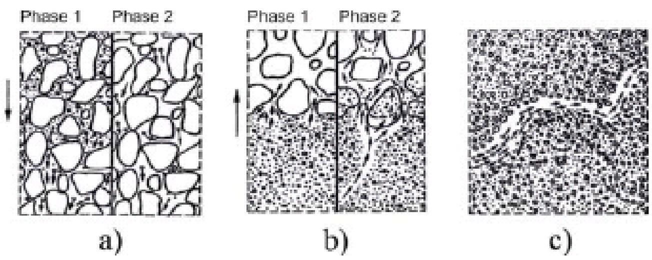


Figure 4. Hydraulic fracture of dykes or flood protection dams due to seepage through or beneath the dyke/dam (Ziems 1967).
(a) Suffusion (fine particles move into pore voids of coarse grain fractions)
(b) Contact-erosion at the interface of soil strata
(c) Internal erosion at steady state flow condition

Eurocode 7 (EN 1997-1) states that in situations where the pore-water pressure is hydrostatic (negligible hydraulic gradient) it is not required to check other than failure by uplift. In the case of danger of material transport by internal erosion filter criteria shall be used. If the filter criteria are not satisfied, it shall be verified that the critical hydraulic gradient is well below the design value of the gradient at which soil particles begin to move.

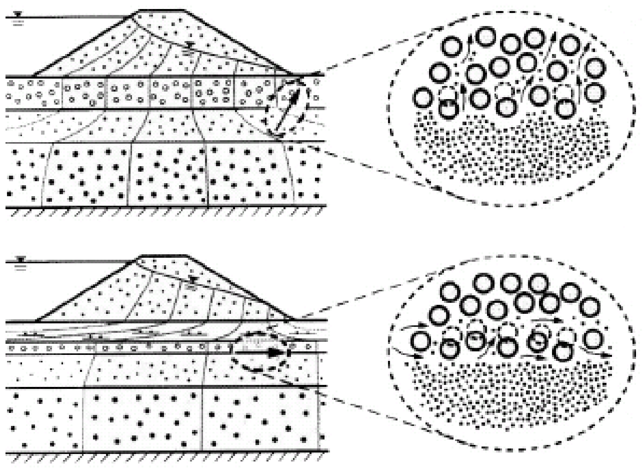


Figure 5. Contact-erosion perpendicular or parallel to an interface of soil strata.

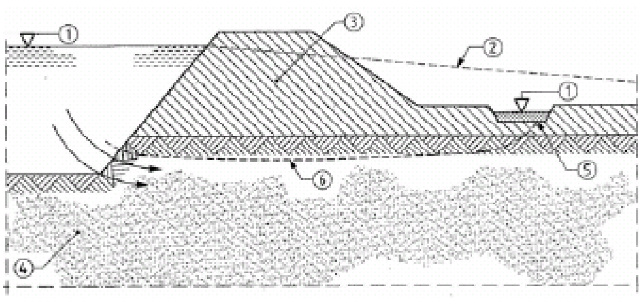


Figure 6. Example of conditions that may cause piping (Eurocode EN 1997-1).

Experience has shown that the magnitude of the critical hydraulic gradient where internal erosion begins is frequently over-estimated. Figure 7 summarizes the critical values on the basis of field observations, geotechnical measurements, literature and long-term experience for different soils A to D. For comparison, the conventional criterion $i_{crit} = \gamma'/\gamma_w$, Lane's criterion, and the critical zones after Eurocode 7 or Chugaev respectively are also plotted in the diagram.

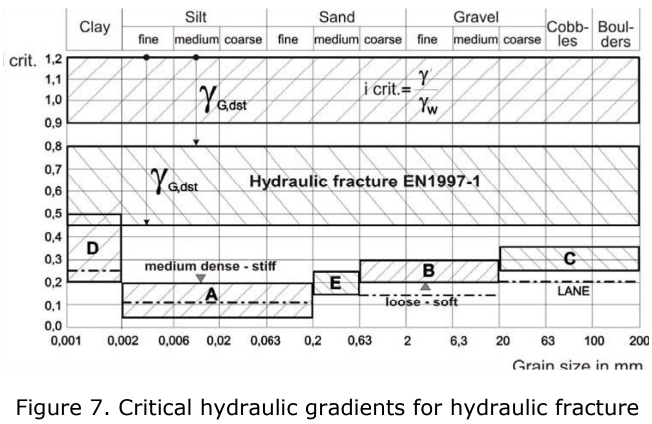


Figure 7. Critical hydraulic gradients for hydraulic fracture (internal erosion) (Brandl, Hofmann 2006).

$i_{crit.}$ depends not only on grain size distribution and density /stiffness but also on flow pressure.

$\gamma_{G,dst}$ = partial safety factor for permanent unfavourable effects.

Filter protection is generally provided by use of non-cohesive granular material (natural soil) that fulfils adequate design criteria for filter materials. Filter geotextiles have been used increasingly since the early 1970ies. Common filter criteria for soils are from Terzaghi and Sherard, and for geotextiles from Giroud (2003, 2010) and Heibaum (2006). All criteria have particular limitations, whereby noncohesive and cohesive soils have to be distinguished. While two criteria are sufficient for granular filters (permeability criterion, retention criterion) four criteria are required for geotextile filters (Giroud, 2010): Porosity criterion and thickness criterion have to be considered additionally.

1.4 Quality assessment of dykes/dams

When assessing the quality of dykes or flood protection dams the purpose and risk potential of the structure should be taken into consideration. Dykes as fill dams directly along reservoirs, rivers or canals are continuously monitored, whereas flood protection dams are loaded by water only during floods. The later are therefore more critical, as experience has shown.

Besides levelling of the crest and control of the geometric data the following (geotechnical) investigations have proved successful:

- Seasonal field observations, especially during floods;
- Documentation of water discharge and localization of wet slope spots/zones during floods, depending on magnitude and duration of the flood wave;
- Arial photographs, especially during and after floods (and in normal periods for comparison);
- Spot checking by soundings, borings, exploratory pits;
- Field tests, laboratory tests;
- Geophysical investigations;
- Tracer testing;
- Infrared photographs for localization seepage;
- Roller-integrated continuous compaction control (CCC).

Investigations in steps provide the best results and are most economical. Conventional spotchecking gives only random data, whereas investigations covering the entire area provide rather detailed information. Consequently, geophysical methods have been increasingly used during the past ten years. However, experience has disclosed that a reliable interpretation is only possible if at least two to three different methods are applied (e.g. geoelectrics, micro-gravimetry, combined surface wave / refraction seismic). Geophysical methods and electric tracers are also used to check the effectiveness of sealing elements (surface sealings, core sealings, cut-off walls). Multi-sensor survey systems together with spatially targeted electrical tracers enable the localization of leakage areas, hence leaks. Moreover, they have proved suitable for continuous leakage monitoring of sealing systems (liners, cutoff walls).

A complete quality assessment of the dyke/dam crest is possible by the roller-integrated continuous compaction control (CCC). This method registers the interaction between roller and ground (details see Chapter 5). The measuring depth reaches 2 to 2.5 m and thus covers the most critical zone of dykes/dams. Weak points are easily localized which is especially important for structures with crests that can be overflowed (spillway section). However, if the crest has a top cover of concrete, asphalt, rip-rap, or if it is very uneven, CCC cannot be used.

The advantage of CCC is not only a full control but also a compaction of weak zones. Figure 8 shows some results along an old dyke which was severely attacked by the previous flood. The weak points/zones were not visible along the crest, but could be clearly localized by CCC already during the first roller pass. After six roller passes the quality of the top 2 m of the dam had improved significantly though relatively weak spots still existed. However, the control values exceeded the lower limit value (30 for this particular fill material) with only one exception (23).

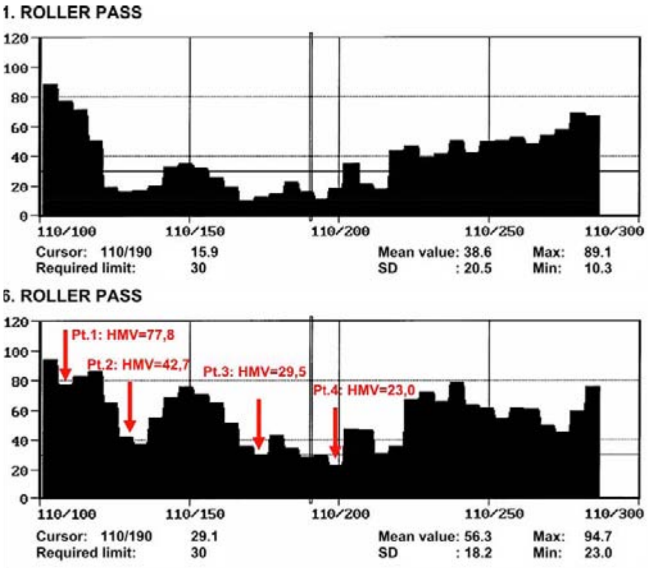


Figure 8. Results from rollerintegrated continuous compaction control (CCC) of an old flood protection dam after a severe flood. Weak spots clearly visible, also improvement after six roller passes.
Compaction degree/stiffness (dimensionless value) is plotted against the chainage of the dam.

1.5 Geosynthetics application for dykes/dams

Most dykes and flood protection dams are lower than 10 to 15 m, but are clearly longer than dams for hydropower generation. Moreover, the latter are cross structures whilst dykes / flood protection dams usually represent longitudinal barriers. Consequently, different design rules, maintenance and monitoring aspects have to be considered. This leads to significantly more application of geosynthetics for dykes and flood protection dams.

Geosynthetics have proved successful for emergency and temporary measures as well as for permanent purposes – and for contingency plans (successful dyke/dam defending in case of severe floods). Furthermore, geosynthetic sensor mats serve for monitoring of critical zones.

1.5.1 Emergency or temporary measures

Placing filter stable counterweights (berms) can prevent hydraulic failure of the dyke / dam by seepage, uplift, or by internal erosion and piping. Sandbags are preferred for local stabilization, and sheets of filter geotextiles (covered with sand, gravel or other granular material) for larger critical zones (Figs. 9, 10). Water outflow must not be prevented, as it would create excessive pore-water pressures and favour sudden failure. Consequently, placing impermeable geomembranes or plugging weak spots with clay at the land side is counterproductive (though it is in many cases the emotional reaction of dyke/dam defending persons).

Mobile flood protective systems may be stiff panels inserted in fixed toe elements and other modular systems of metal, reinforced concrete or synthetic material. Geosynthetics are a promising alternative whereby semi-permeable and impermeable systems have proved successful. Geotextile container solutions show a great versatility: Hand bags, big bags, containers, tubes and mattresses. Many of them are also used for permanent purposes (Saathoff et al., 2007).

Pre-filled sandbags, which can be easily carried and placed by one person, are frequently preferred to bigger geotextile sand containers. They have been successfully used for decades already. However, the better the access and placement conditions for technical equipment, the bigger containers may be used. Finally, if there is no access to critical zones on land big bags can be placed by helicopter.

In all cases the geotextile should have high robustness (puncture resistance), high elongation behaviour and interface friction. Flexible behaviour allows the geotextile container to mould itself in with the existing features and also allows a certain degree of self-healing of the structure. Unavoidable damage from driftwood etc. is also minimized then.

Figure 9. Hydraulic failure prevention by covering local water outflow with permeable sandbags (geotextile hand bags).

Figure 10. Hydraulic failure prevention (emergency measure) by covering the entire dam slope and berm with gravel placed on a filter geotextile.

The permeability of structures made of geotextile sand containers depends primarily on the size of the container elements and the method of placement. The grain size distribution of the infill is negligible. Recio and Oumeraci (2008) showed that the flow through the structure is solely governed by the gaps between neighbouring containers and that the flow through the sand fill in the containers can be neglected. Figure 11 compares two models (out of eleven) providing the worst and best results. The permeability coefficients obtained are in the order of 5 x 10-2 m/s and 7 x 10-3 m/s respectively despite a rather similar width of the structures. Furthermore, placement of the containers such that the contact areas among containers were maximized resulted in the highest stability against wave action.

Figure 12 shows a modular system of metallic grid-boxes lined with non-woven geotextiles and filled with sand. Such elements are preferably used along roads and in settlements, cities. By placing side-by-side and one on top of the other massive and high barrier structures can be constructed. Water discharge is widely negligible, roughly like through a wall of closely placed geotextile sandbags. Synthetic impermeable tubes (usually of PVC) are easily to transport. They are connected by straps, then blown up with air, moved in a fixed position and filled with water (Fig. 13). Geotextile tubes to mitigate the wave-breaking process in meandering rivers or to fill broken sections of dams are less used. But they have proved very suitable as lowcrested submerged structures to reduce the incident wave energy on shorelines.

Figure 12. Mobile flood protection system with metallic gridboxes lined with non-woven geotextiles and filled with sand. Structure finally withstood a 100-year flooding event.

Figure 13. Mobile flood protection system with synthetic tubes.

1.5.2 Permanent measures for dykes and flood protective dams

Figure 14 gives a schematic selection of geosynthetics application for dykes and flood protective dams. Not included are geosynthetics for overflow (spillway) sections, for vertical cut-off walls, for geotextile sand containers and tubes, etc. The latter are used for erosion control, scour protection, scour fill in, groynes and breakwaters. Experiences gained in coastal engineering can be widely adapted to rivers. There are, of course, certain differences between constructing new dykes and flood protective dams or repairing/refurbishing old structures. Nevertheless the basic design requirements are widely the same.

Dykes or dams fully reinforced with geosynthetics, as indicated in Figure 14 are rather exceptional cases, for instance along the outer bank of a river or at a local narrowing of the cross-sectional flow. Transition zones from rigid structures (e. g. culverts) to the flexible dam fill and overflow sections may also exhibit geosynthetic reinforcement.

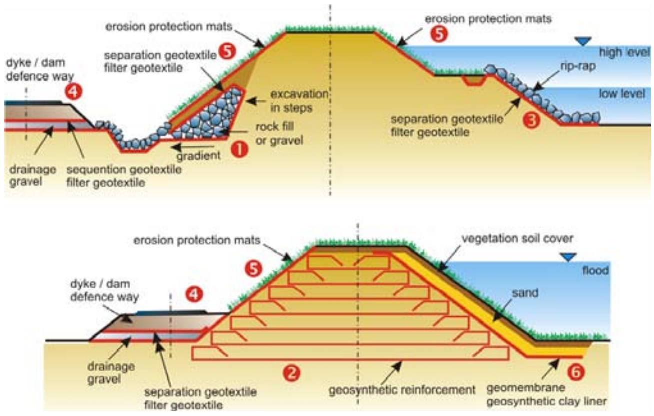


Figure 14. Application of geosynthetics for dykes and flood protective dams. Schematic examples, not to scale.
1 = filter geotextile for drainage fill
2 = geosynthetic reinforced dyke/dam
3 = filter geosynthetics below rock fill or rip-rap
4 = geosynthetics for the access way for dyke/dam defence during floods
5 = geosynthetics for erosion protection
6 = geomembrane or geosynthetic clay liner

Figure 15 shows versatile application of geosynthetics for the rehabilitation of a section of the dyke at the river Elbe after the great flood in 2002. Slope sealing at the waterside of the dyke was widely used there, because geomembranes or geosynthetic clay liners can be placed very quickly.

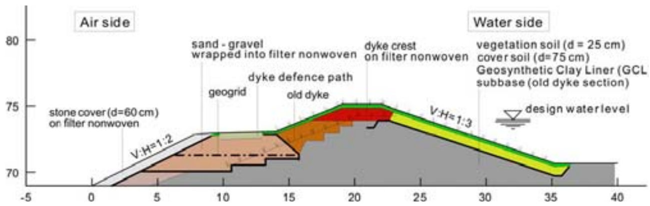


Figure 15. Dyke remediation in a section along the river Elbe (Heerten 2006).

River bank and canal slope protection has also an influence on possible flood damages. Solutions with geosynthetics generally have lower construction and lifetime costs than rigid structures.

To sum up, geosynthetics serve the following purposes for dykes and flood protective dams:

- Horizontal, vertical and inclined filters and separationelements;
- Dyke slope sealing at the water-side;
- Reinforcement of fill, crest zone, access road (for dyke defence);
- Surface erosion protection;
- Vertical cut-off walls;
- Protection against voles and beavers.

Inclined clay cores of flood protective dams commonly lead to a lower slope stability than vertical cores or cut-off walls. This refers especially to a quick drop of the water level causing superficial sliding of the water side slope, leaving the clay core unprotected against erosion. In the case of geomembranes or geosynthetic clay liners near-surface slips are also critical but can be repaired more easily than along softened, partially eroded clay.

Hydraulic failure may be prevented mainly by two measures on the land side of a dyke or flood protective dam:

- Installing trenches or relief columns or drainage wells.
- Filling of berms, thus displacing the possible starting point of inner erosion/piping farther away from the structure and decreasing the hydraulic gradient at this point. Furthermore, such berms should be constructed as access road for quick and easy dam defence in case of severe floods.

In many cases berms move the hydraulic problem only farther away from the dyke or dam, and retrogressive inner erosion may finally reach it in the long-term (after several floods). Boiling and internal erosion could be observed up to 20 to 50 m away from dykes/dams though they were only 3 to 6 m high. Moreover, wide berms are frequently not possible under confined space conditions. Therefore, drainage trenches are preferred then. However, trenches excavated in very soft soil collapse immediately before geotextiles and fill material can be placed. The installation of trussed retaining panels would be too expensive. These problems could be overcome by developing "relief granular columns", jacketed with a filter geotextile:

Jacketed (coated) stone or gravel columns have been installed in Austria since 1992. At first they were mainly used for drainage purposes, for instance as drainage walls to improve the stability of old flood protective earth dams. This method has significant construction advantages over conventional drainage trenches in loose or soft soil. In critical cases the coated columns are combined with other measures for dam refurbishment (Fig. 16). The drainage material (usually clean 4/32 mm, 8/32 mm or 16/32 mm – grain) is lowered by vibroflotation whereby the vibrator is wrapped with a non-woven geotextile (tied together at the toe of the vibrator).

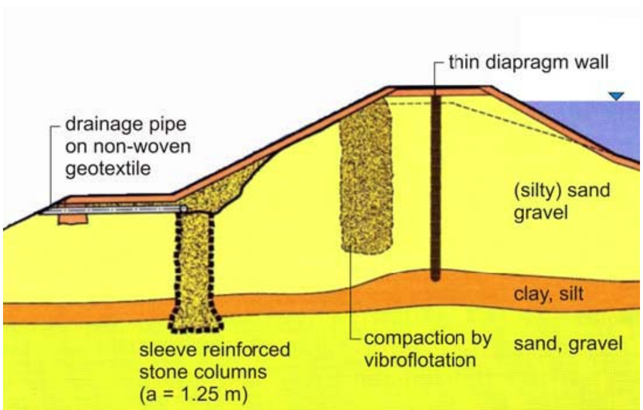


Figure 16. Improvement of static and hydraulic stability of an old flood protective dam.

The conventional top-feed process of the vibro technique is not suitable for jacketed granular columns. In this case the sophisticated vibroflotation technique with bottom-feed vibrators is required. The main advantage of this method is that the vibrator remains in the ground during installation making the technique ideal for unstable ground and high groundwater levels. The granular material is discharged from skips into the chamber at the top of the vibrator and placed at depth (Fig. 17). To avoid geotextile damage, the vibrator is sometimes at first lowered without the geotextile sleeve into the ground to displace soil. This has proved suitable in coarse or stiff subsoil but is not necessary in finegrained soft ground. A leak in the geotextile is quickly recognised due to an infill increasing volume of fill material during vibrating.

Figure 17. Installation of geotextile jacketed drainage columns (relief gravel columns) by bottom-feed vibroflotation.

Figure 18 shows an alternative for relatively short gravel columns (up to 6 m). Column excavation is performed with a continuous flight auger piling equipment. Then a steel tube with slightly greased skin and by a filter geotextile is lowered into the ground. When being withdrawn the pipe is filled with gravel, whereby the geotextile remains wrapped in the ground, thus enveloping the gravel.

Figure 18. Installation of geotextile jacketed drainage columns by inserting a sleeved steel pipe into a pre-bored pile excavation. After pipe withdrawal, the gravel-filled geotextile hose remains in the ground.

Commonly, mechanically bonded continuous filament nonwoven geotextiles of polypropylene are used. In the case of large construction sites, the coating is already prefabricated by the manufacturer and distributed in a tubular shape with needled seam fitting to the site-specific vibrator of the vibrofloatation equipment. The following geotextile characteristics have proved successful:

CBR puncture resistance 3.85 kN
Strip tensile strength 24/24 kN/m
Elongation at max. load 80/40 kN/m
Cone drop test (hole Ø) 15 mm
Mass 325 g/m^2
Thickness at 2 kN/m^2 2.5 mm

An optimized design of position, spacing, depth, diameter and hydraulic capacity of geotextile jacketed relief gravel columns requires numerical modelling for three-dimensional unsteady flow conditions. Multi-layered soil systems always exhibit permeability coefficients different in horizontal and vertical direction. Therefore, numerical models and calculations should be calibrated by site measurements/observations along previous projects or by test fields.

The top of relief columns should be covered with coarse drainage material, wrapped in filter geotextiles (Figs. 19, 20) for longitudinal or/and transversal drainage. This drainage layer should carry an access road for easy dam defence in case of severe floods.

Figure 19. Top of geotextile jacketed drainage columns embedded in coarse drainage layer as sub-base of the access road for quick dam defense in case of severe floods.

Figure 20 shows the cross section through a new flood protection dam after removing the old one that had been destroyed by a severe flood. The coated gravel columns (Ø = 0.7 m) usually exhibit a spacing between 1.5 to 7.0 m depending on local factors (geotechnical and ecological parameters, infrastructure, risk potential etc.); spacing is commonly about 4 m. The water-side dam slope is covered by a protective net against beavers (see Chapter 7).

In many regions crest levels of dykes and flood protective dams need to be raised in the future. However, due to the climate change the prognoses involve several uncertainties. Therefore these structures should be adapted to allow overflow in spillway sections or at least wave overtopping. This minimizes random dyke/dam failures. The results of comprehensive field tests on sea defences subject to wave overtopping (Akkerman et. al, 2007) can be applied also for dykes/dams along rivers. Threedimensional reinforcement grids with additional soil erosion protection represent a flexible alternative to rip-rap or even stiffer structures (Fig. 21).

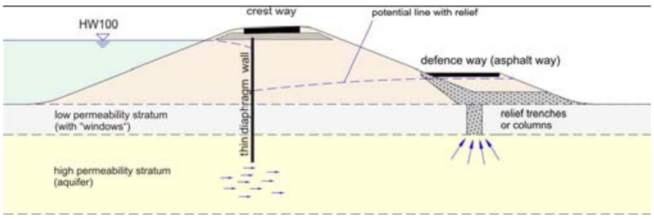


Figure 20. Standard cross section of a new 75 km long flood protective dam in Austria.

Figure 21. Stiff crest structure of a wave overtopping and spillway section of a dyke damaged by flood.

1.5.3 Prevention of seepage

From strict theory seepage through dykes or flood protective dams cannot be prevented, but practically it may be limited nearly to zero (considering evaporation etc.). This is achieved by several methods:

- "Homogeneous" dyke/dams of low permeability fill material;
- Zoned dykes/dams with clay core (vertical in the centre; inclined at the water side);
- Cut-off walls (independent of the fill material).

Geosynthetics have proved suitable as an alternative to clay cores and diaphragm cut-off walls (slurry trench walls). Geosynthetic cut-offs are inserted into the dyke/dam by vibration or by lowering them into a deep trench that is backfilled then (Fig. 22). Leak detection is possible by checking the hydraulic potential on either side of the sealing element, by thermometry, and by geophysical methods (mainly geoelectrics). Moreover innovate systems with integrated leak detectors are available.

Slurry trench walls or deep-mixing walls, however, have a higher resistance against beaver attack than geosynthetic screens, they have a statical function (though theoretically neglected for thin diaphragm walls) and may reach significantly deeper into the natural ground to reduce or prevent seepage beneath the dyke/dam. Furthermore, they have proved suitable for defined fracture sections of dams: In case of emergency it may be necessary to open the dam at particular points in order to avoid irregular failures at random zones. A domino-like progressive failure extending from the designed fracture section must be prevented. Consequently, sheet pile elements are also used for such structures.

Figure 22. Back-filled geomembrane cut-off wall for low flood protective dams.

1.5.4 Mitigation of flood damages

In Austria nearly 3000 flood protection projects are under construction, design and pre-planning until the year 2015. This requires multi-disciplinary cooperation, also considering aspects of infrastructure, of environmental protection, and local, public and legal aspects. Furthermore, plans of dam maintenance, of precaution measures and of emergency actions have been developed since the flood disasters of 2002 and 2006. Contingency plans (based on risk analyses) comprise scenarios from slight but frequent floods to worst-case events (maximum credible water levels).

Another essential prerequisite for a successful mitigation of flood effects is the comprehensive education/training of task forces comprising authorities, organizations, professional groups and volunteers. Flood wave prognoses in the case of dam overtopping or failure, and contour maps of water level and warning, alarm and evacuation plans have become standard already, and they are highly appreciated by the Austrian public.

Each new or refurbished dyke or flood protective dam will be equipped with a parallel flood defence road that makes quick access possible, so that necessary mitigating measures can be taken without delay. Materials for such measures (mainly filter geotextiles) are stored in the vicinity already now.

These concepts correspond to a judgment of the European Court for Human Rights stating the States' responsibility to protect their citizens from natural disasters to an "appropriate" (not "absolute") extent.

2 GEOSYNTHETIC REINFORCED BARRIERS AGAINST ROCKFALL – MODELL TESTS

2.1 Measures against rockfall

In mountainous regions large-scale rockfalls have become an essential factor for regional planning (Fig. 23). Not only single, well-known disasters drew the attention of the public and the authorities to extensive measures for protecting critical areas. Optimized investment comprises several questions concerning risk-analyses, like the probability of a severe event and particularly its amount of damage. Taking into account economical matters leads to a limited risk reduction in many cases of rockfall protection.

During the past years the prediction of rockfall underwent a great advancement concerning the tratjectories of the falling blocks by highly developed computer modelling, that considers mass, geometry, damping, vegetation, etc. In the end this yields the block's kinetic energy at any position. Nevertheless, in most cases the prediction of the behaviour of an unstable block itself involves uncertainties and therefore often requires a wide parameter-variation in calculation and design.

Figure 23. Severe rock fall along an expressway.

The development of structures against rockfall received an enormous boost during the last 15 years in the field of nettings, where the cost-benefit ratio could be perceptibly improved by highly absorbing deformation energy. These results were facilitated also by full-scale fall-tests, where not only further developed types of nettings but also new forms of evaluating the absorbed forces were applied.

Descoeudres (1997) cites various structures against rockfall rated according to their deformation energy (Fig. 24). Among these passive measures against rockfall earthfill barriers and especially reinforced embankment dams (barriers) take up the utmost position. In contrast to flexible nettings, the energy-absorbing effect of protective barriers is mainly given by their mass. Enough space and a topography that makes the rockfall's so called "transit-area" sufficiently known are prerequisites for the choice of an embankment barrier. Furthermore, the risk and the quantity of the expected rockfall have to justify the costly option of constructing an earth-fill barrier instead of nettings.

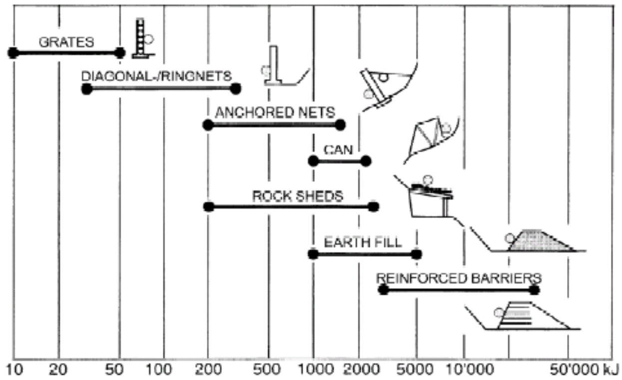


Figure 24. Structures against rockfall according to their deformation energy (Descoeudres, 1997).

As different trajectories of the falling blocks and their rotational behaviour can be predicted to a high degree by computer modelling, it seems appropriate to take this into account not only when positioning the barrier but also in the design of its upper (mountain facing) slope. Obviously, geosyntheticreinforced slopes offer more possibilities in "shaping" the embankment properly to prevent the blocks from running over the barrier. Additionally, the catch basin can be enlarged by steeply sloped barriers. Questions on the comparability of barriers with regard to their overall-resistance against dynamic impact seemed hitherto unanswered as well as economica aspects of such structures.

Full-scale tests commonly require big effort when comparing various kinds of earth-fill barriers under particular conditions. Therefore, qualitative and semi-quantitative model tests were preferred for parametric studies and to investigate the interacting factors of the geosynthetics arrangement, anchoring lengths, and degree of compaction.

2.2 Test set ups and performance

Walz (1982) describes qualitative model tests in soil mechanics primarily as a method to recognize the failure mechanism of stability problems. Starting from this "philosophy" a series of 20 dynamic 1gmodel tests were carried out on protective barriers against rockfall, scaled 1:50 (Blovsky, 2002). In the soil mechanics laboratory at the Vienna University of Technology special attention was directed on the measurement of forces, acceleration and deformations in order to gain comparable results and to enable systematic parametric studies.

In order to exactly record the dynamic impact, the rockfall was simulated by a rigid pendulum that contained dynamic force- and acceleration transducers. For exact measurements it was necessary to leave the six degrees of freedom that a single falling block has. Interpretation of the measured data was supported by deformation-gauges in the embankment and optical recording by two digital video cameras. Figure 25 shows the pendulum with its hemispherical penetration surface, dynamic force and acceleration transducers and a thread pole for additional weight.

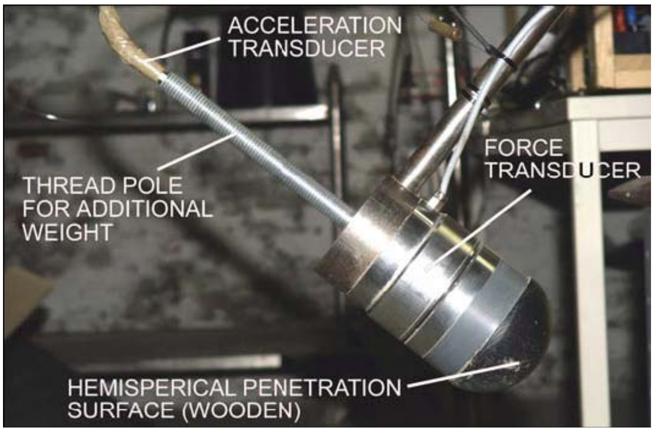


Figure 25. Pendulum simulating the rockfall.

The models were constructed in a steel frame structure with form boards. As side walls for the cross sections of the barriers served two parallel guides. To achieve a uniform density the model soil (sand with low degree of uniformity) was compacted in thin laterally boarded layers in such a way that the required soil mass could be weighed and controlled (Fig. 26a). Without the formwork for the embankment the conditions in the impact area would not have been reproducible.

To achieve a maximum slope inclination of 3:1 in the models, the soil had to be compacted at Proctor water content, thus gaining a certain amount of apparent cohesion, which of course would not exist within a non-reinforced barrier prototype for a long time. These steep slopes were carried out to compare non-reinforced and reinforced barriers.

As already mentioned, the dynamic impact was simulated by a rigid pendulum to provide an exact placement of the transducers. The pendulum was orientated horizontally in half of the barrier height at the moment of impact. To gain information about the behaviour of the barriers under that kind of load, each test contained several strokes (= "rockfalls") beginning with little impulse and increasing to a maximum impulse to destroy the model. The "loading history" of each test had to be exactly the same, to achieve full comparability. To control the increasing impulse, additional weight and/or the release height were varied (Fig. 26b). The release of the pendulum from its certain height was conducted by a steel wire and a special clamp that provided releasing almost without jerk. This method was essential, since triggering of the measurement was executed via the acceleration signal of the pendulum in order to gain sufficient data from the whole period of impact at a maximum measuring rate. Optimizing these parameters led to a possible measuring time of max 2.5 seconds at 2400 Hz dynamic rate.

