Guidelines Document on the Use of CIRCLY to Evaluate the Benefits of Geogrids for Subgrade Stabilisation in Pavement Structural Design

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Table of Contents

1. INTRODUCTION ........................................................................................................................................... 1
2. DESIGN GUIDELINES..................................................................................................................................... 4
3. DESIGN EXAMPLES......................................................................................................................................... 9
4. STEP 7. CONSTRUCTION - Important Notes ................................................................................................. 17
REFERENCES ..................................................................................................................................................... 17
APPENDIX A - REVIEW OF SUBGRADE STABILISATION .............................................................................. 18
APPENDIX B - DESIGN CHARTS ................................................................................................................... 30
APPENDIX C - CONSTRUCTION GUIDELINES ............................................................................................... 35
1. INTRODUCTION

Sites with weak subgrade soils (i.e., CBR ≤ 3%) pose many challenges for pavement design and construction. Such soils provide little support for construction and may even be weakened by construction activities. Even if the pavement section could be constructed, there is a significant potential that a portion of the base course aggregate thickness will be compromised due to penetration of the aggregate into the subgrade during construction and the inability to adequately compact the lower portion of the base course aggregate layer. Continued strength loss and reduction in drainage capabilities of the base layer will likely continue to occur over the life of the pavement due to migration of fines into the base course material. Even if additional aggregate thickness is incorporated into the design, the actual strength loss is highly variable and cannot be accurately predicted, leading to a significant risk that the modulus of the aggregate is underestimated over the life of the pavement system. There is also a potential for localized compression of these soils resulting in differential settlement and correspondingly increased surface fatigue, which would also lead to a risk of premature failure.

Some form of treatment or, alternatively, the placement of a working platform, are thus required to enable construction to proceed, avoid delays in construction, assist in compaction of subsequent pavement layers, and provide improved stability over the life of the pavement as described in Section 3.14.1 Soft Subgrades of Austroads 2012 Guide to Pavement Technology Part 2: Pavement Structural Design (Austroads, 2012). An effective technique that provides minimal disruption of construction and improved pavement reliability is the use of a granular layer in conjunction with a geogrid reinforcing layer to provide a working platform or cap layer over the soft subgrade. Geogrids used in combination with quality aggregate minimize disturbance and allow construction equipment access to sites where the soils are normally too weak to support the initial construction work. They also allow compaction of initial lifts on sites where the use of ordinary compaction equipment is very difficult or even impossible. Geogrids reduce the extent of stress on the subgrade and prevent base aggregate from penetrating into the subgrade, thus reducing the thickness of aggregate required to stabilise the subgrade. Geogrid/geotextile composites also act as a separator to prevent subgrade fines from pumping or otherwise migrating up into the base. Geosynthetics have been found to allow for subgrade strength gain over time. However, the primary long-term benefit is preventing aggregate-subgrade mixing, thus maintaining the thickness of the base and subbase. In turn, rehabilitation of the pavement section should only require maintenance of surface pavement layers.

Additional long-term pavement performance improvement may also result from the use of geogrid reinforcement. For relatively thin base sections, less than 400 mm in thickness, including any additional stabilisation aggregate, the lateral confinement of the base that occurs due to interface shear stresses between the aggregate and the reinforcement continues to grow with traffic load applications, meaning that the lateral confinement of the aggregate increases with increasing load applications. This additional improvement is not currently considered in the Austroads 2012 Guide, but is an additional long term benefit to consider when selecting alternative stabilisation methods.
This document provides specific guidelines for determining the stabilisation requirements for the working platform when using geogrid reinforcements and how to incorporate the stabilised layer into the design of the pavement section. The guidelines will include how to use the CIRCLY mechanistic pavement design software to assist with design of pavements incorporating geogrids for stabilisation of soft subgrade soils.

CIRCLY was developed about 30 years ago for analysing layered elastic media subjected to surface loads. The development of CIRCLY has closely tracked the evolution of the Austroads mechanistic flexible pavement design procedures and is directly referenced by Austroads (Wardle, 2011). CIRCLY automates many requirements of the Austroads procedure such as sublayering of unbound granular layers. A unique parametric analysis feature can fine-tune layer thicknesses, which can be used for the evaluation of the benefits of using geosynthetics. However, at this time neither the specific inputs for geogrid reinforcement nor the method for performing this analysis are included in the current version of CIRCLY.

Briefly, the design process involves considering both the Construction Phase and the Post-Construction Phase (Figure 1).

![Figure 1: Phases considered for design of pavement incorporating Geogrid and Stabilisation Aggregate](image)

The pavement structure for the Construction Phase is designed based on the magnitude and size of anticipated construction traffic and tolerable rut criteria. The design of the geogrid reinforcement is primarily based on:

1) the strength required to survive (i.e., without significant damage) the stresses anticipated during the Construction Phase, and

2) opening characteristics that are compatible with the stabilisation aggregate. The subgrade is stabilised using the geogrid overlaid by a gravel "stabilisation layer" to provide a working platform that will allow construction traffic to operate over the subgrade without significant rutting.
The pavement structure for the Post-Construction Phase is designed using the usual Austroads approach taking account of the design traffic and the final pavement configuration. Two design methods are available for considering the working platform in the design of the pavement section:

1) a design CBR of 3% can be assumed at the new subgrade level (i.e., top of the stabilisation layer) for the subgrades below a CBR of 3%.

2) a mechanistic evaluation can be performed to verify effective subgrade strengths in excess of CBR = 3%.

These two methods were previously mentioned in the Austroads Guide Part 2 (2010); however, they no longer appear in the current guide. Nataatmadja et al. (2012) indicate that the omission of these approaches is likely due to the generally accepted practice of taking the thickness and strength of the working platform layer into account to achieve the nominated effective subgrade strength. Both methods are also recommended by the US the Federal Highway Administration (FHWA) (Holtz et al., 2008). These approaches are similar to assigning an "equivalent" CBR\textsubscript{E} to the subgrade, geogrid and stabilisation layer as is covered in Section 9.3.2 Effective Subgrade Strength of Austroads 2012 for rigid pavement design. However, specific guidelines are not provided for developing the stabilisation requirements for geogrids (e.g., base course thickness and geogrid property requirements) or the mechanistic analyses of the resulting section (i.e., to verify additional improvement provided by the additional aggregate used to stabilise the subgrade).

It should be noted that very good general information on the application and characterization of geosynthetics is provided in Austroads Publication No. AGPT04G/09, Guide to Pavement Technology Part 4G: Geotextiles and Geogrids (Austroads, 2009) and is highly recommended as a supporting reference to this design guide. That document provides broad coverage on haul road and temporary unbound pavements with a focus on geotextiles but does not provide specific design or property requirements for geogrids in stabilisation applications.

This document provides step by step guidelines for determining the stabilisation requirements when using geogrids. The guidelines include how to use CIRCLY for performing the required mechanistic analyses to determine the required section and verify improvement provided by the geogrid. The guideline includes the use of both of the pavement design approaches in Austroads 2012 Guide mentioned above. Again, these approaches are specifically for designing pavements with a subgrade CBR below 3% at the time of construction.

