ISO/TC 221/WG 6
Design using geosynthetics

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ISO TC221 WG6 N314 PG5 Stabilisation TR 18228-5

Replaces:	N 290
Document type:	Working draft
Date of document:	2019-10-03
Expected action:	INFO
Background:	This version of TR 18228-5 takes account of all of the changes that were discussed and agreed during the ISO TC221 WG6 PG5 meeting in Tel Aviv. Some significant changes have been made to the document as a result of rewrites to many sections. All members are therefore requested to review this document carefully and provide comments to the PG5 Leader, Derek Smith on the bottom of the comments sheet provided at N315. Note that N315 contains a number of comments that have not yet been addressed from the Tel Aviv meeting and still need to be addressed. All comments received by 8th November 2019 will be collated and circulated prior to the meeting of PG5 in Beijing, where the comments will be reviewed.
Committee URL:	https://isotc.iso.org/livelink/livelink/open/tc221wg6

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Date: 3rd October 2019

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Design Using Geosynthetics: Part 5 – Stabilisation

Conception utilisant géosynthétiques - Partie 5: Stabilisation

WD stage

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Published in Switzerland.

Contents

Foreword

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The committee responsible for this document is ISO/TC 221, Geosynthetics, WG 6, Design using geosynthetics.

ISO/TR 18228 consists of the following parts.

- Part 1: General
- Part 2: Separation
- Part 3: Filtration
- Part 4: Drainage
- Part 5: Stabilisation
- Part 6: Protection
- Part 7: Reinforcement
- Part 8: Surface Erosion Control
- Part 9: Barriers
- Part 10: Asphalt Pavements

Introduction

ISO/TR 18228 *Design using geosynthetics* is a Technical Report containing guidance for designs using geosynthetics for soils and below ground structures in contact with natural soils, fills and asphalt and/or other materials. The Technical Report is in 10 separate parts. Part 1 contains general guidance relating to the 10 separate parts. Parts 2 to 10 relate to 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.

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.

TR 18228-5 includes information relating to the stabilisation function.

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

The first mechanism provides stabilisation 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 aggregate layer, thereby controlling deformation under load (Serviceability Limit State - SLS).

The second mechanism provides reinforcement by way of a geosynthetic material anchored each side of a loaded area by friction and/or interlock, deforming out of the plane under load to create a tensioned membrane. In doing so it provides support to the aggregate layer, thus decreasing deformations (SLS) and increasing bearing capacity (Ultimate Limit State - ULS).

1 Scope

This part of the Technical Report summarises guidance, from a number of documents, for the design of geosynthetics to fulfil the function of stabilisation of soils in contact with natural soils, fills, asphalt and / or other materials.

The concepts of the presented guidance are always based on installed soils, the installation process and on the strength and/or deformation behaviour of geosynthetics.

This document contains guidance for the design of unbound layers of paved and unpaved roads, working platforms and foundations utilizing the stabilisation function of geosynthetics. This is typically for Serviceability Limit State (SLS). Ultimate limit state (ULS) design, as required for some applications, e.g. slab foundation design, working platform design etc., may need to be proven separately. This technical report is a state of practice report and guidelines are 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 separate Technical Reports.

2 Terms and definitions

For the purposes of this document, the terms and definitions given in EN ISO 10318-1 and the symbols and pictograms in EN ISO 10318-2 apply.

This section to be completed once remainder of document is finalised.

surface resilient modulus?

Equivalent Standard Axel Loads (ESAL's): Of 80kN on single tyres.

3 Concepts and Fundamental Principles

3.1 General

As stated in the introduction there are two primary mechanisms by which geosynthetics can improve the performance of a granular layer, the confinement mechanism and the tensioned membrane mechanism. Figure 1 illustrates the difference between the two mechanisms.

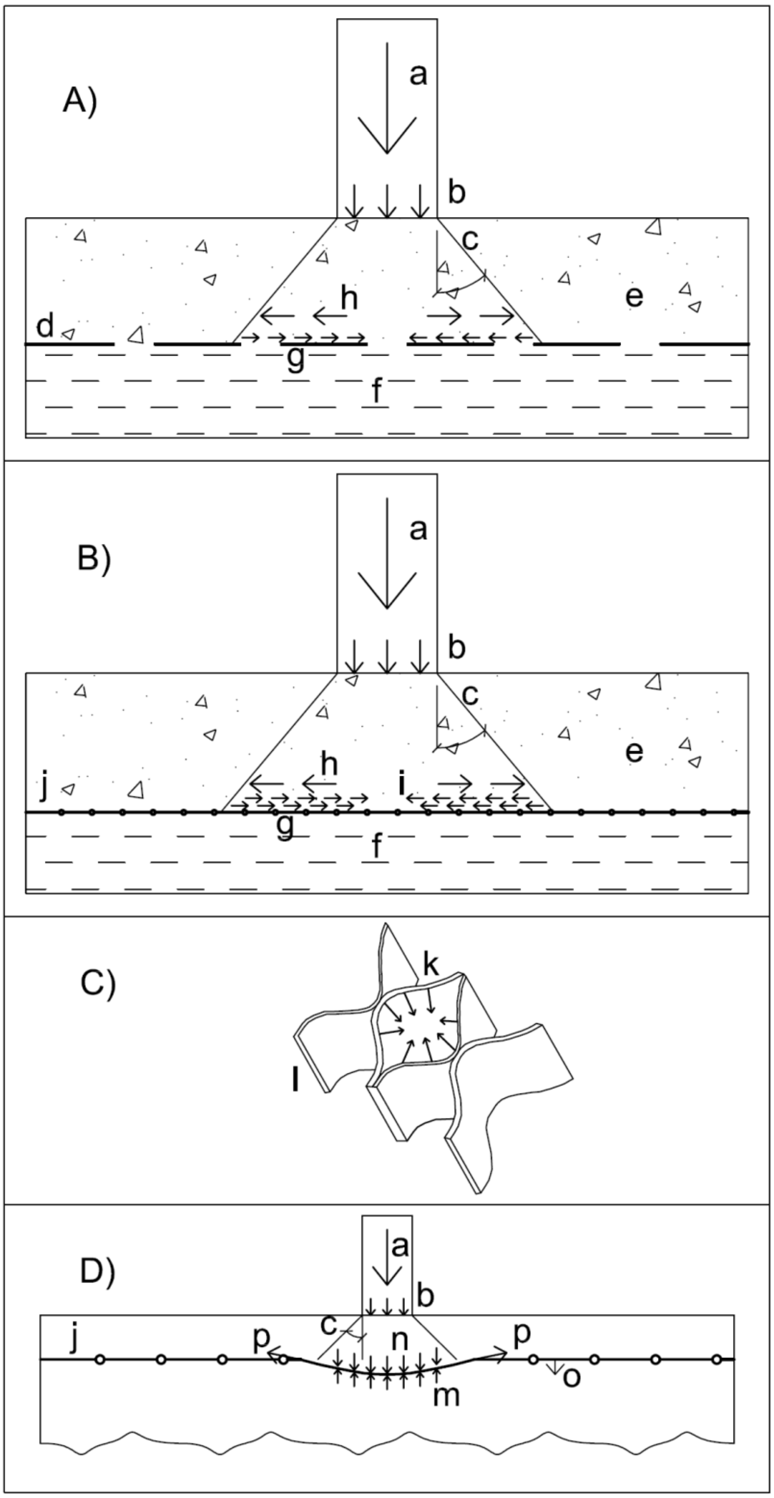
Figure 1: Primary mechanisms by which geosynthetics can improve the performance of a granular layer

A) confinement by direct shear only;

B) confinement by direct shear and interlocking;

C) confinement by 3D structure; and

D) tensioned membrane mechanism



In stabilisation (confinement), the geosynthetic operates most effectively at relatively low levels of strain. Stabilisation will be less influential in designs where high levels of strain are anticipated. Where high levels of strain are anticipated, the tension membrane effect (reinforcement) will be dominant.

There is considerable discussion in the literature about the relative magnitude of the strain within the geosynthetic in the stabilisation and reinforcement mechanisms. The boundary between the operational strain envelopes of these two mechanisms and indeed the nature of any transition between them has not been defined adequately 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.

Following from the above, the design life of the project is also suggested as a key consideration and, as a result, the rate of deformation. Designing for geosynthetic stabilisation 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 may require large deformations to mobilise the tensile strength of the geosynthetic for it to be effective. This depends on the tensile stiffness of the geosynthetic. As such, the geosynthetic performs a reinforcement function. Reinforcement is covered in ISO TR18228-7. However, for ease of user reference, a discussion of the tensioned membrane reinforcement mechanism is provided as Annex A to this document.

3.2 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 reference section of this report. In these cases, the geosynthetic and aggregate together form a stabilised layer.

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

(1) effectively withstand service-loading pressures,
(2) control subgrade and unbound aggregate layer deformation within a range suited to the in-service requirements, and
(3) do so without progressively deteriorating over time through either aggregate deformation, breakdown and/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.

3.3 Confinement / Restraint

Stabilisation by geosynthetics requires the minimisation of particle movement through confinement. Minimisation of particle movement is achieved by particle restraint. In order for geosynthetics to provide particle restraint, they would normally have adequate tensile stiffness and sufficient interaction with the soil.

Confinement is the dominant stabilisation mechanism at low levels of strain (typically less than 1%, but possibly up to 2%) of the geosynthetic within the granular material, depending also on the importance of the stabilised system (e.g. highway versus haul road) and the position of the geosynthetic within the stabilised system (usually at lower levels higher strains are acceptable). If strain values are expected to be above this, then 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.

3.3.1 Internal Confinement – Description of Mechanism

Internal confinement is the intimate interaction of a bi-dimensional geosynthetic with aggregate in a compacted granular layer thereby creating a pseudo-composite material of improved shear strength and stiffness. The interaction can occur via interlock and / or surface friction. For interlock to be effective the geosynthetic is required to 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 physical restraint of the aggregate particles. The stiffness provided by the geosynthetic reduces development of lateral tensile strain and stress in the base aggregate over a defined height above the geosynthetic - the stabilised 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 layer from the geosynthetic (unconfined zone), Figure 2 illustrates the various zones for (A) a planar two-dimensional geotextile without apertures; (B) a planar two-dimensional geogrid with apertures; and (C) a three dimensional geosynthetic.

