Discrete Element Modeling of Geogrid Reinforced Soil

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Latin Characters

\(a\) \[mm\] Aperture size of geogrid

\(a_L\) \[mm\] Aperture width between two longitudinal tensile members

\(a_c, a_n, a_t\) \[-\] Second-order coefficients of the contact normal anisotropy as well as the average normal and tangential contact force anisotropy

\(A\) \[m^2\] Area of parallel bond cross section

\(A\) \[m^2\] Average cross-sectional area of soil specimen

\(b\) \[mm\] Breath of one geogrid tensile member

\(B\) \[mm\] Length of strip foundation

\(B\) \[mm\] Thickness of geogrid transverse tensile member

\(d\) \[mm\] Particle sizes

\(D\) \[mm\] Depth of soil specimen

\(d_1\) \[mm\] Width of 2-ball clump

\(d_2\) \[mm\] Height of 2-ball clump

\(d_{50}\) \[mm\] Median soil particle size

\(d_B\) \[mm\] Distance between strip foundation and edge of slope crest

\(d_c\) \[mm\] Distance of particle centers in 3-ball clumps

\(D_r\) \[-\] Relative density

\(e\) \[-\] Test void ratio in laboratory tests

\(e_{\text{max}}\) \[-\] Maximum void ratio in laboratory tests

\(e_{\text{min}}\) \[-\] Minimum void ratio in laboratory tests

\(f\) \[-\] Friction coefficient

\(F\) \[N\] Geogrid tensile force/pullout force

\(\bar{f}_0\) \[N\] Average normal contact force over all contacts

\(F_{\text{bn}}\) \[N\] Normal contact bond force

\(F_{\text{bottom}}\) \[N/m\] Vertical force on bottom plate in 2D modeling

\(F_{bs}\) \[N\] Shear contact bond force

\(F_{cn}\) \[N\] Normal contact bond strength

\(F_{cs}\) \[N\] Shear contact bond strength
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<td>[N]</td>
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<td>$\bar{F}_i$</td>
<td>[N]</td>
<td>Parallel bond force</td>
</tr>
<tr>
<td>$F^n_i$</td>
<td>[N]</td>
<td>Normal component of contact force</td>
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<td>$\bar{F}^n_i$</td>
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<td>$F^s_i$</td>
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<td>$\bar{F}^s_i$</td>
<td>[N]</td>
<td>Shear component of parallel bond force</td>
</tr>
<tr>
<td>$F_{\text{max}}^s$</td>
<td>[N]</td>
<td>Maximum allowable shear contact force</td>
</tr>
<tr>
<td>$f^k_n$</td>
<td>[N]</td>
<td>Contact normal force in each orientation interval</td>
</tr>
<tr>
<td>$F_{\text{top}}$</td>
<td>[N/m]</td>
<td>Vertical force on top plate in 2D modeling</td>
</tr>
<tr>
<td>$F_v$</td>
<td>[N]</td>
<td>Vertical load on top of soil specimen</td>
</tr>
<tr>
<td>$g$</td>
<td>[m/s$^2$]</td>
<td>Body force acceleration</td>
</tr>
<tr>
<td>$G_{\text{specimen}}$</td>
<td>[N/m]</td>
<td>Self-weight of soil specimen in 2D modeling</td>
</tr>
<tr>
<td>$h$</td>
<td>[mm]</td>
<td>Vertical distance between each consecutive layer of geogrid</td>
</tr>
<tr>
<td>$H$</td>
<td>[mm]</td>
<td>Height of soil specimen</td>
</tr>
<tr>
<td>$H_{\text{knot}}$</td>
<td>[mm]</td>
<td>Height of knot particle in 2D geogrid</td>
</tr>
<tr>
<td>$I$</td>
<td>[kg·m$^2$]</td>
<td>Moment of inertia</td>
</tr>
<tr>
<td>$J_{0-2%}$</td>
<td>[kN/m]</td>
<td>Geogrid tensile stiffness at 2% tensile strain</td>
</tr>
<tr>
<td>$k_{i,j}^n$</td>
<td>[Pa/m]</td>
<td>Parallel bond normal stiffness at each parallel bond contact</td>
</tr>
<tr>
<td>$k_n$</td>
<td>[N/m]</td>
<td>Normal contact stiffness of entities</td>
</tr>
<tr>
<td>$\bar{k}_n$</td>
<td>[Pa/m]</td>
<td>Parallel bond normal stiffness</td>
</tr>
<tr>
<td>$K^n$</td>
<td>[N/m]</td>
<td>Normal contact stiffness at the contact</td>
</tr>
<tr>
<td>$k_s$</td>
<td>[N/m]</td>
<td>Shear contact stiffness of entities</td>
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<tr>
<td>$\bar{k}_s$</td>
<td>[Pa/m]</td>
<td>Parallel bond shear stiffness</td>
</tr>
<tr>
<td>$k^s$</td>
<td>[N/m]</td>
<td>Shear contact stiffness at the contact</td>
</tr>
<tr>
<td>$L$</td>
<td>[mm]</td>
<td>Length of soil specimen</td>
</tr>
<tr>
<td>$\bar{L}$</td>
<td>[mm]</td>
<td>Bonding length in parallel bond</td>
</tr>
<tr>
<td>$L_R$</td>
<td>[mm]</td>
<td>Length of geogrid specimen</td>
</tr>
<tr>
<td>$m$</td>
<td>[kg]</td>
<td>Mass of particle or clump</td>
</tr>
<tr>
<td>$\text{mob } L$</td>
<td>[mm]</td>
<td>Mobilized length in front of transverse tensile member</td>
</tr>
<tr>
<td>$M_i$</td>
<td>[N·m]</td>
<td>Resultant moment of particle</td>
</tr>
<tr>
<td>Symbol</td>
<td>Unit</td>
<td>Description</td>
</tr>
<tr>
<td>------------</td>
<td>---------</td>
<td>------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$\bar{M}_i$</td>
<td>[N·m]</td>
<td>Total moment associated with parallel bond</td>
</tr>
<tr>
<td>$N$</td>
<td>[ - ]</td>
<td>Number of geogrid layers</td>
</tr>
<tr>
<td>$N$</td>
<td>[ - ]</td>
<td>Number of orientation intervals</td>
</tr>
<tr>
<td>$N$</td>
<td>[ - ]</td>
<td>Number of parallel bond contacts</td>
</tr>
<tr>
<td>$n_{2D}$</td>
<td>[ - ]</td>
<td>Test 2D porosity</td>
</tr>
<tr>
<td>$n_{2D,max}$</td>
<td>[ - ]</td>
<td>Maximum 2D porosity based on theoretical calculations</td>
</tr>
<tr>
<td>$n_{2D,min}$</td>
<td>[ - ]</td>
<td>Minimum 2D porosity based on theoretical calculations</td>
</tr>
<tr>
<td>$n_{3D}$</td>
<td>[ - ]</td>
<td>Test 3D porosity</td>
</tr>
<tr>
<td>$n_{3D,max}$</td>
<td>[ - ]</td>
<td>Maximum 3D porosity based on theoretical calculations</td>
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<td>$n_{3D,min}$</td>
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<td>Minimum 3D porosity based on theoretical calculations</td>
</tr>
<tr>
<td>$n'_{3D}$</td>
<td>[ - ]</td>
<td>Quasi 3D porosity</td>
</tr>
<tr>
<td>$n'_{3D,max}$</td>
<td>[ - ]</td>
<td>Maximum quasi 3D porosity</td>
</tr>
<tr>
<td>$n'_{3D,min}$</td>
<td>[ - ]</td>
<td>Minimum quasi 3D porosity</td>
</tr>
<tr>
<td>$n_i$</td>
<td>[ - ]</td>
<td>Unit normal vector</td>
</tr>
<tr>
<td>$n_{lab}$</td>
<td>[ - ]</td>
<td>Test porosity in laboratory tests</td>
</tr>
<tr>
<td>$n_{lab,max}$</td>
<td>[ - ]</td>
<td>Maximum porosity in laboratory tests</td>
</tr>
<tr>
<td>$n_{lab,min}$</td>
<td>[ - ]</td>
<td>Minimum porosity in laboratory tests</td>
</tr>
<tr>
<td>$N_p$</td>
<td>[ - ]</td>
<td>Number of balls in the clumps</td>
</tr>
<tr>
<td>$p$</td>
<td>[kPa]</td>
<td>Foundation pressure</td>
</tr>
<tr>
<td>$P_R$</td>
<td>[kN/m]</td>
<td>Total pullout resistance</td>
</tr>
<tr>
<td>$P_{RB}$</td>
<td>[kN/m]</td>
<td>Bearing resistance in front of geogrid transverse members</td>
</tr>
<tr>
<td>$P_{RS}$</td>
<td>[kN/m]</td>
<td>Frictional resistance between soil and geogrid surface</td>
</tr>
<tr>
<td>$R$</td>
<td>[mm]</td>
<td>Particle radius</td>
</tr>
<tr>
<td>$\bar{R}$</td>
<td>[mm]</td>
<td>Radius of parallel bond</td>
</tr>
<tr>
<td>$r_c$</td>
<td>[mm]</td>
<td>Particle radius in 3-ball clumps</td>
</tr>
<tr>
<td>$S$</td>
<td>[mm]</td>
<td>Spacing between adjacent geogrid transverse tensile members</td>
</tr>
<tr>
<td>$t$</td>
<td>[mm]</td>
<td>Thickness of disc along the plane of paper</td>
</tr>
<tr>
<td>$u$</td>
<td>[mm]</td>
<td>Depth of top geogrid layer below the bottom of strip foundation</td>
</tr>
<tr>
<td>$u_{Clamp}$</td>
<td>[mm]</td>
<td>Clamp displacement</td>
</tr>
<tr>
<td>$U^n$</td>
<td>[mm]</td>
<td>Overlap between ball and ball/wall</td>
</tr>
<tr>
<td>$U^s$</td>
<td>[mm]</td>
<td>Total shear displacement</td>
</tr>
</tbody>
</table>
List of Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\nu$</td>
<td>[mm/min] Tensile rate of geogrid</td>
</tr>
<tr>
<td>$V_i^s$</td>
<td>[m/s] Shear velocity</td>
</tr>
<tr>
<td>$w$</td>
<td>[%] Water content of soil specimen</td>
</tr>
<tr>
<td>$W$</td>
<td>[mm] Width of soil specimen</td>
</tr>
<tr>
<td>$W_R$</td>
<td>[mm] Width of geogrid specimen</td>
</tr>
<tr>
<td>$X$</td>
<td>[ - ] Normalized distance from front clamp</td>
</tr>
<tr>
<td>$x_i$</td>
<td>[m] Centroid location of ball/clump or location of contact point</td>
</tr>
<tr>
<td>$\ddot{x}_i$</td>
<td>[m/s²] Particle acceleration</td>
</tr>
<tr>
<td>$Y$</td>
<td>[ - ] Normalized geogrid displacement</td>
</tr>
<tr>
<td>$z$</td>
<td>[m] Embedded depth of geogrid</td>
</tr>
</tbody>
</table>