A design example for each approach with inputs and outputs from CIRCLY follows the design section. A detailed review of subgrade stabilisation is provided in Appendix A including a description of the geogrid reinforcement mechanisms along with secondary geogrid functions, a demonstration of pavement performance and benefit from the reinforcement layer in full-scale tests, and geosynthetic material properties and tests pertinent to the application.
2. DESIGN GUIDELINES

For either of the two approaches listed in the previous section, the subgrade is stabilised using the geogrid overlaid by a gravel “stabilisation layer” to provide a working platform that will allow construction traffic to operate over the subgrade without significant rutting. The geogrid is a more cost effective alternative to additional unbound material that might be placed to support construction operations and provides some additional support for the pavement system. If the subgrade has a CBR of less than 3%, and the aggregate thickness is determined based on a low rutting criteria in the following steps, the support for the composite system is then determined from one of the two approaches:

1) Assumed to have a design CBR = 3%.

2) Mechanistically modelled to achieve the effective subgrade strength in excess of CBR = 3%.

The pavement design then proceeds exactly according to standard procedures using the subgrade layer strength values, as if the geosynthetic and additional aggregate were not present. The following provides the details of the necessary steps required to complete the design.

STEP 1. Identify properties of the subgrade, including CBR, location of groundwater table, soil classification, and sensitivity. Tests should also be performed when the soils are in their weakest condition, when the water table is the highest, etc. Alternatively, a saturated soaked laboratory CBR test (AS 1289.6.1.1) could be performed to model wet conditions in the field (e.g., for compacted soils that will be exposed to wet conditions).

STEP 2. Determine additional aggregate thickness determine its thickness, \( t_r \), with the geogrid (and for cost comparison, \( t_0 \), without the geogrid) needed for establishment of a working platform using one or more of the chart methods included in Appendix B and CBR or the cohesive strength, \( c \), of the subgrade soil from Step 1. The undrained shear strength of the soil, \( c \), can be obtained from the following relationships:

- From field CBR, \( c \) (in kPa) = 30 x CBR
- From resilient modulus \( M_r \), \( c \) (in kPa) = 300 x \( M_r \)
- From vane shear test, \( c \) is directly measured

The procedure requires the use of curves for aggregate thickness for the expected single tyre pressure, the allowable rut criterion (typically no more than 75 mm for pavement structural section construction) and the number of vehicles loads anticipated during construction (typically less than 100 ESAs). For access roads, greater traffic should be anticipated.
STEP 3. Check the aggregate gradation for compatibility with the geogrid and the subgrade or use a geogrid/geotextile geocomposite and check the separation/filtration requirements for the geotextile used in the geocomposite based on the following criteria. The important measures include the equivalent opening size (EOS), the permeability (k), and permittivity (ψ) of the geotextile. These values will be compared to a minimum standard or to the soil properties as follows:

- Geogrid / Aggregate Compatibility
  - Aperture Size $\geq D_{50}$ of aggregate above geogrid
  - Aperture Size $\leq 2D_{85}$ of aggregate above geogrid

- Aggregate / Subgrade Separation Compatibility
  - $D_{15}$ of aggregate above geogrid $< 5D_{85}$ subgrade, and
  - $D_{50}$ aggregate / $D_{50}$ subgrade $\leq 25$
  - Otherwise use separation geotextile with geogrid (i.e., a geocomposite)

- Nonwoven separation geotextile
  - $EOS \leq 1.8 D_{85}$ but no greater than 0.3 mm
  - $k_{geotextile} \geq k_{soil}$
  - $\psi \geq 0.1$ sec$^{-1}$

STEP 4. Determine geogrid survival criteria. The design is based on the assumption that the geosynthetic cannot function unless it survives the construction process. For geogrid survivability, survivability requirements and the opening requirements for interlock and separation shown in Table 1 have been developed by the US, the Federal Highway Administration (FHWA) (Holtz et al., 2008). For stabilisation of soils, the default is Class 1. These requirements may be reduced to Class 2 based on conditions and experience, as detailed in the footnotes in table. Austroads (2009) provides additional explanation of the survivability class in terms of ground conditions, construction equipment and stabilisation aggregate lift thickness.

STEP 5. Determine the support value from the geogrid and additional stabilisation aggregate using one of the following two methods from Austroads Part 2: Pavement Structural Design.
Table 1: Geogrid survivability property requirements\textsuperscript{1,2,3} for stabilisation and base reinforcement applications (after Holtz \textit{et al.}, 2008).

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Units</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SURVIVABILITY</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultimate Multi-Rib Tensile Strength</td>
<td>ASTM D 6637</td>
<td>kN/m</td>
<td>\textbf{18}</td>
</tr>
<tr>
<td>Junction Strength\textsuperscript{5}</td>
<td>ASTM D 7737</td>
<td>N</td>
<td>\textbf{110}\textsuperscript{5}</td>
</tr>
<tr>
<td>Ultraviolet Stability (Retained Strength)</td>
<td>AS 3706.11</td>
<td>%</td>
<td>50% after 500 hours of exposure</td>
</tr>
</tbody>
</table>

\textbf{Geogrid Class}

<table>
<thead>
<tr>
<th>CLASS 1\textsuperscript{4}</th>
<th>CLASS 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>18</td>
<td>12</td>
</tr>
</tbody>
</table>

\textbf{NOTES:}

1. Acceptance of geogrid material shall be based on ASTM D 4759.
2. Acceptance shall be based upon testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer’s certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354.
3. Minimum; use value in weaker principal direction. All numerical values represent minimum average roll value (\textit{i.e.}, test results from any sampled roll in a lot shall meet or exceed the minimum values in the table). Lot samples according to ASTM D 4354.
4. Default geogrid selection. The engineer may specify a Class 2 geogrid for moderate survivability conditions, based on one or more of the following:
   a) The Engineer has found the class of geogrid to have sufficient survivability based on field experience.
   b) The Engineer has found the class of geogrid to have sufficient survivability based on laboratory testing and visual inspection of a geotextile sample removed from a field test section constructed under anticipated field conditions (see note 5).
5. Junction strength requirements have not been fully supported by data, and until such data is established, manufacturers shall submit data from full scale installation damage tests in accordance with ASTM D 5818 documenting integrity of junctions. For soft soil applications, a minimum of 150 mm of cover aggregate shall be placed over the geogrid and a loaded dump truck used to traverse the section a minimum number of passes to achieve 100 mm of rutting. A photographic record of the geogrid after exhumation shall be provided, which clearly shows that junctions have not been displaced or otherwise damaged during the installation process.
Method 1 – Equivalent CBR = 3%

Use an equivalent CBR = 3% (resilient modulus of 30 MPa) and design the pavement without consideration of a geosynthetic and stabilisation aggregate as illustrated in the following Figure 2.

Figure 2. Pavement Section showing Equivalent Subgrade CBR = 3% provided by Geogrid plus Stabilisation Aggregate

Method 2 – Mechanistically Derived Subgrade Support

According to the Austroads 2012 Guide, the stabilisation layer used for construction is modelled as a Selected Subgrade Material (i.e. as opposed to a Normal Granular Material) as illustrated in the following Figure 3 (refer to the Austroads Guide Part 2 - Section 8.2.2 Procedure for Elastic Characterisation of Selected Subgrade Materials).

Figure 3. Pavement Section showing Mechanistically Derived Stabilisation Support
As reviewed in the Austroads 2012 (Guide Section 8.2.2) the modulus of granular materials used for the stabilisation layer is dependent on the intrinsic characteristics of the materials, on the stress level at which they operate, and the stiffness of the underlying layers. Due to the difficulty in performing numerical models, which will account for all of these factors, for mechanistic modelling, the stabilisation layer should be divided into five sublayers.

