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Design using geosynthetics —

Part 5:
Stabilization

Conception utilisant des géosynthétiques —

Partie 5: Stabilisation

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Foreword

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This document was prepared by Technical Committee ISO/TC 221, *Geosynthetics*.

A list of all parts in the ISO/TR 18228 series can be found on the ISO website.

Any feedback or questions on this document should be directed to the user's national standards body. A complete listing of these bodies can be found at www.iso.org/members.html.

Introduction

The ISO 18228 series provides guidance for designs using geosynthetics for soils and below ground structures in contact with natural soils, fills and asphalt. The series contains 10 parts which cover designs using geosynthetics, including guidance for characterization of the materials to be used and other factors affecting the design and performance of the systems which are particular to each part, with ISO/TR 18228-1 providing general guidance relevant to the subsequent parts of the series.

The series is generally written in a limit state format and guidelines are provided in terms of partial material factors and load factors for various applications and design lives, where appropriate.

Ultimate limit state (ULS) design is necessary for some applications, e.g. slab foundation design, working platform design etc., but usually must be proven separately. This document is a state of practice report and information is provided in terms of the application of the mechanisms and design methods. A discussion on separation, filtration and other relevant engineering issues addressed with geosynthetics are addressed in the separate parts of ISO 18228.

This document includes information relating to the stabilization function. Details regarding design methodologies adopted in a number of regions are provided.

Design using geosynthetics —

Part 5:
Stabilization

1 Scope

This document provides a summary of general guidance for the design of geosynthetics to fulfil the function of stabilization of granular layers in contact with natural soils, fills, asphalt or other materials.

The concepts of the summarised guidance are based on installed materials, the installation process and on either the strength or deformation behaviour, or both, of geosynthetics.

This document provides general considerations to support the design of unbound layers of paved and unpaved roads, working platforms and foundations utilizing the stabilization function of geosynthetics. This is typically for the serviceability limit state (SLS).

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 10318-1, *Geosynthetics — Part 1: Terms and definitions*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 10318-1 apply.

ISO and IEC maintain terminology databases for use in standardization at the following addresses:

— ISO Online browsing platform: available at https://www.iso.org/obp

— IEC Electropedia: available at https://www.electropedia.org/

4 Concepts and fundamental principles

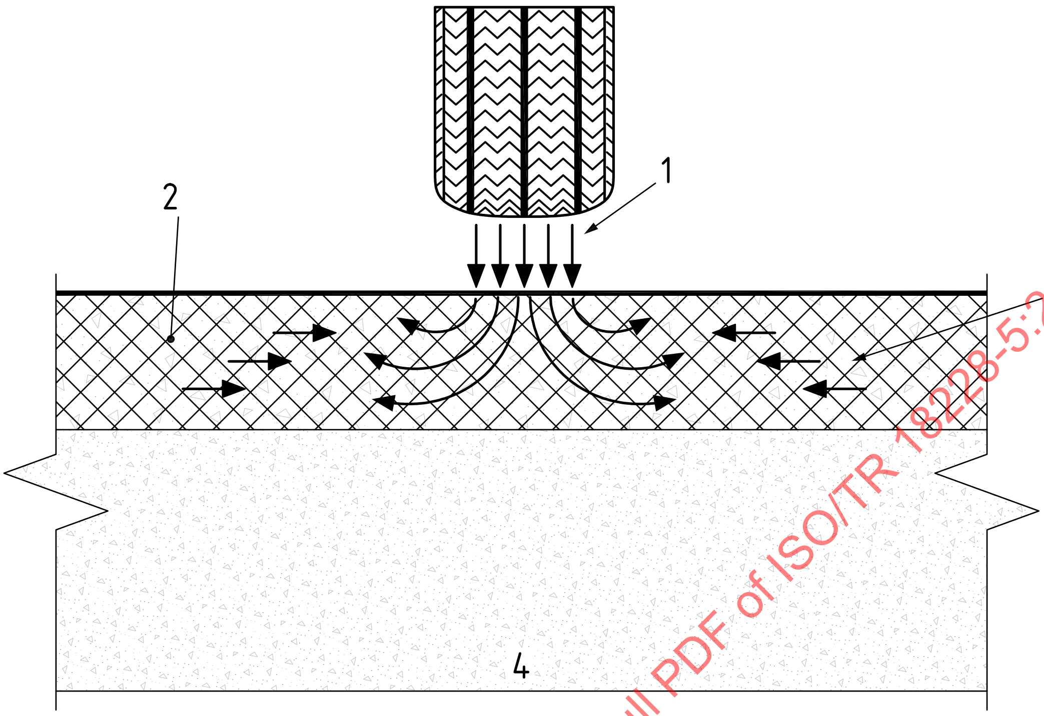
4.1 General

There are two primary mechanisms by which geosynthetics can improve the performance of a granular layer, the confinement mechanism and the tensioned membrane mechanism. The distinction between these two mechanisms and their relevant applications must be understood.

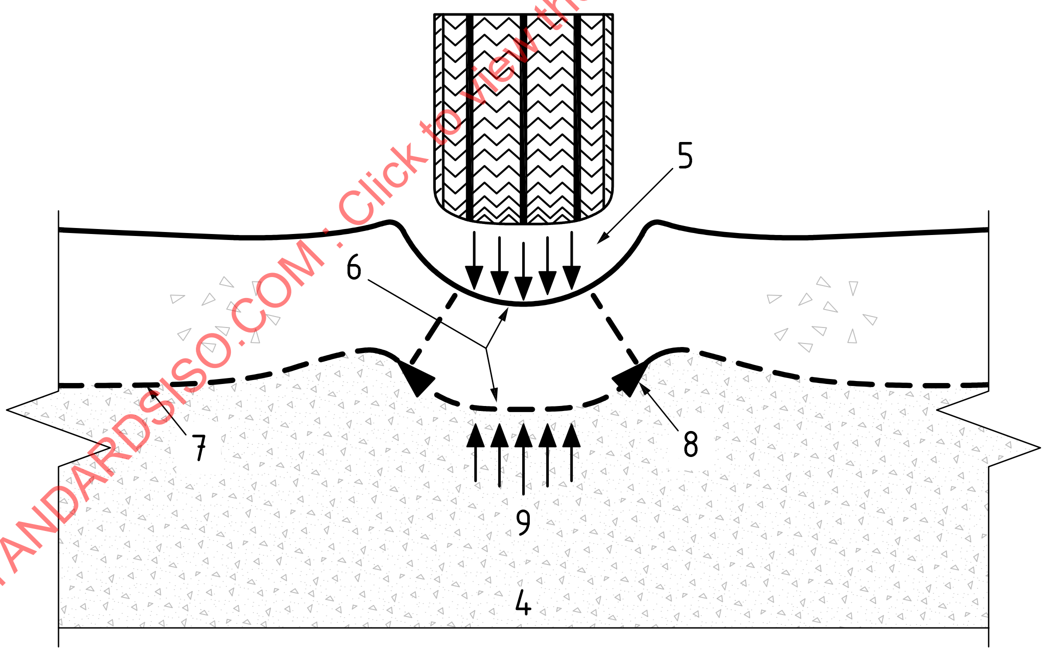
The first mechanism provides stabilization by way of particle confinement, or lateral restraint. By minimizing the movement of aggregate particles, confinement increases the shear resistance and widens the load distribution angle, improving the mechanical properties of the granular (i.e. aggregate) layer, thereby controlling deformation under load (i.e. SLS).

The second mechanism provides reinforcement by way of friction, or interlock, or deforming, or a combination of these three, out of the plane under load. In this case, the geosynthetic material is anchored on each side of the loaded area to create a tensioned membrane. In doing so, it provides support to the granular (i.e. aggregate) layer, thus decreasing deformations (i.e. SLS) and increasing bearing capacity (i.e. ULS).

Figure 1 illustrates the difference between the two mechanisms.



a) Lateral restraint



b) Tensioned membrane

Key

1 wheel load creates stresses which, unchecked, cause strains as shown
2 stabilized composite layer of aggregate and geosynthetic
3 geosynthetic can be within or beneath the layer of aggregate to provide lateral restraint, inhibit strains, and thereby support the vertical load
4 subgrade
5 wheel load
6 wheel path rut
7 geosynthetic
8 membrane tension in geosynthetic
9 vertical support component of membrane

NOTE Figure 1 shows the primary mechanisms by which geosynthetics can improve the performance of a granular layer.

Figure 1 — Mechanisms to improve the performance of a granular layer

In stabilization (i.e. confinement), the geosynthetic operates most effectively at relatively low levels of strain. Stabilization is less influential in designs where high levels of strain are anticipated. Where high levels of strain are anticipated, the tension membrane effect (reinforcement) is dominant.

There is considerable discussion in the literature about the relative magnitude of the strain within the geosynthetic in the stabilization and reinforcement mechanisms. The boundary between the operational strain envelopes of these two mechanisms and the nature of any transition between them has not been adequately defined by any research to date. This is partly because it is extremely difficult to measure the level of strain of a buried geosynthetic. This means that, a universally recognised design methodology based on this parameter is not yet available.

The design life of the project is also suggested as a key consideration and, as a result, the rate of deformation. Designing for geosynthetic stabilization results in the successful control of the rate and level of system deformation to that which is tolerable within the design life of the project.

The tensioned membrane mechanism can require large deformations to mobilize the tensile strength of the geosynthetic for it to be effective. Due to the importance of the tensile strength, the geosynthetic in a tensioned membrane mechanism is considered to be performing a reinforcement function. This is not the only mechanism through which geosynthetics perform the reinforcement function and the reader is referred to ISO/TR 18228-7 where other situations are described. However, because the tensioned membrane mechanism can be used in the application of unpaved roads, for ease of reference, a discussion of the tensioned membrane reinforcement mechanism is provided in Annex A of this document.

4.2 Benefits

Geosynthetics are utilized to facilitate construction and improve the performance of unbound aggregate layers over subgrades of varying strength. The benefits of geosynthetics have been well documented in numerous case histories. These cover the range of full-scale laboratory experiments to instrumented field studies. Many of these are highlighted in the Bibliography. In these cases, the geosynthetic and aggregate together form a stabilized layer.

Further, stabilization of the unbound aggregate leads to an enhancement in both the surface resilient modulus of unbound layers or subgrade and bearing capacity of the stabilized layer. The composite structure of aggregate fill, geosynthetic and subgrade must:

a) effectively withstand service-loading pressures;

b) control subgrade and unbound aggregate layer deformation within a range suited to the in-service requirements;

c) not progressively deteriorate over time through either aggregate deformation, breakdown or contamination.

The corresponding functions of separation and filtration can also contribute to an improvement in performance where site conditions require them to be provided.

4.3 Confinement and particle restraint

4.3.1 General

Stabilization by geosynthetics necessitates the minimization of particle movement through confinement. Minimization of particle movement is achieved by particle restraint. For geosynthetics to provide particle restraint, they must have adequate tensile stiffness and sufficient interaction with the soil.

Confinement is the dominant stabilization mechanism at low levels of strain (typically less than 1 %, but possibly up to 2 %) of the geosynthetic within or beneath the aggregate layer, depending also on the importance of the stabilized system (e.g. highway versus haul road) and the position of the geosynthetic within the stabilized system (usually at lower levels higher strains are acceptable). If strain values are expected to be above this, the designer can consider whether other mechanisms need to be included. For the purposes of this document, two types of confinement have been considered and named internal and external.

4.3.2 Internal confinement — Description of mechanism

Internal confinement is the intimate interaction of a two-dimensional geosynthetic with aggregate in or underneath a compacted granular layer thereby creating a pseudo-composite material of improved shear strength and stiffness. The interaction can occur via interlock, surface friction or both. For interlock to be effective, the geosynthetic must have apertures (e.g. a geogrid) into which granular particles can penetrate.

While vertically loaded, additional shear stress is transmitted from the aggregate to the geosynthetic which in turn results in deformation (strain) in the geosynthetic. The shear resistance caused by friction and mechanical interlock generates a lateral restraint of the aggregate particles. The stiffness provided by the geosynthetic reduces development of lateral deformation in the base aggregate over a defined height above the geosynthetic (the stabilized or "confined zone") by preventing the development of explicit displacements of the aggregate.

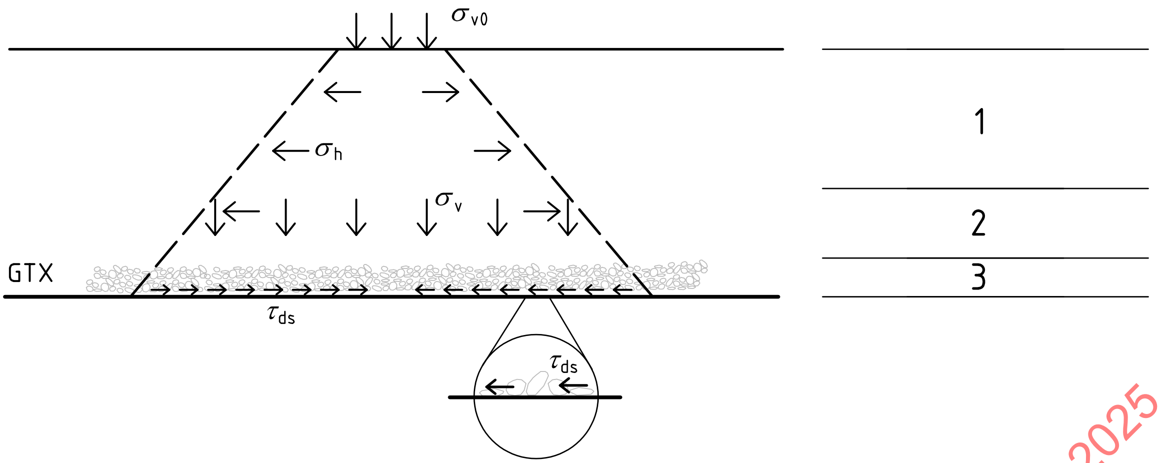
The confined zone has a limited thickness. Above it, a transition zone is developed which extends until there is no influence on the granular (i.e. aggregate) layer from the geosynthetic (i.e. unconfined zone). Figure 2 illustrates the various zones.

The efficiency of confinement and thickness of the confined and transition zones varies with different geosynthetic and soil types. The details therefore are usually defined for each type of geosynthetic and soil individually. From Figure 2 it is evident that, when a relatively high aggregate thickness is required, designing with multiple layers of geosynthetics allows a reduction in or elimination of the unconfined zone, thus affording a more effective stabilization.

