Special applications of geosynthetics in geotechnical engineering

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ABSTRACT: The paper focuses on the interaction between geosynthetics and soil or other granular material, and it considers the systems as structural units or composite bodies. Selected examples are retaining structures, road and railway structures (incl. freezing-thawing resistance), protective embankment dams against large-scale rockfalls or avalanches, designing against impact loads, barrier and reactive walls, landfills, and tunnels. A special chapter refers to jacketed piles and diaphragm wall elements, to geotextile-coated sand columns and sleeve-reinforced stone columns in soft ground and to flexible geotextile forms for grouting in ground with large voids. Finally, attention is directed to proper compaction of geosynthetic-soil systems.

1 GEOSYNTHETICS FOR ROADS AND RAILWAYS

Geosynthetics are used in all layers of road structures, exhibiting the following major functions: separation, reinforcement, filtration, drainage, moisture barrier. Furthermore, geofoam has proved suitable as lightweight fill material on soft ground, for soil exchange beneath the sub-base and for temperature isolation to prevent freezing-thawing damages.

The use of geofoam is increasingly under debate also in railway engineering and is therefore briefly discussed in the following:

Geofoam is usually block-molded expanded polystyrene (EPS), whereby the joints between the blocks are the most critical aspect of this system. Open joints significantly affect the behaviour of pavement structures with an unbound road base. This negative influence can be neutralised by placing a cement-treated layer above the EPS subgrade/sub-base. But generally, open blocks should be avoided by all means, and they should not coincide with each other in various layers. The longitudinal joints between the EPS blocks should not be close to a wheel track. These installation recommendations contribute to a significant increase in the pavement design life (Duskov, 1997)

The most important factors for the use of EPS in road (and railway) construction are:

- Density;
- Behaviour under static loads and cyclic compression;
- Shrinkage behaviour;
- Water absorption.

The deformation, hence density of EPS blocks should be such that the estimated total compression under static and dynamic loads after 50 years is less than $\varepsilon = 1.5\%$, unless repeated surface relevelling is accepted. For subgrades of roads (sub-bases of railways), the minimum density should be 20 kg/m^3 and the thickness of the cover with granular layers $d \geq 0.6$ m.

Time dependent behaviour (creep and relaxation) of EPS (and polymeric foams in general) can be significant and should be considered, especially for high traffic expressways and high-speed railways. Figure 1 illustrates the considerable difference between rapid and long-term loading (1 year) of a 13 kg/m^3 specimen under unconfined axial compression.

Pavement rutting is related to the relative stiffness of the pavement material and its subgrade/subsoil. Recent test series have disclosed (Zou et al. 2000) that the resilient deformation of

EPS geofoam at the subgrade level plays the most important role. It manifests itself as a deeper rut on the pavement surface than in case of granular subgrade. The rut depth could be reduced, however, by placing a thicker or stiffer pavement – or by using stiffer geofoam. The magnitude of cumulative permanent deformation increases non-linearly with loading. The EPS geofoam block size and the lateral restraint do not significantly influence pavement performance.

The use of geosynthetics to reinforce soft subgrade or subsoil, or to separate cohesive ground from granular road structures is a wide application area which has been extensively investigated theoretically and practically world-wide. It is therefore not a special application in the sense of this keynote contribution.

The reinforcement of asphaltic concrete and reflective crack prevention in bituminous pavement overlays are widely used in the USA., but less in Europe. Besides geogrids and geotextiles, also glass fibres, metal fibres and steel meshes have been installed. The most common method, however, is placing full-width fabric sheets over the entire pavement surface, which is waterproofed with asphalt emulsion and then overlaid with the final bituminous surfacing. For further details se e.g. Koerner (1998).

The serviceability index and lifetime of concrete pavements can be increased by placing a nonwoven geotextile between the concrete pavement and the asphaltic base. This should be performed over the entire area and not only below the joints between the concrete slabs. The best results have been achieved with a two-layered geocomposite consisting of a mechanically bonded nonwoven with a covering filter layer that is stable against the initial mortar phase of the concrete. A high stability against erosion along the interface is obtained if the nonwoven geotextile can hold up the fine cement particles and take surplus water without being fully water-saturated. This also increases density and strength as well as resistance against frost, chloride penetration and carbonising of the concrete (Wilmers, 1999).

Full-area installation of a nonwoven below a concrete pavement has also proved suitable in the case of a cement-bound base, though this method contradicts conventional technology. There is no compound effect between the two layers then, and the cemented base cannot be statically taken into account for the design of the concrete pavement. But, on the other hand, the separation prevents stress constraints in the two-layer system and thus minimises especially horizontal cracking in the cement-bound base. Moreover, erosion of lower quality wearing courses is reduced.

Nonwoven geotextiles between concrete pavement and unbound gravel bases exhibit the following advantages (Wilmers, 1999):

- Improvement of the concrete quality in the base zone of the pavement.
- Filtration of (holding back) unbound base particles, which is critical along joints.
- Homogenising of the bedding conditions.
- Easier injection beneath the concrete slabs.

Due to the high pH-value of concrete and cement-bound wearing courses, only geotextiles of polypropylene or polyethylene ought to be used. The products must exhibit sufficient cross-plane and in-plane permeability under traffic loads and in the long-term.

Well graded (angular) gravels and sands make – beyond question – the best-wearing courses for both paved and unpaved roads. But in many areas of the world, such soil types are simply not available. Furthermore, thinner layers would be cost saving and the rational use of raw material resources has become a major concern for society. Therefore, geosynthetics have been increasingly used as reinforcing elements of road structures. As an example, Figures 2 and 3 show the test set-up and the results of systematic investigations of geogrids (polyester) and geocomposites in connection with angular gravel (crushed stone, 0/45 mm grains). The subsoil exhibited a modulus of E_{V2} = 10 MN/m^2 (derived from load plate tests; reloading curve), and on top of the layered system a modulus of E_{V2} = 45 MN/m^2 should be achieved which is required for the subgrade of roads according to German codes. Figure 5 clearly illustrates that a proper reinforcement of the layered system leads to a significant reduction of the necessary thickness.

However, the effect of reinforcing angular gravel (crushed rock) with geosynthetics is to be questioned in many cases. Crushed material possesses already intrinsically a high quality, and it may locally over-stress the geosynthetic inclusions due to its sharp edges.

If the thickness of a road structure should be reduced by installing geosynthetic reinforcement, two critical aspects have to be taken into account:

- Depth of frost penetration and frost susceptibility of subsoil (subgrade) as well as frost resistance of sub-base and base.
- The thickness of a gravel wearing course should be at least 15 to 20 cm, or three times the maximum grain size of the material. Thinner layers cannot be compacted properly.

The best geosynthetic reinforcement cannot avoid freezing-thawing damages if wearing courses (base and sub-base of roads or ballast, sub-ballast of railways) exhibit frost susceptible material, or if the frost penetrates too deeply into frost susceptible subgrade. A reliable assessment of the freezing-thawing behaviour of all layers of a road or track structure is therefore essential when designing and constructing a multi-layered system which incorporates geosynthetic reinforcement. Ice lensing in winter and soil softening in spring causes local stress concentration which must be taken over by the geosynthetic inclusions.

The frost susceptibility of fine-grained soil or bound material can be assessed best by freezing-thawing tests. Coarse material, commonly used for wearing courses, can be checked more easily by a mineral criterion which represents significant improvement of the classical grain size criterion, etc.

Comprehensive laboratory tests on thousands of samples as well as 35 years of field observation have disclosed that, in addition to the granular composition of soils (or artificial granular mixtures), the mineral composition of their fines is of decisive importance. Long-term investigations dealt with kaolinite, illite, smectite (e.g. Na^+, Ca^{++} montmorillonites), quartz, feldspar, dolomite, calcite, serpentine, chlorite, vermiculite, mica, muscovite and their weathered products such as (hydro-) biotite, paragonite, etc. The results can be summarised as follows:

A relatively high percentage of fines is allowable in the case of carbonates or quartz, whereas chlorites and their weathered products have proved to be critical. Granular mixtures, which consist of a larger content of chlorite (and muscovite) but also of montmorillonite, are particularly susceptible: Chlorite and muscovite cause primarily frost damage, but, in general, do not reduce significantly the bearing capacity during thawing. Montmorillonite, on the other hand, increases the danger of thawing damages, whereby there is a great difference between sodium and calcium montmorillonite. Kaolinite causes mainly frost heaves. The influence of metal hydroxides and organic components as well as the soil chemistry were also examined: it was, for instance, found that already a low percentage of X-ray amorphous iron hydroxides (gel form) reduces the freezing-thawing resistance of granular mixtures considerably.

Figure 4, derived from freezing-thawing tests with an open system illustrates the outstanding influence of the mineralogical composition of the fines on the freezing-thawing behaviour. This is especially noticeable with regard to maximum frost heave. It can be seen that the results may differ nearly by a factor of ten for silty-sandy gravel (e.g. curve 1 compared to curve 4) – in spite of the same grain size distribution.

Consequently, a basic distinction must be made between minerals showing a neutral behaviour and minerals which reduce the freezing-thawing resistance of soils or granular media:

Neutral minerals are all rock-forming minerals such as carbonates, quartz and feldspars, with the exception of the laminated silicates mentioned below.

Minerals which reduce the freezing-thawing resistance are mainly laminated silicates belonging to the kaolinite group, chlorite group, vermiculite group, smectite (e.g.montmorillonite) group, mica group and iron hydroxides resulting from weathering.

The percentage of the critical fines < 0.02 mm must not exceed the maxima listed in Table 1, determined after a modified Proctor test with a material of almost optimum water content (slightly below w_{opt}) or after compaction at the construction site.

The values of Table 1 apply to soils and granular mixtures with a uniformity degree of $C_u = d_{60}/d_{10} \geq 15$ and a maximum (theoretical) grain size of 45 mm (square-perforated screen, ISO series R 20/3). A maximum grain size of 63 mm is allowable for crushed material with a rather continuous grading and max. 25 % oversize > 45 mm. Additional oversized grain has to be removed only in the sieving test but may be included in the fill material on the site. Furthermore, the application of Table 1 requires a compaction degree of at least D_{Pr} = 100 % of the standard Proctor density. Poorly compacted layers exhibit a lower freezing-thawing resistance.

Usually, base courses of roads and highways consist of soils or granular mixtures with a uniformity coefficient of $C_u = d_{60}/d_{10} \geq 15$ in order to achieve proper compactability and bearing-deformation behaviour. Therefore, the values of Table 1 refer to such wide-grained material. Uniform soils exhibit a higher frost resistance. Consequently, 10 to 15 % < 0.02 mm are allowable in the case of $C_u \leq 5$. Intermediate values can be linearly interpolated. From a practical point of view, the percentage of fines should be limited to 10 % < 0.02 mm, also for uniform materials of $C_u \leq 5$ if they are part of a pavement structure and exposed to freezing-thawing.

Critics of the mineral criterion argue that the determination of mineralogical components would be very complicated and that the results would scatter too much. But 30 years of practical experience in Austrian road and highway engineering has disclosed that such arguments will not stand up. Firstly, mineralogical tests are necessary only if the material's properties are completely unknown. Consequently, they are widely limited to suitability tests and hardly necessary during the construction period, therefore not hindering or delaying the construction work. Secondly, mineralogical tests sufficiently accurate for the application of the mineral criterion are cheap and can be performed within one to two days. Thirdly, varying results from different laboratories can definitely not be used as a strong argument against the determination of the mineral composition, because varying results are also obtained in case of other tests. For instance, comparative ring tests among numerous soil mechanics laboratories investigating the same material have disclosed that the results do also scatter in case of Proctor values, Atterberg limits, grain size curves, shear parameters, etc. But nobody would, for that matter, omit these tests which have proved suitable in practice since decades, The uncertainty of the so-called "laboratory factor" can be minimised only by engaging laboratories with an excellent reputation and specific experience.

The mineral criterion does not apply to recycling materials, such as slag, fly ash, etc., nor to soil treatment with lime, cement, bitumen or chemicals. On the other hand, Figure 4 and Table 1 represent a reliable tool to assess the frost susceptibility of mineral liners and geosynthetic clay liners of waste deposits.

Frost heave of road pavements can be mitigated by installing capillary barriers. They consist of a layer of coarse, porous material which interrupts water transport from the deeper soil layers to the 0° C –isotherm when frost penetrates into the road structure and the subgrade. Therefore, they have to be placed between the groundwater table and the freezing front. The barrier material can be granular material, tyre or synthetic chips and geosynthetics. Due to their large pore size, geocomposites or geotextiles and geonets are most appropriate.

If the soil above the capillary barrier is frost susceptible and rather saturated, freezing-thawing damages occur in spite of the capillary barrier. The pore water migrates to the frost front where ice lensing occurs, and during thawing the layer loses capacity. Such a situation can be compared to freezing tests with a closed system, i.e. with limited groundwater supply – contrary to the open system where groundwater suction to the freezing front occurs without hindrance. Consequently, ice lensing in the upper layers is only reduced then by capillary barriers, but is not prevented. Furthermore, it should be emphasised that even the best road pavement is not absolutely water tight. Infiltration of water from above may occur at least locally, and this increases inevitably the water content of the (sub-) base, hence leading to local damages if the material does not exhibit sufficient freezing-thawing resistance.

Aspects of freezing-thawing are of course also of interest in railway engineering, especially for high speed trains which are very sensitive towards differential deformations of the tracks. The mineral criterion has proved suitable as in road engineering, whereby long-term deterioration of the (sub-) ballast has to be considered. This may be caused by grain crushing and attrition due to the high dynamic traffic loads, by pumping of the soft soil into the coarse layer, or by subgrade erosion. Pumping and erosion can be prevented by the installation of filter-stable nonwoven geotextiles. This method has proved to be superior to bituminous layers or soil treatmentwith cement or chemicals. Long-term investigations have been running since the year 1973, whereby specimens have been taken periodically from geosynthetics below the ballast.

Furthermore, new methods of track maintenance and renewal make increasingly use of geotextiles, geogrids and geomembranes. A special application of geosynthetics in railway engineering are reinforced sleeper beds. They consist of tubular geotextile bags filled with sand or rounded gravel and they are of economic interest for those countries where conventional coarse ballast is not available. Technological advantages are a considerable reduction of vibration and noise.

The increasing traffic on roads and railways raises the question as to the dynamic behaviour of geosynthetics under numerous load cycles. Hitherto investigations have disclosed that the long-term behaviour of most products does not worsen if a proper design and careful installation have been applied. For instance, HDPE – grids subjected to 10^7 load cycles showed slightly reduced stretchability and even slightly increased strength. The latter is similar to a form of steel quality improvement: the load cycles cause an optimised orientation of the material and hence a strength increase. Furthermore, the high plastic deformations reduce stress constraints within internal inhomogeneities of the fabric and at external notches. Geosynthetics which cover piles or granular columns of "piled" embankments have been monitored for several years already. They show adequate behaviour even in the case of rather low embankments (≥ 1 m) and under intensive dynamic load cycling from high-speed trains.

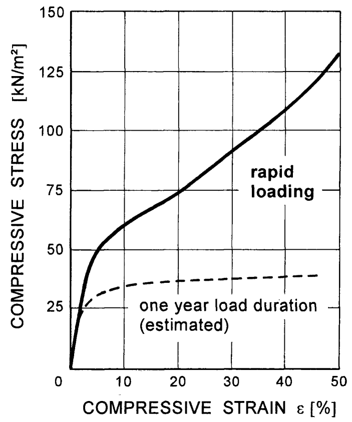


Figure 1. Effect of load duration on stress-strain behaviour of block-molded EPS in unconfined axial compression. EPS density = 13 kg/m^3 (Horvath, 1997).

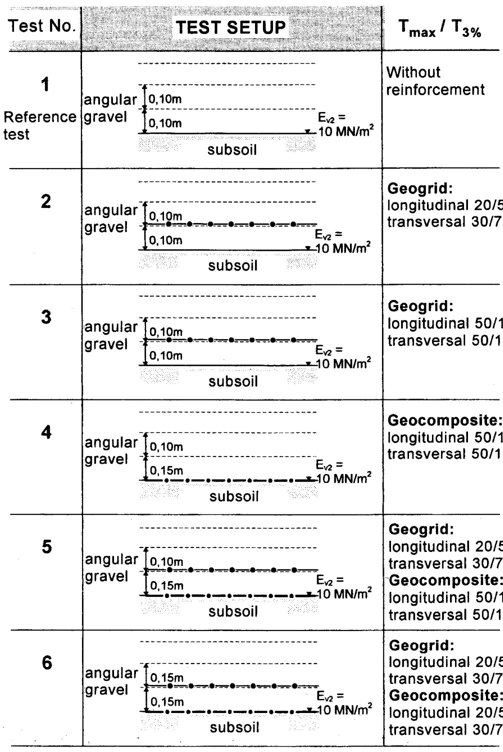


Figure 2. Test set-up to investigate the influence of geosynthetic reinforcement on the bearing-deformation behaviour of a multi-layered system for roads and railways (Göbel et al 1997).

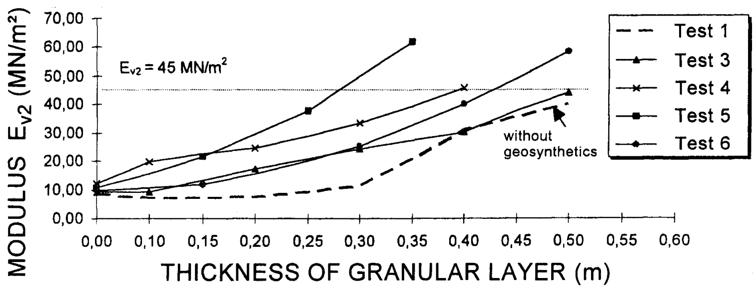
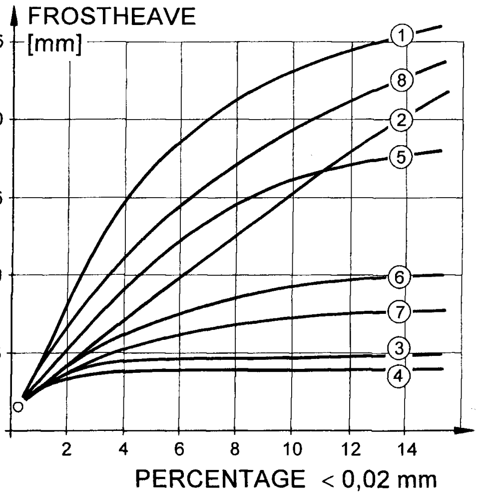


Figure 3. Results to Figure 2.



1 ... 50 % quartz + 50 % kaolinite
2 ... 80 % quartz + 20 % Ca-montmorillonite
3 ... 90 % quartz + 10 % Na-montmorillonite
4 ... 70 % quartz + 30 % Na-montmorillonite
5 ... 50 % kaolinite + 50 % Ca-montmorillonite

F igure 4. Influence of percentage of the fines < 0.02 mm and their mineralogical composition on frost-heave (maximum value after a testing period of 15 days). Sandy gravel mixed with fines of different clay mineral mixtures.

Table 1. Mineral criterion. Freezing-thawing resistance of soils.

Max. allowable fines < 0.02 mm	MINERAL COMPOSITION OF FINES < 0.02 mm
3 %	Determination of mineral components is not necessary
5 %	Standard Case: Additional tests are not necessary if the mineral composition, or the material behaviour under freeze-thaw conditions (in the field or in the laboratory) respectively, are known from earlier investigations.
5 %	In case of materials whose characteristics are unknown : 1) Non-active minerals 2) Mixture of 1) and a maximum of a) 10 % kaolonite group b) 30 % chlorite group c) 30 % vermiculite group d) 40 % smectite (montmorillonite) group e) 50 % mica group Marginal condition : mixture of 1) and a maximum of f) 60 % mica group + chlorite group g) 50 % mica group + chlorite group + kaolinite group h) 50 % mica group + kaolinite group i) 40 % mica group + chlorite group + kaolinite group + smectite (montmorillonite) group Further mixtures, not listed here, of laminated sillicates are allowable up to a total sum of a maximum of 40 %. If this limit value is exceeded, freezing-thawing tests are necessary. 3) Freezing-thawing tests should be performed, if an intensive reddish-brown colour of the material indicates that iron hydroxides are contained.
8 %	Non-active minerals, with a maximum of 1 % by mass < 0.002 mm

Geosynthetic-soil retaining structures developed from the reinforced earth system and have been increasingly used since about 25 years. Geosynthetic inclusions for retaining structures are mainly sheets, strips, loops, but also cellular systems and fibres. Geosynthetic nails are under development but hitherto limited to short length (compared to steel nails). Furthermore, geofoam is used to reduce the earth pressure on retaining walls.

The construction of reinforced earthfills differs widely from ground nailing, though the rupture mechanism and safety analyses correspond to a certain extent. Reinforcing an in-situ soil by installing nails (which can be also of synthetic material) requires a top to bottom excavation sequence, whereas typical geosynthetic walls are constructed in fill layers from bottom to top. Further differences between soil nailing walls and filled soil-geosynthetic walls lie in the stress-strain and creep behaviour. Figure 5 illustrates this for the maximum tensile force line within a reinforced body, comparing rather rigid inclusions and very stretchable geosynthetics in a fill. In the case of very stretchable inclusions, the maximum tensile force line runs close to the Coulomb failure plane (Fig. 5c), and the lateral displacements, especially on top of the structure, mobilise an active state of stress. Rigid inclusions, however, exhibit a nearly vertical maximum tensile force line in the upper wall zone (Fig. 5b), hence roughly earth pressure at rest there, but the active value in the lower half of the wall height.

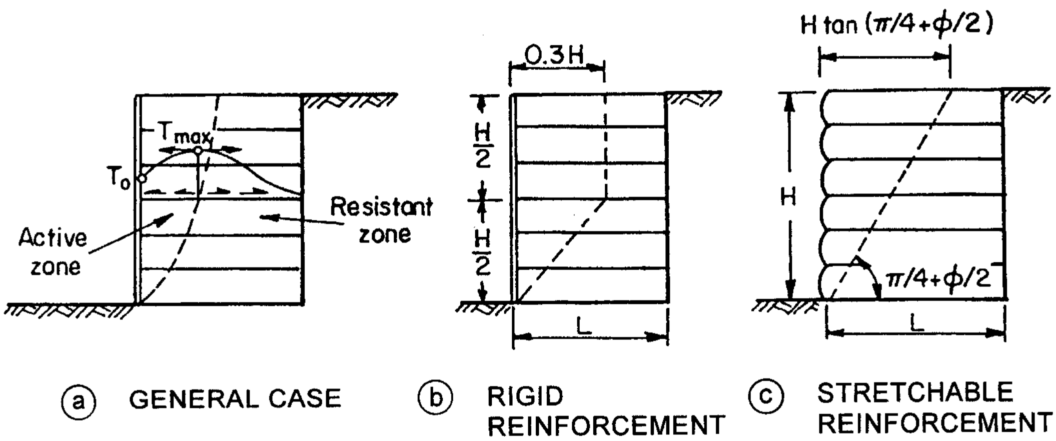


Figure 5. Line of maximum tensile force in a reinforced soil wall. Schematic after Schlosser (1991).

Waste tyres have also proved suitable for soil reinforcement, based on the same principle like soil-geosynthetic retaining structures. The tyres are then connected to each other by synthetic strips, ribbons etc. Gabions, finally, are a sort of soil reinforcement too. Gabions (consisting of geosynthetics) and modular cellular systems (rows of box-shaped cells of a grid lined with a nonwoven) are other forms of geosynthetic-soil retaining structures or walls.

In all cases, the fill material can be soil or other granular material (e.g. recycling aggregates), and the reinforced body behaves like a quasi-monolithic composite structure with an anisotropic cohesion. Anisotropy is not only caused by horizontal inclusions but also by the compaction process of the fill layers.

The compound effect of reinforced bodies increases with density of the in-situ ground or fill material. For soil-geosynthetic structures this can be achieved by intensive compaction of the fill layers, thus leading to a significant increase in slope stability if unstable slopes have to be supported.