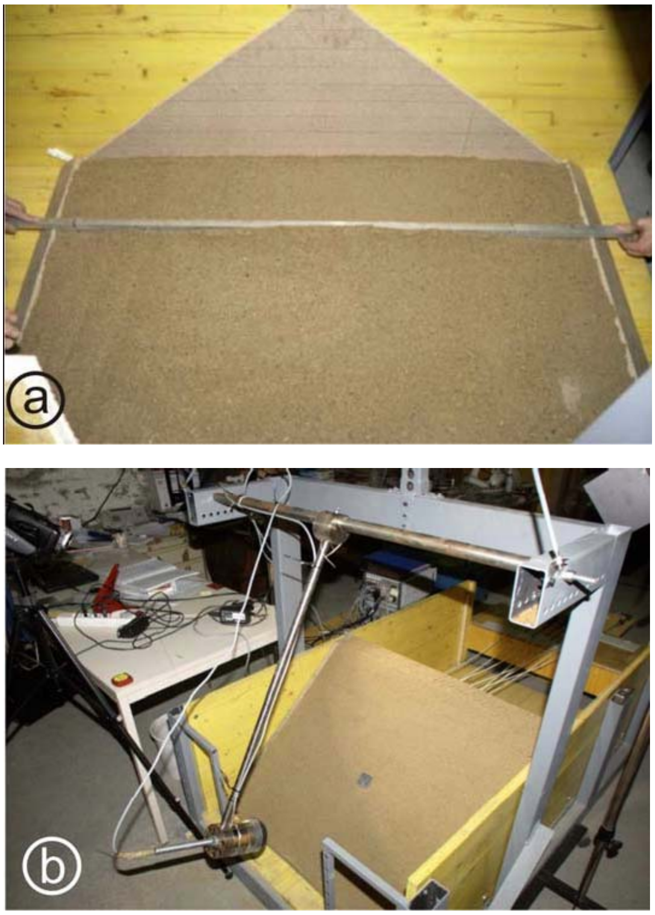


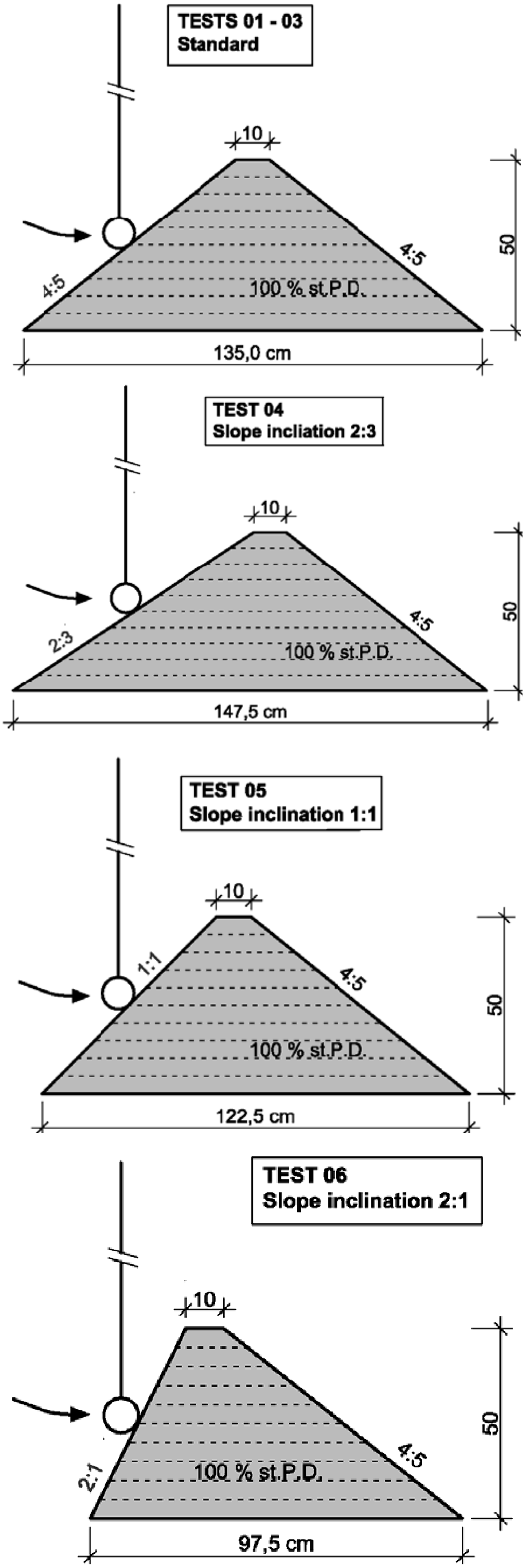
Figure 26 a, b. Construction of the "Standard" model for simulating rockfalls on barrier fill dams.

2.3 Parametric studies

The main goal of the qualitative model tests was to compare influences of geometry and compaction on the one hand, and effectiveness of geosynthetic reinforcement on the other hand. Therefore, 20 model tests were arranged as shown in Figure 27 (not reinforced) and Figure 28 (reinforced). Each group started from so-called "Standard" tests (No. 01-03 non-reinforced and No. 10 reinforced), where both slopes were inclined 4:5 and the compaction was 100 % standard Proctor density. In the following, single parameters like slope inclination, degree of compaction and the arrangement of reinforcement were varied. Combinations of changed parameters were not subject of this first series of model tests.

The first three tests were conducted under exactly the same conditions to verify reproducible results. Geometric variations included different inclinations of the uphill slope from 2:3 to 3:1 (33.7 to 71.6°).

The compaction was tested in a model with 100% standard Proctor density, one with 90% standard Proctor density and a third called "Compaction in zones", with a looser cushion (90% St.P.D), situated as "buffer" in front of a well-compacted core. All those variations were built up both reinforced and not reinforced. Concerning the arrangement



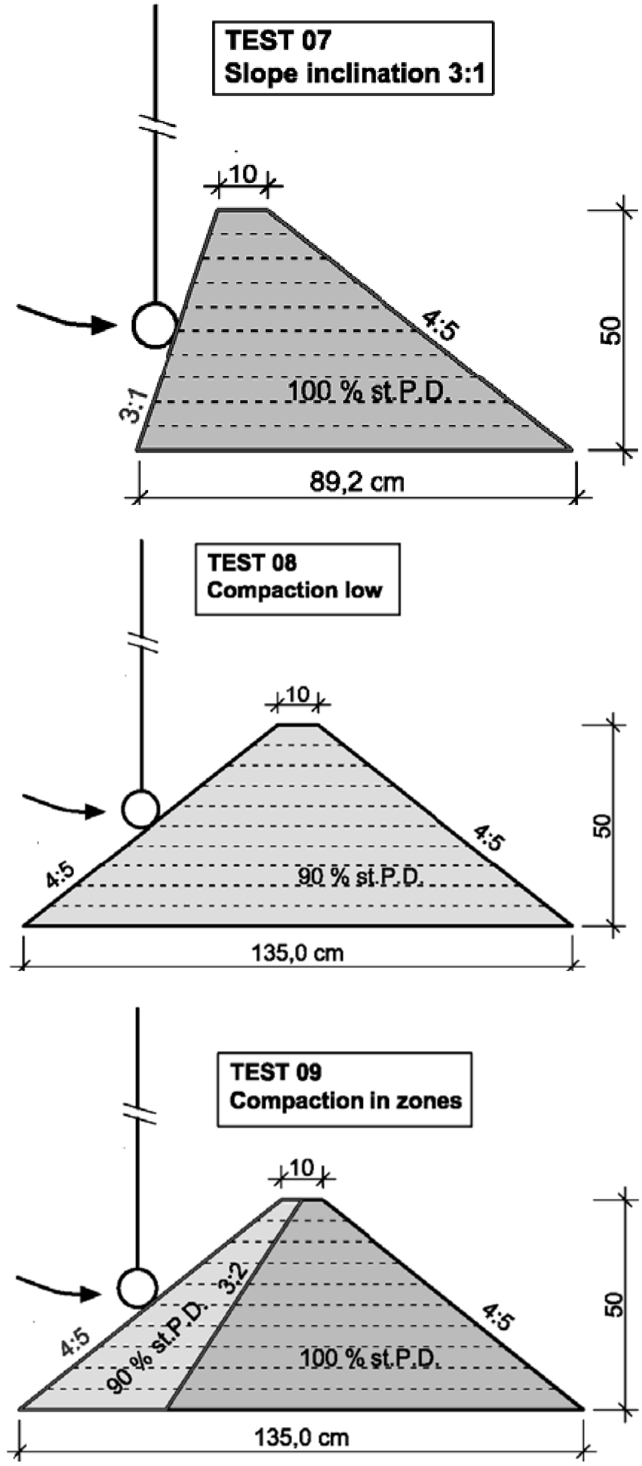


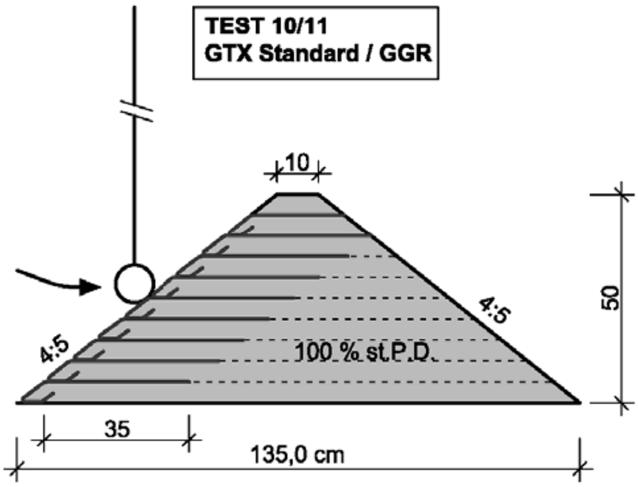
Figure 27. Cross sections of the non-reinforced barrier models.

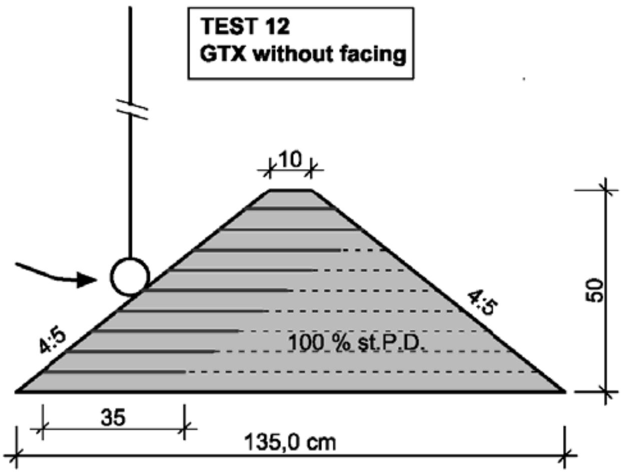
of geosynthetic elements several alternatives were investigated by varying anchoring length, sandwich- respectively compound-types and facing methods.

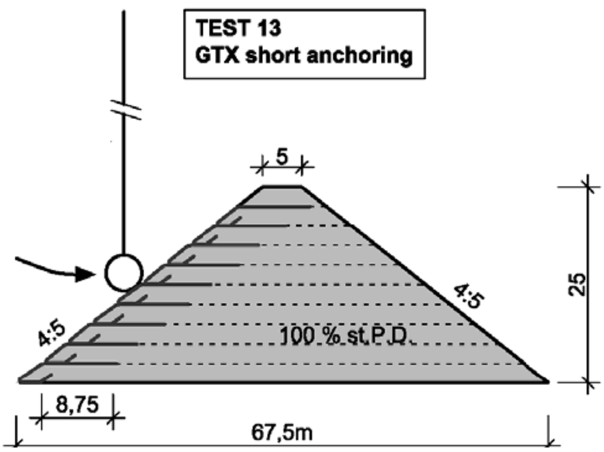
2.4 Analyses and results

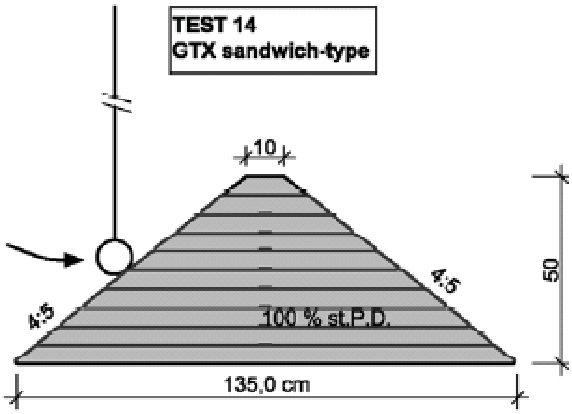
The measured data were analysed first by a detailed comparison of each registered signal (i.e. force and acceleration during impact) for the first three strokes of each test and secondly by an overall comparison, regarding every stroke until failure of the model.

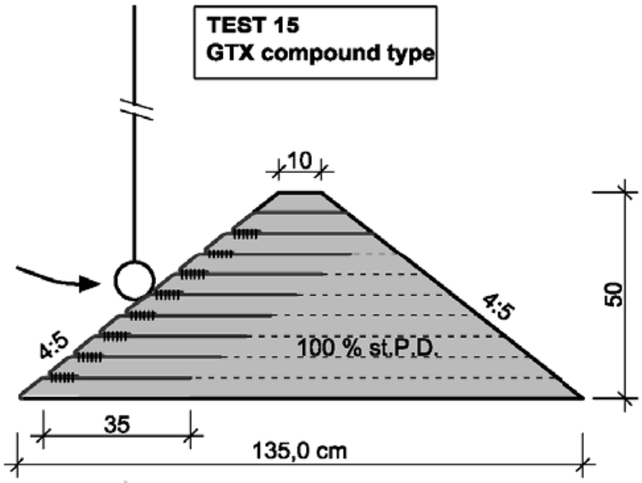
The detailed comparison facilitated to define a useful term or characteristic parameter for evaluating the resistance of earth-fill barriers: Single values like peak values of force or acceleration showed certain dependencies on the condition (local compaction) of the impact area - especially for the first "soft" impacts of each test. Therefore a combined term like impulse or energy seemed more reasonable in this manner. Figure 29 gives an example of the force-time relation for both reinforced and not reinforced standard barriers. While the long impact periods for strokes on a not reinforced barrier caused just low peak values in the force signal, the reinforced tests showed the opposite behaviour. Therefore, the area beyond the signal (the impulse) could be used to reasonably describe the resistance of the barriers.

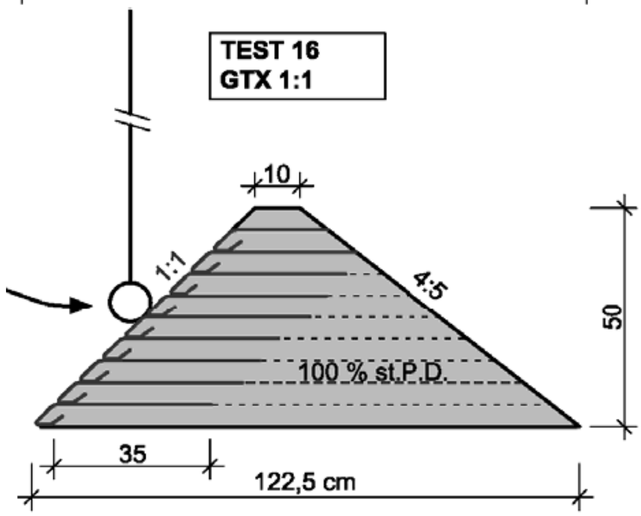


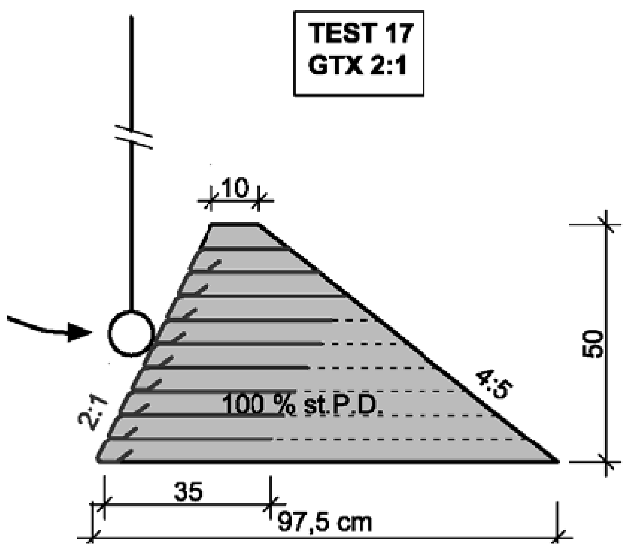












reinforced pendant from the upper camera position, as shown in Figures 30 a, b.

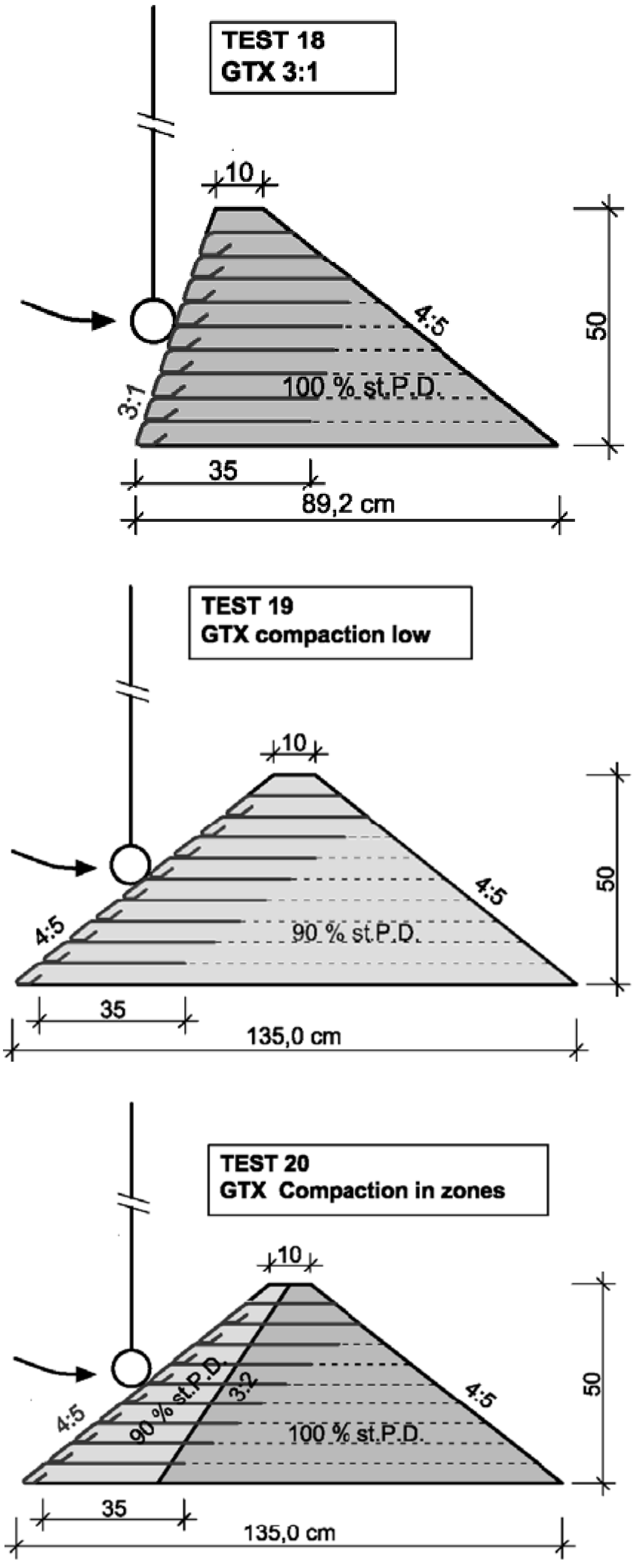


Figure 28. Cross sections of the reinforced models of barrier fill dams

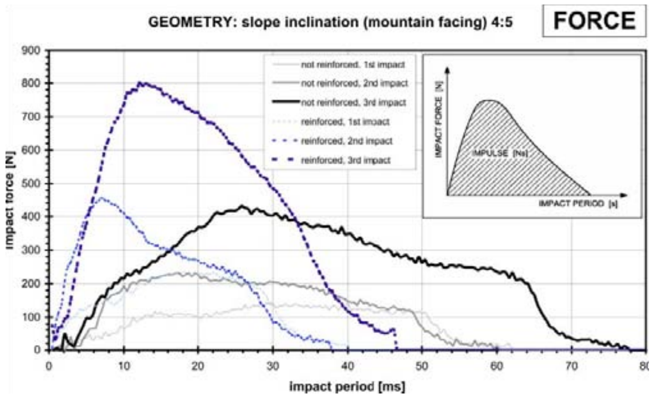


Figure 29. Force-time relation for the first three impacts (reinforced/ non-reinforced). Exemplarily.

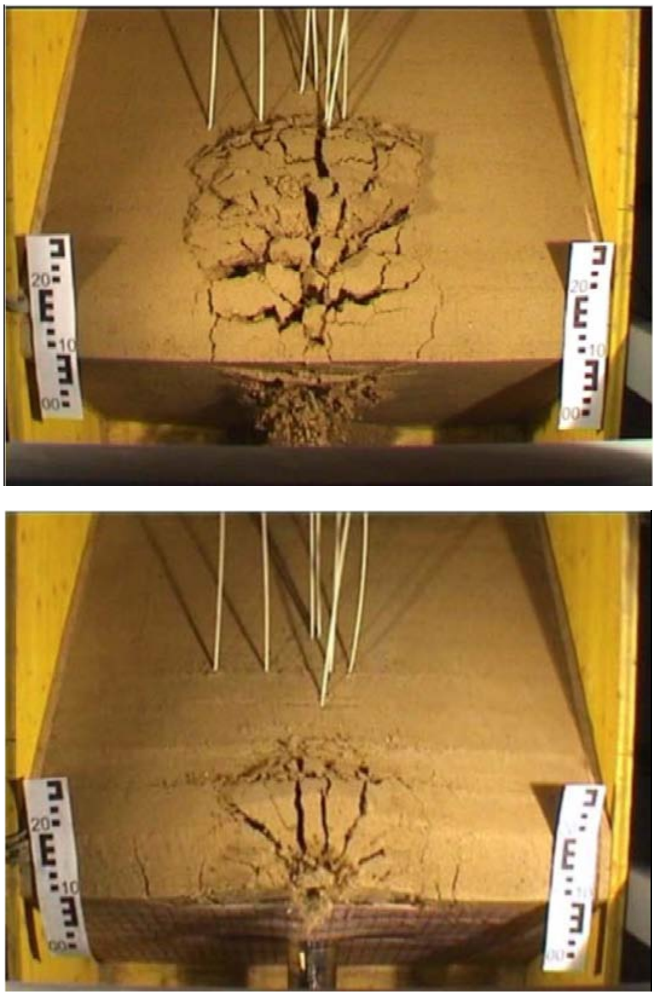


Figure 30 a, b. Cracks in a non-reinforced/reinforced model caused by the impact of the rockfall pendulum.

The evaluation disclosed that in this example the transmitted impulse of impacts on reinforced barriers reached higher values than on not reinforced ones. This result was also obtained when comparing other data from reinforcement tests. Analysing compaction-tests in detail showed that proper compaction of the barrier caused better shear resistance and consequently a higher impulse at impact.

The effects of absorbed impulse can be clearly visualized by comparing for example a reinforced model with its non-

The results of the detailed comparison were similar when evaluating not only the first three strokes but also their number which was required to completely destroy the model. First, the number of strokes differed for each of the tested variations, and secondly the measured and calculated data for each impact yielded different values. Figure 31 summarizes the overall sum of impulse for each tested model, whereby the value of the not reinforced standard barrier was assumed to be 100%.

Most remarkable was the general difference between reinforced and not reinforced barriers. A clear relation to the mass (geometry) could also be obtained: The steeper the slope inclination (the lower the mass), the lower impulses were registered. Lower compaction always caused lower resistance of the barrier, whereby the difference to the standard values was more significant in the case of nonreinforced models.

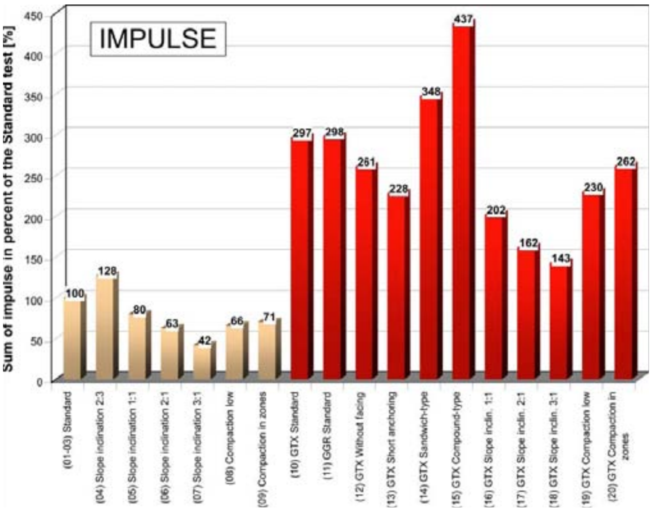


Figure 31. Sum of impulse for all tested models in percent of the non-reinforced standard test.

Two variations were clearly ahead of all others, namely the compound-type and the sandwich-type. Those models were primarily designed to take highest loads in two different ways: The sandwich-type required a lot of geosynthetic reinforcement to cover each layer completely, the compound-type used a relatively small area at the facing, where upper and lower end of the geosynthetics were bonded.

2.5 Economical efficiency

To include economic matters, cost calculations for the corresponding prototypes of the model barriers were carried out. The following assumptions simulated a particular project:

Barrier length of 200 m, constructed in 0.5 m thick layers;

- Fictitious construction time of 7 months;
- 0.3 m overlapping of the geosynthetic reinforcement;
- 2 km transport distance for the fill material;
- No direct passing over the reinforcement by skip lorries;
- Wedge of humus for a vegetated facing of the reinforced barrier zone.

In the case of reinforced structures, obstructions between filling and reinforcing working teams on the site were taken into account: Due to the barrier's geometry, filling with constant efficiency yields increasing headway of the filling-crew. To maintain sufficient utilization of the expensive filling machinery, the reinforcement-crew was accordingly resized. Even so, a certain loss of efficiency had to be considered when filling the top (narrow) layers.

Figure 32 shows clear relations between volume and cost as well as an about 40% higher cost level for reinforced barriers compared to non-reinforced ones. Filling without high quality compaction does not yield significant lower cost, due to the fact that the vibrating roller is neither an expensive equipment nor on the critical path regarding efficiency. The remarkable higher level of the sandwich-type barrier results from the higher amount of reinforcement in all fill layers.

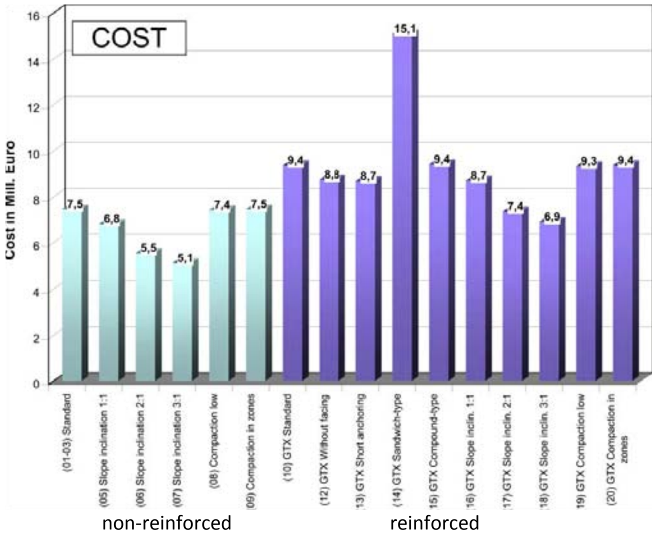


Figure 32. Calculated cost in millions of euros.

To outline the economical efficiency of the barrier variations, a value-cost ratio was determined finally by dividing the sum of impulse by the calculated cost. Figure 33 shows the results:

The generally higher impulse/cost level of reinforced structures underlines, that geosynthetics reinforcement provides more effectiveness than additional expenses. Bonding layers for a "compound facing" provides a much higher resistance against dynamic impact at comparatively low cost. The effectiveness of sandwich-type barriers is relatively small due to the very high amount of reinforcement; its value ranges even behind the 1:1 inclined variation with a lower mass. The low-compacted barriers (test-no. 08 and 09) rank at last but two and last but one position. From the detailed comparison and video analyses it can be concluded that obviously a lack of shear resistance causes the poor results for these barrier types.

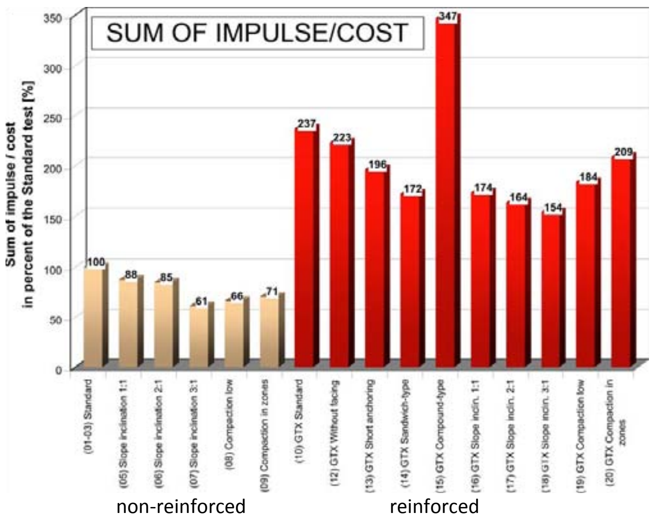


Figure 33. Value-cost ratio in percent of the non-reinforced standard test.

2.6 Practical conclusions from the model tests

20 qualitative model tests on rockfall protection barriers provided detailed information about their deformation- and failure behaviour. The effects of reinforcement, compaction degree and geometry/mass could be clearly determined. From the applied measuring equipment, the exact high-

dynamic registration of the impact-process provided most significant results - supported by single frame analyses of video recordings.

To evaluate the overall-resistance of such barriers, the sum of impulse proved to be a reasonable term. The test series on different structures showed a clear advantage of reinforced barriers that could be verified also by comparative studies considering economical aspects.

Besides a wider load distribution reinforced barriers offer more possibilities for a proper design of the uphill slope and the protection against overtopping. The tests pointed out, that predominantly that layer was stretched, which was directly exposed to the impact. By creating a compound between the adjacent geosynthetic inclusions (sewing, welding, bonding, etc.) upper and lower fill-layers could be additionally activated as load distributors.

If no compound-type is used, both upper and lower anchoring length should be sufficiently dimensioned, otherwise heavy dynamic impacts would over-stretch the reinforcement.

Already a plain placement of geosynthetics without cover at the slope facing (test 12) improves the resistance of the structure significantly. If a short construction time is required, this could be a reasonable alternative or compromise respectively.

Comparing just reinforced alternatives, it has to be stated, that the filling volume/mass cannot be substituted by a special arrangement of the reinforcement (except compound-types). Nevertheless, reinforced barriers of lower mass showed a higher resistance than non-reinforced barriers with a higher mass.

Regarding the orientation of the geosynthetic strips, static and dynamic effects have to be distinguished: The best orientation from a static view is transverse to the barrier according to reinforced earth structures or retaining walls. Overlapping is not required then in the main direction of tension.

In the case of a rock impact, however, the reinforcement is strained mainly in the longitudinal direction. A transverse orientation of the geosynthetic strips would therefore be disadvantageous A compromise is an alternating placing of transversal and longitudinal reinforcement in subsequent fill layers. Moreover, an isotropic behaviour of geosynthetics would be favourable, and various kinds of connections instead of overlapping should be applied. Finally, the models tests showed that the zoned dams (e.g. test no. 09 and 20) did not perform in a better way than the homogeneous fill dams. A kind of loose "cushion zone" (buffer) would possibly cut the peaks of internal forces, but decrease the barrier's resistance against puncture or break through. If barriers are designed with an extremely slender cross section for space saving, stability against overturning and internal restraining forces of the structure decrease.

3 GEOSYNTHETIC-SOIL BARRIERS AGAINST ROCKFALL, AVALANCHES AND DEBRIS FLOWS – DESIGN AND CASE HISTORIES

3.1 Design assumptions

The design of geosynthetic-soil barriers against rockfall, avalanches and debris flow involves inherently more uncertainties than in the case of other geotechnical structures, especially, if they are situated in seismic areas (seismic aspects, however, are not discussed in this paper).