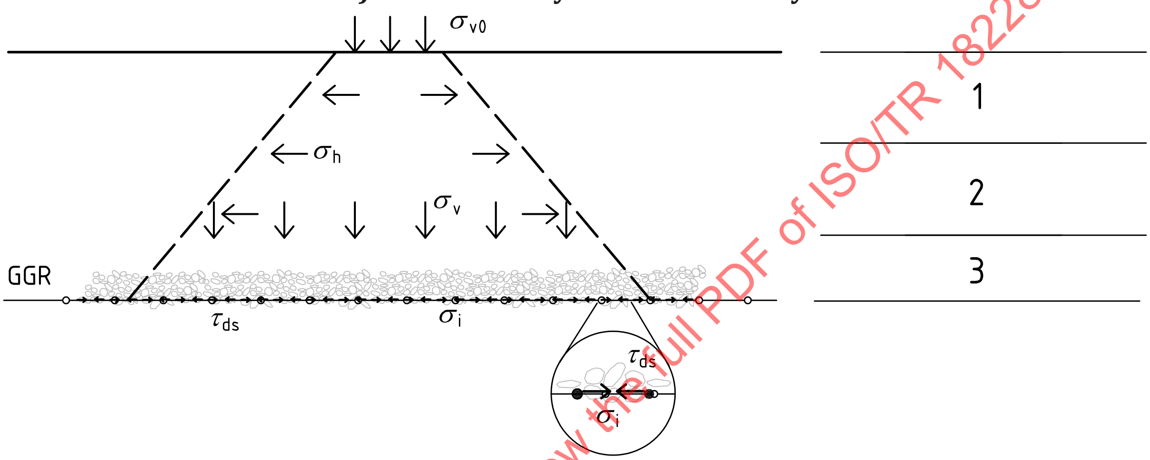
The magnitude by which the horizontal and vertical strain in the aggregate layer can be reduced depends on the stiffness of the composite layer. This is, in turn, a function of the geosynthetic tensile stiffness required for the stress equilibrium (especially at low strain levels) as well as on the efficiency of the aggregate and geosynthetic interaction.

During the application of load to the granular (i.e. aggregate) layer (e.g. trafficking or compaction), the interaction discussed above distributes stress throughout the stabilized granular (aggregate) layer and geosynthetic, thus reducing any stresses transmitted to the underlying subgrade. The limitation of movement under load provided in this way via a geosynthetic is referred to as the provision of lateral restraint.

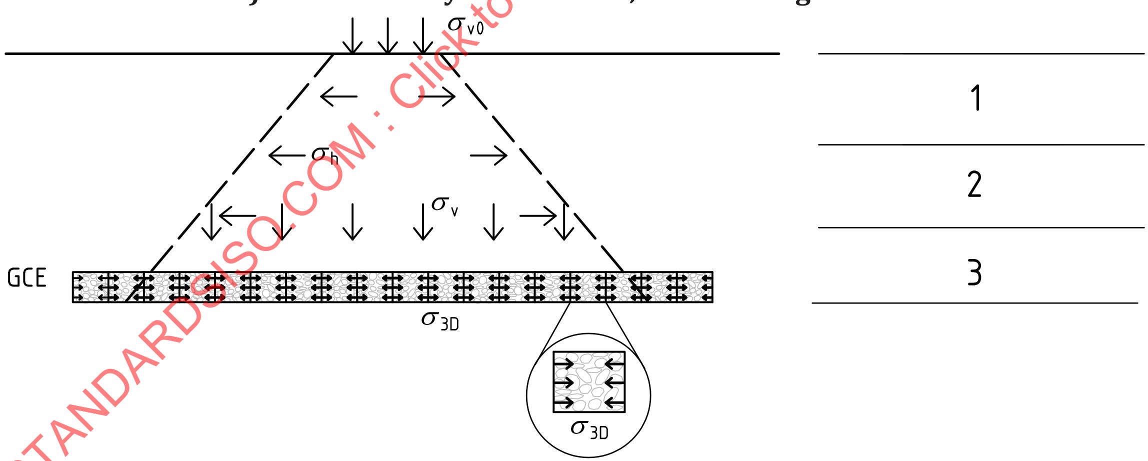
The creation of the confined zone with limited particle movement naturally limits the deformation of the granular (i.e. aggregate) layer as a whole. The resultant reduced stress transmission to the subgrade limits its deformation. It is typically the underlying subgrade that is the weakest material in the construction section and one of the principle aims in developing a confined and stabilized granular (i.e. aggregate) layer is to limit any stress and strain transmission to this weaker layer.



a) Interaction by direct shear only



b) Interaction by direct shear, interlocking or both



c) Interaction with geocell

Key

σ_{v0}	vertical stress applied at the top surface
σh	horizontal stress
σv	vertical stress at the subgrade interface
τds	direct shear stress
σi	interlocking confining stress
GTX	geotextile

GGR geogrid

GCE geocells

1 unconfined zone

2 transition zone

3 confined zone

Figure 2 — Interaction of granular (aggregate) material with geosynthetics

Reducing the horizontal strain leads to a decrease in the Poisson ratio of the soil and geosynthetic composite material compared to the Poisson ratio of the unstabilized soil. Reducing the Poisson ratio increases the horizontal stiffness which means that the geosynthetic stabilized soil layer is able to distribute the vertical stresses σ_{v0} applied at the top surface on a wider area, as shown in Figure 3.

In terms of the well-known concept of load distribution angle, for the unstabilized soil layer the vertical stress on the subgrade σ_{vu} will have a load distribution angle α_u [Figure 3a)]; while for the two dimensional and three-dimensional geosynthetic stabilized soil layer, the vertical stress on the subgrade σ_{vs} will have a much wider and more uniform distribution according to the increased load distribution angle α_s [Figures 3b) and 3c)].

On the other hand, it is evident that at equal maximum value of σ_{vu} and σ_{vs}, the load that the stabilized soil layer can support will be much higher than the load supported by the unstabilized soil layer. In other words, the improved vertical load distribution on the subgrade affords a higher bearing capacity of the stabilized system compared to that of the unstabilized system.

These concepts have been demonstrated by research and monitored stabilized versus unstabilized soil layers under different types of loads and are widely used in the presently available design methods.

4.3.3 External confinement — Description of mechanism

External confinement occurs when a volume of material is confined by a three-dimensional geosynthetic system, which can be made of a geocell or a factory or in-situ three-dimensional assembling of geosynthetic components. For ease of use, reference will be made to geocells only in this document, although the details apply equally to the factory and in-situ 3D assembling of geosynthetic components.

The geocell stabilization mechanism limits horizontal infill soil deformation via the geocell walls, thereby confining the infill soil. The limitation of horizontal deformation is based on four factors.

a) Hoop tension forces in the cell walls.

b) Resistance from the surrounding cells.

c) Friction between cell walls and infill material.

d) Connection strength of joined walls.

Under vertical load, horizontal earth pressure is restrained by the cell walls. The resulting strains in the cell walls mobilize hoop stresses within the loaded cell (Figure 4). The magnitude of the activated hoop stress depends on the geocell material, stress-strain behaviour, magnitude of load, number of load cycles, location of the applied load, type and properties of infill material, and the foundation characteristics.

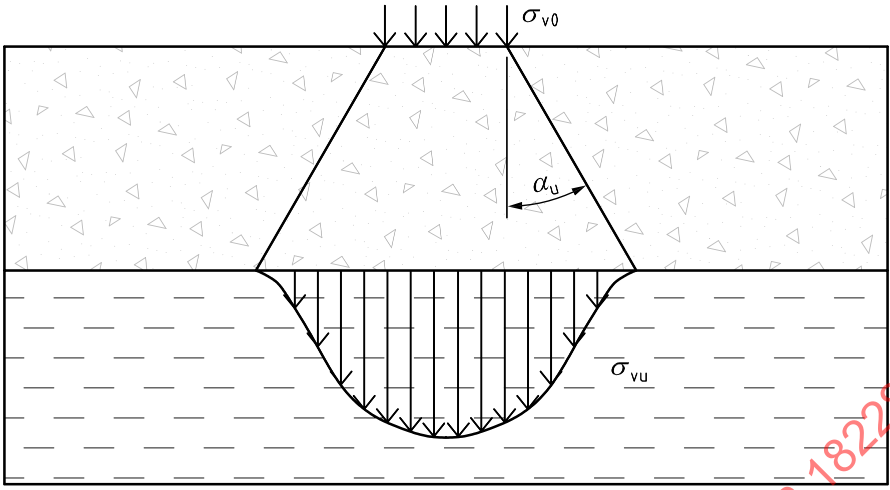
The hoop stresses and resistance provided by surrounding cells restrict lateral deformation of the fill by producing confining stresses σ_{3D} [Figure 3c)]. The intensity of the confining stresses depends on the height to diameter ratio of the geocell, the height of surcharge and the tensile properties of the geocell.

The confined zone shown in Figure 3c) includes two parts.

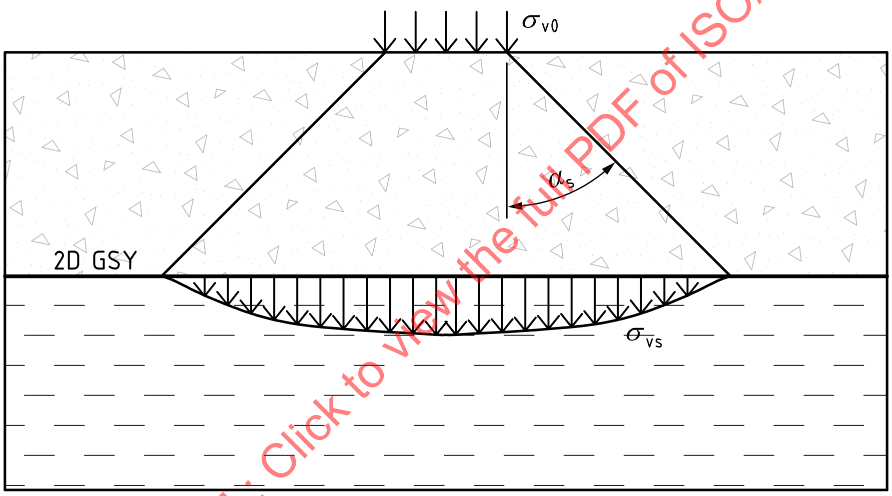
1) Cell height.

2) Limited thickness above and, possibly, beneath the geocell.

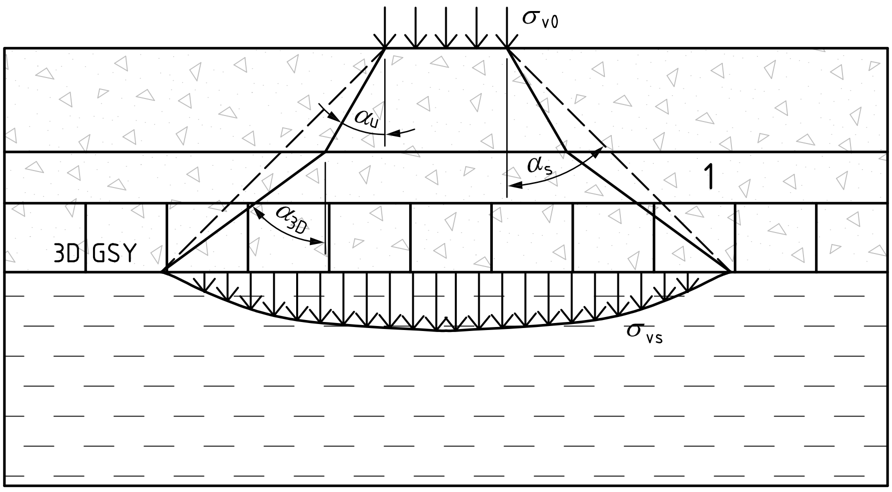
Above the confined zones, a transition zone is developed which extends until there is no influence on the granular (i.e. aggregate) layer from the geocell (i.e. unconfined zone). Figure 3c) illustrates the increased load spread angle under load.



a) Unstabilized granular (aggregate) layer



b) Two-dimensional confinement of granular (aggregate) layer



c) Three-dimensional confinement of granular (aggregate) layer

Key

$\sigma v0$ vertical stress applied at the top surface

σ_{vu} vertical stress on the subgrade with unstabilized granular (aggregate) layer

σ_{vs} vertical stress on the subgrade with stabilized granular (aggregate) layer

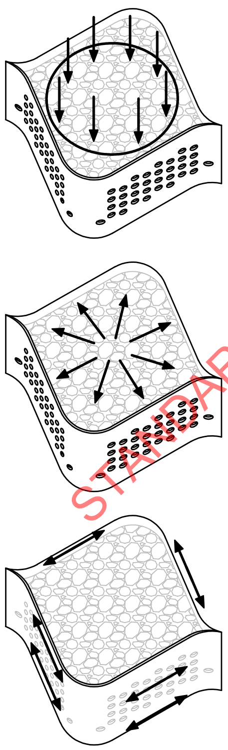
α_u load spreading angle with unstabilized granular (aggregate) layer

α_s load spreading angle with stabilized granular (aggregate) layer

α_{3D} load spreading angle within three-dimensional structure

1 confined zone

Figure 3 — Increase of the load distribution angle for bearing capacity increase



a) External vertical stress

b) Lateral stress on cell walls

c) Lateral confinement or restraint — Hoop tensile stress in cell walls

Figure 4 — Confinement mechanisms in geocells: Development of cell hoop stress by external vertical stress

4.4 Vehicular action

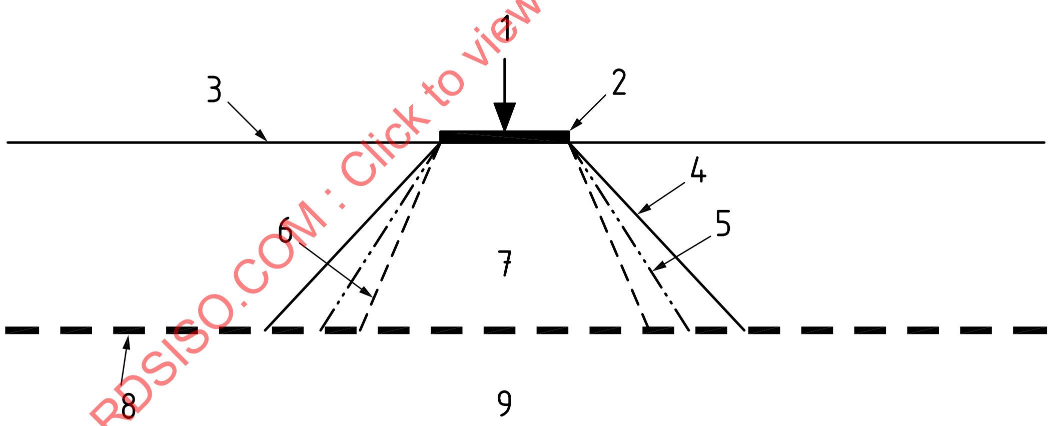
Vehicular loads applied to the road surface create stresses within the aggregate. As the wheels approach and then pass a given location along the pavement, the main principal stress for an individual aggregate particle is increasing and additionally rotating. This stress rotation is linked to a strong horizontal force component (i.e. shear stress), which finally leads to lateral spreading (i.e. shear strain) of the base aggregate particles. Geosynthetics are used to limit these deformations and the adverse effects related to it (e.g. surface rutting and decrease of bearing capacity).