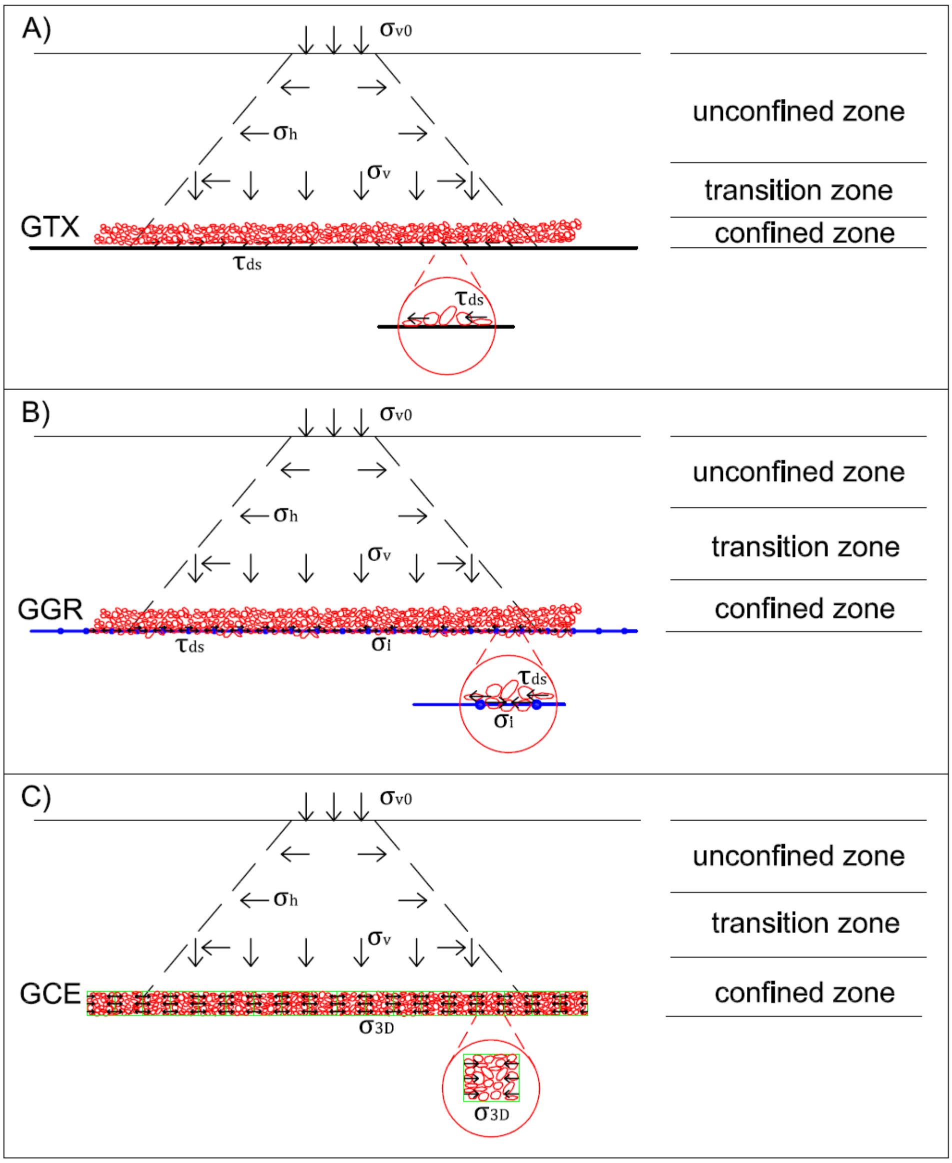
The efficiency of confinement and thicknesses of the confined and transition zones varies with different geogrids and soil types. The details therefore have to be defined for each type of geogrid individually.

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/geosynthetic interaction.

During the application of load to the granular layer (e.g. trafficking or compaction) the interaction discussed above distributes stress throughout the stabilised granular 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 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 stabilised granular layer is to limit any stress and strain transmission to this weaker layer.

Figure 2: Zones of confinement



3.3.2 External Confinement - Description of Mechanism

External confinement is when a certain volume of material is confined by a three-dimensional geosynthetic system. A three-dimensional geosynthetic system could be formed by any combination of geosynthetics that act together to effectively perform as a single system. The remainder of this section refers specifically to three-dimensional geosynthetic systems that are known as geocells.

The principle geocell stabilisation mechanism limits horizontal infill soil deformation via the geocell walls thereby confining the infill soil. The limitation of horizontal deformation is based on three factors:

a) hoop tension forces in the cell walls,

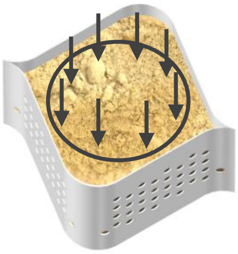
b) resistance from the surrounding cells, and

c) friction between cell walls and infill material

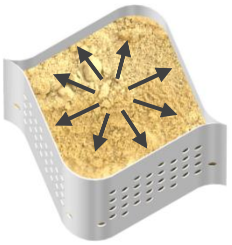
Under vertical loads, horizontal earth pressure is restrained by cell walls, activating hoop tension forces (Figure 3). The resulting strains in the cell wall mobilise hoop stresses within the loaded cell. The magnitude of the activated hoop stress depends on the geocell material, stress-strain behaviour and load level, number of load cycles, the location of the applied load, the type of infill material, and the foundation characteristics.

The hoop stresses and resistance provided by surrounding stabilised cells restrict lateral deformation of the fill. This mechanism increases the layer stiffness when compared to non-stabilised soil, which acts like a semi-rigid slab with increased load spread angle and improved bearing capacity. Vertical stress and vertical deformation on top of the subgrade are reduced.

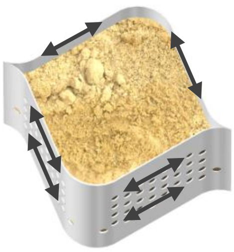
Figure 3. Confinement Mechanisms in Geocells: Development of Cell Hoop Stress by External Vertical Stress



1) External vertical stress



2) Lateral stress on cell walls



3) Lateral confinement / restraint - Hoop tensile stress in cell walls

3.4 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 (shear stress), which finally leads to lateral spreading (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, decrease of bearing capacity).

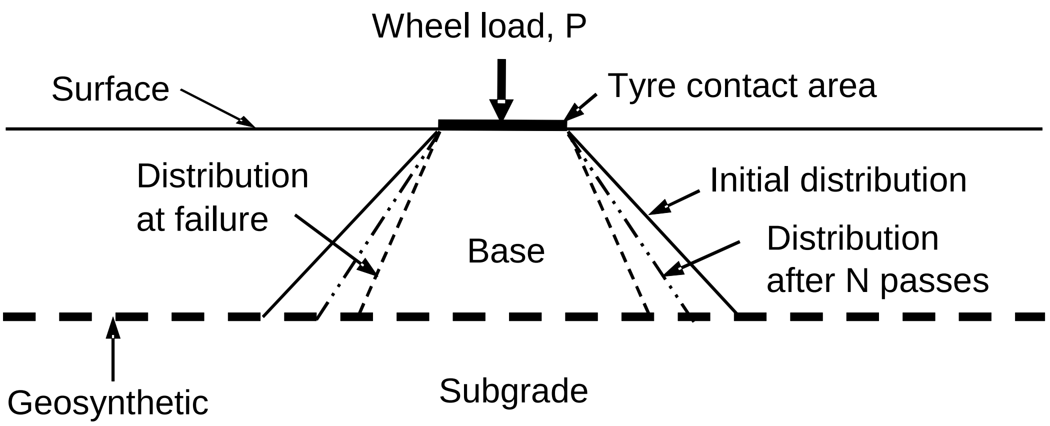
When unbound aggregate is installed and compacted on top of geosynthetics, shear resistance is generated between the two components because of frictional interaction. Other modes of interaction can be based on aperture interlocking (for geogrids) or cell / confinement fixation (for geocells). In such 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 and 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 may be put into tension which causes development of corresponding strain and stress. The amount of strain depends on several factors; main factors are 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.

3.5 Loading Conditions (Cyclic / 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 4), 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/subgrade interface tends to increase over time.

Figure 4. Stress distribution angle (Giroud and Han, 2011)



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 (Giroud and Han, 2011). 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 (Giroud and Han, 2011).

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

3.6 Multi component systems

Multi-component geosynthetic systems typically increase the thickness of the stabilised layer and provide improved confinement. As a result, a multi-component geosynthetic system would normally result 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 and/or external confinement within a single system.

4 Typical Applications

4.1 Stabilised Granular Layers in Trafficked Areas

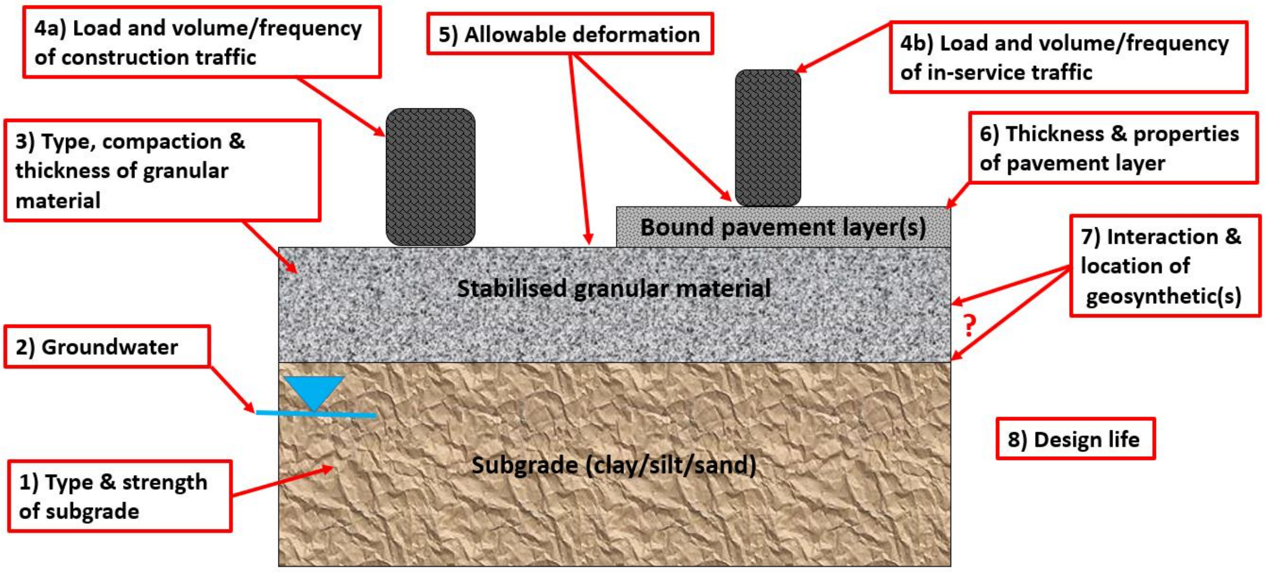
Typically, there are three principle design scenarios when considering the use of stabilised granular layers in dynamically loaded trafficked areas and highways.

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

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

C. As B above but the influence of the stabilisation of the granular layer is included in the analysis of the overall pavement performance and its ability to carry the in-service traffic loads.

Figure 5. Stabilised Granular Layer – Key factors influencing design & performance in trafficked areas: Scenarios A, B & C – Trafficking on unbound & overlying bound pavement layers



4.2 Stabilised Granular Layers in Railways

Typically, there are five principle design scenarios when considering the use of stabilised granular layers in dynamically loaded rail track-bed applications.

D. Stabilised sub-ballast or foundation layers to be designed to carry the loads from construction vehicles and acts as a working platform for placement of overlying ballast or bound layer which supports the sleepers & rails and accommodates in service rails loads

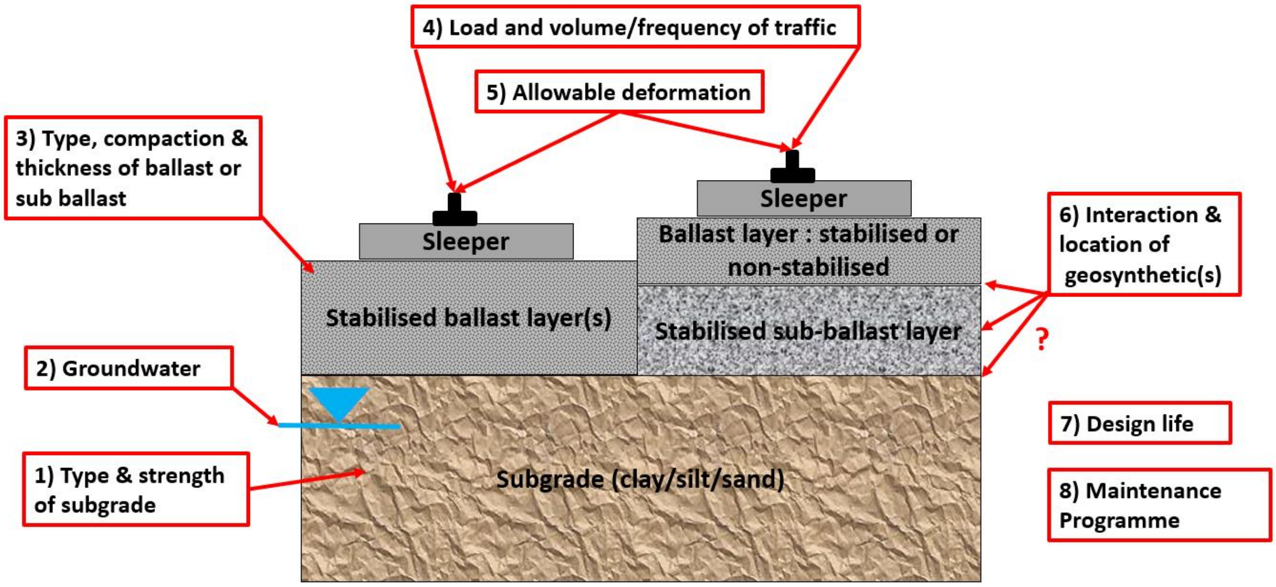
E. As above but the influence of the stabilisation is included in the analysis of the overall rail track bed performance and its ability to carry the in-service traffic loads

F. Stabilised ballast layer directly constructed on subgrade and therefore designed to accommodate construction traffic and in-service rail loads.