- The vertical modulus of the top sublayer \( E_v \) is the minimum of 10 times the design CBR of the selected subgrade material and that dependent on the support provided by the underlying material (i.e. in situ subgrade or another selected subgrade material) determined using:

\[
E_{v\text{ granular sublayer}} = E_{v\text{ subgrade}} \times 2^{(\text{total granular thickness} / 150)}
\]

The vertical modulus of the subgrade \( E_{v\text{ subgrade}} \) is determined from field or laboratory tests and the total granular thickness is the stabilisation aggregate thickness, \( t_1 \), (i.e., with the geogrid) determined using one or more of the chart methods included in Appendix B. For example, \( E_{v\text{ subgrade}} \) for a subgrade with a CBR = 1% is approximately equal to 10 MPa. Using a geogrid, the required stabilisation aggregate layer thickness = 300 mm for construction traffic of less than 100 passes with a loaded 90 KN axle load (see next section Example 2 and Figures B2 and B3a in Appendix B). Therefore, \( E_v = 10 \times 2^{(300/150)} = 40 \) MPa and correspondingly a CBR = 4% for the stabilisation layer. This is the maximum \( E_v \), but this value ignores any additional improvement from the geogrid and is considered to be appropriate for the analysis of the pavement section. As previously indicated, the geogrid provides a confining effect at the bottom of the layer through lateral restraint of the aggregate, improving the modulus of the gravel in the vicinity of the geogrid and effectively decreasing strain at the subgrade level. The actual modulus of the geogrid-stabilisation layer could be verified in the field (e.g., using standard falling weight deflectometer, FWD, a light FWD, or cyclic plate load tests) and used in the analysis.

- The ratio of moduli of adjacent sublayers is given by:

\[
R = \left[ \frac{E_{\text{top granular sublayer}}}{E_{\text{subgrade}}} \right]^{1/5}
\]

- The modulus of each sublayer is then calculated from the modulus of the adjacent underlying sublayer, beginning with the subgrade or upper sublayer of selected subgrade material as appropriate, the modulus of which is known. A check needs to be made that the vertical modulus calculated for each sublayer does not exceed the maximum modulus the granular material in the sublayer can develop due to its intrinsic characteristics (id. Sections 6.2.2 and 6.2.3 of the Guide). If this condition is not met, a material with a higher modulus needs to be used in this sublayer or an alternative pavement configuration selected.
- The other stiffness parameters required for each granular sublayer including the horizontal modulus $E_h$ and the stress parameter $f$ may be calculated from the following relationships:

$$E_h = 0.5 E_v$$

$$f = \frac{E_v}{(1 + v_v)}$$

Where, $v_v$ = the vertical Poisson’s ratio of the granular sublayer.

STEP 6. Design the pavement using CIRCLY and an effective subgrade support value determined in step 5.

3. DESIGN EXAMPLES

The following provides two design examples to illustrate the steps required for each approach. The first example provides a review of the steps required for determining the geogrid and stabilisation lift requirements and illustrates the use of Method 1 for the design of the pavement structural section. The second example focuses on the Method 2 approach and the use of CIRCLY to determine the effectiveness of the stabilisation layer and the pavement requirements. The examples also provide a comparison of the increased aggregate that would be required to design the pavement section without a geogrid beneath the stabilisation layer; however, it should be recognized that construction of this section may also require a separation layer to prevent the stabilisation aggregate from penetrating the subgrade, thus reducing the section, and fine grain soils (sils and clays) from migrating into the stabilisation aggregate over time reducing its resilient modulus and drainage characteristics.

**Example 1 - Method 1 – Equivalent CBR = 3%**

**GIVEN DATA**

- Traffic: Urban Traffic Mix (SAR/DESA=1.6)
  - Design ESAs = $10^6$

- Subgrade: surficial soils: high plasticity silt with sand, MH (CBR = 1%)
  - Gradation tests: $D_{85} = 1.37\text{mm}$, $D_{50} = 0.045\text{mm}$, $D_{15} = 0.01\text{mm}$
  - low-lying topography with poor drainage
  - other nearby streets and roads require frequent maintenance

- Aggregate: angular base and stabilisation aggregate is locally available
  - Base Course Modulus, $E_v = 500\text{ MPa}$
SOLUTION

STEP 1. Identify properties of the subgrade

Low CBR, saturated subgrade, and poor performance history with conventional design.

STEP 2. Determine additional aggregate thickness $t_1$, with the geogrid and $t_0$, without the geogrid

Use chart solutions in Appendix B.

Assume:
- $CBR = 1\%$
- loaded highway legal tip trucks
- $< 100$ passes
- 3 in. (75 mm) rut depth acceptable

Solution:
- $c = 30 \text{ kPa} \times CBR = 30 \text{ kPa}$

From Appendix B, Figure B1 (80 kN axle, 750 kPa tire pressure)
- $t_1$ aggregate depth with geogrid = 240 mm
- (Note: $t_0$ aggregate depth without geogrid = 380 mm)

From Appendix B, Figure B3a (45 kN Single Wheel Load, 90 kN Axle load, 550 kPa tyre pressure)
- $N_c = 5.8$ (e.g., for moderate 50-75 mm rutting and $< 100$ passes)
- $c \times N_c = 30 \text{ kPa} \times 5.8 = 174 \text{ kPa}$
- $t_1$ aggregate depth with geogrid from Figure B3a = 300 mm
- (Note: $t_0$ aggregate depth without geogrid from Figure B3a = 450 mm based on $cN_c = 30 \text{ kPa} \times 3 = 90 \text{ kPa}$)

The two stabilisation design approaches are in good agreement for the given conditions with Figure B3a providing more conservative results for the assumed higher axle load.

STEP 3. Check the aggregate gradation for compatibility with the geogrid and the subgrade

For biaxial geogrids, standard Aperture Size = 25 to 35 mm, use 32 mm
Guidelines Document on the Use of CIRCLY to Evaluate the Benefits of Geogrids for Subgrade Stabilisation in Permanent Roadway Design

- Geogrid / Aggregate Compatibility
  - Aperture Size $\geq D_{50}$ of aggregate above geogrid
  - Aperture Size $\leq 2D_{95}$ of aggregate above geogrid

  Therefore, aggregate should have $D_{85} \geq 16$ mm and $D_{50} \leq 32$ mm, or an alternate geogrid must be used.

- Aggregate / Subgrade Separation Compatibility
  - $D_{15}$ of aggregate above geogrid $\leq 5D_{85}$ subgrade, and
  - $D_{50}$ of aggregate above geogrid $\leq 25D_{50}$ subgrade

  For the silt type soil with sand, $D_{85} = 1.37$ mm and $D_{50} = 0.045$ mm,

  Therefore, either the aggregate should have $D_{15} \leq 6.8$ mm and $D_{50} \leq 1.1$ mm, or use a geocomposite (geogrid + separation geotextile) (e.g., NAUE Combigrind)

- For geocomposite, the nonwoven separation geotextile requirements are:
  - $EOS \leq 1.8 D_{85} \leq 2.5$ mm, but no greater than 0.3 mm

    Therefore use $EOS \leq 0.3$ mm

  - $k_{geotextile} \geq k_{soil}$

    For fine grain silt type soil the permeability is $\sim 10^{-5}$ cm/sec, which practically all geotextiles can meet, therefore permittivity, $\psi$, requirement will control

  - $\psi \geq 0.1$ sec$^{-1}$

STEP 4. Determine geogrid survival criteria.