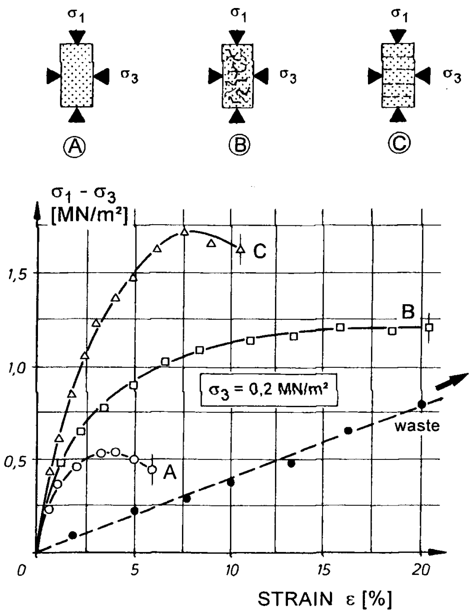


Figure 6. Triaxial tests on sand without (A) and with (B, C) reinforcement. B = irregular tangle of synthetic fibres; C = regular pattern of geosynthetics. Solid municipal waste for comparison.

Usually, soil reinforcement is placed within a fill in a regular pattern, but irregular tangles of synthetic fibres are also used (e.g. Texsol technique). Such structures exhibit a more flexible behaviour as can be seen from triaxial tests with reinforced sand. Figure 6 furthermore illustrates that solid municipal waste is also a sort of reinforced material due to its inclusions of long fibres, plastics and other fabrics.

The comparison of numerous tests performed on reinforcement walls with geosynthetic reinforcement shows a fairly good concurrence of Austrian, French and German test results. Nevertheless, there are differences in the calculation methods, whereby straight rupture lines passing through the toe of the wall are preferred in Germany. The differences are not significant, it is rather a question of design philosophy, because lastly the slip lines may run through soil zones which are completely, partially or not at all reinforced. In the end, all failure mechanisms for external, internal and local stability analyses have to be investigated, comprising straight, curved and combined (polygonal) failure lines.

In the case of high loads near the wall face, slip circles are the most critical failure mode. For ground with negligible cohesion and for high loads at the rear end of the reinforcement, translation with two sliding bodies provides the lowest safety factor. Figure 7 shows an example for evaluating the stability of such a retaining structure (see also Gässler 1987 for soil nailing). The safety factors F may be defined as follows:

$$F_N = \frac{A_{existing}}{A_{limit}} \tag{1}$$

$$F_P = \frac{P_{limit}}{P_{existing}} \tag{2}$$

$$F_\Phi = \frac{\Phi_{existing}}{\Phi_{limit}} \tag{3}$$

$A_{existing}$ = sum of the axial nail forces which may be mobilised at the maximum.
$P_{existing}$ = actual existing surcharge load.
The suffix “limit” means the individual values in the state of limit equilibrium.

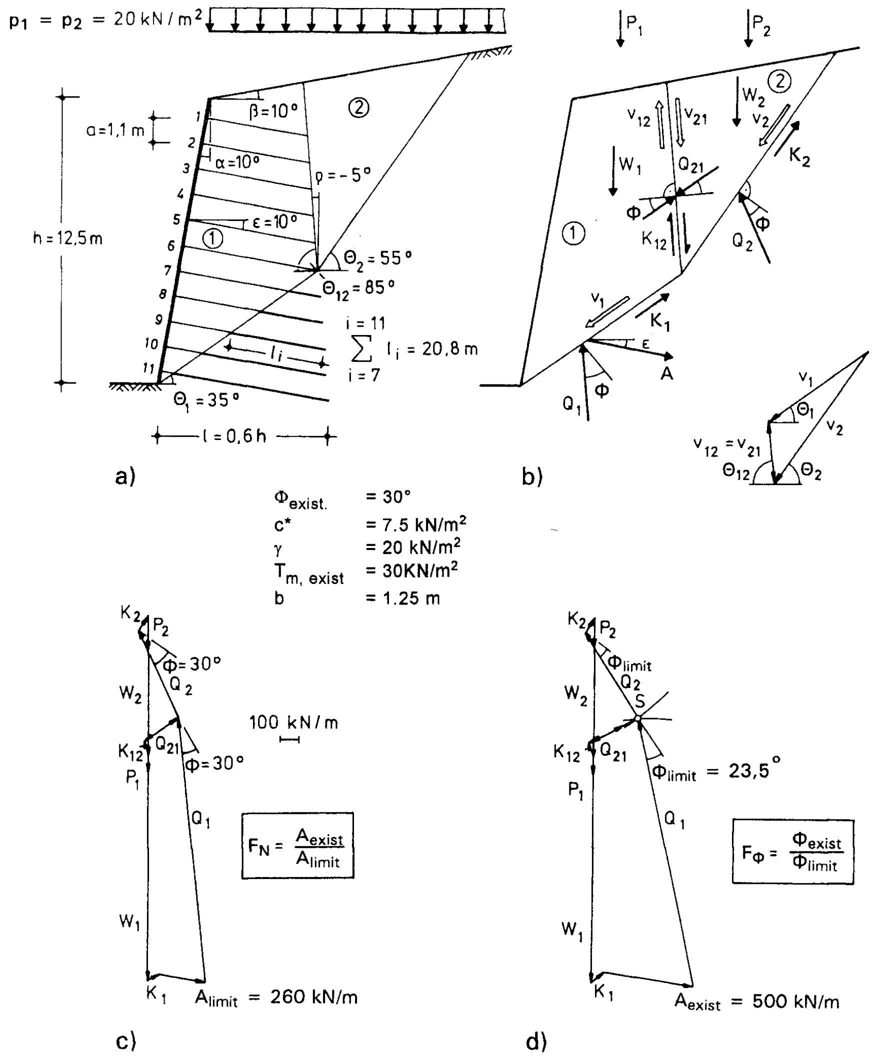


Figure 7. Stability analysis of a soil nailing structure (after Gässler, 1987). Example of a translatory failure mechanism consisting of two rupture bodies: (a) Cross section; (b) Forces acting on the system and displacement polygon; (c) Force polygon, closed with A_{limit} leads to F_N; (d) Force polygon closed with ϕ_{limit} leads to $F\phi$; Cohesion forces $K_i = C_i$ and $K_{ij} = C_{ij}$; v_i, v_{ij} = displacements in the failure plane i, ij.

According to the hypothesis of minimum stability, the sliding geometry is relevant for the state of limit equilibrium which leads to the safety factor of F = 1 as a minimum. The inclination of the main failure planes θ_1 and θ_2 in Figure 7 are either variable in the case of fairly homogeneous soil, or they are given by pre-existing slip surfaces in the natural ground behind the filling.

Parametric studies of the effects of variation of wall geometry on the behaviour of conventional reinforced soil walls disclosed that the most important geometric parameter is the reinforcement length to wall height ratio. Ho, Rowe (1996) varied reinforcement length, number of layers of reinforcement, distribution of reinforcement, and wall height, whereby the ground surface was assumed to be horizontal and the backfill soil to be cohesionless. It was found that the common approach of

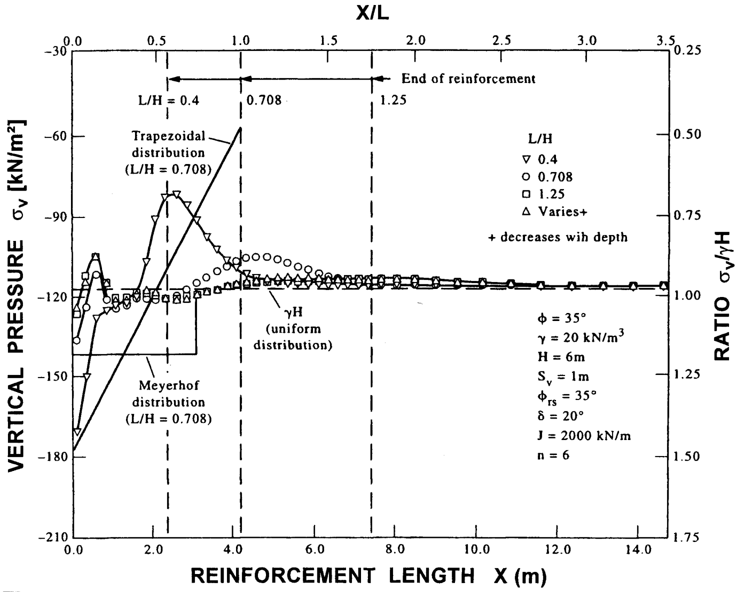


Figure 8. Effect of reinforcement length to wall height ratio (L/H) on vertical stress distribution at the base of a soil reinforced wall (σ_v). Results of numerical simulations (Ho, Rowe 1996).

providing equally spaced reinforcement with L/H = 0.7 provides a relatively efficient distribution of reinforcement force. For reinforcement schemes with a ratio of less than 0.7, the forces in the reinforcement increase as the length of the reinforcement decreases. A very small ratio even affects the vertical stress distribution at the base of the wall (Fig. 8).

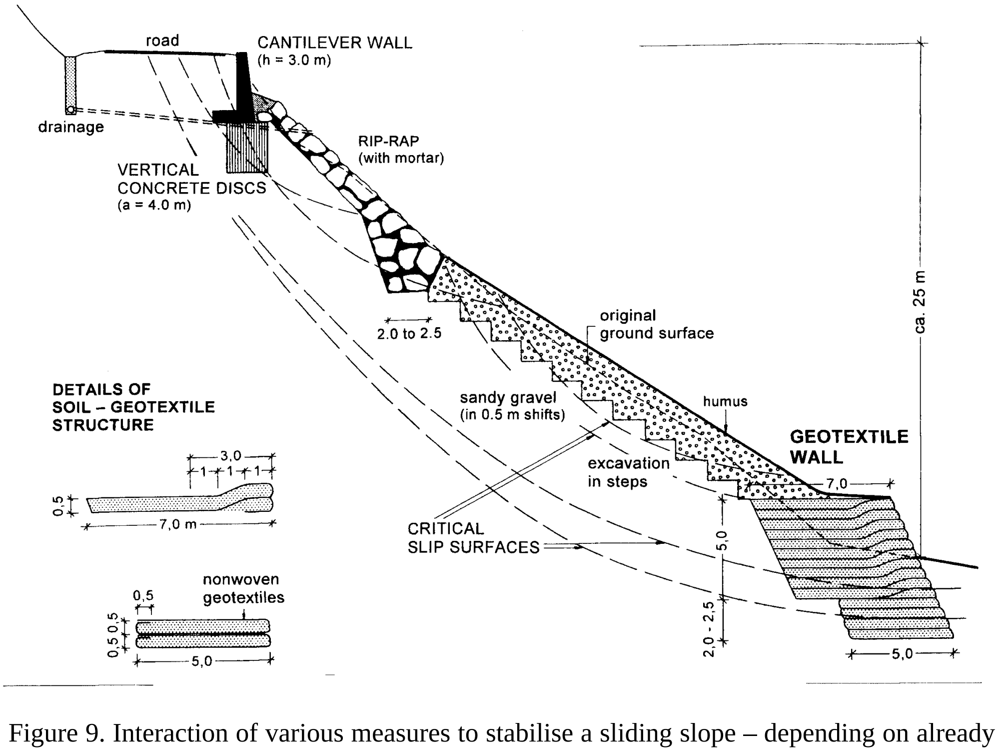


Figure 9. Interaction of various measures to stabilise a sliding slope – depending on already existing and further possible slip surfaces. Geotextile-reinforced toe structure as main support.

Soil reinforcement with geosynthetics is not only used for retaining walls but also for slope stabilisation. Such ground reinforcement is very flexible, can be installed very quickly - also in difficult terrain - and is rather cost-effective, especially when combined with other stabilising or restraining measures. Figure 9 illustrates an example, where the slip surfaces in an unstable slope propagated progressively into the ground. After the occurrence of surface-near failures and severe road damages, a geotextile-soil toe fill was constructed to support the slope and prevent the activation of deep-seated slides. Another slope supporting structure is shown in Figure 10.

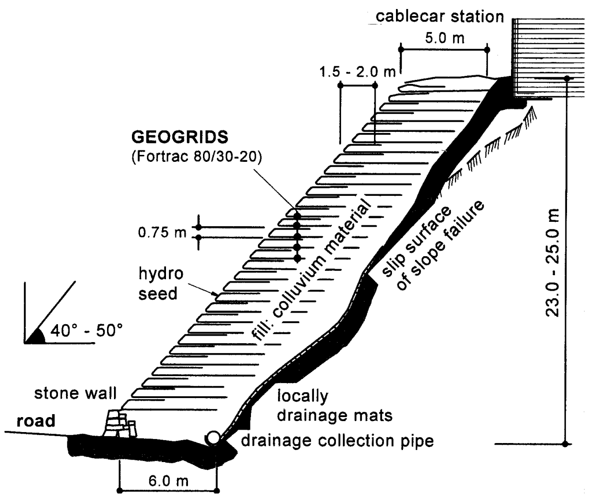


Figure 10. Support of a sliding slope by a geogrid-soil structure (modified after Huesker).

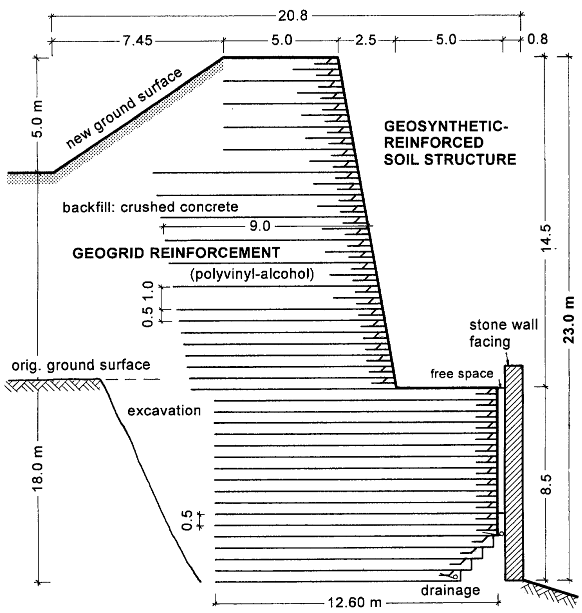


Figure 11. 23 m high geogrid-soil structure (modified after Huesker) to raise the level of ground surface under tight conditions.

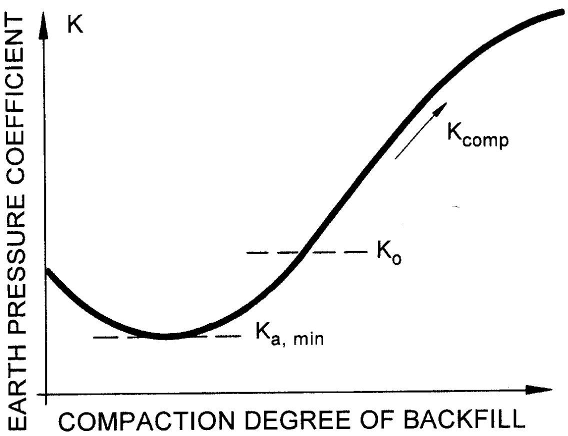


Figure 12. Earth pressure coefficient behind a retaining wall versus compaction degree of backfill. Schematic.

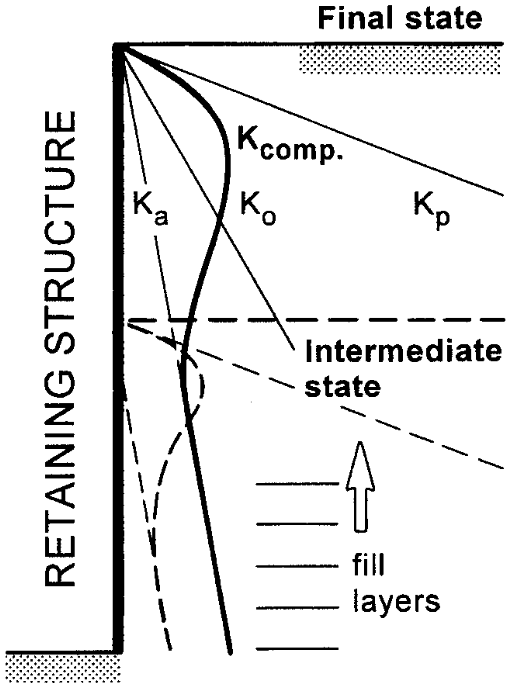


Figure 13. Lateral earth pressure distribution due to compaction of the backfill behind a retaining structure. Schematic.

Structures higher than about 20 m should – if possible – exhibit one or more benches of at least 3 m width for (long-term) monitoring, maintenance and to facilitate possible post-strengthening of the structure (e.g. Fig. 11). The backfill of Figure 11 consisted of crushed concrete (0/45 mm) and had to be compacted at a degree of $D_{Pr} \geq 100$ % of standard Proctor density. The reinforcement was achieved with geogrids. Because of the alkaline behaviour of crushed concrete, the geogrids consisted of polyvinyl-alcohol. This is a synthetic polymer of high strength and low creep, similar to polyester, but at the same time resistant to high pH-values. The structure is 23m high and more than 200 m long. The stone wall facing on toe of the reinforced soil structure was constructed only for architectural reasons and has no static function. The facing of the grid-reinforced structure exhibits sprinkling equipment for watering the vegetation. Such structures require especially good compaction of the fill layers, whereby the large width facilitates the application of continuous compaction control (see chapter 8).

The stability of geosynthetic-soil retaining structures increases with density, hence with the compaction degree of the fill layers. Because of the flexible behaviour of such systems, critical stress constraints caused by compaction hardly occur. However, conventional retaining walls, especially stiff systems, may be loaded by excessive lateral earth pressures due to local (over-)compaction (Fig. 12). Such stress increase may locally approach even the limit value of passive earth pressure, and it occurs predominantly in the upper part of the backfill (Fig. 13). Though it diminishes somewhat during the ongoing construction periods, the residual value of this “compaction pressure” may be significantly higher than the active one or the earth pressure at rest. A possibility to avoid such stress concentration is to compact more properly and/or to place a compressible inclusion behind the structure. This also facilitates controlled yielding of the wall, hence a decrease of the earth pressure at rest towards the active limit value in the case of rather rigid structures (e.g. stiff bridge abutments). Compressible inclusions are mainly geofoams but may also be geocomposites, and they may be placed behind conventional concrete walls or in connection with soil reinforcement walls.

Another special case of excessive lateral pressure or retaining structures is creep pressure:In unstable slopes undergoing creep, retaining structures may be stressed by a lateral pressure E_{cr}, that significantly exceeds the theoretical earth pressure at rest E_0. This creep pressure, E_{cr}, may also be considered as “sliding pressure” or “stagnation pressure” on a retaining structure. Its magnitude can be evaluated according to Figure 14. A sliding surface AB is assumed which enables, even for a nearly immovable wall, further creep of the slope. Corresponding with this assumption, in situ measurements have shown greater deformation of the ground than of the retaining structure (Brandl 1980). For theoretical simplification, the curved sliding surface is replaced by the plane A’B which is surcharge loaded by the mass above. Within the wedge A’BC, the secondary failure surface has to be determined, which provides the maximum earth pressure force E_{cr}. This can be calculated like the classical theory (body NMB as surcharge). Because of this surcharge the ground close to the retaining wall is compressed. Depending on the inclination, δ_1 of the pressure force D, the creep pressure E_{cr} varies according to the limit values $0 \leq \delta_1 \geq \Phi$. If assuming a (quasi) cohesionless mass and limit equilibrium ß = Φ (Figure 14), the creep pressure becomes a special case of an increased Rankine earth pressure.

$$E_{cr} = m(\Phi)\gamma \frac{h^2}{2} \cos\Phi \qquad (4)$$

The multiplication factor m (Φ) also depends on the stiffness of the retaining structure: The limit values of Figure 15 were obtained from numerous in situ measurements. They have proved very useful for practical design in the Alpine sliding areas of Austria for about 25 years. Geosynthetic-soil retaining structures are typically flexible systems and are therefore exposed to smaller creeping pressures than gravity walls or other rigid structures.

In the special case of short plane or curved retaining structures (e.g. protection shells around bridge piers or masts the friction forces on both ends of the wall) cause an increasing pressure. This can be taken into account by assuming an increased influence width of the retaining structure (B_{calc}), depending on the geometry of the wall, the ground parameters and inclination of the slope: mainly B_{calc} = 1.2B to 1.5B, whereby B is the actual length.

Figure 16 shows some example of “deadmen” walls where the “anchor” forces caused by lateral earth pressure or external loads are taken over by the backfill of the facing elements. Loop shaped anchor strips connecting front and rear side of a soil retaining structure differ widely from conventional soil reinforcing elements, because loops do hardly act as reinforcement, and friction along the tension elements is only of secondary importance. Nevertheless, such structures (mostly modular) can also be considered a soil-reinforcement body in a wide sense, whereby polymer tension elements have increasingly substituted steel members.

Geosynthetic-reinforced soil retaining walls with modular (segmental) blocks offer a wide range of aesthetic facings, also resulting in more economical structures compared to traditional gravity wall structures. The external, internal, and local stability of segmental modular walls with a soil reinforcement has to be proved by assuming various failure mechanisms. This is indicated in Figure 17 which is basically valid for all soil-reinforced element walls, independently of the inclusions in the backfill and the details of facing – hence also for gabion walls and other modular/segmental structures tied back with soil reinforcement. In detail the internal and local failure modes are widely influenced by individual structural properties. For most segmental systems it is recommended to place compression materials between modular blocks to allow for greater differential settlement and maintain a possible skin friction.

Stability investigations should also include kinematic analyses. They are a valuable supplement, especially for slope stability assessment and also provide data on possible displacements when rather rigid ground bodies undergo translation as (quasi-) monolithic blocks. Additionally, the ground bodies themselves may consist of separated blocks which move against each other along inner slide surfaces.

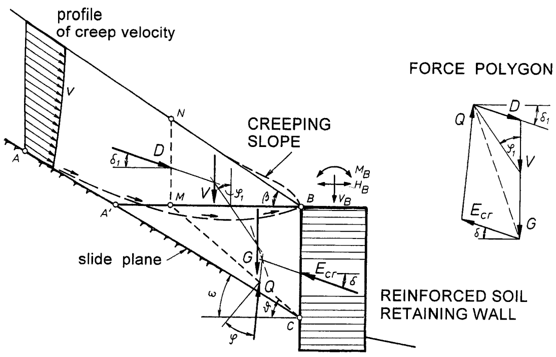


Figure 14. Theoretical assumptions for assessing the creeping pressure, E_{cr}, on reinforced soil retaining structures supporting unstable slopes.

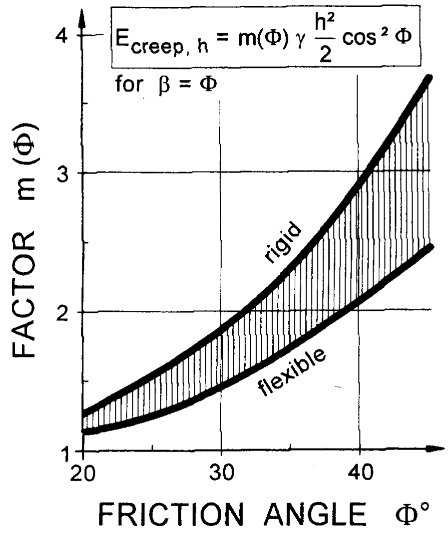


Figure 15. To Figure 14: Influence of the rigidity of the structure on the creeping pressure, $E_{cr,h}$. (Slope inclination = β, angle of internal friction = Φ).

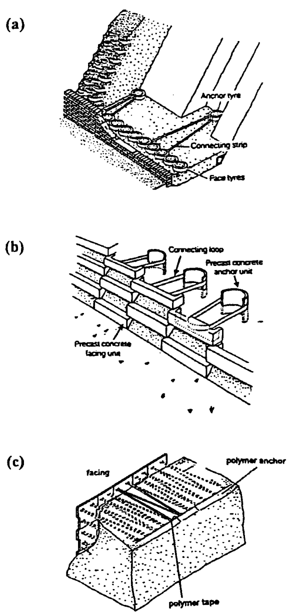


Figure 16. Anchored soil retaining wall systems based on the "deadmen" anchoring principle (Jones, 1995). (a) Tyre anchored wall system; (b) Loop anchored wall system; (c) Polymer tape and anchor wall system.