When the unbound aggregate is installed and compacted on top of geosynthetics, shear resistance is generated between the two components because of frictional interaction. Other modes of interaction can be based on aperture interlocking (for geogrids) or cell and confinement fixation (for geocells). In such cases, granular soil particles can partially penetrate through the geogrid apertures or into the geocells and interlock with them.

One or both interaction mechanisms, friction or interlocking, are a prerequisite for limiting lateral movement of the aggregate particles on top of the geosynthetic. At the initial stage, where the fill material above the installed geosynthetic is compacted, the geosynthetic can be put into tension which causes development of corresponding strain and stress. The amount of strain depends on several factors, with the main factors being the stiffness of the subsoil, the degree and efficiency of interaction between aggregate and geosynthetic and intrinsic properties of the geosynthetic, in particular the stiffness.

4.5 Loading conditions (cyclic and static)

For some of the systems under discussion, the loading conditions immediately beneath the surface are cyclic. As vehicles continue to traffic a pavement overlying unbound aggregate the stress distribution angle within the unbound aggregate typically decreases (see Figure 5) due to cyclic loading. The accumulation of plastic deformations due to loading and unloading cycles normally leads to a reduction of the shear strength of the base course material. As a result, the maximum stress at the base and subgrade interface tends to increase over time.



Key

1 wheel load, P
2 tyre contact area
3 surface
4 initial distribution
5 distribution after N passes
6 distribution at failure
7 base
8 geosynthetic
9 subgrade

NOTE See Reference [1] for further information.

Figure 5 — Stress distribution angle

Bearing capacity failure of the subgrade occurs when the stress distribution angle decreases to a point at which the stress at the interface exceeds the mobilized shear strength of the subgrade.[1] The utilised shear strength of the subgrade depends on the undrained shear strength of the subgrade, the surface deformation or rut depth, the tyre contact area, and the thickness of the base.[1]

Under static loading conditions, the load distribution angle remains the same over time for an uncontaminated layer of unbound aggregate.

4.6 Multi component systems

Multi-component geosynthetic systems typically increase the thickness of the stabilized layer and provide improved confinement. As a result, a multi-component geosynthetic system normally results in improved performance.

A multi-component system can be either multiple layers of the same geosynthetic which provide either internal or external confinement only, or one which contains layers of different geosynthetics, thereby providing both internal or external confinement (or both) within a single system.

4.7 Static loads — Settlement control and limitation

The degree and extent to which loads are transferred from the surface of a stabilized layer to the underlying foundation soil is directly related to the amount of overall settlement that will occur over the life of the structure. For building loads and paved or unpaved roads constructed on embankments, refer to the appropriate national standard that highlights the design approach for estimation of settlement and the limitations of the approach. For each of these cases, the designer normally considers the loading area and amount of load transferred to the subgrade. However, for paved or unpaved roads, rutting potential or reduction in surface deformation is generally of greater concern.

4.8 Reduction in deformation from trafficking

As stated, confinement has been identified as the geosynthetics stabilization mechanism. In simplest terms, the geosynthetic restricts the lateral movement of aggregate fill placed upon or inside it. Many full-scale trials have shown that geosynthetic stabilized aggregate layers are significantly more resistant to surface deformation than non-stabilized layers when subjected to repeated trafficking loads. Furthermore, it has also been shown that deformation of the subgrade beneath the stabilized layer is also significantly reduced. This deformation reduction relies heavily on effective lateral restraint of the aggregate particles and the resulting confinement which creates a stiffer or higher modulus composite layer. Since elastic stiffness or resilient modulus of unbound aggregate is proportional to its confining pressure, the net effect of increasing the fill's modulus or stiffness is a spreading of the vertical stress distribution and a corresponding reduction in the deformation on top of the subgrade.[2] So, under repeated traffic loading, the geosynthetic not only restricts the movement of aggregate particles and thereby reduces deformation of the aggregate layer, the stiffer composite stabilized layer also restricts deformation of the subgrade.

4.9 Reduction in unbound aggregate degradation

The results of modelling and those of full-scale laboratory testing and traffic studies have demonstrated a positive improvement in the amount of trafficking that can occur over stabilized layers in comparison to control sections of non-stabilized granular structures. Comparing a non-stabilized and stabilized aggregate layer in full-scale study has shown both life extension and improved aggregate performance for the stabilized layer.[3] More aggregate degradation normally occurs in the control sections than the stabilized test sections.[4]

It is accepted that degradation of granular (i.e. aggregate) material increases the content of fines, reducing the water permeability and increasing the sensitivity of the granular (aggregate) material to reduced bearing capacity by increased water content and pore water pressure. For this reason, in addition to lateral confinement, the positive influence of separation, filtration and drainage geosynthetics can become significant.

4.10 Extension of design life

It follows from the preceding sections that retention of unbound aggregate particle shape, size and grading as well as minimization of particle movement will occur through lateral confinement, and where required, separation and filtration. The lateral confinement is provided by both particle to particle and particle to geosynthetic interaction. The degree of interaction is affected by aggregate quality, geosynthetic characteristics, placement conditions and level of compaction.

For some design situations, there is a direct correlation between one or more geosynthetic characteristics and performance. However, for other design situations, or for design conditions which are different from those in the specific design method, an extension in design life can only be determined by performing full scale or laboratory scale validation trials, or both. The aim of the trials is to calibrate the performance of a specific geosynthetic and aggregate combination for specific stress conditions. Refer to local and national guidance that can apply for pavement design. There are numerous local and national design approaches utilized around the world.

4.11 Ground and groundwater conditions

For stabilization, the geosynthetic must work in combination, as a composite, with aggregate fill to support applied loads (see Reference [5]). With respect to building over soft soils, either bearing-capacity-based (see References [2] and [6]) or serviceability-based (see References [7], [8] and [6]), design methodologies yield appropriate aggregate fill thickness for a given set of input parameters. Likewise, presuming the subgrade is soft because of moisture content, it is important to keep in mind that the aggregate fill is usually clean (cohesionless, with preferably single-digit percent fines) and drainable (see Reference [9]). Insufficient thickness, poor quality, or both can compromise the beneficial functions of the geosynthetic, and jeopardize stability of the system.

In all cases where a geosynthetic stabilized layer is incorporated within a pavement structure, the groundwater typically must remain below the influence of surface loads within the structural layer as far as practicable. Free drainage of the pavement is normally essential. If groundwater rises within the layers, the ability of the aggregate to transfer shear, and consequently maintain bearing capacity, is compromised. As a result, proper drainage and inclusion of lateral drains is normally essential in the proper functioning of the pavement system.

To ensure the aggregate fill is not compromised, it is normal to consider including a correctly designed separation or filtration geotextile that does not clog and allows free movement of water. Drainage geocomposites can also be used to provide separation, filtration and drainage functions.

4.12 Climatic conditions

With regard to both environmental and climatic conditions, Reference [10] reviews 140 long term pavement performance (LTPP) sites for paved roads covering environmental and climatic conditions throughout New Zealand. The findings suggest that roads in more sensitive climatic zones have an increased rut rate (by approximately 0,1 mm/year) as compared to the more stable climatic areas. For example, at an equivalent single axle load (ESAL) of 400 axles per day, the rut rates are:

— 0,16 mm/year for low sensitivity (dryer regions on more stable geological formations) areas;

— 0,28 mm/year (0,12 mm difference) for high sensitivity (wet regions on less stable geological formations) areas.

However, this research established the fact that the condition and presence of drainage, where needed, was much more important than just the environmental conditions alone. Observations from the data revealed that the rut rate of low volume roads was 2,5 times higher on poor drainage sections compared to sections where adequate drainage was provided throughout the pavement section. The researchers also established that sections having poor or inadequate drainage deteriorate much faster under heavy traffic volumes.

4.13 Sustainability

Sustainability is often described as considering three primary principles: social, environmental, and economic. The goal of sustainability is the satisfaction of basic social and economic needs, both present and future, and the responsible use of natural resources, all while maintaining or improving the well-being of the environment on which life depends (see Reference [11]).

Paved and unpaved roads constructed with geosynthetics meet all these objectives by reducing construction time, the amount of resources required to build these structures and the extension in use of or reduction in maintenance intervals.

Further, since one role of a stabilized layer is to protect the subgrade, the future rehabilitation of these roads are normally limited to restoration of the surfacing layers. The carbon emissions and embodied energy costs of incorporating a geosynthetic are normally seen as integral to a project. The designer typically considers the effect of the geosynthetic and calculates the net effect. For example, inclusion of a geosynthetic for stabilization can potentially result in a reduction in the use of aggregate material or enable the use of marginal quality or recycled soils or an increase in the service life of the road.[12] It can also result in more than one of these effects.[12] Case studies on the use of geosynthetics have been reported by the UK Waste Recycling and Packaging Organisation [13] and the European Association of Geosynthetic Manufacturers.

In terms of economic sustainability, stabilization with geosynthetics reduces aggregate costs and increases a pavement lifespan, thereby reducing maintenance costs.[14] The total carbon emission for the construction of a road, including maintenance etc, up to the end of the design life, is much less when using geosynthetics. Hence, geosynthetics are an effective method for carbon footprint reduction.

4.14 Impact of filtration, separation and drainage requirements

As a practical matter, water cannot be prevented from infiltrating an unpaved road structure, or even one that is paved.[15] Thus, it is prudent to assume that saturated, or elevated, moisture conditions exist within the upper reaches of the subgrade, particularly if the subgrade exhibits California bearing ratio (CBR) values of less than 3 %.

With a geosynthetic placed directly on top of the subgrade, the overlying aggregate fill (i.e. either base or sub-base material) is normally sufficiently graded to provide subgrade filtration and prevent soil migration. Filtration can be defined as allowing the free flow of water while separation means preventing intermixing of adjacent dissimilar soil and fill materials. For both separation and filtration, a geosynthetic or geosynthetic related product can be used. However, a filter stability calculation on the adjacent soil layers is normally be undertaken.

In the case of geocells, a semi-uniform perforation along and across the entire cell wall of a proper diameter are an essential factor for drainage. The hole size is normally small enough to prevent aggregate partition across cells and the perforation pattern is normally semi-uniform to allow free drainage.

5 Typical applications

5.1 Stabilized granular (aggregate) layers in trafficked areas

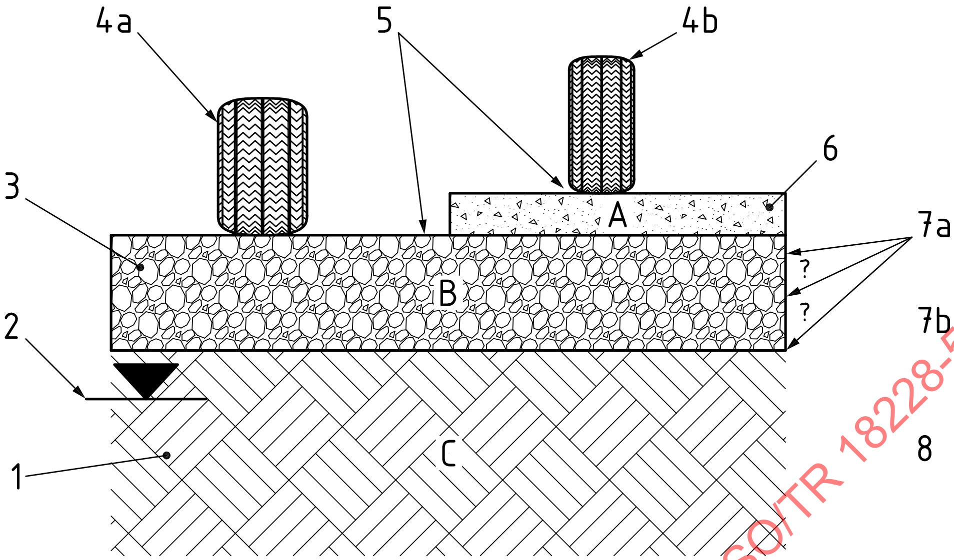
Typically, there are three principal design scenarios when considering the use of stabilized granular (i.e. aggregate) layers in dynamically loaded trafficked areas and highways.

a) Layers are designed to carry the loads from construction vehicles and in-service traffic without additional bound or unbound pavement layers. Examples include haul roads and minor (perhaps rural) unsurfaced and unbound roads.

b) Layers are designed to carry the loads from construction vehicles only and overlying pavement layers is designed to carry all in-service traffic with no influence from the stabilization assumed. Examples include highways where the stabilized granular (aggregate) layer forms part of the road foundation (base and sub-base) and bound pavement layers (typically asphaltic or cementitious) are constructed above.

c) Layers are the same as 5.1 b), but the design of the overlying pavement includes influence from the stabilization of the granular (i.e. aggregate) layer.

Key factors influencing design and performance are shown in Figure 6.



Key

A bound pavement layer(s)
B stabilized granular material
C subgrade (clay, silt or sand)
1 type and strength of subgrade
2 groundwater
3 type, compaction and thickness of granular material
4a load and volume or frequency of construction traffic
4b load and volume or frequency of in-service traffic
5 allowable deformation
6 thickness and properties of pavement layer
7a location(s) of geosynthetic(s)
7b type and properties of geosynthetic(s)
8 required design life and maintenance program

Figure 6 — Factors influencing design and performance in trafficked areas on unbound and overlying bound pavement layers

5.2 Stabilized granular (aggregate) layers in railways

Typically, there are five principal design scenarios when considering the use of stabilized granular (aggregate) layers in dynamically loaded rail track-bed applications.

a) Stabilized sub-ballast or foundation layers are designed to carry the loads from construction vehicles and to act as a working platform for placement of overlying ballast or bound layer which supports the sleepers and rails and accommodates in service rails loads.