Use Table 1 and the values for the default Class 1, high survival geogrids
- Ultimate Multi-Rib Tensile Strength (ASTM ASTM D 6637) $\geq 18$ kN/m
- Junction Strength (ASTM D 7737) $\geq 110$ N
- Ultraviolet Stability (Retained Strength) (AS 3706.11) $\geq 50\%$ after 500 hours of exposure

STEP 5. Determine the support value from the geogrid and additional stabilisation aggregate

Use Method 1 – Equivalent CBR = 3% for geogrid + 300 mm of stabilisation aggregate.
STEP 6. Design the pavement using CIRCLY and an effective subgrade support value determined in step 5.

Reference Case (without Geogrid):

Asphalt (not modelled)
Base Course Layer
Stabilisation Aggregate
No Geogrid
Subgrade

\[ t_0 = 450 \text{ mm} \]
\[ \text{CBR}_{\text{Equiv}} = 3\% \]

Design Example (including Geogrid and Stabilisation Aggregate):

Asphalt (not modelled)
Base Course Layer
Stabilisation Aggregate
Geogrid
Subgrade

\[ t_1 = 300 \text{ mm} \]
\[ \text{CBR}_{\text{Equiv}} = 3\% \]
Example 2 - Method 2 – Mechanistically Derived Subgrade Support

As previously indicated this example focuses on the use of CIRCLY to determine the effectiveness of the stabilisation layer and the pavement requirements. The first four steps should be worked out the same as in the previous example.

Assume:
Design ESAs = 10^6
Urban Traffic Mix (SARu/DESA=1.6)
Base Course Modulus, $E_v = 500$ MPa
Subgrade CBR = 1%

Reference Case (without Geogrid):
(Standard evaluation using CIRCLY for the analysis)

Asphalt (not modelled)
Base Course Layer
Additional Base Course required by CIRCLY
No Geogrid
Subgrade

Design Example (With Geogrid and Stabilisation Aggregate):

Asphalt (not modelled)
Base Course Layer
Stabilisation Aggregate
Geogrid
Subgrade

Note: for modelling Stabilisation Aggregate modulus, $E_v = 40$ MPa (see calculations in Section 2, Step 5 on page 9). Again, a higher value may be used if verified in the field (e.g., using FWD, LFWD, or cyclic plate load tests).
The following provides the CIRCLY 5 screen shots for the analysis of the above pavement structural section using the mechanistically derived subgrade support.

**Assumed Traffic**

$10^6$ ESAs

**Layered System**

**Pavement Design**

- Our goal is to design the thickness of the Base layer
- We use a trial thickness (say, 600 mm) for the initial CIRCLY analysis

**Using Trial thickness for Base layer**
Pavement Design

- Using the trial Base Layer thickness (600 mm) gives maximum CDF = 0.287 for the subgrade. The goal is for the maximum CDF to be as close as possible to 1.0, but less than 1.0.

- We now use CIRCLY's *Automatic Thickness Design* feature to determine the Base Layer thickness.
Using Automatic Thickness Design to determine design thickness of Base layer

1. Run Analysis

2. Unrounded Design Thickness

- Design thickness of layer highlighted below
- Minimum Thickness
- Maximum Thickness
- Current Thickness
- CDF

<table>
<thead>
<tr>
<th>No.</th>
<th>ID</th>
<th>Title</th>
<th>Minimum Thickness</th>
<th>Maximum Thickness</th>
<th>Current Thickness</th>
<th>CDF</th>
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<tbody>
<tr>
<td>1</td>
<td>Gran_500</td>
<td>Granular, E=500 MPa</td>
<td>518.58</td>
<td>518.58</td>
<td>518.58</td>
<td>0.985</td>
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<td>2</td>
<td>substCB4</td>
<td>Select Subgrade, CBR=4</td>
<td>300.00</td>
<td>300.00</td>
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<td>1.81E+01</td>
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<td>3</td>
<td>Sub_CBR1</td>
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<td>0.00</td>
<td>0.00</td>
<td>1.00E+00</td>
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</table>

Performance Criteria and Traffic multipliers:
- Use in Max CDF
- Material Type
- Performance Criterion
- Multipliers

<table>
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<tr>
<th>No.</th>
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<th>Material Type</th>
<th>Performance Criterion</th>
<th>Multipliers</th>
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<tr>
<td>2</td>
<td>Yes</td>
<td>Subgrade</td>
<td>Austroads, 2004</td>
<td>1.60</td>
</tr>
<tr>
<td>3</td>
<td>Yes</td>
<td>Subgrade</td>
<td>Subgrade failure criterion (Austroads, 2004)</td>
<td>1.60</td>
</tr>
</tbody>
</table>

Pavement Design

- This gives the Base Layer thickness as 518.58 mm
- We now round the Base Layer thickness up to 520 mm and re-analyse

3. Run Analysis

4. Confirm CDF < 1.0

- Using Base Layer thickness = 520 mm gives the maximum CDF = 0.985
4. **STEP 7. CONSTRUCTION - Important Notes.**

Field installation procedures introduce a number of special concerns. Concerns and criteria for field installation include, for example, the overlap requirements, construction sequencing and quality control. Recommendations for construction are included in Appendix C and should be incorporated into construction specifications, modified to suit local conditions and contractors. Key items related to performance of the geogrid-aggregate stabilisation layer include:

- Always maintain a minimum of 150 mm aggregate between construction equipment and geogrid
- Track in the first lift with a continuous pass of the dozer before compaction of the stabilisation lift.
- Condition the compacted geogrid-stabilisation layer before placing subsequent pavement layers by proof rolling with 3 to 4 passes of a heavily loaded, rubber-tyre vehicle such as a loaded dump truck. Conditioning seats the aggregate into the geogrid and prestresses aggregate-geogrid system, optimizing its performance.
- Fill in all ruts, never blade them down.

**REFERENCES**


Guidelines Document on the Use of CIRCLY to Evaluate the Benefits of Geogrids for Subgrade Stabilisation in Permanent Roadway Design

APPENDIX A

REVIEW OF SUBGRADE STABILISATION
(after Perkins et al., 2011)

Geosynthetics are placed on weak, often saturated subgrades in combination with a granular layer to stabilise the subgrade and to provide a working platform for construction of unpaved roads and either flexible or rigid pavement systems. The granular layer may be the base or subbase component of the pavement system or it may be additional gravel required to support initial construction and not considered as part of the pavement section. Soft, weak subgrade soils provide very little lateral restraint. When roadway aggregate moves or shoves laterally, vertical deformation occurs and ruts develop on the aggregate surface. A geogrid with good interlocking capabilities can provide reinforcement in the form of tensile resistance to lateral aggregate movement. The reinforcement provides lateral restraint and initiates increased horizontal stress on the aggregate, thus reducing the mobilization of the soil and reducing plastic deformations (i.e., reduced rutting). The geosynthetic also increases the system bearing capacity by forcing the potential bearing surface under the wheel load to develop along an alternate, longer mobilization path and thus higher shear strength surfaces.

Other functions of the geosynthetic play an equally important role in this application. The geosynthetic must provide separation to prevent intermixing of the granular material and the subgrade. Only a small amount of fines penetrating into the granular layer will negatively affect its structural characteristics (i.e., reduced shear strength, lowered permeability and increased frost susceptibility). Geogrids prevent aggregate penetration into the subgrade, depending on the ability of the geogrid to confine and prevent lateral displacement of the base/subbase. However, the geogrid does not prevent intrusion of subgrade soils up into the base/subbase course and the granular layer used with the geogrid must provide this function or a geotextile must be used in combination with the geogrid.