Greek Characters

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_B$</td>
<td>[ - ] Fraction of each bearing frontal area</td>
</tr>
<tr>
<td>$\alpha_S$</td>
<td>[ - ] Fraction of geogrid surface area that is solid</td>
</tr>
<tr>
<td>$\beta$</td>
<td>[ - ] Particle shape coefficient ($\beta=2/5$ for spherical particles and $\beta=1/2$ for disk-shaped particles)</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>[kN/m³] Unit weight of soil</td>
</tr>
<tr>
<td>$\gamma_d$</td>
<td>[kN/m³] Dry unit weight of soil</td>
</tr>
<tr>
<td>$\delta$</td>
<td>[ ° ] Friction angle between soil and geogrid</td>
</tr>
<tr>
<td>$\Delta F_i^n$</td>
<td>[N] Normal force increment in parallel bond</td>
</tr>
<tr>
<td>$\Delta F_i^s$</td>
<td>[N] Shear force increment</td>
</tr>
<tr>
<td>$\Delta F_i^s$</td>
<td>[N] Shear force increment in parallel bond</td>
</tr>
<tr>
<td>$\Delta M_i$</td>
<td>[N·m] Moment increment in parallel bond</td>
</tr>
<tr>
<td>$\Delta t$</td>
<td>[s/step] Timestep</td>
</tr>
<tr>
<td>$\Delta U^s$</td>
<td>[mm] Increment of shear displacement</td>
</tr>
<tr>
<td>$\Delta \theta$</td>
<td>[ ° ] Angular interval</td>
</tr>
<tr>
<td>$\Delta \sigma_1$</td>
<td>[kPa] Additional vertical stress</td>
</tr>
<tr>
<td>$\varepsilon_1$</td>
<td>[%] Vertical strain of soil specimen</td>
</tr>
<tr>
<td>$\varepsilon_{gg}$</td>
<td>[%] Tensile strain within geogrid</td>
</tr>
<tr>
<td>$\varepsilon_{gm}$</td>
<td>[%] Average strain of geogrid tensile member</td>
</tr>
</tbody>
</table>
List of Symbols

\[ \varepsilon_j \quad [\%] \quad \text{Strain at any two bonded particles in 2D geogrid} \]
\[ \varepsilon_v \quad [\%] \quad \text{Volumetric strain of soil specimen} \]
\[ \theta \quad [^\circ] \quad \text{Principal contact direction} \]
\[ \theta_c, \theta_n, \theta_t \quad [^\circ] \quad \text{Principal directions of contact normal anisotropy as well as average normal and tangential contact force anisotropy} \]
\[ \lambda \quad [-] \quad \text{Radius multiplier in parallel bond} \]
\[ \mu \quad [-] \quad \text{Friction coefficient at the contact} \]
\[ \rho_{2D} \quad [\text{g/cm}^3] \quad 2\text{D density of numerical specimen} \]
\[ \rho_d \quad [\text{g/cm}^3] \quad \text{Dry density of soil specimen} \]
\[ \rho_{pr} \quad [\text{g/cm}^3] \quad \text{Proctor density of soil specimen} \]
\[ \rho_s \quad [\text{g/cm}^3] \quad \text{Density of solids} \]
\[ \rho_{soil} \quad [\text{g/cm}^3] \quad \text{Testing density of soil specimen} \]
\[ \sigma_1 \quad [\text{kPa}] \quad \text{Total vertical stress} \]
\[ \sigma_{1,max} \quad [\text{kPa}] \quad \text{Maximum vertical stress} \]
\[ \sigma_3 \quad [\text{kPa}] \quad \text{Confining stress} \]
\[ \sigma_b \quad [\text{kPa}] \quad \text{Bearing stress in front of a geogrid transverse member} \]
\[ \bar{\sigma}_c \quad [\text{Pa}] \quad \text{Parallel bond normal strength} \]
\[ \sigma_{\text{max}} \quad [\text{Pa}] \quad \text{Maximum tensile stress acting on the bond periphery} \]
\[ \sigma_N \quad [\text{kPa}] \quad \text{Normal stress in geogrid plane} \]
\[ \sigma_v \quad [\text{kPa}] \quad \text{Vertical stress applied on loading plate} \]
\[ \tau_b \quad [\text{kPa}] \quad \text{Shear stress on top and bottom of the mobilized soil area} \]
\[ \bar{\tau}_c \quad [\text{Pa}] \quad \text{Parallel bond shear strength} \]
\[ \tau_{\text{max}} \quad [\text{Pa}] \quad \text{Maximum shear stress acting on the bond periphery} \]
\[ \varphi \quad [^\circ] \quad \text{Soil friction angle} \]
\[ \omega_i \quad [\text{rad/s}] \quad \text{Angular velocity of particle} \]
\[ \dot{\omega}_i \quad [\text{rad/s}^2] \quad \text{Angular acceleration of particle} \]

Common Abbreviations

2D \quad \text{Two Dimensional} \\
3D \quad \text{Three Dimensional} \\
ASTM \quad \text{American Society for Testing and Materials}
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>DEM</td>
<td>Discrete Element Method</td>
</tr>
<tr>
<td>DIC</td>
<td>Digital Image Correlation</td>
</tr>
<tr>
<td>DIN</td>
<td>Deutsches Institut für Normung (German Institute for Standardization)</td>
</tr>
<tr>
<td>FCC</td>
<td>Face Centered Cubic</td>
</tr>
<tr>
<td>F-DEM</td>
<td>Finite-Discrete Element Method</td>
</tr>
<tr>
<td>FDM</td>
<td>Finite Difference Method</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite Element Method</td>
</tr>
<tr>
<td>FSA</td>
<td>Fourier Series Approximation</td>
</tr>
<tr>
<td>GRS</td>
<td>Geosynthetic Reinforced Soil</td>
</tr>
<tr>
<td>HDPE</td>
<td>High density polyethylene</td>
</tr>
<tr>
<td>HCP</td>
<td>Hexagonal Close Packing</td>
</tr>
<tr>
<td>L</td>
<td>Longitudinal tensile member of geogrid</td>
</tr>
<tr>
<td>LIM</td>
<td>Linear Interpolation Method</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear Variable Displacement Transducer</td>
</tr>
<tr>
<td>MD/XMD</td>
<td>Machine Direction/Cross Machine Direction</td>
</tr>
<tr>
<td>PFC</td>
<td>Particle Flow Code</td>
</tr>
<tr>
<td>PP</td>
<td>Polypropylene</td>
</tr>
<tr>
<td>S</td>
<td>Transverse tensile member of geogrid</td>
</tr>
<tr>
<td>SG</td>
<td>Secugrid</td>
</tr>
</tbody>
</table>
Abstract

Due to the economic and ecologic advantages of geogrids, this kind of geosynthetic material has been widely used in practice to reinforce various soil structures. Geogrid reinforcing effects are performed via the interaction of geogrids together with the surrounding soil. In order to improve the understanding of geogrid–soil interaction, discrete element modeling (DEM) based on 2D software Particle Flow Code (PFC$^2$D) was carried out in this study. Compared with 3D modeling, 2D investigations have the particular advantage of giving insights into key mechanisms with less computational time.

In the numerical modeling of unbonded soil particles, a new iterative approach was suggested for the determination of reasonable 2D porosities in DEM studies. In the DEM investigations of geogrids, a piecewise linear model was developed to characterize the nonlinear tensile behavior of geogrids.