The "usual" uncertainties regarding scatter and stress-dependent change of soil parameters and interactive/composite effects between soil and geosynthetics are superimposed here by the imponderability of dynamic impacts, magnitude and frequency of disastrous events. For instance, rock blocks of several tons (Fig. 23) and fall heights of 100 m and more have a fall energy that cannot be taken by any protective structure without material plasticization and local damage. Accordingly, flexible geosynthetic reinforced barriers or covers have clear advantages over rigid protective structures. They are designed after the "semi-empirical design method based on calculated risk and contingency plans" (Brandl, 1979):

Assuming the worst ground parameters and maximum credible natural event would make it impossible to achieve theoretically required safety factors in many mountainous regions. Consequently, local damages are accepted, but the possibility to easily repair or strengthen the structure must be given. In following this design philosophy the safety factors may drop to F = 1.0 at the moment of a rock fall impact causing large deformations. Rock penetration of 1 to 2.5 m into a geosynthetic reinforced barrier dam can be repaired more easily than severely fractured reinforced concrete barriers.

The so-called "acceptable safety factor" for permanent loads must be higher, depending on "risk of failure", failure potential and other factors (similar to Chapter 4). The "acceptable safety factor", commonly used for embankment stability as well as in practically all types of structural instability, is an empirical method to ensure a certain level of risk of failure that the owner (and the public) consider as acceptable. After all, no structure is absolutely safe, and the objective of a design is to ensure a certain (acceptable) level of risk of failure rather than a certain safety factor.

Numerical modelling of rockfalls requires knowledge about the penetration depth of rock boulders into the barrier structure and about the dynamic forces resulting from the impact. Both depend on the fall height, the boulder mass, and the indentation resistance of the structure.

For barrier dams against avalanches other design assumptions are relevant than for rockfall protective dams. They are rather similar to those for barriers against mudflows or debris flows. Mass, density and front velocity at impact are dominating parameters. Values of more than 20 m/s have been observed in many cases.

Large scale tests with avalanches in the field are widely impossible due to the high risk potential. Therefore computer-aided events are simulated, thus providing valuable data for design of barrier systems. There are several types of avalanches, with widely differing dynamic characteristics: High speed dust avalanches (Fig. 34), lower speed wet snow avalanches of high density, etc. Avalanches do not create such excessive local energy peaks on the barrier facing as huge blocks of rockfall. There is rather a quasi-"uniform" load distribution. Nevertheless, high puncture resistance of the geosynthetics is required, because avalanches may include tree trunks that perform like spears.

Common material models in geotechnical engineering are suitable to describe the behaviour of soils, but hardly the complex interaction of various types of soils and geosynthetics. Therefore, reliable numerical simulations require both, adequate material modelling and identification of the involved material parameters. The scattering of material parameters in geotechnical engineering renders the parameter identification process (PI) a challenging task (Pichler, 2003). Due to the non-linear behaviour of the shear parameters of soil reinforced with geosynthetics it is recommended to determine the parameters under similar conditions as in-situ (stress level, compaction degree).

Many soils exhibit a decrease of shear strength with increasing shear deformation until the residual value (φ_r; c_r = 0) is reached. This may occur also along soil-geosynthetic interfaces. Impacts from avalanches and especially from rockfall are very short. Consequently, despite possibly large deformations of barrier structures the calculation may be based on design shear values of

$$\varphi_{res} < \varphi_{designs} < \varphi_{peak} \tag{1}$$

In most cases a design value close to the peak value is tolerable, supposed that the decrease of the friction angle and residual shear angle with increasing normal stress is considered. This refers to soils as well as to geosynthetic interlayers and interface friction, respectively.

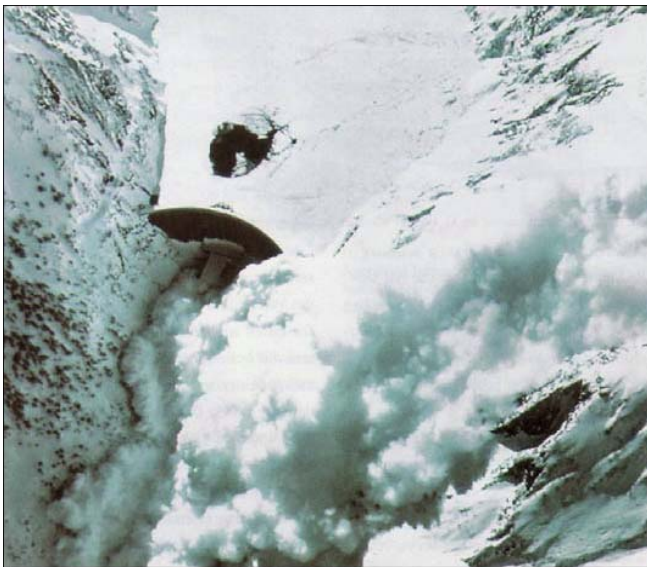


Figure 34. Dust avalanche near a high concrete dam.

Usually, the uphill slope of barrier dams should be clearly steeper than the downhill one. This provides a larger catch area and reduces the risk of crest overrunning by rock blocks, snow or debris. Additionally, catch fences are installed on most barrier dams. This causes strong stress constraints at the crest, requiring particular geosynthetic reinforcement. The fences themselves are hardly of synthetic material but of high-tensile steel wire (coated nets and meshes, etc.). Presently, the maximum retention capacity of fences is 5,000 kJ. Synthetic fences are only used on top of critical mountain zones to already avoid the formation of avalanches (Fig. 35).

Figure 35. Protective fences to prevent the formation of avalanches already on top of a mountain.

Figure 36 shows different applications for reinforcing steep soil dams and embankments. Usually, geosynthetic reinforcement exhibits equal spacing and length of the geosynthetic inclusions (a). An irregular spacing pattern reflects those cases where stresses are to be expected higher on the top than in the lower regions (b). Short edge strips (secondary reinforcement) represent a surface protection if the spacing of the geosynthetic layers is large (c). A similar effect as in the case (b) can be achieved by keeping the layers equally spaced but varying the length (d); short facing layers serve as surface protection and, furthermore, they facilitate surface compaction to achieve high compaction at the edge of the slope.

The focus in the design of geosynthetic reinforced dams, embankments or barrier structures lies on internal and external stability considering the sitespecific aspects like surface and facing details.

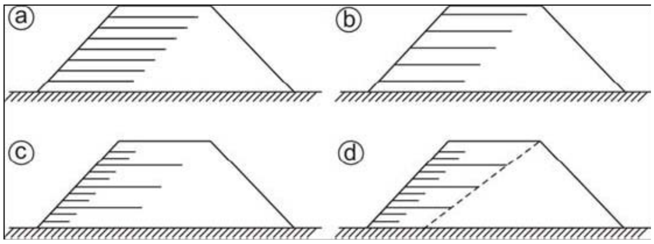


Figure 36. Various geotextile deployment schemes for stabilising steep soil dams. (a) Evenly spaced, same length. (b) Unevenly spaced, same length. (c) Evenly spaced, same length with short facing layers. (d) Evenly spaced, different length with short facing layers (after Koerner 1998).

3.2 General stability considerations

There are different possibilities to design geosynthetic reinforced barrier dams. In this chapter a sophisticated method is developed. It was derived from conventional calculation methods of non-reinforced barrier structures (Brandl, Adam, 2000). The design progresses in steps, as follows:

- Internal stability is first addressed to determine geosynthetic spacing, geosynthetic length, and overlapping. Geometry, surcharge loads, soil parameters, like angle of internal friction and cohesion, geosynthetic parameters, and interaction parameters like adhesion between soil and geosynthetic are therefore taken into account.

- External stability calculations against global slope failure, sliding and base failure (especially on soft soil) have to be carried out in the next step.

- Furthermore, a transition from internal to external slope stability has to be considered. The usual geotechnical engineering approach to slope stability problems is to use limit equilibrium concepts assuming curved or plane failure surfaces, thereby yielding an equation for the factor of safety (Figure 37). The problem can be solved using total stresses or effective stresses. Use of total stress analysis is recommended for embankments where water is not involved, or when the soil is not saturated. Effective stress analyses are preferred for conditions where water and saturated soil are involved. Equations (2) for total stress analysis and equation (3) for effective stress analysis describe limit equilibrium for a geosynthetic reinforced dam. Parameters with an overbar represent effective values while the same expressions without an overbar are total values. The factor of safety FS results:

$$FS = \frac{\sum_{i=1}^{n} \left(N_i \tan\varphi + c\,\Delta l_i\right)R + \sum_{i=1}^{m} T_{Gi}\zeta_i}{\sum_{i=1}^{n} \left(w_i \sin\theta_i\right)R} \tag{2}$$

$$FS = \frac{\sum_{i=1}^{n} \left(\overline{N}_i \tan\overline{\varphi} + \overline{c}\,\Delta l_i\right)R + \sum_{i=1}^{m} T_{Gi}\zeta_i}{\sum_{i=1}^{n} \left(w_i \sin\theta_i\right)R} \tag{3}$$

For saturated fine-grained cohesive soils whose shear strength can be estimated from undrained conditions, the problem can be simplified. Slices need not be taken, since the soil does not depend on the normal force on the shear plane then. Figure 38 shows details of this situation and results in equation (4):

$$FS = \frac{c\,L_{arc}R + \sum_{i=1}^{m} T_{Gi}\zeta_i}{W\,\eta} \tag{4}$$

- Last but not least, details referring to the surface and facing of the protective structure have to be considered. When using geosynthetics with different properties in both directions, it is important to recognise how to place the geosynthetics in an optimum way. For two dimensional cases, the maximum stress is typical in the direction of the dam face. For three dimensional cases, i.e. for local impacts, placing in the transversal direction can be more effective.

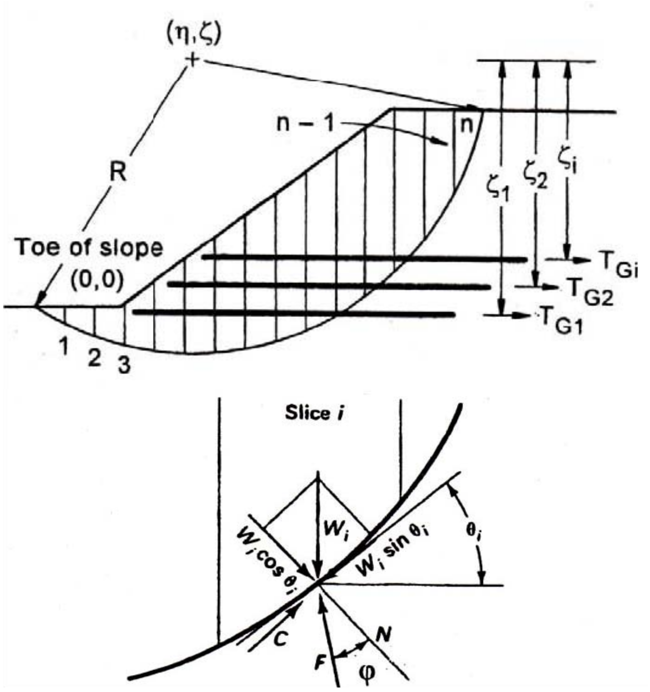


Figure 37. Details of circular arc slope stability analysis for (c, φ) shear strength soils (Koerner 1998).

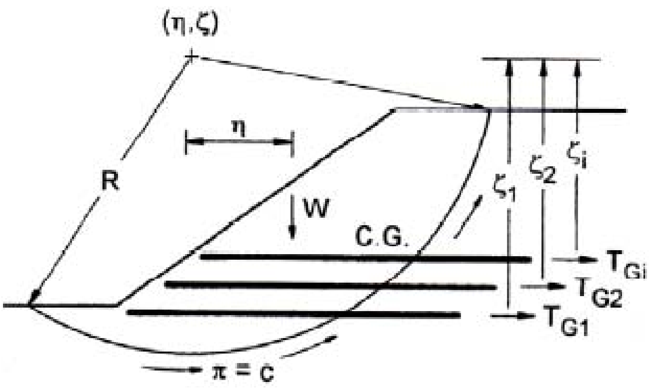


Figure 38. Details of circular arc slope stability analysis for soil strength represented by undrained conditions (Koerner 1998).

3.3 Impact load on geosynthetic reinforced barrier dams

The consideration of local dynamic impacts is essential for protective embankment dams. On the one hand it is difficult to predict the area and the magnitude of the impact force, on the other hand "exact" calculation procedures are complicated and costly. In this paper a calculation method is presented using physical simplifications and approximations. The procedure is derived from the simple case of a wedge-shaped dam cross section and can be extended to more complicated cross section shapes.

3.3.1 Idealization of fill dam structure

The dam cross section is idealized as a wedge (Fig. 39). One slice with constant width in the dam axis is considered. Due to the "stocky" shape of the vertical beam bending deformations can be neglected compared with shear deformations. Accordingly, horizontal load impacts cause primarily horizontal shear deformations and horizontal shear forces in the dam.

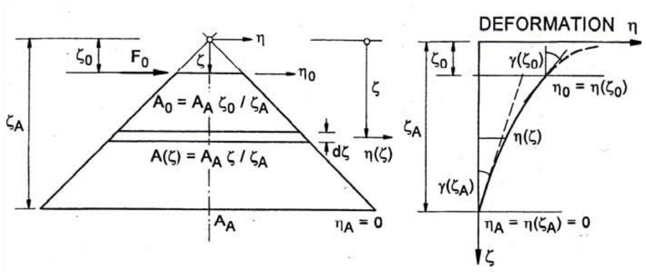


Figure 39: Idealized shear deformable dam cross section.

Considering a differential segment *A* $d\zeta$ of the shear beam (Fig. 40) the differential shear deflection $d\eta$ can be written as:

$$d\eta = \frac{F}{A'\,G}\,d\zeta = \gamma\,d\zeta \tag{5}$$

where *F* is the horizontal shear force, *G* the shear modulus of the dam and *A'* the corrected area derived from dividing the area *A* by the geometric correction coefficient κ.

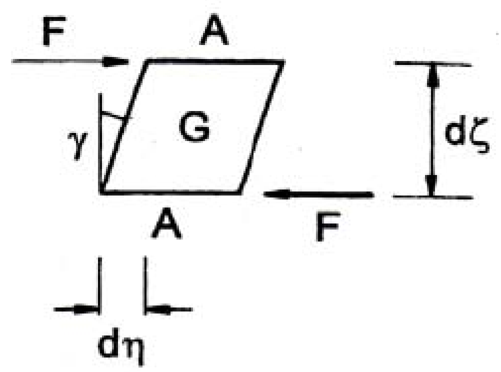


Figure 40. Infinitesimal shear deformable element.

The integration of equation (5) taking into consideration the boundary conditions as shown in Figure 39, i.e.

$$\eta(\zeta) = \int_{\zeta}^{\zeta_A} \frac{F}{A'(\zeta)\,G}\,d\zeta \tag{6}$$

results in equation (7) for the horizontal displacement according to a horizontal force on the top of the dam:

$$\eta(\zeta) = -\frac{F\,\kappa\,\zeta_A}{A_A\,G}\ln\frac{\zeta}{\zeta_A} \tag{7}$$

The displacements on the top η_o and on the foundation basis η_A are:

$$\eta(\zeta_0) = \eta_0 = -\frac{F\,\kappa\,\zeta_A}{A_A\,G}\ln\frac{\zeta_0}{\zeta_A} \tag{8a}$$

$$\eta(\zeta_A) = \eta_A = 0 \tag{8b}$$

The first derivation of equation (7) corresponds to the rotation of shear γ:

$$\eta'(\zeta) = \gamma(\zeta) = -\frac{F\,\kappa\,\zeta_A}{A_A\,G}\frac{1}{\zeta} \tag{9}$$

The described formulas represent the exact static solution of the vertical shear beam with variable area representing a "slice" of the wedge-shaped dam according to a horizontal load on the dam crown which serves as fundamental solution for the dynamic behaviour considered in the following.

3.3.2 Rayleigh-Ritz approximation method

The formulation of the dynamic continuum problem results in a set of differential equations with an infinite number of degrees of freedom. The basic differential equations of such distributed parameter systems, with associated boundary and initial conditions, even in the actual case of linear elastic solids, but with non-simple geometry, cannot be solved in an exact manner. In this case the *Rayleigh-Ritz* approximation method is used to overcome these difficulties: The essential boundary conditions are implemented into the approximation which is not a solution of the basic differential equations. The basic idea is to approximate the displacement *η*(ζ,t)* of the shear deformable dam by one function separable in space *φ(ζ)* and time *q(t)*, the so-called *Ritz* approximation (Ziegler, 1998):

$$\eta^*(\zeta, t) = q(t)\,\varphi(\zeta) \tag{10}$$

where *q(t)* is the generalized coordinate of the single degree of freedom (SDOF) equivalent system of the continuum. The function *φ(ζ)* is properly selected, in order to meet the requirements of the essential boundary conditions. The function must necessarily comply with the geometric boundary conditions and should as far as possible also take into account any dynamic boundary condition. In the actual case the function *φ(ζ)* is selected from the exact static solution derived in the section above which was determined in a sense of best fit for the dynamic problem.

The function is normalized in such a way that the deflection on top of the dam is $\varphi(\zeta_o)=1$

$$\varphi(\zeta) = \left(\ln\frac{\zeta_0}{\zeta_A}\right)^{-1}\ln\frac{\zeta}{\zeta_A} \tag{11}$$

From equation (11) the normalized rotation of shear is achieved:

$$\Phi(\zeta) = \varphi'(\zeta) = \left(\ln\frac{\zeta_0}{\zeta_A}\right)^{-1}\frac{1}{\zeta} \tag{12}$$

The approximated rotation of shear *γ*(ζ,t)* is then defined:

$$\gamma^*(\zeta, t) = q(t)\,\Phi(\zeta) \tag{13}$$

The original system can be rewritten in an equivalent system of a *Lagrange* equation of motion of a SDOF taking into account energy considerations. With the normalisation of the *Ritz* approximation, the generalised coordinate *q(t)* is well illustrated as the measure of the translational motion of an equivalent mass *m** and the kinetic energy becomes:

$$E_{kin} = \frac{1}{2}\int_{\zeta_0}^{\zeta_A}\rho\,A(\zeta)\,\dot{\eta}^{*2}(\zeta,t)\,d\zeta = \frac{1}{2}m^*\dot{q}^2(t) \tag{14}$$

From equation (14) the equivalent mass *m** can be determined by using the *Ritz* separation approximation. Integration yields:

$$m^* = \frac{\rho A_A}{4\zeta_A}\left(\ln\frac{\zeta_0}{\zeta_A}\right)^{-2}\left[\zeta_A^2 - \zeta_0^2\left(1 + 2\ln\frac{\zeta_0}{\zeta_A} - 2\left(\ln\frac{\zeta_0}{\zeta_A}\right)^2\right)\right] \tag{15}$$

The potential energy is approximated by the strain energy of an equivalent spring coefficient *k**, deforming according to equations (10) and (13):

$$E_{pot} = \frac{1}{2}\int_{\zeta_0}^{\zeta_A}G\,A'(\zeta)\,\gamma^{*2}(\zeta,t)\,d\zeta = \frac{1}{2}k^*q^2(t) \tag{16}$$

Integration of equation (20) results in the effective stiffness *k**:

$$k^* = -\frac{G\,A_A}{\kappa\,\zeta_A}\left(\ln\frac{\zeta_0}{\zeta_A}\right)^{-1} \tag{17}$$

The resulting *Lagrange* equation of motion of the idealised dam is that of the linear oscillator with the natural frequency ω_o:

$$\ddot{q}(t) + \omega_0^2\,q(t) = 0,\quad \omega_0 = \sqrt{\frac{k^*}{m^*}} \tag{18a, b}$$

The knowledge about the motion behaviour of the dam can be used for dynamic analyses, i.e. earthquake calculations. In the following, the basic solution will be used to design the dam loaded by a local dynamic impact, e.g. a severe rockfall penetrating into the dam. The dynamic incident is idealized by a punctual inelastic impact.

3.3.3 Idealized inelastic impact

Impact is a process of sudden exchange between two colliding bodies within a short time of contact. With respect to a single impacted body or structure, loading in such a process acts with high intensity during this short period of time. As a result, the initial velocity distribution is rapidly changed. Such rapid loading in the contacting area is a source where waves are emitted which propagate with finite speeds through the dam body absorbing the effective energy.

The most critical case is characterised by a horizontal hit of a rigid body at the dam crown (Figure 41). The rigid body, i.e. a rock with the mass m_S approaches the dam with the velocity v_S. A plausible assumption of the velocity distribution must be made which renders deformation in the subsequent motion over time. By considering the static deformation of the linear elastic dam under the action of a dead weight load F0 applied at the crown of the dam pointing in the same horizontal direction a compatible velocity distribution can be assumed and has the same linear distribution. Consequently, the generalized velocity after impact can directly derived from the Rayleigh-Ritz approximation defined in equation (10):

$$\dot{\eta}^*(\zeta, t) = \dot{q}(t)\,\varphi(\zeta) \tag{19}$$

In the case of utmost dissipation, it is assumed that the colliding bodies do not separate immediately after impact. The surface points of contact take on a common component of velocity in the direction of the impact at the end of the collision process:

$$\dot{\eta}'^*(\zeta_0, t) = \dot{q}'(t)\,\varphi(\zeta_0) = v'_S \tag{20a}$$

or

$$\dot{\eta}'^*_0 = \dot{q}'(t) = v'_S \tag{20b}$$

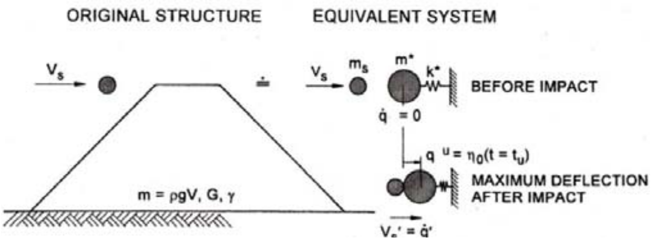


Figure 41. Original cross section of a barrier fill dam and equivalent system loaded by an idealized inelastic single body impact.

Applying the momentum relation on each partial system taking into account the condition of idealised inelastic impact (equation 10) yields the velocity to common both bodies:

$$\dot{\eta}'^*_0 = \dot{q}'(t) = v'_S = \frac{m_S}{m_S + m^*} v_S \tag{21}$$

Taking into account the conservation of energy maximum deflection $\eta_o{}^u = q^u$ of the dam can be calculated. Conservation of energy requires:

$$E'_{kin} + E'_{pot} = E^u_{kin} + E^u_{pot} \tag{22}$$

whereby it is obvious that $E'_{pot}=0$ at the moment of collision and $E^u_{kin}=0$ at the moment of maximum deflection since deformation velocity is zero. Kinetic energy at the moment of collision is a maximum and the potential energy is a maximum at the moment of maximum deflection:

$$E'_{kin} = \frac{1}{2} m^* \dot{q}'^2 + \frac{1}{2} m_S v'^2_S \tag{23}$$

$$E^u_{pot} = \frac{1}{2} k^* q^{u2} \tag{24}$$

Combining equations (23) and (24) with (22) finally yields the maximum deflection of the dam according to a horizontal impact of a rigid body:

$$\eta_0{}^{*u} = q^u = \frac{m_S v_S}{\sqrt{k^*(m_S + m^*)}} \tag{25}$$

The maximum deformation function over the total height of the dam is given by equations (7) and (10) combined with equation (25):

$$\eta^{*u}(\zeta) = q^u\,\varphi(\zeta) = \frac{m_S v_S}{\sqrt{k^*(m_S + m^*)}} \left(\ln\frac{\zeta_0}{\zeta_A}\right)^{-1} \ln\frac{\zeta}{\zeta_A} \tag{26}$$

3.3.4 Magnification factor

Due to the linear elastic behaviour of the considered system the introduction of a magnification factor is possible. Thus, a reasonable design method is provided since static calculations can be carried out in a first step and in a second step these results can be multiplied with the magnification factor χ in order to get maximum forces (stresses) and deformations (strains) from dynamic impacts.

It is recommended to perform static calculations introducing a horizontal force F_o on top of the dam with the following magnitude:

$$F_o = m_S\, g \tag{27}$$

Applying this expression to equation (22a) yields the maximum horizontal static displacement on the top of the dam:

$$\eta_0{}^{stat} = \eta_0 = -\frac{m_S\, g\, \kappa\, \zeta_A}{A_A\, G} \ln\frac{\zeta_0}{\zeta_A} \tag{28}$$

The magnification factor χ is defined by dividing the maximum dynamic deformation (equation 25) by the static deformation (equation 28):

$$\chi = \frac{\eta_0{}^{*u}}{\eta_0^{stat}} = -\frac{v_S}{\sqrt{k^*(m_S + m^*)}} \frac{A_A\, G}{g\, \kappa\, \zeta_A} \left(\ln\frac{\zeta_0}{\zeta_A}\right)^{-1} \tag{29}$$

3.3.5 Limit equilibrium design

Following the considerations above, the impact area should be designed in such a way that the impact load can be distributed to a larger dam region, so that maximum forces and deformations can be reduced in the near field of impact. Nevertheless, nonlinear material behaviour and ultimate stresses and strains should be considered in an area near the impact zone. Geosynthetic reinforcement significantly improves the stability against local failure caused by heavy impacts.

A finite section of the geosynthetic-reinforced dam according to Figure 42 is considered. The crown of the dam is horizontally loaded with the static force $m_S g$ multiplied by the magnification factor χ. Equilibrium in the horizontal direction renders that the resulting shear force is constant in every horizontal plane of the dam. The constant driving force T^S is:

$$T^S = T(\zeta) = \chi\, m_S\, g = \frac{1}{\kappa} A(\zeta)\, G \frac{d\eta(\zeta)}{d\zeta} = \text{const.} \tag{30}$$

Nevertheless, the shear stress due to the horizontal load is decreasing with increasing ζ.

In the case of failure the driving force equals or exceeds the maximum allowable shear force resulting from shear resistance of the dam in failure surfaces assumed to be vertical and horizontal surfaces creating a body shown in Figure 42. The resistance of soil body is composed of two components, geosynthetics contribute as third component:

- Shear resistance in horizontal plane $T^1_R(\zeta)$ taking into account the shear parameters φ and c:

$$T_1^R = \frac{A_0}{2}\left(1 + \frac{\zeta}{\zeta_0}\right) \rho\, g\, (\zeta - \zeta_0) \tan\varphi + A_0 \frac{\zeta}{\zeta_0} c \tag{31}$$

  The horizontal shear resistance increases with increasing depth and is a reliable shear resistance component.

- Lateral shear resistance in vertical planes $T^2_R(\zeta)$ taking into account the shear parameters φ and c:

$$T_2^R = \frac{l_A}{\zeta_A} \lambda_0 \rho g \tan\varphi \left[\frac{1}{3}\left(\zeta^3 - \zeta_0^3\right) - \frac{1}{2}\zeta_0\left(\zeta^2 - \zeta_0^2\right)\right] + \frac{l_A}{\zeta_A}\frac{c}{2}\left(\zeta^2 - \zeta_0^2\right) \tag{32}$$

The lateral shear resistance also increases with increasing depth but looses effect depending on the opening angle due to total separation of the failure body from the remaining dam at large deformations. Therefore, the lateral shear resistance force should not be considered in a limit equilibrium design. Nevertheless, it serves as a "hidden safety" covering heterogeneity and local lower shear parameters.

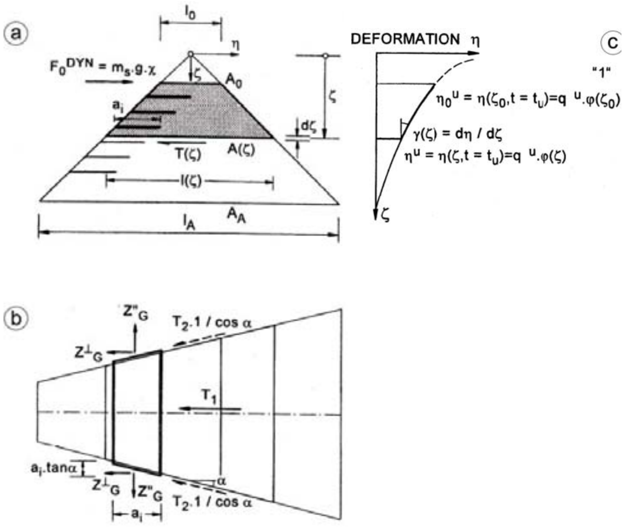


Figure 42. Limit equlibrium design of a geosynthetic reinforced protection dam. The failure body is assumed to be wedge shaped. (a) Cross section, (b) Ground view, (c) Maximum deformation curve.

- Geosynthetic reinforcement provides a significant increase of resistance against shear failure. It serves as a "load distributor", thus, the affected soil body absorbing the impact energy can be assumed to be significantly larger. Depending on their stress-strain characteristics geosynthetic inclusions are mobilised in different states of impact loading of the dam. Linear tensile stressstrain behaviour of the geosynthetics is recommended, whereby the resulting secant stiffness should be in the range of the elastic modulus of soil to take into account the strain compatibility between soil and geosynthetics. In this case, soil and geosynthetics would be mobilised simultaneously. Furthermore, different stress-strain properties and ultimate tensile strength have to be considered. From Figure 42b it is obvious that the geosynthetic properties should be equal in both directions in the case of a local impact. If there are different properties the design must be based on the minimum tensile strength. Consequently, the geosynthetic resistance force T_G^R can be easily calculated:

$$T_G^R = a_i \tan\alpha \min\left|Z_G^{normal}, Z_G^{parallel}\right| \quad (33)$$

A limit equilibrium design requires a clear definition of the safety factor. In the case of a geosynthetic reinforced dam it is proposed to use partial factors of safety γ^S and γ^R. Ultimate strength values can be adapted to allowable values for the design as follows:

$$T^S \gamma^S \le \frac{1}{\gamma^R}\left[T_1^R\left(+2\,T_2^R\right)+2\,T_G^R\right] \quad (34)$$

In Figure 43 the driving forces T^S due to a dynamic impact on the dam crest and the resisting forces T_1^R are shown in dependence of the dam height. In the top area the safety requirements are not met, thus, a geosynthetic reinforcement must be installed. The design strength of the geosynthetics can be determined exactly for each dam region. It is obvious that the geosynthetic reinforcement must be concentrated in the top region of the dam, both, length and spacing can be varied or geosynthetics with higher tensile strength can be applied.

When a barrier dam is impacted on a locally limited area, primary shear forces are produced which may possibly cause failure. Furthermore, in a three dimensional consideration the dam can also be affected by a global bending moment due to a local impact, especially in the upper regions of the barrier. In an earthfill and a rockfill dam tensile and flexural stresses cannot be taken, so that failure may occur due to this kind of load. Geosynthetic inclusions with high ultimate tensile force placed in the zone of the dam slope lead to a distinctive increase in the factor of safety. Compound between geosynthetics and soil is essential.

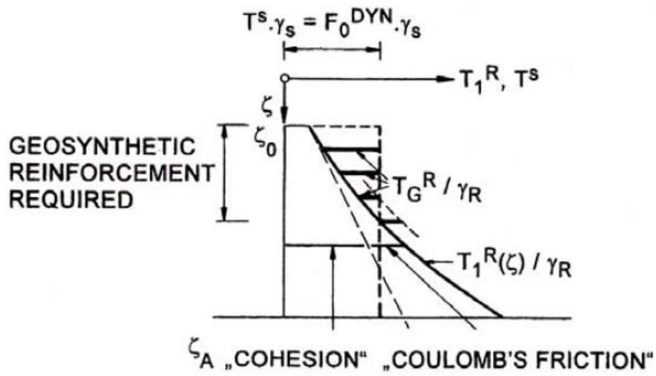


Figure 43. Limit equilibrium design. Driving forces T^S versus resisting forces T_1^R.

Geogrids, geotextiles, geonets and geocomposits are suitable to achieve practicable and reliable solutions. Furthermore, high friction between geosynthetics and soil is required in order to transfer the tensile forces into the soil along the embedded geosynthetics. An approximate calculation can be performed to estimate the additional bending moment taken by the geosynthetic layer. Therefore, a finite section of the dam is considered shown in Figure 44. The bending moment can easily be calculated:

$$\Delta M(\zeta) = Z_G^{parallel}\delta(\zeta) \quad (35)$$

whereby $\delta(\zeta)$ must be estimated, the dam pressure ΔP_{DAM} can be approximated by using earth pressure considerations taking into account the geometry and the overburden load.

