Chapter 5

Conclusions

5.1 Introduction

During the last couple of decades, geocell reinforcement of soil has been applied in several new and technically challenging applications, some of which tested the boundaries of the current knowledge and understanding of the functioning of these systems. One such application is the proposed use of geocell-reinforced soil to form support packs.

The objective of this study was to investigate the stiffness and strength behaviour of geocell support packs to provide a better understanding of the functioning of geocell support packs under uniaxial loading. This was achieved by studying the constitutive behaviour of the fill and membrane material and their interaction, as well as the influence of multiple cells on the composite structure.

Practical considerations limited this study to one soil, one type of membrane and only one aspect ratio. These limitations were necessary to allow for a manageable project. The knowledge and insight gained and the models and calculation procedures developed as part of this study, however, are not limited to the materials and configuration used in the experimental programme.

This chapter provides the conclusions flowing from the previous chapters. The study contributes to the current knowledge and understanding in the following areas:

- Understanding and modelling of the constitutive behaviour of cycloned gold tailings.
- Understanding and modelling the behaviour of the HDPE membranes under uniaxial loading.

- Understanding and quantifying the constitutive behaviour of soil reinforced with a single geocell.
- Understanding and quantifying the influence of multiple geocells on the composite behaviour.

5.2 Geocell reinforcement of soil – general conclusions from literature

Although the research that has been performed on geocell reinforced soil encompass a wide variety of geometries and loading mechanisms, there seems to be consensus on several issues from which the following qualitative conclusions can be drawn:

- A geocell reinforced soil composite is stronger and stiffer than the equivalent soil without the geocell reinforcement.
- The strength of the geocell/soil composite seems to increase due to the soil being confined by the membranes. The tension in the membranes of the geocells gives rise to a compression stress in the soil, resulting in an increased strength and stiffness behaviour of the composite.
- The strengthening and stiffening effect of the cellular reinforcement increase with a decrease in the cell sizes and with a decrease in the width to height ratio of the cells. The optimum width to height ratio of the cells seems to be dependent on the specific geometry of the geocell system used in an application.
- The effectiveness of the geocell reinforcement increase with an increase in the density, for a particular soil.
- The strength and stiffness of the geocell reinforced composite increase with an increase in the stiffness of the geocell membranes.

5.3 Classified gold tailings

- *Elastic behaviour*: The non-linear model for the elastic behaviour of the classified tailings, based on the assumption of a linear relationship between the voids ratio and the logarithm of the mean effective stress seem to adequately model the elastic behaviour of the cycloned gold tailings for the higher intermediate and large strain range.

- *The stress-dilatancy theory*: Rowe's stress-dilatancy theory provides a useful framework for the interpretation of the constitutive behaviour of the classified tailings.

- *Dilation*: The dilation parameter at peak, D_{max}, seems to be about 1.6 for the classified gold tailings material in its densest state and therefore does not support the generally accepted assumption that the value of D_{max} is about 2 for sands in their densest state. This could be attributed to the fact that the soil consists mainly of flattened and elongated particles as the flatness of the particles would result in a suppressed dilation behaviour, compared to soils consisting of more rotund particles.

  Bolton's (1986) equation for obtaining the dilation parameter of the material, D_{max}, from the relative density and the mean effective stress, in its current form, seems not to be applicable to the cycloned tailings as it overestimates the dilational behaviour of the cycloned tailings for a particular relative density. Good estimates of the value of ϕ'_{cv} can, however, be obtained from Bolton's work by using measured values of D_{max} and ϕ'.

- *The limiting angles of granular soil*: There seems to be a relationship between the values of the two limiting angles ϕ'_{μ} and ϕ'_{cv} of granular soils, applicable to Rowe's stress-dilatancy theory. This relationship can be approximated by the following polynomial equation:

$$\phi'_{cv} = 0.0001373\phi'^{3}_{\mu} - 0.019\phi'^{2}_{\mu} + 1.67\phi'_{\mu} \qquad (4.21)$$

- *The plastic shear strain at peak*: The value of $(\varepsilon_s^p)_{peak}$ is influenced by the density, the confining stress and the sample preparation method of which the sample preparation method seem to have the largest influence. The plastic shear strain at peak increases with an increase in the confining stress and a decrease in the density.