Based on the developed models of soil and geogrids, the geogrid–soil interaction was investigated under different experimental loading conditions. In the numerical compound tensile tests, the frictional interaction between one geogrid tensile member and soil was visualized by the force development and soil particle rotations in the specimen. Based on the discrete element modeling of geogrid pullout tests, the bearing resistance caused by the geogrid transverse members was obtained. The numerically obtained normal stress distribution in the geogrid plane was found to be not constant with increasing clamp displacement but on average in agreement with the prescribed normal stress. In the DEM investigations of biaxial compression tests, the compound stress–strain behavior of geogrid reinforced specimen was significantly improved with increasing number of geogrid longitudinal and transverse members. The geogrid reinforcement mechanisms were visualized by the kinematic behavior and load transfer behavior of unreinforced and reinforced specimens.

Besides the DEM investigations under each single experimental load, the developed DEM models were applied in the numerical modeling of real geogrid reinforced soil structures under combined loading conditions. The geogrid reinforcing effects in such practical loading conditions were visualized by the responses of soil and geogrids.

The DEM simulation results of this study demonstrate that PFC$^2$D can be used as a practical tool to investigate the complex interaction between geogrid and soil. The visualization results provide researchers detailed insights into the geogrid–soil interaction and an improved understanding of geogrid reinforcement mechanisms at a microscopic scale under different loading conditions.
Kurzfassung


1 Introduction

1.1 Background and Motivation

The term “reinforced earth” was first proposed by Vidal (1969) with a definition that the reinforced earth is a composite material formed by combining earth and reinforcement. Soil alone is able to carry mainly compressive and shear forces. However, by using geosynthetics as reinforcement materials, soil structures can be built to carry tensile forces (Heerten, 2007). As an important geosynthetic material, geogrids have been widely used in dealing with various geotechnical problems, e.g. stabilizing earth structures, increasing the bearing capacity of base courses and reducing rut depths in flexible pavements, etc. (Yoo and Kim, 2008; Leshchinsky et al., 2010; Al-Qadi et al., 2011; Heerten, 2012; Yang et al., 2012; Indraratna et al., 2013; Koerner and Koerner, 2013; Qian et al., 2013; Santos et al., 2013; Allen and Bathurst, 2014). Geogrid reinforcement effects are performed via the interaction of geogrids together with the surrounding soil. Hence, the geogrid–soil interface behavior has been regarded as a significant factor to investigate the geogrid reinforcement mechanisms (Lopes and Ladeira, 1996; Palmeira, 2009).

Figure 1.1 presents a typical application of geogrids as reinforcement in an earth structure in practice. Failure is assumed to happen within the geogrid reinforced structure along a potential slip plane. In order to investigate the failure mechanisms of geogrid reinforced structures, many researchers have conducted various laboratory tests on the interface behavior between geogrid and soil as well as the general compound behavior of the composite material (Palmeira and Milligan, 1989; Ochiai et al., 1996; Palmeira, 2004; Moraci and Recalcati, 2006; Sieira et al., 2009; Giang et al., 2010; Abdi and Arjomand, 2011; Naeini et al., 2013; Ruiken, 2013; Arulrajah et al., 2014; Liu et al., 2014). They investigated the influencing factors and their effects on the composite material, which provides useful hints to the design and construction of real geogrid reinforced structures. Numerical approaches, such as the finite element method (FEM), the finite difference method (FDM) and the discrete element method (DEM) (Cundall and Strack, 1979), have also been used to evaluate the geogrid–soil interface behavior. The FEM and the FDM can be regarded as important supplements to laboratory tests for the investigation of reinforcement mechanisms at macroscopic scales. However, with the continuous mechanical approach, the FEM and the FDM are not able to model discrete soil particles and hence they do not take the particle rolling and sliding within geogrid reinforced soil structures into consideration. As the geogrid reinforcement mechanisms highly depends on discrete soil particle shape and distribution, they cannot be satisfactorily investigated only by FEM/FDM, especially at a microscopic scale. The DEM, which has particular advantages in capturing the kinematic behavior of discontinuous media at a microscopic level (Zhang and Thornton, 2007; Stahl and Konietzky, 2011), has been successfully used by many researchers to study the geosynthetic reinforced soil behavior (McDowell et al.,
2006; Zhang et al., 2007; Bhandari and Han, 2010; Ferellec and McDowell, 2012; Chen et al., 2013; Tran et al., 2013; Ngo et al., 2014; Stahl et al., 2014). All the above DEM investigations provide detailed observations at the geogrid–soil interface. However, due to the varying elastic-plastic properties of geogrids and soils as well as the high sensitivity of their interaction to many influencing factors, the compound stress–strain behavior of geogrids embedded in soil is very complex. Therefore, the geogrid reinforcement mechanisms have not been described conclusively up to now and it is still an essential issue to investigate the geogrid–soil interface behavior.

Figure 1.1 Tests to investigate the geogrid–soil interaction mechanisms and compound behavior of geogrid reinforced soil (modified from Ochiai et al., 1996).

1.2 Research Objectives

This study aims to investigate the geogrid–soil interface behavior and to evaluate the geogrid reinforcement mechanisms experimentally and numerically under different loading conditions, which can be used to optimize the design of geogrid reinforced soil systems. In order to achieve this aim, the following research activities are required:

(1) Discrete element modeling of frictional interaction between one geogrid tensile member and soil under a compound tensile load;

(2) Experimental and DEM investigations of pullout tests with geogrids embedded in granular soil;

(3) Experimental and DEM investigations of large-scale biaxial compression tests with unreinforced and geogrid reinforced soil;
1.3 Dissertation Structure

This dissertation is structured into eight chapters as follows.

Chapter 1 presents a brief background and motivation as well as the research objectives of this study.

Chapter 2 summarizes the geogrid reinforcement mechanisms under different loading conditions and the current achievements of discrete element modeling of geogrid and soil as well as their interaction under different testing conditions.

Chapter 3 describes the basic knowledge of the DEM and the used DEM software Particle Flow Code in 2 Dimensions (PFC\textsuperscript{2D}) first. Then a new suggestion for the determination of 2D porosities is proposed based on the review of current approaches for linking 2D and 3D porosities in DEM investigations. After determining 2D porosities for the soil samples in this study, direct shear tests are simulated to calibrate the DEM models and micro input parameters for sand and gravel, which can be used for further DEM investigations. Finally, a piecewise linear model is developed in this chapter to describe the nonlinear tensile strength behavior of one geogrid tensile member and normal geogrids with regular transverse members, followed by the corresponding calibration results for the investigated geogrid samples.

Chapter 4 focuses on the frictional interaction between one geogrid tensile member and soil under a compound tensile load using PFC\textsuperscript{2D}. The load transfer behavior between one geogrid tensile member and soil is visualized by the force, displacement and strain distributions along the geogrid tensile member, the contact force changes in the specimen and the rotations of soil particles in the vicinity of the geogrid tensile member at different clamp displacements.

Chapter 5 investigates the geogrid–soil interaction under pullout loads experimentally and numerically. In the laboratory tests, two types of geogrids with different tensile stiffness are modified with different numbers of transverse members. The influence of geogrid tensile stiffness and number of geogrid transverse members on the pullout resistance are studied. In the discrete element modeling, the numerical geogrids are also prepared with different numbers of transverse members. Detailed insights into the geogrid–soil interaction under pullout loads can be described not only by the qualitative force distributions along the geogrid and in the specimen but also by the quantitative geogrid force, displacement and strain distributions along the geogrid with different numbers of geogrid transverse members. Moreover, based on the Fourier Series Approximation (FSA) method, the reorientations of contacts and forces in the specimen are
presented at different clamp displacements. Furthermore, the normal stress distributions in the geogrid plane, which is a decisive parameter that can only be evaluated indirectly in the experimental pullout tests, are obtained directly using the FSA method in the numerical modeling.

Chapter 6 presents the compound stress–strain behavior of geogrid reinforced soil under biaxial compression loads with both experimental and numerical approaches. In the laboratory tests, different numbers of geogrid longitudinal and transverse members are used to investigate the contributions of longitudinal and transverse members to the compound stress–strain behavior of geogrid reinforced soil. The kinematic behavior of unreinforced and geogrid reinforced soil is visualized with the particle displacements and rotations. Moreover, the tensile strain distributions along the geogrid are obtained using the attached strain gauges on geogrid samples. In the DEM investigations, the geogrid reinforcing effects are described by the contributions of geogrid longitudinal and transverse members to the compound stress–strain behavior of geogrid reinforced specimens and the numerically obtained kinematic behavior of both unreinforced and reinforced specimens. The load transfer behavior between soil and geogrid is visualized not only by the qualitative force distributions in the specimen and along the geogrid but also by the quantitative tensile force and tensile strain distributions along the geogrid.

Based on previously developed DEM models, Chapter 7 investigates the geogrid reinforcement mechanisms in real geogrid reinforced soil structures, i.e. reinforced base courses and reinforced slopes loaded with surface strip foundations. The numerical simulation results are validated by the laboratory test results from the literature. The geogrid reinforcing effects are evaluated by the responses of soil and geogrids.