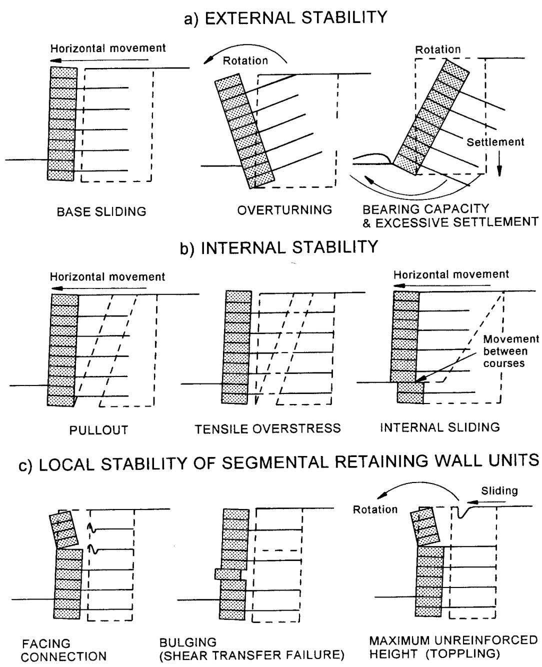


Figure 17. Failure mechanisms for external, internal and local stability analyses for segmental retaining walls including reinforced or anchored soil.

3 GEOSYNTHETICS FOR PROTECTIVE EMBANKMENT DAMS AGAINST ROCKFALLS, AVALANCHES AND DEBRIS FLOWS

3.1 *Introduction*

The use of geosynthetics has proved to be an excellent technique for the reinforcement of embankments and earthfill dams. The obvious benefit is the possibility to significantly increase the slope angles compared to non-reinforced structures. Special cases are protective embankments against large scale rockfalls, avalanches, and debris flow. They are primarily stressed by dynamic impact loads superposed by static loads, whereby local pore water pressures may also occur.

Figure 18 shows different applications for the stabilising of 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 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, as surface compaction aid to achieve high compaction at the edge of the slope.

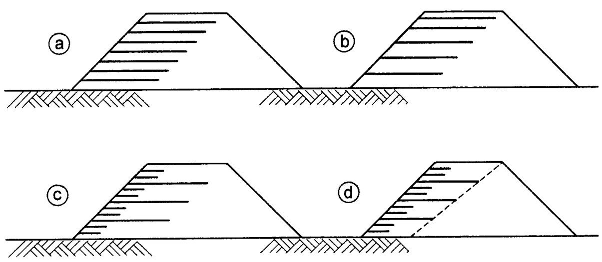


Figure 18: Various geotextile deployment schemes for stabilising steep soil dams. (a) Even spaced, even length. (b) Uneven spaced, even length. (c) Even spaced, even length with short facing layers. (d) Even spaced, uneven length with short facing layers (after Koerner 1998).

The focus in the design of geosynthetic reinforced dams lies on internal and external stability under consideration of the site-specific aspects like surface and facing details.

The design of protection dams has to meet specific requests of these type of structures:

- Protection dams are constructed to safeguard threatened (especially inhabited) areas against dangerous impacts from avalanches, mud and debris flows, rockfalls and landslides.
- Thus, the protecting structure must be stable not only against static dead loads and "quasi-static" life loads but has to resist sudden large-scale or locally heavy dynamic impacts. The prediction of such events is difficult, as is the calculation and the design of a reliable protective structure considering also economical aspects.

Geosynthetic reinforcement essentially contributes to meeting the high demands of protective structures. In a first step the general stability of geosynthetic reinforced dams is considered, in a second step a dynamic calculation model is introduced and discussed taking into consideration possible impacts on an embankment dam.

3.2 *General stability considerations*

There are different possibilities to design geosynthetic reinforced embankments. In this chapter a new method is developed. It was derived from conventional calculation methods of non-reinforced

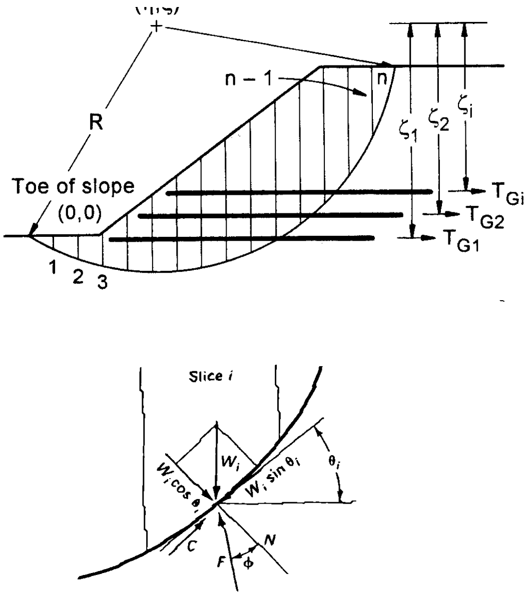


Figure 19: Details of circular arc slope stability analysis for (c, φ) shear strength soils (Koerner 1998).

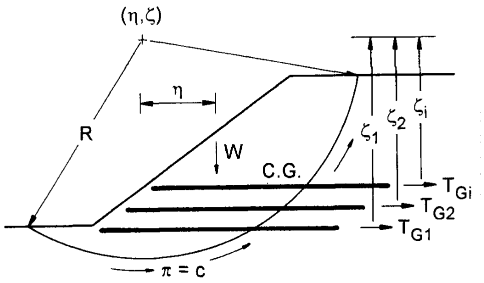


Figure 20: Details of circular arc slope stability analysis for soil strength represented by undrained conditions (Koerner 1998).

embankment dams and was extended and adapted for reinforced embankment dams. 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 19). 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 (6) for total stress analysis and equation (7) 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 in:

$$FS = \frac{\sum_{i=1}^{n} \left(N_i \tan\varphi + c\,\Delta l_i\right) R + \sum_{i=1}^{m} T_{Gi}\zeta_i}{\sum_{i=1}^{n} \left(w_i \sin\theta_i\right) R} \tag{6}$$

$$FS = \frac{\sum_{i=1}^{n} \left(\overline{N}_i \tan\overline{\varphi} + \overline{c}\,\Delta l_i\right) R + \sum_{i=1}^{m} T_{Gi}\zeta_i}{\sum_{i=1}^{n} \left(w_i \sin\theta_i\right) R} \tag{7}$$

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 20 shows details of this situation and results in equation (8):

$$FS = \frac{c\,L_{arc} R + \sum_{i=1}^{m} T_{Gi}\zeta_i}{W\,\eta} \tag{8}$$

- 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.

3.3 *Impact load on geosynthetic reinforced embankment dams*

The consideration of local dynamic impacts is essential for protection 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 *Idealisation of embankment dam structure*

As shown in Figure 21, the dam cross section is idealised as a wedge. 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.

Considering a differential segment $A\,d\zeta$ of the shear beam (Fig. 22) the differential shear deflection $d\eta$ can be written as:

$$d\eta = \frac{F}{A' G} d\zeta = \gamma\, d\zeta \tag{9}$$

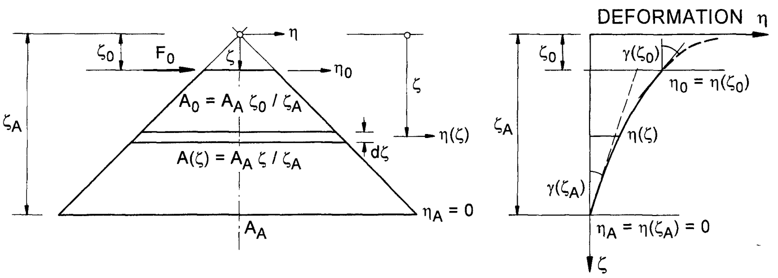
where F is the horizontal shear force, G the shear modulus of the dam and A' the corrected area derived from dividing the area A by the geometric correction coefficient κ.

The integration of equation (9) taking into consideration the boundary conditions as shown in Figure 21, i.e.

$$\eta(\zeta) = \int_{\zeta}^{\zeta_A} \frac{F}{A'(\zeta)\, G} d\zeta \tag{10}$$

results in equation (11) for the horizontal displacement according to a horizontal force on the top of the dam:

$$\eta(\zeta) = -\frac{F\, \kappa\, \zeta_A}{A_A\, G} \ln \frac{\zeta}{\zeta_A} \tag{11}$$



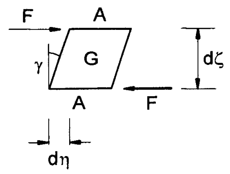


Figure 21: Idealised shear deformable dam cross section

Figure 22: Infinitesimal shear deformable element.

The displacements on the top η_0 and on the foundation basis η_A are:

$$\eta(\zeta_0) = \eta_0 = -\frac{F\, \kappa\, \zeta_A}{A_A\, G} \ln \frac{\zeta_0}{\zeta_A} \qquad \eta(\zeta_A) = \eta_A = 0 \tag{12a, b}$$

The first derivation of equation (11) corresponds to the rotation of shear γ:

$$\eta'(\zeta) = \gamma(\zeta) = -\frac{F\, \kappa\, \zeta_A}{A_A\, G} \frac{1}{\zeta} \tag{13}$$

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 $\eta^*(\zeta,t)$ of the shear deformable dam by one function separable in space $\varphi(\zeta)$ and time $q(t)$, the so-called *Ritz* approximation (Ziegler, 1998):

$$\eta^*(\zeta,t) = q(t)\, \varphi(\zeta) \tag{14}$$

where $q(t)$ is the generalised coordinate of the single degree of freedom (SDOF) equivalent system of the continuum. The function $\varphi(\zeta)$ 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 $\varphi(\zeta)$ 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.

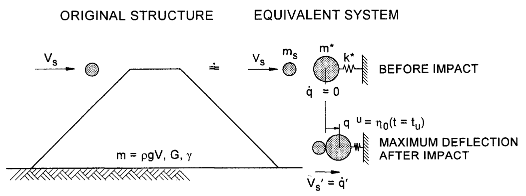


Figure 23: Original dam structure and equivalent system loaded by an idealised inelastic single body impact.

The function is normalised in such a way that the deflection on top of the dam is $\varphi(\zeta_0) = 1$:

$$\varphi(\zeta) = \left(\ln \frac{\zeta_0}{\zeta_A} \right)^{-1} \ln \frac{\zeta}{\zeta_A} \tag{15}$$

From equation (15) the normalised rotation of shear is achieved:

$$\Phi(\zeta) = \varphi'(\zeta) = \left(\ln \frac{\zeta_0}{\zeta_A} \right)^{-1} \frac{1}{\zeta} \tag{16}$$

The approximated rotation of shear $\gamma^*(\zeta,t)$ is then defined:

$$\gamma^*(\zeta,t) = q(t)\,\Phi(\zeta) \tag{17}$$

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:

$$E_{kin} = \frac{1}{2} \int_{\zeta_0}^{\zeta_A} \rho\, A(\zeta)\, \dot{\eta}^{*2}(\zeta,t) d\zeta = \frac{1}{2} m^* \dot{q}^2(t) \tag{18}$$

From equation (18) the equivalent mass m^* can be determined by using the *Ritz* separation approximation. Integration yields:

$$m^* = \frac{\rho\, A_A}{4\,\zeta_A} \left(\ln \frac{\zeta_0}{\zeta_A} \right)^{-2} \left[\zeta_A^2 - \zeta_0^2 \left(1 + 2 \ln \frac{\zeta_0}{\zeta_A} - 2 \left(\ln \frac{\zeta_0}{\zeta_A} \right)^2 \right) \right] \tag{19}$$

The potential energy is approximated by the strain energy of an equivalent spring coefficient k^*, deforming according to equations (14) and (17):

$$E_{pot} = \frac{1}{2} \int_{\zeta_0}^{\zeta_A} G\, A'(\zeta)\, \gamma^{*2}(\zeta,t) d\zeta = \frac{1}{2} k^* q^2(t) \tag{20}$$

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

$$k^* = -\frac{G\, A_A}{\kappa\, \zeta_A} \left(\ln \frac{\zeta_0}{\zeta_A} \right)^{-1} \tag{21}$$

The resulting *Lagrange* equation of motion of the idealised dam is that of the linear oscillator with the natural frequency ω_0:

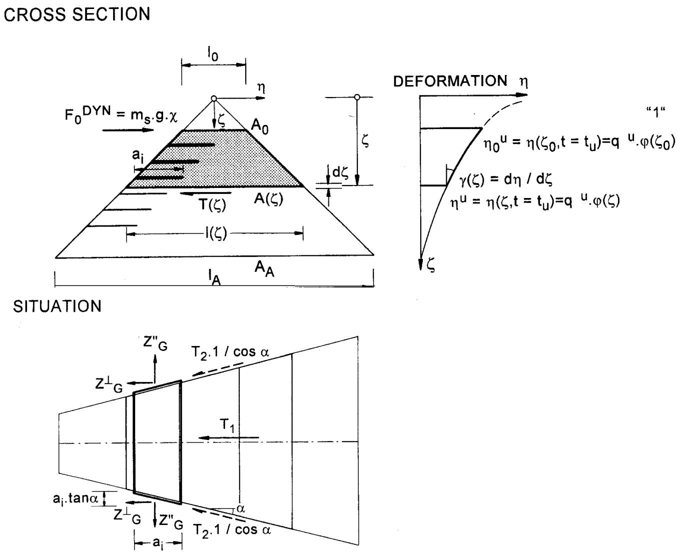


Figure 24: Limit equlibrium design of a geosynthetic reinforced protection dam. The failure body is assumed to be wedge shaped. (a) Cross section. (b) Situation. (c) Maximum deformation curve.

$$\ddot{q}(t)+\omega_0^2\, q(t)=0, \quad \omega_0=\sqrt{\frac{k^*}{m^*}} \tag{22a, b}$$

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 idealised by a punctual inelastic impact.

3.3.3 *Idealised 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 23). 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 F_0 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 generalised velocity after impact can directly derived from the *Rayleigh-Ritz* approximation defined in equation (14):

$$\dot{\eta}^*(\zeta,t)=\dot{q}(t)\,\varphi(\zeta) \tag{23}$$

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:

$$\dot{\eta}'^*(\zeta_0,t)=\dot{q}'(t)\,\varphi(\zeta_0)=v'_S \qquad \text{or} \qquad \dot{\eta}_0'^*=\dot{q}'(t)=v'_S \tag{24a, b}$$

Applying the momentum relation on each partial system taking into account the condition of idealised inelastic impact (equation 14) yields the velocity to common both bodies:

$$\dot{\eta}_0'^*=\dot{q}'(t)=v'_S=\frac{m_S}{m_S+m^*}\,v_S \tag{25}$$

Taking into account the conservation of energy maximum deflection $\eta_0{}^u = q^u$ of the dam can be calculated. Conservation of energy requires:

$$E'_{kin} + E'_{pot} = E^u{}_{kin} + E^u{}_{pot} \tag{26}$$

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:

$$E'_{kin} = \frac{1}{2} m * \dot{q}'^2 + \frac{1}{2} m_S \, v_S'^2 \tag{27}$$

$$E^u{}_{pot} = \frac{1}{2} k * q^{u^2} \tag{28}$$

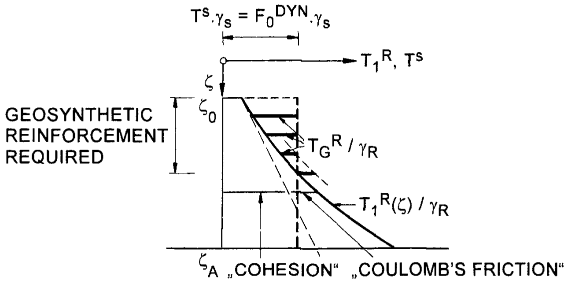


Figure 25: Limit equilibrium design. Driving forces T^S versus resisting forces $T_1{}^R$.

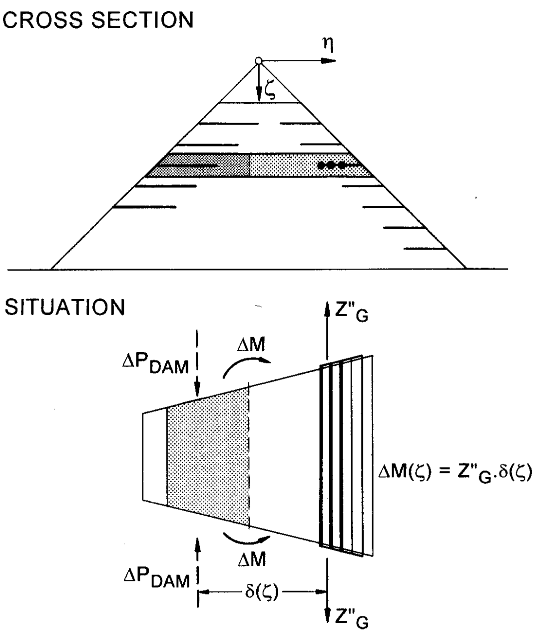


Figure 26: Limit equilibrium design. Flexural stresses from bending moment covered by geosynthetic reinforcement embedded in the outer dam slope. The maximum reinforcement tensile force is dependant on the ultimate dam pressure ΔP_{DAM}.

Combining equations (27) and (28) with (26) finally yields the maximum deflection of the dam according to a horizontal impact of a rigid body:

$$\eta_0 *^u = q^u = \frac{m_S\, v_S}{\sqrt{k*(m_S + m*)}} \tag{29}$$

The maximum deformation function over the total height of the dam is given by equations (11) and (14) combined with equation (29):

$$\eta *^u(\zeta) = q^u\, \varphi(\zeta) = \frac{m_S\, v_S}{\sqrt{k*(m_S + m*)}} \left(\ln\frac{\zeta_0}{\zeta_A} \right)^{-1} \ln\frac{\zeta}{\zeta_A} \tag{30}$$

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.

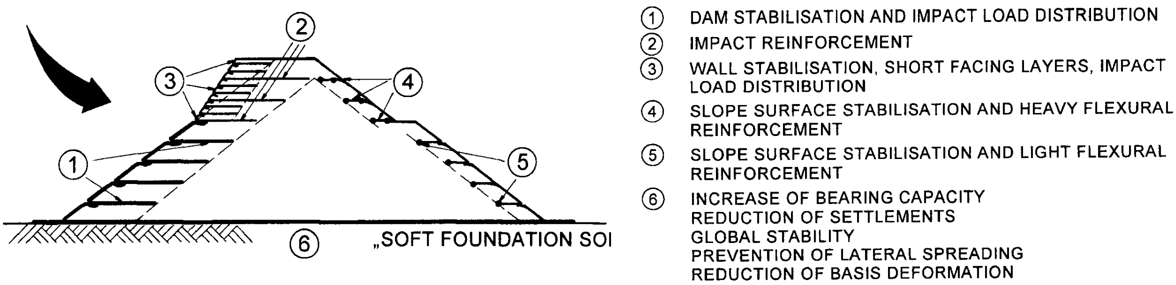


Figure 27: Example of a geosynthetic reinforced protection dam. Effect of different geosynthetic reinforcement layers.

It is recommended to perform static calculations introducing a horizontal force F_0 on top of the dam with the following magnitude:

$$F_0 = m_S\, g \tag{31}$$

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

$$\eta_0^{stat} = \eta_0 = -\frac{m_S\, g\, \kappa\, \zeta_A}{A_A\, G} \ln\frac{\zeta_0}{\zeta_A} \tag{32}$$

The magnification factor χ is defined by dividing the maximum dynamic deformation (equation 29) by the static deformation (equation 32):

$$\chi = \frac{\eta_0 *^u}{\eta_0^{stat}} = -\frac{v_S}{\sqrt{k*(m_S + m*)}} \frac{A_A\, G}{g\, \kappa\, \zeta_A} \left(\ln\frac{\zeta_0}{\zeta_A} \right)^{-1} \tag{33}$$

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, non-linear 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 24 is considered. The crown of the dam is horizontally loaded with the static force $m_S g$ multiplied with 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:

$$T^S = T(\zeta) = \chi\, m_S\, g = \frac{1}{\kappa} A(\zeta)\, G \frac{d\eta(\zeta)}{d\zeta} = const. \tag{34}$$

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 24. 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:

$$T_1^R = \frac{A_0}{2}\left(1+\frac{\zeta}{\zeta_0}\right)\rho\, g\,(\zeta-\zeta_0)\tan\varphi + A_0\frac{\zeta}{\zeta_0}c \tag{35}$$

  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:

$$T_2^R = \frac{l_A}{\zeta_A}\lambda_0 \rho g \tan\varphi\left[\frac{1}{3}\left(\zeta^3-\zeta_0^3\right)-\frac{1}{2}\zeta_0\left(\zeta^2-\zeta_0^2\right)\right]+\frac{l_A}{\zeta_A}\frac{c}{2}\left(\zeta^2-\zeta_0^2\right) \tag{36}$$

  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.

- 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. A linear tensile stress-strain 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 24b 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:

$$T_G^R = a_i \tan\alpha \min\left|Z_G^{normal}, Z_G^{parallel}\right| \tag{37}$$

A limit equilibrium design requires a clear definition of safety factor. In the case of a geosynthetics 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:

$$T^S \, \gamma^S \leq \frac{1}{\gamma^R}\left[T_1^R \left(+2\, T_2^R\right) + 2\, T_G^R\right] \tag{38}$$

In Figure 25 the driving forces T^S due to a dynamic impact on the dam crown 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 dam is impacted on a locally limited area, primary shear forces are produced which can 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 dam. In an earthfill and a rockfill dam tensile and flexural stresses cannot be taken over, so that failure can occur due to this kind of load. Geosynthetic inclusions with high ultimate tensile force placed in the outer dam slope lead to an distinctive increase in the factor of safety. Compound between geosynthetics and soil is essential. 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 over along the embedded geosynthetics. An approximate calculation can be performed in order to estimate the additional bending moment taken by the geosynthetic layer. Therefore, a finite section of the dam is considered shown in Figure 26. The bending moment can easily be calculated:

$$\Delta M(\zeta) = Z_G^{parallel}\, \delta(\zeta) \tag{39}$$

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.

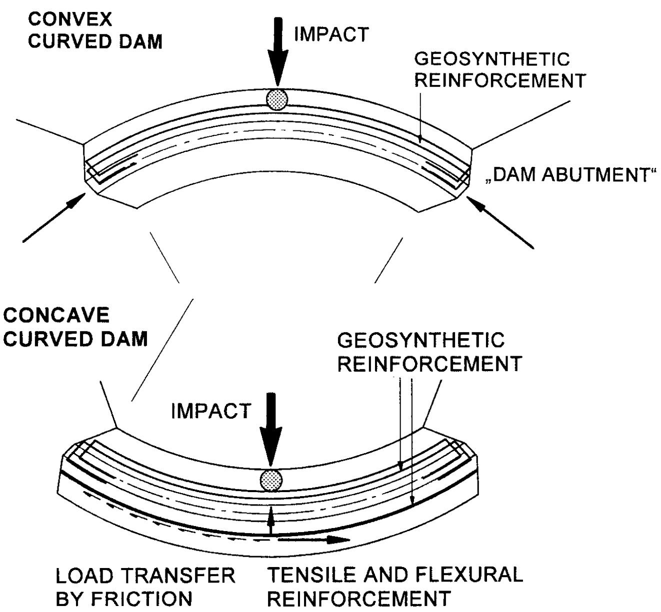


Figure 28: Different ground plans of geosynthetic reinforced protection dams. (a) Convex curved dam. (b) Concave curved dam.

3.4 *Design of geosynthetic reinforced protection dams*

Geosynthetic reinforcement of a protective dam improves the resistance of the structure significantly. Slope angles can be increased over the non-reinforced condition. Furthermore, long term surface protection is provided, whereby 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.

With respect to design, the process involves modification to conventional calculation procedures. As shown in the chapters above 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 27 an example of an 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)

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 (Figure 28a) 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 (Figure 28b).

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

3.5 *Case histories*

Figure 29 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 29 was constructed in 1981 and has withstood extreme impacts since. Long-term monitoring has confirmed excellent behaviour; some data were published in Brandl 1998.