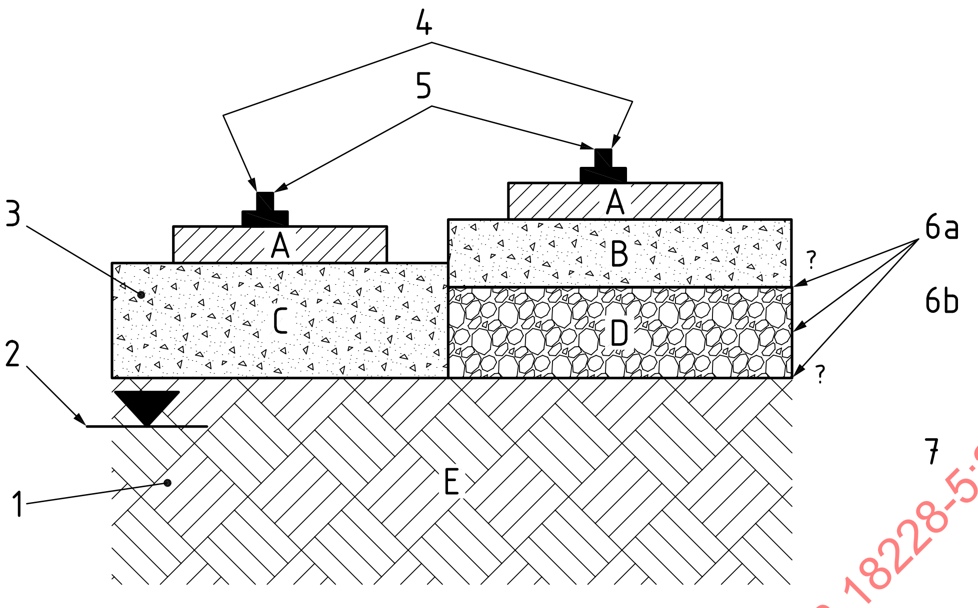
b) Layers are same as 5.2 a), but the influence of the stabilization is included in the analysis of the overall rail track bed performance and its ability to carry the in-service traffic loads.

c) Stabilized ballast layer is directly constructed on subgrade and therefore designed to accommodate construction traffic and in-service rail loads.

d) Both the sub-ballast layer and the bound or ballast layer are stabilized.

e) Stabilization of the ballast layer is for the purpose of reducing ballast particle movement under service conditions, but with no load carrying benefit assumed.

Key factors influencing design and performance are shown in Figure 7.



Key

A sleeper
B ballast layer: stabilized or non-stabilized
C stabilized ballast layer(s)
D stabilized sub-ballast layer
E subgrade (clay, silt or sand)
1 type and strength of subgrade
2 groundwater
3 type, compaction and thickness of ballast or sub ballast
4 load and volume or frequency of traffic
5 allowable deformation
6a location(s) of geosynthetic(s)
6b type and properties of geosynthetic(s)
7 required design life and maintenance programme

Figure 7 — Factors influencing performance in rail track bed stabilized sub-ballast or ballast layer

The train wheel load is applied on the rail and then from the rail to the sleeper. The load then spreads, with a set distribution angle, from the top to the bottom face of the sleeper. Hence, the loaded area on top of the bound or ballast layer is rectangular with a width equal to the width of the sleeper and length equal to the width of the rail increased according to the set load distribution angle in the sleeper. Stabilized layers for railways usually must be designed for direct construction traffic.

5.3 Stabilized granular (aggregate) layers in working platforms

Typically, the types of loading to be accommodated when considering the use of stabilized granular (aggregate) layers in stabilized working platforms are as follows.

a) Dynamic loads associated with construction traffic as previous scenarios.

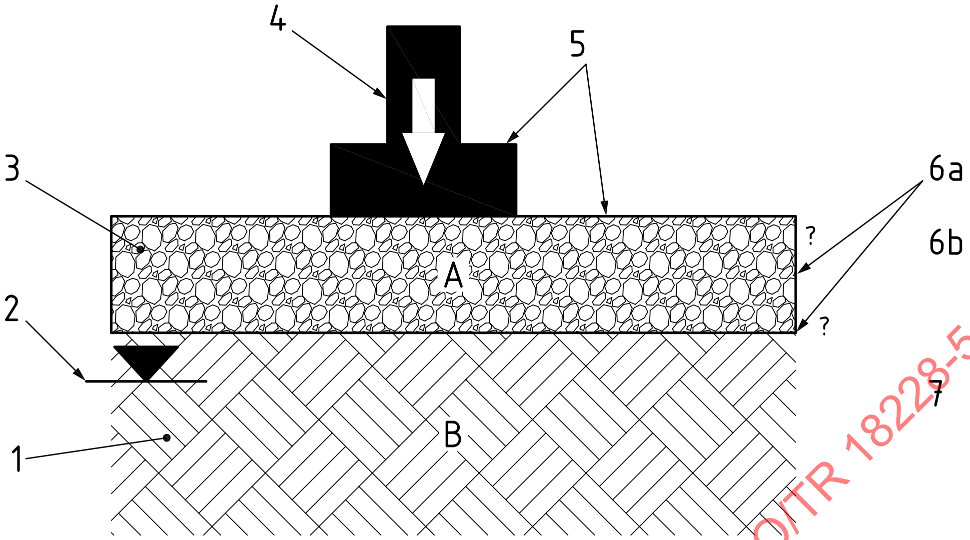
b) Slow moving, high magnitude low frequency dynamic loads (effectively static) associated with large items of plant and machinery travelling slowly across site, perhaps stationary for extended periods (days or weeks). The static load can be considered as applied on a circular area in the case of tyred machinery and on a rectangular area for tracked machinery.

c) High magnitude medium frequency dynamic loads associated with the operation of large items of plant (e.g. piling rigs and cranes). These loads are typically imparted via tracks, pads or outriggers and can be critically high point loads.

d) Parking decks, container areas and other situations where a load transfer platform, made from unbound granular (aggregate) layers, is required. Also, in these cases, loads can be considered as static, with usually lighter loads than in the above, but can be applied over large areas.

The design for slab foundation and working platform is normally proven at the ULS in terms of both local and global stability.

Key factors influencing design and performance are shown in Figure 8.



Key

A	stabilized granular material	4	magnitude and volume or frequency of loading
B	subgrade (clay, silt and sand)	5	allowable deformation and ultimate bearing capacity
1	type and strength of subgrade	6a	location(s) of geosynthetic(s)
2	groundwater	6b	type and properties of geosynthetic(s)
3	type, compaction and thickness of granular material	7	required design life and maintenance program

Figure 8 — Factors influencing performance in working platforms with high magnitude pseudo static loads

6 Design methods

6.1 General

The design methods for many unbound layers (e.g. roadways) are characterized by their empirical origins based on the particular nature of the experiences in each part of the world. Many methods are based on the application and provision of standard thicknesses of various layers of aggregate. Others provide empirical or semi-empirical design rules and equations.

At present, it is not yet possible to include a single generic product characteristic of a geosynthetic in most of the design methods to quantify the stabilizing effect to the construction. Geosynthetics have been included in some designs to provide an empirically observed improvement of the system parameters, e.g. improvement of modulus. Evidence of the improvement in system parameters can also alternatively be provided by field and full-scale laboratory testing of the specific product in the appropriate aggregate and loading for the application.

Clause 6 provides details of some design methods that are adopted in various applications.

6.2 Unpaved roads

6.2.1 Giroud-Han (2004) method

The Giroud-Han[7] method provides a rational empirical framework that makes it possible to quantify the benefits of all types of geosynthetics for the stabilization of unbound granular (aggregate) layers. It is important to note that, for valid use and to ensure reliable results, the Giroud-Han method necessitates calibration for each specific type of geosynthetic under consideration.

The method has only been calibrated for geogrids using the aperture stability modulus (measured by ASTM D7864) as the characteristic property of the geogrids. Other properties, or groups of properties, can be used to calibrate the method for all types of geogrids. In the past several years, hundreds of unpaved roads and areas in the United States, Canada and Latin America have been designed in a consistent manner using the Giroud-Han method and there have been no reported performance problems on projects designed using this method.

The Giroud-Han method has been added to the US Federal Highway Administration (FHWA) Geosynthetics Training Manual as a recommended design method. It is different from other methods, as it makes it possible to quantify the influence of geogrid and geotextile properties.

The Giroud-Han method provides the following formula for calculating the required thickness of the unbound granular (aggregate) layer:

$$h=\frac{\left\{0{,}868+CF\left(\frac{r}{h}\right)^{1{,}5}\right\}Log_{10}N}{\{1+0{,}204[R_{\mathrm{E}}-1]\}}\times\left(\sqrt{\frac{\frac{P}{\pi r^{2}}}{\left(\frac{s}{f_{\mathrm{s}}}\right)\left\{1-0{,}9\exp\left[-\left(\frac{r}{h}\right)^{2}\right]\right\}N_{\mathrm{C}}C_{\mathrm{M}}}}-1\right)r \quad (1)$$

where

h	is the required compacted aggregate (gravel) thickness (m);
CF	is the calibration factor for the geosynthetic used in design for a “specific set” of punched and drawn biaxial geogrids;
N	is the number of axle passes;
R_{E}	is the limited modulus ratio of compacted aggregate to subgrade soil (maximum = 5,0);
P	is the wheel load (kN);
r	is the radius of the equivalent tyre contact area (m);
s	is the allowable rut depth (mm; for rut depths between 50 mm and 100 mm);
f_{s}	is the reference rut depth (75 mm);
N_{c}	is the bearing capacity factor (3,14 for unreinforced; 5,14 for geotextile reinforced; 5,71 for geogrid reinforced);
C_{u}	is the undrained shear strength of subgrade (taken as 30 kPa x CBR of the subgrade soil for CBR’s between 1 % and 5 %); and
$P/\pi r^2$	is the tyre contact pressure (kPa), and is equivalent to the tyre pressure (p).

The base thickness h appears on both sides of the above equation, which can therefore be solved by iterations.

The original Giroud-Han method, to which Formula (1) refers, has been developed for geotextiles and biaxial extruded geogrids, hence the method necessitates calibration in the case of extension to other types of stabilizing geosynthetics.

Limits and applicability of the Giroud-Han method are provided in References [7],[8],[1],[16] and [3].

6.2.2 Leng-Gabr (2006) method

The Leng and Gabr[17] method for the design of stabilized unpaved roads shows some similarity to the Giroud-Han method, but has important differences in the theoretical approach and in the parameters used to characterize the effect of stabilizing geosynthetics.

The method is based on a simplified analysis of vertical stress diffusion, on the correlation between the degree of bearing capacity mobilization, and on the elastic secant modulus of stabilizing geosynthetics at 2 % strain (see ISO 10319). The method has been validated with experimental tests results.

In the past several years, hundreds of unpaved roads and areas around the world have been designed in a consistent manner using the Leng-Gabr method and there have been no reported performance problems on projects designed using this method so far.

On the basis of plate loading tests and in-situ tests, it has been demonstrated that the inclusion of stabilizing geosynthetics within the base course delays and minimizes the load spreading angle degradation at an increasing number of passes. Based on experimental tests and empirical criteria, geogrid influence is introduced through the tensile strength at 2 % elongation $T_{2\,\%}$ (see ISO 10319). If $T_{2\,\%}$ is different between the longitudinal and transversal directions, the average value is normally assumed.

The Leng-Gabr method provides the required thickness of the base of an unpaved road through the following formula:

$$h=\frac{0{,}85a(1+k_2\log N)}{\tan\alpha_1}\left(\sqrt{\frac{p}{\mathrm{mN_c}C_\mathrm{u}}}-1\right) \tag{2}$$

With:

$$k_2=(\mathrm{a/h})^{0{,}081}\,\mathrm{Max}\left[\left(0{,}58-0{,}000\,046\mathrm{T_{2\,\%}}^{4{,}5}\right),0{,}15\right] \tag{3}$$

$$m=\left[1-e^{-0{,}78(\mathrm{a/h})}\right]\frac{r}{r_\mathrm{cr}}\leq 1$$

$$r_\mathrm{cr}=0{,}025\cdot(0{,}125\log N+1{,}5)$$

where

h is the aggregate base thickness (m);

J is the aperture stability modulus (N-m/deg);

p is the tyre pressure (kPa);

a is the radius of tyre footprint (m);

N is the number of standard 80 kN axle passes;

N_c is the subgrade bearing capacity factor;

- = 3,80 (unstabilized base);
- = 5,69 (geotextile stabilized base);
- = 6,04 (geogrid stabilized base);

C_u is the subgrade undrained shear strength (kPa);

α_1 is the initial load spreading angle (for N = 1);

k_2 is the coefficient defining the degradation of the load spreading angle with increasing number of passages;

$T_{2\,\%}$ is the tensile strength of the stabilizing geosynthetic at 2 % strain (kN/m);

r is the rut depth (m);

r_{cr} is the critical rut depth, reached when the full bearing capacity is mobilized (m).

Note that the base thickness h appears on both sides of the above equation, which can be therefore solved by iterations.

Refer to Reference [17] for information regarding the limits and applicability of the Leng-Gabr method.

6.2.3 Pokharel (2010) method for geocells

Based on studies of geocell stabilization mechanisms, numerical modelling and field trials, methods for design with geosynthetics were modified and adapted to geocells. The modifications include changing geosynthetic dependent parameters (such as torsional stiffness and tensile strength at 2 % strain) to geocell dependent parameters (such as elastic stiffness, creep resistance less than 2 % and tensile strength).

Pokharel[18] modified the Giroud and Han[7] design methodology for use with geocells. A specific set of geocell products, with specific properties were validated by laboratory cyclic plate loading tests and full-scale moving wheel tests. In the design methodology, a maximum allowable rutting is set (together with all other parameters) and the pavement thickness is determined by:

$$h = \frac{(0{,}868 + k'\log N)}{\eta\{1 + 0{,}204(R_E - 1)\}} \times \left(\sqrt{\frac{P}{\pi r^2 m 5{,}14 c_u}} - 1\right) r \tag{4}$$

where

h is the required base course thickness (m);

r is the radius of tire contact area (m);

N is the number of ESAL;

P is the wheel load (kN);

c_u is the undrained cohesion of the subgrade soil (kPa);

R_E is the modulus ratio of base course to subgrade soil;

m is the bearing capacity mobilization factor;

η is the conversion factor (0,689 for cyclic plate loading tests, 1,0 for field tests);

k' is the geocell calibration factor for the product used in design.

Giroud and Han[7] proposed a factor (k) that controls the rate of reduction in the stress distribution angle, which depends on the (r/h) ratio and the aperture stability modulus of the geogrid. Since the aperture stability modulus is not suitable for geocells, Pokharel[18] proposed a factor (k') to replace the *CF* (calibration factor) for the design of geocell bases over weak subgrade as shown in the equation above.

The reduction in the load distribution angle with the number of passes caused by the deterioration of the base course material under the repeated loading in the laboratory was observed by Gabr[19] for geogrid stabilization. Recent research showed that geocell confinement slows down the rate of deterioration and increases and maintains the modulus of the base course.