G. Both the sub-ballast layer and the bound/ballast layer are stabilised.

H. Stabilisation of the ballast layer for the purpose of reducing ballast particle movement under service conditions, but with no load carrying benefit assumed.

Figure 6. Stabilised Granular Layer – Key factors influencing performance in rail track bed: Scenario D, E, F & G – Stabilised Sub-ballast and/or ballast layer



4.3 Stabilised Granular Layers in Working Platforms

Typically, the types of loading to be accommodated when considering the use of stabilised granular layers in stabilised working platforms are as follows: -

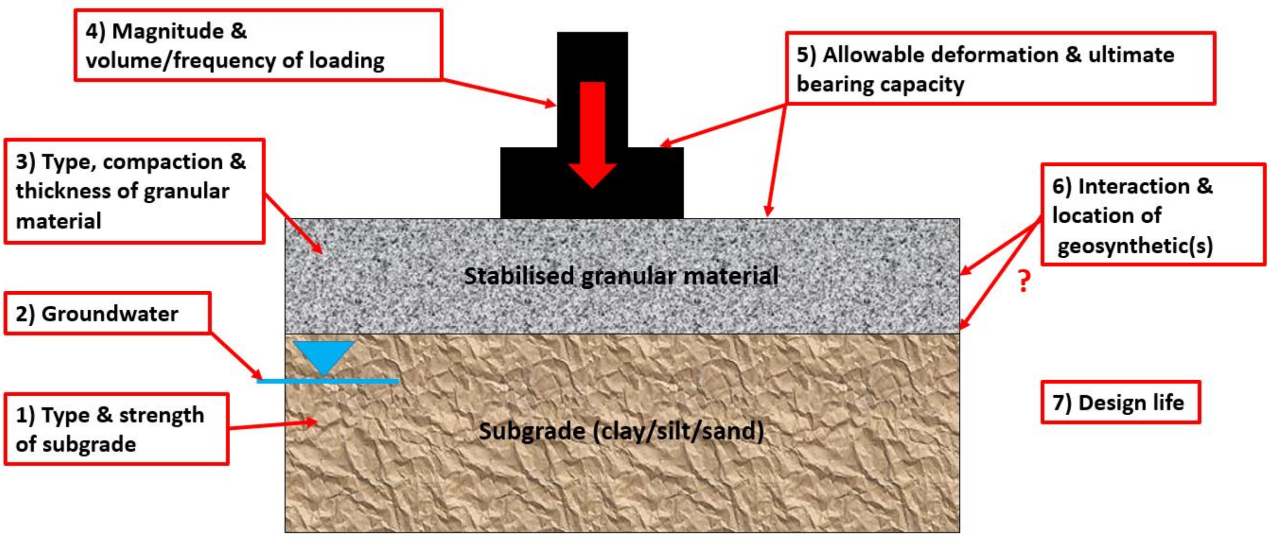
I. Dynamic loads associated with construction traffic as previous scenarios.

J. 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).

K. 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.

The design for slab foundation and / or working platform is normally proven at the Ultimate Limit State (ULS) in terms of both local and global stability.

Figure 7. Stabilised Granular Layer - Factors influencing performance in working platforms: Scenario H– High magnitude pseudo static loads



5.0 Principles of Design

5.1 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 provide standard thicknesses of various layers of aggregate. Others provide empirical or semi-empirical design rules and equations.

As yet, it is impossible to include a generic product characteristic of a geosynthetic in most of methods to quantify the stabilising effect to the construction. Geosynthetics have been included in some designs to provide an empirically obtained improvement of the system parameters, e.g. improvement of modulus. Evidence of the improvement in system parameters may also alternatively be provided by field and full-scale laboratory testing of the specific product in the appropriate aggregate and loading for the application.

The remainder of this section provides details of some design methods that are adopted in various applications.

5.2 Unpaved roads

5.2.1 Giroud- Han (2004) Method

The Giroud& Han (2004) 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 layers. The method requires calibration in the case of geogrids, whereas no calibration is needed in the case of geotextiles. It is important to note that, for valid use and to ensure reliable results, the Giroud- Han method requires calibration for each specific type of geogrid under consideration. It has been calibrated only for geogrids using the aperture stability modulus (measured by ASTM D7864) as the characteristic property of the geogrids. Other properties, or group 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 performance problems on projects designed using this method. The Giroud- Han method has recently been added to the FHWA (US Federal Highway Administration) Geosynthetics Training Manual as a recommended design method. The Giroud- Han method 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 layer:

$$h = \frac{0.868 + (0.661 - 1.006J^2)\left(\frac{r}{h}\right)^{1.5} \log N}{\left[1 + 0.204(R_E - 1)\right]} \left(\sqrt{\frac{P/\pi r^2}{\frac{s}{f_s}\left[1 - 0.9e^{-\left\{\frac{r}{h}\right\}^2}\right] N_c f_c s_u}} \right) r \qquad (1)$$

h = aggregate base thickness (m)
J = aperture stability modulus (N-m/deg)
P = wheel load (kN)
r = radius of tire footprint (m)
N = number of ESAL (Equivalent Standard Axle Load of 80 kN on single tyres)
N_c = bearing capacity factor
= 3.14 (unstabilized)
= 5.14 (geotextile)
= 5.71 (geogrid)
s = rut depth (m)
f_s = rut depth factor (0.075 m)
s_u = subgrade undrained shear strength (kPa) = 30·CBR

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

The original Giroud - Han method, to which the above reported equation refers, has been developed for geotextiles and biaxial extruded geogrids; hence the method requires calibration in the case of extension to other types of stabilizing geosynthetics.

Limits and applicability of the Giroud - Han method in the papers reported in References (in particular: Giroud – Han 2004, 2006, 2011, 2012; Han et al, 2011).

5.2.2 Leng – Gabr (2006) Method

Leng and Gabr (2006) presented a method for the design of stabilized unpaved roads, which shows some affinity with Giroud – Han method, but also important differences in the theoretical approach and in the parameter used to characterize the effect of stabilizing geosynthetics.

In fact, the method is based on a simplified analysis of vertical stresses diffusion, on the correlation between the degree of bearing capacity mobilization, and on the elastic secant modulus of stabilizing geosynthetics at 2 % strain (measured by 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 performance problems on projects designed using this method reported so far.

On the base 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 increasing number of passages. Based on the above-mentioned experimental tests and on empirical criteria, geogrid influence is introduced through the tensile strength at 2 % elongation $T_{2\%}$ (measured by ISO 10319). If $T_{2\%}$ is different between longitudinal and transversal directions, the average value is assumed.

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

$$h = \frac{0.85 \quad a \left(1 + k_2 \log N\right)}{\tan \alpha_1} \left(\sqrt{\frac{p}{m N_c C_u}} - 1 \right) \tag{2}$$

With:

$$K_2 = (a/h)^{0.81} \mathrm{Max}[(0.58 - 0.000046\ T_{2\%}{}^{4.5}), 0.15] \tag{3}$$

$$m = \left[1 - e^{-0.78(a/h)}\right] \frac{r}{r_{cr}} \leq 1 \tag{4}$$

$$r_{cr} = 0.025 \cdot (0.125 \log N + 1.5) \tag{5}$$

where:
h = aggregate base thickness (m)
J = aperture stability modulus (N-m/deg)
p = tyre pressure (kPa)
a = radius of tire footprint (m)
N = number of standard 80 kN axle passes
N_c = subgrade bearing capacity Factor
= 3.80 (unstabilized base)
= 5.69 (geotextile stabilized base)
= 6.04 (geogrid stabilized base)
C_u = subgrade undrained shear strength (kPa)
α_1 = initial load spreading angle (for N = 1)
k_2 = coefficient defining the degradation of the load spreading angle with increasing number of passages.
$T_{2\%}$ = tensile strength of the stabilizing geosynthetic at 2 % strain (kN/m)
r = rut depth (m)
r_{cr} = 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 must be therefore solved by iterations.

Limits and applicability of the Leng - Gabr method can be found in Leng&Gabr (2006), reported in References.

5.2.3 Pokharel (2010) method for geocells

Based on studies of geocell stabilisation 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 (2010) modified the Giroud and Han (2004) design methodology for geosynthetic stabilization of unpaved roads by changing planar 2D geosynthetic dependent parameters (such as aperture modulus) to geocell dependent parameters. These parameters were calibrated by laboratory cyclic plate loading tests and full-scale moving wheel tests on NPA (Novel Polymeric Alloy in a Nano matrix) based geocell stabilized granular bases over weak subgrade. In the design methodology a maximum allowable rutting is set (together with all other parameters), and the pavement thickness is determined by:

$$h = \frac{\left(0.868 + 0.52\left[\frac{r}{h}\right]^{1.5} \log N\right)}{\{1 + 0.204(R_E - 1)\}} \times \left(\sqrt{\frac{P}{\pi r^2 m 5.14 c_u}} - 1\right) r \tag{6}$$

Where:
h = required base course thickness (m)
r = radius of tire contact area (m)
N = number of equivalent standard axle load (ESAL)
P = wheel load (kN)
c_u = undrained cohesion of the subgrade soil (kPa)
R_E = modulus ratio of base course to subgrade soil
m = bearing capacity mobilization factor

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 laboratory was observed by Gabr (2001) for geogrid stabilisation. Recent research showed that NPA geocells significantly slowed down the rate of deterioration in the base quality. This phenomenon is attributed to 3D geocell confinement to increase and maintain the modulus of the base course.

The Modulus Improvement Factor I_f was proposed by Han et al (2007) to account for this benefit:

$$I_f = E_{bcs} / E_{bcu} \tag{7}$$

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:

$$R_E = I_f \cdot (E_{bcu} / E_{sg}) = \max(7.6;\ I_f \cdot [(3.48\ CBR_{bc}^{0.3}) / CBR_{sg}]) \tag{8}$$

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

Han et al (2207) reported that the modulus ratio with geocell stabilization ranges between 4.8 and 10.0. Anyway, based on cyclic plate loading tests and accelerated moving wheel tests, Pokharel suggests to limit the value of R_E to 7.6 for NPA geocell stabilized bases.