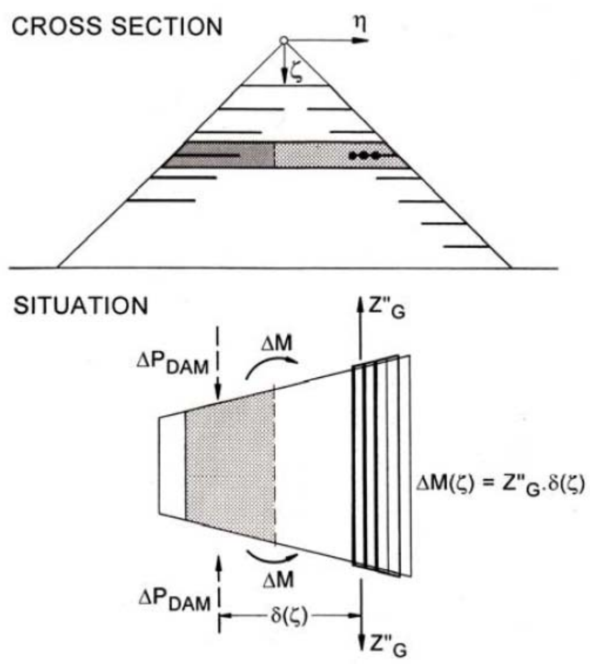


Figure 44. Limit equilibrium design. Flexural stresses from bending moment covered by geosynthetic reinforcement embedded in the outer zones of the dam slopes. The maximum reinforcement tensile force depends on the ultimate dam pressure ΔP_{DAM}.

3.4 Design of geosynthetic-soil barrier dams

Geosynthetics reinforcement of a protective dam improves the resistance of the structure significantly. Slope angles can be clearly steeper than for nonreinforced barriers. Furthermore, long term surface protection is provided, the facing can be designed in various alternatives. An essential advantage is achieved by the load distributing effect of the geosynthetics, so that the danger of local damages and overall failure decreases significantly. The impacted energy is transferred to a larger dam area then, stress and strain peaks are reduced.

Consequently, the design process involves a modification to conventional calculation procedures. As shown in the previous chapters the design comprises two steps with two different kinds of loads:

- Continuous dead loads and "quasi-static" life loads;
- Local heavy dynamic impact loads.

In Figure 45 an example of a geosynthetic reinforced protection dam is shown, especially designed to restrain rock falls and other impacts. Geosynthetics are applied in the dam to meet following requirements:

- Increase of the slope angles (1), (2), (3);
- Global stability of dam (1);
- Distribution of impacted load (1), (3) ;
- Stability against local impacts in the top region (2);
- Steep slope stabilisation (3);
- Facing, surface shaping, and surface compaction aid (1), (3), (4), (5);
- Heavy flexural reinforcement covering impacts (4);
- Light flexural reinforcement covering impacts (5);
- Reinforcement of weak and/or unstable foundation soils (6).

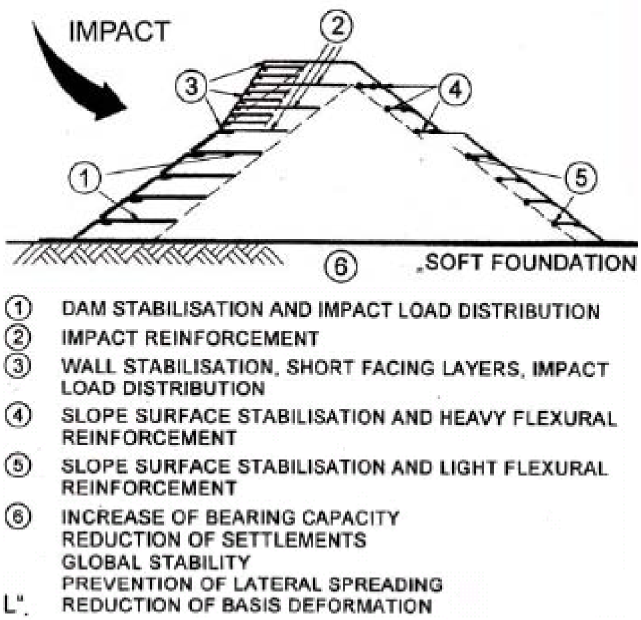


Figure 45. Example of a geosynthetic reinforced protection dam. Effect of different geosynthetic barrier or reinforcement layers.

The mass distribution of the dam regarding impact resistance can be improved significantly by installing a geosynthetic reinforcement: It facilitates the placement of relatively more mass in the top zone than in the lower zone of the protective body. Furthermore, the bottom area of the dam can be reduced to a minimum while the dam volume is kept constant serving as an energy absorber.

If the topography allows various ground plans the most effective dam shape is that of a convex curvature. Load is diverted to the abutments by compression in the dam body arch-like (Fig. 46a) and to the dam basis respectively. In dams with a straight axis and especially in concave curved dams tensile stresses can be caused by horizontal impacts which can be taken only by a tensile reinforcement embedded in the outer slope of the dam (Fig. 46b).

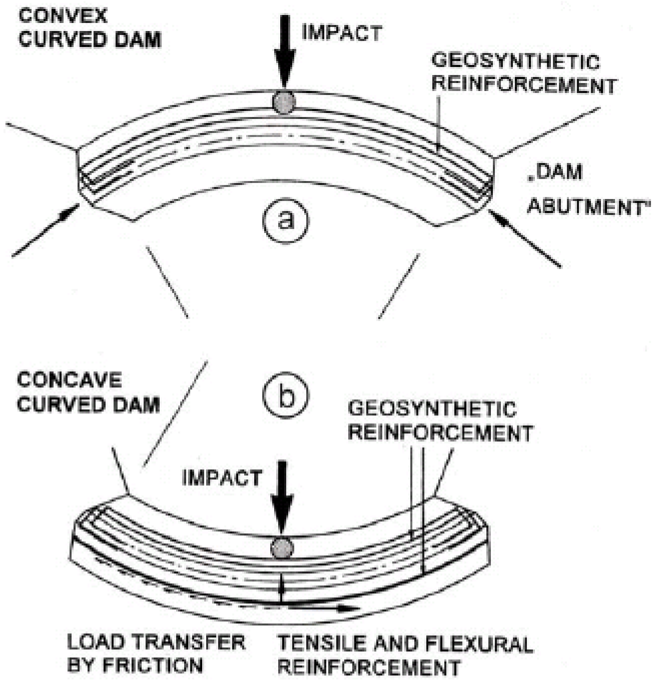


Figure 46. Different ground plans of geosynthetic reinforced fill barriers (protective dams).
(a) Convex curved dam, (b) Concave curved dam.

The height of such protective structures is more or less unlimited. Depending on the sub-soil conditions heights up to an order of 100 m (or even more) are throughout possible.

3.5 Case histories

In 1999 a severe rockfall occurred in Tyrol/Austria that required the evacuation of about 300 persons due to expected further rockfalls of about 300 000 m3 or even more (1 million m3). The protective measures comprised two reinforced geosynthetic barrier dams, the bigger one with a height of 25 m and a crest length of 170 m. The fill material was partly gained by excavating an uphill catch basin (Fig. 47) and had to be placed within two months, always under the risk of further rockfalls. Downslope of the barriers the natural slope steepened, thus requiring detailed slope stability analyses and future monitoring. Figure 48 shows the top zone of the barrier dam, which exhibited strong geosynthetic reinforcement, also for inserting a catch fence:

Continuous filament mechanically bonded (needle punched) nonwoven geotextile of polypropylene, strengthened with high-strength polyester yarns, whereby the orientation of its reinforcement is bidirectional. The main geosynthetic characteristics are:

Tensile strength	10 kN/m
Elongation at break	13 %
Tensile strength at 5 %	30 kN/m

Long-term design strength (FS creep = 120 years)	31.3 kN/m
Thickness	3.0 mm
Mass	580 g/m2

Figure 47. Rockfall barrier (geosynthetic renforced earth dam) with upslope catch basin for 300 000 m³.

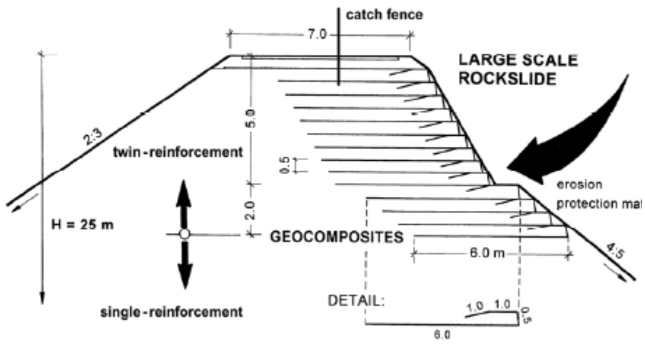


Figure 48. Protective embankment dam against large scale rockfalls and landslides. Soil reinforcement on upper part with geocomposites; slope cover with humus and geosynthetic erosion mat.

The compaction of the barrier dams was optimized and controlled by roller-integrated continuous compaction control (CCC – see Chapter 5). Finally, erosion protection of the barrier slopes was achieved by placing a three-dimensional mat of extruded polypropylene monofilaments strengthened by an incorporated geogrid.

Figure 49 shows a geosynthetic-soil protective embankment dam against large-scale avalanches in the Austrian mountains. The upper part of the structure exhibits a modular (segmental) block scheme consisting of geosynthetic loops. This system acts like a composite body according to the "deadman" principle, whereby friction along the anchor elements is by far less important than in the case of conventionally reinforced soil structures. Consequently, numerous site measurements and observations have disclosed that the internal stability of loop anchored structures is actually higher than assessed by conventional calculation. Such walls can be idealised as truss-like structures, whereby the loops are considered truss elements under tension, and the soil between the loops and modular units represents the truss elements under compression. Another calculation method is similar to that of cofferdams.

The protective dam/structure of Figure 48 was constructed in 1981 and has withstood extreme impacts since. Long-term monitoring has confirmed excellent behaviour:

The worldwide largest protective fill barrier against avalanches has been recently finished. It is a 600 m long and 27 m high earth dam of 500 000 m3 in Tyrol, Austria, reinforced with geosynthetics (Fig. 50): Geogrids of high-tensile strength polyester yarns with polymer coating. Their quality was adapted to the locally prevailing statical requirements varying between

Tensile strength:
- longitudinal 58 – 168 kN/m
- transversal 30 kN/m

Elongation at break:
- longitudinal 10.5 kN/m
- transversal 25 kN/m

Mesh width:
- longitudinal 25 mm
- transversal 35 – 30 mm

Lost formwork of galvanized steel mesh was used to locally achieve a facing of 2:1, and an erosion protection grid covered the vegetation soil.

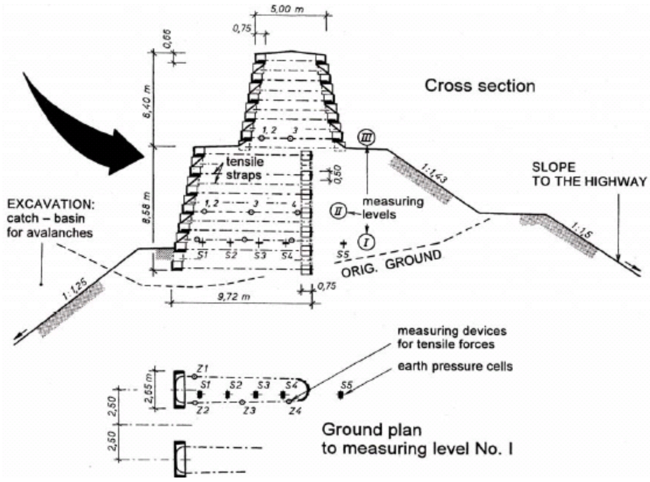


Figure 49. Protective embankment dam against large-scale avalanches and debris flows. Soil reinforcement on upper part with a loop-anchored wall system. Cross section and ground plan with measuring details.

Figure 50. Barrier dam against avalanches: 500 000 m³ L = 600 m, H = 27 m.

The design of high geosynthetic reinforced earth structures should consider that friction angle φ and residual shear angle φ_r decrease with increasing normal stress. On the other hand, under low normal stress levels higher in-situ friction angles can be observed than determined in conventional shear tests. The inclined plane test provides more realistic results for interface shear strengths, especially if differing between "sudden sliding" and "gradual sliding" (Pitanga et al. 2009).

4 STABILITY OF FLOATING EMBANKMENTS AND BARRIER DAMS IN CREEPING SLOPES

4.1 General

Floating embankments for roads and highways, and (large-scale) toe fills as stabilizing counterweight to unstable or creeping slopes represent an increasing field of geosynthetics application. Embankments along creeping slopes have become a promising alternative to bridges if they are not too high or if they are situated in the toe zone of an unstable slope (Figs. 51, 52). The deeper the creeping layer reaches and the higher the seismic activity of an area is, the more advantages gain geosynthetic-reinforced "floating" earth structures. Sometimes a combination of flexible and rigid elements may provide the optimal solution.

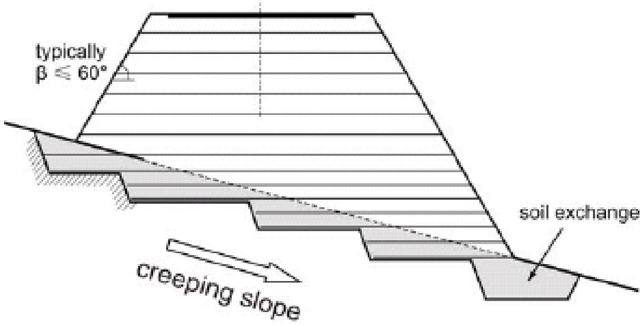


Figure 51. Floating embankment in creeping slope.

Figure 52. 60 m high geosynthetic-reinforced floating embankment in rugged steep terrain and seismic zone.

Geosynthetic reinforcement of floating embankments or other floating fills serves the following targets.

- Minimizing the embankment mass due to steep fill slopes.
- Creating a composite body that acts like a "quasi-monolith" and can move with the creeping slope without (relevant) damages.
- Providing sufficient overall stability of the embankment - natural ground - system.
- Providing high earthquake resistance.

Monitoring of such structures is therefore inevitable. On the other hand, they allow to construct in areas which commonly have been considered as too risky. Only the transition zones from creeping to stable slope sections require increased maintenance.

4.2 Dominating design parameters

The dominating parameters for designing floating embankments and barrier dams in creeping slopes are shear strength and ground water conditions.

Figure 53 shows a histogram which clearly indicates the important effect of weather on the number and magnitude of landslides in a certain region: Heavy, long lasting rainfalls in spring 1975 caused numerous, disastrous slides in some regions of Austria as never experienced during the past 150 years. They were favoured by a preceding very wet autumn and heavy snowfalls during winter which left the ground soaked like a sponge and rock joints filled with water already before heavy spring rains began. Above all, Figure 53 raises the question of "worst case design parameters" and underlines the importance of "semi-empirical design with calculated risk" based on the observational method and contingency plans. It would be rather uneconomic to construct most expensive protective structures by throughout assuming and superposing the most unfavourable parameters. In many mountainous regions this is technologically even impossible. Future additional measures – even in connection with remedial works – have proved by far less costly than a fully engineered design based on rather high theoretical factors of safety.

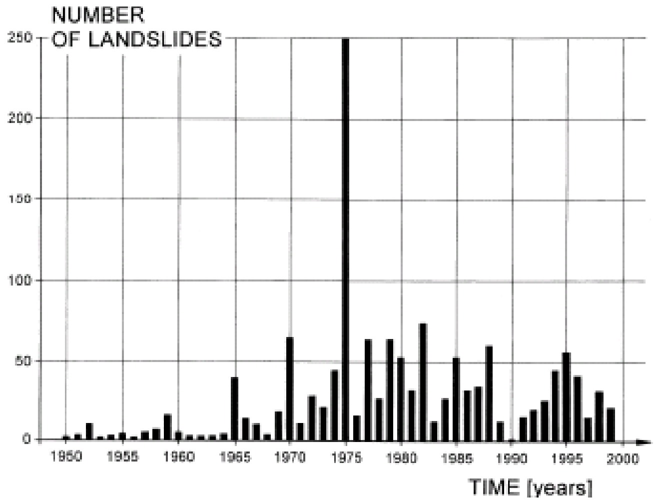


Figure 53. Landslides in Lower Austria between 1953 and 1999. Singular weather conditions in 1975 caused excessive mass movements.

Risk assessment and stability analyses of slopes should always involve the determination of the residual shear strength. This is especially important for creeping and other unstable slopes. Creeping represents a long-term process with progressively decreasing shear resistances - contrary to sudden deformations caused by rockfall, debris flow or avalanches. This deterioration depends not only on the soil parameters (mainly grain size distribution, grain shapes, mineral contents and density) but also on the degree of saturation and on the level of effective normal stress (Fig. 54).

Consequently, if the normal stress at shear tests is too small, the measured value of φ_r is not the theoretical minimum. As φ_r of soils and geosynthetics mostly decreases with increasing normal stress, the overburden should be taken into account when assessing the possible residual shear strength in the field. Deep-seated slide planes are more critical than those near to the surface.

An increasing degree of water saturation favours the tendency towards slickensides and decreasing Φ_r (Fig. 54). Therefore, shear or triaxial tests should be performed on saturated specimens to obtain the minimum value of Φ_r for lower border analyses.

The rate of displacement has an influence on the residual strength of a range of soil types, fillings and weathered

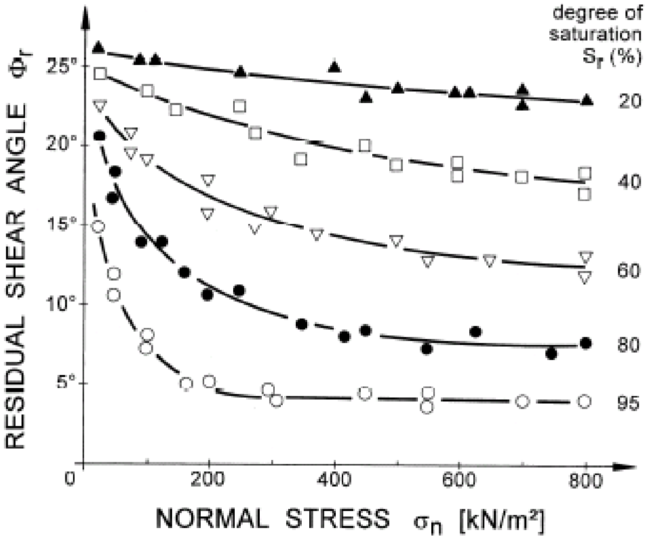


Figure 54. Residual shear angle, Φ_r, versus effective normal stress, σ_n; degree of saturation, S_r, as parameter. Results of direct shear tests with silty-clayey soil.

decomposed rock with a high proportion of fines. In granular soils or rock fills the effect of rate of shearing on the ultimate strength is negligible. In cohesive material different behaviour may occur when a shear zone, formed at a residual strength by slow drained shearing, is then subjected to more rapid rates of displacement.

Creeping soil slopes close to the limit equilibrium (F = 1) and exhibiting a low residual shear strength tend towards progressive failure with a gradual transition from creeping to (sudden) slip failure. The risk of tertiary creep increases with decreasing Φ_r. Longterm monitoring is therefore essential for a reliable risk-assessment and to start stabilizing, strengthening or retaining measures in time. If sufficient data exist, a creeping factor can be deduced, and future extrapolation is possible (Fig. 55)

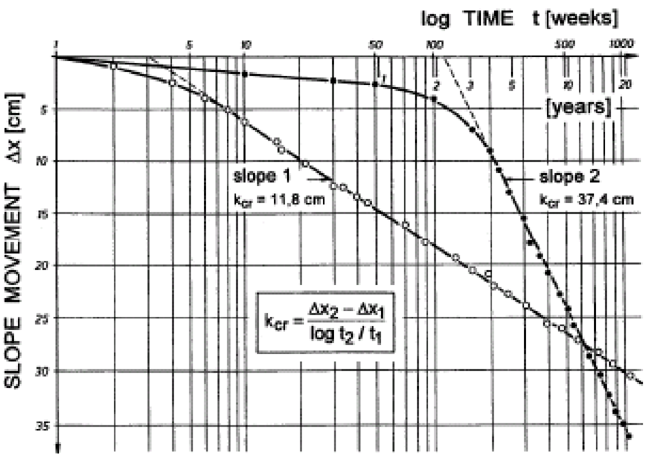


Figure 55. Steady creeping behaviour of two unstable slopes in weathered schists. Definition of creeping factor.

In the case of geosynthetic reinforced structures in creeping slopes or at the toe of unstable slopes, slip surfaces running through the ground and the structure have to be investigated. Therefore the interface shear strength (peak and residual value) between geosynthetics and fill material, and the composite shear strength of the reinforced fills (compound body shear strength) should be determined. Floating embankments and barrier dams in creeping slopes may undergo relatively large deformations. Consequently, residual strength is an essential parameter for risk assessment and proper design.

4.3 Creeping pressure on barrier dams in unstableslopes

The main factors influencing the creep rate of an unstable slope are

- slope angle,
- pattern of discontinuities, rock joint fillings,
- shear parameters,
- water pressure,
- external loads or unloading.

In a slope undergoing creep, retaining structures may be stressed by a lateral pressure E_{cr}, that exceeds significantly the theoretical earth pressure at rest E_o. This creep pressure E_{cr}, may also be considered as "sliding pressure" or "stagnation pressure" on a retaining structure. If assuming a (quasi) cohesionless mass and limit equilibrium β= φ, the creep pressure becomes a special case of an increased Rankine earth pressure according to Figure 56:

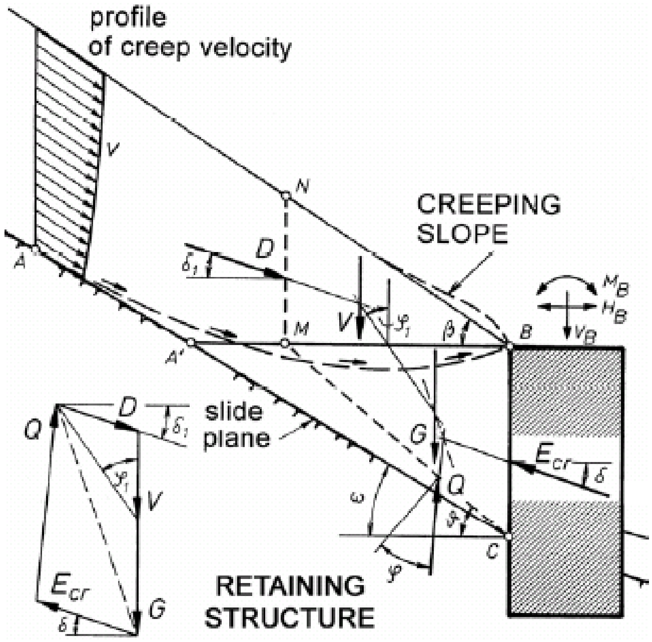


Figure 56. Theoretical assumptions for calculating the creeping pressure, Ecr, on retaining structures or protective embankment dams (barrier fills) in a sliding slope.
VB, HB, MB ... external forces on top of the structure.

$$E_{cr} = m(\varphi).\gamma.1/2.h^2.\cos\varphi \qquad (36)$$

β= slope angle; h= height
φ = fictitious friction angle, including effects of water pressure (and small cohesion)

The multiplication factor m(φ) also depends on the stiffness of the retaining structure. Hence, geosynthetic soil structures attract less creeping pressure than concrete walls. The enveloping values of Figure 57 have been obtained from numerous in-situ measurements for about 35 years already.

4.4 Stability analyses of geosynthetic reinforced earth structures in slopes

Stability analyses of geosynthetic-reinforced earth structures in slopes are based mainly on slope failure calculations taking into account the retaining forces in the reinforcing layers/inclusions. Frequently, external/global and internal stability are considered entirely separately in a rather formalistic way: Global failure mechanisms commonly assume slip surfaces that run completely outside the reinforced earth structure – thus not cutting the geosynthetic layers/inclusions. The analysis of internal safety, however, is frequently based on failure lines running only within the reinforced earth/slope.

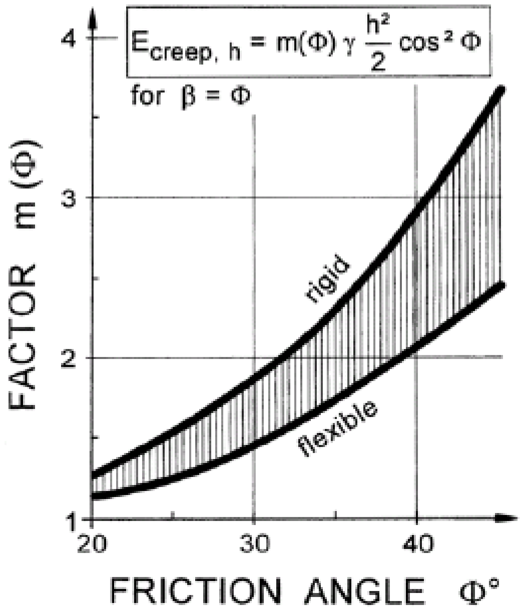


Figure 57. Influence of stiffness of structure on the creeping pressure Ecr; β = slope angle, Φ = (fictitious) friction angle

In daily design practice, such a formal separation of stability analyses frequently results in investigations being limited to a very narrow scope, namely failure mechanisms just outside and/or (predominantly) inside the reinforced part (e.g. a combination of two sliding bodies with plane slip surfaces). In many cases, an investigation of potential failure mechanism of various shapes and positions running partly outside and partly inside the reinforced zone is not carried out.

However, it is specifically such a failure mechanism – often called "mixed mode" or "compound mode" – that frequently turns out to be the most probable form of failure, thus providing the lowest factor of safety with regard to slope stability. Ignoring them may lead to a critical underdesign of the structure, and hence to an increased risk of failure.

Therefore, the length of the reinforcement should be calculated from the earth pressure theory as well as from slope stability analyses. A proper design has to consider all possible slip surfaces to gain the most critical failure mechanism. This means that cylindrical slip surfaces have to be investigated as well as plane, logarithmic, or any form of combined or polygonal failure surfaces, including potential slip surfaces in the ground and/or within the geosyntheticreinforced earth structure (e.g. pre-existing or fossil slip surfaces in the ground; geosynthetic-soil interfaces, structural interfaces, etc.). An exclusive reliance on conventional stability analyses proving internal and external stability without considering mixed forms of failures ("compound modes") is absolutely insufficient, as numerous failure histories have shown.

4.5 Embankments instead of bridges in creeping slopes

Modern compaction equipment, optimization and control, and geosynthetic reinforcement have opened the possibility of constructing high embankments instead of bridges for roads, highways and railways likewise (Fig. 58).

Experience has disclosed that generally valid criteria for weighing up the advantages of embankments in comparison to bridges are rather limited. Whether a bridge or an embankment is most suitable has to be decided for each project specifically. The main factors, which influence the pros and contras of an embankment instead of a bridge, are the following:

- Local situation, including existing buildings or settlements;
- Geomorphology; stability of existing slopes;
- Ground properties, including ground- and slope water conditions;
- Depth of slip surface(s);
- Seismic activity;
- Allowable total and differential settlements of the embankment crest (with regard to the evenness of road pavements or rail tracks);
- Availability of proper fill material;
- Material balance of soil excavation or slope cut and of fill volume within a certain construction section;
- Length and quality of access ways for the transport of fill materials;
- Number, diameter, length and location of possible culverts in the bottom of the embankment or within the fill;
- Statical system of the bridge;
- Schedule of construction operation;
- Construction costs;
- Costs for long-term maintenance;
- Local climate; environmental and aesthetic aspects.

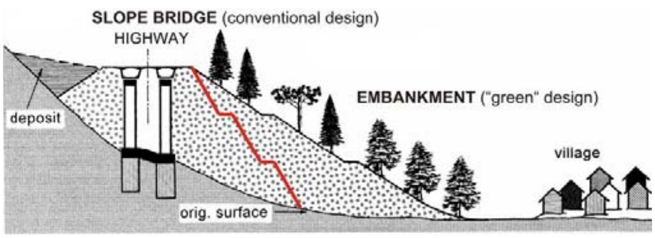


Figure 58. High embankment instead of a bridge for a highway or railway. Scheme, illustrating the acoustic and optic screening of a nearby village through proper vegetation of the embankment slope. Also indicated is the possibility of a steeper embankment slope due to geosynthetic reinforcement and of a deposit upslope of the embankment.

In the case of statically very sensitive slope bridges (e.g. with continuous girder superstructures) the foundation requires a high resisting moment (e. g. large diameter sockets/caissons), sometimes with multiple permanent anchorage. That means that rather rigid (and deep) footings have to be designed. Moreover, such buildings must be protected upslope by a flexible retaining structure, which acts as a first barrier (= "primary" retaining system) against excessive slope pressures (e. g. Fig. 59). This may require very long anchors (up to 100 m or even more) for fixing them in stable ground. As slope pressures may change with time, long-term monitoring of sensitive structures in creeping slopes is essential.

To sum up, bridges along creeping slopes are very expensive, have a relative high risk potential, require continuous monitoring and comprehensive maintenance. Moreover, they are more sensitive towards earthquake than geosynthetic reinforced embankments.

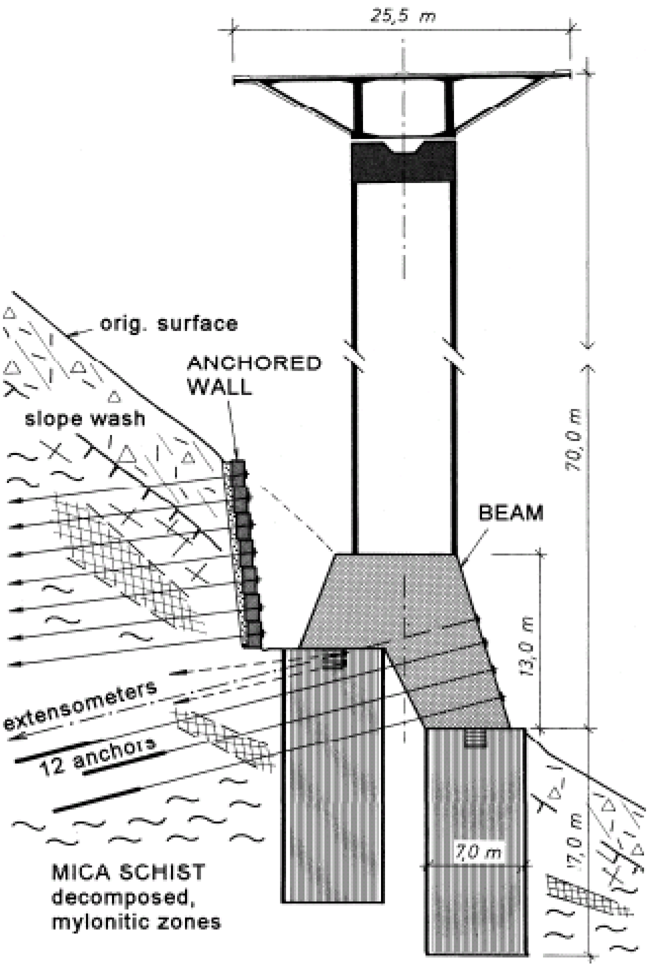


Figure 59. Typical foundation and permanent anchoring of a bridge pier in a steeply inclined, creeping slope. Statically sensitive superstructure (2,6 km long bridge) completely separated from the slide-protection measures (tied back retaining structures) around each bridge pier.

If comparing bridges and (reinforced) embankments along creeping slopes the following aspects should be considered:

- Embankments may replace bridges if the fill does not significantly increase the slope's creep rate, and the expected deformation remains within an allowable limit value.
- The construction costs of (geosynthetic reinforced) embankments are smaller than for bridges.
- Commonly, conventional embankments are superior to bridges only in connection with toeweighting of the creeping slope; i.e. if the local geomorphology allows a toe fill as counterweight. However, in the case of very deep reaching creeping strata floating earth structures are superior, because bridge construction and monitoring become uneconomical.
- Using light weight products as fill material reduces the load applied on creeping slopes. But, on the other hand, such materials exhibit a minor earthquake resistance because they tend to liquefaction (and embankment spreading), especially if they have a uniform grain size distribution. Moreover, the composite effect between fill and geosynthetic reinforcement decreases with lower density.
- Geosynthetic reinforcement makes a reduction of the fill mass possible, due to steeper embankment slopes. Furthermore, such composite earth structures have a significantly higher resistance against earthquake, spreading and local failure. The overall slope stability is higher than in the case of conventional fills, but nevertheless the creep rate of an unstable slope will increase unless drainage measures are successful.
- Dowelling of the creeping zone with large diameter piles or reinforced concrete sockets beneath the embankment is another alternative to slope bridges.
- Running along creeping slopes or crossing them requires for bridges as well as for embankments contingency plans. The structures should exhibit the possibility of future strengthening and/or relevelling if the results of long-term monitoring require such.
- The maintenance of embankments is clearly minor than for bridges.
- Embankments are environmentally more friendly than bridges, and their slopes can be planted. In many cases the earth structure finally looks likenatural terrain (e.g. Fig. 60).