Figure 30 shows another example where a small town had to be protected from large-scale rockfall and severe rockslide. The design assumed about 200.000 m^3 of rockfall material, whereby possible masses of up to 1 million m^3 were discussed. The 200 m long protective embankment dam had to be constructed in the flattening zone of a slope consisting of natural soil as well as of old mining deposits. About 180,000 m^3 fill material had to be placed within two months, always under the pressure of active rockfalls. In order to achieve a large catchment volume at the hillside of the embankment and to increase its resistance towards large-scale impacts, the upper zone was steepened and reinforced with geocomposites: 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/m^2

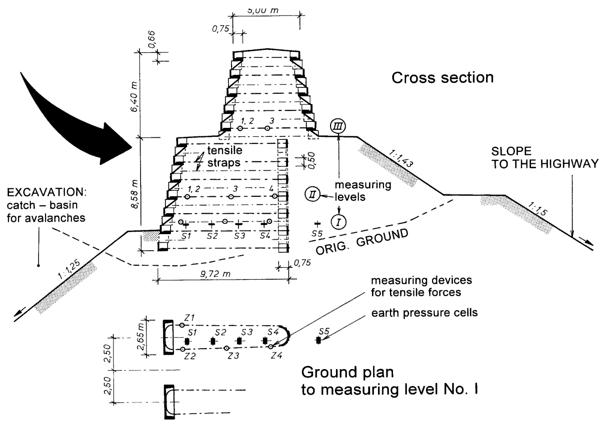


Figure 29. Protective embankment dam against large-scale avalanches and debris flows. Soil reinforcement on upper part with a loop-anchored wall system. Cross section and ground plan with measuring details.

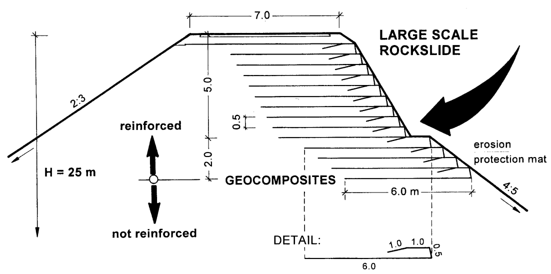


Figure 30. Protective embankment dam against large scale rockfalls and landslides. Soil reinforcement on upper part with geocomposites; slope cover with humus and geosynthetic erosion mat.

4 GEOSYNTHETICS FOR JACKETING PILES, GRANULAR COLUMNS AND GROUT PIPES

4.1 *Pile jacketing*

Geotextiles/geosynthetics used in these applications serve multiple functions: filtration, separation and reinforcement (by containment).

Jacketing of in-situ-cast concrete piles has been successfully used in Austria since 1973 (Brandl 1977). The first utilisation goes back to pile installation in very soft, saturated soil to avoid local constriction or uncontrolled bulging. Moreover, the negative skin friction of piles below high fills can be thus reduced (e.g. for abutment piles adjacent to high embankments). In very coarse material, on the other hand, pile jacketing prevents running off of fresh concrete or its being washed out. Figure 31 shows an example where a temporary support of a collapse endangered highway bridge was required. The temporary bridge was founded on piles through a coarse grained access fill and river boulders. The flow velocity of the river was locally about 10m/s.

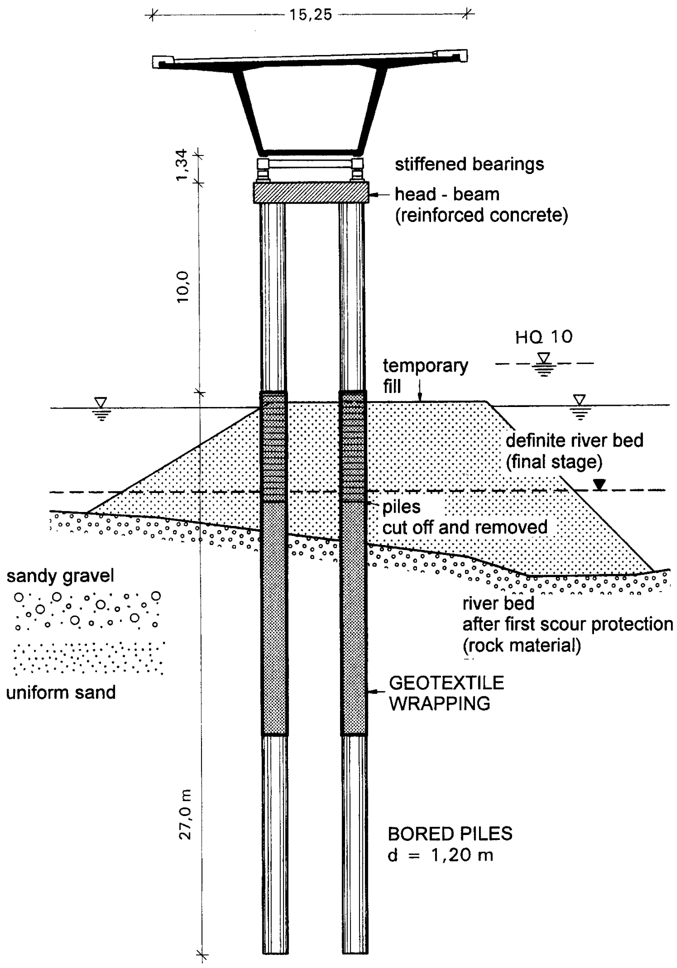


Figure 31. Cross section through the temporary support of a collapse-endangered river bridge. Jacketing of large diameter bored piles.

Pile jacketing has also proved suitable in ground exhibiting large voids, cavities, known or unknown mining openings etc. Further experience could be gained in loess areas with historical galleries and cellars excavated in this cohesive soil. The position and extent of such subsurface openings is frequently uncertain, which would cause excessive loss of pile concrete and of pile bearing capacity. Similar problems exist in urban areas due to old subsurface voids (e.g. from abandoned pipes, shafts etc.).

Large diameter bored piles are sometimes directly used as columns of multi-storeyed subsurface buildings, especially for underground garages. A cost effective top to bottom construction is possible by jacketing the piles which are then unwrapped during or after soil excavation, thus representing columns which are architecturally integrated in the building.

Furthermore, pile jacketing may be essential if the piles are used simultaneously for foundation and for heating or cooling. Such “energy piles “ (Brandl, 1998) require high integrity, and constricted zones would reduce their geothermal efficiency significantly.

Pile jacketing can be performed with nonwoven geotextiles or with stretchable geocomposites which are wrapped around the reinforcement cage. The seam of the geosynthetic must be welded or mechanically connected, overlapping alone is not sufficient. When the pile casing is withdrawn, the lateral pressure of the fresh (liquid) concrete stretches the geosynthetic radially, whereby sufficient cover of the steel reinforcement is achieved.

The stress-strain behaviour of the geosynthetic should be adapted to the lateral earth pressure which acts against the liquid concrete pressure within the geosynthetic sleeve.The earth pressure coefficient may vary between K_o = 0 to 1, whereby K_o = 0 represents voids and K_o = 1 soft, saturated soil of a liquid consistency.

According to the pressure vessel formula, the ring tensile force in the geosynthetic sleeve is

$$T_r = p\, r = K_{0,c}\, \gamma\, h\, r \qquad (40)$$

where p = radial inner pressure
$K_{o,c}$ = 1 for liquid concrete
γ = density of concrete
r = pile radius
h = depth of pile (or height of column).

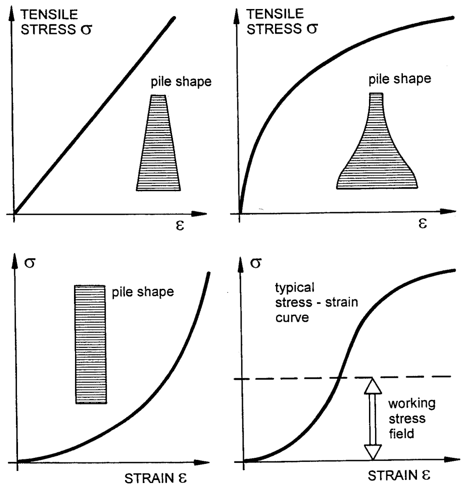


Figure 32. Stress-strain characteristics of geosynthetic sleeves for pile jacketing and subsequent pile deformation. Schematic

From equation (40) it can be seen that the ring force in the geosynthetic is proportional to h if no lateral earth pressure acts, i.e. in cavities etc. Three basic cases of stress-strain behaviour and pile deformation may be distinguished then (Fig. 32): A linear σ - ε correlation leads to truncated cone-shaped piles (a), whereas underlinear behaviour (tension stiffening) creates vertical pile forces with the exception of local constriction on pile top if there is no vertical load (b). Overlinear σ - ε behaviour (tension softening) leads to a bell-shaped pile (c), whereby instability is indicated by increasing deformation. Consequently, geotextiles with a relatively large initial strain followed by a tension stiffening period have proved most suitable. The working load should be clearly below the transition zone from tension stiffening to tension softening.

A proper stress-strain characteristic of the geotextile is especially important if the piles should later on be used as visible columns of a subsurface structure, e.g. for deep underground garages. Otherwise a conventional formwork must be placed around the piles/columns during the unwrapping procedure.In the case of sufficient lateral ground support, the ring tension force in the sleeve is relatively low and radial deformation of secondary importance.

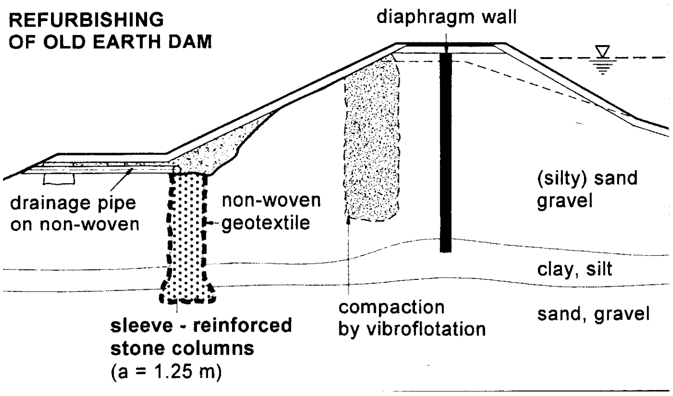


Figure 33. Improvement of static and hydraulic stability of an old embankment dam against floods.

Jacketed piles are frequently installed in connection with piled embankments, whereby the pile loads are covered with geosynthetics. If this cover is designed as a multi-layered soil geosynthetics composite body, it acts like a damping cushion against dynamic forces. Comprehensive in-situ measurement on low railway embankments have disclosed that the forces from the traffic acting on the pile heads are significantly smaller than hitherto assumed (and required in several codes, standards or regulations).

Pile jacketing with geosynthetics is also used for the rehabilitation of old, deteriorated piles. Basically, this concept uses a jacket of geotextile as a concrete form. The annular space between the form and pile can be filled with either concrete grout or specially formulated epoxy (Koerner 1998). The advantages of these flexible geotextile forms are light weight, ease of installation, adaptability to any configuration and low cost.

The optimum stress-strain characteristics of geotextiles jacketing old piles differ from those for jacketing cast-in-situ concrete piles under construction: The elongation of the geotextile under load should be kept to a minimum for the rehabilitation of old piles (to save costs), but should be relatively large for casting new concrete piles (to achieve sufficient cover of steel reinforcement).

Proper opening size of the geotextile must prevent loss of cement and sand. Some bleeding of the fresh concrete (or grout) will occur, but this will increase rather than decrease the strength.

4.2 *Jacketing of granular columns*

Jacketed (coated) stone and sand 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 protection earth dams. This method has 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. 33). The drainage material (usually clean 4/32 mm or 8/32mm – grain) is lowered by vibroflotation whereby the vibrator is wrapped with a nonwoven geotextile (tied together at the toe of the vibrator).

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 ships into the chamber at the top of the vibrator and placed at depth. In order to avoid damaging the geotextile, the vibrator is sometimes at first lowered without the geotextile geotextile sleeve into the ground to displace soil. This has proved suitable in coarse or stiff subsoil but is not necessary in fine-grained soft ground.

A leak in the geotextile is quickly recognised due to an increasing volume of backfill material during vibrating. 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 ∅)	15 mm
Mass	325 g/m^2
Thickness at 2 kN/m^2	2.5 mm

The next innovative step was the development of sleeve-reinforced stone and sand columns which primarily have a static function. Conventional stone columns may fail as shown in Figure 34. Coating is an effective means against excessive bulging and shear failure. Therefore, coated granular columns (mostly consisting of sand or gravel) are increasingly used for "piled" embankments on very soft ground – e.g. in peat where conventional stone columns are hardly proper unless they are partly grouted. Coated granular columns serve as load bearing elements, for primary settlement (=consolidation) acceleration and for stability increase against base failure due to excessive pore water pressure. It should be emphasised that secondary settlement (creep) cannot be accelerated by sand or gravel columns – just as it is not possible to do so by vertical drains. Consequently, the application of jacketed granular columns (and vertical drains) depends widely on the settlement properties of the soil, if settlements are the primary design criterion and less load-bearing aspects (Fig. 35).

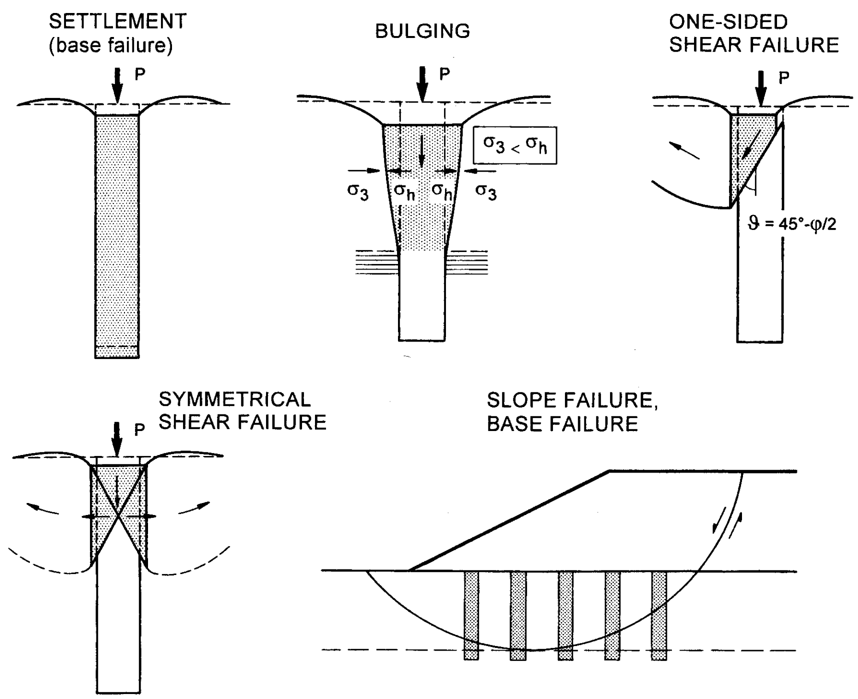


Figure 34. Failure modes of granular columns (stone or sand columns, etc.).

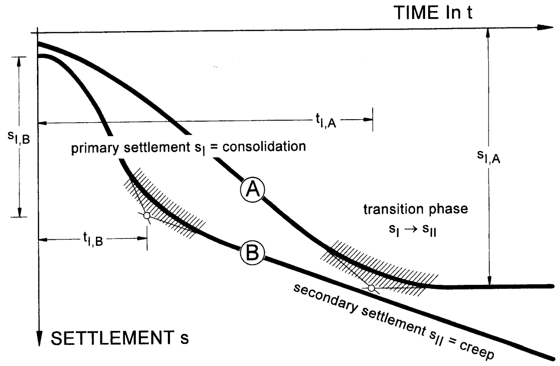


Figure 35. Time-settlement behaviour of soft soils and suitability of jacketed granular columns (or vertical drains). Schematic. A = suitable; B = not suitable.

Jacketed granular columns have proved suitable especially in the case of embankment widenings on very soft ground. The method minimises differential settlements between the old and the new parts of the embankment. Figure 36 shows an application for widening a highway on peat, whereby settlements of 0.5 to 1.0 m were expected. The fill material was 8/16 mm grain and the coating a high-strength woven geotextile. This sleeve was wrapped around a tube (∅ 0.2 m) which was pressed into the ground and withdrawn after filling. The columns were 5 to 6 m long and installed in a 1.5 m grid (Dietrich, Bräu 2000). In this case, settlement acceleration was the dominating factor rather than load transfer. The columns could be installed very quickly (about 150 pieces per day), and they are – in case of large settlements - more resistant to damages than conventional vertical drains.

The main field of application of jacketed granular columns as load bearing elements (and for settlement acceleration) are “piled” embankments. Commonly, the heads of the columns are covered with geosynthetics or multi-layered geosynthetic-soil granular material composite systems to optimise load distribution and to facilitate a proper compaction of the further fill layers of embankments. Figure 37 shows an example when column jacketing was not jet experienced. In order to avoid failure in the soft peat, stone columns were locally grouted or (unreinforced) concrete was used as backfilling material for the vibroflotation technique.

Today, geosynthetic-reinforced granular columns with a relevant load bearing effect (in addition to settlement acceleration and decrease of pore water pressure) exhibit a diameter of typically 0.6 to 1.5 m. They are installed by the vibroflotation technique – or by the soil replacement or soil displacement method, using steel tubes which are then withdrawn. Commonly, the columns are placeed in a regular grid, but their spacing may be varied according to locally prevailing ground properties.

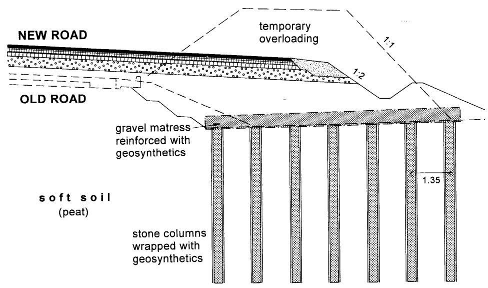


Figure 36. Widening of a highway over peaty soil (after Dietrich, Bräu 2000). Geosynthetic-coated stone columns covered by a basal mattress (geosynthetic-reinforced gravel) as embankment foundation.

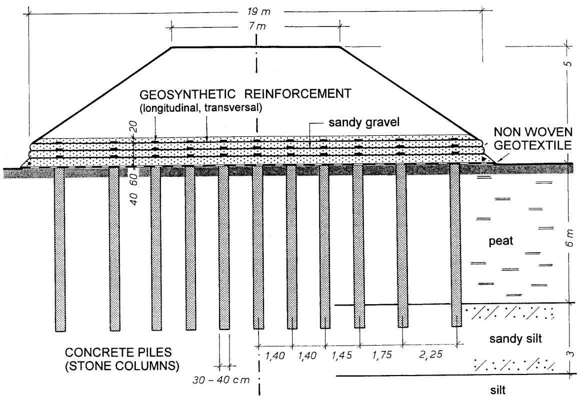


Figure 37. Piled embankment with geosynthetic-soil composite body on top of the piles or stone columns. Early solution (1973) with non-woven geotextiles geosynthetic strips.

Figure 37. Piled embankment with geosynthetic-soil composite body on top of the piles or stone columns. Early solution (1973) with non-woven geotextiles geosynthetic strips.

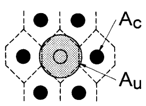


Figure 38. Partial view of an embankment foundation with geosynthetic-coated granular columns (sand, gravel or recycling products). Schematic ground plan with "unit cell" (hatched).

Contrary to jacketed stone columns installed by vibroflotation, the steel tube technique uses a casing. After lowering the tube by displacing or excavating the soil, the geosynthetic jacket (commonly a polyester woven geotextile) is placed into the steel tube with typically a larger diameter than the tube. During backfilling the jacket, the casing is withdrawn continuously – similar to in-situ cast concrete piles.

Geotextile-jacketed granular columns can be installed also in very soft subsoil where conventional stone columns are not suitable, i.e. in soils with an undrained shear strength lower than about $c_u = 15$ kN/m^2. The jacket provides filter stability between the granular column and fine grained or peaty soil.

The soil displacement method leads to stress constraints and local densification of the surrounding soil, hence to an improvement of the bearing-deformation behaviour of the entire system. On the other hand, it may reduce the hydraulic conductivity around the column due to a smear zone, thus somewhat increasing the consolidation period.

Because of the arch effect and the load distribution in the case of a flexible foundation, sufficient surcharge load is essential. Consequently, "piled" embankments for high-speed railways (and highways) should exhibit a certain minimum height: usually about ≥ 1.0 m, depending on the traffic loads and on the stiffness ratio of columns to soil, on the diameter and spacing of columns, and on the performance of the column head cover (e.g. geosynthetic-reinforced soil).

Contrary to conventional stone columns, the sleeve reinforcement of coated columns can take relevant parts of the horizontal stress within the granular infill by ring tension forces. This reduces the deformation of the columns significantly and hence the settlement of the embankment.

Figure 38 shows a partial ground plan of a column grid and an idealised “unit cell” which is the basis of a calculation model according to Figure 39. The “unit cell” is defined by a transformation of the influence area A_u into a circle of equal area (Raithel, Kempfert 1999). This bearing system simplifies the stress conditions within and around the jacketed column but provides, nevertheless, practicable results which are in good correspondence with site measurements.

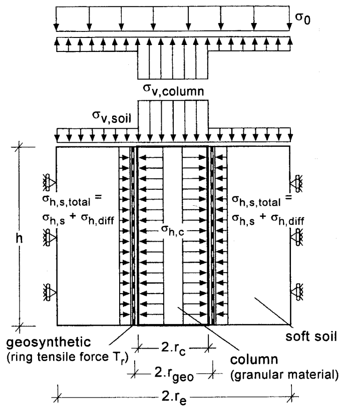


Figure 39. Bearing system and calculation model to Figure 38 (after Raithel, Kempfert 1999).

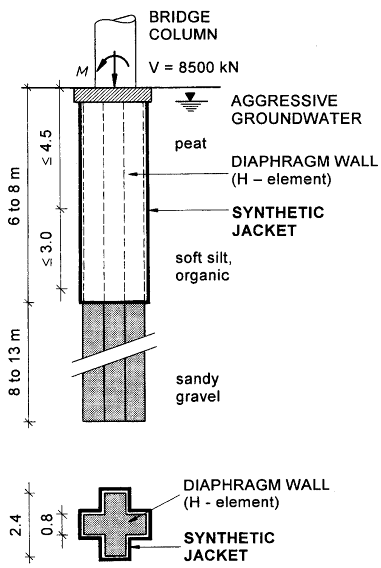


Figure 40. Deep foundation of a bridge pier in highly aggressive soil. H-shaped diaphragm wall elements protected by a geosynthetic coating.

Sleeve-reinforced granular columns exhibit a flexible and self regulating bearing behaviour, because the external loads are automatically transferred more to the surrounding soil if the columns yield. Simultaneously, the horizontal deformation activates an increasing soil resistance (passive lateral earth pressure) which causes a stress redistribution, hence an increasing bearing capacity of the column.

The calculation of the jacketed columns can be performed numerically or analytically, whereby the installation method has to be taken into account. Displacement technique provides a higher lateral earth pressure coefficient ($K_o < K < K_p$) in the surrounding soil than the replacement method ($K \leq K_o$). The analytical calculation makes use of the "pressure vessel formula" which is of sufficient accuracy in practice (Raithel, Kempfert 1999): The horizontal tensile stress in the geosynthetic is

$$\Delta\sigma_{h,geo} = \frac{\Delta T_r}{r_{geo}} \tag{41}$$

where ΔT_r is the ring tensile force in the geosynthetic jacket with a radius of r_{geo} , which is typically somewhat larger than the installation radius of the column r_c.

From the different horizontal stresses in the column ($\sigma_{h,c}$), the soil ($\sigma_{h,s}$), and the geosynthetic ($\sigma_{h,geo}$), a differential stress, $\Delta\sigma_{h,Diff}$, can be deduced:

$$\Delta\sigma_{h,Diff.} = \Delta\sigma_{h,c} - (\Delta\sigma_{h,s} + \Delta\sigma_{h,geo}) \tag{42}$$

This corresponds to the mobilisation of an additional earth pressure in the surrounding soil until an equilibrium of lateral stresses is reached. The interaction is illustrated schematically in Figure 8, whereby the differential stress (equation) results in an expansion of the column.

Parametric studies disclosed that the ring tensile forces and settlements depend significantly on the stiffness of geosynthetic sleeves and on the grid of the columns. The compression modulus of the soil and the surcharge load (embankment height) have, of course, also an essential influence on the load settlement behaviour of the jacketed granular column system.

The allowable ring tensile force for long-term function, hence the design tensile strength, depends not only on the factor of safety (FS) but also on geosynthetic-typical reduction factors (RF):

$$T_{allow} = T_{ult}\left(\frac{1}{RF_{CR} + RF_{IN} + RF_{CC} + RF_{BC} + RF_{DYN}}\right)\frac{1}{FS} \tag{43}$$

where

T_{allow} = allowable tensile strength ($\leq\Delta T_r$),
T_{ult} = ultimate tensile strength (from short-term tests),
RF_{ID} = reduction factor for installation damage,
RF_{CR} = reduction factor for creep,
RF_{CD} = reduction factor for chemical degradation,
RF_{BD} = reduction factor for biological degradation,
RF_{DYN} = reduction factor for dynamic influences,
FS = factor of safety for site-specific additional terms.