The subgrade soils in this application are fine grained soil with high water content. The granular layer supported by the geogrid must have a gradation that is compatible with the subgrade, based on standard geotechnical graded granular filter criteria. If a geotextile separation layer is used with a geogrid, must also provide filtration to allow excess pore water pressure to dissipate into the aggregate base course and, in cases of poor-quality aggregate, through the geotextile plane itself. It is the reinforcement, separation, filtration and drainage functions that combine to provide the mechanical stabilisation for weak subgrade soils (Holtz, et al., 2008).

Geosynthetics are primarily used in stabilisation applications to facilitate construction. Even if the finished roadway can be supported by the subgrade, it may be virtually impossible to begin construction of the embankment or roadway without some form of stabilisation. Geosynthetics offer a cost-effective alternative to other expensive foundation stabilisation methods such as dewatering, demucking, excavation and replacement with select granular materials, utilization of thicker stabilisation aggregate layers, or chemical stabilisation. This mechanically stabilised layer also enables contractors to meet minimum compaction specifications for the first two or three aggregate lifts.

While the stabilisation application is primarily used for initial construction, geosynthetics also provide long-term benefits and improve the performance of the road over its design life. The geosynthetic continues to perform by maintaining the roadway design section and the base course material integrity by preventing the aggregate from penetrating the subgrade. In addition, the separation function provided by geotextiles, geogrid/geotextile geocomposites or
geogrids with appropriately designed filter aggregate prevent the migration of fines into base/subbase materials, especially into open graded bases, maintaining the support and drainage characteristics of the base over the life of a pavement system. In essence, the geosynthetic improves the reliability of the pavement system performance, especially during overload and/or seasonally weak subgrade conditions. Thus, the geosynthetic should ultimately increase the life of the roadway.

1 Mechanisms of reinforcement

The two primary mechanisms with this application are increased bearing capacity and lateral restraint, both of which significantly contribute to load-carrying capacity. When an aggregate layer is loaded by a vehicle wheel or dozer track, the aggregate tends to move or shove laterally and is restrained by the subgrade or geosynthetic reinforcement, a pavement reinforcement mechanism as shown in Figure 1a. Components of this mechanism include: (i) restraint of lateral movement of base, or subbase, aggregate (confinement); (ii) increase in modulus of base aggregate due to confinement; (iii) improved vertical stress distribution on subgrade due to increased base modulus; and (iv) reduced shear strain along the top of the subgrade. In addition, the geosynthetic reinforcement forces the potential bearing capacity failure surface below the wheel load, which is analogous to a footing on foundation soil, to follow an alternate higher strength path as shown in 1b. This tends to increase the bearing capacity of the subgrade soil.

A third possible geosynthetic reinforcement function is membrane-type support of wheel loads, as shown conceptually in Figure 1c. In this case, the wheel load stresses must be great enough to cause plastic deformation and ruts in the subgrade. If the geosynthetic has a sufficiently high tensile modulus, tensile stresses will develop in the reinforcement, and the vertical component of this membrane stress will help support the applied wheel loads. As tensile stress within the geosynthetic cannot develop without some elongation, wheel path rutting (in excess of 100 mm as determined by Giroud and Noriay, 1981) is required to develop membrane-type support. Therefore, this mechanism is generally limited to temporary roads or the first aggregate lift in permanent roadways, if significant rutting can be tolerated. These reinforcement mechanisms were originally described by Bender and Barenberg (1978) and later elaborated on by Kinney and Barenberg (1982) for geotextile-reinforced unpaved roads.

The influence of each reinforcement mechanism will diminish with stronger subgrade conditions and as additional layers of base and the pavement system are placed. The effect of the reinforcement also increases with increasing acceptable deformation (rutting). As previously indicated, when little or no rutting of the subgrade occurs, the membrane tension support does not exist. Also, with stronger subgrades, bearing capacity is not an issue and lateral movement of the gravel under construction equipment diminishes. Research indicates that stabilisation for construction is generally no longer required for subgrade soils with a soaked CBR value approximately greater than three to four (CBR < 3 to 4%), shear strengths greater than approximately 90 to 120 kPa, and resilient modulus greater than approximately 30 to 40 MPa. From a foundation engineering point of view, clay soils with undrained shear strengths of 2000 psf (90 kPa) or higher are considered to be stiff clays (Terzaghi and Peck, 1967) and are generally quite good foundation materials. Simple stress distribution calculations show that for static loads, such soils will readily support reasonable truckloads and tire pressures, even under relatively thin granular bases.
Figure 1. Possible reinforcement functions provided by geosynthetics in roadways: (a) lateral restraint, (b) bearing capacity increase, and (c) membrane tension support (Holtz et al., 2008, after Haliburton, et al., 1981).

As the thickness of the gravel layer increases or stiffer components of the pavement section are added, the stress at the geosynthetic decreases to a point where there is little or no geosynthetic deformation and correspondingly little or no reinforcement. The actual thickness where this occurs is related to the subgrade strength, the type and magnitude of the wheel load, and the number of vehicle passes. Thus design solutions should evaluate each of these elements.

Again, the reinforcing function can be compromised if separation and filtration are not provided. Several case histories have documented poor performance of reinforcements, when the separation function was not achieved. For example, the US Army Corps of Engineers
District in Baltimore documented a subgrade restraint failure due to separation problems in a test strip pilot study to evaluate optimal subgrade stabilisation (US Army Corps of Engineers, 1999). Severe rutting and localized bearing failures were attributed to intermixing between the subgrade soils and a well graded granular base in a test section where a geogrid was placed directly on the subgrade. Separate test strips under the same conditions found that a stabilisation geotextile performed adequately with some rutting and a geogrid over a separation geotextile performed with minimal rutting. Another indication of the importance of these functions is the number of research studies that have identified varying performance by different geosynthetics as discussed in the next section on full scale performance.

2 Full Scale Performance

The use of geosynthetics for subgrade stabilisation to solve problems encountered in constructing unpaved and paved (both flexible and rigid pavements) roads over soft, wet subgrades was well established internationally in the 1970s. The performance of geosynthetics used in stabilisation applications in low-volume roads have been well documented in numerous case histories, full-scale laboratory experiments, and instrumented field studies, some of which include Steward et al. (1977), Bender and Barenberg, (1978), Haliburton and Barron (1983), Haas et al. (1988), Austin and Coleman (1993), Tsai (1995), Fannin and Sigurdsson (1996), Knapton and Austin (1996), Hayden et al. (1999), Gabr et al. (2001), Leng and Gabr (2002), Tingle and Webster (2003), Hufenus et al. (2004), Watts et al. (2004), Christopher and Lacina (2008), and Christopher and Perkins (2008). Summaries of much of this research is contained in Berg et al. (2000), Christopher et al. (2001), Watn et al. (2005) and in the European study Cost 348 WG1 (2004).

As indicated in the previous section, the results of these studies vary in terms of the performance of different geosynthetic types. For example, in some studies geogrids were found to perform better than geotextiles (e.g., Barksdale et al., 1989); in some studies, geotextile and geogrid performance has been found to be essentially the same (e.g., Fannin and Sigurdsson, 1996 and Hayden et al. 1999); in others, geotextiles were found to perform better than geogrids (Al-Qadi et al., 1994 and Christopher and Lacina, 2008); and in all cases, where composite geogrid/geotextile systems were used, they always performed the best (Fannin and Sigurdsson, 1996, Christopher and Perkins, 2008, and Christopher and Lacina, 2008). Recent work by Christopher et al. (2009) has found that these varying results may be the result of pore water pressure development and the ability of pore water pressure to dissipate during loading is a critical factor to performance.