Figure 60. 120 m high embankment (large scale counterweight) instead of a highway bridge in a creeping slope, 25 years after construction.

4.6 Long-term performance of high embankments or barrier dams in creeping slopes

From more than 35 years of personal experience with numerous embankments and barrier dams of 30 m to 135 m height and a length of about 150 m to 700 m it can be summarized as follows:

Usually, the stability factors against slope failure or ground failure of high embankments are at their lowest just at the end of construction. Excessive pore water pressures may occur in fine-grained, wet fill zones and/or in the natural ground. In the long-term a gradual decrease in safety is possible if

- the drainage systems fail;
- low quality fill material was used and insufficiently compacted;
- the fill material and/or the natural ground tend to progressive failure due to a very low residual shear strength.

Slope stability assessment of high embankments should consider a possible decrease of the friction angle in the lower fill zone depending on grain size distribution, grain shape and strength of the fill material: High overburden causes grain crushing, thus leading to a flattening of the Mohr-Coulomb rupture line in the σ-τ-diagram. Interface friction between geosynthetics and soil usually also drops with σ_n.

High embankments should be designed with benches at about 20 to 50 m of vertical spacing depending on fill material, slope angle, slope stability and local climate. These benches should have a width of at least 3 m. They serve for maintenance, facilitate local strengthening (if necessary) and act as a braking zone in the case of local slope sliding or soil liquefaction (e.g. during heavy rainfalls or earthquakes).

Benches as well as the toe zone of slopes exhibit local stress concentration (Fig. 61). Therefore these parts should be reinforced with geosynthetics (Fig. 62). In seismic zones the installation of geosynthetic reinforced zones parallel to the sloped surface of high embankments is recommended: Width at least 10 m. Barrier dams frequently should be reinforced over the entire area.

Figure 61. Stress concentration at the bench or toe zone of a slope. Results from photoelastic model tests (Hoek, 1969).

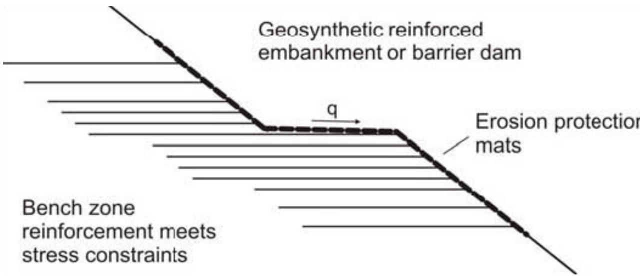


Figure 62. Recommended geosynthetic reinforcement of bench zones of high embankments or barrier dams in unstable slopes; schematic.

Moreover, it can be useful to install reinforced interlayers covering the entire ground plan of an embankment: For example, for slender embankments along steep slopes and without a toe buttress towards the opposite site of a valley or without the possibility of a counterweight fill in a valley bottom.

Such reinforced interlayers should be at least 3 m thick and be placed at a vertical spacing of about ¼ H to ⅓ H of the maximum embankment height (H) and on top of the embankment. They serve as loaddistributing members (reducing differential settlements) and increase slope stability.

In the case of proper fill material and careful compaction the self-settlements (ss) of high embankments (i.e. without ground settlements) lie typically below 1 % of the embankment height (H). If high-quality fill material is available and intensively compacted, self-settlements may even be reduced to about ss = 0.1 to 0.2 % of H. On the other hand, poor compaction and too many silty-clayey interlayers (e.g. placed in a sandwich-like mode) may lead to self-settlements of about ss = 1 to 2 % of H and more, depending on portion, thickness and quality of "soft" interlayers.

Spreading of high embankments is negligible if they are properly interlocked / teethed with the natural ground (by cutting steps).

Long-term experience with numerous highway embankments has disclosed that such earth structures exhibit in many cases significant advantages over bridges. They are environment-friendly, they facilitate a balance of cut and fill volume during construction, and they require only negligible longterm maintenance. If proper fill material is used and carefully compacted, the differential settlements of the embankment crest, hence also of the road pavement, are usually smaller than in the case of low embankments on soft or heterogeneous ground.

Until about the early 1990ties, continuous compaction control (CCC) and roller-integrated compaction optimization were not yet generally available but rather in an early development phase. The nowadays improved compaction equipment, technology and control methods clearly facilitate the construction of high embankments for highways (and even for high-speedrailways). This could be a cost effective, environmental-friendly alternative to bridges which, at the same time, also provides increased factors of safety.

High embankments crossing unstable slopes in seismic areas should have local reinforcement with geosynthetics. Such inclusions have proved very suitable as they improve significantly internal and external (overall) stability of the embankments. Moreover, they reduce differential settlements of the embankment crest.

4.7 Case histories

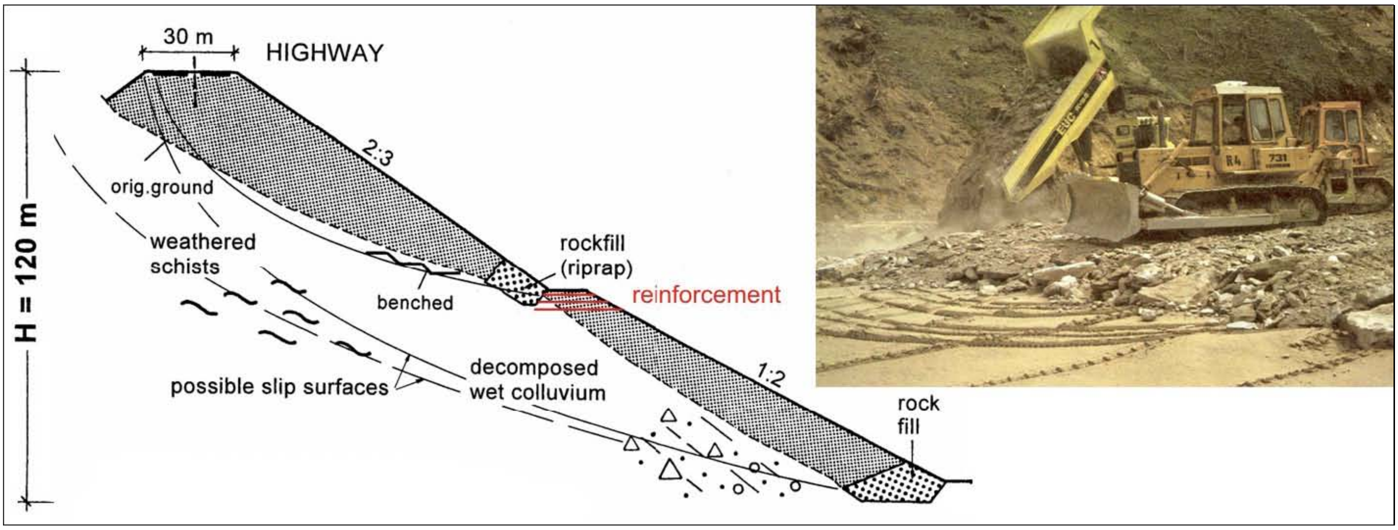
Figure 63 shows the side zone of a 120 m high embankment, which was constructed instead of a slope bridge along an unstable slope. The low slope stability required a local reinforcement with geosynthetics; moreover nonwoven geotextiles were placed in the base to provide sufficient long-term function of the drainage blanket. This earth structure was constructed between 1978 and 1979; meanwhile it can hardly be distinguished from the natural slope (see Fig. 60). Maintenance is minimal (contrary to the adjacent slope bridges), and the stability of the natural slope could be increased significantly.

Floating embankments instead of bridges have proved suitable also in rugged, steep terrain. Figure 52 gives a partial view of a 60 m high reinforced embankment in a seismic zone. The curved crest connects twin tunnels on either side of the fill that is intensively teethed with the natural ground. Earthquake induced deformations would therefore be of minor effect than in the case of a bridge.

Geosynthetic reinforcement of floating embankments or barrier dams becomes especially important in areas of high seismic activity and if the shear strength of a creeping slope is already progressively decreasing towards the residual value. This is demonstrated at a spectacular case history:

A 100 m high slope cut for a new highway triggered a landslide superimposing old creeping. The moving area finally had a lengths of about 600 m, a width of 270 m, a height difference of 330 m, and a total mass volume of 13 millions m3. Inclinometers disclosed the dominant slip surfaces in a depth of 83 m and displacements of 20 mm per month there. Furthermore, multiple slip surfaces had to be considered. Geological observations identified fissures and open cracks up to 2 m width in the order of 300 m upslope the crest of the slope cut.

The entire mountainside where this "Bit Cut" was excavated comprises a disordered, "chaotic" mass in which very weak ophiolites are erratically distributed. Moreover, the ophiolites are serpentinised, sheared, weathered, and with passages transformed to clayey material. A certain bedding seems to be dipping into the slope at a gentle angle of



Extremely heterogeneous fill material (sandy silt to rockfill) placed in sandwich form (not mixed).

Figure 63. Side zone of a 120 m high embankment instead of a highway bridge. Earth structure serves simultaneously as counterweight for the creeping slope and is intensively teethed with the natural ground.

approx. 10°, but it is intensively tectonized and severely cut by step-like discontinuities (Fig. 64). Numerous discontinuities more or less parallel to the slope or cut surface respectively govern local and overall stability. Moreover, clayey interlayers (mylonites) with low residual shear strength contributed the instability of the "Big Cut".

Several alternative alignments were discussed how to cross safely the geologically sensitive and seismically active area of this "Big Cut". Due to the uncertainties of a deep seated landslide the present alignment and as well tunnel options were excluded.

Given the magnitude of the instability and earthquake danger, the proposed solution in this area was to shift both carriageways away from the toe of the "Big Cut" – as far as possible, taking into account the highway geometrical restrictions deriving from the presence of already constructed tunnels on both sides of the landslide. This required comprehensive measures to reach acceptable safety factors, whereby geosynthetics were used for reinforcement, drainage/filters and erosion protection (Fig. 64):

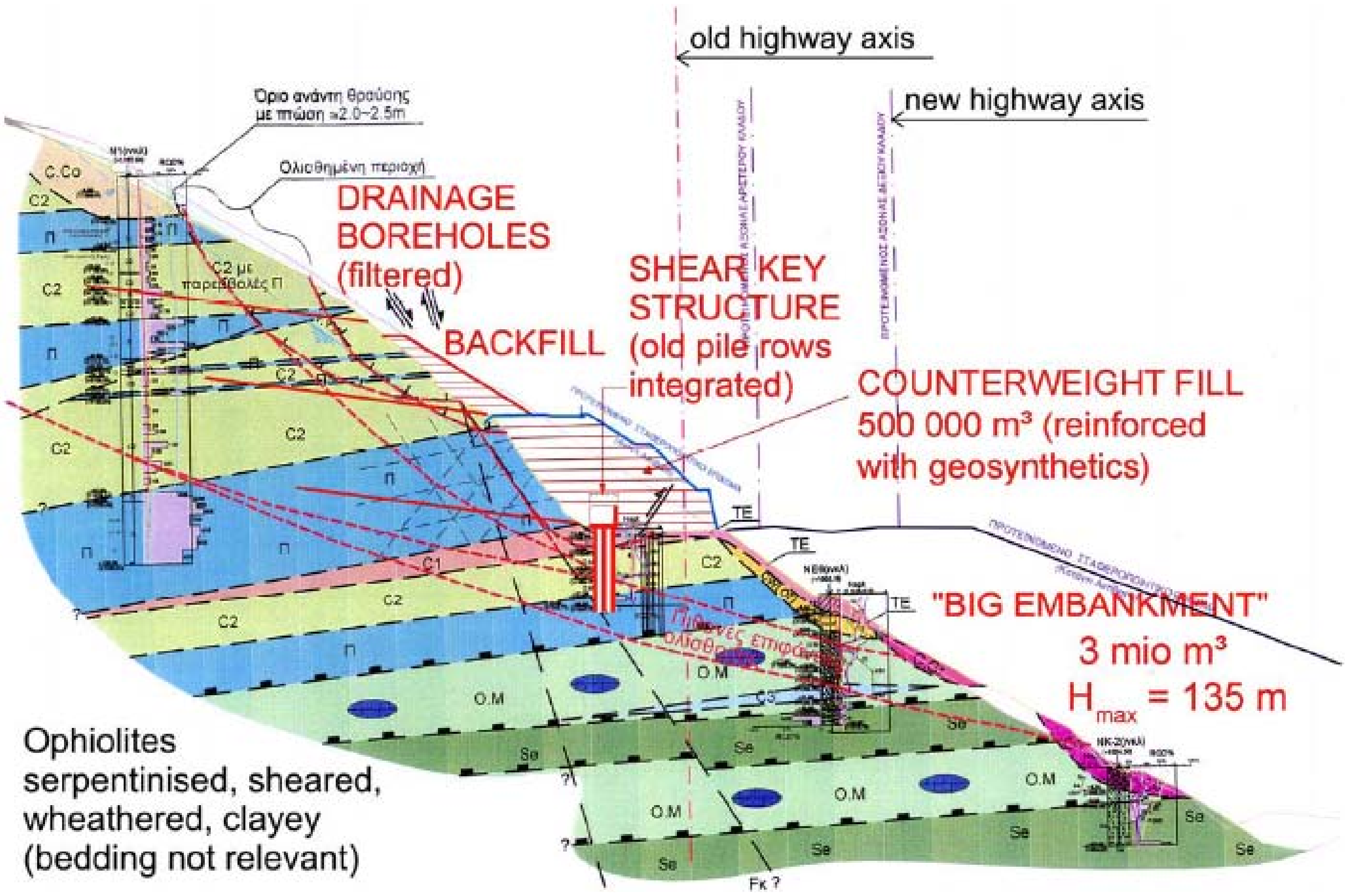


Figure 64. Geotechnical cross section through a landslide triggered by a 100 m high slope cut ("Big Cut") and stabilizing measures: Counterweight fill and upslope backfill fully reinforced with geosynthetics, "Big Embankment" includes only locally geosynthetics.

- Local slope dowelling (shear keys) with large diameter bored piles, barrettes or elliptical sockets on toe of the "Big Cut".

- Backfill of the toe zone of the cut and adding a counterweight fill of 500 000 m³; each layer over the ground view fully reinforced with geosynthetics. This should restrict mainly superficial slides but also deep-seated slides.

- Partial backfill of the slope cut uphill the counterweight to avoid local slides. The toe zone near the berm should be fully geosynthetic reinforced.

- Counterweight embankment ("Big Embankment", Fig. 64) of about 3 mill. m³, 700 m long and max. 135 m high to increase the overall stability for the creep area, to prevent deep reaching slips, and to carry the highway after realignment.

- Arch-shaped geosynthetic reinforced support dam on toe of the "Big Embankment" where the slope forms a canyon-like valley.

- Comprehensive drainage measures (deep drainage borings, drainage trenches, drainage blankets), using filter geotextiles.

- Open cracks treatment, slope reshaping.

- Contingency plans to immediately act to critical monitoring results.

Counterweight fill, upslope backfill and downslope big embankment were placed on granular drainage blankets using filter geotextiles if locally necessary. Contrary to the upslope fills and the downslope archdam supporting the "Big Embankment" the latter was reinforced only locally: In the bench zones, sandwich-like at vertical-spacings of about ¼ H of the maximum embankment height (H), and within the top 5 m of the embankment. The upper reinforcement served mainly for stress distribution and differential settlement reduction. The structural reinforcement consisted of flexible high strength geogrids manufactured from high-tenacity polyester yarns with low creep and an environmentally inert coating: Longitudinal tensile strength is 45 and 110 kN/m respectively, and elongation at break is 12.5 %. Additionally, three-dimensional reinforcement grids with erosion protection were placed on the slope surface.

Roller-integrated continuous compaction optimization and control (CCC) was recommended for all earthworks.

5 OPTIMIZED COMPACTION OF GEOSYNTHETIC-SOIL SYSTEMS

The interaction between geosynthetics and soil (or other granular fill material) depends mainly on the interface shear strength, the stiffness ratio, geometry of inclusions, and on the compaction of the fill layers. A high and uniform compaction degree favours the composite behaviour, thus improving the bearing-deformation characteristics of the entire system. In addition, homogeneous compaction avoids local stress concentration and stress constraints in multi-layered composite systems consisting of geosynthetics and soil or other granular material.

A high and uniform compaction is especially important for reinforced soil barrier dams, floating embankments, counterweight earth structures and other retaining structures. But also new dykes/dams or their enlargement should be compacted intensively and uniformly.

So far, compaction control has been carried out mainly by means of punctual test methods with the purpose to check the density or stiffness of the compacted layer. Conventional tests to determine density, and load plate tests for checking soil stiffness are based on spot checking by more or less random selection; and they are only superficial. The uniformity cannot be approved, and weak points can hardly be detected by such spot tests. This involves an unavoidable residual risk. Moreover, all spot test methods are relatively expensive and time consuming. Finally, conventional testing frequently delays construction work because construction activities must not be carried out in the vicinity of a spot test, as ground vibrations might affect the test results.

Therefore the conventional methods of compaction control are not sufficient any more for high quality projects, especially for high embankments along steep or unstable slopes and in seismic areas, for barrier fills, and for dams. Increasing demands on engineered earth structures require as much as possible continuous compaction optimization and control already during the compaction procedure. The roller-integrated CCC-technique, which first was used for road structures (Brandl, Adam, 1997), represents a distinctive improvement, because all control data are already available during the compaction process and over the roller-compacted area. Moreover, CCC allows a reduction of the number of conventional acceptance tests and provides all data already during compaction. Thus, the compaction procedure can be optimized and expedited.

Composite structures of geosynthetic reinforced soil require intensive and homogeneous compaction, whereby already the subgrade should be compacted properly (as far as possible). Theoretical investigations, field experiments and site measurements have disclosed that there is an intensive interaction between the soil or other granular material (fill layers), the geosynthetic inclusions and the compaction equipment. Measuring this interaction provides an excellent tool for three important goals of compaction:

- Compaction optimization Refers to quality of compaction, to the required compaction energy and time, and to the required geotechnical parameters of the compacted material. Over-compaction and re-loosening of layers should be just as much avoided as heterogeneous compaction degrees. Therefore, a main goal of cooperation between geotechnical and mechanical engineering has been the development of "intelligent" compaction equipment which itself reacts to locally varying soil/granular material properties by automatically changing its relevant machine parameters. Rollers with automatically regulating compaction systems (vibratory or oscillatory) are already a significant step in this direction, which raises compaction from a mere routine craft to a scientifically based high-tech process.

- Compaction documentation The CCC technology involves an automatic registration of all data in such a way that the results cannot be manipulated. These data collection is essential not only for site acceptance but also for quality control and long-term risk assessment. Furthermore, such information is very helpful for future rehabilitation of dams and embankments (after floods, earthquakes, etc.).

- Compaction control Should be widely performed already during the compaction procedure. A calibration of the control data based on the reaction between ground and compaction equipment is essential. Control tests after compaction should be increasingly reduced to conventional spot checking, whereas continuous compaction control (compaction equipment-integrated) should be promoted.

Soil properties inherently vary within a more or less wide range. Furthermore, locally different fill materials and compaction degrees on the construction site, but also by test-inherent uncertainties increase the scatter of compacted fill material characteristics. Consequently, reliable quality assessment and stability analyses may become somewhat difficult. Such a situation can be improved significantly by applying roller-integrated continuous compaction control (CCC), which facilitates compaction optimization already during the compaction procedure:

Vibratory roller compaction takes place by means of a vibrating drum, which is excited by a rotating mass. Oscillations of the roller drum changes depending on the soil response. This fact is used by CCC in order to determine the stiffness of the ground. Accordingly, the drum of the vibratory roller is used as a measuring tool (Fig. 65): Its motion behaviour is recorded (A), analysed in a processor unit (B) where a dynamic compaction value is calculated, and visualised on a dial or on a display unit (C) where data can also be stored. Furthermore, an auxiliary sensor is necessary to determine the location of the roller (D). By means of GPS (Global Positioning System) the position of the roller can be located up to an accuracy of 5 cm.

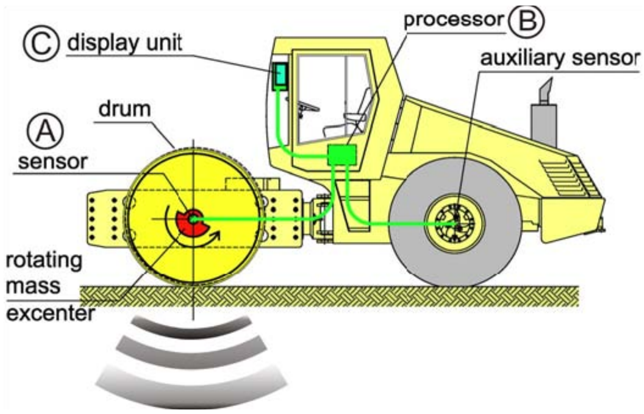


Figure 65. Principle of roller-integrated continuous compaction control (CCC) – and compaction optimization.

The dynamic compaction values have to be calibrated on the basis of conventional tests, e.g. compaction degree (D_{Pr}) or density ρ, or deformation modulus E_v (rather E_{v1}- than E_{v2}-values from static load plate tests and/or E_{vd} from dynamic load plate test). The main advantages of this control method are the following:

- Continuous control of the entire area;
- Results are already available during the compaction process, hence no hindering or delay of the construction work;
- Optimisation of the compaction work, including prevention of local over-compaction (which causes near-surface re-loosening of the layer);
- Full and permanent documentation of the entire area.

CCC possesses the essential advantage that the measuring equipment can be easily mounted on vibratory or oscillatory rollers (smooth rollers or sheep foot rollers). Experience has shown that the roller operators, site supervisors, etc. have very quickly familiarised themselves with this control method. Low quality rollers, which provide only low compaction quality, can be eliminated, and the documented data cannot be manipulated. CCC has proved suitable on many construction sites, and has therefore become obligatory for several years already in Austria: Mainly for roads and railways, for embankments, for dykes and dams, for barrier fills, for geosynthetic-soil structures, and for clay liners of waste deposits.

Furthermore, the measuring depth of rollerintegrated continuous compaction control is significantly larger than in case of conventional methods: Whereas density measurements commonly reach only a depth of 0.1 to 0.3 m and standard load plate tests about 0.5 to 0.6 m, CCC reaches to a depth of about 2 to 2.5 m.

Figure 66 illustrates this with a case history: The raft foundation of a power plant required a deep soil improvement and partial soil exchange to minimize differential settlements. Soil improvement of the loose river sediments (sandy silts) was performed with stone columns, which were then covered by a geotextile and layers of sandy gravel. The exact location and diameter of the stone columns was registered during the entire earthwork and could still be observed by CCC on the top layer that covered the stone columns by 1.5 m (Fig. 66).

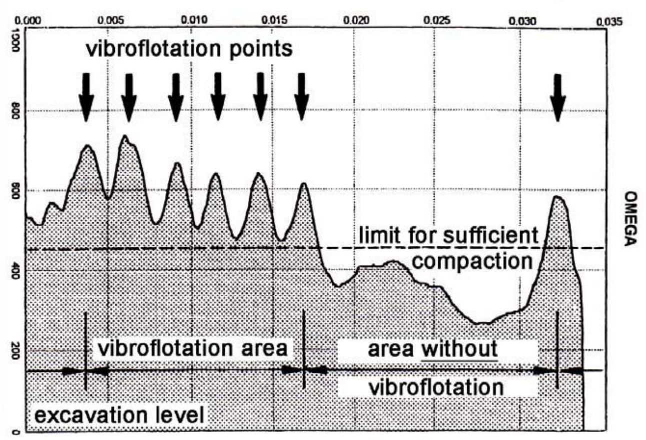


Figure 66. Results of roller-integrated continuous compaction control (CCC) on a 1.5 m thick fill on top of subsoil improved by deep vibroflotation. The positions of the stone columns are still clearly visible on the CCC-diagram. OMEGA = dimensionless value of compaction degree or soil stiffness respectively.

Consequently, CCC would easily detect those areas where geosynthetic reinforcement locally had been omitted in the sublayer(s) despite design requirements.

High quality compaction provides numerous advantages for dykes, flood protection and barrier dams, floating embankments, counterweight fills and retaining structures with and without geosynthetic reinforcement. Moreover, it has proved successful for roads, highways, railways and all kinds of earthworks, reinforced retaining structures etc.. Compared to conventional spot checking CCC provides:

- Increase in safety;
- Improvement of serviceability;
- Improvement of seismic response;
- Increase in life-time;
- Reduction of costs and maintenance;
- Reduction of construction time;
- Detailed monitoring of top zone (2.5 m) of existing dykes or dams (see Fig. 8).

Furthermore, CCC increases the installation survivability of geosynthetics placed in multi-layered structures: The uniformly compacted, smooth surface of each fill shift reduces the risk of wrinkles in the geosynthetic or it's punching; furthermore, local over-compaction of the fill layers (hence stress constraints in the geosynthetic inclusions or cover) is avoided.

Intensive and uniform compaction of geosynthetic reinforced dykes, dams, barrier fills and other retaining structures improves their earthquake resistance significantly. If the layers are not properly compacted they will be more damaged during stronger seismic events. A rigid facing of geosynthetic reinforced retaining walls makes higher compaction of the outer fill zone possible. Consequently, such structures undergo smaller damage. Too intensive compaction, however, should be avoided in any case, because it creates excessive lateral earth pressures behind rigid facings or moves flexible elements outwards. CCC is an excellent tool to optimize compaction intensity depending on structural requirements.

The increase in lifetime and serviceability of structures is especially important for road / highway pavements and for railways, but also for dykes and dams, and for barrier fills and other retaining structures. Weak spots in dykes/dams may promote seepage, inner erosion or piping. Hence, not only the absolute degree of compaction but also its uniformity is essential. This can be checked best by continuous compaction control, which is a significant progress over hitherto statistical quality control. Until now the different selection procedures of spot checking have been: grid pattern, random selection, subjective selection, and subjective selection using existing criteria.

The CCC-method also involves new statistical criteria because the uniformity of the compacted layer as well as the increase in compaction degree during subsequent roller passes are recorded. Mean value, max. and min. value, standard deviation, and increase of the compaction values represent relevant parameters. Moreover, the "minimum quantile" can be used as soon as sufficient experience with CCC is gained on the particular construction site.

Hitherto used statistical parameters do not allow the assessment of the distribution of control data within a section: The plots indicated in Figure 67 have the same mean value, max. and min. value, and standard deviation. Nevertheless, they exhibit different qualities. Figure 67(a) shows only one limited weak zone, which can easily be improved, whereas the area (b) requires comprehensive measures to achieve sufficient and homogeneous quality.

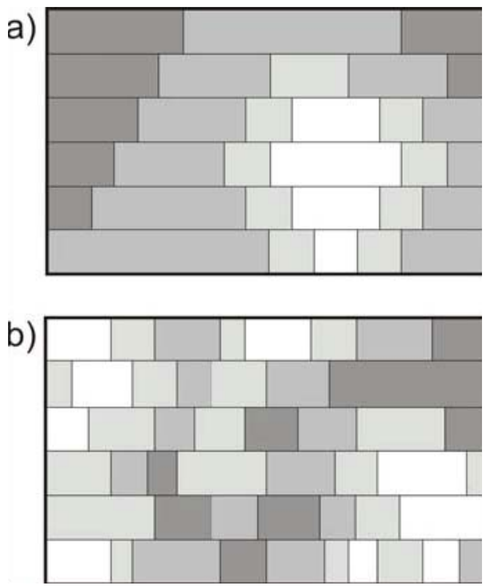


Figure 67. Control plots of compacted areas with the same conventional statistical characteristics of compaction (schematic). The strips indicate the roller lanes, controlled with CCC. The different colours show different compaction degrees. Statistical evaluation can be improved with "Variography" or "Kriging".

To statistically judge the distribution of control values within a defined area, the statistical methods of "Variography" and "Kriging" can be used. These methodologies were originally developed in order to interpret geophysical data and seem to be a useful tool to improve the quality assurance of compacted layers, hence also of geosynthetic-soil structures A reliable interpretation of the CCC-data is only possible if the operation conditions of the vibratory roller drum are taken into consideration (see Reference - RVS 8.S.02.6). The significant operating conditions depend on the roller and soil data and on the interaction between roller and soil (or other granular material) – Adam, 1996 and Brandl, Adam 1997.

Roller compaction technologies have been sophisticated significantly during the past 15 years and they now provide a wide range of possibilities to select the adequate roller for the particular purpose. A further development is the automatically controlled roller "Vario Control", whereby the direction of excitation is controlled automatically by using defined control criteria. "Vario Control" compaction provides uniform compaction, less roller passes, improved compaction both in deeper layers and on surface, and reduction of lateral vibrations, e.g. when operating closely to sensitive structures.

Meanwhile also smaller compaction equipment (plate vibrators, etc.) exists that facilitates continuous compaction control for backfills under confined site conditions or for slender geosynthetic-soil structures.

To sum up, roller-integrated continuous compaction control (CCC) represents a significant improvement for high-quality management systems. Compaction control is integrated in the compaction process, and data are provided all over the compacted area. Because of the outstanding advantages of CCC, this technology should be used for the compaction of soil structures as much as possible, especially in case of geosynthetic reinforcement.

To a certain extent, compaction control can be also achieved by the spectral analysis surface wave method (SASW) or continuous surface wave technique (CSW). Both methods are non-intrusive; however, a continuous compaction optimization (i.e. already during rolling) is not possible.

6 THERMO-ACTIVE GEOSYNTHETICS FOR TUNNELS ("ENERGY-GEOSYNTHETICS")

6.1 General

Natural disasters are increasingly caused also by climate change. Consequently, measures against climate change help to mitigate natural disasters. Thermo-active ground structures represent such a contribution to environmental protection, and moreover they provide substantial long-term cost savings and minimized maintenance (Brandl, 2006). Energy foundations and energy tunnels make use of earthcontact concrete elements that are already required for structural reasons, and simultaneously they work also as heat exchangers. Absorber pipes filled with a heat carrier fluid are installed within conventional structural elements (piles, barrettes, diaphragm walls, basement slabs or walls, tunnel linings) forming the primary circuit of a geothermal energy system. The natural ground temperature is used as a heat source in winter and for cooling in summer. Hence no additional elements have to be installed below surface. The primary circuit is then connected via a heat pump to a secondary circuit within the building.

6.2 Energy tunnels with geocomposites

"Energy tunnels" are infrastructure tunnels that are used simultaneously for heating and/or cooling buildings, road pavements, or bridge decks. Until recently geothermal heating from tunnels was used only in connection with hot waters, mostly without heat pumps. But the heat potential along a tunnel can also be utilized by using the tunnel support and lining as energy absorbers. These may be anchors, rock/soil nails, geosynthetics and secondary concrete linings. Anchors or nails reaching deeply into the surrounding ground can activate a relatively large mass for geothermal utilization. "Energy-tunnel" may be excavated as closed systems, e.g. by the New Austrian Tunneling Method – NATM (Fig. 68) or by the cut and cover method.

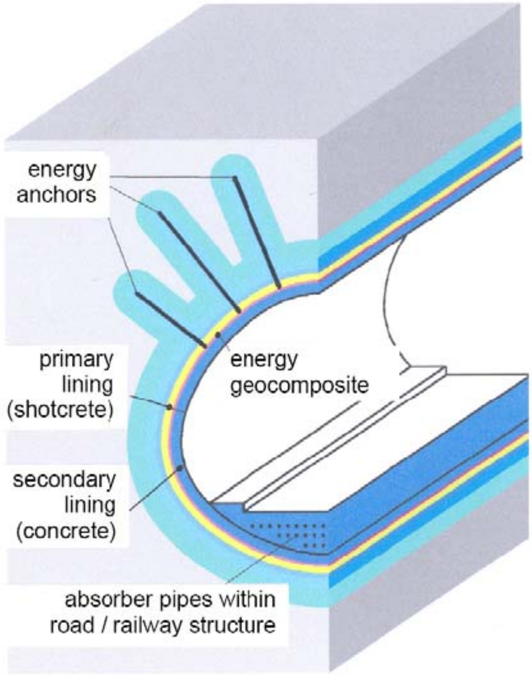


Figure 68. Absorber elements for energy tunnels.