The bearing capacity mobilization coefficient (m) is given by:

$$m = \left(\frac{s}{f_s}\right)\left\{1 - 0.9\exp\left[-\left(\frac{r}{h}\right)^2\right]\right\} = \left(\frac{s}{75mm}\right)\left\{1 - 0.9\exp\left[-\left(\frac{r}{h}\right)^2\right]\right\} \quad (9)$$

where
s = limit rut depth at the top of the stabilized base course (mm)
f_s = rut factor = 75 mm.
Since usually a separation nonwoven geotextile is placed below the geocells, the bearing capacity factor N_c for geocell stabilized unpaved road base is assumed, according to Giroud& Han (2004) as:

$$N_c = 5.14 \text{ (geotextile)} \quad (10)$$

The factor used by Giroud – Han (2004) to control the rate of reduction in the load distribution angle, which depends on the aperture stability modulus J for geogrids, for NPA-based geocell stabilization have been replaced by the term:

$$\left(0.868 + 0.52\left[\frac{r}{h}\right]^{1.5} \log N\right) \quad (11)$$

The parameters of the Pokharel method, including MIF, R_E, rate of reduction in the load spreading angle, bearing capacity, and others, have been calibrated for NPA geocells (Pokharel, 2010; Pokharel et al, 2011; Pokharel et al, 2015; Pokharel et al, 2016).

The extension of the method to other types of geocells requires different parameters, which can be calibrated by laboratory cyclic plate loading tests and full-scale moving wheel tests on the specific geocells.

5.3 Paved roads

5.3.1 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 AASHTO Guide for Design of Pavement Structures (AASHTO 1993), which has been aligned with the requirements of the AASHTO standard practice document R50-09 (AASHTO 2013). The AASHTO design method was developed in the early sixties based on the AASHO Road Test and later revised multiple times until the final version in 1993.

AASHTO R 50-09 reports that an engineer may want to develop an approved list of products that are considered appropriate for a particular pavement application, based on successful past applications and long-term performance (AASHTO 2010). The AASHTO R50-09 document references the GMA White

Paper II (GMA 2000) for specifics on the design process as it relates to the inclusion of geosynthetics in pavements. Benefits of pavement stabilisation associated with life extension can be incorporated into an empirical design (e.g., AASHTO '93) using a Traffic benefit ratio (TBR) or a modified layer coefficient ratio (LCR) which accurately represent the additional number of ESAL's which can be carried by the stabilised pavement. Products submitted as equivalent should have documented equivalent or better performance in pavement stabilisation 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 readings for international roughness index (IRI) and pavement condition index (PCI) over the life of the pavement.

Accelerated Pavement Testing (APT) should be the primary means utilised to gather the data needed for developing the design inputs required for the stabilised layer (TBR or LCR) and verification of the ability of the stabilised layer to extend the life of a pavement. A minimum of three full-scale accelerated pavement test (APT) sections should be constructed and tested following the protocol defined in NCHRP Report 512 "Accelerated Pavement Testing: Data Guidelines" (Saeed and Hall, 2003) or other appropriate guidance. The APT tests should be planned to cover different ranges of subgrade strengths, variation of pavement and base thicknesses, different type of geogrids, and their location within the pavement section. Further, the test sections should be designed and constructed such that a minimum of 200,000 Equivalent Standard Axel Loads (ESAL's) can be achieved with a permanent deformation of 12.5mm or less during the APT program.

A non-stabilized control section should also be included for comparison and calibration purposes. If the control section, however, does not have the same layer thickness and/or modulus as the geogrid stabilized pavement, parameters for use in design may be addressed through the computation of an effective base coefficient as described by Tingle, et. al. (2017) or through mechanistic analysis. The effective base coefficient would be used to determine the LCR for the specific conditions of the test. The results of such testing as reported within the GMA White Paper II (GMA 2000) and Tingle, et. al. (2017) demonstrate that both the TBR and LCR are not fixed numbers. Therefore, engineering judgement must be applied when utilizing these values in design.

The AASHTO method uses empirical equations based on multilayer elastic theory to model the pavement cross section. The structure is characterized 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:

$$logW_{18} = Z_R S_0 + 9.36\log(Sn+1) - 0.2 + \frac{\log\left(\frac{\Delta PSI}{2.7}\right)}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 logM_R - 8.07 \tag{12}$$

where W_{18} = number of 80 kN equivalent single axle load (ESAL) applications over the design life of the pavement, Z_R = standard normal deviate, S_0 = standard deviation, ΔPSI = allowable loss in Present Serviceability Index and M_R = Resilient modulus of the subgrade.

Once the required SN is determined, the pavement structure can be designed through a series of iterations solving Eq. (13):

$$SN = a_1 \cdot D_1 + a_2 \cdot D_2 \cdot m_2 + a_3 \cdot D_3 \cdot m_3 \tag{13}$$

where a_1, a_2 and a_3 = layer coefficients of surface, base and subbase respectively (relative strength of the layer), D_1, D_2 and D_3 = thicknesses of surface, base and subbase layers respectively, m_2 and m_3 = drainage coefficients of base and subbase.

According to the GMA White paper (Berg et Al., 2000), 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 Bearing Ratio (TBR): ratio of number of load applications of the geosynthetic stabilized section over the number of load applications of the unstabilised 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 unstabilised one.
3) Layer Coefficient Ratio (LCR): ratio of the layer coefficient of the geosynthetic stabilized base layer over the layer coefficient of the unstabilised one.

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

When designing a geosynthetic stabilised pavement structure, one of the first steps AASHTO R50-09 suggests doing 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.

When using the TBR, Eq. (12) can be modified as shown in Eq. (14) to evaluate the increased total number of ESAL due to the beneficial effect of the geosynthetics:

$$W_{18,\, stabilized} = TBR \cdot W_{18,\, unstabilised} \tag{14}$$

Typical TBR values range from 1.5 to 10 for geotextiles and 1.5 to 70 for geogrids (Shukla, 2002).

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:

$$BCR = 1 - \frac{D_{stabilized}}{D_{unstabilized}} \tag{15}$$

Hence Eq. (12) can be modified as:

$$SN = a_1 \cdot D_1 + \frac{1}{1 - BCR_2} D_{2,BCR} \cdot a_2 \cdot m_2 + \frac{1}{1 - BCR_3} D_{3,BCR} \cdot a_3 \cdot m_3 \tag{16}$$

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

$$D_{2,BCR} = \frac{SN - a_1 \cdot D_1 - a_3 \cdot D_{3,BCR} \cdot m_3}{BCR_2 \cdot a_2 \cdot m_2} \tag{17}$$

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 Eq. (13) 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 follow:

$$SN = a_1 \cdot D_1 + (LCR_2 \cdot a_2) \cdot D_2 \cdot m_2 + (LCR_3 \cdot a_3) \cdot D_3 \cdot m_3 \tag{18}$$

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

Following this method, the designer may quantify the performance benefits of the geogrid reinforcement through both the increase in service life (i.e. number of ESAL) and decrease in layer thickness required. The standard practice for all these three approaches is very 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.

It is easy to derive the relationship between LCR and BCR:

$$LCR = \frac{1}{1 - BCR} \tag{19}$$

Hence 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 recently shown (Isola et al, 2018) that TBR and LCR approaches can be used independently, but LCR approach leads to more conservative results.

5.3.3 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 should be 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 (Zornberg, 2012). The proper use of the MEPDG method with geosynthetics for stabilization would require the definition of rut and fatigue laws for stabilized roads, which would require extensive testing on stabilized sections. Since such work has not been carried out yet, the application of MEPDG to stabilized roads is still highly questionable.

5.3.4 Mechanistic-Empirical method for geocells

For paved roads on soft or stable subgrades the design method for incorporating geocells can be based on the elastic behaviour of pavement structures following the Mechanistic-Empirical design procedure (Kief, 2015b). A mechanistic model of each pavement layer can be developed by including its thickness, elastic modulus and Poisson's ratio into commercially available layered-elastic analysis programs for pavements; the mechanistic model produces an elastic response of tensile and compressive strains for fatigue and rutting failure modes, respectively.

The Mechanistic-Empirical method applied to geocell stabilization of paved roads normally utilizes the following parameters:

- Resilient modulus of subgrade
- Number of ESAL (80 kN equivalent single axle load) in the design life
- Modulus Improvement Factor (MIF)

The MIF of the layer stabilised with geocells relates to the improvement of the layer modulus (base and/or sub-base), which is expressed by the following formula:

$$MIF = E_{\text{with cellular confinement system}} / E_{\text{without cellular confinement system}} \tag{20}$$

where:

$E_{\text{with cellular confinement system}}$ = modulus of geocell stabilized base / sub-base
$E_{\text{without cellular confinement system}}$ = original modulus of the unstabilised base / subbase

MIF for geocells is obtained through performance testing including validation of specific geocell properties for the design life (Rajagopal, et al, 2014), specific infill type and Resilient modulus of subgrade.

MIF is higher for:

- Smaller cell size
- Higher cell height
- Higher ratio of cell height to cell size
- Lower modulus of infill materials
- Higher resilient modulus of subgrade / support
- Higher tensile strength and stiffness of geocell strips

Measured MIF values for NPA (Novel Polymeric Alloy) geocell are typically in the range 1.8 - 4.5, depending on the above factors.

The original modulus of the unstabilised base / subbase depends on:

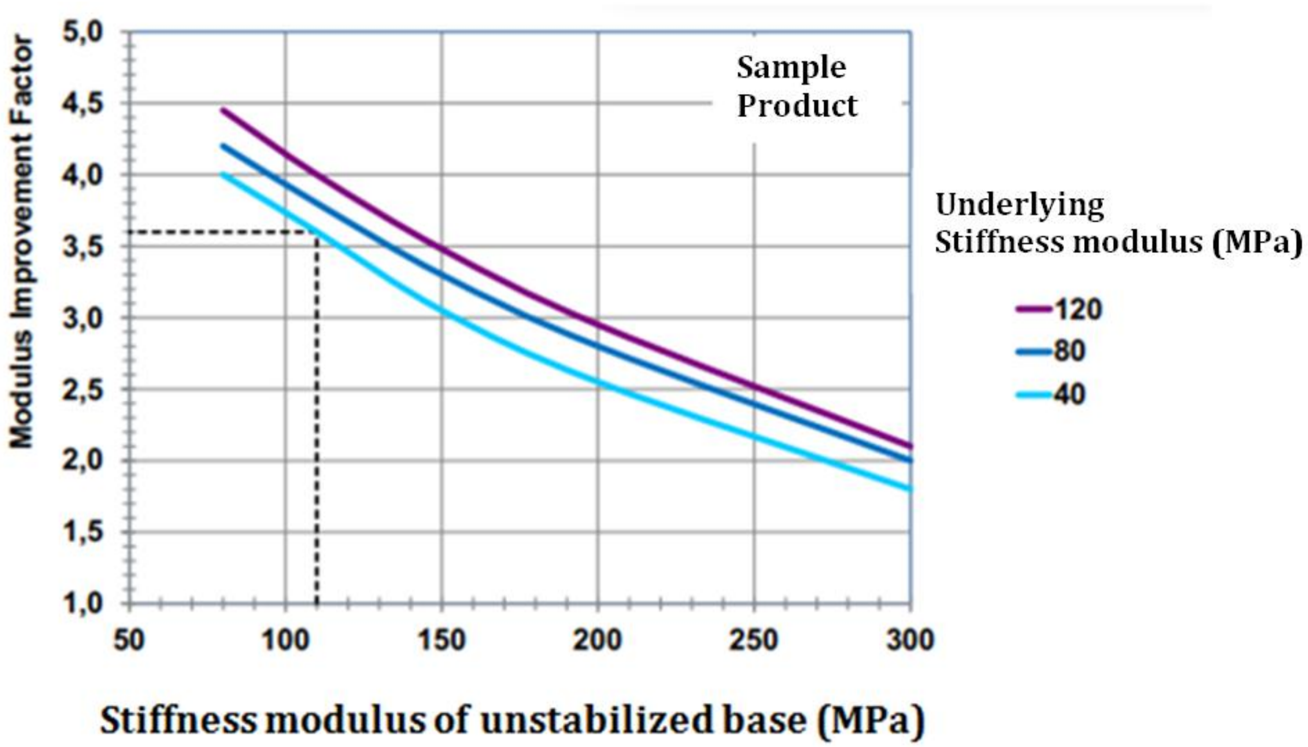
- the bearing capacity of the underlying layers and subgrade;
- the layer thickness of the subbase.