In full scale laboratory tests performed to evaluate geosynthetics used in both stabilisation and base reinforcements on a number of different geosynthetics in several separate studies (Perkins et al., 2004, Christopher and Lacina, 2008, and Christopher et al., 2009) have observed the development and increase of pore water pressure measured in the wet, nearly saturated subgrade during cyclic loading. As indicated in these references and the representative results shown in Figure 2, the pore water pressure measurements in most of the tests were found to directly correspond to the performance of the geosynthetic.
The largest amount of deformation per cycle was found to occur in the tests with the highest developed pore pressure (e.g., the control tests in Figure 2) and the best performing tests (least amount of rutting under the same number of cycles) were in the sections with the lowest measured pore pressure (e.g., see Figure 2). The full scale studies found that the reinforcement action of an open geogrid (e.g., GG\textsubscript{wd-pp} in Figure 2) positively results in lower pore water pressure development than measured in control tests (i.e., with no geosynthetics) performed on the same subgrade. The addition of a nonwoven geotextile to the reinforcement geogrid (e.g., GC\textsubscript{gg-nwgt} in Figure 2) provides additional separation and filtration features that further limit the development of excess pore water pressure and significantly further reduces rutting.

**Figure 2.** Representative results from full scale laboratory stabilisation tests on geogrids and silt type subgrade with pore pressure measurements (Christopher and Perkins, 2009)
These results indicate that the performance of the geosynthetics varies with both the subgrade type and conditions (i.e., a geosynthetic may perform well in one condition and not so well under other conditions). Geosynthetics could influence the development and magnitude of pore water pressure through: 1) a reduction in stress in the subgrade (Berg et al., 2000); 2) separation, which would reduce point stress and corresponding pore pressure developed from gravel penetration into subgrade layers (Christopher and Lacina, 2008); and/or, 3) pore pressure dissipation in the plane of some geosynthetics when the in plane permeability is greater than the permeability of the base layer (e.g., poorly draining base layers containing fine grained soils) (Holtz et al., 2008).

Most of these studies are focused on short-term (i.e., during construction and initial traffic) performance. Several studies are currently underway to monitor long-term performance. The full scale test section constructed by the Maine DOT had several test sections using stabilisation geotextiles for comparison with the geogrid base reinforcement test sections (Hayden et al. 1999). Stabilisation aggregate required in the control section was eliminated in all of the geosynthetic test sections. Monitoring of a Washington DOT pavement test section in which separation/stabilisation geotextiles were used is anticipated to be continued over the full pavement design life (Black and Holtz 1997). Al-Qadi and Appea (2003) also reported on an eight year study investigating the effects of geogrid and geotextile reinforcement placed between the base course and subgrade. In the first eight years of performance, only the thinnest, 100 mm thick base course has realized a measurable increase in service life and pavement quality. A stabilisation research project is currently being conducted by the University of Wisconsin in cooperation with the Wisconsin DOT, in which geosynthetic stabilisation test sections are being compared with sections stabilised using fly ash and bottom ash (Woon-Hyung et al., 2005). The geosynthetic sections are instrumented, strain gages are mounted on the geosynthetics and a control section was installed. The Geosynthetic Research Institute (GRI) has an ongoing study to monitor test sections where geosynthetics have been used as separators (GRI, 2001). Although the study is focused on the long-term benefits of geotextile separators, in many of the projects that are being monitored, geosynthetics were initially used for subgrade stabilisation. A database of full-scale field test sites has been developed and is maintained at GRI. Monitoring is proposed for up to 20 years.

3. Geosynthetic Material Properties and Tests

As with any geosynthetic application, the material properties required for design are based on: 1) the properties required to perform the primary and secondary function(s) for the specific application over the life of the system, and 2) the properties required to survive installation. The separation and filtration functions are related to opening characteristics and are determined based on the gradation of the adjacent layers (i.e., subgrade, base and/or subbase layers). Some strength is, of course, required for the reinforcing function, which is based on the requirements in the specific design approach. If the roadway system is designed correctly, then the stress at the top of the subgrade due to the weight of the aggregate and the traffic load should be less than the bearing capacity of the soil plus a safety factor, which is generally a relatively low value compared to the strength of most geosynthetics. However, the stresses applied to the subgrade and the geosynthetic during construction may be much greater than those applied in-service. Therefore, the strength of the geosynthetic in roadway applications is usually governed by the anticipated construction stresses and the required level of performance. This is the concept of geosynthetic survivability -- the geosynthetic must survive the construction operations if it is to perform its intended function. For subgrade stabilisation, the geosynthetic survivability tends to control the strength requirements and not the reinforcement function.
In the US, the Federal Highway Administration (FHWA) (Holtz et al, 2008) and the American Association of Highway and State Transportation Officials (AASHTO) (AASHTO M288, 2006) provide tables specifically for stabilisation applications that relate geosynthetic index properties defined by the American Society for Testing and Materials (e.g., wide width strength and strength for geogrids) to survivability of geosynthetics. The geosynthetics are classified as High (Type 1), Moderate (Type 2) and Low (Type 3) survivability geosynthetics and the types are matched to specific installation conditions. Opening characteristics for geogrids based on the relation to the granular layer particle size and for geotextiles based on separation and filtration requirements are also included in the tables plus permittivity requirements are specified for geotextiles.

A similar approach has been developed in the Nordic countries of Finland, Sweden and Norway through the establishment of the NorGeoSpec system in 2002 (Moe, 2008). In Denmark, the road design procedure equally requires that the geosynthetic be flexible and strong enough to withstand stresses from the surrounding gravel or rocks without being damaged. The requirement for the geotextiles could be based on a design model such as that proposed by Steen (2004).

Other properties, such as stiffness, aperture size and interlock effect, may be required for the specific design method as discussed in Section 5.2. Almost no correlations have been developed between properties and field performance of geosynthetics in subgrade stabilisation applications. In order to develop such correlations, Berg et al, 2000 has recommended that the following properties of interest be provided with any future full scale studies or long-term pavement studies: 2% & 5 % secant moduli, Coefficient of Pullout Interaction, Coefficient of Direct Shear, Aperture Size, and Percent Open Area. A proposal for guidelines for reinforcement in road structures in Norway emphasizes the necessity of linking the reinforcement function to the dominating deterioration mechanisms. The required reinforcement properties are then given based on an evaluation of different deterioration mechanisms. Generally the required properties can be grouped into strength and stiffness, interaction properties with surrounding material and survivability (Øiseth and Hoff, 2006).

4. Subgrade Stabilisation (Design)

Design methods for geosynthetic reinforcement in pavement sections are either based on empirical and analytical considerations or analytical models modified by experimental data. Summaries of much of this research are contained in Berg et al., 2000. Geosynthetic stabilised unpaved roads are typically designed using methodologies based on not exceeding the bearing capacity of the underlying subgrade materials (e.g., Bender and Barenberg, 1978; Giroud and Noiray, 1981; Steward et al., 1977). Empirical modifications to bearing capacity theory are generally added to account for the level of traffic for which the roadway should be designed.

Several design methods also exist for the use of reinforcement geosynthetics in subgrade restraint (or stabilisation) for permanent road construction. Since some rutting is allowed for the initial lift in constructing these sections, these design techniques mainly rely on the tensioned membrane approach and bearing capacity theory to analyse the reinforcing requirements for these sections. The two most widely used procedures in the US are the Steward et al. (1977) procedure and the Giroud and Han (2004) procedure.