The reduction factors could also be interpreted as partial factors of safety, and FS would represent then an additional site-specific term. This would, of course, lead to other values for RFs. But the more appropriate terminology refers to reduction factors, and if they are chosen properly, the (additional) factor of safety could be assumed FS = 1.

4.3 *Jacketing of diaphragm walls and grout pipes*

For deep foundations in aggressive groundwater diaphragm wall elements with chemically resistant synthetics may be necessary if concrete additives and/or enlarged cross-sectional areas are not sufficient. In the case of diaphragm elements, these protective jackets must be relatively stiff prefabricated "boxes" which are lowered into the suspension of the slurry trench (Fig. 40). The reinforcement cage should be placed then very cautiously, and finally the concrete is cast according to the two-phase construction technique of such deep foundations (thus replacing the suspension). Usually, the reinforcement cage is wrapped with a rather stretchable geomembrane or geocomposite incorporating a geomembrane and nonwoven geotextile(s) as protective cover. The elongation of the geomembrane must provide sufficient concrete cover for the reinforcement when casting the concrete. If a stiff jacket is mounted to the reinforcement cage, spacers must guarantee a certain gap between steel reinforcement and geosynthetic in order to achieve sufficient concrete cover.

This method has been used successfully in Austria since 1972. In addition to their chemically protective function, the smooth geosynthetic jackets have the following geotechnical advantages:

- Minimisation of concrete volume because uncontrolled lateral bulging (flowing out of the fresh concrete into the soil) is prevented.
- No local constriction of the fresh concrete by soft (squeezing) soil.
- Reduction of negative skin friction.

Grouting in soil or fills with large voids and cavities is facilitated by using geotextile-wrapped grout pipes in the first phase of grouting. This method has proved successful to save bridge piers near collapse because of deep river bed scouring (Brandl 1994), and it is also used to stabilise abandoned mines (Koerner 1998). Grout is injected under controlled pressure, and expansion of the geosynthetic occurs where only small or no resistance is met, i.e. where voids exist. Thus, a primary structure is formed in the ground, and subsequent grouting can be performed without excessive loss of grout material.

5 GEOSYNTHETICS FOR BARRIER WALLS

Cut-off walls have been utilised in geotechnical and hydraulic engineering since decades. In environmental geotechnics they are increasingly used as vertical barriers to encapsulate contaminated ground or contaminant sources (abandoned landfills, special industrial plants etc.) and for landfill containment. Design and construction have to distinguish:

- If the cut-offs are temporary or permanent measures.
- If the cut-offs are reaching into an aquitard or only into a lower permeable stratum (hydraulically “incomplete” screens).
- If the cut-offs should be only a hydraulic barrier or also a barrier against diffusion of contaminants.
- If the cut-off walls have only a barrier effect or also a statical function.

Consequently, there are several systems: Table 2 gives an overview of the currently used cut-off wall technologies and their capacity. For permanent waste containment diaphragm walls and vibrating beam slurry walls (“thin diaphragm walls”) predominate.

Geosynthetic cut-off walls or “geomembrane barrier walls” are increasingly installed in soft or loose ground, whereas combined slurry trench walls with integrated geosynthetic screens are used in all ground foundations. The following technologies have proved suitable:

- In-situ installed HDPE-panels (lowered by vibration). Depth usually 2 to 20m.
- Excavation of a trench and placing a seamed geomembrane in it and then backfilling it, thereby pushing the liner to one side of the trench. Depth limited to 1.5 to 15 m.

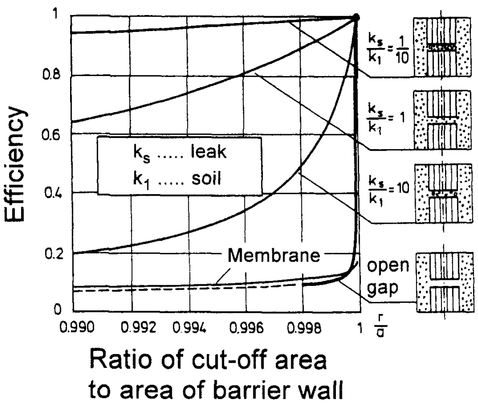


Figure 41. Barrier effect of cut-off walls with a leak (joint). Comparison of geomembrane and thick wall, and influence of partial clogging of joints (after Brauns, 1978). Example for a soil with effective grain size d_{w1} = 0.2 mm.

a = axial spacing of cut-off panels (= axial distance of joints).

r = effective length of cut-off panels.

k_s = hydraulic conductiviy of leak filling (due to clogging),

k_1 = hydraulic conductivity of surrounding soil.

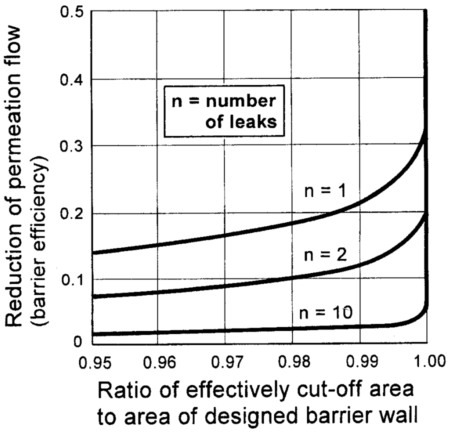


Figure 42. Barrier effect of cut-off walls with one or more leaks of the same total (cumulative) area. Behaviour of a geomembrane for comparison (after Dachler, 1936).

- Installation of geomembranes or geocomposites in slurry trench walls (conventional diaphragm walls) or vibrating beam slurry walls (thin diaphragm walls). Depth up to about 50 m. This technology is – at least in Europe – the most common one. Usually, only one geomembrane sheet is installed. Since most geomembrane materials have a specific gravity near 1, it is necessary to weight the bottom, so that it sinks properly.

Geomembrane cut-off walls are very sensitive to leaks. A leak of only 1 % of the screen area makes already a significant discharge of seepage water possible (Figure 41). The barrier efficiency is only 10 to 20 % then (depending on theoretical hydraulic assumptions), and it drops further if there are more leaks which in total exhibit the same area as one large leak (Figure 42). On the other hand, clogging of leaks leads to a significant improvement of the barrier effect (Figure 41). Consequently, single geomembrane walls should be used rather for secondary purposes, in the case of low contaminant potential and for temporary measures. For the permanent containment of polluted areas with an excessively high contaminant potential slurry trench walls or geosynthetic twin-walls, as well as systems with leak detection and leakage removal are recommended.

A geosynthetic composite with a central drainage within the slurry trench wall enables the collection and removal of polluted seepage water which might have penetrated through one side of the double liner system. Another advantage of this system is that each section of the cut-off wall can be monitored and repaired if bulkheads are installed at certain intervals. Otherwise it would be rather difficult to locate leaks in a long and deep wall. The seam in the interlocks is achieved by cutting a slot and subsequent welding. In case of leaks, the space between the geomembranes can be grouted.

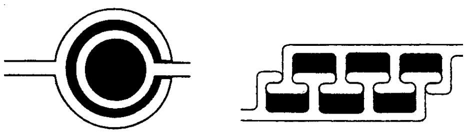


Figure 43. Locks of vertical geomembrane liners which can be grouted.

Another alternative is the insertion of sandwich-like composite panels into the slurry trench wall (diaphragm wall). In this case, a metal foil (usually aluminium) is placed between two HDPE-geomembranes. The method is theoretically promising if seepage or groundwater is heavily contaminated by chlorinated hydrocarbons, and against certain vapour migration. Comprehensive practical experience does not yet exist. The compatibility of the incorporated screen and the surrounding sealing mix must be proven design- and site-specifically.

The main problem of slurry trench walls with integrated geomembranes is the interlocking of the panels. Several variations of connections of one sheet to the next are shown in Brandl (1990) and Koerner and Guglielmetti (1995). Welding or grouting leads to a significantly higher barrier effect than a mere plug-in connection (Figure 43). Consequently, homogeneous extrusion welding is used increasingly, but also interlocks with hydrophile seals (Figure 44). Furthermore, such composite cut-off walls can be constructed only after the slurry trench single-phase method whereby special mixes with additives to retard setting are required. The final density of the single-phase sealing masses is typically lower than in the case of twin-phase construction. Consequently, the barrier effect against contaminant propagation by diffusion is lower then.

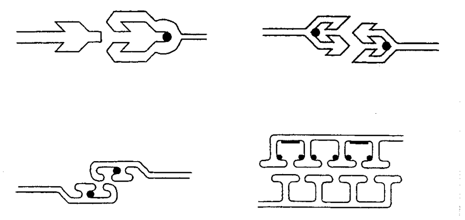


Figure 44. Locks of vertical geomembrane liners with hydrophile sealings.

The main purpose of cut-off walls with integrated geomembranes is a minimisation of hydraulic permeation. Vapour, transmission and diffusion are also reduced, whereby the thickness and density of the liner play in important role. Nevertheless, in the case of a high concentration ratio, a residual diffusion will always occur, even through high quality cut-off walls. But diffusion can be practically prevented by an inverse hydraulic gradient which is achieved by lowering the groundwater table within the cut-off wall.

Slurry walls are the most common type of cut-off walls used for environmental protection and permanent cut-off of waste deposits or contaminated land. Therefore, the following paragraphs focus on (thick) diaphragm walls and (thin) vibrating beam slurry walls.

The required hydraulic conductivity of cut-off walls should be defined in connection with the thickness and the system of the barrier. For instance, a cellular barrier system of vibrating beam slurry walls provides a higher safety than a conventional diaphragm wall ("one-screen system"). Consequently, the permittivity ψ should be considered the predominant parameter describing the hydraulic performance of a cut-off wall:

$$\Psi = \frac{k}{d} \quad \text{in sec}^{-1} \tag{44}$$

where k = hydraulic conductivity in m/s
d = thickness of the wall in m

The following values have proved suitable, depending on the cut-off wall system, the soil properties, degree of underground contamination, environmental risk etc.:

10^{-9} sec^{-1} $\leq \Psi \leq 10^{-7}$ sec^{-1}
for diaphragm walls (d $\geq$ 60 cm)
10^{-8} sec^{-1} $\leq \Psi \leq 10^{-6}$ sec^{-1}
for vibrating beam slurry walls (d $\geq$ 5 cm)

Consequently, a low k-value must be achieved for thin walls, and a higher hydraulic conductivity is allowed for thick diaphragm walls. Thus the width of the wall could largely be left to the choice of the contractor (according to his equipment etc.) unless other design criteria dominate. Commonly, the following permittivity, Ψ, should be required for the in-situ behaviour of the barrier system:

$\Psi \leq 10^{-8}$ sec^{-1} ... single cut-off walls
$\Psi \leq 10^{-7}$ sec^{-1} ... cellular cut-off wall systems

Additionally, the following hydraulic conductivity, k, is recommended, depending on specific design criteria:

k $\leq 10^{-7}$ to 10^{-10} m/s after 7 days*
k $\leq 10^{-8}$ to 10^{-10} m/s after 90 days*
*of samples cured under water

Table 2. Overview of methods for cut-off wall construction. Approximate values for common width d (m) and currently maximum wall depth t_{max} (m).

TECHNOLOGY	CUT-OFF SYSTEM	GROUND PLAN (schematical)	DIMENSIONS d (m)	DIMENSIONS t_{max} (m)
Permeability reduction of in-situ soil	compaction wall		0,4-1,0 [1)]	10-20
	grouting wall		1,0-2,5	20-80
	soil freezing wall		≥ 0,7	50-100
	jet grouting wall		0,4-2,5	30-70
			≥0,15-0,3 [2)] (lamella)	20-30
	soil mix wall		0,8-1,5	30-60
	cut-mix-grout wall		≥ 0,7	10
Soil displacement methods	geomembrane wall		≥ 0,005 (0,002) [6)]	20-40
	sheet pile wall		≈ 0,02	20-30
	vibrating beam -slurry wall "thin (diaphragm) wall"		≥0,05-0,2 [3)]	10-35
	earth concrete driven-sheet pile wall		≥ 0,4	15-25
Excavation methods	secant bored pile wall		0,4-1,5	20-40
	diaphragm wall (with hydrofraise)		0,4-1,6 [4)]	100-170
	diaphragm wall (with grab)		0,4-1,0	40-70
	diaphragm wall with incorporated liner(s)		0,6-1,0 (0,4-1,6) [5)]	20-50

1) vibrocompaction, vibroflotation (vibrodisplacement, vibroreplacement)
2) total width of the lozenge-shaped jet grouting walls: ≥ 0,5 m
3) near the flanges of the vibrating beam significantly wider
4) up to 3,0 m in special cases
5) in special cases
6) in the case of twin walls

The in-situ global hydraulic conductivity is typically higher than that of small test specimens. Here the interlocking of wall panels and the interface between sealing mass and surrounding ground play and important role. The in-situ barrier behaviour can be controlled best by piezometers on either side of the cut-off wall and especially by pumping tests (Groundwater lowering within the cut-off area – Brandl, 1994).

The higher values within the range of ψ and k can be accepted for cellular cut-off wall systems. Moreover, such barriers allow a somewhat smaller embedment depth in the low-permeability stratum than in case of single cut-off walls (Brandl 1994).

Slurry trench walls should meet these criteria independently whether they exhibit a geosynthetic inclusion or not. The above values are required for a high quality sealing mass which, in the end, interacts with the geomembrane by forming a twin-barrier.

Several regional regulations include lower permissible values for the hydraulic conductivity of cut-off walls. But the assumption that the barrier effect of cut-offs increases with decreasing k is not generally valid:

If an inverse hydraulic gradient is achieved by lowering the groundwater within the containment, a hydraulic conductivity of $k \leq 10^{11}$ m/s may be even disadvantageous with regard to diffusion. The properties of the sealing mix, the hydraulic gradient and the diffusion parameters interact closely and should therefore be adapted to each other. A proper design can practically prevent contaminant migration by diffusion.

This is demonstrated at the Vienna central waste deposit (Radl 1996, Brandl 1994). The site comprises 600.000 m^2 and was cut-off by vertical barriers in the year 1985. Organic substances, heavy metals, and especially chloride were selected as relevant tracer pollutants for comprehensive parametric studies, risk analyses, and quality assurance. Cl^+ ions exhibit hardly adsorption or precipitation and are very mobile, hence diffusing relatively easily through barriers.

The parameters of the cut-off walls were well documented during the construction period and have been continually monitored since. Accordingly, calculatory values leading to the results of Figure 45 are:

- thickness of thin diaphragm walls, d = 0.1 m,
- thickness of slurry trench walls, d = 0.4m to 1.2m,
- hydraulic conductivity of cut-off walls, $k = 10^{-8}$ to 10^{-12} m/s, with a mean value of $k = 10^{-9}$ m/s,
- hydraulic gradient between outer and inner groundwater level, i = 0.5 to 5,
- porosity, n = 0.2 to 0.4,
- Cl^+ concentration in natural groundwater, 5 to 50 mg/l,
- Cl^+ concentration below the waste deposit, 100 to 10.000 mg/l,
- diffusion coefficient for Cl^+, 2×10^{-10} m^2/s.

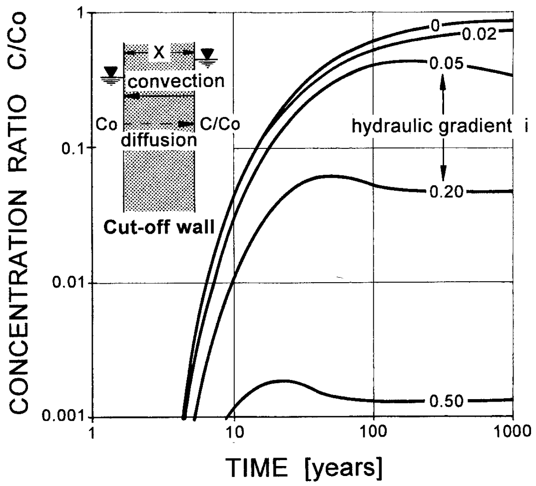


Figure 45. Contaminant diffusion against inverse hydraulic gradient through a cut-off wall (after Radl et al.1996). C/C_0 = concentration ratio.

The parametric studies and comparative calculations as well as the results from monitoring (since the year 1985) can be summarised as follows: For concentration of C_o= 100 to 1000 mg/l and C = 10 mg/l, the ration C/ C_o varies between 0 to 0.1. The hydraulic gradient usually varies between i = 1 to 3, corresponding to a minimum water table difference of 0.2 to 0.3 m. Local and temporary values of about i = 0.5 and i = 4 to 8 occur, whereby the later dominate. Advection and sorption may be neglected in relation to diffusion. Therefore the partial differential equation for contaminant transport through a cut-off wall reduces to non-stationary diffusion.

$$D_e \frac{\partial^2 C}{\partial x^2} = \frac{\partial C}{\partial t} \quad (45)$$

where D_e = coefficient of diffusion (m^2/s)
C = concentration of solution
x = way (m), t = time (s)

Figure 45 illustrates that under such conditions a contaminant migration by diffusion through the cut-off wall is negligible.

Theoretical investigations, site experience (especially construction difficulties with geomembrane liner installation) and long-term monitoring have disclosed that an inverse groundwater gradient has a higher and more reliable barrier effect against contaminant migration than the installation of geomembrane liners in slurry trench walls. Usually, a difference of groundwater tables inside and outside the cut-off of about 0.3 to 0.5 m is sufficient. This measure has also proved to be very cost-effective, and furthermore it facilitates leak detection, especially if the cut-off exhibits a cellular structure (two parallel screens with cross walls in about 50 to 100 m intervals) – Brandl, 1994.

Commonly, barrier walls are tight cut-off walls thus forming a passive containment. However, for in-situ contaminated groundwater cleaning, permeable walls are also utilised, designed as reactive walls of funnel and gate systems (Fig. 46). The contaminant plume is then flowing through a straight or curved wall or is directed to a gate. Groundwater cleaning in the reactive wall or gate is performed site-specifically, whereby physical, chemical, and/or micro-biological measures are possible. Several systems contain exchangeable geosynthetic filter panels, but also geotextiles to encapsulate special (granular) reactive material.

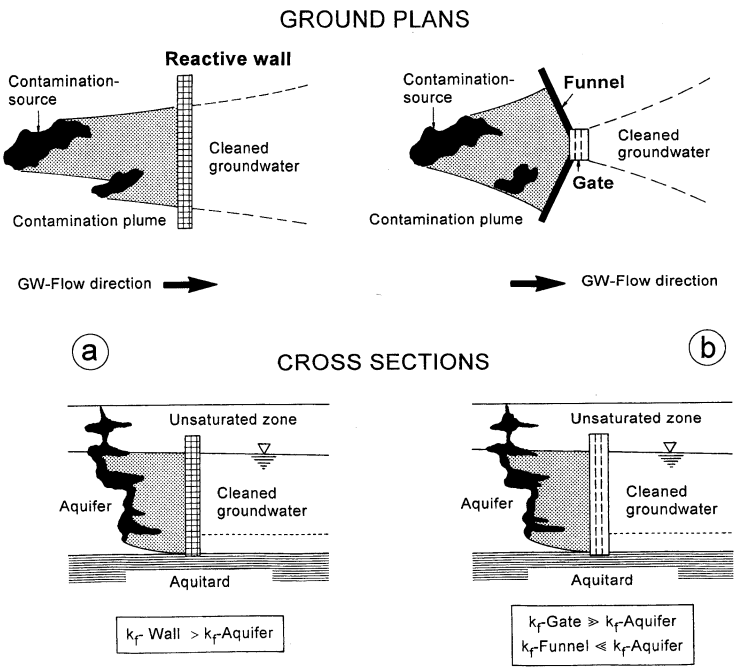


Figure 46. In-situ groundwater cleaning with permeable reative walls (a) or "funnel and gate" system (b).

A barrier wall may also consist of a row of consecutive wells for groundwater cleaning, or of an alternating sequence of cut-off wall elements and reactive walls. Figure 47 shows a reactive barrier system where trichloroethene contaminated groundwater is channelled by cut-off walls to flow through reactor vessels with iron filling. There the contaminant, a chlorinated solvent, is broken down to water and carbon dioxide. The reaction medium (in this special case zero-valent iron) is enclosed by a geotextile filter in its base and on top. Because of the inlet pipe being installed at a higher level than the outlet, a possible precipitation would collect on the geotextile above the iron fillings where it can be easily removed (Soudain, 1997).

Barrier walls with reactive elements have been increasingly used for in-situ groundwater remediation since about 1995. A main advantage of this method is that once installed, it usually requires little maintenance and monitoring.

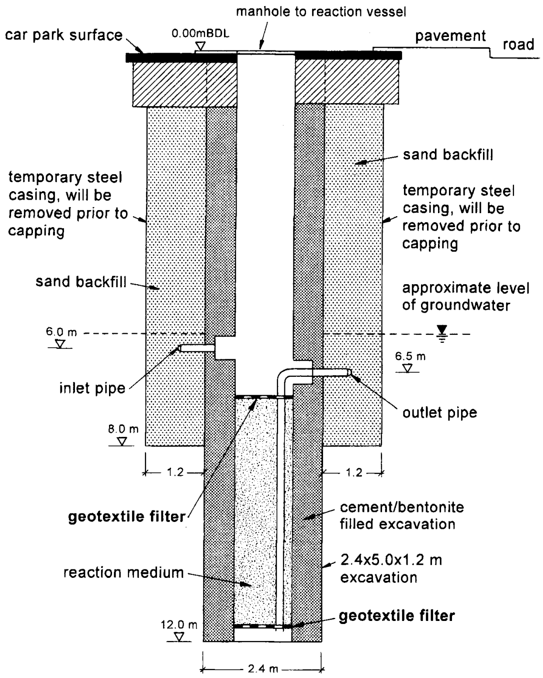


Figure 47. In-situ groundwater cleaning in a reactor vessel installed within a slurry trench cut-off wall (Soudain, 1997).

6 GEOSYNTHETICS FOR WASTE CONTAINMENT

6.1 *General*

Disposal facilities of municipal/household waste, industrial waste and especially of hazardous waste should be designed and constructed according to the “multi-barrier system”. This term originates in nuclear engineering and originally defines a security system consisting of several protective measures (“barriers” which act independently from each other). It was then taken over and extended in the waste disposal terminology and has been widely used in Germany and Austria since.

The multi-barrier concept comprises natural and man-made (“technical”) barriers. In case of waste deposits above ground, these barriers include (Fig. 48):

- Natural barrier (“geological” barrier), incorporating proper site characteristics from a geotechnical and hydrological point of view;
- Horizontal barrier (bottom liner and drainage system);
- Capping barrier (cover and/or liner and drainage system);
- Vertical barrier (cut-off walls) - not obligatory.
- In a broader sense, “multi-barrier systems” also include the deposit and the waste itself.

The pre-treatment of the waste and the operation technology of the landfill therefore play a significant role within the framework of a safe, well-managed disposal facility.

Figure 49 gives an overview of several geotechnical aspects which have to be considered for the design, construction, operation, and aftercare of a waste deposit. The main problems with regard to geotechnical engineering are large differential settlements in the base of the waste deposit, slope stability and long-term behaviour of the liners, drainage and leachate removal systems. This has opened a wide spectrum of geosynthetics application which could be discussed in a keynote paper of its own. Therefore, only some special aspects are selected, focusing on the interaction between soils/granular materials and geosynthetics.