With increasing values of bearing capacity of underlying medium and increasing thickness of the subbase, the stiffness of a subbase in an unstabilised situation increases.

Figure 8 can be used for a preliminary evaluation of the MIF value as a function of the modulus of the unstabilised base / subbase and of the modulus of the layer below. The MIF from the chart are based on geocells with perforated strips having tensile strength (ISO 10319) higher than 16 kN/m with less than 3 % accumulated permanent deformation, evaluated according to ASTM D6992 with three isothermal steps at 44^{0}C, 51^{0}C and 58^{0}C (Vega et al, 2018).

If additional unstabilised layers are placed above the geocell stabilized layer, then their modulus may be improved as well due to the stronger support provided by the geocell stabilized layer.

Fig. 8. Chart providing the MIF value of NPA (Novel Polymeric Alloy) geocell stabilized layers as a function of the modulus of the unstabilised base / subbase and of the modulus of the layer below (modified from Vega et al, 2018).



5.4 Working platforms and load transfer platform

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 play 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, named Static method and BCR method.

5.4.1 Static method for clay subgrade

When a vertical static load is applied to a load transfer platform involving a gravel aggregate – 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 may reduce the bearing capacity of the clay to as little as one half the value for purely vertical loading. If a stabilising 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 (1965), Barenberg (1980), and other researchers, showed that the bearing capacity of a soft clay subgrade - unstabilised granular layer system can be approximated as:

$$q_u = \pi\, c_u \tag{21}$$

while the bearing capacity of a soft clay subgrade - stabilized granular layer system can be approximated as:

$$q_r = 2\,\pi\, c_u \tag{22}$$

Rutting and deformations of the granular platform can be limited by reducing the allowable bearing capacity through a proper Factor of Safety FS: a value FS = 2 already affords good reduction of deformations and displacements; a value FS = 3 shall be selected when allowable deformations shall be minimal:

$$q_{ua} = q_u \;/\; FS; \qquad q_{ra} = q_r \;/\; FS \tag{23}$$

The static method derives from the above described basic research 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 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 Factor of Safety. This technique assumes that the vertical pressures are distributed through the platform soil layer according to Boussinesq theory for circular or rectangular loaded area. 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 from the medium characteristics. The Boussinesq theory provides the stress components in every point along the vertical line passing by the centre of the circular area or by 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 tire pressure p, on a circular area whose equivalent radius is:

$$r = (P / \pi\, p)^{0.5} \qquad (24)$$

Introducing Eq. (24) into Boussinesq equation for uniform vertical stress on a circular area and solving for depth z, the required thickness of the unstabilised layer is:

$$z_u = r / [(1 - q_{ua} / p)^{-2/3} - 1]^{1/2} \qquad (25)$$

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

The required thickness of the unstabilised layer is:

$$z_r = r / [(1 - q_{ra} / p)^{-2/3} - 1]^{1/2} \qquad (26)$$

Similar formulas, using the Boussinesq equation for uniform vertical stress p on a rectangular area, allows to calculate the required thicknesses z_u and z_r in cases of rectangular loads, like tracks and bearing pads (Rimoldi& Simons, 2013).

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 of 30 % to 50 % by using planar 2D stabilising geosynthetics.

After setting the platform thickness, the stabilizing geosynthetics layout have to be designed. Section 5.5 provides actually available design methods.

The best results are usually obtained with 2 - 3 layers of equally spaced stabilizing geosynthetics at vertical centers of 150 – 300 mm.

Further details can be found in Rimoldi & Scotto (2014).

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 stabilised layer, rather than on Boussinesq theory.

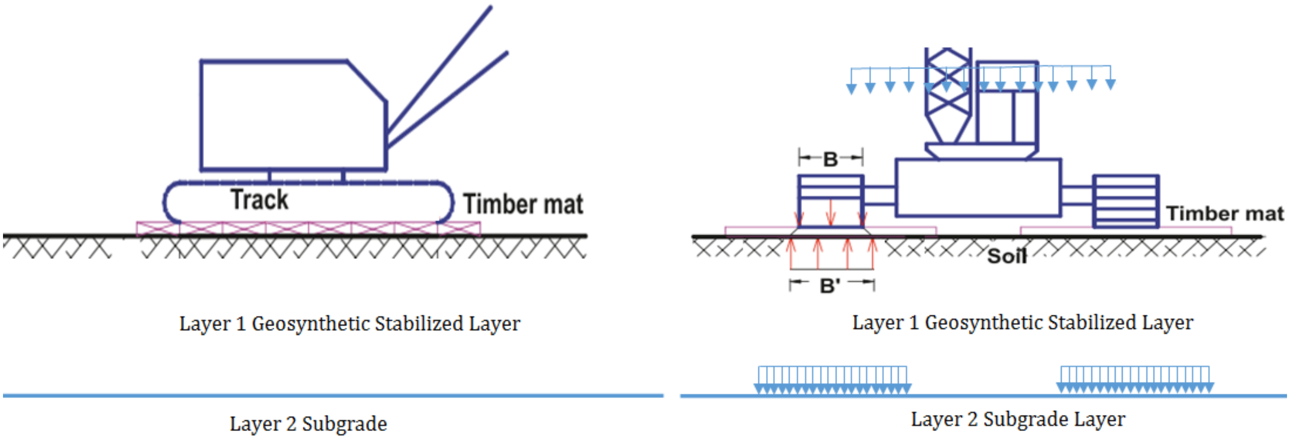
Westergaard's analysis is based on the assumption that the soil on which load is applied is stabilised by closely spaced horizontal layers (geosynthetics in this case) which prevent horizontal displacement. The effect of the Westergaard assumption is to reduce the stress at the top of layer 2 in Figures 9 below those obtained by Boussinesq equations.

Westergaard 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 Westergaard theory are less conservative than methods based on Boussinesq theory.

As such, it is advisable to check in advance whether the Westergaard's theory is actually applicable in the considered design situation.

Figure 9. Example of rectangular loaded area: tracked crane (lateral and front view), optionally staying on timber mat



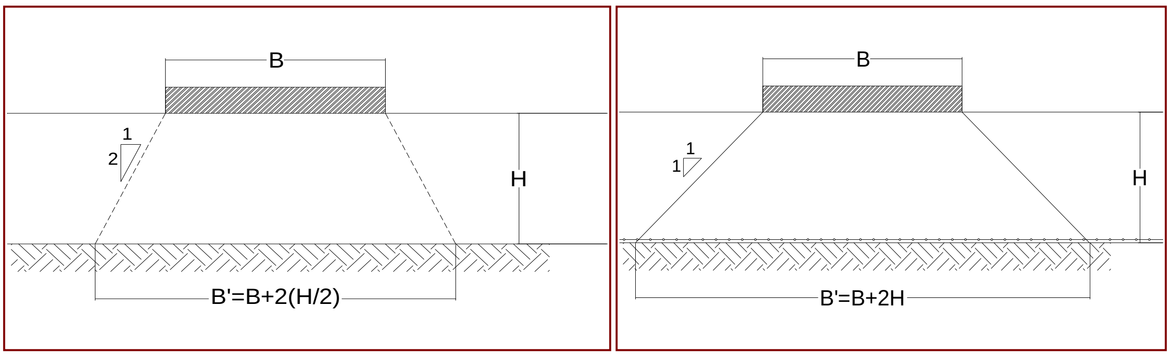
5.4.2 BCR method for soft sand subgrade

The BCR method is based on the assumption 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 stabilising geosynthetics in the granular layer is able to increase the bearing capacity, through a widening of the load distribution area, as shown in Fig. 10.

Experimental researches have demonstrated that the insertion of a stabilising geosynthetics affords the load spreading angle to increase from a batter of approx. 2V/1H to a batter of approx. 1V/1H, as shown in Fig. 10.

Fig. 10 - The load distribution area widens when the load spreading angle increases from a batter of 2V/1H to a batter of 1V/1H



The stabilised granular 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, as shown in Fig. 10.

The bearing capacity is then calculated using the well-known Terzaghi equation, like for a foundation having dimensions B' and L':

$$q_{lim}=1/2\ \gamma'\ B'\ N_\gamma s_\gamma + c'\ N_c s_c + q'\ N_q s_q \qquad (27)$$

where N_q, N_c, N_γ are the bearing capacity factors, and 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 (20) the bearing capacity q_u and q_r can be calculated for the unstabilised and stabilized granular layer condition, for any value of the deck thicknesses h_u and h_r.

The Factors of Safety are then calculated as:

- unreinforced deck: $FS_u = q_u / \sigma_{vu}$ (28)

- reinforced deck: $FS_r = q_r / \sigma_{vr}$ (29)

The Bearing Capacity Factor BCR is calculated as:

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

Optimal results are usually obtained for BCR = 1,60 – 1,75, affording FS_r= 2.0 – 3.0.

Further details can be found in Rimoldi & Scotto (2014).

5.5 Railways

5.5.1 US Federal Railway Administration

A comprehensive field case study conducted by the US Federal Railway Administration showed that geocell stabilisation performance extended the maintenance cycles (Palese, et al, 2017).

5.5.2 Network Rail (UK) Design Specifications

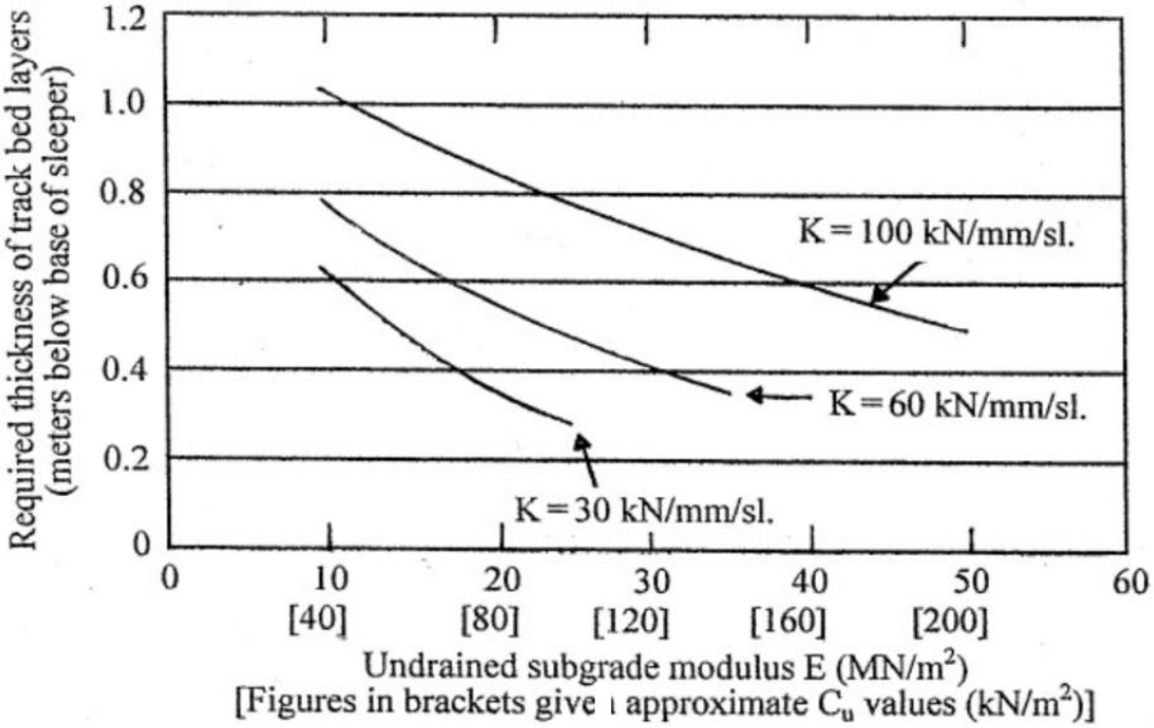
As reported by Das (2016), the use of geogrids is beginning to be incorporated into the railroad design codes of several countries, particularly in Europe. As an example, following is a summary of the guidelines adopted by the Network Rail (2005). According to the guidelines the required dynamic sleeper support stiffness (K) is given in Tab. 1.