- *The hardening/softening behaviour of the classified tailings*: The following empirical equation (Equation 4.27) adequately models the pre-peak hardening and the post-peak softening of the classified tailings material:

$$D = \begin{cases} (D_{max} - D_0)\cdot f_1 + D_0 & \varepsilon_s^p \le (\varepsilon_s^p)_{peak} \\ (D_{max} - 1)\cdot f_2 + 1 \quad \text{for} & (\varepsilon_s^p)_{peak} < \varepsilon_s^p \le (\varepsilon_s^p)_{cv} \\ 1 & \varepsilon_s^p > (\varepsilon_s^p)_{cv} \end{cases} \tag{4.27}$$

With:

$$f_1 = \frac{2\cdot\sqrt{\varepsilon_s^p \cdot (\varepsilon_s^p)_{peak}}}{\varepsilon_s^p + (\varepsilon_s^p)_{peak}} \tag{4.30}$$

$$f_2 = 1 - A^2 \cdot (3 - 2\cdot A) \tag{4.31}$$

With:

$$A = \left(\frac{\ln(\varepsilon_s^p) - \ln((\varepsilon_s^p)_{peak})}{\ln((\varepsilon_s^p)_{cv}) - \ln((\varepsilon_s^p)_{peak})}\right)$$

Where:

ε_s^p = the hardening parameter, plastic shear strain,

$(\varepsilon_s^p)_{peak}$ = the plastic shear strain at peak,

$(\varepsilon_s^p)_{cv}$ = the plastic shear strain at which the dilation parameter can be assumed to be 1.

The post-peak softening behaviour of the material seems not to be sensitive to the value of $(\varepsilon_s^p)_{cv}$. The value of $(\varepsilon_s^p)_{cv}$ seems to be constant for the cycloned tailing over the densities and confining stresses under which it was tested.

The strength of the classified tailings is influenced by the particle shearing direction during the shearing process. This component of the material behaviour can be accounted for by assuming the Rowe friction angle, ϕ_f, to change from ϕ'_μ to ϕ'_{cv} as a function of the plastic shear strain in the material as:

$$\phi'_f = (\phi'_{cv} - \phi'_\mu)\cdot\left(1 - e^{-b\cdot\varepsilon_s^p}\right) + \phi'_\mu \tag{4.32}$$

Where:

b = a parameter governing the rate of change of Rowe's friction angle between the two limiting angles.

- *Plastic flow*: Rowe's stress dilatancy theory provides a simple non-associated flow rule for granular material, which seems to be adequate for modelling the non-associated flow of the classified tailings.

- *The constitutive model*: The constitutive model presented in Chapter 4, adequately models the material behaviour for cycloned gold tailings under triaxial compression loading. All the material parameters necessary for the model can be obtained from triaxial tests.

5.4 HDPE membrane behaviour

- *Strain distribution in membranes*: The strain distribution and the engineering Poisson's ratio are strain, but not strain rate dependent. The engineering Poisson's ratio is not dependent of the loading history. The theory presented by Giroud (2004) accurately predicts the engineering Poisson's ratio for the HDPE membranes.

  For a membrane specimen with an aspect ratio (width/length) of 0.5 a uniaxial stress in the central half of the specimen and a uniform stress in the central quarter can be assumed. The difference between the axial strain in the test specimen over the total length (between clamps) and over the central quarter of the specimen is small for axial strains smaller than 0.5.

  The measurement of the lateral strain during the test is not necessary. The relationship between the longitudinal and lateral strain can be obtained from direct measurements after completion of the tests, provided that the membranes did not rupture or fail due to localised necking (cold drawing).

- *The stress-strain behaviour*: Transition strain for the HDPE membranes under uniaxial loading seems to be independent of strain rate. The transition stress seems to be linearly related to the logarithm of the strain rate for a wide range of strain rates but seems to reach an asymptote both at very low and very high strain rates. The shape of the stress-strain curve is weekly dependent on strain rate.

  The strain-rate-dependent stress-strain curve of HDPE membranes under uniaxial tensile loading can be adequately modelled by the hyperbolic-linear function and the exponential function presented in Chapter 4. The parameters necessary for the successful implementation of both these models can easily be obtained from uniaxial tensile tests performed at commercial laboratories.

Extrapolation of the two presented models outside of the range of laboratory tested strain rates provides a rational procedure for obtaining design stress-strain curves at low strain rates not achievable in the laboratory.

5.5 The behaviour of cycloned gold tailings reinforced with a single cell geocell structure

- *The "dead zone"*: The shape and size of the "dead zone" adjacent to the confined ends in geocell structures filled with granular soils can be related to the mechanical properties of the soil. The angle between the confined ends and the boundary of the "dead zone" at the confined end, β, for circular geometries can be estimated with:

  $$\beta = \frac{\phi'_{mob} + \psi_{mob}}{4} + 45^\circ \qquad (4.52)$$

  Where:

  ϕ'_{mob} = the mobilized Mohr-Coulomb friction angle,

  ψ_{mob} = the mobilized dilation angle.