The modulus improvement factor I_f was proposed by Han et al[20] to account for this benefit:

$$I_f = E_{bcs} / E_{bcu} \tag{5}$$

where E_{bcs} and E_{bcu} are the moduli of stabilized and unstabilized base course.

Considering I_f, the modulus ratio R_E in the Pokharel formula can be expressed as:

$$R_E = I_{f^*}\left(E_{bcu} / E_{sg}\right) = \max\left(10{,}0;\ I_{f^*}\left[\left(3{,}48\ CBR_{bc}{}^{0{,}3}\right) / CBR_{SG}\right]\right) \tag{6}$$

where CBR_{bc} and CBR_{sg} are the CBR values of the unstabilized base course and of the subgrade.

Han et al[20] reported that the modulus ratio with geocell stabilization ranges between 4,8 and 10,0, suggesting the value for R_E is limited to no greater than 10,0, depending on the particular geocell being used.

The bearing capacity mobilization coefficient (m) can be calculated by:

$$m = \left(\frac{s}{f_s}\right)\left\{1 - 0{,}9\exp\left[-\left(\frac{r}{h}\right)^2\right]\right\} = \left(\frac{s}{75\,mm}\right)\left\{1 - 0{,}9\exp\left[-\left(\frac{r}{h}\right)^2\right]\right\} \tag{7}$$

where

s is the limit rut depth at the top of the stabilized base course (mm);

f_s is the rut factor = 75 mm.

The factor used by Giroud-Han[7] to control the rate of reduction in the load distribution angle, which depends on the aperture stability modulus J for geogrids, has been replaced by the term:

$$0{,}868 + k'\log N \tag{8}$$

Pokharel[18] determined that it is possible to use cyclic plate loading tests and full-scale moving wheel tests to develop a unique geocell calibration factor (k') for a particular geocell product. Accordingly, designers can consult with geocell product manufacturers to better understand the suitability of using a specific value of k' for use in unpaved road design.

6.2.4 Emersleben (2009) method for geocells

Emersleben[21] developed an SLS design of a geocell stabilized system used for unpaved road constructions. The model is verified by large-scale model and in-situ tests and gives the expected settlements for a stabilized layer depending on the subgrade strength, the traffic amount and the geocell material strength.

a) Calculation of settlements:

The cumulative settlements are the sum of settlements within the geocell layer and subsoil settlements.

$$S_{Ges} = S_G + S_U \tag{9}$$

where

S_{Ges} is the cumulative settlement;

S_G is the settlement in geocell layer;

S_U is the subsoil settlement.

b) Calculation of geocell settlement:

The settlement within the geocell and the geocell strains are linked according to the following.

$$S_{G,b} = \left(1 - \frac{{d_0}^2}{(d_0 + \varepsilon \cdot d_0)^2}\right) \cdot h_0 \lambda \tag{10}$$

where

$S_{G,b}$ are settlements within the geocell layer;

d_0 is the geocell installation diameter;

ε is the geocell material strain dependent on vertical stress;

h_0 is the geocell height at installation;

λ is the settlement reduction factor (between 0,3 and 0,7).

The load dependent strain ε can be calculated for a strain level up to 2 % as indicated in formula below:

$$\varepsilon_{(<2\,\%)} = -\frac{\left(\dfrac{2 \cdot t \cdot a \cdot E + a_1 + a_2 - a_3}{(a_1 + a_2)}\right)}{2} + \sqrt{\left(\frac{\left(\dfrac{2 \cdot t \cdot a \cdot E + a_1 + a_2 - a_3}{(a_1 + a_2)}\right)}{2}\right)^2 + \frac{a_3}{(a_1 + a_2)}} \tag{11}$$

where:

$$a_1 = \frac{1}{0{,}04} \cdot \gamma \cdot z \cdot k_{pgh} \cdot \mu_E \cdot d_0 \cdot A$$

$$a_2 = \frac{1}{0{,}02} \cdot c \cdot k_{pch} \cdot \mu_C \cdot d_0 \cdot A$$

$$a_3 = \left(\sigma_v \cdot k_{pgh}\right) \cdot d_0 \cdot A$$

For strains in excess of 2 %, the load dependent strain can be calculated based on the formula below:

$$\varepsilon_{(\varepsilon \geq 2\,\%)} = \frac{\left(\sigma_v \cdot k_{agh} - b_1 - b_2\right) \cdot d_0 \cdot A}{\left[E \cdot a \cdot 2 \cdot t - \left(\sigma_v \cdot k_{agh} - b_1 - b_2\right) \cdot d_0 \cdot A\right]} \tag{12}$$

where

$$b_1 = \frac{1}{2} \cdot \gamma \cdot z \cdot k_{pgh} \cdot \mu_E$$

$$b_2 = c \cdot k_{pch} \cdot \mu_C$$

where

σ_v is the vertical stress on geocell;

d_0 is the geocell installation diameter;

E is the geocell secant modulus from radial strain tests;

a is the area cross-section with provision for perforation;

A is the area cross-section without provision for perforation;

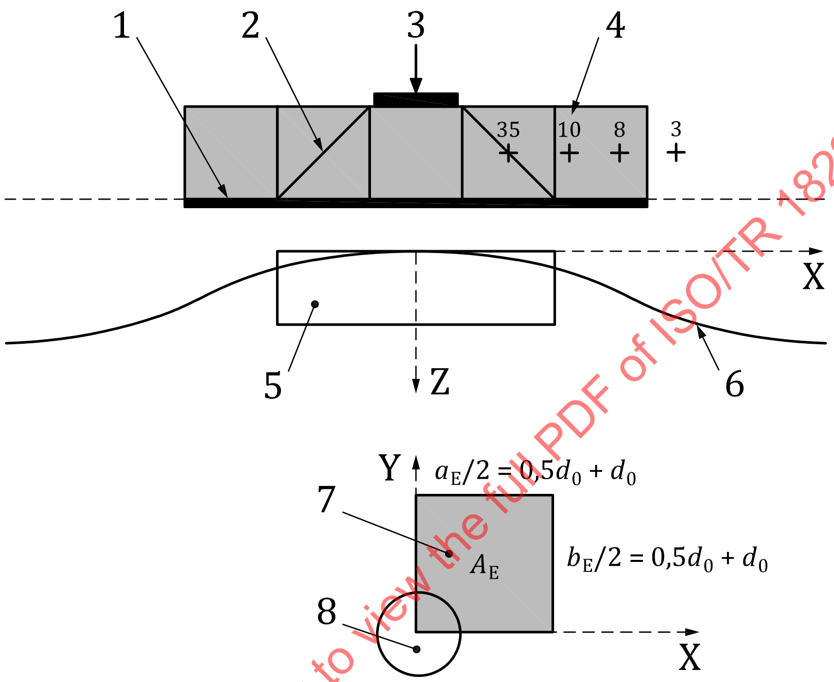
μ_E is the coefficient for three-dimensional earth pressure;

μ_C is the coefficient for three-dimensional cohesion;

c is the cohesion due to geocell soil stabilization.

c) Subsoil settlement calculation:

Geocells lead to an improved distribution of applied loads when compared to a non-stabilized soil layer. This effect can be measured in radial- and vertical strain tests by an increase of measured horizontal soil stresses in geocell stabilized structures. A safe side assumption for settlement calculations is to use the soil friction angle as a load distribution angle.



Key

1 surrogated load area

2 load distribution

3 load

4 measured horizontal stresses at a vertical load of 500 kN/m^2

5 idealized stress distribution beneath surrogated load area

6 stress distribution (model tests)

7 surrogated load area for consideration of stress distribution due to geocells

8 load area

Figure 9 — Load distribution model for a geocell stabilized soil and derived surrogated load area (A_E)

Settlements of the soil layers below the geocell (S_U) can be approximated using a closed form solution and a surrogated load area according to DIN 4019 as indicated in Formula (13)

$$S_U = \frac{\sigma_{0,E} \cdot b_E}{E_M} \cdot f_0 \quad (13)$$

where

S_U is the settlement of soil layers below geocells;

$\sigma_{0,E}$ is the normal stress on surrogated load area A_E;

b_E is the surrogated width using geocell load distribution angle;

E_M is the mean stiffness value of the relevant soil layer;

f_0 is the settlement factor.

Traffic amount can be considered using Formula (14) or other available empirical correlations between static and dynamic loads.

$$\sigma_{v,zykl} = \sigma_{v,stat} \cdot N^{0,046\,8} \tag{14}$$

where

$\sigma_{v,zykl.}$ is the surrogated static load to stimulate N load cycles;

$\sigma_{v,stat.}$ is the static reference load;

N is the load cycles;

$N^{0,046\,8}$ is the load factor to consider cyclic loads.

6.3 Paved roads

6.3.1 Modified American Association of State Highway and Transportation Officials (AASHTO) (1993) method

In North America, the state of practice for the design of flexible paved roads with geogrid stabilization in the base layer is in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures[22] which has been aligned with the requirements of the AASHTO standard practice document.[23] The AASHTO design method was developed in the early sixties based on the AASHTO road test and later revised multiple times until the final version in 1993.

The AASHTO method uses empirical equations based on multilayer elastic theory to model the pavement cross section. The structure is characterised by a structural number (*SN*), that reflects the combined structural capacity of the flexible pavement to carry a predefined traffic load. During the design process, *SN* is selected such that the pavement cross section will be able to support the anticipated traffic load without experiencing a loss in serviceability no greater than what established by the project requirements. The required *SN* is determined by solving the following performance equation:

$$\log W_{18} = Z_R S_0 + 9{,}36 \log(Sn+1) - 0{,}2 + \frac{\log\left(\dfrac{\Delta PSI}{2{,}7}\right)}{0{,}4 + \dfrac{1\,094}{(SN+1)^{5{,}19}}} + 2{,}32 \log M_R - 8{,}07 \tag{15}$$

where

W_{18} is the number of 80 kN ESAL applications over the design life of the pavement;

Z_R is the standard normal deviate;

S_0 is the standard deviation;

ΔPSI is the allowable loss in present serviceability index (PSI);

M_R is the resilient modulus of the subgrade.

Once the required *SN* is determined, the pavement structure can be designed through a series of iterations:

$$SN = a_1 \cdot D_1 + a_2 \cdot D_2 \cdot m_2 + a_3 \cdot D_3 \cdot m_3 \tag{16}$$

where

a_1, a_2 and a_3 are the layer coefficients of surface, base and subbase respectively (relative strength of the layer);

D_1, D_2 and D_3 are the thicknesses of surface, base and subbase layers respectively;

m_2 and m_3 are the drainage coefficients of base and subbase.

According to the GMA white paper,[24] the structural contribution of geosynthetics to the overall structural capacity of the pavement structure can be included in the design through three factors.

1) Traffic benefit ratio (*TBR*): Ratio of number of load applications of the geosynthetic stabilized section over the number of load applications of the unstabilized section with the same geometry, material and same defined failure state.

2) Base course reduction factor (*BCR*): Percent reduction in the thickness of the geosynthetic stabilized base layer compared to an unstabilized one.

3) Layer coefficient ratio (*LCR*): Ratio of the layer coefficient of the geosynthetic stabilized base layer over the layer coefficient of the unstabilized one.

Each of these three factors is derived from empirical data specific to each geosynthetic. It means that each of these parameters must be determined from laboratory and field tests carried out by each manufacturer. Specifically, full-scale test trials independently certified must generally justify the data properly and identify the geosynthetic benefits.

When designing a geosynthetic stabilized pavement structure, one of the first steps AASHTO R50-09[43] suggests is assessing the target benefit of using a geosynthetic in terms of service life, reduction of base thickness or both. This leads the designer to choose whether to apply the *TBR*, the *LCR*, or the *BCR* approach.

The *TBR* can be modified to evaluate the increased total number of ESAL due to the beneficial effect of the geosynthetics:

$$W_{18,\text{ stabilized}} = TBR \cdot W_{18,\text{ unstabilized}} \tag{17}$$

Typical TBR values range from 1,5 to 10 for geotextiles and 1,5 to 70 for geogrids.

On the other hand, if the design target is to reduce the base layer thickness, then the *BCR* approach can be followed, where *BCR* is defined as:

$$BCR = 1 - \frac{D_{\text{stabilized}}}{D_{\text{unstabilized}}} \tag{18}$$

This can be modified as:

$$SN = a_1 \cdot D_1 + \frac{1}{1 - BCR_2} D_{2,BCR} \cdot a_2 \cdot m_2 + \frac{1}{1 - BCR_3} D_{3,BCR} \cdot a_3 \cdot m_3 \tag{19}$$

When only the base layer is stabilized, the reduced base layer thickness can be calculated as follows:

$$D_{2,BCR} = \frac{SN - a_1 \cdot D_1 - a_3 \cdot D_{3,BCR} \cdot m_3}{BCR_2 \cdot a_2 \cdot m_2} \tag{20}$$

Typical base course thickness savings range between 20 % and 30 % for geotextiles and 30 % to 50 % for geogrids, resulting in *BCR* = 0,2 - 0,3 for geotextiles and *BCR* = 0,3 - 0,5 for geogrids.

In the *LCR* design approach, the coefficient is directly applied to implement the benefit of the geosynthetic into the design method to reduce the base layer thickness. However, the *LCR* is calculated from field test results as the ratio of layer coefficients instead of layer thicknesses. It therefore represents the impact that a specific geosynthetic provides to the layer coefficient of the layer in which the geosynthetic is placed. The performance benefits are therefore implemented in the design method as follows:

$$SN = a_1 \cdot D_1 + (LCR_2 \cdot a_2) \cdot D_2 \cdot m_2 + (LCR_3 \cdot a_3) \cdot D_3 \cdot m_3 \tag{21}$$

The *LCR* is usually obtained and plotted as a function of the *CBR* of the layer below. Therefore, for the same geosynthetic, two different *LCR* values are normally used for the base (having subbase below) and subbase (having subgrade below).

Following this method, the designer can quantify the performance benefits of the geogrid reinforcement through both the increase in service life (i.e. the number of ESAL) and decrease in layer thickness. The standard practice for all these three approaches is empirical in nature and relies on the experience of the designer and the published data from the manufacturer on full scale test results to determine the performance benefit coefficients.

The relationship between *LCR* and *BCR* is derived as follows:

$$LCR = \frac{1}{1 - BCR} \tag{22}$$

The *LCR* and *BCR* approaches are just two faces of the same coin, since *LCR* and *BCR* values, for the same geosynthetic and the same *CBR* of the layer below, are directly linked by a simple correlation. However, a direct relationship cannot be established between *LCR* or *BCR* and *TBR*. It has been shown (see Reference [25]) that *TBR* and *LCR* approaches can be used independently, but the *LCR* approach leads to more conservative results.