Table 1 - Required dynamic sleeper support stiffness (K) for maximum axle load of 25 tonnes (after Network Rail, 2005)

		Minimum dynamic sleeper support stiffness (K) kN/mm/sleeper end
Absolute value		30
Existing main lines	With geogrid	30
	Without geogrid	60
New track	Up to 100 mph	60
	Above 100 mph	100

The dynamic sleeper support stiffness (K) is defined as "the peak load divided by the peak deflection of the underside of a rail seat area of an unclipped sleeper subjected to an approximately sinusoidal pulse load at each rail seat; the pulse load being representative in magnitude and duration of the passage of a heavy axle load at high speed." Accordingly, Figure 11 can be used to obtain the required trackbed thickness with known values of K and undrained subgrade modulus E (or undrained cohesion C_u).

Figure 11. Determination of thickness of trackbed layers (Network Rail, 2005).



5.5.3 Ev2 Method

Moduluses Eo on subgrade and Ec on selected level of the construction layers are measured, e.g. by bearing plate test.

Design procedures is then usually as follows:

- *Eo* modulus on subgrade is measured
- *Ec* modulus on selected level of the construction according to the railway /road category is defined (standard range 20-180MPa)
- *E* modulus of fill material should be known (typically 50 - 90 MPa)
- ***hs*** thickness of unstabilized layer is derived from design chart below
- ***hs*** thickness of mechanically stabilized layer is derived from design chart below is reasonably smaller but it is not generic, has to be defined by manufacturer on the basis of independent research
- Construction layer *hc* is to be used e.g. to achieve frost protection thickness

Figure 12. Typical design chart for determination of *hs*

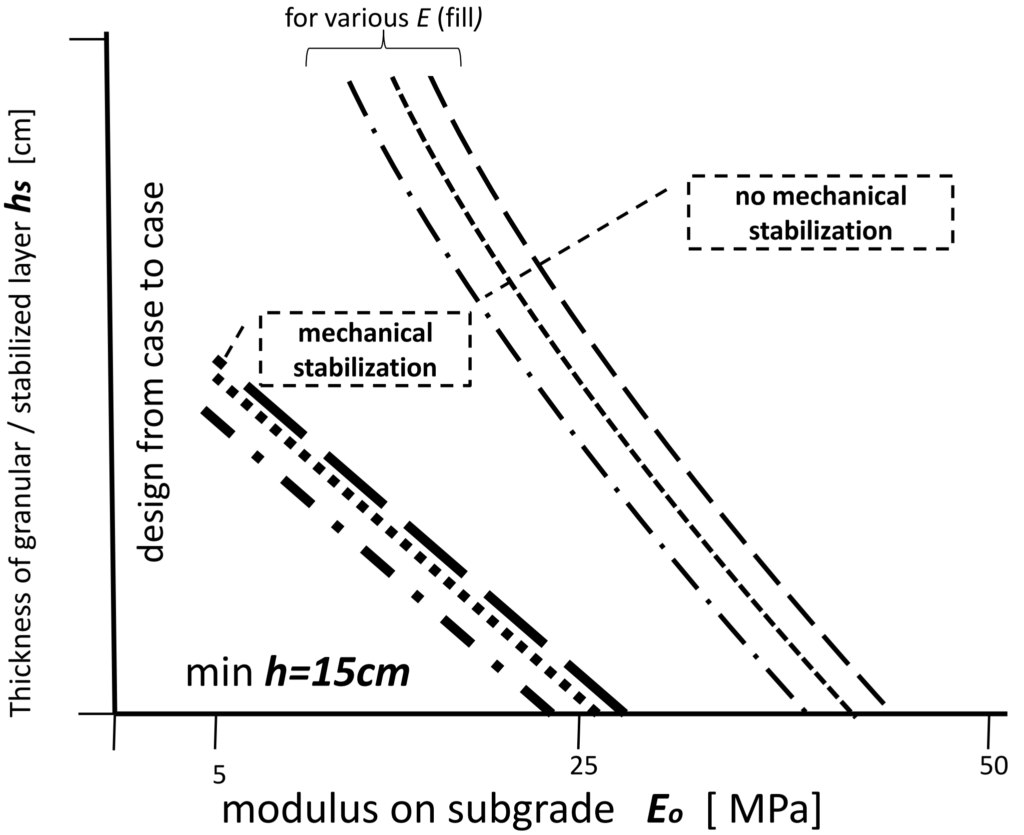
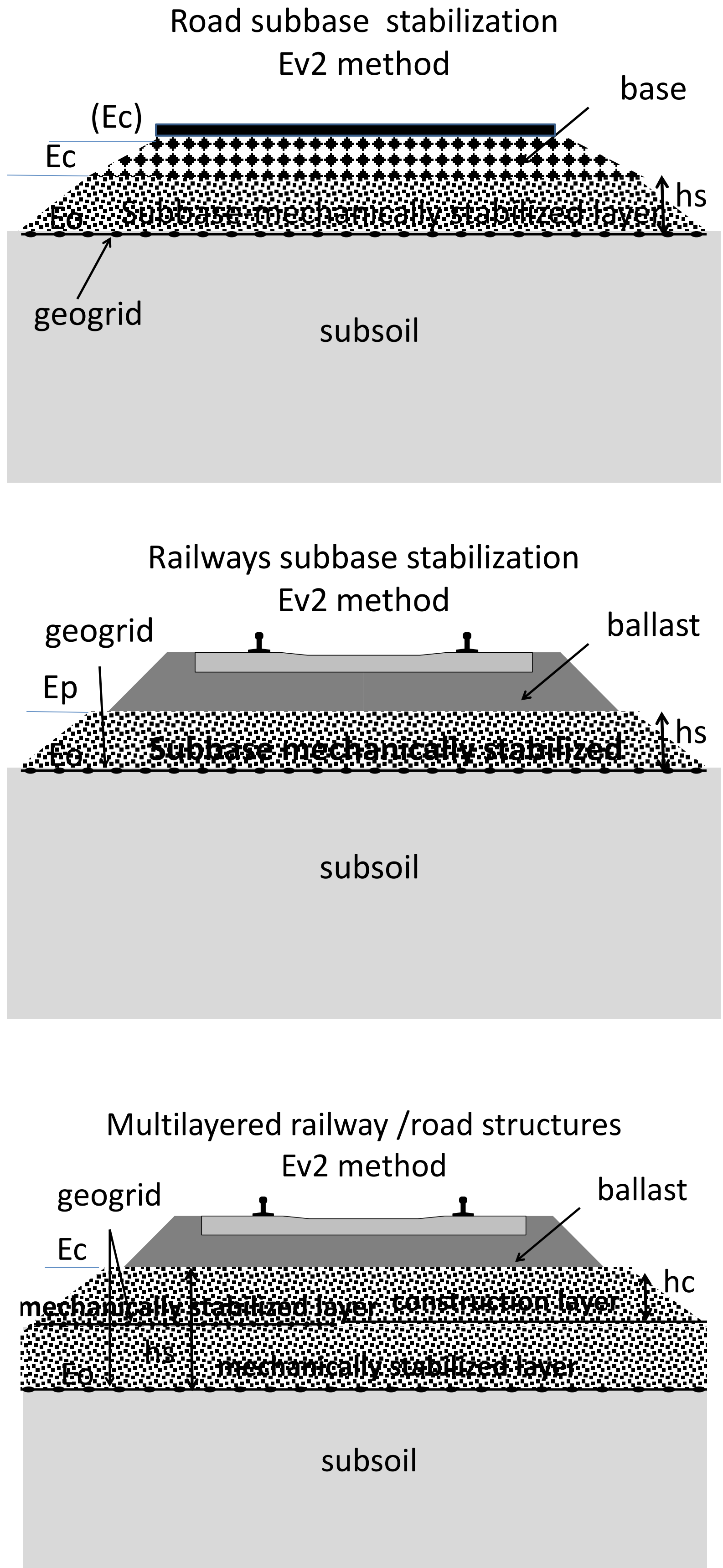


Figure 13.



a)

b)

c)

5.5.4 Railway Ballast

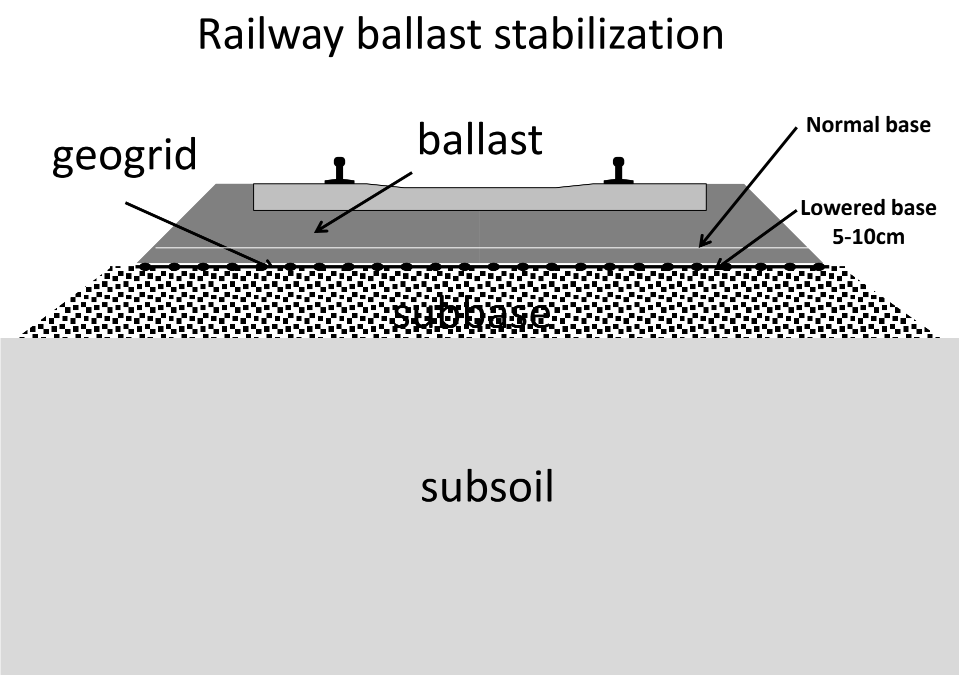
There is no direct method for the design of mechanical stabilisation of railway ballast. However, there are parameters that influence the design.

Design principles that can be applied are as follows: -

- Service life of the ballast layer can be extended by several years due to restriction of particle displacement and therefore less abrasivity of grains,
- A geogrid is to be placed at the base of the ballast layer
- A geogrid with adequate aperture size has to be selected

Increasing the thickness of the ballast layer by 5-10 cm (Fig.14) is recommended to avoid damaging the geogrid during tamping or clearing operations.

Figure 14. Typical layout of railway ballast stabilization



5.5.5 Design method for railway stabilization with geocells

A primary contribution of geocells to a railway structure is the improved modulus (MIF) of the stabilised layer, and of layers above it, if exist. This concept is detailed in Section 5.3.4.