Near the portals of transportation tunnels with geothermal equipment the following groups may take the available energy:

- The owner or operator of the tunnel;
- Private users (especially large residential blocks, but also one-family houses);
- Commercial, industrial users;
- Public users (municipal, federal).

Moreover, seasonal operation makes cooling during summer possible by using the ground for heat storage via the energy geocomposites.

An example from a railway tunnel in Vienna underlines these advantages: About 1200 private flats could be supplied with geothermal energy, but also large public buildings.

Energy tunnels are an exciting challenge to geotechnical engineering whereby the optimization of energy extraction or feed/storage, of transfer and distribution requires a multi-disciplinary cooperation. Ground investigation and geotechnical design should incorporate geothermal aspects already at an early stage. The main advantages of this innovative technology are:

- Commonly, tunnels are situated in a depth, where the seasonal ground temperature is widely constant.
- Tunnels exhibit large interfaces between structure and ground, thus favouring the extraction and/or feed, hence storage of geothermal energy.
- Very deep-seated mountain tunnels can make use of great geothermal gradients.
- In long tunnels significant inner heat is available, mainly due to the waste heat of transportation. In metro-tunnels, for instance, temperatures of more than +20°C are possible even during the winter months.
- Utilizing clean and self-renewable energy from tunnels is environmentally friendly and economical. Therefore energy tunnels have a high public acceptance and political support which makes the approval procedures easier.

Until now only cut-and-cover tunnels and open face tunnels (excavated after the NATM) have been equipped with thermo-active geosynthetics or anchors/nails. But bored tunnels with segmental lining can also be used as energy tunnels as they exhibit earth-contact structural elements. Furthermore, optimized energy tunnels may use not only the ground temperature from their large underground contact area but also inner heat sources from traffic, lighting etc.

A supply pipe (e.g. along the side walls) feeds those cellular geocomposites where the absorber fluid takes the energy from the surrounding ground. The warmed up fluid is then transported from the absorber pipes via collector line (HDPE pipe) to a heat pump. Prefabricated geocomposites with integrated absorber pipes make the installation of independent thermo-active cells possible, which can be easily monitored and separated from the overall circuit for repair. The geocomposites should be prefabricated in plant and transported in rolls to the construction site. The rolls should have the standard width (typically about 2.5m – Fig. 69) and a length adapted to the cross section of the tunnel. The installation procedure in the tunnel is similar to the placement of conventional non-woven geotextiles. The energy geocomposite exhibits adaptors on both ends and they have to be overlapped in the longitudinal direction of the tunnel. Additional works are only the fixing of absorber pipes when casting the concrete and checking the watertightness of the pipe system.

Figure 69. Unrolling of an energy geotextile (geocomposites).

Absorber pipe systems have to be air-free. Degassing is therefore essential for heat transfer and to obtain high efficiency. Details on installation and degassing can be found in Markiewicz, 2004.

6.3 Large-scale tests in a test hall

Large-scale tests were performed in a geosynthetics production hall prior to the installation of "energy geosynthetics" in tunnels.

Since the year 2002 several geosynthetics have been tested at the Institute for Soil Mechanics and Geotechnical Engineering of the Vienna University of Technology for utilization as geothermal absorber elements in shotcrete supported open face tunnelling ("New Austrian Tunnelling Method"). According to the present state of the art non-woven geotextiles serving simultaneously for mechanical protection and in-plane drainage have proved most suitable. Non-woven geotextiles are required in tunnelling mainly for watertight geomembrane protection, and they improve roof and sidewall drainage. Fixing plastic pipes between two geotextiles provides a thermo-active absorber system with a closed circuit for energy extraction (or storage).

6.4 Installation of thermo-active geocomposites on construction site: Full-scale tests

In principle, energy geocomposites are installed in a tunnel like non-woven geotextiles. But special attention must be given to the connecting pipes leading from the individual heat exchangers (absorber pipes) within the geocomposites through the secondary tunnel lining to the main collector. This detail varies in dependence of the tunnel structure being mainly influenced by the waterproofing system: umbrella or full circumference waterproofing, waterproofing with geomembrane, watertight secondary lining (Fig. 70). In the case of groundwater under high pressure special pipe passage elements are required.

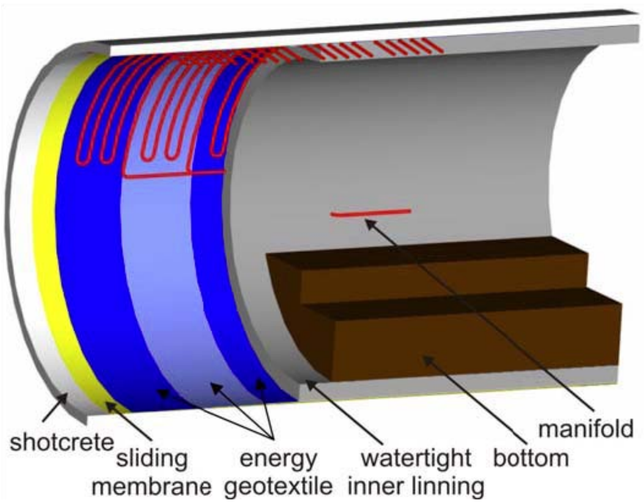


Figure 70. Scheme for installing energy geotextiles in the case of a watertight inner lining.

In order to optimize the production and installation of thermo-active geocomposites ("energy geotextiles") full scale tests have performed. The tunnel was excavated in fractured to intact rock (flysch), the overburden in the test section varied between 15.6 and 17.3 m. Figure 71 shows the cross section of this single tube railway tunnel close to the test fields. The niche allowed the placement of a heat pump, whereas the test fields extended along the standard tunnel cross section without niche. Figure 70 illustrates the scheme with watertight secondary lining. Hence, no waterproofing geomembrane was needed, only a sliding geomembrane between thermo-active geotextile and watertight concrete.

The following structural elements were used:

- Non-woven polypropylene geotextile with the following technical data:

  Mass per unit area: 285 g/m²
  Tensile strength: 21.5 kN/m
  Elongation at maximum load: 100 % (MD); 40 %(CD)
  Static puncture resistance (CBR-Test): 3300 N
  Cone drop test (hole Ø): 17 mm
  Permeability vertical: 70 l/m²s
  Opening size O_{90}: 95 µm
  Thickness (2 kPa): 2.5 mm

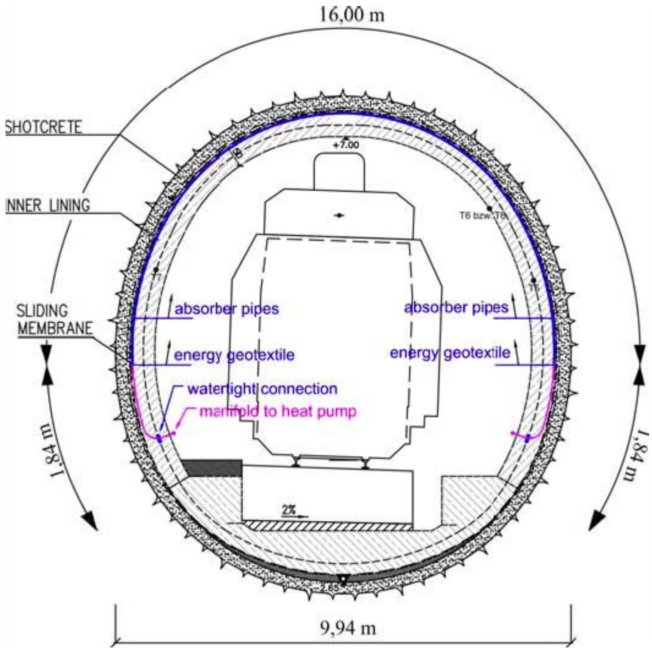


Figure 71. Cross section of the test field.

- Strips of 2.5 x 16.0 m whereby 1.0 m end zones and 0.5 m wide longitudinal zones were not fitted with absorber pipes. The longitudinal strip parts served for fixing the absorber pipes and for overlapping the geotextiles.

- Absorber pipes of linear polyethylene with copolymere octane, outer diameter 25 mm, wall thickness 3.5 mm, exhibiting
  - high flexibility and stability;
  - suitability for bending with small radius but small rebound forces;
  - proper long-term behaviour.

The geotextiles were fixed to the primary tunnel lining (shotcrete) with common nails and, additionally, the absorber pipes were fixed with pipe clamps to avoid deformations during concreting of the secondary lining (Fig. 72). Four strips of energy geocomposites were connected to one thermo-active unit with one manifold for absorber entry and another one for absorber return. Therefore, main attention during filling the absorber system with antifreeze must be paid to a complete filling and degassing of all absorber pipes. Preliminary tests had been conducted to find a suitable technique for filling the absorber system, as there is no possibility for degassing at the top(s) of the absorber system. The passage of the collecting absorber pipes require special measures as illustrated in the examples of Figures 73, 74. In principle, there are four options: plastic screws, flexible hose, niches, and reinforcement cross.

The pipes have to withstand the outer pressure of casting the concrete. Therefore, they must be kept under inner pressure (compressed air, $p_i \geq 2.0$ bar) during concreting. The formwork transport equipment for the secondary tunnel lining needs special openings for the pipe passages. The absorber pipes coming from the individual geosynthetic strips have to be connected to the collecting pipes by special Tshaped elements (Fig. 75). The collector pipes should run as long as possible between inner and outer reinforcement of the tunnel lining (Fig.76). After finishing all welding work the entire pipe system has to undergo a detailed tightness control (Fig. 77). The tightness of the pipe system must be checked after each construction phase, i. e. before placing the reinforcement, after welding the bars etc.

Figure 72. Installation of energy geotextiles at the testing plant „LT22 – Bierhäuselberg". Pipe loops clearly visible.

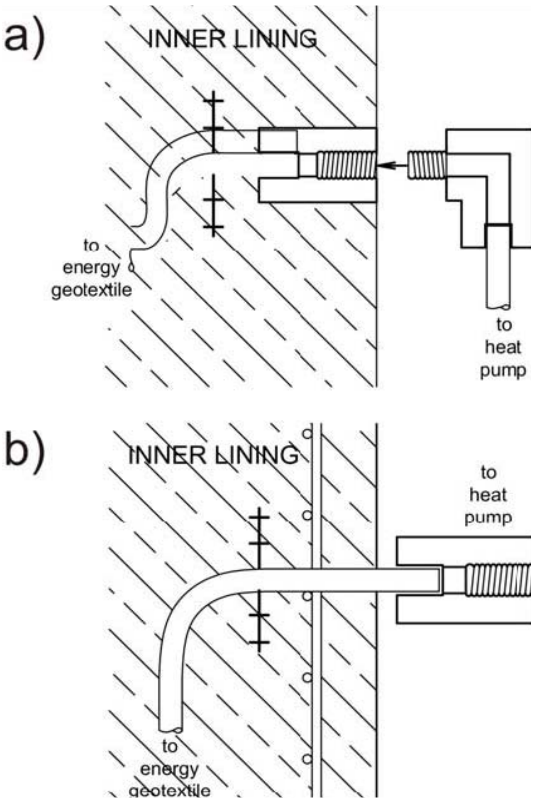


Figure 73. Watertight passage of the collecting absorber pipes through inner lining with screws (a) or flexible hose (b).

Figure 74. Pipe passage through the secondary (= inner) tunnel lining. Detail with criss-cross reinforcement.

Figure 75. T-shaped elements to connect the absorber pipe from the individual strips of energy geosynthetics to the collector pipes.

Figure 76. Collector pipe running between the inner and outer reinforcement of the tunnel lining.

Figure 77. Tightness control after finishing all welding work at the pipe system.

6.5 Test results

The energy extracted from the ground was transferred to a heat pump and then to a radiator that emitted the produced heat to the air in the tunnel. Thus, the efficiency of the thermo-active system could be measured. For parametric studies different operating features were investigated. The measurements comprised (Markiewicz 2004):

- Temperature of brine solution fluid in the absorbers (entrance, exit);
- Temperature of ground/groundwater and tunnel (air, lining);
- Heat production of the radiator;
- Flow velocities in the heating circuit and absorber fluid circuit;
- Electric current consumption of heat pump and circulating pumps;
- Photographical documentation of the surface of secondary tunnel lining with heat picture cameras to check the operational serviceability of the individual strips of the energy geocomposite.

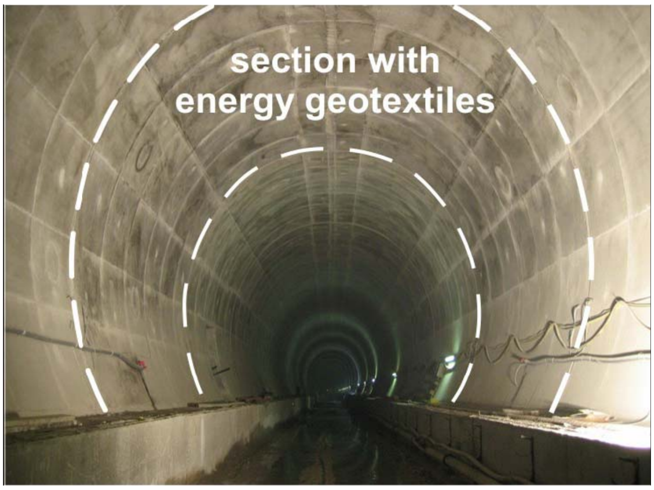
Immediately after the thermo-active operation had started the temperatures of the secondary lining changed. This proved an excellent contact between energy geotextile and waterproof concrete.

In the first operating phase the entrance temperature of the absorber fluid circuit dropped to −1° C due to an overcapacity of the heat pump. The temperature difference between entrance and exit was 3.5° C, and the entrance temperature of the heating circuit went up to 21° C. Therefore, a heat capacity of only 31.4 W/m² could be achieved corresponding to a performance factor of $\beta = 4.5$. In order to avoid the formation of ice lenses in the ground the capacity of the heat pump was reduced then.

The measurements have disclosed that a heat capacity of 15 to 20 W/m² can be gained from the thermo-active geocomposites, whereby a significant portion of heat comes from the tunnel air. Function tests with a thermal imaging camera showed that two rings of the energy tunnel have a reduced capacity (Fig. 78). This was caused by a not proper degassing of an absorber pipe section.

6.6 Recommendations for practice and further innovations

Energy geocomposites consisting of non-woven geotextiles



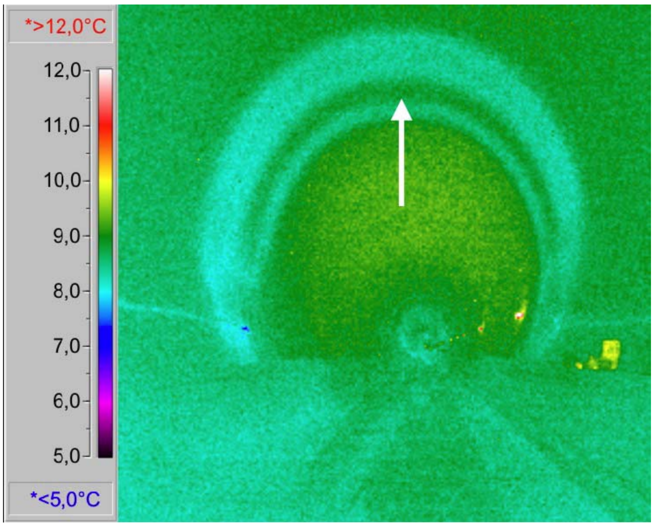


Figure 78. Pictures of the test field taken with a normal camera (above) and a thermal imaging camera (below). Collector pipes along the side walls of the tunnel clearly visible on the normal photo. The brighter one of the thermal photo indicates a test zone with a spot of lower thermal activity.

fitted with absorber pipes can be installed like conventional geotextiles. It is not necessary to use absorber pipes of such a high flexibility as at the large-scale test. Conventional plastic pipes are sufficient if the allowable minimum bending radius is considered.

- Fusion-welding of the absorber pipes may cause the formation of bulges inside the pipe seam, thus reducing the fluid discharge area and eventually the efficiency. Therefore, pipe sockets (e.g. electric welding sockets) should be preferred.
- Conventional attachment of thermo-active geocomposite to the primary tunnel lining (shotcrete) is sufficient. Additional fixing of the absorber pipes by means of pipe clamps is not necessary.
- The number of pipe passages through the secondary lining (waterproof concrete) should be minimized. The collecting pipes should not be placed along the sidewalls of the tunnel but rather in the base concrete of the invert. At a central point (e.g. emergency exit) they can be led outside then.

Commonly, tunnel structures with waterproof concrete consist of the following elements: Primary lining (shotcrete – structured geomembrane with naps (for drainage) – sliding membrane (to minimize transfer of shear forces) – secondary lining (= waterproof structural concrete). With regard to thermoactive geocomposites the following innovative structure was designed: Primary lining – energy geocomposite – secondary lining. This geocomposite consists of three elements: non woven geotextile – absorber pipes – sliding membrane. Geotextiles with high in-plane drainage capacity (high transmissivity) can take the drainage function of a structured geomembrane, if only moderate water ingress is expected. Figure 79 shows an installation detail for such "energy geocomposites.

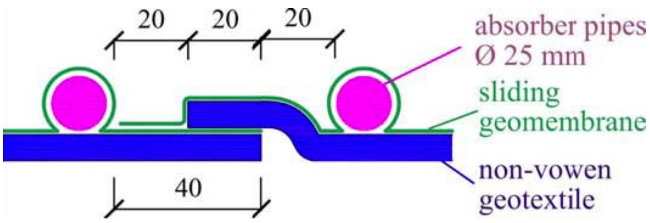


Figure 79. Installation detail for overlapping thermo-active "energy composites": Non-woven geotextile (with high transmissivity), absorber pipes, sliding geomembrane.

Sliding membranes of 0.2 mm thickness, commonly used on construction sites, have proved suitable. Small leaks would not affect their serviceability, because they serve not for waterproofing but for separation with a smooth interface between geocomposite and waterproof concrete. This reduces stress constraints within the tunnel lining and minimizes the transfer of shear forces on the waterproof concrete (according to the "white tank" technology).

6.7 Résumé

Thermo-active ground structures (energy foundations, retaining walls, tunnels, etc.), but also energy wells are a promising innovation regarding sustainable and clean energy consumption. A significant advantage of such systems is that they are installed within elements that are already needed for statical/structural or geotechnical reasons. Hence, no separate/additional structural or hydraulic measures are required. Foundations, walls (below and above ground) or tunnel linings can be used directly for the installation of absorber pipes for heat exchange.

Energy tunnels using geosynthetics fitted with absorber pipes are a key improvement over the conventional geothermal methods like (deep) borehole heat exchangers or near-surface earth collector systems. Comparative large scale tests and site experience have shown that composites, consisting of non woven geotextiles fitted with absorber pipes provide the best results. Usually, a temperature difference of only ΔT = 2°C between absorber fluid inflow and returnflow from the primary circuit is sufficient for an economical operation of the energy system. Consequently, such geothermal systems represent low temperature systems. Experience has shown that the electricity required for operating the entire system commonly varies between 20 to 30% of the total energy output. If no heat pump is necessary (e.g. for free cooling) this value drops to 1 to 3 % for merely operating a circulation pump.

7 FURTHER APPLICATIONS OF GEOSYNTHETICS

In addition to the previous chapters some special applications of geosynthetics are mentioned:

- Rockfall shelters:

Roads and railways along steep rock slopes need shelters if protective fences are not sufficient to withstand severe impacts. Large-scale rock blocks falling from great height may destroy even prestressed reinforced concrete structures if they hit them directly (Fig. 80). Therefore, shelters under high dynamic load should be covered by sloped soil fill, preferably reinforced with geosynthetics (Fig. 81). Thus, the blocks can over-roll the structure without damage.

Figure 80. Damaged concrete shelter due to heavy rockfall.

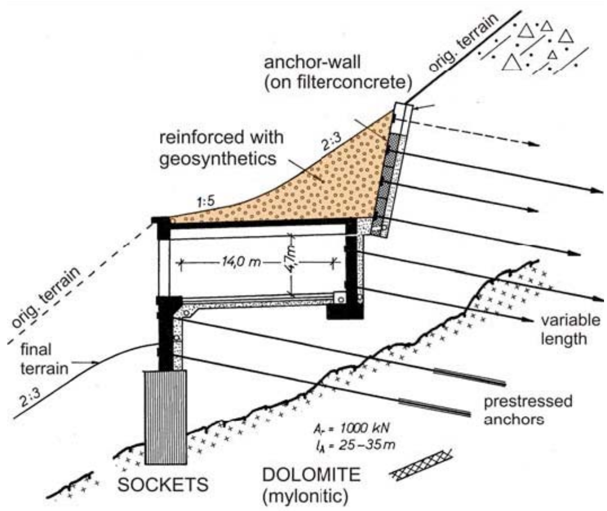


Figure 81. Reinforced concrete shelter (gallery) against rockfall, debris flow and avalanches. Covered by protective fill reinforced with geosynthetics.

- "Sandwich-reinforcement" of embankments and barrier dams:

The stability and deformation behaviour of high embankments and barrier dams along steep or unstable slopes or in seismic areas can be significantly improved by geosynthetic "sandwich-reinforcement". The scheme of Figure 82 demonstrates that the reinforced zones should preferably be placed at levels where benches are designed. The top reinforcement is recommended if the embankment carries traffic routes. It serves for better load distribution and reduces differential settlements. Moreover, sandwichreinforcement provides a high earthquake resistance.

- Glacier melting:

Glacier melting progressively creates unstable slopes, when ground that had been frozen into great depth is thawing. High pore-water pressures and seepage reduce local and global slope stability, and trigger debris flows, multiple slips or deep reaching block sliding. Moreover, glacier melting becomes a problem in touristy regions, especially where "Alpine" skiing dominates. Therefore, several precautious measures and contingency plans have been developed in Austria. A special application of geosynthetics is the local cover of exposed glacier areas and zones around lift columns with white nonwoven geotextiles (Fig. 83). Lift columns are frequently founded/anchored in creeping ice that undergoes increasing differential movements. A local reduction of glacier melting by geotextile cover prolongs the intervals of column readjustment.

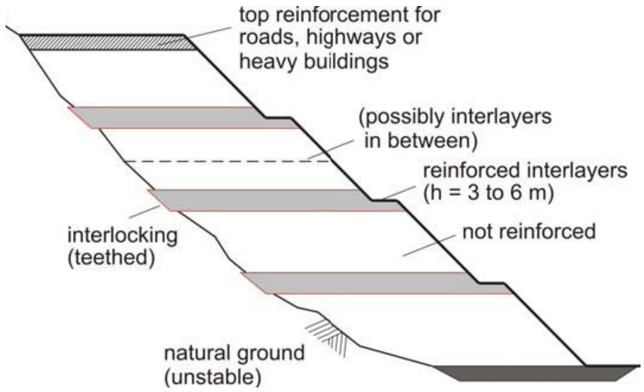


Figure 82. "Sandwich-reinforcement" of high embankments and barrier dams along steep or unstable slopes and in seismic zones. Scheme with steepened fill slope; not to scale.

Figure 83. Local geotextile cover of glaciers to reduce melting of particular zones.

Measurements disclosed that glacier melting could be reduced by about 60 %. The main characteristics of the most suitable protective geosynthetics (found often comparative in-situ tests) are:

Non-woven polypropylene geotextile:

Mass per unit area 340 g/m²
Thickness (at 2 kPa) 2.0 mm
UV-resistance (residual strength) > 75%
Tensile strength (EN ISO 10319) 23 kN/m
Opening size (EN ISO 10319) 0.06 mm
Thermal resistance 0.0593 m²K/W
Water permeability in plane 2 x 10-6 m³/m.s
Water penetration resistance > 8 cm
Light-reflexion capacity 78 %
(wave length 500 mm)

- Beaver barriers:

An increasing population of beavers represents also an increasing danger to dykes and flood protective dams. Therefore field tests with several geosynthetics as beaver barriers were conducted in cooperation with zoologist (Fig. 84). The tests disclosed that beavers finally broke through all geosynthetics but not through metal wire mesh with synthetic cover. Such wire meshes are installed in Austria since the year 2008 along critical sections through and beneath dykes/dams. On toe of the dam they are fixed in a trench filled with concrete (Fig. 85), at the crest they are covered by the layers of the road structure.

Figure 84. Geosynthetics (a, b) and wire mesh (c = right) to prevent damage to flood protective dams caused by beavers. Field tests in cooperation with zoologists.

Figure 85. Installation of nets against beaver actions along flood protective dams.

8 FINAL REMARKS AND CONCLUSIONS

"Geosynthetics-engineering" offers a broad range of applications for damage prevention, natural disaster mitigation, and for rehabilitation measures. Coastal protection, erosion protection, seismic aspects, sinkhole bridging, or hazardous waste containment, for instance, are topics that have been treated in numerous publications. Therefore they are not discussed in this Giroud Lecture, though referred to in most chapters.

The selected geosynthetic applications emphasize the close links between theory and practice, and between geosynthetics and geotechnical engineering. Design by function, or semi-empirical design with calculated risk ("interactive design"), combined with the observational method represents an essential feature of geosynthetics application. For geosyntheticsoil systems in critical areas (unstable slopes, seismic zones, etc.) contingency plans should exist (e. g. possibilities of in-time strengthening).

Due to their flexibility geosynthetic-soil structures have clear advantages over rigid structures with "classical" construction materials: Higher impact absorbency of rockfalls or avalanches, lower sensitiveness to earthquake and creeping slopes, rapid installation (flood defence, etc.). But also ecological advantages can be found when comparing the total energy consumption for the pre- and finalproducts, the transportation from the manufacturer to the construction site, and for the installation.

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Giroud Lecture, 9th International Conference on Geosynthetics, 23-27 May 2010, Brazil

ΘΕΣΕΙΣ ΕΡΓΑΣΙΑΣ ΓΙΑ ΓΕΩΤΕΧΝΙΚΟΥΣ ΜΗΧΑΝΙΚΟΥΣ

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› **Responsibilities**

- Ongoing projects such as Cityringen and Amagerforbrænding
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For further information, please contact Head of Department Thomas Carentius Larsen +45 4520 4189, or visit www.geo.dk.

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GEO's 200 highly qualified staff members offer Denmark's most comprehensive soil and water consulting services. GEO has offices in Copenhagen and Aarhus.

Ground Engineering Opportunities

Recruiter	: Aurecon
Posted	: 07 November 2011
Ref	: NCE777879
Location	: New Zealand
Sector	: Geotechnical
Category	: Geotechnical Engineer
Job Type	: Permanent
Salary	: Market Rate

Further information

Aurecon provides world class professional consulting engineering services to clients across New Zealand, Australia and internationally. An independent, employee owned company, Aurecon has more than 6,500 staff in over 80 offices worldwide.

Our Ground Engineering team is well established as one of the largest within New Zealand, with broad capability across the country. We seek skilled Geotechnical Engineers and Engineering Geologists to help us deliver an expanding and diverse workload of road, rail, water, tunnelling and telecommunications infrastructure, residential townships, asset management, mining, industrial, energy and commercial projects. We are heavily involved in providing geotechnical assessments and solutions for infrastructure impacted by the Canterbury earthquakes. The roles we seek range from experienced graduates through to lead positions across New Zealand.

We seek both skilled specialists and well-rounded generalist ground engineering practitioners to join us. Ideal candidates will clearly demonstrate their technical abilities through their experience and understanding in site investigation and ground characterisation, parameter evaluation, state of the art analysis and applied engineering design. A working knowledge of modern interpretative, database and analytical software is an advantage and a thorough grounding in first principles is essential.

If you are looking for a challenging and rewarding position in a forward thinking team then look no further. We offer opportunities for you to work alongside passionate, world class peers, who strive to be the best in their field.

To register your interest please apply now. Applications are sought from direct applicants only.

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The seismic group at Arup is leading the way in the application of state-of-the-art design methodologies to mitigate againstthe impact of seismic activity. Our skills allow us to provide clients with innovative nad cost effective solutions which will protect both life and capital investment. We bring together the best international practice, research and people to create a highly dynamic team to achieve these goals.

Senior Geotechnical Engineer, London and Hong Kong

The roles require significant earthquake engineering expertise and an extensive working knowledge of earthquake codes on practice, the process of seismic hazard and liquefaction assessment, sismic foundation design, dynamic soil structure interaction, performance based design and other related fields. These are key positions in the seismic group and we are looking for candidates who are self motivated and can provide leadership, inspiration and technical guidance to a growing team.

Junior Geotechnical Engineer, London

The role will be based in the specialist sub-field of earthquake engineering where you will work on a wide variety of projects including the Energy, Transport, Building and Risk sectors. Higher degree (MSc or PhD) in geotechnical earthquake engineering or a related subject is desirable.

For further information or to apply, please contact ivan.harrison@arup.com quoting @Seismic@ in the email.

We are committed to equal opportunities **www.atup.com**

ΑΝΑΣΚΟΠΗΣΗ ΓΕΩΤΕΧΝΙΚΩΝ ΕΚΔΗΛΩΣΕΩΝ

Landslides and Geo-Environment Geotechnical Symposium in Balkan Region

Κατά το διάστημα 20 έως 22 Οκτωβρίου 2011 έλαβε χώρα στα Τίρανα της Αλβανίας το "Geotechnical Symposium in Balkan Region" με θέμα "Landslides and Geo-Environment" στο Polis University των Τιράνων. Το Συνέδριο διοργανώθηκε από την Albanian Geotechnical Society υπό την Προεδρεία της Καθ. L.Bozo και την αιγίδα της ISSMGE.

Κατά τη διάρκεια του Συμποσίου παρουσιάστηκαν έντεκα προσκεκλημένες διαλέξεις (keynote lectures) από τους: Καθ. Jean Louis Briaud, Καθ. Pedro Seco e Pinto, Καθ. Ivan Vanicek, Καθ. Ανδρέα Αναγνωστόπουλο, Καθ. Mario Del Prete, Καθ. Iacint Manoliu, Καθ. Alfred Frasheri, Καθ. Fioreta Luli, Καθ. Antonio Federico, Καθ. Violeta Mircevka και τον Καθ. Sanda Manea. Παρουσιάστηκαν επίσης και 33 εργασίες.

Η Ελληνική συμμετοχή στο Συμπόσιο περιελάμβανε και τις δημοσιεύσεις:

- Μία προσκεκλημένη Διάλεξη:

  Α. Αναγνωστόπουλου και Γ. Μπελόκα με τίτλο "The Stability of Natural and Cut Slopes in Stiff Clays", η οποία παρουσιάστηκε από τον Καθηγητή Α. Αναγνωστόπουλο

- Τρεις εργασίες:

  Π. Βέττα, Μ. Παχάκη, Γ. Χλιμίντζα, Κ. Πλύτα, Α. Μπαλτζόγλου, Χ. Κουταλιά και Α. Αναγνωστόπουλου με τίτλο "Landslide Stabilization at the Egnatia Odos Highway, Greece – Instrumentation, Remediation Options and Numerical Modeling" , η οποία παρουσιάστηκε από τον Δρ Γ. Χλιμίντζα

  Κ. Σταματόπουλου, Π. Πετρίδη, Λ. Μπάλλα, Ι. Παρχαρίδης, Μ. Φουμέλης, Δ. Φουντούλης, Σ. Λαλέχος, Χ. Μεταξάς με τίτλο "Predicting Ground Subsidence Induced by Pumping Combining Space Measurements and Geotechnical Modelling: Application in the Thessaly Region, Greece", η οποία παρουσιάστηκε από την κα Λ. Μπάλλα.

  Κ. Σταματόπουλου, Π.Πετρίδη, Λ. Μπάλλα, S. Allkja, L. Bozo, Γ. Βατσέλα, Γ. Λουκάτου, A. Modaress, F. Lopez – Caballero, A. Small με τίτλο "Preloading to Mitigate Seismic Liquefaction Risk", η οποία παρουσιάστηκε από την κα Λ. Μπάλλα.