6.3.2 MEPDG methods

In 2008, AASHTO officially adopted the Mechanistic-Empirical Pavement Design Guide (MEPDG). The method uses mechanistic principles to minimize design reliance on empirical observation and maximize reliability. Mechanistic properties of materials of each pavement layer are input in the design method. Therefore, the MEPDG platform is normally considered as better suited to incorporate the benefits of geosynthetic stabilization. In the available MEPDG models, geosynthetic stabilization is often incorporated as an equivalent resilient modulus and Poisson ratio. However, depending on the asphalt layer thickness, the geosynthetic contribution has been incorporated into the properties of the base course also as an equivalent delay in the onset of fatigue cracking. Therefore, the benefits of the geosynthetic reinforcement have not been consistently defined (see Reference [26]). The use of the MEPDG method with geosynthetics for stabilization necessitates the definition of rut and fatigue laws for stabilized roads, which necessitates extensive testing on stabilized sections. The application of MEPDG to stabilized roads is still under development.

6.4 Working platforms and load transfer platforms

6.4.1 General

When dealing with working platforms for heavy machinery or with load transfer platforms for parking decks, container yards and similar applications, the loads can be considered as static.

In these applications, geosynthetics still provide the stabilization function. That is, the confinement mechanism and sometimes the tensioned membrane mechanism are provided by the geosynthetics, but the reference limit state is the ULS for bearing capacity.

There are two typical soil conditions which can produce inadequate bearing capacity for a working platform or a load transfer platform.

— Soft silt or clay.

— Loose sand.

These two soil conditions are addressed by two different design methods: the Static method and the BCR method.

6.4.2 Static method for clay subgrade

When a vertical static load is applied to a load transfer platform involving a gravel aggregate and clay subgrade system, horizontal and vertical stresses are generated. The horizontal stresses in the aggregate result in outward shear stresses on the surface of the subgrade. These shear stresses can reduce the bearing capacity of the clay to as little as one half the value for purely vertical loading. If a stabilizing geosynthetic is present, these shear stresses can be carried by the geosynthetic (depending on interface friction), allowing the full bearing capacity of the clay to be mobilized.

Research by Rodin[27] and Barenberg,[28] among others, showed that the bearing capacity of a soft clay subgrade, unstabilized granular (aggregate) layer system can be approximated as:

$$q_u = \pi c_u \tag{23}$$

while the bearing capacity of a soft clay subgrade, stabilized granular (i.e. aggregate) layer system can be approximated as:

$$q_r = 2\pi c_u \tag{24}$$

Rutting and deformations of the granular (aggregate) platform can be limited by reducing the allowable bearing capacity through a proper factor of safety (FS). An FS of 2 results in good reduction of deformations and displacements. An FS of 3 is normally selected when allowable deformations need to be minimal:

$$q_{ua} = q_u / \mathrm{FS}; \tag{25}$$

$$q_{ra} = q_r / \mathrm{FS}.$$

The static method derives from the research given in References [27] and [28], among others, and is applicable for the design of load transfer platforms built over soft clay soil.

The static method is based on the distribution of pressures generated at the surface of the soft clay layer by a circular or rectangular loaded area on top of the granular (aggregate) layer through a planar 2D stabilizing geosynthetic. The design method ensures that the pressure at the top of subgrade is less than the allowable bearing pressure of the subgrade soil divided by a proper FS. This technique assumes that the vertical pressures are distributed through the platform soil layer according to the Boussinesq theory for circular or rectangular loaded areas. If a vertical pressure (q) is applied, evenly distributed on a circular or rectangular area, on a homogeneous elastic half space, since the overburden stress ($\gamma \cdot h$) is constant, the application of the pressure (q) generates equilibrium conditions in every point of the medium. Hence the vertical stress is independent of the medium characteristics. The Boussinesq theory provides the stress components at every point along the vertical line passing through the centre of the circular area or through one corner of the rectangular area.

The wheel load (P) applied by a vehicle is approximately equivalent to a uniform vertical stress, equal to the tyre pressure (p), on a circular area where the equivalent radius is:

$$r = (P / \pi p)^{0,5} \tag{26}$$

Introducing this into the Boussinesq equation for uniform vertical stress on a circular area and solving for depth (z), the required thickness of the unstabilized layer is:

$$Z_\mathrm{u} = \mathrm{r} / \left[(1 - q_\mathrm{ua} / p)^{-2/3} - 1 \right]^{1/2} \tag{27}$$

where z_u is the thickness of platform (m) at which the bearing capacity q_u becomes equal to the vertical stress induced by the tire pressure p on the circular area of radius r.

The required thickness of the unstabilized layer is:

$$Z_\mathrm{r} = r / \left[(1 - q_\mathrm{ra} / p)^{-2/3} - 1 \right]^{1/2} \tag{28}$$

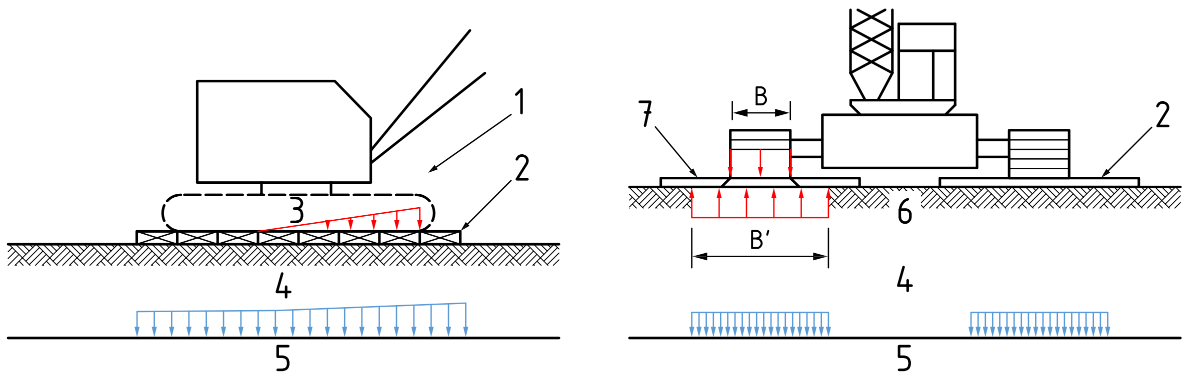
Similar formulas, using the Boussinesq equation for uniform vertical stress p on a rectangular area, allow calculation of the required thicknesses z_u and z_r in the cases of rectangular loads, like tracks and bearing pads (see Reference [29]).

Comparison of z_r and z_u for equal vertical stress and size of the loaded area show that the platform thickness can be reduced by 30 % to 50 % by using planar 2D stabilizing geosynthetics.

After setting the platform thickness, the stabilizing geosynthetics layout is usually designed. The best results are usually obtained with 2 to 3 layers of equally spaced stabilizing geosynthetics at vertical centres of 150 mm to 300 mm. Further details can be found in Reference [30].

Other similar methods are based on Westergaard's layered elastic theory for the determination of the amount of vertical stress in the subgrade beneath a stabilized layer, rather than on the Boussinesq theory. Westergaard's analysis is based on the assumption that the soil on which load is applied is stabilized by closely spaced horizontal layers (geosynthetics in this case) which prevent horizontal displacement. The effect of the Westergaard assumption is to reduce the stress at the top of layer 2 in Figure 10, below those obtained by Boussinesq equations.

Westergaard's theory assumes that the Poisson modulus is equal to zero since it assumes that the thin horizontal layers afford infinite stiffness, that is with no horizontal strain. Hence methods based on the Westergaard theory are less conservative than methods based on the Boussinesq theory. As such, it is advisable to check in advance whether the Westergaard theory is applicable in the considered design situation.



Key

1 pick loading
2 timber mat
3 track pressure
4 layer 1 geosynthetic stabilized layer
5 layer 2 subgrade
6 soil
7 walkway loading

Figure 10 — Example of rectangular loaded area: tracked crane

6.4.3 BCR method for soft sand subgrade

The BCR method assumes that the static load transmitted by uniformly loaded circular or rectangular areas is distributed throughout the platform structure down to the loose sand layer. The insertion of stabilizing geosynthetics in the granular (aggregate) layer can increase the bearing capacity through a widening of the load distribution area. Experimental research has demonstrated that the insertion of stabilizing geosynthetics allows the load spreading angle to increase from a batter of approximately 2V/1H to a batter of approx. 1V/1H, as shown in Figure 11.

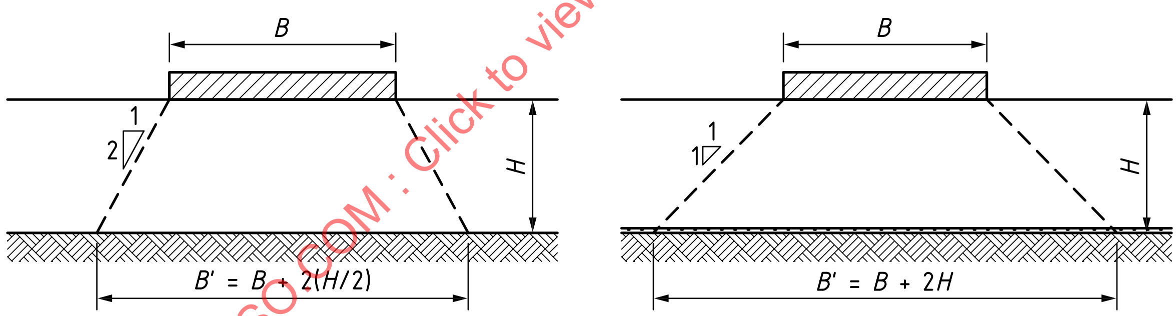


Figure 11 — The load distribution area widens when the load spreading angle increases from a batter of 2V/1H to a batter of 1V/1H

The stabilized granular (aggregate) layer is assumed to act as a semi-rigid foundation, whose bearing capacity can be calculated with the classical Terzaghi equation. The load spreading angle modifies the width *B'* and the length *L'* of the load area at the subgrade top surface. The bearing capacity can then be calculated using the well-known Terzaghi equation, like for a foundation having dimensions *B'* and *L'*:

$$q_{\text{lim}} = 1/2\gamma' B' N_\gamma s_\gamma + c' N_c s_c + q' N_q s_q \qquad (29)$$

where

N_q, N_c, N_γ are the bearing capacity factors;

s_q, s_c, s_γ are the shape coefficients (functions of the ratio B'/L').

NOTE For a circular or square loaded area, $B' = L'$, hence: $B'/L' = 1{,}0$.

Using the Terzaghi equation, the bearing capacity q_u and q_r can be calculated for the unstabilized and stabilized granular (aggregate) layer condition, for any value of the deck thicknesses h_u and h_r.

The FS can then be calculated as:

— unreinforced deck: $FS_u = q_u/\sigma_{vu}$

— reinforced deck: $FS_r = q_r/\sigma_{vr}$

The BCR can be calculated as:

$$BCR = FS_r / FS_u = (q_r / q_u) \cdot (\sigma_{vu} / \sigma_{vr}) \quad (30)$$

Optimal results are usually obtained for BCR = 1,60 – 1,75, affording FS_r= 2,0 – 3,0.

Further details can be found in Reference [30].

6.5 Railways

6.5.1 Network Rail (UK) design specifications

As reported by Das,[33] the use of geogrids is beginning to be incorporated into the railroad design codes of several countries, particularly in Europe. As an example, Table 1is a summary of the guidelines adopted by the Network Rail in the United Kingdom.

The philosophy of the Network Rail track bed design is based on achieving certain sleeper support stiffness targets for different lines [existing line or green field (virgin ground)]. The Network Rail track bed design is as shown in Table 1.

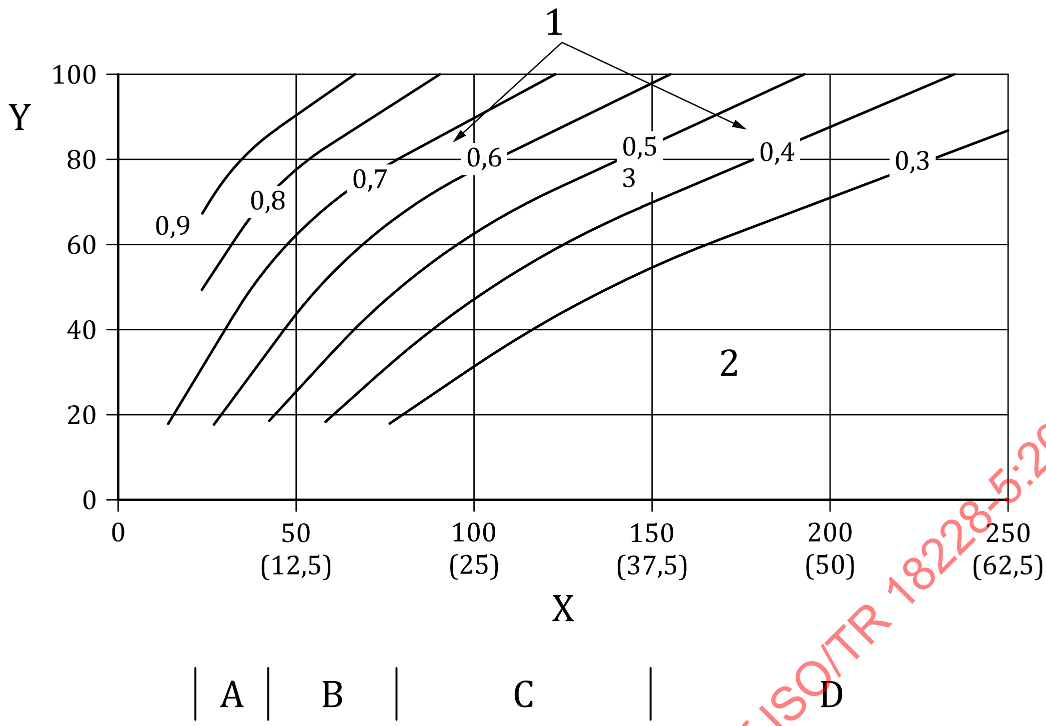
Table 1 — Dynamic sleeper support stiffness (*K*)

Site type		**Minimum dynamic sleeper support stiffness (*K*) – kN/mm/sleeper end**[b]
Existing lines, linespeed < 80 km/h (50 mph)		30 (2,0)
Existing lines, linespeed 80 km to 201 km/h (50 mph to 125 mph)		60 (1,0)[a]
Greenfield sites	Up to 160 km/h (100 mph)	60 (1,0)
	Above 160 km/h (100 mph)	100 (0,6)

[a] The required track bed performance can also be achieved on softer formations with the use of Geogrid reinforcement, provided that the dynamic sleeper support stiffness is at least 30 kN/mm/sleeper end.