Chapter 8 summarizes the research activities performed in this study. Recommendations for further research are also provided in this chapter.
2 Literature Review

2.1 Introduction

In geogrid reinforced earth structures, geogrid reinforcement effects are performed through complex interaction mechanisms between geogrid and its surrounding soil. Additionally, different loading conditions may lead to different load transfer behavior between geogrid and soil. Hence, the geogrid reinforcement mechanisms under different loading conditions are reviewed in this chapter based on related studies.

In order to further investigate the geogrid–soil interface behavior using the DEM in this study, a sufficient understanding of the current discrete element modeling of geogrid and soil as well as their interaction is imperative. Therefore, the current achievements of discrete element modeling of geogrid and soil as well as their interaction under different testing conditions are summarized in this chapter.

2.2 Geogrid Reinforcement Mechanisms

Figure 2.1 shows a schematic view of a geogrid, which is composed of longitudinal and transverse tensile members via junctions. Due to the typical grid structures, geogrid reinforcement effects are performed through various interaction mechanisms between geogrid and soil.

![Figure 2.1 Schematic view of a geogrid (modified from Jewell et al., 1984; Müller and Saathoff, 2015).](image)

2.2.1 Geogrid Reinforcement Mechanisms under Pullout Loads

A geogrid pullout test has been regarded as the most appropriate way to study the pullout failure mechanism that develops in mechanically stabilized earth walls (Palmeira, 2009). When a relative
displacement between geogrid and soil occurs under pullout loads, two basic interaction mechanisms between geogrid and soil can be identified (Jewell et al., 1984), as shown in Figure 2.2. The total pullout resistance ($P_R$) is commonly calculated as a sum of the frictional resistance between soil and geogrid surface ($P_{RS}$) and the bearing resistance in front of geogrid transverse members ($P_{RB}$) using the following equation:

$$P_R = P_{RS} + P_{RB}$$  \hspace{1cm} (2.1)

(a) Frictional resistance between soil and geogrid surface

(b) Bearing resistance in front of geogrid transverse members

Figure 2.2 Two mechanisms between geogrid and soil (modified from Jewell et al., 1984).

According to the studies by Jewell et al. (1984), the frictional resistance between soil and geogrid surface ($P_{RS}$) can be calculated with the following equation:

$$P_{RS} = 2 \cdot W_R \cdot L_R \cdot a_S \cdot \sigma_N \cdot \tan \delta$$  \hspace{1cm} (2.2)

where $W_R$ is the width of geogrid specimen;
$L_R$ is the length of geogrid specimen;
$a_S$ is the fraction of geogrid surface area that is solid;
$\sigma_N$ is the normal stress in geogrid plane;
$\delta$ is the friction angle between soil and geogrid.

Jewell (1990) suggested to calculate the bearing resistance in front of geogrid transverse members ($P_{RB}$) with the following equation:

$$P_{RB} = \left( \frac{L_R}{N} \right) a_B \cdot \sigma_b \cdot B \cdot W_R$$  \hspace{1cm} (2.3)

where $\frac{L_R}{N}$ is the number of geogrid transverse members for bearing resistance;
$a_B$ is the fraction of each bearing frontal area;
$\sigma_b$ is the bearing stress in front of a geogrid transverse member;
2.2 Geogrid Reinforcement Mechanisms

$B$ is the thickness of a geogrid transverse member.

The bearing stress can be calculated based on two typical shear failure modes, i.e. a general shear failure mode proposed by Peterson and Anderson (1980) and a punching shear failure mode proposed by Jewell et al. (1984). The two shear failure modes provided the upper and lower bounds, respectively, for theoretically estimating the bearing stresses in front of grid transverse members, which were normalized by the vertical stress in grid plane as a function of the soil friction angle, as shown in Figure 2.3. The dots in Figure 2.3 present the measured bearing stress ratios from related studies, which were summarized by Palmeira and Milligan (1989) and Jewell (1990). Most measured results locate between the upper and lower bounds, which demonstrates that the above theoretical estimations fit the experimental data very well.

\[
\frac{\sigma_b}{\sigma_N} = e^{n\cdot\tan \varphi} \cdot \tan \left(45^\circ + \frac{\varphi}{2}\right)
\]

\[
\frac{\sigma_b}{\sigma_N} = e^{(90^\circ + \varphi)\cdot\tan \varphi} \cdot \tan \left(45^\circ + \frac{\varphi}{2}\right)
\]

**Figure 2.3** Bearing stress ratio against soil friction angle (modified from Peterson and Anderson, 1980; Jewell et al., 1984; Palmeira and Milligan, 1989; Jewell, 1990).

In addition to the above studies, Ziegler and Timmers (2004) proposed a simple model to describe the displacement-dependent bearing resistance in front of welded geogrid transverse members with a mobilization function, as shown in Figure 2.4. The bearing resistance ($P_{RB}$) increases with increasing mobilized length of soil in front of geogrid transverse members. However, the bearing resistance cannot increase any more once the mobilized area reaches the neighboring transverse member. Therefore, the bearing resistance in front of a transverse member is limited by the distance to the adjacent transverse member, which is known as the interference between transverse members.
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Figure 2.4 Soil mobilization in front of geogrid transverse members (modified from Ziegler and Timmers, 2004).

The interference behavior between grid transverse members was investigated by Dyer (1985) based on laboratory pullout tests using metallic grids with different spacing between transverse members embedded in a photo-elastic medium. The bearing resistance can be visualized by intensity variations of light in front of each transverse member, as shown in Figure 2.5. It is clear that the degree of interference between grid transverse members increases significantly with reducing spacing between them.

Figure 2.5 Interference between grid transverse members (after Dyer, 1985, modified from Palmeira, 2009).

The transverse member thicknesses with respect to soil particle sizes also influence the bearing stresses in front of transverse members and related studies have been conducted by Palmeira (2009). Figure 2.6 shows the bearing stress normalized by the in-soil shear stress against the ratio of transverse member thickness to median soil particle size ($d_{50}$). The presented results in Figure 2.6 are summarized from those tests on isolated transverse members with different shapes of cross sections (e.g. round, square, hexagonal and rectangular) embedded in dense sands or crushed glass. The fitted curve shows that the influence of grid transverse member thicknesses
with respect to soil particle sizes on the bearing stress ratio can be neglected when the ratio of grid transverse member thickness to median soil particle size \(d_{50}\) is larger than 12 (Palmeira, 2009).

\[
\frac{\sigma_b}{\sigma_N \tan \varphi}
\]

\[\varphi = \text{soil friction angle}\]

\[\frac{B}{d_{50}}\]

**Figure 2.6** Pullout test results on isolated transverse members with different cross sections (modified from Palmeira, 2009).

Following the above estimations and methods, some researchers (e.g. Palmeira, 2004; Moraci and Gioffrè, 2006; Sieira et al., 2009; Jacobs et al., 2014) developed theoretical models to predict the geogrid–soil interaction under pullout loads. The predicted results based on those models showed good agreement with the experimental pullout test results as well as with the field observations.

In order to investigate the contributions of frictional and bearing resistance to the total pullout response separately, laboratory pullout tests of geogrids with and without transverse members have been carried out (e.g. Farrag et al., 1993; Ziegler and Timmers, 2004; Teixeira et al., 2007; Ziegler et al., 2007; Jacobs et al., 2014). The investigation results showed different contributions of frictional and bearing resistance to the total pullout response, e.g. the frictional resistance contributes approximately from 35 % to 79 % to the total pullout resistance based on the above studies. The huge differences are caused by many influencing factors, such as soil type and density, geometrical characteristics and mechanical properties of geogrids, stress level as well as boundary conditions in pullout tests (Palmeira and Milligan, 1989). Based on finite element modeling of pullout tests, Wilson-Fahmy and Koerner (1993) reported the development of each pullout resistance component for stiff and flexible geogrids, as shown in Figure 2.7. It is clear that the frictional resistance of longitudinal members of both geogrid types decreases with increasing pullout loads, while the bearing resistance of transverse members increases. Moreover, the bearing resistance of transverse members for flexible geogrids was activated much later than that for stiff geogrids. When the pullout load was less than 25 % of the ultimate pullout force for flexible geogrids, almost all of the pullout resistance was contributed by the frictional resistance of longitudinal members (see Figure 2.7).

Besides the above mentioned conventional pullout tests, Ezzein and Bathurst (2011, 2014) developed a novel pullout test apparatus with a transparent bottom wall to evaluate the geogrid–soil interaction using transparent granular soil. The transparent soil is fused quartz particles
saturated with white mineral oils, which have the same refractive index as the quartz. Using the digital image correlation (DIC) technique, it is possible to visualize the relative horizontal displacement between geogrid and its surrounding soil directly. The experimental data can be used to quantify the geogrid–soil interaction and to develop interface shear models for describing the load transfer behavior in the anchorage zones of geogrid reinforced earth structures (Bathurst and Ezzein, 2015). Additionally, Ferreira and Zornberg (2015) developed a 3D transparent pullout test device, in which the bottom plate and the sidewalls of the pullout box were transparent. The setup led to direct 3D visualization of the geogrid–soil interaction not only in the plan view of the geogrid but also in the side view of the geogrid–soil interface.

**Figure 2.7** Components of pullout resistance for stiff and flexible geogrids (modified from Wilson-Fahmy and Koerner, 1993).