  Κ. Σταματόπουλου, Π.Πετρίδη, Λ. Μπάλλα, Ι. Παρχαρίδη, Μ. Φουμέλη, Ν. Νικολάου, Ν. Σπανού με τίτλο "Use of Displacement Space Measurement in Geotechnical Engineering: Application at Kerasia Slide", η οποία παρουσιάστηκε από την κα Λ. Μπάλλα.

Τις εργασίες του συμποσίου παρακολούθησαν μεγάλος αριθμός Συνέδρων από τις Βαλκανικές χώρες, καθώς και Σύνεδροι από την Αγγλία και Ιταλία. Η όλη διοργάνωση ήταν επιτυχής, υπήρχε παράλληλα μία ενδιαφέρουσα "Τεχνική Έκθεση", οι δε κοινωνικές εκδηλώσεις του Συμποσίου (υποδοχής και Gala) ήταν άριστα διοργανωμένες και διακρίθηκαν από ένα άριστο πνεύμα φιλοξενίας. Η τελευταία μέρα του Συμποσίου αφιερώθηκε σε Τεχνικές Επισκέψεις και Επισκέψεις σε ενδιαφέροντες Αρχαιολογικούς Χώρους της Αλβανίας.

ΠΡΟΣΕΧΕΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΚΔΗΛΩΣΕΙΣ

Για τις παλαιότερες καταχωρήσεις περισσότερες πληροφορίες μπορούν να αναζητηθούν στα προηγούμενα τεύχη του «περιοδικού» και στις παρατιθέμενες ιστοσελίδες.

Spritzbeton - Tagung 2012
12. und 13. 1. 2012, Alpbach, Tyrol, Austria
www.spritzbeton-tagung.com

Prof. Wolfgang Kusterle and his team welcome you to the Conference and Exhibition Shotcrete2012 at the Alpbach Conference Centre, Tyrol, Austria, January 12 and 13, 2012. Knowledge and experience do not help, if they remain hidden. This platform has gathered shotcrete specialists for twenty years, in a surrounding field where the exchange easily takes place.

Sekretariat
Agneta Kusterle
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Tel +43 (0)650 8244610
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Veranstalter und Tagungsleiter
Prof. Dipl.-Ing. Dr. techn. habil. Wolfgang Kusterle

cs ∞

Ετήσια Επιστημονική Συνεδρία ΕΓΕ, 6 Φεβρουαρίου 2012, Αθήνα, http://www.geosociety.gr

4th International Conference on Grouting and Deep Mixing, February 15-18, 2012, New Orleans, Louisiana, USA, www.grout2012.org

3rd International Seminar on Earthworks in Europe, 19 – 20 March, 2012, Berlin, Germany, www.fgsv.de/veranstaltungen_international.html?&tx_jullee vents_pi1[showUid]=85&cHash=4153b585bc

Practices and Trends for Financing and Contracting Tunnels and Underground Works, 22-23 March 2012, Athens, www.tunnelcontracts2012.com

6th Colloquium "Rock Mechanics - Theory and Practice" with "Vienna-Leopold-Müller Lecture", 22-23 March 2012, Vienna, Austria, christine.cerny@tuwien.ac.at

GeoCongress 2012 State of the Art and Practice in Geotechnical Engineering, Oakland, California, USA, March 25-29, 2012, www.geocongress2012.org

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UNDER CITY
Colloquium on Using Underground Space in Urban Areas in South-East Europe
April 12 -14, 2012, Dubrovnik, Croatia
www.undercity2012.com

Using of underground space is a modern way of protecting environment where we live. It requires a general public understanding and knowledge about advantages, procedures and technologies that should be applied. Therefore it needs time and effort to learn society and to present new ideas and technologies to people that have to accept new solutions.

In that sense the use of underground space in urban areas is the most difficult task that can be performed in civil enginnering. Because it needs a lot of time and a lot of effort: to show, to learn, to educate, to convince , to prepare, to start, to develop underground projects in cities. Today we look to the region of Soth-East Europe where is a enormous potential to develop infrastructure and especially undergorund infrastructure. To be on time and ready to change all ideas in to projects we would like to support this develpmet by organizing the colloquium on using of underground space in urban areas in the region. Just as the start.

The Colloquium will be organized by ITA Croatia - Croatian Association for Tunnelling and Underground Structures with the support of members of ITA from all other neighbouring countries and with the cover of sponsoring by ITA-AITES.

SPECIFIC TOPICS:

1. **Planning, Design and Geological Baseline** – easibility and planning, structural concept and operation purpose, functional assessment, environmental circumstances, inputs to design, risk and cost estimation, influence of life time cycles on condition of structures, hereditary influence, design measures on old structures

2. **Urban Traffic Development using Underground -** traffic development using underground space, metro systems, underground city railway, underground roads, garages and underground railway depots

3. **Underground Structures for Storage, Energy, Water Supply and Sewer -** cable tunnels, sewers , water supply systems, storages for water, fuel, gas and others.

4. **Construction Methods and Technologies, Maintenace and Rehabilitation** – new materials, new technologies, construction methods providing long term quality level, maintenance , reconstruction and rehabilitation technologies

5. **Fire Protection, Safety, Equipment of Underground Structures** – application of fire protection design, safety approaches and decision-based safety procedures, damage scenarios and hazard evaluation, ventilation and electro- mechanical equipment elements and systems

6. **Contractual Development and Project Preparation in Using Urban Underground Space** - PR techniques by project development, project preparation and development, project financing, cost control, project sustainability

Two themes of the colloquium will be introduced with special attention and introduction:

7. **Using Underground Space (under cover of: ISOCARP)** - underground urbanism as a part of city plans and traffic studies, underground cities, protection of ecology

through using of underground space, better cities by using underground

8. **Immersed Tubes – (under cover of: ITA-AITES-WG11)** - immersed tubes in cities, immersed technology for connections in cities on the coast, connections land-islands, application in seismic regions

Contact us on+385 (0)51 410 447 or email tanja.rabar@hubitg.com

Conference on the Mechanical Behavior of Salt, 16 – 19 April 2012, MINES ParisTech, Paris, France, www.saltmech7.com

TERRA 2012 XIth International Conference on the Study and Conservation of Earthen Architecture Heritage, 22 – 27 April 2012, Lima, Peru, http://congreso.pucp.edu.pe/terra2012/index.htm

UNDERGROUND INFRASTRUCTURE & DEEP FOUNDATIONS SAUDI ARABIA

22 - 24 April, 2012, Jeddah, Saudi Arabia, Middle East
www.undergroundfoundationsaudi.com

Underground Infrastructure and Deep Foundations Saudi Arabia will highlight the key solutions to the latest challenges in KSA's deep foundations and underground infrastructure networks.

This event will feature technical presentations from key experts, demonstrating best-practices in deep foundations, piling and underground infrastructure implementation and construction.

To find out about discounts, email enquiry@iqpc.ae or call +971 4 364 2975.

3rd International Conference on Shaft Design and Construction 2012

24 - 26 Apr 2012, London, UK
www.iom3.org/events/sdc2012

Following on from the UK conferences Shaft Sinking and Tunnelling 1959 (53 years ago) and Shaft Engineering 1989 (23 years ago), which covered the topic globally, this conference is again targeted at design and construction professionals worldwide. The scope will include both civil engineering shafts and mine shafts and all areas of design and construction will be considered.

It is anticipated that the programme will cover:

- Planning
- Geotechnical and hydrogeological data acquisition
- Ground temporary support
- Permanent lining design
- Construction materials
- Excavation methods
- Ground water control
- Shaft furnishings
- Rehabilitation
- Performance monitoring
- Risk management
- Case studies

Organiser details:
IOM Communications on behalf of the Mining Technology Division of the Institute of Materials, Minerals and Mining
Contact Name: Paul Harris
Email: paul.harris@iom3.org
Organiser Phone: +44 (0)20 7451 7302

GEOAMERICAS 2012 II Pan-American Congress on Geosynthetics, Lima, Perú, 2 - 5 May 2012 www.igsperu.org

16th Nordik Geotechnical Meeting, 9-12 May, 2012, Copenhagen, Denmark www.ngm2012.dk

Second Southern Hemisphere International Rock Mechanics Symposium SHIRMS 2012, 14–17 May 2012, Sun City, South Africa, www.saimm.co.za

ITA-AITES WTC 2012 "Tunnelling and Underground Space for a global Society", Bangkok, Thailand, 18 to 23 May, 2012, www.wtc2012.com

Fifth International Symposium on Contaminated Sediments: Restoration of Aquatic Environment, May 23 - 25 2012, Montreal, QC, Canada, www.astm.org/SYMPOSIA/filtrexx40.cgi?+-P+EVENT_ID+1857+/usr6/htdocs/astm.org/SYMPOSIA/callforpapers.frm

EUROCK 2012 - ISRM European Regional Symposium - Rock Engineering and Technology, 27 – 30 May 2012, Stockholm, Sweden, www.eurock2012.com.

SECOND INTERNATIONAL CONFERENCE ON PERFORMANCE-BASED DESIGN IN EARTHQUAKE GEOTECHNICAL ENGINEERING, May 28-30, 2012, Taormina, Italy, www.associazionegeotecnica.it

INTERNATIONAL SYMPOSIUM & SHORT COURSES TC 211 IS-GI Brussels 2012 Recent Research, Advances & Execution Aspects of GROUND IMPROVEMENT WORKS, 30 May – 1 June 2012, Brussels, Belgium, www.bbri.be/go/IS-GI-2012

12th Baltic Sea Geotechnical Conference "Infrastructure in the Baltic Sea Region", Rostock, Germany, 31 May – 2 June, 2012, www.12bsgc.de

80th Annual Meeting - 24th ICOLD Congress, June, 2nd to 5th, 2012 - June, 6th to 8th, 2012, Kyoto, Japan, http://icold2012kyoto.org/

ISL 2012 NASL 11th International Symposium on Landslides, 3 ÷ 8 June 2012, Banff, Alta, Canada, corey.froese@ercb.ca, www.ISL-NASL2012.ca

☙ ❧

The International Symposium on Sustainable Geosynthetics and Green Technology for Climate Change (SGCC2011), which also serves as the Retirement Symposium of Prof. Dennes T. Bergado, will be **rescheduled on 20 and 21 June 2012** instead of 7 and 8 December 2011 due to difficulties of the current flood calamity in Bangkok. It will be held at the Grand Centara Convention Hotel, Bangkok, Thailand. This Symposium is hosted by the Asian Center for Soil Improvement and Geosynthetics (ACSIG) in the Geotechnical and Earth Resources Engineering Program (GTE) under the School of Engineering and Technology (SET) at the Asian Institute of Technology (AIT), the Southeast Asian Geotechnical Society (SEAGS), the International Geosynthetics Society-Thailand Chapter (IGS-Thailand), and Suranaree University of Technology (SUT), under the auspices of the International Geosynthetics Society (IGS).

www.set.ait.ac.th/acsig/sgcc2011

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46th U.S. Rock Mechanics Geomechanics Symposium, Chicago, USA, 24 – 27 June 2010, www.armasymposium.org

XII International Symposium on Environmental Geotechnology. Energy and Global Sustainable Development "Unveiling the Pathways to Global Sustainability", Los Angeles, USA, June 27 – 29, 2012, www.isegnet.org/2012/

ASTM Symposium on Dynamic Testing of Soil and Rock: Field and Laboratory, June 28 - 29 2012, San Diego, CA, USA, www.astm.org/D18symp0612.htm

Protection and Restoration of the Environment XI July 3-6, 2012, Thessaloniki, Greece, www.pre11.org

Shaking the Foundations of Geo-engineering Education, International Conference on Geotechnical Engineering Education, 4-6 July 2012, NUI Galway, Galway, Ireland, bryan.mccabe@nuigalway.ie

ANZ 2012 "Ground Engineering in a Changing World" 11th Australia-New Zealand Conference on Geomechanics, Melbourne, Australia, 15-18 July 2012, www.anz2012.com.au

A Symposium on EXPERIMENTAL STUDIES WITH GEOSYNTHETICS In Conjunction with 15th INTERNATIONAL CONFERENCE ON EXPERIMENTAL MECHANICS (ICEM15), Porto, Portugal, July 22-27, 2012, http://paginas.fe.up.pt/clme/icem15

Geotechnique Themed Issue 2012 "Offshore Geotechnics", www.geotechnique-ice.com

34th International Geological Congress 5 ÷ 15 August 2012, Brisbane, Australia, http://www.ga.gov.au/igc2012

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ISRM Regional Symposium
II South American Symposium on Rock Excavations
8 – 10 August 2012, San Jose, Costa Rica

Contact Person: Marlon Jiménez
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EYGEC 2012 Gothenburg
22nd
European Young Geotechnical Engineers Conference
Gothenburg, Sweden, August 26th to 29th, 2012
www.sgf.net

Invitation

On behalf of the ISSMGE and the Swedish Geotechnical Society the organizing committee have the pleasure of inviting you to Gothenburg, Sweden where the 22nd European Young Geotechnical Engineering Conference will be held from Sunday 26th of August to Wednesday the 29th of August.

All sessions will be held at Chalmers University of Technology which is located in the centre of Gothenburg. The city of Gothenburg is Sweden's second largest city and is situated on the West coast. For more information about the city refer to www.goteborg.com.

Conference Topics

Papers covering the following topics will be accepted:

* Site investigations and laboratory testing
* Design parameters and modelling
* Shallow and deep foundations
* Deep excavations and retaining structures
* Tunnelling and underground structures
* Slope stability and landslides
* Infrastructure projects
* Ground improvement
* Environmental geotechnics

Local Conference Organizing Committee

Chairman Victoria Svahn, victoria.svahn@swedgeo.se
Secretary Tara Wood, tara.wood@chalmers.se
Members Ulrika Isacsson, ulrica.isaksson@wspgroup.se
Henrik Möller, henrik.moller@tyrens.se

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ICSE-6, 6th International Conference on Scour and Erosion, 27-31 August 2012, Paris, France, www.icse-6.com

Baltic Piling Days 2012
Tallinn, Estonia, 3-5th September 2012
www.balticpiling.com

In Baltic States problems concerning pile foundations are being discussed since 1967. These meetings were called "Baltic Piling Days".

Last similar meeting took place in 1999 and it was attended by representatives from 7 different countries.

In May 2010 managing boards of Estonian, Latvian, Lithuanian and Finnish Geotechnical Societies decided to re-establish before mentioned event in interaction with ISSMGE. As a result conference concerning pile foundations will be held in Tallinn. "Baltic Piling Days" is scheduled to take place on 3-5th September 2012.

A supposed agenda of the conference stands as follows:

1. Day. Loaded piles behaviour in soil and bearing capacity of piles:

   a) vertically loaded pile and its behaviour
   b) laterally loaded pile
   c) load test and bearing capacity of piles (assessment, evaluation)
   d) dynamic method, integrity testing
   e) CPTU, DPT, SPT (application)
   f) calculations based on soil properties
   g) settlements of pile foundations and interaction of grillage and piles
   h) Reception.

2. Day. Reinforcement of deep pit and slopes.

   a) assessment of soil properties for calculations of pits and slopes
   b) main calculation schemes and accordance with parameters of calculations
   c) methods of reinforcements of pit – sheet-pile wall, secant wall, slurry wall
   d) anchors, types and bearing capacity
   e) bearing capacity of laterally loaded piles
   f) case histories
   g) interaction between pit, grillage, piles and building
   h) Banquette.

3. Day. Technologies of piles. Appliance, materials, manufacturing.
   a) driven piles
   b) bored piles
   c) micropiles
   d) slurry wall
   e) new pile technologies

Technical visit in Tallinn

4. Technical visit in Pärnu

Information and Conference Secretary
Scientific Secretary
M. Mets, mait.mets(at)gmail.com phone +372 6312402
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2nd International Conference on Transportation Geotechnics, 10 - 12 September 2012, Sapporo, Hokkaido, Japan, http://congress.coop.hokudai.ac.jp/tc3conference/index.html

7th International Conference in Offshore Site Investigation and Geotechnics: Integrated Geotechnologies, Present and Future, 12-14 September 2012, London, United Kingdom, peter.allan@geomarine.co.uk; zenon@tamu.edu

EUROGEO5 - 5th European Geosynthetics Conference, 16 - 19 September 2012, Valencia, Spain, www.eurogeo5.org

IS-Kanazawa 2012 The 9th International Conference on Testing and Design Methods for Deep Foundations 18-20 September 2012, Kanazawa, Japan, http://is-kanazawa2012.jp

ISC' 4 4th International Conference on Geotechnical and Geophysical Site Characterization, September 18-21, 2012, Porto de Galinhas, Pernambuco – Brazil, www.isc-4.com

1st Eastern European Tunneling Conference
THE TUNNEL CONNECTS

September 18-21, 2012, Budapest, Hungary
www.eetc2012budapest.com

It is our pleasure to inform you, that Hungarian Tunnelling Association (HTA) is organizing the 1st Eastern European Tunnelling Congress in Budapest on September 18-21, 2012.

The idea was recommended to the East European Associations several times at the President meeting in Harkány November 08, 2011, in Dubrovnik, April 2011 and at the final discussion at WTC, Helsinki.

The common aim of EETC 2012 is to share our experiences and exchange our knowledge of design, construction management, research results and technical developments of tunnels completed by the regional associations and experts. The planned regional sub European conference is open to all other co-organizers and participants as well as to those who having ongoing or completed projects, research works in this area.

I do hope that the planned collaboration of Eastern European associations will establish a biannual routine for EETC that will further facilitate the growth of this region's tunnelling industry and affirm professional relationships.

Tibor Horvath
President of HTA

Main Topics:

- Research
- Contractual practices in underground construction
- Health, Safety and Risk of underground structures
- Maintenance and repair of underground structures
- Immersed and floating tunnels
- Use of Sprayed Concrete
- Mechanization of underground excavations
- Underground and environment
- Long and great depth tunnels
- Training - Conventional tunnelling
- Urban problems - Underground solutions

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SAHC 2011, 8th International Conference on Structural Analysis of Historical Constructions, October 15 – 17, 2012, Wroclaw, Poland, www.sahc2012.org

7th Asian Rock Mechanics Symposium, 15-19 October 2012, Seoul, Korea, www.arms7.com

International Conference on Ground Improvement and Ground Control: Transport Infrastructure Development and Natural Hazards Mitigation, 30 Oct - 2 Nov 2012, Wollongong, Australia www.icgiwollongong.com

ACUUS 2012 13th World Conference of the Associated Research Centers for the Urban Underground Space Underground Space Development – Opportunities and Challenges, 7 – 9 November 2012, Singapore, www.acuus2012.com

32. Baugrundtagung with exhibition "Geotechnik", Mainz, Germany, 26 – 29 November 2012

GEOSYNTHETICS ASIA 2012 (GA2012) 5th Asian Regional Conference on Geosynthetics, Bangkok, Thailand, 10 - 14 December 2012, www.set.ait.ac.th/acsig/igs-thailand

First International Congress FedIGS, 12 – 15 November 2012, Hong Kong – China, www.fedigs.org/HongKong2012

GA2012 - Geosynthetics Asia 2012 5th Asian Regional Conference on Geosynthetics, 10 - 14 December 2012, Bangkok, Thailand, www.set.ait.ac.th/acsig/GA2012

Geotechnical Special Publication, ASCE "Foundation Engineering in the Face of Uncertainty". Abstracts to Mohamad H. Hussein at: MHussein@pile.com.

Geotechnical Special Publication, ASCE "SOUND GEOTECHNICAL RESEARCH TO PRACTICE", http://web.engr.oregonstate.edu/~armin/index_files/Holtz GSP

Themed Issue on Geotechnical Challenges for Renewable Energy Developments, Geotechnical Engineering 2013, ben.ramster@icepublishing.com

Pam-Am UNSAT 2013 First Pan-American Conference on Unsaturated Soils, 20-22 February 2013, Cartagena de Indias, Colombia, panamunsat2013.uniandes.edu.co

TU-SEOUL 2013 International Symposium on Tunnelling and Underground Space Construction for Sustainable Development, March 18-20, 2013, Seoul, Korea www.tu-seoul2013.org/

Fifth International Conference on Forensic Engineering Informing the Future with Lessons from the Past, 15-17 April 2013, London, United Kingdom, http://ice-forensicengineering.com

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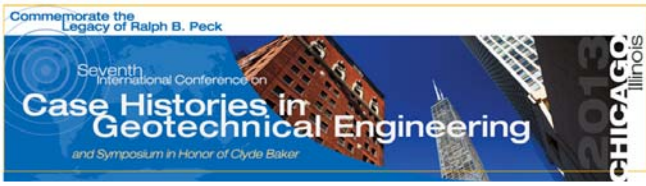

Conference to Commemorate the Legacy of Ralph B. Peck, 7th International Conference on Case Histories in Geotechnical Engineering & Soil Dynamics and Symposium in Honor of Clyde Baker, Chicago, USA, 29 April – 4 May, 2013, http://7icchge.mst.edu

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ITA-AITES WTC 2013 "Underground – the way to the future", Geneva, Switzerland, 10 to 17 May 2013, www.wtc2013.ch/congress

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Effective and Sustainable Hydraulic Fracturing - an ISRM Specialized Conference
20-22 May 2013, Brisbane, Queensland, Australia

The Conference will focus on three technical themes:

1. Advancing Effectiveness presenting the latest advances in simulation, theory, field and laboratory experimentation, and case studies with an emphasis on petroleum resources

2. Exploring Versatility presenting methods and lessons from a diversity of application domains

3. Promoting Sustainability driving toward differentiation between real and perceived risks, deployment of viable controls, and beneficial public engagement.

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STREMAH 2013
13th International Conference on Studies, Repairs and Maintenance of Heritage Architecture
25 – 27 June 2013, New Forest, UK
carlos@wessex.ac.uk

STREMAH 2013 is the 13th International Conference on Studies, Repairs and Maintenance of Heritage Architecture. The meeting has taken place on a regular basis since the first conference started in Florence (1989) and continued in Seville (1991); Bath (1993); Crete (1995); San Sebastian (1997); Dresden (1999); Bologna (2001); Halkidiki (2003); Malta (2005); Prague (2007); Tallinn (2009) and Tuscany (2011). It has become a major international event attracting specialists from all over the world.

Modern society exerts considerable strain on our architectural heritage, partly due to the increase of population and the associated demands on the built environment, and also because of the pernicious effects of pollution. The difficulties and costs of repairs and maintenance contribute to the neglect of important buildings in times of economic stress. It is hence important that our research addresses not only technical questions, but solutions that can be implements in practice, where social and economic considerations may be paramount. The STREMAH conference fulfils the important function of bringing together experts from different disciplines and from all over the world to discuss problems of common interest and put forward solutions.

The STREMAH Conference over the years has addresses interdisciplinary topics and evolved into a meeting that attracts a wider range of expertise. It has also produced many outstanding contributions, with the conference books distributed throughout the world. Furthermore, all papers published since 1993 are available mostly in open access format from the WIT eLibrary (http://library.witpress.com). They constitute a permanent record of the most significant work in the field.

The conference brings together scholars and professionals to discuss a variety of topics related to architectural and maritime heritage.

The success of the conference is due both to the importance given today to architectural heritage and the high quality of the presentations at the meetings. Each volume has gathered the most recent advances in research and up to date studies of heritage buildings. This has resulted in STREMAH becoming the most important conference of its kind.

TOPICS

- Heritage architecture and historical aspects
- Learning from the past
- Surveying and monitoring
- Performance and maintenance
- Structural restoration of metallic structures
- Preservation and monitoring
- Earth construction
- Modern (19th/20th century) heritage
- Maritime heritage
- Heritage masonry buildings
- Stone masonry walls
- Wooden structures
- Simulation and modelling
- Material characterization
- New technologies or materials
- Corrosion and material decay
- Seismic vulnerability
- Non-destructive techniques
- Assessment and re-use of heritage buildings
- Heritage and tourism
- Social and economic aspects in heritage
- Guidelines, codes and regulations for heritage

CONFERENCE CHAIRMAN

C.A. Brebbia, Wessex Institute of Technology, UK (carlos@wessex.ac.uk)

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The 6th International Symposium on Rock Stress
20-22 August 2013, Sendai, Japan

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18th International Conference on Soil Mechanics and Geotechnical Engineering "Challenges and Innovations in Geotechnics", 1 – 5 September 2013, Paris, France www.paris2013-icsmge.org

Géotechnique Symposium in Print on Bio- and Chemo-Mechanical Processes in Geotechnical Engineering, www.elabs10.com/content/2010001471/SIP%202013.pdf

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EUROCK 2013
ISRM European Regional Symposium
Rock Mechanics for Resources, Energy and Environment
23-26 September 2013, Wroclaw, Poland

Contact Person: Prof. Dariusz Lydzba
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ANDORRA 2014 14th International Winter Road Congress 2014, 4-7 February 2014, Andorra la Vella (Andorra), www.aipcrandorra2014.org

EUROCK 2014
ISRM European Regional Symposium
Rock Engineering and Rock Mechanics: Structures in and on Rock Masses
26-28 May 2014, Vigo, Spain

Contact Person: Prof. Leandro Alejano
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E-mail: alejano@uvigo.es

8th European Conference "Numerical Methods in Geotechnical Engineering", Delft, The Netherlands, 18-20 juni 2014, www.numge2014.org

10th International Conference on Geosynthetics – 10ICG, Berlin, Germany, 21 – 25 September 2014 www.10icg-berlin.com

13th ISRM International Congress on Rock Mechanics
Innovations in Applied and Theoretical Rock Mechanics
29 April – 6 May 2015, Montreal, Canada

The Congress of the ISRM "Innovations in Applied and Theoretical Rock Mechanics" will take place on 29 April to 6 May 2015 and will be chaired by Prof. Ferri Hassani.

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ΝΕΑ ΑΠΟ ΤΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΝΩΣΕΙΣ

ΕΛΛΗΝΙΚΗ ΕΠΙΤΡΟΠΗ ΣΗΡΑΓΓΩΝ ΚΑΙ ΥΠΟΓΕΙΩΝ ΕΡΓΩΝ

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Μεταλλειόγος Μηχανικός ΕΜΠ
Υπουργείο Υποδομών, Μεταφορών και Δικτύων
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ΕΝΔΙΑΦΕΡΟΝΤΑ ΓΕΩΤΕΧΝΙΚΑ ΝΕΑ

Eurocode 7 underlines need for broad knowledge

Implementation of Eurocode 7 (geotechnical) has underlined the need for engineers to have a broad knowledge and not to be too specialist, Buro Happold technical director Peter Scott said this week.

Speaking at the Construction Industry Research and Information Association (Ciria) conference on Geotechnical Issues in Construction, Scott highlighted the need for better understanding. "Too often site investigation is undertaken by geologists and the analysis by an engineer in the office," he said. "But with the arrival of Eurocodes there is a need for engineers to understand geology and for the geologists to understand design."

Eurocode 7 came into force in April last year and marked a change in style with procedures that "have to be applied" with no alternatives as were available in previous British Standards. "This prescriptive approach has been difficult for the geotechnics industry, which is part science and part art," said Scott.

"Eurocode 7 demands that there is continuity and communication between eh data collection, design and construction operations. This takes the industry back to how it was in the 1970s when design engineers had a broader knowledge and were less specialist."

Scott also highlight the difficulties of sampling in accordance with Eurocode 7 and the lack of differentiation in reliability and quality of different in situ testing methods. "The main benefit is that the Eurocodes do set a minimum level of site investigation," he said. "It also calls for a ground investigation report that includes geotechnical parameters, which removes assumptions being made further down the line."

(Claire Symes / NCE, 10 November 2011)

Caisson foundations needed for next generation of wind farms, says expert

Future deep water wind farm developments and government targets will demand a different foundation design approach from the current monopile technique used in the UK, an Oxford University expert has said.

Oxford University lecturer Byron Byrne said monopile foundation construction was too expensive and time-con-suming to allow government targets for wind energy capac-ity to be met by 2020, and other foundation designs would be more time and cost effective. Byrne was speaking at last night's Geotechnique lecture at the ICE in London.

"Construction of monopiles for the turbines currently being installed is a costly and lengthy process and to meet government plans for 33GW capacity from wind by 2020, faster foundation solutions are needed," he said. "To meet this aim around 800 wind turbines must be installed each year and a further 300 year will need to be replaced, so the industry needs to find an alternative."

Byrne said mono or multiple suction caissons could be a viable alternative as they can be installed quickly. He said he was pleased that the Dogger Bank development was to use the system for its met masts. "While a caisson solution is not suitable for every ground condition, this approach will drive down the cost and time to build wind farms," Byrne said. He urged developers to carry out field trials to prove the technology.

Wave, wind and load challenges

Byrne highlighted the issues of designing monopile foundations to cope with wave, wind and vertical loads imposed by the turbine and location.

He said future generations of wind farm projects would face increased challenges.

"Round 1 wind farms were relatively closer to shore but the current Round 3 developments are in deeper water. Round 4 sites are in deeper water still," he said.

"The natural frequency of the pile must avoid the frequencies generated by the blades and the rotational frequency," Byrnee said.

He went onto look at the issues related to the performance under cyclic loading of monopiles and the stiffness response to accumulated rotations from the various forces. He said these issues could have serious impact on fatigue in the structure and more field testing is needed to fully understand the potential problems.

(Claire Symes / NCE, 10 November 2011)

600-foot section of road quietly slips into ocean

A 600-foot section of a bluff-top roadway in San Pedro collapsed into the Pacific Ocean following heavy weekend rains, instantly carving a sheer, gaping canyon into the shoreline.

The earth and asphalt moved as a giant block, slipping away gently and swiftly about 3 p.m. Sunday, L.A. City Engineer Gary Moore said.

"This entire coast along here is a cliff," Moore told reporters Monday, standing about 25 feet from the edge of the newly formed drop-off. "So nature has created a new cliff."

The 100-foot-high coastal bluff on Paseo Del Mar between Western and Weymouth avenues has been moving slowly seaward for several months, but Sunday marked the first time that a sizable mass of earth slid away at one time.

None of the 15 or so contractors working at the site at the time was injured. Authorities say no structures or homes in a nearby neighborhood are at risk.

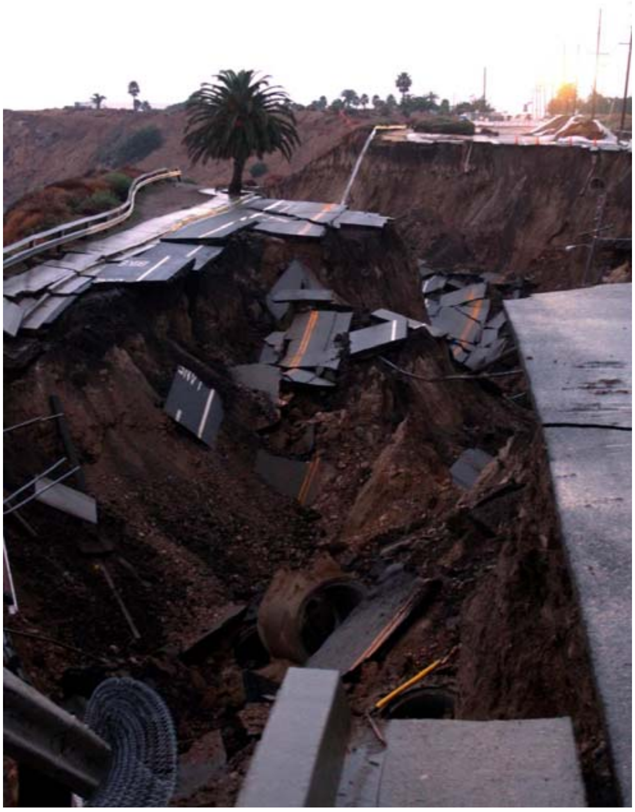
One worker who was on scene Sunday reported the slide was nearly silent, with the earth slipping away in a matter of seconds, L.A. County Department of Public Works area engineer Hector Bordas said. Before the worker could even grab his camera, it was over.

The slide transformed what was once a flat roadway into a chasm up to 75 feet deep, its bottom littered with chunks of as-phalt and concrete, fallen utility poles, railings, electrical wiring and concrete pipes.

"Someone called it a mini-Grand Canyon," said Richard Lee, a spokesman for the public works department. "I wouldn't go that far, but it doesn't look good."

In a statement, Los Angeles Mayor Antonio Villaraigosa called it a "significant landslide" and urged residents and visitors to keep away from the stretch of roadway, which has been closed since September.

"The landslide area remains unstable and presents a life-threatening hazard," he said.