[b] Figures in brackets in this column show FWD sleeper deflection in mm.

Based on the selected sleeper support stiffness and subgrade shear strength or modulus, a total thickness of granular layers can be determined using Figure 12.



Key

X	undrained shear strength - C_u (kN/m^2) (figures in brackets give approximate sub-grade modulus E values – MN/m^2)	A	soft clay
Y	dynamic sleeper support stiffness K – kN/mm/sleeper end	B	firm clay
1	total thickness of granular track bed layers (m)	C	stiff clay
2	note that this chart does not apply where drainage is poor	D	very stiff

Figure 12 — Determination of thickness of trackbed layers

6.5.2 Ev2 method

The design procedure is usually as follows:

— E_o modulus on subgrade is measured;

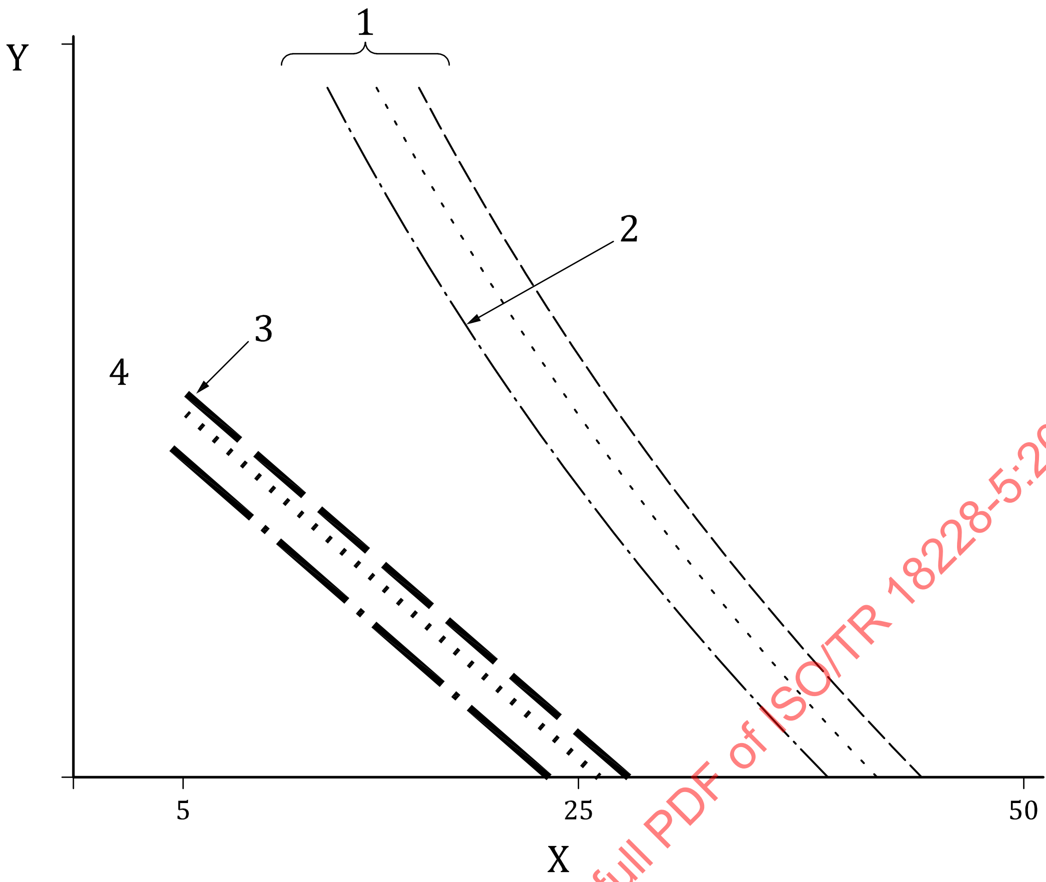
— E_c modulus on selected level of the construction according to the railway or road category is defined (standard range 20 MPa to 180 MPa);

— E modulus of fill material is normally known (typically 50 MPa to 90 MPa);

— h_s thickness of unstabilized layer (Figure 14) is derived from design chart in Figure 13;

— h_s thickness of stabilized layer (Figure 14) is derived from design chart in Figure 13; this is reasonably smaller but it is not generic and has to be defined by the manufacturer on the basis of independent research;

— construction layer h_c (Figure 14) is to be used, e.g. to achieve frost protection thickness.

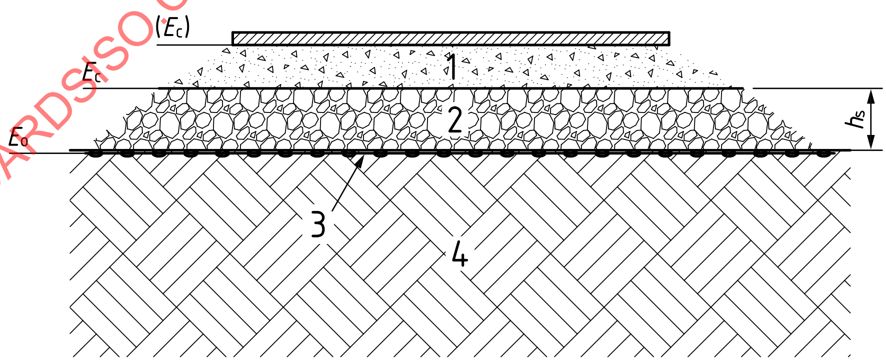


Key

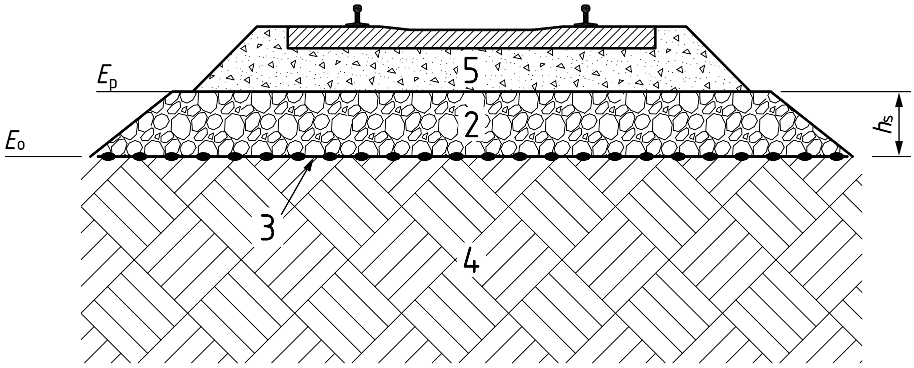
X modulus on subgrade – E_0 [MPa]
Y thickness of granular or stabilized layer - h_s [cm]
1 for various *E* (fill)
2 no stabilization
3 stabilization
4 design from case to case

NOTE The minimum *h=15 cm.*

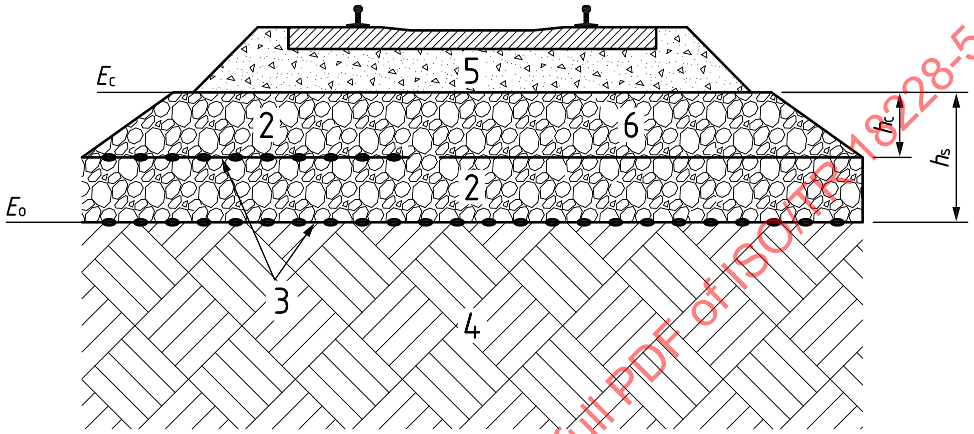
Figure 13 — Typical design chart for determination of h_s



a) Road subbase stabilization — Ev2 method



b) Railways subbase stabilization — Ev2 method



c) Multilayered railway or road structures — Ev2 method

Key

1 base
2 stabilized layer
3 geogrid
4 subsoil
5 ballast
6 construction layer

Figure 14 — Layers in Ev2 method

6.5.3 Railway ballast stabilization

There is no direct method for the design of stabilization of railway ballast. However, there are parameters that influence the design and empirical methods for the design of railway ballast using geosynthetics.

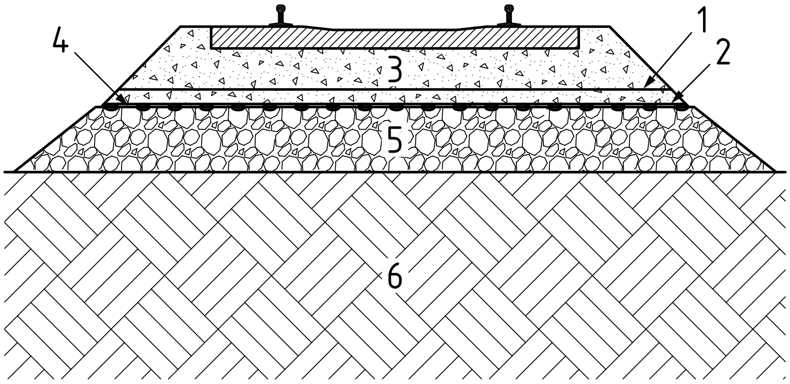
Design principles that can be applied are as follows:

— The service life of the ballast layer can be extended by several years due to restriction of particle displacement and therefore result in less abrasivity of grains.

— A geotextile or geogrid can be placed at the base of the ballast layer to restrict particle displacement.

— A geogrid with compatible aperture size is normally selected to suit the grading of the ballast to be used.

Increasing the thickness of the ballast layer by 5 cm to 10 cm (see Figure 15) avoids damaging the geogrid during tamping or clearing operations.



Key

1	normal base	4	geogrid
2	lowered base, 5 cm to 10 cm	5	subbase
3	ballast	6	subsoil

Figure 15 — Typical layout of railway ballast stabilization

6.5.4 Design method for railway stabilization with geocells

A comprehensive field case study conducted by the US Federal Railway Administration showed that geocell stabilization performance extended the maintenance cycles (see Reference [32]).

A primary contribution of geocells to a railway structure is the improved modulus (i.e. MIF) of the stabilized layer, and of layers above it, if they are present. This concept is detailed in 6.2.3.

Based on the MIF value, the geocell stabilization increases the overall stiffness of the sub-ballast structure. The stabilized sub-ballast layer can then be optimised in terms of reduced thickness, and the use of lower quality or marginal materials. The design criteria in such a structure can include the required E_{V2} modulus (modulus of subgrade reaction measured in a plate loading test) on top of the sub-ballast structure, according to railway standards (typically between 20 MPa and 100 MPa).

The different layer moduli can be defined considering that the modulus of each layer is dependent on the layer underneath it, as listed in Table 2.

Table 2 — Formulas for different layer moduli

Elasticity parameters		**Notes**
Base	$E_b = E_{ab} \times (1+0{,}006\,7 \times h_b\ [\text{mm}])$ $v_b = 0{,}35$	$E_b\ [MPa] \leq 700$
Sub-base	$E_{sb} = E_{sg} \times (1+0{,}003 \times \text{h}_{sb}[\text{mm}])$ $v_{sb} = 0{,}35$	$E_{sb}[\text{MPa}] \leq 300$
Subgrade	$E_{sg}[MPa] = 14 \times \text{CBR}\ [\%]$ $v_{sg} = 0{,}40$	$2 < \text{CBR}\ [\%] < 12$
SOURCE: Reference [33], reproduced with the permission of the authors.		

where

h_{sb}	is the subbase layer thickness [mm];
h_b	is the granular (aggregate) base layer thickness [mm];
E_{sg}	is the subgrade elastic modulus [MPa];
E_{sb}	is the subbase elastic modulus [MPa];
E_b	is the granular (aggregate) base elastic modulus [MPa];
ν_b, ν_{sb}, ν_{sg}	are the Poisson's ratio of base, subbase and subgrade, respectively.

After calculating E_{sg} and E_{sb} with the formulas in Table 2, it is possible to evaluate the MIF value (based on the specific data and parameters for different types of geocells). If the subbase is stabilized with geocells, the modulus of the stabilized subbase E_{sbs} is:

$$E_{\mathrm{sbs}} = E_{\mathrm{sb}} \cdot \mathrm{MIF} \tag{31}$$

Once the layer thicknesses and moduli of the unstabilized and geocell stabilized railway sub ballast have been set, the vertical settlement on the sub-ballast surface and the vertical stress on the subgrade surface below the centre of the sleeper can be calculated for both stabilized and unstabilized railway structures using standard stress and strain software for railway design.

The stress and strain characteristic of a railway substructure is dependent on the frequency and the size of the individual axle load applications. Accordingly, the loading on the subgrade is inversely proportional to the number of loading cycles raised to a power λ, according to the following formula:

$$\frac{\sigma_1}{\sigma_2} = \left(\frac{N_2}{N_1}\right)^{\lambda} \tag{32}$$

where

σ_1, σ_2	are the vertical stresses corresponding to N1, N2 loading cycles respectively;
λ	is the exponent with a mean value of 0,2.

If P denotes the load per axle and T denotes the daily traffic tonnage the equation above becomes:

$$\frac{\sigma_1}{\sigma_2} = \left(\frac{T_2/P_2}{T_1/P_1}\right)^{\lambda} \tag{33}$$

For constant axle loads, $P_1 = P_2$ and the equation above becomes:

$$\frac{\sigma_1}{\sigma_2} = \left(\frac{T_2}{T_1}\right)^{\lambda} \tag{34}$$

where T_1, T_2 are daily traffic tonnage corresponding to N_1, N_2 loading cycles respectively.

The above formulae allow calculation of the improvement in loading cycles and daily traffic tonnage that can be accommodated by the geocell stabilization design.

6.6 Design of the geosynthetics layout

The above illustrated design methods usually assume that the granular (aggregate) layer is stabilized with just one geosynthetic layer, but that the required geosynthetic layout is yet to be designed.