### 2.2.2 Geogrid Reinforcement Mechanisms under Wheel Loads

Geogrids have been widely used in road engineering to increase the bearing capacity of base courses and to reduce rut depths in flexible pavements or unpaved roads (Collin et al., 1996; Fannin and Sigurdsson, 1996; Al-Qadi et al., 2011, 2012). Figure 2.8 shows the performance of unpaved roads without and with geogrid reinforcement under truck wheel loads. Good performance with geogrid reinforcement can be seen in Figure 2.8b.
2.2 Geogrid Reinforcement Mechanisms

Studies of roadways reinforced with geosynthetics have identified three possible reinforcement mechanisms, i.e. lateral restraint, increased bearing capacity and tensioned membrane effect, as shown in Figure 2.9.

When an unpaved road is subjected to repetitive wheel loads, the base course soil under the wheel tends to move laterally. Such lateral movement can be restrained by placing a geogrid layer horizontally in the base course. The vertical wheel load is transferred from soil to geogrid due to the frictional interaction and the interlocking effect between soil and geogrid. Hence, the lateral movement of base course soil is restrained by the geogrid, as shown in Figure 2.9a. The experimental and numerical studies on geogrid reinforced ballast and granular assembly under cyclic loading have quantified the lateral restraint behavior of geogrids (Indraratna et al., 2013; Chen et al., 2015).

Figure 2.9b illustrates the second possible geosynthetic reinforcement mechanism under wheel loads. The potential shear failure of a reinforced foundation develops along a higher shear strength path, which tends to increase the bearing capacity of the roadway. The increased bearing capacity can be explained quantitatively with two mechanisms. One is the “cohesion concept”, in which the geosynthetic reinforcement is said to give an added cohesion to the soil (Schlosser and Long, 1972). The other is the “confining effect concept”, in which the geosynthetic reinforcement is considered to increase the effective confining pressure of the soil (Yang, 1972). In order to understand how geosynthetics contribute to the improvement of bearing capacity, many researchers carried out plenty of large-scale triaxial and biaxial compression tests on geogrid reinforced soil (e.g. Haeri et al., 2000; Ruiken and Ziegler, 2008; Ruiken et al., 2012; Chen et al., 2014b).

The third mechanism of geosynthetic reinforcement is the tensioned membrane effect, as shown in Figure 2.9c. The tensioned membrane effect develops in the geosynthetic reinforcement with increasing vertical deformation of the roadway under wheel loads. The developed tensile force in the geosynthetics provides vertical support to the wheel load. Thereby, the vertical stress on the base course soil can be reduced as well as the wheel rut depth. Collin et al. (1996) carried full-
scale highway load tests of flexible pavement systems with unreinforced and geogrid reinforced base courses. Their testing results showed that geogrids could significantly improve the rutting resistance. Similar observations have been reported in the field study of unpaved roads by Fannin and Sigurdsson (1996) and in the laboratory loaded wheel tests on unpaved base courses by Wu et al. (2015). Moreover, based on a full-scale accelerated testing system, Al-Qadi et al. (2011, 2012) identified the optimum location of a single geogrid layer in a low-volume flexible pavement, i.e. upper one third of the base course for thick granular base layers.

**Figure 2.9** Three possible reinforcement mechanisms of geosynthetics in roadways (after Haliburton et al., 1981; modified from Holtz et al., 1998).
2.3 Investigation of Geogrid Reinforced Soil with DEM

Due to the advantages of capturing detailed insights of discontinuous media at a microscopic scale, the discrete element modeling has been regarded as an ideal tool for investigating the geogrid–soil interaction (Konietzky et al., 2004). In this section, current achievements of discrete element modeling of geogrid and soil as well as their interaction are summarized.

2.3.1 Discrete Element Modeling of Soil

In discrete element modeling, sand particles were always modeled as circular or spherical particles (Wang and Leung, 2008; Bhandari and Han, 2010; Lin et al., 2013; Tran et al., 2013; Zhang et al., 2013). Since the computational time in the DEM simulations is highly dependent on the particle numbers, the technique of “up-scaling” for soil particles has been used in DEM studies to balance the computational cost against the scaling effect on the sample response. The up-scaling factor in those studies ranged from 1.25 to 15. In order to compensate the lack of angularity for such circular particles, large friction coefficients have been used to reach an equivalent internal friction angle of the soil (Lin et al., 2013; Zhang et al., 2013).

Another possible way to reach the equivalent internal friction angle of the soil is to simulate irregular shapes of the real particles by bonding circular or spherical particles together to form clusters or clumps, as shown in Figure 2.10. Overlaps may exist in each cluster and each clump. The difference between a cluster and a clump is that the cluster can be used to simulate grain crushing since the intragranular bonds can be broken, while the intragranular bonds in the clump are always considered to be infinitely stiff and strong (Li and Holt, 2002). Without using large friction coefficients for the clusters and clumps, high values of the macroscopic internal friction angles of the soil can be achieved. However, due to the high requirement of computing power, this technique is currently only applied for simulating medium/coarse gravel or ballast (Lu and McDowell, 2007; Ferellec and McDowell, 2010a, 2010b; Lackner, 2012; Chen et al., 2014a; Ngo et al., 2014; Stahl et al., 2014).

In order to get the equivalent internal friction angle of the soil with less computational time, a clump consisting of limited numbers of circular discs or spheres provides a good solution. This technique has been successfully used in the numerical modeling of geogrid–ballast interaction under cyclic loads and shear band development in granular materials as well as 2D DEM calibrations of large size plane strain compression tests (Chen et al., 2012; Gu et al., 2014; Rui et al., 2015). Figure 2.11 shows examples of 2-ball, 3-ball and 4-ball clumps reported in the literature.
(a) Discrete element modeling of real particles with 2D clusters (after Indraratna et al., 2010)

(b) Discrete element modeling of real particles with 3D clumps (after Lackner, 2012)

**Figure 2.10** Discrete element modeling of real particles in 2D and 3D.

(a) a 2-ball clump in 3D (after Chen et al., 2012)  (b) a 2-ball clump in 2D (after Gu et al., 2014)

(c) 3-ball and 4-ball clumps in 2D (after Rui et al., 2015)

**Figure 2.11** Examples of clumps with limited numbers of circular discs or spheres in 2D and 3D.

### 2.3.2 Discrete Element Modeling of Geogrids

In discrete element modeling, geogrids were modeled by bonding and/or overlapping circular or spherical particles to form similar geometries of real geogrids as those used in laboratory tests.
Investigation of Geogrid Reinforced Soil with DEM

(McDowell et al., 2006; Zhang et al., 2008; Chen et al., 2012; Han et al., 2012; Lackner, 2012; Ngo et al., 2014; Stahl et al., 2014; Bhandari et al., 2015). Quite similar geometries of real geogrids, including transverse members and knots at the junctions, can be easily simulated in 3D modeling, as shown in Figure 2.12. In 2D modeling, however, the geometries of real geogrids has to be simplified. The longitudinal member was simulated with one row of bonded particles without any overlaps. Extra bonded particles on and beneath the longitudinal member or particles with larger diameters at the positions of junctions were simulated to investigate the contributions of geogrid transverse members, as shown in Figure 2.13.

Figure 2.12 Examples of DEM models of geogrids in 3D.

Figure 2.13 Examples of DEM models of geogrids in 2D.

Compared to the geometric modeling of geogrids, the mechanical modeling of geogrids is equally important. The tensile strength of a geogrid is determined by the bond forces between geogrid
particles in discrete element modeling (McDowell et al., 2006; Chen et al., 2012). With the parallel bond model provided by Itasca (2008), it is easily possible to represent the tensile strength properties of a geogrid with a linear force–strain relationship (e.g. Zhang et al., 2008; Han et al., 2012; Ngo et al., 2014). However, most laboratory tensile test results of geogrids show nonlinear tensile strength behavior, as shown in Figure 2.14.

![Figure 2.14 Examples of nonlinear tensile force–strain behavior of geogrids in laboratory tests.](image)

In order to describe the nonlinear tensile behavior of geogrids under tensile loads, a parallel bond radius decreasing law was implemented into the discrete element modeling (Stahl et al., 2014). With increasing parallel bond radius, the parallel bond stiffness was reduced. The parallel bond radius decreasing law mimicked the effects of reducing the amount of cement-like material joining the two bonded particles. The calibrated numerical tensile test results showed good agreement with the corresponding experimental data (Stahl et al., 2014).

### 2.3.3 Discrete Element Modeling of Geogrid–Soil Interaction

#### 2.3.3.1 Pullout Tests

Konietzky et al. (2004) carried out 3D discrete element modeling of pullout tests using spheres to investigate the interlocking effect of geogrids. Qualitative contact force distributions within the specimen at the beginning and the end of the test visualized the influencing zones of the geogrid under pullout loads, as shown in Figure 2.15. Moreover, the recorded normal and shear forces inside the model at the end of the test showed that the interlocking effect was restricted to a relatively small height at both sides of the geogrid (i.e. approximately 20 cm).