Citing safety concerns, the city last week built a chain-link fence around the site and boosted police patrols to keep out trespassers, vandals and spectators.

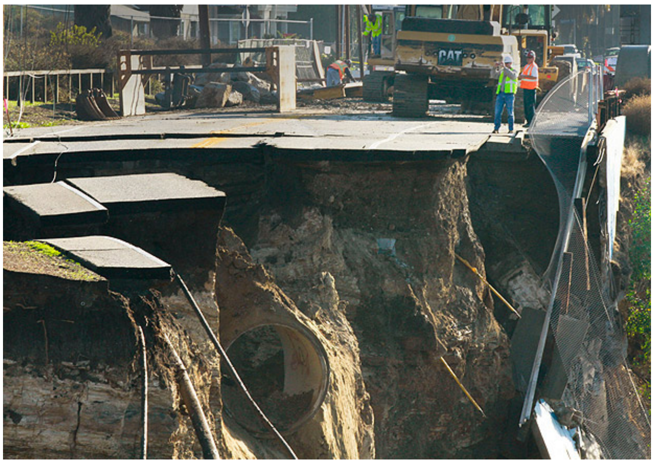
Still, the barricades and warning signs didn't seem to deter dozens of gawkers who gathered Monday to get a glimpse of the destruction by craning their necks along the 8-foot-high fence and peering down from a hillside high above.

U.S. Rep. Janice Hahn (D-San Pedro), a former L.A. councilwoman who lives a few blocks from the site of the slide, said she would ask the governor to consider a disaster declaration and would see if federal emergency funds could be secured.

"It's gone, it's gone. It's fallen into the ocean and it's devastating to me," Hahn told KTLA news. "I'm very concerned about the homes that are just a few feet away and hoping that we're going to figure out a way to stabilize this and not have it go any further."

City engineers don't know exactly what is causing the instability, though "rain doesn't help," Moore said. Whether the bluff will hold its new boundaries or continue to crumble away remains to be seen, he said.

To determine the extent of the slide and, eventually, what might have caused it, the city has hired a geotechnical consultant. In the coming weeks the firm, Shannon & Wilson, Inc. will drill 80 to 100 feet below the ground, taking soil samples and analyzing them in a lab to find the bluff's failure points. The work will cost at least $100,000.



City workers are scrambling to erect an 8-foot fence around the landslide area to keep onlookers from getting close. A 900-foot stretch of Paseo del Mar, shown, has been closed.

The city began monitoring the unstable bluff last spring when cracks opened up on the coastal roadway. The land started sinking, buckling and moving slowly toward the ocean several months ago and had accelerated in recent weeks, exposing huge crevices in the road and adjacent White Point Nature Preserve.

There's no predicting when it might slide again, officials said.

As a precaution, the city plans to keep a close eye on the slide during the Thanksgiving holiday, when more rains are forecast.

Such a dramatic landslide isn't without precedent on the Palos Verdes Peninsula. The coastline has long been prone to geological failure because of slippery rock formations that dip toward the sea and ocean waves that wear away at the base of coastal bluffs.

Crews from L.A.'s Department of Water and Power cut a power line connected to a pole that is now at the bottom of a giant landslide along a large section of West Paseo Del Mar between South Western and South Weymouth avenues in San Pedro.

The so-called "Sunken City," a residential area that started collapsing in 1929, is just a few miles away.

Portuguese Bend, just up the coast in Rancho Palos Verdes, has been geologically unstable since the 1950s, when construction triggered a landslide that destroyed more than 100 homes. A 1999 landslide in the area sent the 18th hole of a golf course crashing into the ocean.

(Tony Barboza / Los Angeles Times, November 22, 2011)

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΣΕΙΣΜΟΙ

New seismic inventory identifies potentially unsafe buildings

As many as 17,000 older concrete buildings in California could be vulnerable during a major earthquake, according to a new inventory by a coalition of volunteer structural engineers, universities and government agencies.

A number of schools, state and local government buildings, and other vital infrastructure – such as police stations and hospitals – made the list.

During the San Fernando, Loma Prieta and Northridge earthquakes, several concrete buildings constructed before the implementation of modern codes collapsed or were catastrophically damaged.

"One of the problems with the concrete buildings is that they tend to be large," said Craig Comartin, director of the Concrete Coalition project, a group of volunteer structural engineers that wrote the report (http://www.eeri.org/wp-content/uploads/Concrete_Coalition_Final_0911.pdf).

"When you have a major apartment building, there could be hundreds of people in the building. When one does come down, the potential for deaths or injuries is high."

The coalition looked at 23 counties with the highest earthquake risk and population, as well as two cities – Fresno and Bakersfield. The group estimates there are between 16,000 and 17,000 potentially vulnerable concrete buildings in the state. More detailed information for each county surveyed is available at the coalition's website (http://www.concretecoalition.org/?page_id=260&page=california_counties).

Not all of the concrete buildings identified are collapse hazards or prone to severe earthquake damage, the report notes. The next stage of the project will be a more careful study of specific buildings in order to better understand which are the riskiest structures.

Unlike unreinforced masonry buildings, which structural engineers say uniformly do not perform well in earthquakes, there's tremendous variability in older concrete buildings.

Understanding what makes a concrete building vulnerable is one of the goals of a National Science Foundation-funded research project at the Pacific Earthquake Engineering Research Center. Under the leadership of Jack Moehle, a civil engineering professor at UC Berkeley, researchers are working on ways to quickly weed out which buildings may need more detailed evaluation and possibly a retrofit.

"If we can provide the tools that would help identify which are the highest risk, then you can develop programs that target those higher-risk buildings," Moehle said.

Some members of the business community have said they're interested in helping mitigate the risk, but expressed concerns about the cost.

"We as an industry group would be happy to get information out," said Martha Cox-Nitikman, senior director for public policy and education for the Building Owners and Managers Association of Greater Los Angeles. But some owners have told her that if required to retrofit the building, they might just tear it down.

It's unclear what the next steps might be to address the older concrete buildings. The state Seismic Safety Commission's most recent attempt to address concrete structures statewide was more than two decades ago, through a bill by the late Sen. Al Alquist.

"We'll have more tools to make better decisions, but at this point, we don't have enough tools in place for me to even surmise," said Fred Turner, the top structural engineer with the Seismic Safety Commission. "We feel a lot more confident to help make informed decisions now than we did back in the mid-'80s in that we have a lot of publications that talk about the financial and social implications of retrofit programs, whether they be voluntary or mandatory."

(Kendall Taggart / CALIFORNIA WATCH, October 31, 2011)

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΛΟΙΠΑ

'Fracking' report is released as protesters halt work at drill site

On-shore gas drilling in Lancashire was the 'highly probable' cause of earth tremors detected in the area, a study has found.

The report into the controversial shale gas hydraulic fracturing or 'fracking' operation by British firm Cuadrilla Resources was released this morning as a group of protesters halted work at the company's drilling site near Southport, Merseyside.

The seismic events, including one in April of 2.3 on the Richter scale and one in May of 1.5, were caused by an 'extremely rare' combination of geological factors at the well site and would be unlikely to happen at other locations, the study said.

But the report also noted that systems to monitor seismic activity trigged by fracking — which involves injecting water and chemicals into the ground at high pressure — are already in place at sites in the Netherlands and Germany, and could be used to prevent the escalation of any future seismicity.

Mark Miller, Cuadrilla's chief executive, said: 'We unequivocally accept the findings of this independent report and are pleased that the report concludes that there is no threat to people or property in the local area from our operations.

'We are ready to put in place the early detection system that has been proposed in the report so that we can provide additional confidence and security to the local community.

'Cuadrilla is working with the relevant local and national authorities to implement the report's recommendations so we may safely resume our operations.'

The report also said that if future drill sites did have similar geology then any seismic events as a result of fracking would be limited to around magnitude 3 on the Richter scale as a 'worst-case scenario'. Activity of less than 2 on the scale can only be detected by seismographic equipment.

Nine protesters from the opposition group Frack Off stormed Cuadrilla's drilling site early this morning. Several scaled the drilling rig using climbing equipment and unfurled banners. They say they plan to stay as long as possible to stop the drilling.

The Southport operation is the first hydraulic fracturing site in the UK. But fracking is well established in the US and has been the subject of much debate due to fears that the process is leading to increased seismic activity and drinking water contamination.

Jenny Boykin, a spokesperson for Frack Off, said: 'Fracking uses huge amounts of water mixed with toxic chemicals, a large fraction of which are never recovered.

'The fracking fluid also leaches chemicals such as arsenic out of the rocks when it is used, making it even more toxic and so the fluid that is recovered becomes a big disposal problem.'

However, Cuadrilla has insisted that its methods are designed to prevent the problems experienced in the US and that the UK's drilling regulations are some of the strictest in the world.

Cuadrilla commissioned the report in consultation with the Department of Energy and Climate Change (DECC). The research was carried out by academic seismic experts directed by Dr Hans de Pater of the Delft University of Technology in the Netherlands. The firm intends to seek a peer review of the report.

A DECC spokesman said: 'The implications of this report will be reviewed very carefully — in consultation with the British Geological Survey, independent experts, and the other key regulators, HSE and the Environment Agency — before any decision on the resumption of these hydraulic fracture operations is made.'

(The Engineer, 2 November 2011)

Fracking-induced tremors are 'extremely rare', says report

The report into Cuadrilla Resources' hydraulic fracturing (fracking) operation in Lancashire found that it probably did cause earth tremors, but the study has also highlighted the rarity of such an event and the relatively small scale of the seismic activity.

The seismic activity occurred when Cuadrilla injected water and chemicals at high pressure — the key part of the fracking process to release shale gas — directly into a tectonic fault that, very unusually for a geological feature of this kind, had some natural permeability.

It was the fact that the fault accepted the fluid that caused the earth tremors, and Stefan Baisch, one the report's authors and general manager of German deep-drilling research firm Q-Con, told *The Engineer* it had been a worst-case scenario for Cuadrilla.

'There have been more than a million similar treatment operations in the world over the last 50 years or so and there are only two cases where similar seismic reactions occurred,' said Baisch, who has a PhD in seismology and has spent 10 years researching induced seismicity.

'With hindsight, with our knowledge now that a fault in the Bowland shale can react like this, it might have been possible that the Preese Hall-1 [well site] would not have been picked because of seismic risk concerns.'

But the rarity of such an event meant that fracking-induced seismic activity was not seen as a concern before drilling started, he said. Although there is a chance the fault could have been detected with a seismic pre-survey, it wouldn't have provided a 100 per cent guarantee of identifying it unambiguously.

However, Prof Stuart Haszeldine, a geologist at Edinburgh University pointed to the relatively high amount of stress in UK rockbeds.

'That stress often needs just a small trigger to make a small earthquake,' he said. 'It's also very hard to see where pre-existing faults are deep down in this location, so predicting how to stay away from future faults is very difficult indeed.

'Although this is a very thorough report, I would be very cautious before proceeding with more commercial drilling. More measurements of deep present-day stresses at different locations in the basin are needed to enable more accurate predictions.'

'Traffic light' warning system

The answer, according to the Cuadrilla-commissioned report, is to use smaller volumes of fluid and to deploy sensitive seismic monitors just below the surface in the area around any future well sites to act as a 'traffic light' warning system if seismic activity reaches a threshold of 1.7 on the Richter scale.

Given that any seismic activity is likely to be bigger the further into the rock the fracking fluid progresses, such monitoring should provide an early warning if minor tremors are detected and the operation can be halted and the pressure released before stronger activity occurs.

These systems have been used to monitor gas exploitation in the Netherlands and have become mandatory in some German states for geothermal drilling, but while they are increasingly widely used, they are not yet a standard precaution for fracking.

Baisch said that using such a traffic light would provide extra reassurance that fracking was safe with regards to seismic activity.

'If we don't know with a 100 per cent guarantee, we put in an extra safety factor,' he said. 'There's always some uncertainty with sub-surface parameters, and with this traffic light system we are sure, even if our expectations about the geological conditions are wrong, that nothing could happen.'

Quake magnitude

Even if further seismic activity were to occur due to fracking, the report also says that it would only ever reach 3 on the Richter scale. The largest event recorded in Lancashire was 2.3 on the scale.

Dr Brian Baptie, head of earthquake seismology at the British Geological Survey, said: 'The proposed maximum acceptable earthquake of a magnitude 2.6 earthquake might, at a depth of 3km, result in an intensity of shaking that would not be expected to cause any damage but would be widely felt by people indoors and out, and may displace objects on shelves.'

Prof Quentin Fisher, professor of petroleum geoengineering at Leeds University, went a step further and said: 'Although hydraulic fracturing always causes microseismic events (i.e. low magnitude earthquakes) I've never heard of large magnitude earthquakes being caused by hydraulic fracturing.

'Indeed, on a theoretical basis, there are reasons to believe that production of gas from shales could actually reduce the longer-term risk of larger magnitude earthquakes. The magnitude of the events at Blackpool was very low — so much so that a passing train would be more noticeable.'

As Baisch points out, this activity is also comparable to activity caused by Britain's substantial mining operations in the past. 'You had thousands of mining-induced seismic events and some of them were up to 3.1 and occurred at a depth of 1km only,' he said. By comparison, fracking tends to occur at depths of at least 1.5km (5,000ft) and the further down the cause of the activity, the less it is felt on the surface.

However, even if the government decides there are sufficient precautions in place to prevent fracking-induced seismic activity in the future, the use of hydraulic fracturing will remain a controversial one, notably due to concerns over drinking water contamination and the importance of moving away from fossil-fuel sources of energy.

(Stephen Harris / The Engineer, 3 November 2011)

Τι είναι το fracking;

Reporting on energy supply is rarely straightforward given the variables that inform the debate.

Throw shale gas into the discussion and opinions often become militantly polarized.

So-called shale or tight gas is extracted from shales using a process called fracking, which - in its simplest terms – involves injectingwater, sand and chemicals into the ground at high pressure to crack the shale rock and release trapped gas.

Advocates believe the process is safe and could unlock around 200 trillion cubic feet of gas whilst opponents claim it is linked to water contamination, health problems and earth quakes.

Both sides are going to come face to face this Wednesday at the Shale Gas Environmental Summit in London.

Event organisers SMi Conferences say the event will examine existing, current and proposed shale gas extraction, with a focus on the associated environmental impacts.

The two day conference will then examine the risks associated with extraction, environmental benefits and challenges, public opposition and support, responsible development and product management.

Opponents from Frack Off, the national anti-fracking network, claim the event is nothing more than 'greenwash' designed to attract investors. They will gather at 1500 on Wednesday to disrupt the event and hold a "people's assembly" to discuss the environmental impacts of fracking.

In a statement, Sophie Choudri, 24, a member of Frack Off, said: 'This conference is all about spin. It is just the fossil fuel industry PR machine trying to tell the people in power that they should let them make lots of money.'

In May this year the UK's Energy Select Committee published a report saying it had found no evidence to suggest that hydraulic fracturing poses a risk to underground water aquifers, provided the drilling well is constructed properly.

Last week, however, energy secretary Chris Huhne distanced himself from shale gas when he said it 'has not yet lit a single room nor cooked a single roast dinner in the UK.'

'Shale gas may be significant,' he said during the Renewable UK conference. 'It is exciting. But we do not yet know enough to bet the farm on it. Faced with such uncertainty we do what any rational investor does with their own pension fund – we spread our risks, we have a portfolio.'

Skills are on the agenda this week at IMechE who host a lecture entitled 'What's your solution for the skills gap? this Wednesday.

Attendees will hear about the Equality Act and its effects on engineering. They will also be able to discuss the ways engineers themselves can broaden engagement with engineering, thereby helping to address the skills gap.

According to the event's publicity material Education for Engineering (E4E) will give a presentation before the debate designed to share the initial results of their research into the diversity profiles of those doing STEM qualifications at age 16.

Sheffield's Business and Innovation Network is hosting the second in a series of free annual events designed to stimulate discussion, promote the sharing of best practice and support collaboration across disciplines in science and between research and industry.

Starting Wednesday, the three day event includes workshops, exhibitions and networking opportunities.

The organizers say the first day has attracted an international line up of speakers to set the context by highlighting current political and funding priorities as well as the importance of universities engaging with industry to translate research into economic and wider social benefits.

Day two and three of the event are focused around moderated and themed workshops and includes a workshop on offshore wind power, asking whether Britain can rely on offshore wind energy.

Finally, tomorrow marks the introduction of a new £50 bank note into circulation that bears portraits of James Watt and Matthew Boulton. The pair formed Boulton & Watt in 1775, an excellent example of how private investment (and patience) can help deliver revenue via innovation.

(Jason Ford, News editor / The Engineer, 31 October 2011)

Μυστήριο στην άμμο

Νεκροταφείο απολιθωμένων φαλαινών αποκαλύφθηκε στην έρημο Ατακάμα

Φάλαινες στην έρημο: σπάνιο θέαμα, ακόμα και για τους παλαιοντολόγους (Φωτογραφία: Associated Press)

Ερευνητές του Ινστιτούτου Smithsonian εργάζονται πυρετωδώς για την απομάκρυνση των δεκάδων σκελετών από προϊστορικά κήτη που βρέθηκαν τυχαία στην έρημο Ατακάμα της Χιλής.

Τα οστά περίπου 75 φαλαινών και άλλων θαλάσσιων θηλαστικών ήρθαν πέρυσι στο φως κατά τη διάρκεια εργασιών για την επέκταση του Παναμερικανικού Αυτοκινητόδρομου. Ο εργολάβος του δρόμου έδωσε στους ερευνητές το περιθώριο να απομακρύνουν τα ευρήματα πριν οι εργασίες ξαναρχίσουν.

Οι σκελετοί, διάσπαρτοι σε μια έκταση όσο δύο γήπεδα ποδοσφαίρου, βρέθηκαν αρκετά χιλιόμετρα από τις ακτές του Ειρηνικού, κοντά στην πόλη του Καλντέρα. Οι παλαιοντολόγοι ακόμα ξύνουν τα κεφάλια τους για το πώς βρέθηκαν μαζεμένες οι φάλαινες στην έρημο.

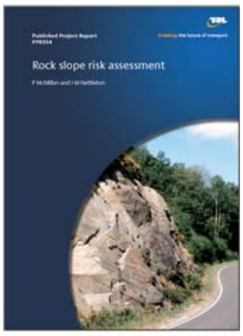
Μια πιθανή εξήγηση είναι ότι τα κήτη εξόκειλαν μαζικά πριν από 2 έως 7 εκατομμύρια χρόνια, όταν η ακτή βρισκόταν πιο βαθιά στην ενδοχώρα. Αργότερα το έδαφος ανυψώθηκε, μετατοπίζοντας την ακτή χιλιόμετρα μακριά.

Μια δεύτερη εξήγηση, την οποία θεωρούν και πιθανότερη οι ερευνητές, είναι ότι οι φάλαινες παγιδεύτηκαν σε μια ρηχή λιμνοθάλασσα λόγω κάποιας καταιγίδας ή μεγάλης κατολίσθησης.

Ελπίζοντας να λύσει το μυστήριο κάποια στιγμή στο μέλλον, το Smithsonian χρησιμοποιεί ειδικό εξοπλισμό για να χαρτογραφήσει την περιοχή σε τρεις διαστάσεις.

(Newsroom ΔΟΛ, με πληροφορίες από Associated Press, 22 Νοε. 2011)

ΝΕΕΣ ΕΚΔΟΣΕΙΣ ΣΤΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΠΙΣΤΗΜΕΣ



Rock slope risk assessment

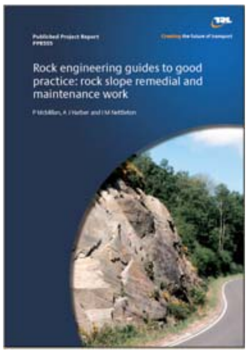
Published Project Report PPR554

P McMillan and I M Nettleton

The management of rock slopes requires knowledge of their location, traffic levels and other geometric parameters as well as the level of the hazard posed to the road user. This information can then be used to prioritise remedial action. This report details a system that was developed to allow such assessment and prioritisation on the Scottish road network.

Ελεύθερο «κατέβασμα» από την ιστοσελίδα:
http://www.trl.co.uk/online_store/reports_publications/trl_reports/cat_ground_engineering/report_rock_slope_risk_assessment.htm

(Transport Research Laboratory, 24.10.2011)



Rock engineering guides to good practice: rock slope remedial and maintenance works

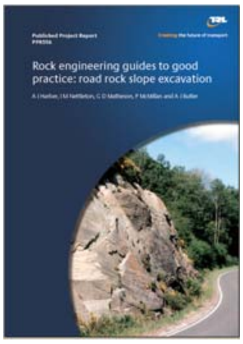
Published Project Report PPR555

P McMillan, A J Harber and I M Nettleton

This report was provides advice and guidance on good practice in rock slope remedial and maintenance works. The subjects covered include rock slope stability, risk assessment, risk management and reduction strategies, the selection of risk reduction strategies, the design and specification of remedial works and environmental considerations.

Ελεύθερο «κατέβασμα» από την ιστοσελίδα:
http://www.trl.co.uk/online_store/download_form/download_complete.htm?id=6714

(Transport Research Laboratory, 05.10.2011)



Rock engineering guides to good practice: road rock slope excavation

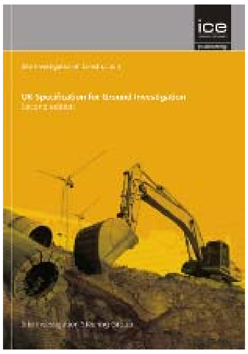
Published Project Report PPR556

A J Harber, I M Nettleton, G D Matheson, P McMillan and A J Butler

This report provides advice and guidance on good practice in road rock slope excavation. The subjects covered include rock slope stability, site investigation, rock slope design, rock slope excavation and environmental considerations.

Ελεύθερο «κατέβασμα» από την ιστοσελίδα:
http://www.trl.co.uk/online_store/download_form/download_complete.htm?id=6715

(Transport Research Laboratory, 05.10.2011)



UK Specification for Ground Investigation, 2nd edition

Site Investigation Steering Group

Endorsed by the Highways Agency, Environment Agency, British Waterways and Network Rail, UK Specification for Ground Investigation is applicable to all ground investigation work. Revised in line with the many advances and regulatory changes affecting ground investigation, particularly in respect of contaminated ground and dealing with waste materials, the Specification has an increased number of sub-clauses to reflect the requirement for many investigations to use combinations of methods and varying sampling techniques.

UK Specification for Ground Investigation

- can be used for contracts of any size, with any form of contract and method of measurement
- meets current standards and complies with established good practice
- includes associated Schedules and a Bill of Quantities, drafted to be compatible with the Specification
- can be modified for a specific investigation; the Schedules provide for modifications to the Specification
- includes Notes for Guidance alongside each clause.

UK Specification for Ground Investigation is an essential document for all ground practitioners involved in ground investigation, including geotechnical engineers, engineering geologists, environmental scientists and archaeologists, as well as specialists in other disciplines involved in specific investigations.

(ICE Publishing, 23.11.2011)

Handbook of Geosynthetic Engineering, 2nd edition

Edited by Sanjay Kumar Shukla

Handbook of Geosynthetic Engineering details the different types of geosynthetics and their practical application for students and for practising engineers seeking efficient, cost-effective solutions to civil engineering problems.

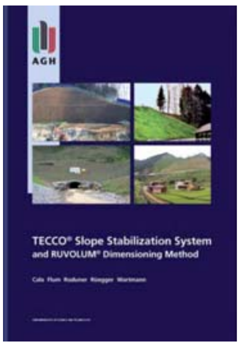
Written by subject specialists, the second edition of *Handbook of Geosynthetic Engineering* details the basic concepts of geosynthetics and their application in major civil engineering projects in an easily-readable form. With new chapters on filters and drains, tunnels, and the use of geosynthetics in sustainable development, the handbook is fully updated in line with current research and developments.

Handbook of Geosynthetic Engineering:

- Explains the technical, economical and environmental advantages of using geosynthetic construction materials over conventional solutions to field problems
- Discusses methods of analysis, design, construction and field monitoring of structures constructed with geosynthetics
- Describes the practical applications of geosynthetics with general and specific application guidelines, including seismic aspects, for the most effective solutions to specific field and research problems
- Presents case histories of geosynthetics use
- Concludes each chapter with a summary of main points and common questions and answers
- Identifies further potential areas for research

An essential resource for practising civil, geotechnical, transport, hydraulic, environmental and mining engineers in search of novel approaches for solving civil engineering problems using geosynthetics; *Handbook of Geosynthetic Engineering* is an invaluable reference for civil engineering researchers, lecturers and students, as well as for manufacturers of geosynthetic products, including polymer Industries, and specialised contractors.

(ICE Publishing, 24.11.2011)



TECCO® Slope Stabilization System and RUVOLUM® Dimensioning Method

Cała, Flum, Roduner, Rüegger & Wartmann

Flexible slope stabilization systems made from conventional wire meshes in combination with nails or nailing are widely used in practice to stabilize soil and rock slopes. They are traditional solutions and provide an alternative to measures based on rigid concrete liner walls, shotcrete applications or massive supporting structures.

Slope protection by means of common wire mesh and wire rope nets is known accordingly, but the transfer of forces by mesh as pure surface protection devices is limited on account of their tensile strength and above all also by the possible force transmission to the anchoring points (nails, anchors).

Strong wire rope nets offer certain possibilities for slope stabilizations with greater distances between nails and anchors. However, they are comparatively expensive in relation to the protected surface and the size of the individual nets is relatively small, resulting in higher installation cost and less fl exibility to local terrain conditions.

Today, apart from solutions using conventional steel wire, new meshes from high tensile steel wire are now also available on the market. The latter can absorb substantially higher forces and transfer them onto the nailing.

A new special method has been developed for the designing of fl exible slope stabilization systems with high tensile steel wire meshes for the use on steep slopes in more or less homogeneous soil or heavily weathered loosened rock.

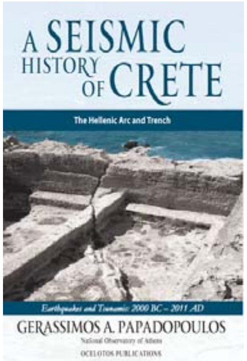
The interaction of mesh and fastening to nails has been investigated in comprehensive laboratory tests. This enabled also to find a suitable fastening spike plate which allows an optimal utilization of the strength of the mesh in tangential (slope-parallel) as well as in vertical direction (perpendicular to the slope).

The trials also confi rmed that the high tensile wire meshes, in combination with suitable plates, enable substantial pretensioning of the system. Such pretensioning increases the effi ciency of the protection system. This restricts deformations in the surface section of critical slopes which might otherwise cause slides and movements as a result of dilatation. Suitable dimensioning models permit to correctly dimension such systems.

Various implemented stabilizations in soil and rock, with and without vegetated face, confi rm that these measures are suitable for practical application and provide useful information for the optimized handling and installation process.

Geotechnical, civil and mining engineers, geologists, professors and students, designers, public authorities as well as any decision takers will receive a detailed insight into the subject of slope stabilization with the new high tensile steel wire mesh and the new dimensioning method for fl exible slope stabilization systems which allow simple and safe concepts including cost saving installation processes. This provides new interesting solutions for traditional geotechnical problems which, in the meantime, are executed and applied globally on all continents.

(Geobrugg AG, end of 2011)

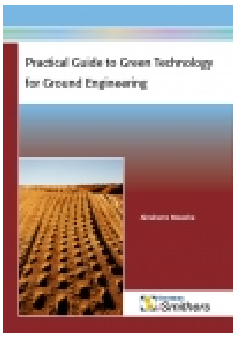


A Seismic History of Crete – Earthquakes and Tsunamis, 2000 BC – 2011 AD

G. Papadopoulos

In this book the geodynamics and seismicity of the Hellenic Arc and Trench in the area of Crete are reviewed. Starting from the Minoan era of about 2000 BC, 190 earthquake events occurring in the pre-historical, the historical and the primitive instrumental period up to 1913, are critically examined with the support of field geological and archaeological observations and of documentary sources in original language and in English translation. A considerable number of those events were unknown in the seismological literature so far. Earthquake focal parameters are evaluated and reliability scales are introduced. Associated phenomena, such as volcanic eruptions, tsunamis, seaquakes, liquefaction in soil, rockfalls and landslides, earthquake precursors and the similar are also described. Cultural items, such as folk songs, poems etc., as well as pictorial material, when related to the earthquake activity, are also inserted along with the rest documentary material. Another set of 29 only strong earthquake events occurring in the early and the modern instrumental era of seismology up to 2011 inclusive are also examined with the support of instrumental records and macroseismic observations. Associated phenomena are again described.

(Εκδόσεις ocelotos, 2011)



Practical Guide to Green Technology for Ground Engineering

Abrahams Mwasha

Over the last 50 years there has been rapid development of construction techniques, analytical methods and materials for use in ground engineering. One of the major techniques which has been developed is soil strengthening or reinforcement whereby man-made elements are included within geological material to provide a stabilised mass. Various products have been developed for retaining systems, slope stabilisation, etc.

More recently, environmental concerns and the focus on sustainable development have led to the examination of materials based on renewable resources for use in ground engineering.

In this book, the applications of both vegetable and man-made fibres in situations where there is a requirement for short-term ground reinforcement are examined and discussed. The use of vegetable fibre geotextiles (VFG), particularly in erosion control and soil reinforcement, is covered in detail, with examples from various civil engineering applications.

(iSmithers Rapra Publishing, 2011)

ΗΛΕΚΤΡΟΝΙΚΑ ΠΕΡΙΟΔΙΚΑ

http://www.issmge.org/web/page.aspx?refid=759

Κυκλοφόρησε το 5ο Τεύχος του 5ου Τόμου του ISSMGE Bulletin (Οκτωβρίου 2011) με ενδιαφέροντα νέα από την διεθνή γεωτεχνική κοινότητα και άρθρα για την επίσκεψη της ATC3 "Geotechnical Natural Hazards" σε περιοχές εκτεταμένων κατολισθητικών φαινομένων στο Bhutan, για υποθαλάσσιες κατολισθήσεις στην νοτιοανατολική Αυστραλία και για βελτιώσεις εδαφών.

Visit of ATC3 Committee on Slope Instability Sites in Bhutan, pp. 20-31, Ikuo Towhata, Mitsu Okamura, Hirofumi Toyota

Submarine Landslides on the South-Eastern Australian Margin, pp. 32-45, S. Clarke, D.W. Airey, P. Yu, T. Hubble

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project, pp. 46-52, P.K. Wong

Geotechnical offshore site investigation and reclamation design at Port Kembla, pp. 53-66, Z. Lai, J. Hsi, T. Rheinberger, T. Andrews

☙ ❧

www.geoengineer.org

Κυκλοφόρησαν το Τεύχος #82 του **Newsletter του Geoengineer.org** (Νοέμβριος 2011) με πολλές χρήσιμες πληροφορίες για όλα τα θέματα της γεωτεχνικής μηχανικής. Υπενθυμίζεται ότι το Newsletter εκδίδεται από τον συνάδελφο και μέλος της ΕΕΕΕΓΜ Δημήτρη Ζέκκο (secretariat@geoengineer.org).

☙ ❧

International Journal of Geoengineering Case Histories, Vol.2, Issue #2

http://campaign.r20.constantcontact.com/render?llr=a6kbovbab&v=001fTAmhMmZbVJbofb5oX3BYPae7ljiVCuFMrYL0-kb8YLZ9KHHIf1LX6-DCcu225e15MK7NSLJYF3ekBIvhGQn31XaFTCJcjG4J4EKUXZMD_8%3D

Papers published in this refereed journal are freely available in color and are accompanied by databases that include the electronic data presented in the paper as well as additional figures (as necessary).

Ground Improvement using Pre-loading with Prefabricated Vertical Drains, pp. 86-104, Ashutosh Sutra Dhar, Abu Siddique, Syed Fakrul Ameen

Underpassing of Angel Underground by London Ring Main Extension Tunnel, pp. 105-126, Benoît Jones

Mississippi River Levee Failures: June 2008 Flood, pp. 127-162, Michelle Bernhardt, Jean-Louis Briaud, Dongkyun Kim, Mathieu Leclair, Rune Storesund, Seok-Gyu Lim, Robert G. Bea, J. David Rogers

Paper Title: Quarry-Induced Slope Instability at a Broadcasting Transmission Plant near Valcava, Lombardia, Italy, pp. 163-181, Monica Barbero, Fabrizio Barpi

ΕΕΕΕΓΜ
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