Geosynthetics provide the following stabilizing mechanisms:

— base course lateral restraint mechanism for horizontal stresses generated by the soil self-weight;

— base course lateral restraint mechanism for horizontal stresses generated by dynamic or cyclic loading (typically wheels).

Each of these mechanisms produce tensile forces in the geogrid layers.

The dynamic or cyclic effect of wheel loading is usually considered in the presently available methods (AASHTO method, Giroud-Han method, Leng-Gabr method, etc.). Therefore, the thickness of the base or subbase resulting from such methods is already appropriate for providing the structural capacity of the road to resist the design number of wheel passages for the whole design life of the road or railway.

When the applied load is static (like in the case of a working platform) or quasi-static (like in the case of a crawler crane), then the required platform thickness can be calculated with the Static method, the BCR method, etc., to prevent bearing failure.

Given the base, subbase or the platform thickness, by considering separately the effect of the static loads (applied static or quasi-static loads, soil self-weight and tensioned membrane mechanism) and the instant effect of wheel load, it is then possible to calculate the distribution of the horizontal tensile forces in the whole road structure. The overall tensile forces generated in each layer of geosynthetic can also be calculated and then an appropriate geosynthetic for each layer can be selected based on the limit strain criterion.

Rimoldi and Scotto[34] developed a model consisting of four soil layers and any number of geosynthetic layers for the design of planar 2D stabilizing geosynthetics (see Figure 16):

— asphalt course (AC);

— base course (BC);

— subbase course (SB);

— and subgrade (SG).

The model assumes that the load is applied as a uniform vertical pressure $\sigma_{v0} = p$ on a circular or rectangular area; this load spreads in the 3 layers of the platform structure (AC, BC and SB) according to their load spreading angles α_1, α_2, α_3.

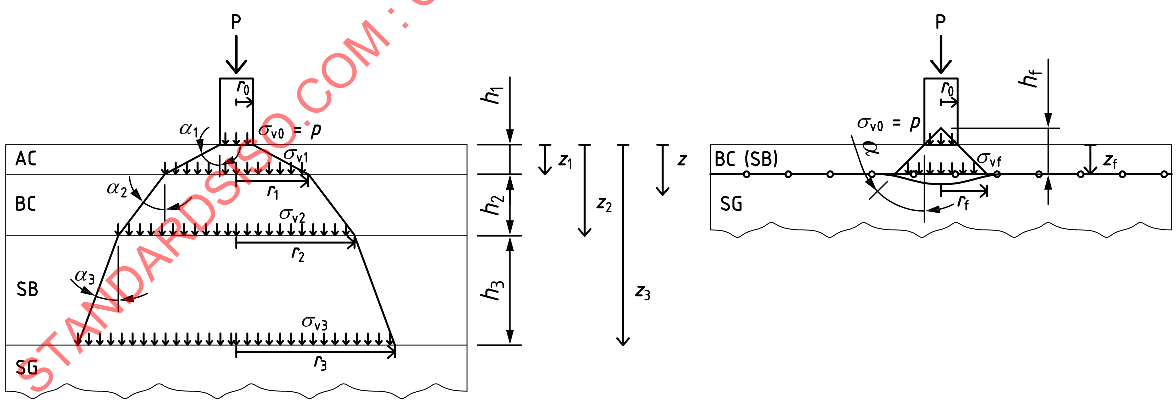


Figure 16 — General scheme of the 4-layers model and of the first geosynthetic layer placed at subgrade interface

The model allows the calculation of the tensile force in each geosynthetic, produced by the following mechanism:

— Force due to the horizontal soil trust:

$$T_{zi} = \int_{(i-1)-th\_GSY}^{i-th\_GSY} \sigma_{h-soil}(Z) \tag{35}$$

where

T_{zi} is the tensile force generated in the i-th geosynthetic by the soil thrust;

Z is the reference depth;

$\sigma_{h\text{-load}}$ is the horizontal stress produced soil trust.

— Force due to the horizontal stresses generated by the uniform circular or rectangular loads at the top surface:

$$T_{pi} = \int_{(i-1)-th\_GSY}^{i-th\_GSY} \sigma_{h-load}(Z) \tag{36}$$

where

T_{pi} is the tensile force generated in the i-th geosynthetic by the circular or rectangular uniform load at top;

Z is the reference depth;

$\sigma_{h\text{-load}}$ is the horizontal stress produced by the circular or rectangular uniform load at top.

— For forces due to membrane mechanism at the interface with the subgrade, the first geosynthetic layer (at the interface with the subgrade) is the most subject to out-of-the plane deformations due to the compaction of the first lift of soil on the soft subgrade, while the upper layers of geosynthetics are far less subjected to vertical displacements.

The tension in the reinforcement can be determined from the following equation:

$$T_m = W_{TCj} \cdot \Omega \cdot r_f \tag{37}$$

where

T_m is the tensioned membrane force in the geosynthetic;

Ω is the dimensionless factor of the tensioned membrane theory, function of geosynthetic strain;

W_{TC} is the uniform vertical load, which is a function of the load cone volume below the loaded area, fill density, wheel load, subgrade resilient modulus R, and the loaded area of the geosynthetic.

The total horizontal force that the i-th geosynthetic layer has to withstand is then:

$$T_{tot\text{-}i} = Tz_i + T_{pi} + T_m \tag{38}$$

where T_m applies only to the first geosynthetic layer at the interface with the subgrade, either of the base course or of the subbase course.

The i-th geosynthetic layer is normally able to provide a tensile force equal to or larger than $T_{tot\text{-}I}$ at the set maximum strain limit.

For important structures, the geosynthetic strain is usually limited to 1 % to 2 %, while for less important structures (or when the design conditions afford slightly larger deformations), a 3 %, 4 % or 5 % limit strain can be acceptable.

Further details and all formulas required for the design of planar 2D geosynthetics layout can be found in Reference [34] for circular loaded area and in Reference [29] for rectangular loaded area.

7 Materials

7.1 Properties of aggregate

An unbound granular (aggregate) layer that is composed of a well graded aggregate plays a structurally important role by providing load distribution. During the load distribution process, the resilient and permanent deformation response of the unbound aggregate base course is known to be influenced by many factors such as aggregate type and physical properties, density and moisture content, stress level (pneumatic, pad foot or steel drum compaction equipment) and grading and fines content (see Reference [35]).

7.2 Key properties for geosynthetics – Internal confinement

The precise quantification of individual properties of geosynthetics for inclusion in the design of an internally confined stabilized layer is not widely accepted. However, some properties have been identified which appear to be important in respect of geosynthetic performance in these systems and some which do not. In the case of properties that appear to be important, the precise quantification of the individual properties of the geosynthetic is required.

Ultimate tensile strength has not been found to be a predictor of performance for geosynthetics in road applications based on experience of full-scale trafficking trials at low deformation levels. Key factors for stabilization (internal confinement) are the long term tensile stiffness of the geosynthetic at low strain levels and degree and efficiency of the geosynthetic and aggregate interaction. Full scale trafficking trials have identified empirically derived parameters known as the traffic benefit ratio (TBR), the base course reduction factor (BCR) and the layer coefficient ration (LCR).

Trials used for calibration of TBR, BCR or LCR need to be as representative as possible of actual conditions in the field. Determination of the design parameters for a specific geosynthetic can be done using full-scale moving wheel tests which can be supplemented by cyclic plate loading tests. Also, documented case histories can provide valuable information complementing the data from full-scale tests, thereby contributing to the validation of the design parameters for a specific geosynthetic.

In order to provide lateral restraint or confinement to the aggregate particles, those geosynthetic elements (bars, ribs, bundles, fibres etc.), which are positioned crosswise or diagonal to the direction of particle displacement, are subjected to shear forces. In the case of geogrids, those shear forces are then transferred into adjacent elements capable of resisting these forces via shear at the element junctions. The strength and stiffness of the connections at the junction area depends upon the manufacturing technology used. The overall stiffness and integrity of the product needs to meet the requirements for the SLS of the stabilized construction as well as the ULS.

The magnitude of lateral restraint or confinement is related to the stiffness of the geosynthetic at low strain. As the quality of interaction between geosynthetic and soil plays a key role in the development of the confined zone, the soil and geosynthetic interaction is usually considered.

7.3 Key properties for geosynthetics – External confinement

The geometry and material properties of the geocell are key properties to maximize the effect of stabilization and to validate it for the design life of the project.

The key material properties of the geocell, related to hoop tension forces in the cell walls, are its elastic stiffness and resistance to permanent deformation. These properties contribute to the magnitude of the stabilization for the design life. Another key property is junction strength.

The stiffer the geocell, the higher the hoop tension stress can be and, thus, the higher the MIF. Material properties can be measured by the hoop tensile test (wide-width perforated cell wall and junction), dynamic mechanical analysis (see ISO 6721-1 and ASTM E2254) and accelerated creep test (ASTM D6992 modified for geocells). The creep test for geocells is normally performed with wide-width perforated cell walls at a fixed load that simulates the applied hoop tension forces according to the location of the geocell layer in the pavement structure. The improvement of the geocell-stabilized layer for the entire design life is dependent on the geocell material properties (tensile, dynamic stiffness and creep) to assure very low deformation of the confining cell walls, i.e., less than 2 % accumulated for the entire design life (a higher value can invalidate stabilization).

The geometry of the geocell including cell size, height, texture and perforation pattern affect the magnitude of stabilization. The size (diameter) of the cells affects the density of confinement. A small-diameter cell size with a higher cell wall (depth) provides a larger MIF than a large-diameter cell size with a lower cell wall height. The perforation pattern is normally distributed as uniformly as possible to maximize the soil-cell friction interaction, permitting water to flow while preventing particle migration between cells. The geocell panels are normally connected in such a way as to restrain lateral and node rotation of the connection.

The stiffness modulus is an indication of the relationship between the force exerted on a material and the associated elastic deformation. The elastic deformation behaviour is the main mechanical property for all types of pavement bases, from unbound to bound. The same principle applies to base stabilization with geocells. The net dynamic stiffness modulus (DMA-test) is determined by ISO 6721-1 or ASTM E2254.

Tensile dynamic loading on a geocell can result in a reduction in strength referred to as fatigue. Fatigue testing of materials is frequently investigated by applying high dynamic loads, establishing the relationship between the load and number of cycles and then extrapolating the number of cycles to failure (see Reference [40]).

Simulating elastic versus plastic behaviour is important to ensure performance in long-term applications. Polymers tend to lose elastic modulus (stiffness) over time, particularly under dynamic loading, and enter the plastic (viscous) range (see Reference [37]).

A geocell normally maintains stiffness under dynamic loading without significant permanent deformation or loss of geometry, which can result in a loss of confinement, invalidate design parameters or cause failure. In project design, the stiffer the geocell is, the less lateral and vertical movement there is, and therefore the compressive stress is increased. The dynamic elastic behaviour is relevant to the improvement of geocell performance as follows (see Reference [38]):

— wider elastic zone endures higher stress without permanent deformation;

— lower elastic deformation enables a higher modulus improvement factor (MIF) and lower settlements of confined infill soil;

— durability for cyclical loading provides resistance to polymeric fatigue.

8 Testing

8.1 General

Many parameters of geosynthetics, such as tensile behaviour or interaction, can be evaluated by laboratory tests, corresponding to so called index testing. However, these index tests cannot completely reflect the complex in-situ behaviour of the geosynthetic and soil system. Hence, when required, so called performance testing is normally carried out for the geosynthetics and for the interaction of geosynthetics and soil (large scale laboratory tests or in situ tests, or both).

The key properties of geosynthetics defined in this document are normally validated according to standard testing methods to ensure the design performance for the duration of the project design life.

8.2 Accelerated pavement tests (APT)

For paved roads, AASHTO R 50-09[43] references the GMA White Paper II[39] for specifics on the design process as it relates to the inclusion of geosynthetics in pavements. Benefits of pavement stabilization associated with life extension can be incorporated into an empirical design (e.g. Reference [43]) using a TBR or a BCR or an LCR which accurately represent the additional number of ESALs which can be carried by the stabilized pavement. Products submitted as equivalent normally have documented equivalent or better performance in pavement stabilization in full-scale accelerated pavement tests (APT), as well as results from completed project experience for the project conditions (base course material and thickness, failure criterion, subgrade strength, etc.). Results from field performance can include, but are not to be limited to, static and repetitive plate load testing to confirm material properties at the time of construction, as well as over the life of the pavement.

APT can be utilized to gather the data needed for developing the design inputs required for the stabilized layer (TBR, BCR or LCR) and verification of the ability of the stabilized layer to extend the life of a pavement. A minimum of three full-scale APT sections are usually constructed and tested following the protocol defined in Reference [40] or other appropriate guidance. The APT tests are usually planned to cover different ranges of subgrade strengths, variation of pavement and base thicknesses, different type of stabilizing geosynthetics, and their location within the pavement section. Further, the test sections can be designed and constructed such that a minimum of 200,000 ESALs can be achieved with a target permanent deformation of 12,5 mm or 25 mm (according to the national design standards) during the APT program.

An unstabilized control section is usually also included for comparison and calibration purposes. If the control section, however, does not have the same layer thickness or modulus as the geosynthetic stabilized pavement, parameters for use in design can be addressed through the computation of an effective base coefficient as described in Reference [41] or through mechanistic analysis. The effective base coefficient is used to determine the TBR, BCR or LCR for the specific conditions of the test. The results of such testing, as reported within References [39] and [41] demonstrate that TBR, BCR, and LCR are not fixed numbers. Therefore, engineering judgement is applied when utilizing these values in design.

8.3 Tests for geotextiles and geogrids

Typical tests that can be undertaken for both geotextiles and geogrids can include (but are not limited to):

- short term tensile strength (see ISO 10319), for identification purposes only;
- tensile creep (see ISO 13431 and ASTM-D6992), only when considering permanent static loads;
- tensile stiffness (see ISO 10319) at strains appropriate to the stabilization function;
- installation damage resistance (see EN ISO 10722);
- coefficients of friction and interaction between geosynthetics and soils (see ISO 12957-1);
- durability;
- UV resistance (see EN 12224) to determine the maximum exposure time before covering the geosynthetic with soil;
- chemical durability in contact with soils and ground water (see EN 12447 or ISO 13438).

Additionally, for geogrids:

- dimensions of the geogrid aperture related to the fill particle size distribution;
- junction strength or efficiency (see ASTM D7737).

8.4 Tests for geocells

Typical tests that can be undertaken for geocells can include (but are not limited to):

- geometry of geocells: determining the cell height and distance between internal junctions;
- single strip tensile strength: testing of strength at yield of the full width of a perforated or non-perforated cell wall (see ISO 10319);
- junction strength (see ISO 13426-1, Method C) for resistance to relative movement, node rotation, lateral and vertical movement;