  The shape of the "dead zone" for circular geometries resembles a paraboloid and the depth of the "dead zone" at the centre of the pack for a circular geometry can be estimated by equation (4.53):

  $$d = \frac{W_0 \cdot \tan(\beta)}{4} \qquad (4.53)$$

  Where:

  d = the maximum depth of the "dead zone" from the confined surface,

  W_0 = the width of the geocell pack at the confined ends,

  β = the angle between the "dead zone" and the confined boundary, at the confined boundary.

- *Calculation procedure for the stress-strain response of a soil element:* The procedure for calculating the stress-strain response of a soil element under triaxial loading presented as part of this study provides a simple method for the implementation of the constitutive model presented in Chapter 4. The calculation procedure compares well with the results of numerical analyses using the same soil model.

- *Correction factors for taking non-uniform strain in a soil cylinder with confined end into account*: Due to the non-uniform stress and strain distribution, the stress and strain in a soil cylinder, of which the ends are constrained, is not the same as that for the soil element. The following correction factors, developed in this study, provide a relationship between the axial strain of the whole cylinder and the mean local axial strain in the cylinder as well as the volumetric strain of the whole cylinder and the mean local volumetric strain:

$$\varepsilon_{ag} = \bar{\varepsilon}_{al} \cdot \left(1 - \frac{Diam_0}{l_0 \cdot (1 - \varepsilon_{ag})} \frac{\tan(\beta)}{4} \right) \quad (4.55)$$

and

$$\varepsilon_{vg} = \bar{\varepsilon}_{vl} \cdot \left(1 - \frac{Diam_0}{l_0 \cdot (1 - \varepsilon_{vg})} \frac{\tan(\beta)}{4} \right) \quad (4.56)$$

Where:

l_0 = the original length of the soil cylinder,

β = the angle between the "dead zone" and the confined boundary, at the confined boundary,

ε_{ag}, ε_{vg} = the axial and volumetric strain measured for the whole soil cylinder,

$\bar{\varepsilon}_{al}$, $\bar{\varepsilon}_{vl}$ = the mean local axial and volumetric strain.

These correction factors, when incorporated into the calculation procedure for the calculation of the stress-strain response of a soil cylinder, seem to adequately correct for the non-uniform strain in the soil cylinder.

- *The stress state in the soil due to the membrane action*: The confining stress in the deformed soil cylinder results from the ambient confining stress and the "hoop stress" of the membrane surrounding the soil cylinder and can be written as:

$$\sigma'_{3h} = \sigma'_{30} + \sigma_m(\varepsilon_{mh}) \cdot \frac{2 \cdot t}{D_h} \cdot f_s \quad (4.61)$$

with:

$$f_s = \frac{1 - \varepsilon_{mh} \cdot \nu_m}{1 - \varepsilon_a}$$

Where:

σ'_{3h} = the confining stress imposed onto the soil at position h,

σ'_{30} = the ambient confining stress,

σ_m = the membrane stress,

ε_{mh} = the hoop strain in the membrane at position h,

t = the thickness of the membrane,

D_h = the diameter of the soil cylinder at position h,

ε_a = the mean axial strain of the soil cylinder,

ν_m = the Poisson's ratio of the membrane.

- *The centre diameter of the deformed geocell/soil cylinder*: Under conditions where the ambient confining stress is high compared to the confining stress resulting from the membrane action, the following equation adequately describes the centre diameter of a soil cylinder in terms of the original volume and length and the volumetric and axial strain of the whole cylinder of soil:

$$D_c = 2 \cdot \sqrt{\frac{5}{16} \cdot \left(\frac{6}{\pi} \cdot \frac{V_0 \cdot (1 - \varepsilon_{vg})}{l_0 \cdot (1 - \varepsilon_{ag})} - \left(\frac{Diam_0}{2} \right)^2 \right)} - \frac{Diam_0}{4} \qquad (4.58)$$

Where:

D_c = the diameter at the centre of the soil cylinder,

V_0, l_0, $Diam_0$ = the original volume, length and diameter of the soil cylinder,

ε_{ag}, ε_{vg} = the axial and volumetric strain measured for the whole soil cylinder.

Under conditions where the ambient confining stress is low compared to the confining stress resulting from the membrane action, the following equation adequately describes the centre diameter of the soil cylinder in terms of the original volume and length and the volumetric and axial strain of the whole cylinder of soil:

$$D_c = \frac{1}{8} \cdot \left(\sqrt{\frac{384}{\pi} \cdot \frac{V_0 \cdot (1 - \varepsilon_{vg})}{l_0 \cdot (1 - \varepsilon_{ag})} - 15 \cdot Diam_0} - Diam_0 \right) \qquad (4.59)$$

- *The calculation procedure for the stress-strain response for a single cell geocell-soil composite:* A combination of the calculation procedure for the stress-strain response of a soil element, the correction factors for the non-uniform straining of the soil cylinder and the calculation of the membrane confining stress resulting from the membrane strain, results in the calculation procedure presented in Chapter 4 for the calculation of the stress-strain response of soil reinforced with a single geocell. The results of the calculation procedure compares well with experimental data and numerical analyses.