The Steward et al. (1977) procedure was developed from an earlier method by Barenberg et al. 1975. Based on lab tests and supported by bearing capacity theory, Barenberg and his colleagues proposed that the bearing capacity factor, $N_c$ (to prevent significant permanent deformation under a small amount of traffic) increased from 3.3 to 6.0 when using a geosynthetic. Based on field tests, Steward et al. (1977) extended this approach for U.S. Forest
Guidelines Document on the Use of CIRCLY to Evaluate the Benefits of Geogrids for Subgrade Stabilisation in Permanent Roadway Design

Service unpaved roads to cover situations where very little rutting was tolerable under high traffic levels (>1000 ESALs) by including an Nc of 2.8 without a geotextile and 5.0 with a geotextile. With these extended factors, both traffic and rut depth could be considered for a variety of wheel loading conditions. The approach was later adopted by the Federal Highway Administration for both temporary roads and the working platform for permanent roads (Christopher and Holtz 1985) and is still in use by the FHWA today (Holtz et al. 2008). The solution of Steward et al. (1977) was modified by Tingle and Webster (2003) for geogrid reinforcement and the proposed modification was adopted in the COE method for design of geotextile- and geogrid-reinforced unpaved roads (USCOE, 2003).

A key feature of the FHWA method is the assumption that the structural pavement design is not modified at all in the procedure. The geosynthetic instead replaces additional unbound material that might be placed to support construction operations, and replaces no part of the pavement section itself. However, this reinforced unbound layer will provide some additional support. If the soil has a CBR of less than 3%, and the aggregate thickness is determined based on a low rutting criteria, the support for the composite system is theoretically equivalent to a CBR = 3% (resilient modulus of 30 Mpa). As with thick aggregate fill used for stabilisation, the support value should be confirmed though field testing using, for example, a cyclic plate load test or FWD test to verify that a minimum composite subgrade modulus has been achieved.

Another method included in the FHWA design manual is the Giroud and Han procedure. Utilizing previous research, Giroud and Han (2004) developed theoretically based and experimentally calibrated design method for geogrid-reinforced unpaved roads that reflects the improvements due to the geogrid-aggregate interlock. They built upon earlier design methods developed by Giroud and Noiray (1981) and Giroud et al. (1985) using recent field and laboratory test data. Giroud and Noiray (1981) developed an empirical solution for unreinforced unpaved roads using field test data and quantified the benefits resulting from geotextile reinforcement. The solution was based on the limit equilibrium bearing capacity theory with a modification to consider the benefit of the tension membrane effect. The Giroud-Han theoretical formulation takes into account the distribution of stresses, strength of base course material, geogrid-aggregate interlock, and geogrid in-plane stiffness in addition to conditions considered in earlier methods (traffic volume, wheel loads, tire pressure, subgrade strength, rut depth and influence of reinforcing geosynthetics of the failure mode of unpaved roads).

In Europe, several empirically derived design methods also exist. The German “Recommendations for Design and Analysis of Earth Structures using Geosynthetic Reinforcement – EBGEO” supports the use of the Giroud and Noiray method for stabilisation design (German Geotechnical Society, 2011). The Danish road design uses and E-value (elasticity modulus) of the subsoil to determine if a geosynthetic (i.e., geotextile) is required. If the E-value is higher than 30 MPa, geosynthetics may be used, but no value for their use is assigned. If the E-value is less than 30 MPa and geotextiles are used, the E-value can be multiplied by 1.8 and the increased value is inserted in the pavement design charts. This is one of the few national methods that allows a reduction in the pavement section for using a geosynthetic.

A general analytical design solution has not been found that addresses all of the many variables that impact performance and, as a result, has been validated by experimental data (Perkins and Ismeik 1997a and b). All empirical design methods are limited by the conditions associated with the experiments of the study(ies) and demonstrated performance of constructed works. The lab test results must be extrapolated to field conditions for application to design. Several methods are based on obtaining a performance level (TBR or BCR) from a laboratory model test.
Recently work has begun to extend mechanistic-empirical pavement design methodologies to low volume and unpaved roads and account for the benefits of stabilisation (Perkins et al, 2008). Mechanistic-empirical design has the advantage of describing the effect of traffic passes on the accumulation of permanent deformation of the roadway materials. The design method uses the elements described earlier in this section for paved roadway evaluation, involving the use of a finite element response model, which contains material models for the base aggregate, subgrade and geosynthetic reinforcement materials. An empirical damage model for permanent deformation of the aggregate and subgrade materials is used to relate the vertical dynamic strain from the response model to rutting as a function of traffic passes. Laboratory testing of the materials used in the unpaved road provided input parameters for the material models contained in the finite element response model. The design method has been calibrated by comparison of unreinforced model predictions to comparable full-scale laboratory test sections. Additional stabilised tests sections containing a layer of geosynthetic reinforcement were then compared to model predictions to assess the ability of the design model to account for the positive effects of the geosynthetic.

Mechanistic-empirical modeling of the unsurfaced pavement test sections showed good agreement with full scale unreinforced tests. Improvements were made in the model by including steps to account for the reduced excess pore water pressure in the reinforced test section as compared to the unreinforced section. The method used the principals of effective stress to reduce the elastic modulus of the subgrade for the influence of pore water pressure. The method is detailed by Christopher et al., 2009 where they show the relation of the initial undrained strength of the soil, $S_{ui}$, to the final undrained strength of the soil, $S_{uf}$, based on measured pore pressure in full scale tests during cyclic loading, $u_e$, and the effective strength of the soil expressed by the effective friction angle of the soil ($\phi'$) and Skempton's pore water pressure parameter at failure, $A_f$, using the following equation.

\[
    u_e = (S_{ui} - S_{uf}) \left[ \frac{2}{\tan^2(45 + \phi'/2) - 1} + 2A_f \right] 
\]  

(1)

The decrease in undrained shear strength is assumed to be proportional to the decrease in elastic modulus. Using this approach, the decrease in the pore water pressure measurement in the reinforced section versus the control section was found to account for approximately 80 % of the reduced rutting.

An additional 20 % is accounted for by modelling the effects of the reinforcement. Overall, the prediction of rutting using the reinforced model was found to be greater than that seen in the reinforced test section, but showed considerable improvement as compared to the unreinforced section and is regarded as a favourable and successful prediction. This design approach will allow for calibration of empirical models for other variables not included in the empirical evidence (e.g., soil type, reinforcement type and porewater pressure development) and ultimately lead to long term performance predictions.
References


Guidelines Document on the Use of CIRCLY to Evaluate the Benefits of Geogrids for Subgrade Stabilisation in Permanent Roadway Design


Guidelines Document on the Use of CIRCLY to Evaluate the Benefits of Geogrids for Subgrade Stabilisation in Permanent Roadway Design

APPENDIX B

DESIGN CHARTS
The following method developed by Giroud and Noiray (1981) is included in Appendix A of the Austroads 2009 “Design of Geotextile Reinforced Haul Roads and Temporary Unbound Granular Pavements,” which explains the method in detail and provides example charts developed from the method. The following provides charts to cover the range of typical traffic for design of the stabilisation lift for permanent pavement systems. The charts can be extended to other traffic conditions using the equations outlined in Austroads 2009. Note that the Giroud and Noiray allows for additional reduction in granular thickness for an increase in geogrid modulus $J_{2\%} \geq 10$ kN/m; however, the rut depth must be greater than 75 mm in order to mobilize sufficient strain in the geogrid for this additional improvement to be effective. Therefore this additional aggregate reduction is generally not applicable to stabilisation of permanent roads and thus not included in the charts in this appendix. However, a minimum geogrid modulus could improve the performance by reducing the potential for larger ruts. Therefore, a minimum geogrid modulus $J_{2\%} \geq 400$ kN/m is recommended.