The subsequent study conducted by McDowell et al. (2006) focused on the effect of the ratio of geogrid aperture size to ballast particle diameter under pullout loads, using clumps by overlapping spheres to represent the actual shape of ballast. For the ballast particles simulated in their study (40 mm in diameter), a ratio of aperture size to particle diameter of around 1.4 showed optimum results in terms of peak resistance mobilized at the smallest displacement.
Zhang et al. (2007) conducted 2D numerical pullout tests on loose and dense soil samples to study the effect of compaction on geogrid pullout response. The development of contact forces and displacement vectors of soil particles in both samples showed that the density had great influence on the 2D dilatancy behavior of geogrid reinforced soil under pullout loads.

Ferellec and McDowell (2012) investigated the influence of ballast shape (i.e. spheres and clumps with angular shapes) on ballast–geogrid interlocking behavior under pullout loads. The clumps used in their study were simulated by overlapping spherical particles. The pullout force was larger for the clumps than that for the spheres, which can be explained by the better interlocking of the geogrid with the clumps than that with the spheres. However, the maximum pullout forces in both simulations were much smaller than the values based on laboratory tests. A higher value of the pullout resistance, which was closer to the experimental results, could be achieved by modeling clumps with sharper edges. However, it required many more spheres for each clump and therefore much more computing power.

Based on 3D discrete element modeling, Chen et al. (2013) reproduced experimental pullout tests using 2-ball clumps. The DEM simulation results showed good agreement with the corresponding experimental data and the distribution of contact forces in the geogrid reinforced ballast system was provided. Their further DEM investigations focused on the influence of clump shapes on both the pullout resistance and the distribution of contact forces (Chen et al., 2014a). The numerical modeling with 8-ball tetrahedral clumps showed the best fitting results with the experimental data.

A coupled finite-discrete framework has been developed by Tran et al. (2013) to investigate the behavior of a biaxial geogrid embedded in granular soil subjected to pullout loading. The geogrid was modeled with finite element and interface elements, while the soil was modeled using

**Figure 2.15** Contact force distributions in pullout tests (modified from Konietzky et al., 2004).
spherical particles. Response of both the geogrid and the surrounding backfill soil has been obtained numerically to visualize the geogrid–soil interaction under pullout loads.

Based on 3D discrete element modeling, Zheng et al. (2013a, 2013b) carried out numerical pullout tests using biaxial geogrids and 3-ball clumps. They investigated the influence of soil particle sizes on the pullout resistance as well as the contribution of frictional resistance and bearing resistance to the total pullout force. The shear strength of the geogrid–soil interface with big particles was larger than that with small particles. At a large clamp displacement, the contribution of bearing resistance to the total pullout force was larger than that of frictional resistance.

Miao et al. (2014a, 2014b) conducted 3D discrete element modeling of pullout tests using triaxial geogrids. The development of fabric anisotropy (e.g. coordinate number, orientation of contact forces) in the pullout model has been illustrated with increasing clamp displacements. Moreover, they also investigated the influence of joint protuberances on the geogrid pullout behavior. The shear strength of the geogrid–soil interface was increased by using joint protuberances.

Biaxial geogrids and granular soil with similar geometries as those used in the laboratory tests have been modeled by overlapping and/or bonding spherical particles in 3D DEM investigations (Stahl et al., 2014). Using the above developed geogrid and soil, numerical pullout tests were carried out. DEM analyses of soil mobilization and geogrid deformation have been presented in their study to illustrate the significance of geogrid interlocking effect as a key mechanism for soil stabilization.

2.3.3.2 Large-scale Triaxial Tests

Based on 3D discrete element modeling, Konietzky et al. (2004) modeled large-scale triaxial tests using spheres to study the confinement zones of geogrids. For aggregates with particle radii between 2 and 11 mm, a single extruded biaxial geogrid layer had a confinement influence zone of approximately 100 mm on both sides of the geogrid. They also conducted further DEM modeling with three geogrid layers (see Figure 2.16a) to investigate the effect of multiple geogrid layers on the displacement response of the sample. The numerical results showed that the vertical and radial displacements of the sample with three geogrid layers were reduced by approximately 50% compared with those with only one geogrid layer. The geogrid interlocking effect and the soil arching effect could be clearly observed by the contact force distribution in the specimen, as shown in Figure 2.16b.
Mishra et al. (2014) carried out numerical large-scale triaxial tests using an imaging-based DEM modeling approach to identify the optimal position for geogrid reinforcement in terms of the maximum shear strength. Specimens with five different reinforcement configurations using one geogrid or two geogrids were tested (see Figure 2.17). Placing two geogrids at 254 mm from top and bottom of the specimen (Configuration F in Figure 2.17) resulted in the maximum shear strength for both biaxial and triaxial geogrids.
2.3.3.3 Direct Shear Tests

Direct shear tests were modeled by Bussert (2009) to study the load transfer mechanisms in both unreinforced and reinforced systems. In both systems, the vertical stress was maintained constant and the lower shear box was moved horizontally. In the unreinforced model, the contact forces developed between the lower “shifted” wall and the opposite upper wall in an “S” shape with the main principal stress being horizontal in the shear plane. However, a different load transfer mechanism has been observed in the reinforced system, in which the geogrid was placed in the upper shear box near the shear plane. The horizontal forces acting on the lower “shifted” wall were restrained by the geogrid and thus almost no horizontal stresses developed in the composite material above the geogrid layer even at a large horizontal shear displacement (e.g. 10 mm), as shown in Figure 2.18. Similar observations have also been reported by Ngo et al. (2014) in the DEM investigations of large-scale direct shear tests with unreinforced and geogrid reinforced ballast (see Figure 2.19).

![Figure 2.18 Contact force distribution in a direct shear test at a horizontal displacement of 10 mm (modified from Bussert, 2009).](image1)

(a) Unreinforced ballast  
(b) Geogrid reinforced ballast

![Figure 2.19 Contact force distributions in direct shear tests at a horizontal displacement of 18 mm (modified from Ngo et al., 2014).](image2)

2.3.3.4 Plate Load Tests

A plate load test is commonly used to evaluate the benefits of placing geosynthetics in foundations (Tingle and Jersey, 2005). Chen et al. (2012) carried out 3D discrete element modeling of cyclic loads on ballast reinforced with biaxial and triaxial geogrids under confined and unconfined
conditions. The ballast particles were simulated with 2-ball clumps. Compared with unreinforced samples, the settlement of geogrid reinforced specimens were greatly reduced. For confined box tests, an optimum location of a single biaxial geogrid layer was found to be 100 mm from the base and the triaxial geogrid showed better performance than the biaxial geogrid. Under unconfined conditions, the optimum location of a single geogrid layer was 50 mm from the sub-ballast layer. The use of two geogrid layers located both near the base and at mid-depth led to a slight improvement compared with the sample reinforced at 50 mm above the sub-ballast.

2.3.3.5 Wheel Load Tests

In order to develop the concept of prestressed reinforced soil during compaction, 3D discrete element modeling of geogrid–soil interaction under a compaction roller was carried out by Lackner and Semprich (2010), as shown in Figure 2.20. Their DEM simulation results showed that the interaction between soil and geogrid could be defined as a composition of frictional and interlocking effects. The frictional effect was influenced by the surface roughness of soil particles and geogrid, while the interlocking effect depended on the ratio of geogrid aperture size to soil particle size.

Bhandari et al. (2015) carried out 2D discrete element modeling of unpaved geogrid reinforced granular bases under a vertical cyclic load. The geogrid reinforcement was placed at the mid-depth or the bottom of the base course. A vertical wheel load was applied cyclically on top of the base course. The reinforced bases showed better performance than the unreinforced base. Moreover, the geogrid placed at the mid-depth of the base performed better than that placed at the bottom of the base. Their numerical results showed that the geogrid tensile force was mobilized under the wheel load, which helped to widen the distribution of contact force chains in the geogrid reinforced system.

Based on 3D discrete element modeling, Jas et al. (2015) investigated the performance of a geogrid stabilized sub-base under a running wheel load. Their simulation results showed that the rutting
behavior of the sub-base was greatly improved by using a triaxial geogrid. The development of geogrid tensile forces and displacements were visualized at different loading cycles.

2.4 Summary

This chapter reviewed the geogrid reinforcement mechanisms under different loading conditions. Experimental and numerical investigations as well as theoretical evaluations help to improve the understanding how geogrids interact with the surrounding soil and how geogrids contribute to the reinforcement effects.

The DEM, which has particular advantages of capturing detailed insights into the geogrid–soil interaction, provides a new perspective to study the geogrid reinforcement mechanism, especially at a microscopic scale. Current achievements of discrete element modeling of geogrid and soil as well as their interaction under different testing conditions has been summarized in this chapter. The DEM investigations provide not only qualitative visualization of the geogrid–soil interaction but also quantitative response of the geogrid and its surrounding soil. Such results provides helpful hints to optimize the design of geogrid reinforced soil systems.