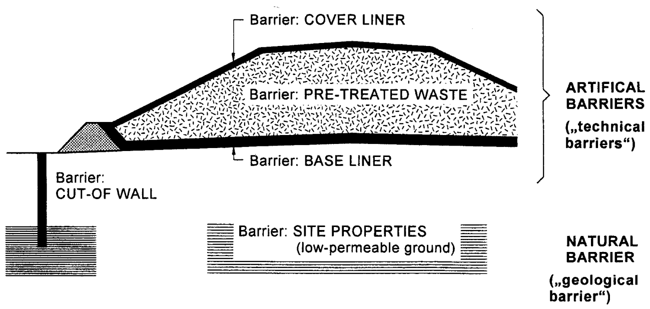
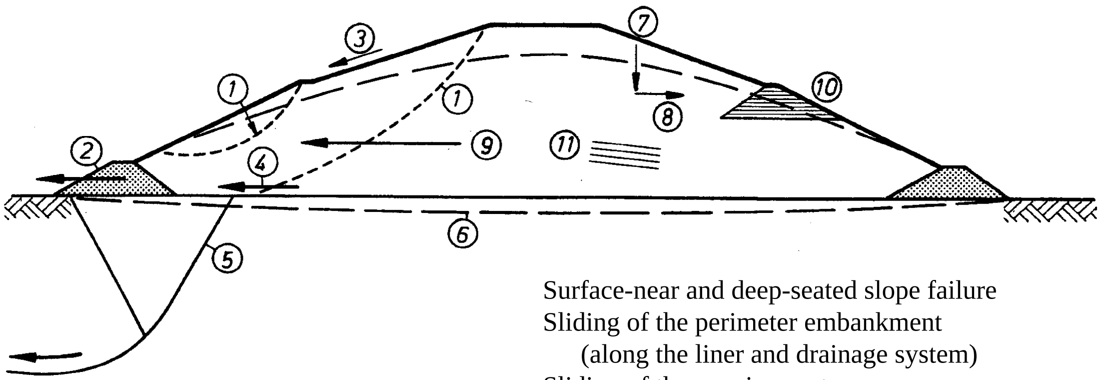


Figure 48. The multi-barrier system of an engineered waste deposit.

Geosynthetics used in landfill engineering are geotextiles (preliminarily non-wovens), geomembranes, geonets, geogrids, geosynthetic clay liners, geocomposites, geocells, and geopipes or plastic pipes



Surface-near and deep-seated slope failure
Sliding of the perimeter embankment
(along the liner and drainage system)
Sliding of the capping system
Spreading in the toe zone
Base failure
Subsoil settlement
Settlement of the waste material
Lateral displacements
Sliding of the waste
Failure of perimeter embankments/berms
Sliding along the working joints of the landfill

Figure 49. Stability and deformation problems which have to be considered for the design, construction, operation, aftercare, and monitoring of a waste deposit.

Most regional and national regulations require geosynthetics for liner and drainage systems of landfills. Geomembranes are an essential component of composite liner systems but are – at least in Europe - less used as single barrier. In the case of not pre-treated household and hazardous waste the minimum thickness should be d = 2mm to minimise contaminant migration by diffusion.

Installing geosynthetics exclusively makes a very thin structure of liner and drainage systems possible. But it has to be considered that the stress-strain behaviour of geosynthetics diverges greatly. This is especially important along slopes. Furthermore, the durability of such mono-systems is somewhat uncertain.

Geosynthetic blankets as drainage systems at the bottom of high/deep waste deposits represent an increased risk with regard to long-term behaviour (clogging). Consequently, such designs are excluded in many regulations.

Experience has shown that the efficiency of geosynthetic barriers is greatly influenced by the installation process. Comprehensive controlling over the years has revealed the following sequence and frequency of defects: About 20 to 30 % of all failures occur during installation of the geomembrane, whereby improper seams and welding defects are predominant. A clear majority of all failures is caused when covering the geomembrane with granular drainage material. A proper design, therefore, has to consider the interaction of maximum grain size of the drainage blanket, the puncture resistance of the geomembrane, and the required protective geotextile. Finally, designers should give up their unrealistic expectations regarding the number of geomembrane holes which can be expected, even with good construction quality control and construction quality assurance.

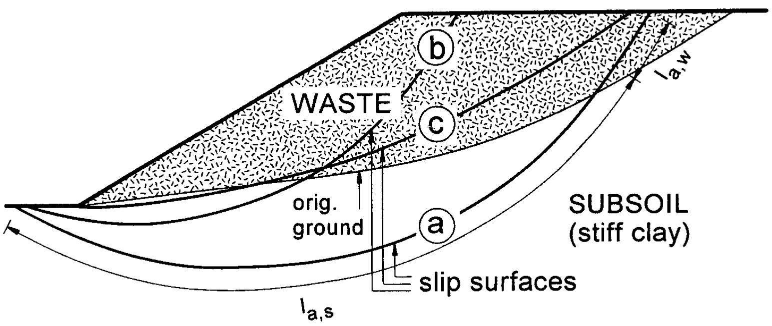


Figure 50. Slope failure investigation of a slope fill with relevant slip surfaces running through the waste deposit (municipal waste) and the subsoil (stiff clay). Slip surfaces a, b, and c assumed as examples.

l_i = length of slip surface in the specific medium (subsoil or waste), e.g.

$l_{a,s}$ = length of slip surface within the soil

$l_{a,w}$ = length of slip surface within the waste

Long-term quality assurance could be significantly increased by performing a continuous quality control (CQC) after installing and covering a geomembrane. CQC involves a leak detection and localisation system (e.g. by placing electrodes in a special pattern) which preferably should be designed in such a way that monitoring can be continued after the closure of the landfill. Though electrical leak detection techniques are very useful, at times they may give a false indication of holes and on some occasions they may not detect actual holes. Therefore, conventional monitoring systems should be installed additionally (e.g. leachate collection/removal control; groundwater monitoring standpipes/wells with chemical analyses from outside the landfill).

6.2 *Slope stability*

The stability of lined slopes of landfills is strongly influenced by the interface shear strength between geosynthetics (geomembrane – geotextile, geosynthetic clay liners etc.). Experience has shown that numerous side slope failures or spreading failures have occurred along such low-friction planes.

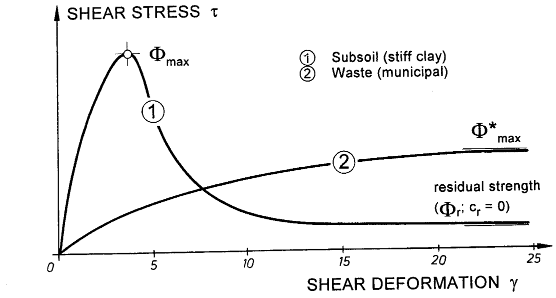
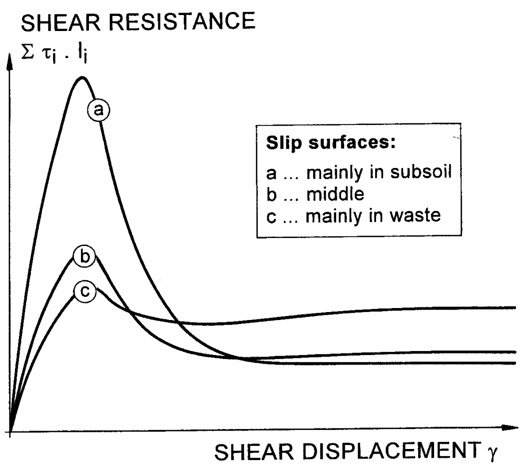


Figure 51. Shear strength - strain diagrams for municipal waste and stiff clayey soil. Schematic: completely different, strain-dependent mobilisation of the shear resistance.

Interface friction can best be determined by direct shear tests. For instance, for measuring soil-to-geomembrane friction, the geomembrane is used in one half of a direct shear box with the opposing soil surface in the other half. The size of the shear box must be adapted to the maximum grain size of the soil. The latter should not exceed about 1/50 of the side length of the shear box and 1/10 of the box height. Otherwise, friction values which are too high may be measured. A series of at least three separate tests at different normal stresses is recommended involving the determination of peak and residual strengths, which are sometimes similar and sometimes very different.

Figure 52. Shear resistance along the slip surfaces a, b, and c in Figure 50, based on the τ-γ - diagrams of Figure 51.



The peak friction angles of soil-to-geomembrane (δ) are always less than soil-to-soil (ϕ), with the smoother, harder geomembranes being the lowest (e.g. HDPE). Depending on the soil type, the values usually scatter between $\tan\delta/\tan\phi$ = 0.5 to 0.65 for smooth HDPE and between $\tan\delta/\tan\phi$ = 0.8 to 0.9 for PVC/smooth to rough. Soil-to-geotextile friction angles are significantly higher: $\tan\delta/\tan\phi$ =0.75 to 1.0 depending on soil type and manufacturing type of the geotextile. Minimum friction angles exist along geomembrane–to-geotextile interfaces. They vary between δ = 5° to 13°

for HDPE smooth and non-wovens or wovens, whereby the manufacturing type of the geotextile plays an important role. Interface friction (and apparent cohesion) can be increased to a peak value of about $\delta = 20°$ to $30°$ by using textured geomembranes, or geomembranes with spikes etc. However, the residual shear strength at large strains remains still low: usually about $\delta_r = 10°$ to $15°$, sometimes up to $\delta_r = 20°$.

Stability analyses become rather complicated if slide surfaces run through the waste body as well as through the subsoil (Fig. 50). In such cases the compatibility of waste and subsoil deformations has to be taken into account, because the mobilisation of shear resistance of waste and subsoil occurs in a quite different way (Fig. 51): Municipal waste requires large shear deformations until its maximum shear strength or friction angle (ϕ^*_{max}) is activated.

Unlike stiff soils, municipal waste does not exhibit a clear fracture. Therefore, the friction angle is a fictitious value, symbolised by ϕ^*. On the other hand, clayey subsoil gains its peak value (ϕ_{max}) already at relatively small strains, and additional shear displacements in the post failure state may cause a significant decrease of the shear resistance towards a very low residual strength (ϕ_r).

A similar discrepancy refers to the cohesion: Large shear deformations in the soil reduce the cohesion from c to finally $c_r = 0$, whereas they still mobilise an increasing fictitious cohesion, c^*, in the municipal waste. Consequently, slope stability analyses of waste deposits have to consider the compatibility of waste and subsoil deformations, especially in the case of slope fills where the slide surfaces may run through the deposit and the subsoil or (partly) along the liners.

Deformation compatibility is especially critical for stiff clays and silts which are overconsolidated and/or tectonically disturbed. Large shear displacements then cause not only a decrease of ϕ towards $\phi_r << \phi_{max}$ but also a reduction of cohesion c towards $c_r = 0$ in the soil – whereas an additional cohesion $c*$ is mobilised in the waste.

Consequently, the following shear strength criteria should be taken into account for landfill slope analyses:

Municipal waste $\quad \phi_r = \phi^*_{max}; \quad c_r = c^*_{max}$ (46)

Subsoil, clay liners $\quad \phi_r = \phi_{min}; \quad c_r = 0$ (47)

The sub-index r symbolises the final shear parameters for large shear displacements.

The different shear stress-strain behaviour of municipal waste and subsoil (or clay liners) and geosynthetics influences not only the safety factor but also the shape and location of the most critical slide surface. Figure 52 shows the shear resistance $\sum\tau_i\, l_i$ along the slide surfaces a, b, c of Figure 50, and it illustrates a dominating influence of the subsoil's parameters which is usual in the case of a very low residual shear strength of cohesive ground. The interaction of all components, hence the stability analysis, becomes still more complicated if the failure surfaces run partly along the geosynthetics or cut through them.

The consequences for slope stability analyses, risk assessment and design are:

- Low-permeable ground or subsoil with a high percentage of active clay minerals (esp. montmorillonite) is advantageous for waste disposal sites. But on the other hand, it exhibits in many cases a low residual shear strength and the tendency to progressive failure. Therefore, a detailed investigation of the shear parameters is essential, especially the determination of ϕ_r.
- The shear properties of municipal waste cannot be described by material "constants" or standardised parameters, and they are therefore not well suited for being included in regulations. Stability analyses should be based at first on a deformation assessment in connection with assumed allowable deformations. The design values for calculation should then be chosen accordingly:

  Waste $\quad \phi^*_{calc} < \phi^*_{max}; \quad c^*_{calc}$ (48)

  Subsoil, clay liners $\quad \phi_r \leq \phi_{calc} < \phi; \quad c_{calc}$ (49)

  where ϕ is the peak value of soil or interface friction angle.

Landfills on slopes with a low residual shear strength require comprehensive monitoring of slope deformations before and during construction and operation of the waste disposal facility, and during the entire aftercare period.

6.3 *Bottom liners of landfills*

The bottom liner usually represents the most important technical component of the multi-barrier system. Therefore, it should consist at least of two different sealing materials to achieve a high, durable, and multi-efficient barrier effect against the transport of contaminant migration. This is achieved by composite liner systems. So-called mono systems, consisting only of one material (e.g. geosynthetics or recycling products) should be limited to waste deposits with a low risk potential.

Composite liner systems may consist of clayey soils ("mineral liners") and/or geosynthetics, asphalt, recycling material, chemically improved soil, metals, etc. Most regulations for municipal solid waste and for hazardous waste prescribe composite liner systems based on clay liners and geosynthetics, also incorporating a proper drainage (leachate collection and removal) system. But the possibility of alternatives should be kept open to encourage further developments.

To evaluate novel systems and to compare their barrier effect to conventional liner/drainage systems, a number of criteria are required:

- Permeability (by convection, diffusion, etc.),
- Resistance (mechanical, physical, chemical, biological),
- Long-term behaviour, durability,
- Cracking and self-healing properties,
- Stress-strain behaviour,
- Friction, adhesion,
- Compatibility of the single elements,
- Overall structure of the entire barrier-drainage system,
- Workability during construction and duration of construction,
- Sensitivity towards weather during the construction period,
- Quality control and assurance,
- Reparability in case of failure,
- Thickness of the structure regarding a loss of deposit volume,
- Environmental impact during the production of the barrier materials,
- Availability of the raw materials for the liner and drainage system, and their future resources,
- Public acceptance.

Innovative systems should exhibit at least the same efficiency as conventional systems, hence, a technical equivalency must be proved. To evaluate this, risk analyses are necessary, also including the properties of the waste and the subsoil, whereby the durability of the components plays an essential role.

Table 3 contains some assumptions referring to clayey and geosynthetic liners. The values actually depend very much on the usage, the structure, the materials, and the installation quality of the liner and drainage system. For instance, geomembranes placed between clay liners, will probably still be intact even after 100 to 200 years. Mono-systems consisting only of geosynthetics make a thinner structure possible but, on the other hand, exhibit a shorter life-time than composite systems.

An optimum resistance to pollutant migration is provided if the mineral liner is firmly covered by a geomembrane (without a geotextile or geosynthetic clay liner between). This reduces not only convective migration in the case of a leak in the geomembrane, but also diffusion through the entire system. The thickness of the mineral liner varies relatively widely in the individual national regulations, whereas geomembranes are rather standardised: "HDPE being at least 1.5 mm thick."

However, in the case of soft clays, HDPE liners may not be suitable due to their relatively stiff behaviour. Soft clay with an optimum mineralogical composition has a high contaminant adsorption capacity and a low permeability. So why not use such a subsoil as natural barrier and accept large (differential) settlements by designing a project which can meet such deformations. This involves, for instance, the use of LDPE or even VLDPE liners. In the end, the barrier effect of the entire multi-barrier system (according to Fig. 48), including the natural (geological) barrier, is crucial and not the barrier effect of single components. According to this design philosophy, municipal landfills were located on (deep reaching) soft clays with expected settlement up to 3 m (Brandl, 1994 – unpublished report).

Table 3. Assumption for a risk analysis evaluating the base liner of a waste deposit.

Phase	Period (years)	Efficiency of the barrier				
		Subsoil/subgrade		Liner and drainage system		
		geological barrier	technical barrier	clay liner, mineral liner	geosyn-thetics	drainage systems
Operation (filling of waste)	0 - 25	++	++	++	++	++
Operation and/or aftercare	25 - 50	++	++	++	++	++, + 1)
Aftercare	50 - 100	++	++	++	++, + 1)	+
Final state 2)	100 -	++	++	++, (+) 1)	+, 0 1)	(+), 0

1) Depending on mechanical, chemo-physical and biological impact, construction, quality of installation and maintenance (e.g. flushability of the drain pipes etc.)
2) No further monitoring, gas and leachate collection

++ intact, in full working order
\+ intact, but probably impaired
0 no longer in working order

Composite liner and drainage systems with a primary and secondary leachate collection and removal system have proved suitable in the case of waste with a high risk potential and subsoil with a low natural barrier quality.

Figure 53 shows a double composite liner design, according to US EPA 1993. It incorporates also geosynthectic clay liners (GCL) which are less used for bottom liners in Europe. Furthermore, GCLs ought not to be placed between a geomembrane and a mineral liner. Europeans are mainly concerned about the following disadvantages of GCLs for bottom liners of non-treated municipal or hazardous waste deposits: sensitive to damage (in spite of local self-healing), sensitive to ion exchange, thin layer facilitates diffusion permeation etc. However, for landfill covers GCLs have certain advantages over conventional clay lines: thin structure (hence more waste volume available), flexible, quickly installed, easy to remove, cost-effective.

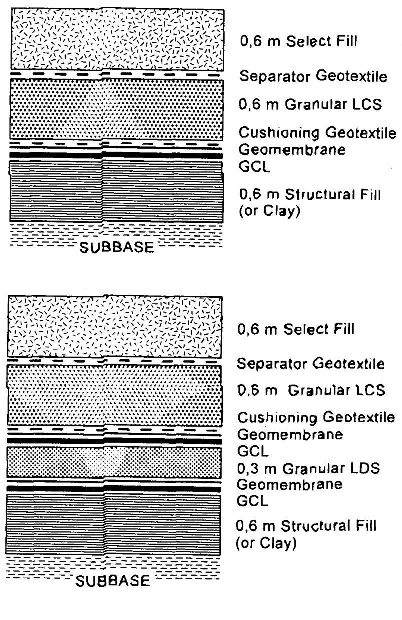


Figure 53. Double composite liner system according to US EPA 1993. GCL = Geosynthetic clay liner.

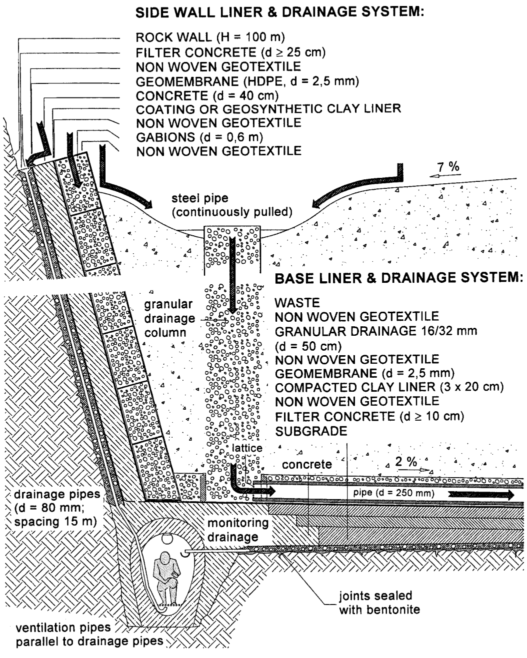


Figure 54. Bottom and side wall liners, drainage, and leak detection system for a 100 m high waste deposit in an abandoned quarry.

As an analogy to fibre concrete, synthetic fibres have been mixed with soil to reinforce clay liners and to minimise crack formation in the case of differential settlements. But, so far, the results are hardly promising.

An example of approved alternatives to common regulations is given in Figure 54. Pre-treated waste from flue gas desulphurisation replaces compacted clay liners at the bottom and along the 100m high side wall of a household and industrial waste deposit in an abandoned quarry. The rock consists of lime stone and dolomite with surface-near fissures (tension cracks and old discontinuities). Austrian Standards require a liner system which consists of at least two different materials. Therefore, a compacted clay liner is also shown in Figure 54. But a waste deposit in a 100 m high quarry with fairly intact rock certainly could not be considered a „standard" case. Consequently, the compacted clay liner was replaced by a recycling product (REA) in a sophisticated design. This alternative proved more economical and easier to construct than the first design with composite liners based on common standards for waste deposits.

Another alternative for steep sidewall bottom liners are thick composites of geosynthetics and sealing masses (as used for slurry trench walls). These systems consist of two sheets of geotextiles joined together at discrete points which creates a form that can be filled with a site-specific sealing mass to form a barrier mattress. Such "mattresses" can be adapted (fitted) easily to any surface condition, independently of its inclination or roughness. Their thickness (typically between 0.2 to 1.0m) and geometry are controlled by internal spacer threads woven into both sheets of geotextile. The sealing mix is chosen according to the risk potential of the encapsulated waste. The space between the geotextiles is in-situ filled by pumping. The modular elements of the double-wall fabric may be sewn directly on the construction site or can be connected by Velcro fasteners. This technique was developed in Austria at the beginning of the Eighties and has since proved successful for sloped bottom liners of landfills.

Incineration of municipal waste leads to slags which consist to a high degree of inorganic oxydes. The reactions between those residues and water are exothermic, hence coupled with a temperature increase. In situ measurements have disclosed that temperatures up to 85°C may develop in slag deposits, independently of the fill speed and the original temperature of the fill material. But there are clear correlations between reaction temperature and volume of deposit, water and salt content, percentage of metals, age and intermediate disposal of the slag. These aspects have to be considered when designing a bottom liner for such residues.

Commonly, geomembranes and geopipes are designed for a permanent temperature of 40°C at the maximum. Long lasting higher temperatures have a negative effect on their function – the same is true for clay liners. Consequently, landfills of slag have to be operated in a special way:

- Temporary intermediate storage of the slag to anticipate alteration.
- Reduction of salt content by washing out.
- Separation of metals (which prevents oxidation under alkaline conditions).
- Total fill height of the deposit less than 10 m.

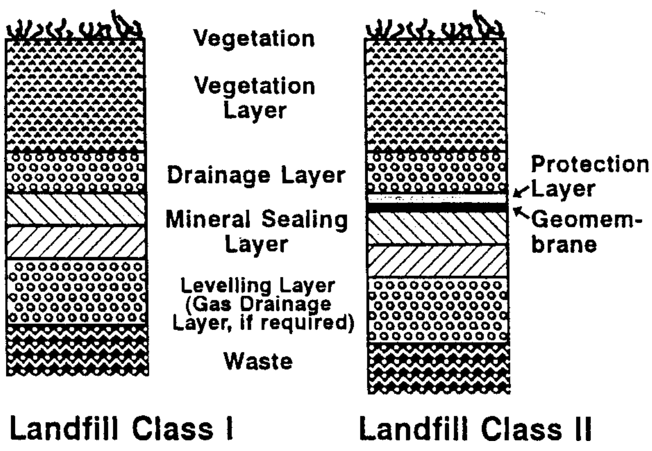


Figure 55. Capping liner and drainage system according to German regulation "TA Siedlungsabfall".

Class 1 = excavation material and debris,

Class 2 = household waste.

6.4 *Landfill covers*

Most national regulations prescribe a rather "non-permeable" capping of the waste deposit after closure. Figure 55 illustrates typical designs which reflect widely used standards. A placing of such liners immediately after finishing waste filling increases mostly the risk of long-term environmental impact because the contamination life-span increases with decreasing infiltration into the landfill. Furthermore, greater settlements of not pre-treated municipal waste inevitably cause local leaks. Both factors worsen with increasing height of the landfill and with decreasing degree of waste pre-treatment. From this point of view, a temporary semi-permeable cover of the landfill is of advantage. Geosynthetic clay liners have also proved suitable as an alternative to multi-layered liner systems because they are rather flexible and easy to remove.

The minimum barrier effect required for gas collection depends on the waste properties. Gas production due to an aerobic bio-chemical degradation of organic substances in a non-treated household waste may exhibit a maximum about 20 years after closure. Therefore, a design life of an engineered waste deposit facility of 30 years after closure is absolutely insufficient – at least for non-treated municipal waste.

During the initial period, the bottom liner and drainage system is still intact, so that an infiltration into the waste and a relatively high amount of leachate do not increase the risk of environmental impact.

Degradation of municipal solid waste can be accelerated by postponing the final cover (until waste is decomposed) and leachate recycling or re-circulation. This forces the landfill to be a massive anaerobic reactor, hence a "bioreactor landfill". As such, the waste is reduced to methane and relatively inert biomass with major losses of cellulose, the principal biodegradable constituent of municipal solid waste (Koerner, 1998). The main goals of this method are to provide for environmentally acceptable residues and to avoid long post closure care periods. According to Pacey et al (1998) a bioreactor landfill is "a sanitary landfill operated for the purpose of transforming and stabilising the readily and moderately decomposable organic waste constituents within five to ten years following closure by purposeful control to enhance microbiological processes". The addition of anaerobic sewage sludge may increase the moisture content and inhibit souring, but raises some geotechnical problems (differential settlement, slope stability).