**Figure B1. Giroud and Noiray chart for highway trucks showing the variation in granular thickness with subgrade cohesion for 75 mm rut depth**
Figure B2. Giroud and Noiray chart for off-road trucks showing the variation in granular thickness with subgrade cohesion for 75 mm rut depth (after Austroads 2009)
B2 – FHWA 2008
The following design method was developed by Steward, Williamson, and Mohney (1977) for the U.S. Forest Service (USFS) and was adopted by the US Federal Highway Administration in 1987 as a preferred method. As noted in Appendix A, the method was later calibrated for geogrid reinforcements and adopted by the US Army Corps of Engineers (USOCE, 2003). This method allows the designer to consider vehicle passes; equivalent axle loads; axle configurations; tire pressures; subgrade strengths; and rut depths. The following limitations apply:

- the aggregate layer must be
  a) high quality fill (e.g., laboratory CBR based on ASTM D 1883 ≥ 80),
  b) cohesionless (nonplastic);
- vehicle passes less than 10,000;
- geotextile survivability criteria must be considered; and
- subgrade undrained shear strength less than about 90 kPa (CBR < 3%).

As with the Giroud and Noiray (1981) procedure, this method is based on bearing capacity theory and uses the bearing capacity factor \( N_c \) for determining the thickness required with geogrid reinforcement \( t_1 \). \( N_c \) is based on allowable subgrade rutting under construction traffic conditions (typically a rut of 50 to 75 mm is generally acceptable during construction), where:

- \( N_c = 5 \) for a low rut criteria (< 50 mm) and > 1000 passes,
- \( N_c = 5.8 \) for moderate rutting (50 – 75 mm) and < 100 passes, and
- For comparison of required aggregate thickness without a geogrid (i.e., \( t_0 \)): \( N_c = 2.8 \) and 3.0 respectively for corresponding low and moderate rutting criteria.

Enter the curve with appropriate bearing capacity factors (\( N_c \)) multiplied by the design subgrade undrained shear strength (c) to evaluate each required stress level (c\( N_c \)).
Figure B3. Thickness design curves with geosynthetics for a) single and b) dual wheel loads (after Steward et al., 1977, and FHWA-NHI-07-092).
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APPENDIX C

CONSTRUCTION GUIDELINES

(After Holtz et al., 2008)
The following step-by-step procedures should be followed, along with careful observations of all construction activities.

1. The site should be cleared, grubbed, and excavated to design grade, stripping all topsoil, soft soils, or any other unsuitable materials. If moderate site conditions exist, i.e., CBR greater than 1%, lightweight proofrolling operations should be considered to help locate unsuitable materials. Isolated pockets where additional excavation is required should be backfilled to promote positive drainage. Optionally, a geotextile wrapped trench drains could be used to drain isolated areas.

2. During stripping operations, care should be taken not to excessively disturb the subgrade. This may require the use of lightweight dozers or grade-alls for low-strength, saturated, noncohesive and low-cohesive soils. For extremely soft ground, such as peat bog areas, do not excavate surface materials so you may take advantage of the root mat strength, if it exists. In this case, all vegetation should be cut at the ground surface. Sawdust or sand can be placed over stumps or roots that extend above the ground surface to cushion the geogrid. Remember, the subgrade preparation must correspond to the survivability properties of the geogrid.

4. Parallel rolls of geogrids should be overlapped or joined as required.

<table>
<thead>
<tr>
<th>CBR</th>
<th>Minimum Overlap</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 2</td>
<td>300 - 450 mm</td>
</tr>
<tr>
<td>1 - 2</td>
<td>600 - 900 mm</td>
</tr>
<tr>
<td>0.5 - 1</td>
<td>900 mm</td>
</tr>
<tr>
<td>&lt; 0.5</td>
<td>Run overlap pullout test or use mechanical connections (e.g., ties, staples or pins)</td>
</tr>
<tr>
<td>All roll ends</td>
<td>900 mm</td>
</tr>
</tbody>
</table>

5. For curves, geogrids should be cut and overlapped in the direction of the turn.

6. When the geogrid intersects an existing pavement area, the geosynthetic should extend to the edge of the old system. For widening or intersecting existing roads where geogrids have been used, consider anchoring the geogrid at the roadway edge. Ideally, the edge of the roadway should be excavated down to the existing geosynthetic and the existing geosynthetic mechanically connected to the new geosynthetic (i.e., with plastic ties to the geogrid, staples, or pins). Overlaps could also be utilized.

7. Before covering, the condition of the geogrid should be checked for excessive damage (i.e., holes, rips, tears, etc.) by an inspector experienced in the use of these materials. If excessive defects are observed, the section of the geosynthetic containing the defect should be repaired by placing a new layer of geosynthetic over the damaged area. The minimum required overlap required for parallel rolls should extend beyond the defect in all directions. Alternatively, the defective section can be replaced.

8. The base aggregate should be end-dumped on the previously placed aggregate. For very soft subgrades, pile heights should be limited to prevent possible subgrade failure. The maximum placement lift thickness for such soils should not exceed the design thickness of the road.
9. The first lift of aggregate should be spread to the design thickness, but no greater than 300 mm, prior to compaction. At no time should traffic be allowed on a soft roadway with less than 300 mm of aggregate over the geogrid. For extremely soft soils, lightweight construction vehicles will likely be required for access on the first lift. Construction vehicles should be limited in size and weight so rutting in the initial lift is limited to 75 mm. If rut depths exceed 75 mm, it will be necessary to decrease the construction vehicle size and/or weight or to increase the lift thickness. For example, it may be necessary to reduce the size of the dozer required to blade out the fill or to deliver the fill in half-loaded rather than fully loaded trucks.

10. The first lift of base aggregate should be compacted by tracking with the dozer, then compacted with a smooth-drum vibratory roller to obtain a minimum compacted density. For construction of permeable bases, compaction shall meet specification requirements. For very soft soils, design density should not be anticipated for the first lift and, in this case, compaction requirements should be reduced. One recommendation is to allow compaction of 5% less than the required minimum specification density for the first lift.

11. Construction should be performed parallel to the road alignment. Turning should not be permitted on the first lift of base aggregate. Turn-outs may be constructed at the roadway edge to facilitate construction.

12. In order to optimize geogrid reinforcement of the aggregate layer, preconditioning of the geosynthetic to maximize geogrid aggregate interaction should be considered. For precondition, the area should be proofrolled by a heavily loaded, rubber-tired vehicle such as a loaded dump truck. The wheel load should be equivalent to the maximum expected for the site. The vehicle should make at least three to four passes over the first lift in each area of the site. Alternatively, once the design aggregate has been placed, the roadway could be used for a time prior to paving to prestress the geogrid-aggregate system in key areas.

13. Any ruts that form during construction should be filled in to maintain adequate cover over the geogrid. Ruts should never be bladed down, as this would decrease the amount of aggregate cover between the ruts.

14. All remaining base aggregate should be placed in lifts not exceeding 250 mm in loose thickness and compacted to the appropriate specification density.