Based on the MIF value, the geocell stabilization affords to increase the overall stiffness of the sub-ballast structure. The stabilised sub-ballast layer can then be optimised in terms of reduced thickness and/or the use of lower quality/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 railways standards (typically between 20 and100 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 Tab. 2 (from Livneh, 2013), where:

h_{sb} = Subbase layer thickness [mm]
h_b = Granular base layer thickness [mm]
E_{sg} = Subgrade elastic modulus [MPa]
E_{sb} = Subbase elastic modulus [MPa]
E_b = Granular base elastic modulus [MPa]
ν_b, ν_{sb}, ν_{sg} = Poisson's ratio of base, subbase and subgrade, respectively

Table 2. Formulas for different layer moduli (Israel Road Authority, 2003)

	Elasticity Parameters	Notes
Base	$E_b = E_{ab}$ x (1+0.0067 x h_b [mm]) $\upsilon_b = 0.35$	E_b [MPa] $\leq$ 700
Sub-base	$E_{sb} = E_{sg}$ x (1+0.003 x h_{sb} [mm]) $\upsilon_{sb} = 0.35$	E_{sb} [MPa] $\leq$ 300
Subgrade	E_{sg} [MPa] = 14 x CBR [%] $\upsilon_{sg} = 0.40$	2 < CBR [%] < 12

After calculating E_{sg} and E_{sb} with the formulas in Table 2, it is possible to evaluate the MIF value using the chart in Fig. 13 (valid only for NPA geocell) or with other charts if available in literature.
If the subbase is stabilized with geocells, the modulus of the stabilized subbase E_{sbs} is:

$$E_{sbs} = E_{sb} \cdot MIF \tag{40}$$

Once the layer thicknesses and moduli of the unstabilised 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 unstabilised railways structure using standard stress/strain software for railway design.

The stress - 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:

$$\frac{\sigma_1}{\sigma_2} = \left(\frac{N_2}{N_1}\right)^{\lambda} \tag{41}$$

where:
σ_1, σ_2 = vertical stresses corresponding to N1, N2 loading cycles respectively
λ = 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:

$$\frac{\sigma_1}{\sigma_2} = \left(\frac{T_2 / P_2}{T_1 / P_1}\right)^{\lambda} \tag{42}$$

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

$$\frac{\sigma_1}{\sigma_2} = \left(\frac{T_2}{T_1}\right)^{\lambda} \qquad (43)$$

Where:
T_1, T_2 = daily traffic tonnage corresponding to N_1, N_2 loading cycles respectively.

Formulas (41), (42), and (43) allows to calculate the improvement in loading cycles and daily traffic tonnage afforded by the geocell stabilization design.

5.6 Design of the geosynthetics layout

The above illustrated design methods usually assume that the granular soil base is stabilized with just one geosynthetic layer, but the actual required geosynthetic layout needs to be furtherly designed.

Geosynthetics provide the following stabilizing mechanisms:

- base course lateral restrain mechanism for horizontal stresses generated by the soil self weight;
- base course lateral restrain mechanism for horizontal stresses generated by dynamic / cyclic loading (typically wheels);
- tensioned membrane mechanism at the base or subbase – subgrade interface.

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

The dynamic / cyclic effect of wheel loading is always considered in the presently available methods (AASHTO method, Giroud – Han method, Leng - Gabr method, etc.): therefore the thickness of base and / or subbase afforded by 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 case of a working platform) or quasi-static (like in 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 and the overall tensile forces generated in each layer of geosynthetic, and then to select the appropriate geosynthetic for each layer based on the limit strain criterion.

The tensile forces produced on the geosynthetic layers by the three active mechanisms need to be defined.

Rimoldi & Scotto (2012) developed a model consisting of four soil layers and any number of geosynthetic layers for the design of planar 2D stabilizing geosynthetics (Fig. 20): 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 σ_{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.

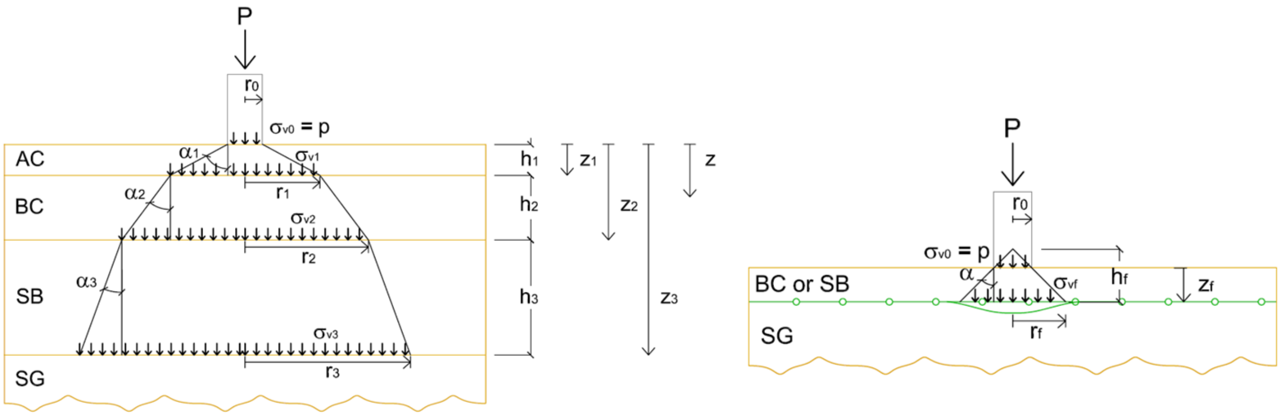


Figure 20. General scheme of the 4-layers model and of the first geosynthetic layer placed at subgrade interface

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

- Force due to the horizontal soil trust:

$$T_{zi} = \int_{(i-1)-th\_GSY}^{i-th\_GSY} \sigma_{h-soil}(Z) \qquad (44)$$

Where: T_{zi} is the tensile force generated in the i-th geosynthetic by the soil thrust; Z is the reference depth; $\sigma_{h\text{-}soil}$ is the horizontal stress produced by the soil trust.

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

$$T_{pi} = \int_{(i-1)-th\_GSY}^{i-th\_GSY} \sigma_{h-load}(Z) \qquad (45)$$

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.

- 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 (Fig. 20), while the upper layers of geosynthetics are far less subjected to vertical displacements.

  The tension in the reinforcement is determined from the following equation (Giroud et al. 1990):

$$T_m = W_{TCj} \cdot \Omega \cdot r_f \qquad (46)$$

  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:

$$T_{tot\text{-}i} = Tz_i + T_{Pi} + T_m \tag{47}$$

where Tm, as said, 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 shall be able to provide a tensile force equal to or larger than Ttot-I at the set maximum strain limit.

For important structures the geosynthetic strain shall be limited to 1 – 2 %, while for less important structures (or when the design conditions afford slightly larger deformations) 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 Rimoldi & Scotto (2012) for circular loaded area and in Rimoldi and Simons (2013) for rectangular loaded area.

5.7 Static Loads - Settlement Control / Limitation

The degree and extent to which loads are transferred from the surface of a stabilised 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, users of this document are referred 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 would normally consider 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 a greater concern.

5.8 Reduction in Deformation from Trafficking

As was stated earlier, confinement has been identified as the geosynthetics stabilisation mechanism. In simplest terms, the geosynthetic restricts lateral movement of aggregate fill placed upon or inside it. Many full-scale empirical trials have shown that geosynthetic stabilised aggregate layers are significantly more resistant to surface deformation than non-stabilised layers when subjected to repeated trafficking loads. Furthermore, it has also been shown that deformation of the subgrade beneath the stabilised layer is also significantly reduced. This deformation reduction relies heavily on effective lateral restraint of the aggregate particles and the resulting confinement which creates and 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/stiffness is a spreading of the vertical stress distribution and a corresponding reduction in the deformation on top of the subgrade (USACE, 2003). 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 stabilised layer also restricts deformation of the subgrade.

5.9 Reduction in Unbound Aggregate Degradation

The results of modelling discussed above and those of full-scale laboratory testing and traffic studies have demonstrated a positive improvement in the amount of trafficking that can occur over stabilised layers in comparison to control sections of non-stabilised granular structures. Based on this observation of extended life, evaluation of aggregate behaviour using an unstabilised and stabilised aggregate layer in full-scale study show both life extension and aggregate performance (Han et al 2011). More aggregate degradation normally occurs in the control sections than the stabilised test sections (Leshchinsky and Ling, 2013).

It is accepted that degradation of granular material increases the content of fines, reducing the water permeability and increasing the sensitivity of the granular material to reduced bearing capacity by increased water content and pore water pressure. Beside lateral confinement the positive influence of separation and filtration effect may become significant.

5.10 Extension of Design Life

It follows from the logic discussed in 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 reflected in specific design methods, there is evidence of direct correlation between one or more geosynthetic characteristic and performance. For other design situations, o for design conditions different from those in the specific method, the extension in design life can be determined by performing full scale and / or laboratory scale validation and calibration of performance of a geosynthetic and aggregate combination for specific stress conditions. This can be carried out in accordance with local and national guidance for pavements and through use of full-scale testing. However, there are numerous design approaches utilized around the world to accomplish this task.

5.11 Ground and Groundwater Conditions

For stabilisation, the geosynthetic is required to work in combination – as a composite – with aggregate fill to support applied loads (Anderson, 2006). With respect to building over soft soils, design methodologies, either bearing-capacity-based (USACE, 2003; EBGEO, 2011; SVG, 2003) or serviceability-based (Giroud and Han, 2004, 2006; EBGEO, 2011; SVG, 2003), yield appropriate aggregate fill thickness for a given set of input parameters. Likewise, presuming the subgrade is soft because of moisture content, it is normally important to keep in mind that the aggregate fill must be clean (cohesionless, with preferably single-digit percent fines) and drainable (Skorseth and Selim, 2000). Insufficient thickness and/or poor quality can compromise the beneficial functions of the geosynthetic, and jeopardize stability of the system (Anderson, 2006).

In all cases where a geosynthetic stabilised layer is incorporated within a pavement structure the groundwater would typically need to 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 then 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/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.

5.12 Climatic Conditions

With regard to both environment and climate conditions Henning, et. Al. (2014) reviewed the performance of 140 long term pavement performance (LTPP) sites for paved roads which covered the environmental and climatic conditions throughout New Zealand. The findings suggest that roads in more sensitive climatic zones will have an increased rut rate (by approximately 0.1 mm/year) as compared to the more stable climatic areas. For example, at an ESAL of 400 axles per day the rut rates are 0.16mm/year and 0.28 mm/year (0.12 mm difference) for the low sensitivity (dryer regions on more stable geological formations) and high sensitivity (wet regions on less stable geological formations) area respectively. 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 as high on poor drainage sections compared to sections where adequate drainage was provided throughout the pavement section. The researchers also established the fact that sections having poor or inadequate drainage will deteriorate much faster under heavy traffic volumes (Henning, et. al., 2014).

5.13 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 (FHWA Sustainable Highways Initiative, 2015).

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 or reduction in maintenance intervals.

Further, since one role of a stabilised layer is to protect the subgrade, the future rehabilitation of these roads should be limited to restoration of the Portland Cement Concrete (PCC) or asphalt layers. It is important that the carbon emissions and embodied energy costs of incorporating a geosynthetic are not seen as additive to a project. The designer would typically need to consider the effect of the geosynthetic and calculate the net effect. For example, inclusion of a geosynthetic for stabilisation will potentially result in a reduction in the use of aggregate material and/or enables the use of marginal quality or recycled soils and/or an increase in the service life of the road (Album, 2014), (Pokharel, et al, 2016). Case studies on the use of geosynthetics have been reported by the UK Waste Recycling and Packaging Organisation (WRAP, 2010) and the European Association of Geosynthetic Manufacturers (Elsing et al 2012).

In terms of economic sustainability, stabilisation with geosynthetics reduces aggregate costs and increases a pavement lifespan, thereby reducing maintenance costs (Han, et al, 2012). 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.

5.14 Impact of Filtration / Separation / Drainage Requirements

As a practical matter, water cannot be prevented from infiltrating an unpaved road structure, or even one that is paved (USDOT-FHWA, 1992). Thus, it is normally prudent to assume that saturated, or elevated, moisture conditions exist within the upper reaches of the subgrade, particularly if the subgrade exhibits CBR values less than 3.