However, due to the varying elastic-plastic properties of geogrids and soils as well as the high sensitivity of their interaction to many influencing factors, the complicated geogrid–soil interaction as well as the compound stress–strain behavior of geogrid reinforced soil have not been described conclusively up to now. Therefore, it is still necessary to conduct further experimental and DEM investigations to describe the reinforcement mechanisms of geogrids embedded in soil.
3 Numerical Modeling and Calibration using PFC\textsuperscript{2D}

3.1 Introduction

The discrete element method (DEM) was introduced by Cundall (1971) for analyzing rock mechanics problems and then applied to soils by Cundall and Strack (1979). Compared to conventional laboratory tests and numerical simulations based on finite element method (FEM), the DEM has particular advantages of capturing detailed insights into the kinematic behavior of discontinuous media at a microscopic level (Zhang and Thornton, 2007; Stahl and Konietzky, 2011). Therefore, this method has been regarded as a powerful supplement to laboratory tests and FEM simulations (Meier et al., 2008; Rahmati et al., 2014). Until now, the DEM has been playing an important role in investigating geotechnical problems, i.e. rock fracturing (Lee and Jeon, 2011; Jia et al., 2013; Yang et al., 2014), geosynthetic–soil interaction (Ferellec and McDowell, 2012; Han et al., 2012; Tran et al., 2013; Ngo et al., 2014; Stahl et al., 2014), etc. The numerical modeling in this study was carried out using Particle Flow Code in 2 Dimensions (PFC\textsuperscript{2D}), which was developed by Itasca (2008) based on the DEM.

This chapter firstly presents the basic knowledge of the DEM and PFC\textsuperscript{2D}, e.g. the calculation cycle, the clump logic and the contact constitutive models. Since the 2D porosity is a required controlling parameter in PFC\textsuperscript{2D}, the 3D porosity of soil in a real problem has to be converted into a 2D value when using PFC\textsuperscript{2D} for DEM investigations. The main approaches for converting porosities from 3D to 2D for DEM studies are reviewed in Section 3.3 and a new suggestion is proposed for determining 2D porosities. Based on the corresponding experimental data, numerical direct shear tests have been conducted to calibrate the model and input parameters for sand and gravel, respectively. Finally, a piecewise linear model is developed in Section 3.4 for describing the nonlinear tensile strength behavior of one geogrid tensile member and normal geogrids. The calibration results are presented in this section.

3.2 Discrete Element Modeling using PFC\textsuperscript{2D}

The basic knowledge of PFC\textsuperscript{2D} as well as the corresponding laws and contact constitutive models are described in this section according to the manual book of this software (Itasca, 2008).

3.2.1 Overview of PFC\textsuperscript{2D}

PFC\textsuperscript{2D} uses rigid entities (particles and walls) and soft contacts to simulate the mechanical behavior of a discrete element system. In such a discrete element system, each particle displaces independently from one another and interacts only at contacts with its neighboring particles or the
boundary walls. The mechanical behavior of the system is then described in terms of the movement of each particle and the inter-particle forces acting at each contact point. Newton’s laws of motion provide the fundamental relationship between particle motion and the forces causing that motion, while the force-displacement law is applied to each contact.

More complex behavior can also be modeled by bonding the particles together at their contact points. When the inter-particle forces acting at any bond exceed the bond strength, that bond is broken. This allows tensile forces to develop between particles. The DEM investigations with PFC\(^2\)D are carried out under the following assumptions (Itasca, 2008):

1. The particles are treated as rigid bodies.
2. The contacts occur over a vanishingly small area (i.e., at a point).
3. Behavior at the contacts uses a soft-contact approach, where the rigid particles are allowed to overlap one another at contact points.
4. The magnitude of the overlap is related to the contact force via the force–displacement law, and all overlaps are small in relation to particle sizes.
5. Bonds can exist at contacts between particles.
6. All particles are circular. However, the clump logic supports the creation of super-particles of arbitrary shape. Each clump consists of a set of overlapping particles that acts as a rigid body with a deformable boundary.

In addition to circular particles, PFC\(^2\)D also includes “walls” as boundaries for modeling. For each wall, both the translational and rotational velocities can be specified. However, it should be noted that an applied force cannot be specified, but it may be measured and used by controlling the wall velocity, e.g. via a numerical servo-mechanism. Since forces acting on a wall do not influence its motion, contacts may not exist between two walls. Thus, contacts in PFC\(^2\)D are either ball–ball or ball–wall.

3.2.2 Calculation Cycle and Basic Laws

The calculation cycle in PFC\(^2\)D is performed via a timestepping algorithm that consists of the repeated application of the law of motion to each particle, a force-displacement law to each contact, and a constant updating of wall positions. The calculation cycle is shown in Figure 3.1. It is noted that the velocities and accelerations are assumed to be constant within each timestep. Based on the resultant force and moment, the law of motion is applied to each particle to update its velocity and position. The wall positions are also updated based on the specified wall velocities. Then the force–displacement law is applied to each contact to update the contact forces based on the relative motion between the two entities at the contact and the contact constitutive model. The calculation is triggered by applying velocities on particles/walls or applying forces on particles. Once the positions of the particles/walls or the contact forces reach the expected values the calculation cycle stops. The force–displacement law and the law of motion are described in this section.
3.2 Discrete Element Modeling using PFC2D

3.2.2.1 Force–Displacement Law

The force–displacement law describes the relative displacement between two entities at a contact due to the contact force acting on them. As mentioned before, the contacts in PFC2D are either ball–ball or ball–wall; hence, the force–displacement law is illustrated for both ball–ball and ball–wall contacts. Figures 3.2 and 3.3 show the ball–ball contact with two spherical particles (labeled A and B) and the ball–wall contact with a spherical particle and a wall (labeled b and w), respectively. In both cases, $U^n$ denotes overlap and $x_i^{[C]}$ represents the location of the contact point.

![Figure 3.1](image1)

**Figure 3.1** Calculation cycle in PFC$^{2D}$ (Itasca, 2008).

![Figure 3.2](image2)

**Figure 3.2** Notation used to describe ball–ball contact (adopted from Itasca, 2008).
The contact force vector \( F_i \), which represents the action of ball A on ball B for ball–ball contact and the action of the ball on the wall for ball–wall contact, can be resolved into normal component \( F_i^n \) and shear component \( F_i^s \) as follows

\[
F_i = F_i^n + F_i^s
\]  

(3.1)

The normal contact force vector \( F_i^n \) is calculated by

\[
F_i^n = K_n \cdot U^n \cdot n_i
\]  

(3.2)

where \( K_n \) is the normal contact stiffness at the contact;

\( U^n \) is the overlap between two entities;

\( n_i \) is the unit normal vector.

The shear contact force is computed in an incremental fashion. When the contact is formed, the total shear contact force is initialized to zero. Each subsequent relative shear–displacement increment results in an increment of elastic shear force that is added to the current value. The shear elastic force increment vector is calculated by

\[
\Delta F_i^s = -k^s \cdot V_i^s \cdot \Delta t
\]  

(3.3)

where \( k^s \) is the shear stiffness at the contact;

\( V_i^s \) is the shear velocity vector;

\( \Delta t \) is the timestep.

The new shear contact force vector \( F_i^s \) is then computed by summing the old shear contact force vector at the start of the timestep with the shear elastic force increment vector

\[
F_i^s = \{ F_i^s \}^{\text{old}} + \Delta F_i^s
\]  

(3.4)
3.2.2.2 Law of Motion

The motion of a single rigid particle is determined by the resultant force and moment vectors acting on it. This can be illustrated in terms of the translational motion of a point in the particle and rotational motion of the particle. The equation for the translational motion is defined as

$$F_i = m (\ddot{x}_i - g_i) \quad (3.5)$$

where $F_i$ is the sum of all resultant forces acting on the particle;

$m$ is the mass of the particle;

$\ddot{x}_i$ is the acceleration of the particle;

$g_i$ is the body force acceleration vector.

The equation for the rotational motion is written as

$$M_i = I \cdot \dot{\omega}_i = (\beta \cdot m \cdot R^2) \cdot \dot{\omega}_i \quad (3.6)$$

where $M_i$ is the resultant moment acting on the particle;

$I$ is the principal moment of inertia of the particle;

$\dot{\omega}_i$ is the angular acceleration of the particle;

$\beta$ is the particle shape coefficient ($\beta=2/5$ for spherical particles and $\beta=1/2$ for disk-shaped particles);

$R$ is the radius of the particle.

The two equations of motion (Equations 3.5 and 3.6) are integrated using a centered finite-difference procedure with a timestep of $\Delta t$. The quantities $x_i$, $\dot{x}_i$, $\ddot{x}_i$, $F_i$ and $M_i$ are calculated at the primary intervals of $t \pm n \Delta t$. The accelerations are computed as

$$\ddot{x}_i = \frac{1}{\Delta t} \left( x_i^{(t+\Delta t/2)} - x_i^{(t-\Delta t/2)} \right) \quad (3.7)$$

$$\dot{\omega}_i = \frac{1}{\Delta t} \left( \omega_i^{(t+\Delta t/2)} - \omega_i^{(t-\Delta t/2)} \right) \quad (3.8)$$

Inserting the above two equations into Equations 3.5 and 3.6, and solving for the velocities at time $(t+\Delta t/2)$, results in

$$x_i^{(t+\Delta t/2)} = x_i^{(t-\Delta t/2)} + \left( \frac{F_i^{(t)}}{m} + g_i^{(t)} \right) \cdot \Delta t \quad (3.9)$$

$$\omega_i^{(t+\Delta t/2)} = \omega_i^{(t-\Delta t/2)} + \left( \frac{M_i^{(t)}}{I} \right) \cdot \Delta t \quad (3.10)$$
Finally, the velocities obtained from Equations 3.9 and 3.10 are used to update the positions of the particle center as

\[ x_i^{(t+\Delta t)} = x_i^{(t)} + \dot{x}_i^{(t+\Delta t/2)} \cdot \Delta t \]  

(3.11)

To sum up, the calculation cycle for the law of motion can be summarized as follows. Given the values of \( \dot{x}_i^{(t-\Delta t/2)} \), \( \omega_i^{(t-\Delta t/2)} \), \( x_i^{(t)} \), \( F_i^{(t)} \) and \( M_i^{(t)} \), the values of \( \dot{x}_i^{(t+\Delta t/2)} \) and \( \omega_i^{(t+\Delta t/2)} \) are obtained using Equations 3.9 and 3.10. Then Equation 3.11 is used to obtain \( x_i^{(t+\Delta t)} \). As illustrated in the calculation cycle in Figure 3.1, the force–displacement law is again used to compute the values of \( F_i^{(t+\Delta t)} \) and \( M_i^{(t+\Delta t)} \) for the next cycle.