There are several leachate recycling methods or flushing techniques: Surface spraying, surface ponding, leach fields, shallow wells and deep wells. Commonly, a combination achieves the most efficient approach to field capacity (Koerner, 1998). The potential of the hydraulic head on the liner is then well in excess of the usually regulated 300 mm. This has to be considered when designing the liner system, whereby three components are possible: Geosynthetic clay liner/geomembrane/ compacted clay liner. This three component composite liner system will often be less expensive than a traditional geomembrane / clay liner in areas where low permeability native clay or wide-grained soil suitable for being mixed with bentonite or other clay powders are not available.

Leachate recycling or flushing requires small deposit depths and low storage densities to guarantee high hydraulic conductivityof the landfill. The leachate collection system must have a high permeability, and the leachate removal system a higher capacity than in the case of conventional landfill operating. Daily covers with soil should be replaced by permeable cover material, because soil interlayers may provide a barrier to the downward flow of leachate. Stability analyses of the bioreactor landfill have to take into account the hydraulic situation which may be significantly worse than for standard landfills. Too high hydraulic heads may lead to instability and failure of the entire waste mass (Koerner, Soong 1999). However, the aftercare period would be significantly reduced. Consequently, flushing or leachate recycling is also a worthwhile alternative for the stabilisation of old municipal solid waste landfills, given that leachate drainage systems are effectively working and there is a proper lining system.

For bioreactor landfills with leachate recycling, the final cover should not be placed until the main settlements have occurred. This may last about 5 to 20 years. Instead, a temporary cover should be placed, preferably geosynthetics which allow water to enter the waste mass, but prevents the gas from escaping (Hullings and Swyka, 1999).

Finally, the leachate recycling system raises the provocative question, why not to grade the temporary landfill cover in a concave rather than convex shape. “Thus the cover would provide a pool, or reservoir, to capture rain and snow, allowing it to percolate into the waste mass, thereby augmenting the recycling of leachate” (Koerner 1999). This concept stands clearly in contradiction to common codes, standards or regulations and would be a significant shift in our thinking. But it is technically appropriate and has, for instance, proved successful in all parts of the 600,000 m^2 central municipal landfill of Vienna where it has been practised for nearly 15 years.

7 GEOSYNTHETICS FOR TUNNELS (AND OTHER UNDERGROUND STRUCTURES)

The main forms of geosynthetics in tunnelling are:

- Geomembranes as liners against ground- and seepage water.
- Nonwoven geotextiles as protective layers for geomembranes.
- Nonwoven geotextiles or geocomposites for full area, two-dimensional drainage.
- Nonwoven geotextiles as separation between primary and secondary tunnel support (primary and secondary lining).
- 5a) Geocomposites for waterproofing and drainage at tunnel entrances.
- 5b) Geogrids, geonets, geomats and geocells.
- Special geotextile drainage systems for collecting sub-surface water and surface run-off on the outside of cut and cover tunnel walls to provide relief drainage by reducing the hydrostatic pressures behind the wall.
- Geopipes for local drainage.
- Fibres for reinforcement of shotcrete.
- Fibres to increase fire resistance of concrete (secondary lining).

Figure 56 illustrates that there is a significant difference between tunnels constructed by soil/rock excavation or by cut and cover. Cut and cover tunnels are surface-near structures and they are not loaded by such high stresses as, for instance, deep-seated tunnels in squeezing rock. Tunnelling in squeezing rock or soft soil may cause large deformations and stress constraints in the entire lining systems, hence also in the geosynthetic drainage and liner. Shallow tunnels in urban areas must be constructed in such a way that excessive deformations are avoided. This requires a relatively stiff tunnel support which incorporates less constraint for the geosynthetic elements.

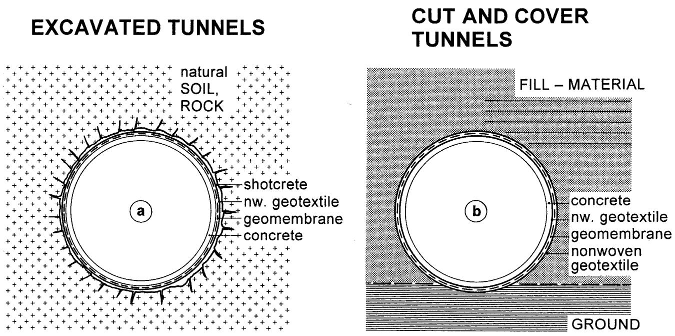


Figure 56. Application of geosynthetics in tunnelling; schematical.
a = tunnel excavated in rock or soil. b = cut and cover tunnel.

There is also a difference between open face tunnelling (tunnel excavation by drilling and blasting) and closed face tunnelling (excavating by tunnel boring machines). The drill and blast technique is more versatile and is commonly combined with “shotcrete supported tunnelling” (New Austrian Tunnelling Method). Machine tunnelling (e.g. slurry and earth pressure balance shield technology) uses segmental lining, hence a different drainage and waterproofing system. The latter may be joint covering strips or joint and fillings between the segmental elements or non-permeable concrete.

A third fundamental difference lies in the amount of water that a tunnel lining system (including waterproofing liner and drainage) has to withstand. Tunnel under the groundwater level should typically exhibit a closed all-round waterproofing liner (Fig. 57).

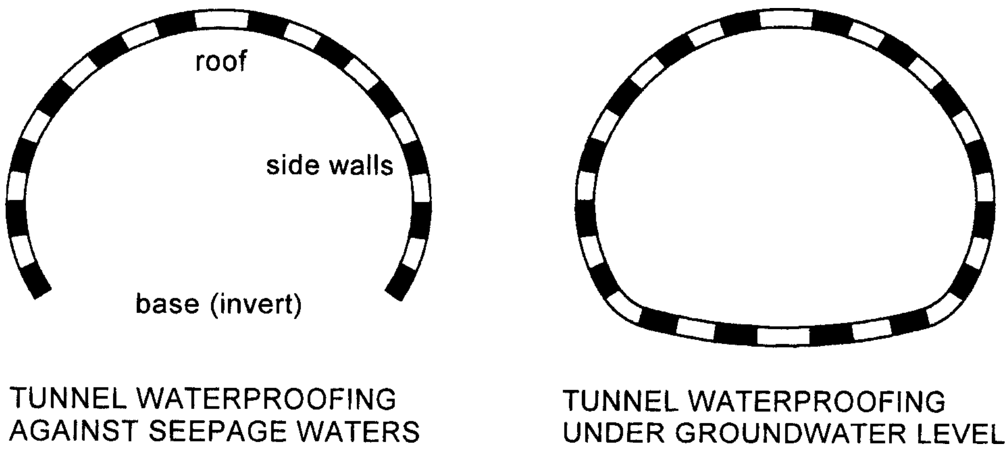


Figure 57. Waterproofing of tunnels in the case of seepage waters or of all-round water pressure.

The separation of primary and secondary tunnel lining causes a significant reduction of local stress constraints in the structure and consequently fewer cracks in the concrete. Long-term observation of tunnels constructed after the New Austrian Tunnelling Method (NATM) have proved the positive influence of nonwoven geotextiles as separating and protective full area blankets. This is illustrated in Figures 58 and 59, whereby the following structures were chosen for comparison:

a) = Secondary lining directly placed onto the primary lining (no geosynthetic interlayer).
b) = Nonwoven geotextile (700g/m^2) as separation between primary and secondary lining.
c) = Nonwoven geotextile (500g/m^2) and PVC-geomembrane (d = 1.5 mm) between primary and secondary lining.

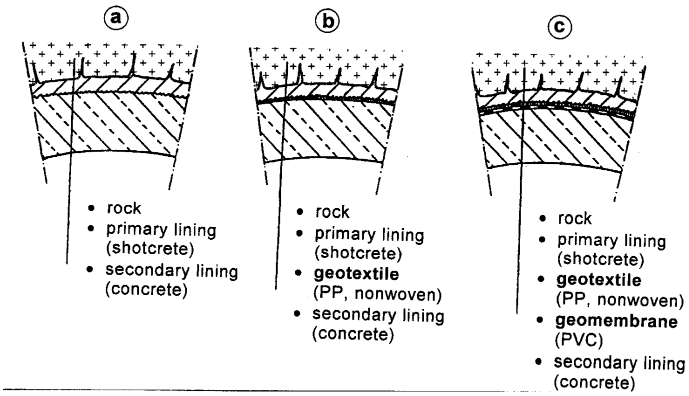


Figure 58. Three different types of tunnel lining in deep-seated squeezing rock. Tunnelling with NATM; concrete quality = C25/30 according to EN206 (after Mannsbarth, 1992).

Primary and secondary linings were identical in all cases: shotcrete (reinforced with wire mesh) and 0.3 to 0.4 m concrete (C30). The nonwoven geotextiles were PP needle-punched continuous filaments. Figure 59 illustrates that the number of cracks decreases already clearly when only a separating geotextile is placed, and that an additional installation of a geomembrane reduces further crack frequency. Crack monitoring has been performed along half the total tunnel length, hence along 6 km.

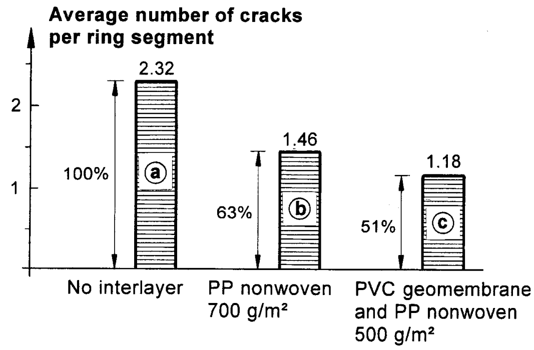


Figure 59. To Fig. 58: Frequency of cracks in the tunnel lining after an observation period of 10 years. The data refer to tunnel rounds of 12 m length along 6 km of tunnel.

The New Austrian Tunnelling Method integrates the surrounding rock- or soil formations of a tunnel into an overall ring-like support structure and optimises the support by allowing a certain deformation. The primary stress p_o in the ground is released with progressive inward displacement $\Delta r/r$, thus leading to a minimum of required ground support ($p_{r,min}$). Theoretically, the minimum load on support is achieved if the support curve intersects the ground response curve at its lowest point. That means that the support should be neither too rigid nor too flexible, and that it should be installed neither too early nor too late. But in practice, a rapid closing of the ground-ring is most of the time essential for optimised tunnelling, at least in weak rock (and soil). Maintaining as much of the rock mass' shear strength as possible is an important principle. Too large deformations loosen the ground and cause a stress increase, hence more support. The increase in radial pressure, p_r, due to loosening of the surrounding ground is limited to that point where the residual strength of the ground is reached.

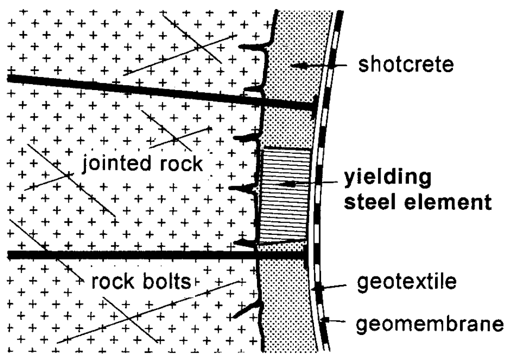


Figure 60. Shotcrete lining in squeezing rock with a yielding steel element (Lining Stress Controller = LSC) installed in a deformation gap to make controlled displacements possible.

In squeezing rock excessive deformations may cause irregular cracks of the reinforced shotcrete ring. In such cases, gaps are left in the shotcrete ring, and the connections of the steel support are not yet closed until the deformations have decreased to a negligible value. This method has provided evidence that the reinforced ground ring is the essential load bearing body and not the shotcrete lining. However, the missing transfer of axial load into the lining means that the bearing capacity of the shotcrete lining is not utilised until the gaps are closed. To overcome this problem, low-cost yielding elements integrated into the shotcrete lining were developed recently (Moritz, 1999). This innovation, called Lining Stress Controller (LSC), consists of multiple steel pipes in a concentric assembly (Figure 60). The bearing capacity of the yielding elements can be tailored to the supporting system, to the ground behaviour and to the deformation speed. The yielding elements prevent uncontrolled crack formation in the shotcrete, hence reduced stress constraints in the geosynthetics placed between primary and secondary lining.

An optimised ground support system depends on numerous factors, e.g. ground properties, overburden and possible failure mechanisms, sectional area of the tunnel, excavation method (e.g. multiple-drift method or full face excavation), speed of excavation and of support installation etc. Ground reinforcement, therefore, has to be adapted to these local conditions which requires intensive monitoring and iterative optimisation. Consequently, these aspects have a significant influence on the geosynthetic elements and on the optimal time and way of the installation.

- Rough tunnel walls (with steel arches and/or anchor heads) as well as large and long-lasting deformations or local constraints require very flexible geosynthetics and appropriate connection.
- Geotextiles and geomembranes should be installed as late as possible, hence after the main ground deformations have occurred.
- Geotextiles, geomembranes and geocomposites should be fitted very cautiously to the surface of the primary tunnel lining. Protruding parts of the shotcrete, anchor or nail heads and supporting steel ribs should be covered with shotcrete to obtain a surface as smooth as possible. Subsequent deformations of the systems should be possible without failure. A rather loose placement of the geosynthetics without permanent tension is therefore essential. A proven method is fixing a nonwoven geotextile with discs and nails onto the shotcrete and then welding the geomembrane onto the discs. This fixation is required only during a short construction period but has no permanent function. In the case of differential deformation between primary and secondary lining, the geotextile-geomembrane connection breaks immediately, and the geomembrane remains undamaged.

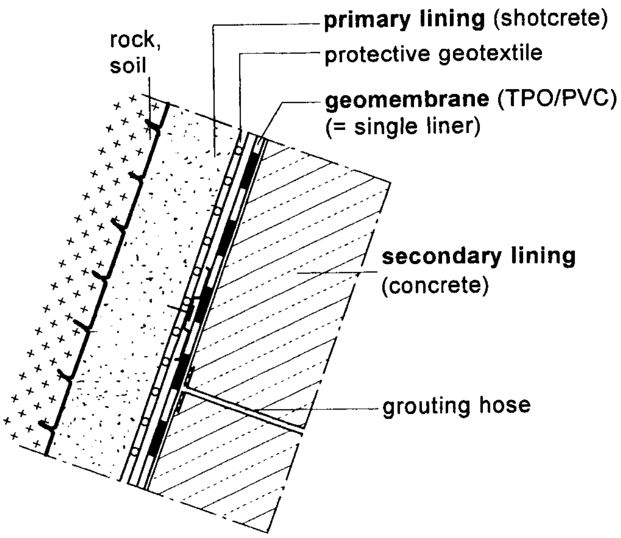


Figure 61. Single-layer geomembrane tunnel liner in the sidewall/roof of an open face excavated tunnel (adapted after Kuhnhenn, 1995).

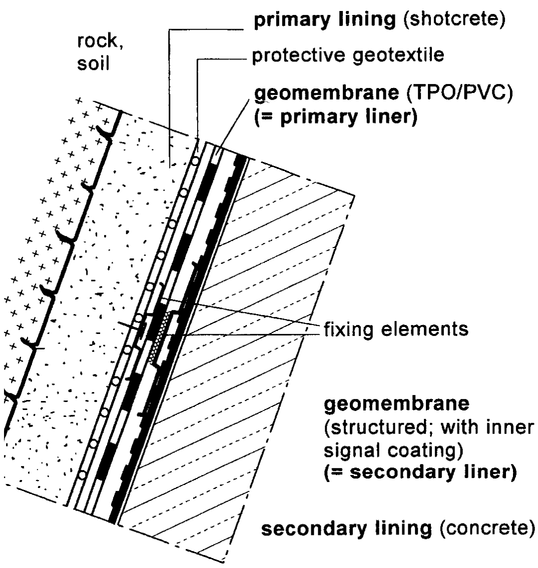


Figure 62. Double-layer geomembrane tunnel liner in the sidewall/roof of an open face excavated tunnel (adapted after Kuhnhenn, 1995).

TPO = thermoplastic olefins.

Geosynthetics in tunnelling and for other underground excavations must possess mechanical, hydraulic, chemical and biological stability. Chemical resistance refers to aggressive seepage- or ground-waters and to reaction products from shotcrete and concrete. Fresh concrete may have a pH-value up to 12, which attacks polyester; furthermore, seepage water from such alkaline zones clogs the drainage.

Thermal resistance of geosynthetics is required in the case of deep-seated tunnels in zones where high temperature seepage waters occur. Temperatures between 40° to 60° C have been measured in several tunnels. Commonly, a design life-time of 25 years is assumed for geotextiles or geotextile-related products exposed to temperatures lower than 25° C in the ground around the tunnel lining. Consequently, higher temperatures require special products and a twin-liner system that can be properly post-sealed (e.g. by grouting) if long-term monitoring disclosed leaks. Very low temperatures (sometimes 5° C to 10° C, due to glacial waters) do not represent a problem for the geosynthetic waterproofing and sealing systems.

PVC geomembranes will probably be increasingly substituted by thermoplastic olefins in the future. Accordingly, PP geomembranes have been already installed in several test sections of tunnels. They are resistant towards alkaline attack and excessive temperature changes (e.g. softening temperature about 125° C for PP, but only 65 to 70° C for soft PVC).

In the case of seepage water only, the invert of a tunnel can be constructed without waterproofing liner, but needs a proper drainage. However, tunnels running below groundwater level require an all-round waterproofing liner. Commonly, a 1.5 mm thick PVC geomembrane is sufficient for proper waterproofing of tunnels. In the case of very high water pressure, geomembranes up to 4 mm thickness are installed, and three-dimensional synthetic geonets with a large open structure for long-term drainage may be useful.

Waterproofing against high pressure groundwater with geomembranes should be constructed in such a way that repair is possible at any time. This requires an integrated injection system which covers the entire liner area (Figs. 61, 62). In the case of hydraulic heads exceeding 30m, a twin-liner system is recommended. This can be either a combination of non-permeable concrete and geomembrane or a double geomembrane solution. The latter should be preferred if concrete-aggressive seepage occurs (then already at hydraulic heads ≥ 10m).

Waterproofing systems with geomembranes should be subdivided in sections with bulkheads and should incorporate a post-grouting system. Single- and twin- layer geosynthetic liners exhibit different schemes of bulkheads in the transversal and longitudinal direction (Fig. 63). The single layer liner requires an additional joint tape sealing of longitudinal sections in the case of non-permeable concrete (presently possible up to 30 m hydraulic head).

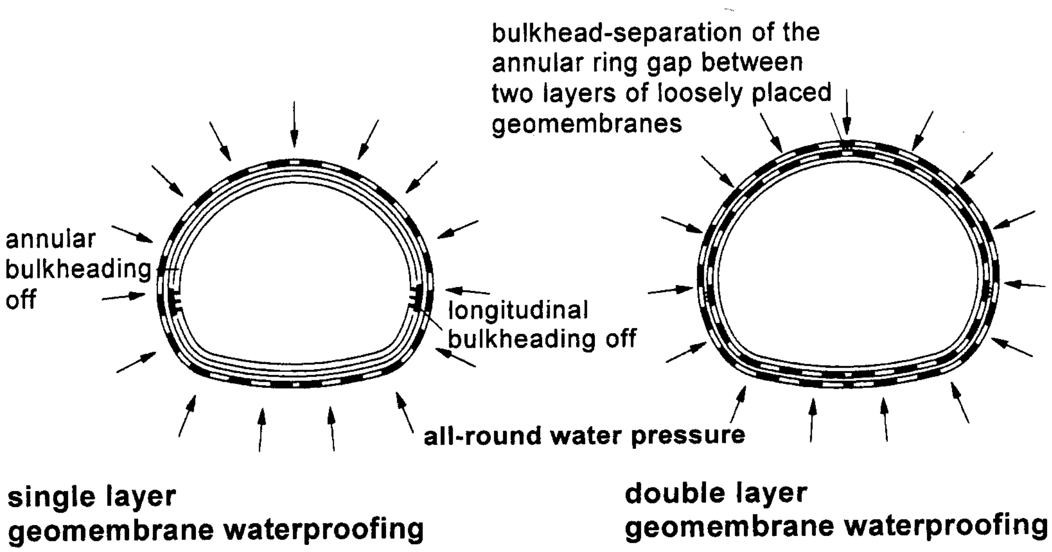


Figure 63. Bulkhead systems for single-layer and twin-layer geomembrane waterproofing of tunnels (after DS853).

Double-liner waterproofing is essential not only for high pressure and/or aggressive ground-water but also in the case of high quality usage of tunnels or other underground openings (e.g. extended use by people, storage of moisture-sensitive materials, or computer systems). Reinforced concrete allows the diffusion of water in the form of vapour and must therefore be covered with a geomembrane.

Full area liners which cover the entire tunnel circumference against waters of high pressure and/or high concrete aggressivity require special bulkheads and joint tapes in order to prevent local seepage. The latter may run along many meters, sometimes over hundreds of meters if no bulkheads have been installed. Special solutions are also required in transition zones where the waterproofing system changes. Figure 64 illustrates this for a single liner system with non-permeable concrete (without geomembrane) to permeable concrete (with geomembrane).

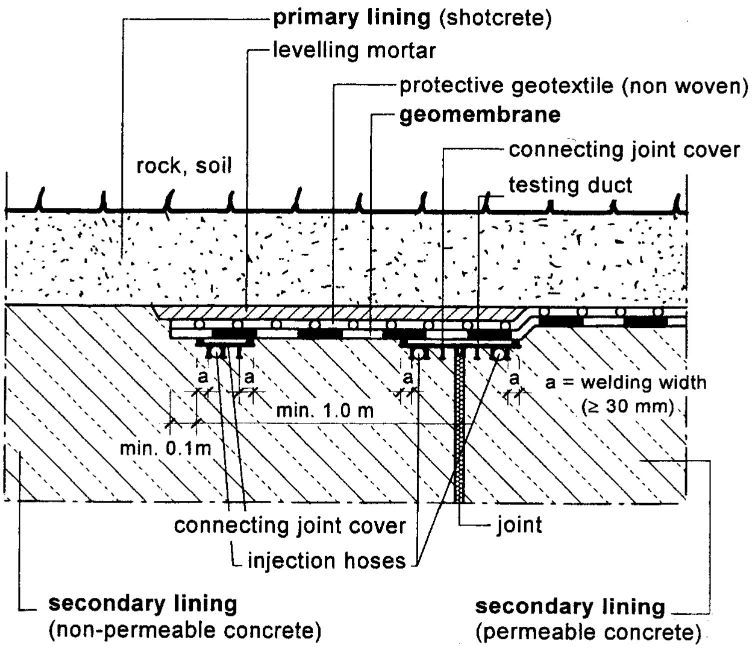


Figure 64. Transition zone for a single-liner system changing from non-permeable concrete (without geomembrane) to permeable concrete (with geomembrane).

Geosynthetics may replace conventional drainage systems (granular blanket, trench, pipes) in the base (invert) of tunnels with a small sectional area if they exhibit sufficient hydraulic capacity. For instance, HDPE drainage layers with wide dimples placed on a geotextile over a fine-grained levelling layer provide waterproofing and drainage simultaneously. Wide channels within the HDPE-drainage layer enable large volumes of water to flow transversally and longitudinally whilst the flat areas of the dimples transfer loads. This system has proved successful for a 8 km long railway tunnel (∅ = 3.5 m) 100 m below sea level, whereby the drainage system was covered by load-transferring blinding concrete and 0.3 m structural concrete.

High-speed railways raise problems for tunnels in erosive soil deposits, especially if they are situated below original ground water level. Heavy dynamic traffic loads may cause excessive pore water pressures, thus resulting in seepage flow which loosens soil particles beneath the tunnel invert. Mud slurry is then pumped out from invert holes (weep holes) with a passing train. This causes loss of tunnel support and uneven deformation of the track that requires costly and difficult maintenance (Hayashi/Shahu 2000). Such problems could be avoided by two alternative measures: First, waterproofing the tunnel instead of permanent groundwater lowering by tunnel invert drainage. This requires a kind of ring-shaped tunnel support but is the most reliable and environment-friendly solution. The second solution comprises permanent groundwater lowering and installation of capable geotextile filters in the tunnel invert.

Long experience with waterproofing of tunnels has led to the following recommendation with regard to construction quality control and quality assurance: The client should not insist on unrealistic “watertightness” but should define, in time, with the contractor a realistic allowable amount of leakage water depending on the usage of the tunnel. For instance, “completely dry” tunnels (for personnel, storage, sensitive machines) may have a daily leakage rate of 0.02 l/m^2 along 10m of tunnel or 0.01 l/m^2 along 100 m. However, traffic tunnels with an allowable capillary moisture may have ten times more.