The calculation procedure slightly under predicts the stress in the single cell structures during the early stages of compression.

5.6 The behaviour of cycloned gold tailings reinforced with a multiple cell geocell structure

- *The "dead zone"*: The equation for the angle β, between the confined ends and the boundary of the "dead zone" which has been presented for circular geometries is also applicable to the "square" geometries.

  For "square" packs, the shape of the "dead zone" resembles a parabola on cross sections at the major symmetry axes.

  The equation for the depth of the "dead zone" at the centre of a circular geometry is also applicable to a "square" geometry.

- *Strain distribution*: The horizontal strain and strain rate in the centre cell of a multi-cell pack at the mid-height, is significantly larger than the horizontal strain of the outer cells. After an axial strain of about 0.08 the horizontal strain of the outer cells seems to cease while the horizontal strain in the centre cell continues with the vertical straining of the pack. The horizontal strain in each cell closer to the centre of the pack exceeds the strain in the cells directly on its outside.

  For the tested packs, it seems that the number of cells in the packs does not significantly influence the horizontal strain distribution in the packs.

- *Stress-strain response of the packs*: The stress-strain response of the 1, 2x2 and 3x3 cell packs shows a sudden stress drop, which seems to be absent in the 7x7 cell packs. This response is a result of strain localization in the 1, 2x2 and 3x3 packs. The increased number of membranes in the 7x7 cell pack is adequate to prevent a shear band from developing.

  The confining stress resulting from the "hoop stress" action for a single cylindrical geocell is directly proportional to the inverse of the cell diameter. The stress-strain response of the single and multi-cell pack configurations can be normalized by the original cell diameter.

There is a systematic change in the stress-strain response of the packs with an increase in the number of cells in the pack. At axial strains of less than about 0.01, the stiffness of the packs increases with an increase in the number of cells. At higher strains, the stiffness and subsequently the strength of the pack decrease with an increase in the number of cells in the pack.

- *The efficiency of multi-cell packs*: The systematic change in the peak strength of the pack with a change in the number of cells can be quantified with the use of an efficiency factor f_{eff}, defined as the ratio of the axial stress in a single cell and multi-cell structure at the same diameter and axial strain rate, that is:

$$f_{eff} = \frac{\sigma_{a\ single\ cell}}{\sigma_{a\ multi\text{-}cell}} \tag{4.65}$$

Where:

f_{eff} = the efficiency factor,

$\sigma_{a\ single\ cell}$ = the axial stress in a single cell structure at a specified diameter and axial strain rate,

$\sigma_{a\ multi\text{-}cell}$ = the axial stress in a multi-cell structure at the same specified cell diameter and axial strain rate.

The "periphery factor", defined in this study, enables the comparison of the data obtained from different geometries. The periphery factor is defined as follows:

$$f_{periphery} = No_{cp} \cdot f_{mp} \tag{4.66}$$

Where:

$f_{periphery}$ = the periphery factor,

N_{ocp} = the number of cells on the periphery of the pack,

f_{mp} = the fraction of membranes belonging to only one cell.

The following empirical relationship, with $a_f = 0.207$, seem to adequately predict the change in the efficiency factor at the peak strength of the pack with an increase in the periphery factor:

$$\left(f_{eff}\right)_{peak} = 1 - a_f \cdot \ln(f_{periphery}) \qquad (4.67)$$

Where:

$(f_{eff})_{peak}$ = the efficiency factor at peak stress,

a_f = the parameter defining the rate of efficiency loss with an increase in the number of cells in the pack,

$f_{periphery}$ = the periphery factor of the pack.

5.7 Recommendations

- Although this study has advanced the current state of knowledge and understanding of the functioning of geocell support packs, it has been limited in its scope and further research needs to be done in the areas that fall outside the scope of this project. The most important of these probably being the influence of the aspect ratio on the strength and stiffness of the support packs. Due to the increased interaction of the two "dead zones" it is reasonable to expect that the strength and stiffness of the pack will increase as the aspect ratio (width/height) increases. This also highlights the need for further research in this area.

- Other aspects that should be researched are the influence of the membrane type and thickness on the composite behaviour. The influence of temperature and damage during installation and during the life of the pack should also be quantified.