With geosynthetic placed directly on top of the subgrade, the overlying aggregate fill (i.e. either base or sub-base material) should be sufficiently graded to provide subgrade filtration and prevent soil migration (Christopher, et al. 2001). Filtration may be defined as allowing the free flow of water into or cross geotextile while separation means preventing intermixing of adjacent dissimilar soil and/or fill materials. For both separation and filtration, a geotextile or geotextile related product may be used. However, a filter stability calculation on the adjacent soil layers would 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 should be small enough to prevent aggregate partition across cells and the perforation pattern should be semi-uniform to allow free drainage.

6.0 Design Factors & Methodology

6.1 Design Factors

There are many factors which would normally be considered when considering the design of a stabilised granular layer of which the interaction of the geosynthetic is only one. The following design factors are product and project specific, and would normally be evaluated based on performance testing where appropriate: -

- Type and strength of subgrade
- Groundwater conditions
- Type, compaction, density and thickness of granular material
- Magnitude, type and frequency of loading
- Allowable deformation
- Design life of layer and overall construction
- Maintenance intervals
- Properties of other layers in the construction
- Climatic conditions
- Tensile stiffness of the geosynthetic
- Type and morphology of the geosynthetic (e.g. geogrid or geocell, aperture size, cell height and diameter etc)
- Interaction and lateral confinement capability of geosynthetic with granular material
- Location in the stabilised layer: -

- On the subgrade – improve bearing capacity of the subgrade – stabilised soil layer system
- In the subbase/base – improve elastic stiffness of the system

6.2 Design Methodology

As a result of the above, any design model is consequently complex. This complexity has thus far meant that there are no widely accepted general stabilisation performance related characteristics of geosynthetics, or precisely quantify individual properties of any geosynthetic which can be input in a theoretical calculation to satisfactorily design these systems. Set against this background, the design of internally confined granular layers has typically been undertaken empirically, based on full scale loading and trafficking trials as appropriate. However, if the properties of the geosynthetic at the actual loading, deformation and stress conditions are known, these may be used to characterise the performance of unique layers within the overall system.

Correctly designed, monitored and repeated full scale trials incorporating individual geosynthetics, potentially augmented by laboratory testing and numerical modelling, permit product specific empirical design rules for those geosynthetics to be established.

For unpaved roads the Giroud and Han (2004) method can be used for design of the stabilized layer thickness. The original method was calibrated using punched and drawn geogrids. Use of other geosynthetics can be accomplished through calibration as described by Giroud and Han (2012).

For paved roads the design method will depend on the methodology selected by local pavement designers. Today there exists both empirical and mechanistic-empirical design methods. In North America the empirical method of pavement design is presented in AASHTO (1993). Within AASHTO (1993) the structural coefficients associated with the unbound aggregate are modified based on protocol described in AASHTO R 50-09 (2009) to consider the influence of the geosynthetic. Adjustments to other empirical design methods utilized world-wide could follow a similar approach as outlined in AASHTO R 50-09 (2009). Within Mechanistic-Empirical design parameters associated with stabilized aggregate resilient modulus, subgrade resilient modulus and permanent deformation have been examined to date. However, full scale laboratory and field testing is required to achieve calibration for each type of geosynthetic.

For unpaved roads and railway aggregate on soft subgrades: the Giroud - Han (2004) or the Leng – Gabr (2006) or the Rimoldi (2012) methods for design with geosynthetics can be 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) as per the modified Giroud-Han methodology (Pokharel, et al, 2015).

For paved roads on soft subgrades: the design method for incorporating geocells may be based on the elastic behavior of pavement structures and follows the Mechanistic-Empirical design procedure (Kief, 2015b). Road design with geocells may be based on the Mechanistic-Empirical design method for flexible pavements using the layer elastic model. A mechanistic model of each pavement layer may be created by including its thickness, elastic modulus and Poisson's ratio into commercially available layered-elastic analysis programs for pavements. The Mechanistic-Empirical method normally utilizes the following parameters: -

- Resilient modulus (Modulus of Elastic Response) of subgrade
- Number of ESAL in the design life
- Modulus Improvement Factor (MIF) - "for geocells," obtained through performance testing "including validation of specific geocell properties for the design life (Rajagopal, et al 2014). Fatigue and rutting criteria for the geocells, obtained through performance testing

The MIF of the layer stabilised with the geocells relates to the improvement of the layer modulus (base and/or sub-base), which may be expressed by the following formula:

$$MIF = E_{\text{with cellular confinement system}} / E_{\text{without cellular confinement system}}$$

Where:

$E_{\text{with cellular confinement system}}$) = the modulus of base / sub-base incorporating the cellular confinement system

$E_{\text{without cellular confinement system}}$ = the original modulus of base / sub-base

7.0 Materials

7.1 Properties of Aggregate

An unbound granular 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 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 and/or steel drum compaction equipment) and history, grading and fines content (Kwon et. al., 2013).

7.2 Key Properties for Geosynthetics – Internal Confinement

As stated above, the precise quantification of individual properties of geosynthetics for inclusion in the design of an internally confined stabilised 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 stabilisation (internal confinement) are the tensile stiffness of the geosynthetic at low strain levels and degree and efficiency of the geosynthetic / 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). The TBR is a performance indicator and is defined as the ratio of the number of passes necessary to reach a given rut depth for a section stabilised with a geosynthetic to the number of passes necessary to reach the same rut depth for a non-stabilised pavement section with the same aggregate, same aggregate layer thickness and subgrade properties. This calibration of individual geosynthetics to provide TBR's, BCR's and LCR's is established using full-scale testing.

Trials used for calibration of TBR, BCR or LCR would necessarily 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 may 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. Those shear forces must then be 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 would normally need to meet the requirements for the serviceability limit state of the stabilised construction.

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, as discussed above, the soil/geosynthetic interaction must be considered.

7.3 Key Properties for Geosynthetics – External Confinement

The impact of geocells was quantified through laboratory testing and large-scale field tests (Pokharel et al, 2011 & Yang et al, 2011). Geocells were found to stiffen granular materials layers by 3D confinement, thereby increasing the modulus of the stabilized layer. This benefit is quantified as the 'MIF', 'Modulus Improvement Factor', as detailed in Section 6.2.

Geocells provide external confinement through: -

a) hoop tension forces in the cell walls,

b) resistance from the surrounding cells, and

c) friction between cell walls and infill material

The geometry and material properties of the geocell are key properties to maximize the magnitude of mechanical stabilisation and to validate it for the design life of the project. These two essential properties are detailed below:

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 mechanical stabilisation for the design life. Other key properties include junction strength, friction etc. The stiffer the geocell, the higher the hoop tension stress will be, and, thus, the higher the MIF. Material properties may be measured by hoop tensile test (wide-width perforated cell wall and junction), Dynamic Mechanical Analysis (ISO 6721-1/ASTM E2254) and accelerated creep test (ASTM D6992 modified for geocells). The creep test for geocells should be 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 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 may invalidate stabilisation).

The geometry of the geocell including cell size, height, texture and perforation pattern affect the magnitude of mechanical stabilisation. The size (diameter) of the cells determines 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 should be 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 should be connected in such a way to restrain lateral and node rotation of the connection.

8.0 Testing

Many parameters of geosynthetics such as tensile behaviour or interaction can be evaluated by laboratory tests, corresponding to so called index testing, for which SO test standards exist. However, these index tests cannot completely reflect the complex in-situ behaviour of the geosynthetic / soil system. If needed, so called performance testing should be carried out for the geosynthetics and for the interaction of geosynthetics and soil.

The Key Properties of geosynthetics defined in previous sections (7.2 and 7.3) should be validated according to standard testing methods to ensure the design performance for the duration of the project design life, for example: tensile strength and stiffness/modulus of the geosynthetic.

Geocells should ensure low level deformation to correspond to the improvement factors in the pavement design, that refers to the increased stiffness (lower deformation) of the stabilized layer (e.g., MIF). A geocell with higher predicted deformation (greater than 2%) may not provide suitable improvement factors.

Typical tests that can be undertaken for geocells are as follows: -

- Single Strip Tensile Strength: testing of strength at yield of the full width of a wall, including perforations (ISO 10319) and Junction strength (ISO 13426-1 Part 1, Method C)

- Dynamic Stiffness: testing of storage modulus by Dynamic Mechanical Analysis (DMA, ISO 6721-1 / E2254)

- Accumulated Creep deformation: testing of Stepped Isothermal Method for wide-width perforated cell wall at fixed loads to simulate hoop stress in base layer and in lower layers (ASTM-D6992)

- UV tests to determine the maximum exposure time before covering the GSY with soil

- Durability tests applicable to the specific polymer of the geocells, e.g. Oxidation Resistance: testing of High-Pressure Oxidative Induction Time – HPOIT (ASTM D5885) @ 150°C

- Geocell panel connection strength (ISO 13426-1) and resistance to relative movement for resistance to node rotation, lateral and vertical movement

Performance testing of geocells and soil interaction should include variety of laboratory tests, including large-scale moving wheel test, and on-site monitored field trials. The improvement factors in the pavement design are based on those tests, for the specific tested geocell type (material and geometric properties).

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Annex A - Tensioned Membrane Design

A1 Description of Mechanism

In the tension membrane effect, the geosynthetic is providing a reinforcement function. Assuming the pavement design requirements can accommodate a degree of deformation beyond that normally associated with stabilisation, then the mobilization of the tensioned membrane effect may make a valuable contribution to the design process.

Based on the principle of internal & external confinement, lower lateral shear strain in the base aggregate and an increased lateral stress state result in less vertical and horizontal deformation at the aggregate/roadway surface. The magnitude, by which the horizontal and vertical strain in the aggregate layer can be reduced, depends on the geosynthetic tensile stiffness required for stress equilibrium as well as on the soil/geosynthetic interaction efficiency.

As a result of repeated traffic loading, plastic deformations within the aggregate as well as the subsoil accumulate to increased strain and therefore an increased stress state of the tensioned geosynthetic. At deflections and irregular surface conditions of the subsoil/aggregate interface, a vertical load component of the tensioned and deflected geosynthetic contributes to the vertical load equilibrium, resulting from increased vertical shear forces and an increased lateral stress state, supporting the applied load and in turn reducing the local intensity of vertical loading transferred to the subgrade.

A2 Design Factors & Methodology

As the tension membrane effect is providing a reinforcement function, guidance on the design of reinforcement geosynthetics can be found within TR 18228-7.

A3 Key Properties for Geosynthetic

The mobilised strength of the geosynthetic depends on the stiffness-modulus of the geosynthetic and the amount of lateral deformation. Assuming full contact between geosynthetic and soil, the mobilised strength is linearly linked to the stiffness modulus of the geosynthetic. As loads appear to act in different directions, the radial tensile strength under confined conditions as well the tensile strength in longitudinal and transverse direction become decisive to generate the lateral restraint effect or an increased stress-state respectively to achieve stress equilibrium.

As the quality of interaction between geosynthetic and soil plays a key-role for the equilibrium of forces as discussed above, the soil/geosynthetic interaction coefficient is usually considered.

In the case of geogrids, as a result of aggregate particle movement within the base course, those geogrid elements (bars, ribs, bundles, etc.), which are positioned crosswise to the direction of particle displacement, are subjected to shear forces. Those shear forces are then transferred into the longitudinal elements via the geogrid junction. The strength of the connections in the junction area depends upon the manufacturing technology. The junction strength influences the performance of a product, to divert loads into unstressed areas of the reinforcement. (Schulz & Witte, 1991). (Christopher, 2007; Christopher & Perkins, 2008) recommend, that the junction strength should conform to approx. the tensile strength at 2% strain, which represents typical working load conditions.