An improvement of the new Austrian Tunnelling Method or „shotcrete supported tunnelling“ is the use of non-alkaline accelerated shotcrete. It is suitable for wet-mix as well as dry-mix shotcrete, and it provides a high quality shotcrete with a significantly lower scatter of material characteristics than standard shotcrete. The main aim of the development of high quality shotcrete was not a high strength (up to C 95/105) but a high density to improve waterproofness and chemical resistance in the case of aggressive sub-surface water. The quality increase can be taken into account when designing the covering geosynthetic liner and drainage system (e.g. less clogging risk).

Another innovation is fibre shotcrete and fibre concrete using polypropylene instead of steel or glass. The aim of using such material is to substitute classical steel reinforcement by synthetic fibres which are either straight, waved, or ruffled, placed separately or connected. The best results were obtained with thick filaments of a waved shape (l = 20 - 25) and a dosage of about 1 to 2 kg/m^3 of polypropylene fibres. The main advantages of such reinforced materials are:

- Improvement of the early shrinkage behaviour, hence favouring high stresses at an early stage without uncontrolled micro-cracks.
- Reduction of material rebound during spraying the concrete.
- Improvement behaviour in the case of fire.

Polypropylene fibres facilitate stress redistribution during the critical shrinking phase but they hardly improve the bearing capacity after the shotcrete/concrete has reached its inherent strength. The bending tensile strength is rather similar to non-reinforced shotcrete or concrete. Hence, static calculation of PP-fibre concrete or shotcrete structures is equal for reinforced and non-reinforced material.

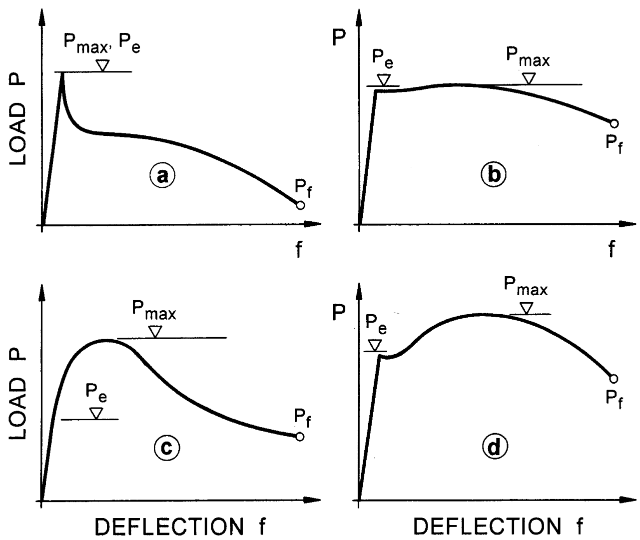


Figure 65. Examples of possible load-deflection curves for fibre reinforced concrete in tunnelling. Results from deformation controlled bending-tension tests.

During fire, the vaporising residual water in concrete may lead to such excessive inner pressures that parts of the structure crack off. Polypropylene fibres, however, crepitate at temperatures above 400°C and leave fine capillaries. Through these voids the vaporising pressure can drop, and the cracks are significantly minimised.

Current research and development focuses on a combination of PP and steel fibres to make use of the advantages of both materials. Figure 65 shows some example for possible load-bending curves of fibre reinforced concrete in deformation controlled bending tension tests. Though the bearing capacity is pretty much the same in all cases, the structural behaviour will differ widely. For instance, curve d) fulfils the required crack distribution in a better way than b) or c).

Commonly, a 1.5 mm thick PVC geomembrane is sufficient for proper waterproofing of tunnels. In the case of very high water pressure, geomembranes up to 4 mm thickness are used and three-dimensional synthetic geonets with a large open structure for long-term drainage have proved successful.

8 COMPACTION OF GEOSYNTHETIC – SOIL SYSTEMS

The interaction between geosynthetics and soil depends mainly on the interface shear strength, the stiffness ratio, geometry of inclusions, and on the compaction of the fill layers. A high compaction degree favours the compound behaviour, thus improving the bearing-deformation characteristics of the entire system. In addition, homogeneous compaction avoids local stress concentration 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 retaining structures and for road structures, but also for reinforced soil improvement beneath the foundation of buildings and for capping layers on top of piles or granular columns of piled embankments. Furthermore, the barrier effect and lifetime of multi-layered combined geosynthetic-clay liners of waste deposits increases significantly with degree and homogeneity of compaction.

Figure 66 gives a schematic overview of composite structures which require intensive and homogeneous compaction, whereby already the subgrade should be compacted properly (as far as possible). Compaction is also essential on sloped area, for instance in the case of landfills (bottom and cover liners) and canals.

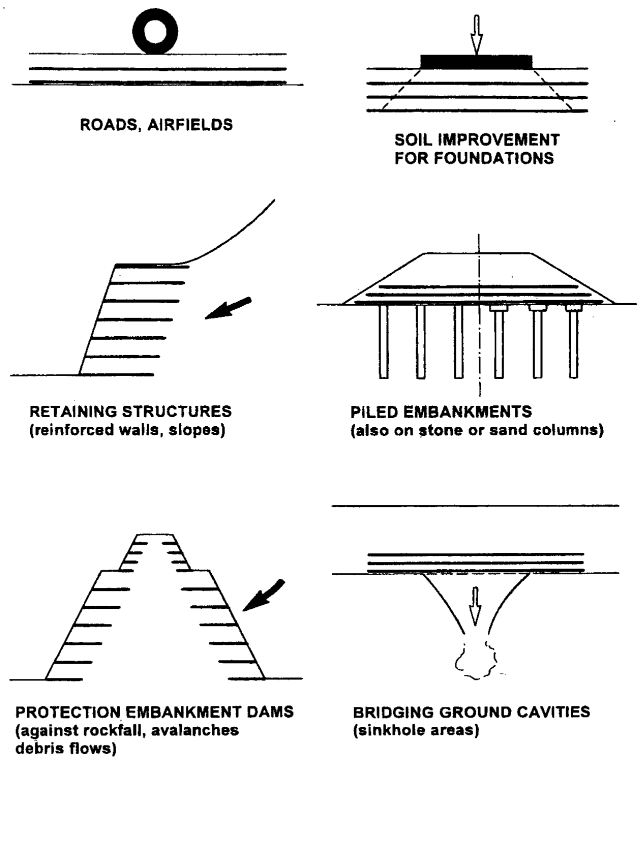


Figure 66. Composite systems of geosynthetics and soil (or other granular material) which should exhibit a high and uniform compaction degree.

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 optimisation*
  Refers to quality of compaction, to the required compaction energy and time, and to the required geotechnical parameters of the compacted material.
  Overcompaction 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/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 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.

Figure 67 shows a geosynthetic-soil retaining structure which was installed for the widening of a highway in a slope of low stability. The results of conventional compaction control (compared to Proctor density measurements) are given in Figure 68. The wide scatter is not only caused by locally different fill materials and compaction degrees but also by test-inherent uncertainties, thus making a reliable quality assessment and stability analysis somewhat difficult. Such a situation can be improved significantly by applying continuous compaction control (CCC):

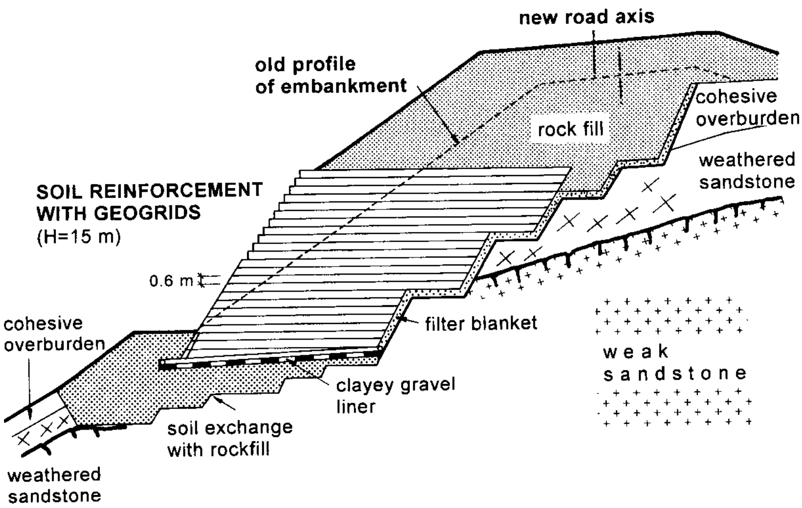


Figure 67. Geosynthetic–soil retaining structure for widening a highway (after Stiegeler, Straussberger).

The conventional methods of soil mechanics have proved suitable for decades, but are based on spot checking by more or less random selection. This involves an unavoidable residual risk, whereas the increasing demands on engineered structures and fills require as much as possible continuous compaction control and optimisation already during the compaction procedure.

The innovative technique represents a distinctive improvement because control data are already available during the compaction process and all over the roller compacted area. Vibratory roller compaction takes place by means of a vibrating drum which is excited by a rotating mass. The oscillation 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. 69): 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.

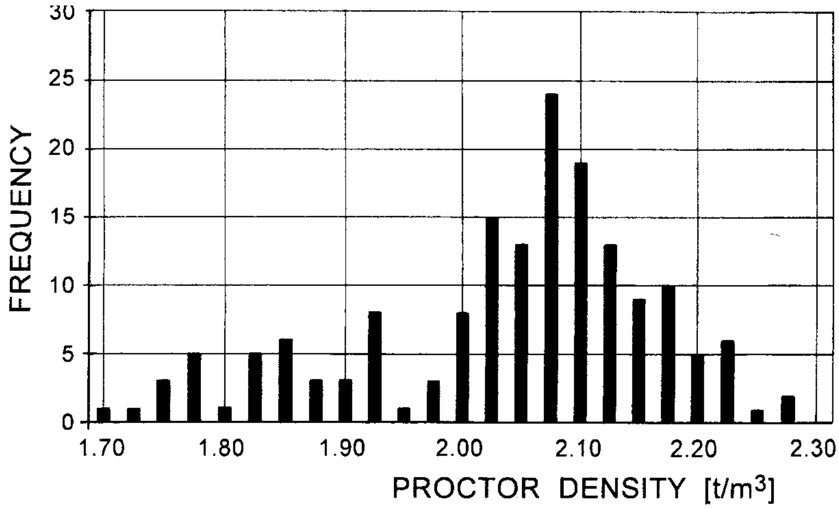


Figure 68. To Figure 67: Results of density measurements.

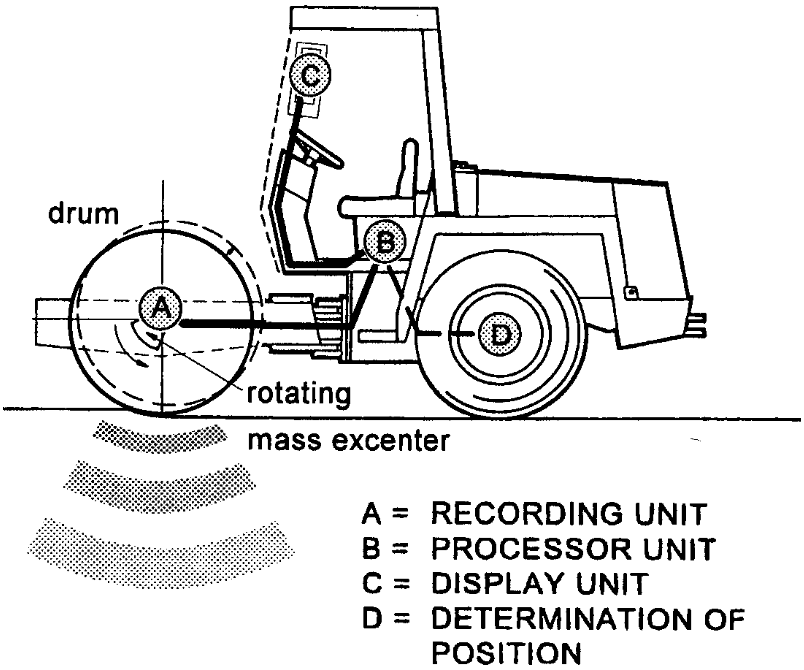


Figure 69. Principle of roller-integrated continuous compaction control (CCC).

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. 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 surface-near 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 federal roads and highways, but also for railways and clay liners of waste deposits, and for embankment dams).

Furthermore, the measuring depth of roller-integrated 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 1.5 m (and even more).

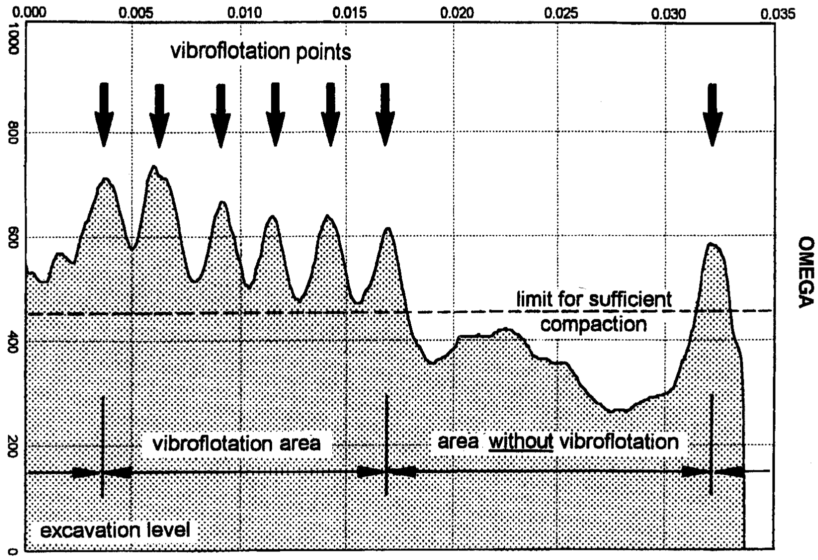


Figure 70. Results of roller-intergrated continuous compaction control (CCC) on a 1.5 m thick fill on top of a subsoil improved with vibroflotation. The locations of the stone columns are still clearly visible on the diagram.

Figure 70 illustrates this with a case history: The raft foundation of a power plant required a deep soil improvement and partial soil exchange to minimise 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 be observed up to the final layer which covered the top of the stone columns by 1.5 m (Fig. 70).

High quality compaction provides numerous advantages for roads, highways, railways, all kinds of earthworks, geotextile-reinforced retaining structures etc.:

- Increase in safety;
- Improvement of serviceability;
- Increase in life-time;
- Reduction of costs and maintenance;
- Reduction of construction time (mostly in case of roller integrated continuous compaction control);

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 its punching; furthermore, local over-compaction of the fill layers (hence stress constraints in the geosynthetic inclusions or cover) is avoided.

The increase in life-time and serviceability of structures is especially important for road/ highway pavements and for railways. Here 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 as illustrated in Figure 71 which shows various selection procedures of spot checking:

This technique 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.

Hitherto used statistical parameters do not allow the assessment of the distribution of control data within a section: The plots indicated in Figure 72 have the same mean value, max. and min. value, and standard deviation. Nevertheless, they exhibit different qualities. Figure 72(a) shows only a limited weak zone which can easily be improved, whereas the area (b) requires comprehensive measures to achieve a sufficient and homogeneous quality.

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 of roads, highways, railways, airfields, embankments, landfill liners and other 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. 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; Brandl, Adam 1997. Roller compaction technologies have been sophisticated significantly during the past two to three years and they now provide a wide range of possibilities to select the adequate roller for the respective purpose. The classic vibratory roller operating in five different conditions is supplemented by rollers with different kinds of excitation. The torsionally behaving oscillatory roller is specially suitable for asphalt compaction, cohesive mineral liners and in the vicinity of sensitive structures. In a “Vario” roller, the vibrating excitation is directed; the direction can be adjusted depending on the soil properties, so that optimised compaction can be achieved. Thus, “Vario” rollers can be employed universally for each soil type and 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.

To sum up, roller-integrated continuous compaction control (CCC) represents an improvement for high-levelled 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 geosynthetic-soil structures as much as possible.

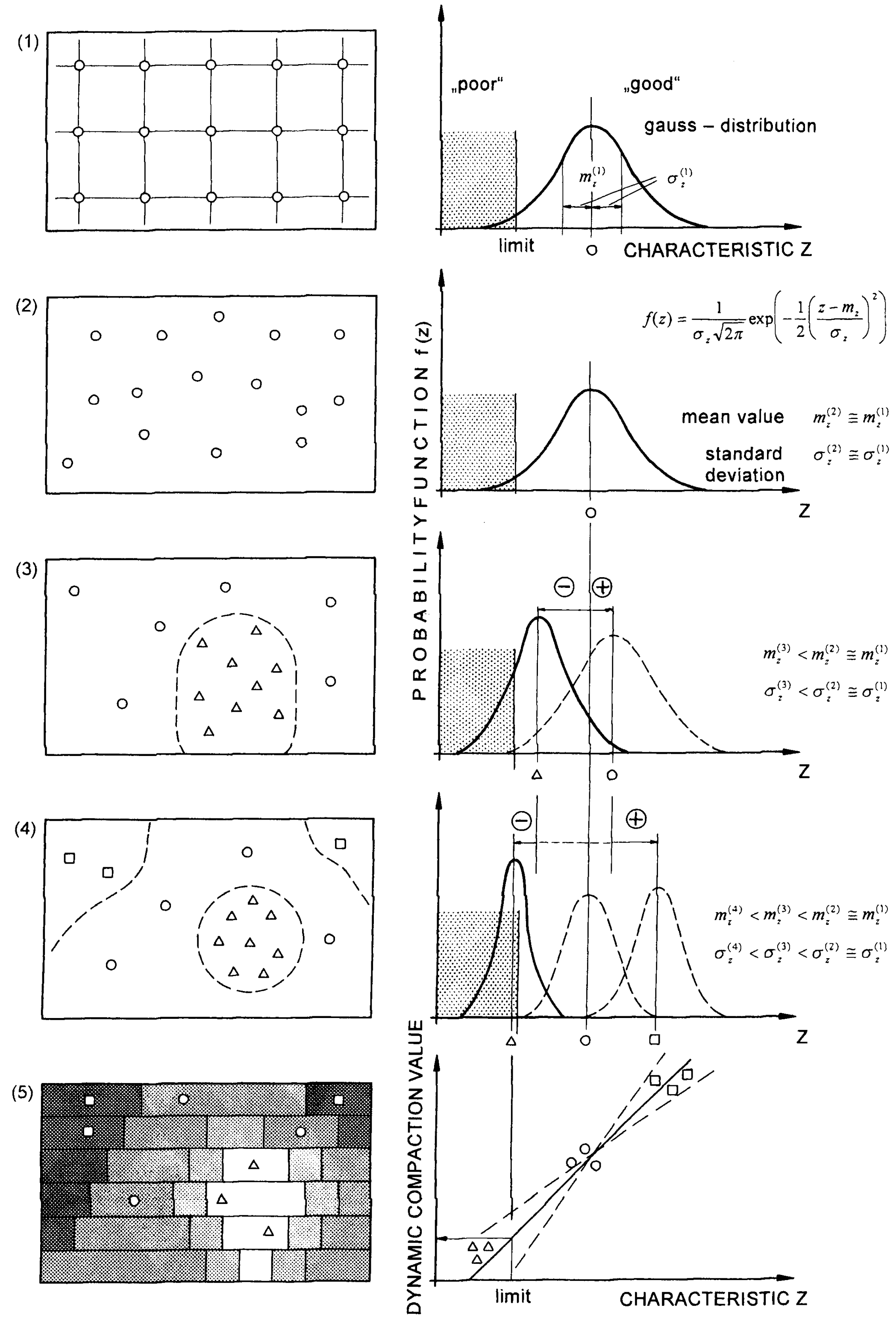


Figure 71. Statistical quality control. Different selection procedures of spot checking.

(1) = grid, (2) = random selection, (3) = subjective selection, (4) = subjective selection using auxiliary criteria, (5) = selection and correlation to continuous methods.

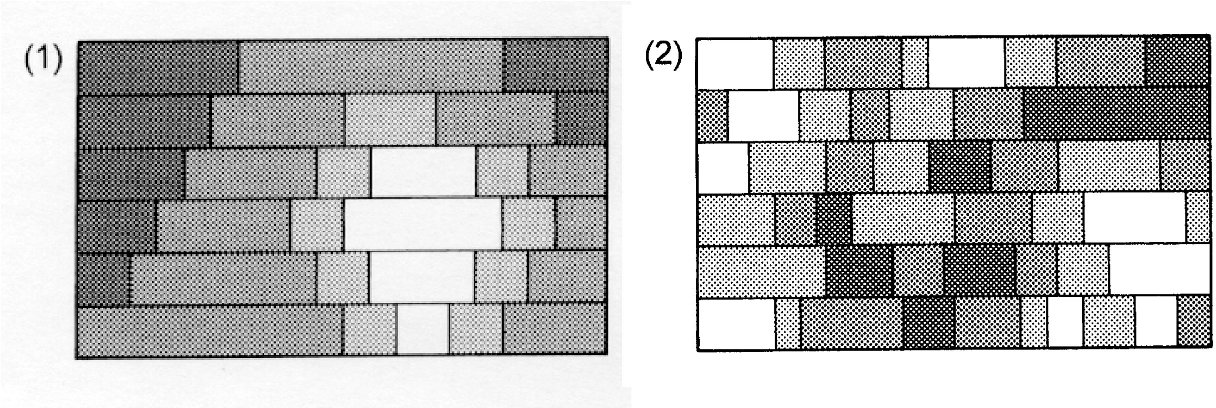


Figure 72. Control plots of compacted areas with the same conventional statistical characteristics of compaction (schematic). The strips indicate the roller lanes, controlled with CCC. Statistical evaluation can be improved with " variography" or "Kriging".

9 CONCLUSIONS

Proper utilisation and optimised application of geosynthetics is closely inter-linked with aspects of soil mechanics and ground engineering. For instance, reinforcement of paved or unpaved roads has to consider in many cases freezing-thawing behaviour of granular layers. For that purpose, a mineral criterion was developed which is also useful to assess frost susceptibly of mineral liners or geosynthetic clay liners (GCLs). Geosynthetics and soil form more or less composite bodies or even structural units, e.g. reinforced soil retaining walls or combined multi-layered landfill liners. Hence, proper compaction of such structures is also essential for high quality, whereby continuous compaction control (CCC) should be preferred to conventional spot checking with random selection. Moreover, CCC facilitates compaction optimisation already during roller operation and provides complete documentation.

For the containment of hazardous waste or untreated municipal waste with a high risk potential, multi-barrier systems are recommended, hence not mono-geosynthetic barriers as bottom liners. Commonly, a combination of mineral and geosynthetic liners provides the optimum solution with regard to long-term behaviour, whereby the mineral liner may consist of clay, stabilised wide-grained soil or granular recycling products. Contaminant diffusion through barrier walls can be minimised rather by an inverse hydraulic gradient than by geomembrane inclusions in slurry trench walls.

Geosynthetics for tunnels in squeezing rock may be exposed to great differential deformations. This can be reduced by installing yielding elements in open joints of the primary lining (shotcrete) and by placing geosynthetics in a loose way that avoids stress constraints. Tunnels exposed to high water pressure and/or aggressive ground/seepage water should exhibit a double-layer waterproofing which can be easily monitored and repaired. Synthetic fibre shotcrete (and concrete) is under development. A main advantage is its behaviour in the case of fire.

Geosynthetics for piled embankments were originally used only for roads but have meanwhile proved successful also for high-speed railways and embankments of very low height. Instead of conventional piles, stone columns are increasingly used which are jacketed with geosynthetics in case of very soft soil. Geosynthetics for settlement acceleration of soft soils comprise not only vertical drains but also jacketed stone or sand columns and capping systems with geomembranes to apply suction forces to the ground.

Typical composite elements are, finally, geosynthetics for reflective crack prevention of bituminous road pavements, geosynthetics as secondary liner within slurry cut-off walls, and synthetic fibres for reinforcing soils or other granular material.

To sum up, since their intensive introduction in civil and geotechnical engineering in the Seventies, geosynthetics have been increasingly used and adapted for numerous special applications. This versatility is a challenge to theory and practise likewise, whereby design by function, combined with the observational method, should be the basis for both. In-situ measurements and well documented case histories are even of greater importance in the field of special applications than for already „classical“ utilisation of geosynthetics.

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