### 3.2.3 Clump Logic

The clump logic provides a method to create groups of slaved balls to model particles with arbitrary shapes. The balls, which are used to comprise a clump, may overlap to any extent but they remain at a fixed distance from each other; hence, contact forces are not generated between these balls within the clump. A clump acts as a rigid body that will not break apart, regardless of the forces acting on it, which is different from a group of balls that are bonded to each other.

The basic mass properties of a clump, i.e. the total mass of a clump \( m \), the location of the mass center of a clump \( x_i^{[G]} \) and the moments and products of inertia \( I_{ii} \) and \( I_{ij} \), are computed by the following equations.

\[ m = \sum_{p=1}^{N_p} m^{[p]} \]  

(3.12)

\[ x_i^{[G]} = \frac{1}{m} \cdot \sum_{p=1}^{N_p} m^{[p]} x_i^{[p]} \]  

(3.13)

\[ I_{ii} = \sum_{p=1}^{N_p} \left \{ m^{[p]} \cdot (x_i^{[p]} - x_i^{[G]}) \cdot (x_i^{[p]} - x_i^{[G]}) + \frac{2}{5} \cdot m^{[p]} \cdot R^{[p]} \cdot R^{[p]} \right \} \]  

(3.14)

\[ I_{ij} = \sum_{p=1}^{N_p} \left \{ m^{[p]} \cdot (x_i^{[p]} - x_i^{[G]}) \cdot (x_j^{[p]} - x_j^{[G]}) \right \} \]  

(3.15)

where \( N_p \) is the number of the balls in the clumps;

\( m^{[p]} \) is the mass of a ball;

\( x_i^{[p]} \) is the centroid location of the ball;

\( R^{[p]} \) is the radius of the ball.
The motion of a clump is determined by the resultant force and moment vectors acting on it. Because a clump is treated as a rigid body, its motion can be described in terms of the translational motion of a point in the clump and the rotational motion of the entire clump. The equation for the translational motion is defined as

$$F_i = m \left( \ddot{x}_i - g_i \right)$$  \hspace{1cm} (3.16)

where $F_i$ is the sum of all externally applied forces acting on the clump;

$m$ is the total mass of the clump;

$\ddot{x}_i$ is the acceleration of the clump;

$g_i$ is the body force acceleration vector.

The equation for the rotational motion is expressed in the matrix form as

$$\{M\} - \{W\} = [I]\{\alpha\}$$  \hspace{1cm} (3.17)

with

$$\{M\} = \begin{pmatrix} M_1 \\ M_2 \\ M_3 \end{pmatrix}$$

$$\{W\} = \begin{pmatrix} \omega_2 \omega_3 (I_{33} - I_{22}) + \omega_3 \omega_1 I_{23} - \omega_2 \omega_1 I_{32} - \omega_3 \omega_1 I_{21} + \omega_1 \omega_3 I_{21} \\ \omega_3 \omega_1 (I_{11} - I_{33}) + \omega_1 \omega_1 I_{31} - \omega_3 \omega_3 I_{13} - \omega_2 \omega_1 I_{12} + \omega_2 \omega_1 I_{32} \\ \omega_1 \omega_2 (I_{22} - I_{11}) + \omega_2 \omega_2 I_{12} - \omega_1 \omega_1 I_{21} - \omega_3 \omega_1 I_{23} + \omega_3 \omega_2 I_{13} \end{pmatrix}$$

$$[I] = \begin{bmatrix} I_{11} & -I_{12} & -I_{13} \\ -I_{21} & I_{22} & -I_{23} \\ -I_{31} & -I_{32} & I_{33} \end{bmatrix}$$

$$\{\alpha\} = \begin{pmatrix} \alpha_1 \\ \alpha_2 \\ \alpha_3 \end{pmatrix} = \begin{pmatrix} \dot{\omega}_1 \\ \dot{\omega}_2 \\ \dot{\omega}_3 \end{pmatrix}$$

where $\{M\}$ is the resultant moment about the center of mass;

$\omega_i$ is the angular velocity about the principal axis;

$\dot{\omega}_i$ is the angular acceleration about the principal axis.

The Equations 3.16 and 3.17 are integrated using a centered finite-difference procedure involving a timestep of $\Delta t$ as described in Section 3.2.2.
3.2.4 Contact Constitutive Models

The constitutive behavior of a material is modeled in PFC2D associating a contact model with each contact. In general, the basic contact constitutive models can be classified into the following four types:

1. Linear contact stiffness model;
2. Slip model;
3. Contact bond model;
4. Parallel bond model.

3.2.4.1 Linear Contact Stiffness Model

The contact stiffnesses relate the contact forces and relative displacement in the normal and shear directions via the force–displacement law. PFC2D provides two types of contact stiffness models: linear and simplified Hertz-Mindlin model. Due to the benefit of computational time (Ngo, 2012), the linear contact stiffness model is used in this study. This model is defined by the normal and shear contact stiffness (\(k_n\) and \(k_s\)) of the two contacting entities (ball–ball or ball–wall). The normal stiffness is a secant stiffness representing the total normal force to the total displacement, whilst the shear stiffness is a tangent stiffness relating the increment of shear force to the increment of shear displacement (Itasca, 2008). According to the manual book, the contact normal and shear stiffness can be calculated as follows.

\[
K_n = \frac{k_n^{[A]} k_n^{[B]}}{k_n^{[A]} + k_n^{[B]}} \quad (3.18)
\]

\[
k_s = \frac{k_s^{[A]} k_s^{[B]}}{k_s^{[A]} + k_s^{[B]}} \quad (3.19)
\]

where the superscripts [A] and [B] denote the two entities in contact.

3.2.4.2 Slip Model

The slip model provides a relation between shear and normal force between two contacting entities, such that they may slip relative to one another. The slip behavior is determined by the friction coefficient at the contact (\(\mu\)) where \(\mu\) is defined to be the minimum friction coefficient of the two contacting entities. The maximum allowable shear contact force is calculated by

\[
F_{max}^s = \mu |F_n^s| \quad (3.20)
\]

If \(|F_i^s| > |F_{max}^s|\), then slip is allowed to occur during the next calculation cycle by setting the magnitude of \(F_i^s\) equal to \(F_{max}^s\) via
\[ F_i^s \leftarrow F_i^s \left( F_{\text{max}}^s / |F_i^s| \right) \]  \hspace{1cm} (3.21)

The contact logic based on the linear contact stiffness model and the slip model is shown in Figure 3.4.

\[ F_n = K_n \cdot U_n \]
\[ \Delta F^s = k^s \cdot \Delta U^s \]

**Slip model**

\[ F^s \leq \mu F_n \] \hspace{0.2cm} (\( \mu = \text{friction coefficient} \))

**Figure 3.4** Contact logic (modified from Itasca, 2008).

### 3.2.4.3 Contact Bond Model

The bonding model in PFC\(^2\)D allows particles to be bonded together at the contacts. Two standard bonding models are supported in the software: contact bond model and parallel bond model. The contact bond model is illustrated in Figure 3.5.

**Contact bond model**

*models adhesion over vanishingly small area of contact point (does not resist moment)*

*breaks if normal or shear force \((F_{bn} \text{ or } F_{bs})\) exceeds the corresponding bond strength*

**Figure 3.5** Contact bond logic (modified from Itasca, 2008).

The contact bond can only transfer forces acting at the contact points and the contact bond model is defined by the following two parameters: normal contact bond strength \((F_{bn}^n)\) and shear contact...
bond strength ($F_c^s$). The contact bond breaks when the contact force exceeds either the normal contact bond strength or the shear contact bond strength, as shown in Figure 3.6.

![Figure 3.6](image)

(a) Normal component of contact force.

(b) Shear component of contact force.

**Figure 3.6** Force–displacement behavior for contact occurring at a point (adopted from Itasca, 2008).

### 3.2.4.4 Parallel Bond Model

The parallel bonds establish an elastic interaction between particles that acts in parallel with a slip or a contact bond model described above. Hence, the existence of a parallel bond does not preclude the possibility of slip. Both forces and moments can be transferred among particles using the parallel bond model, as shown in Figure 3.7.
The force-displacement behavior of the parallel bond is similar to that of the contact bond as shown in Figure 3.6. The parallel bond model is defined by the following five parameters: normal and shear stiffness ($\bar{k}_n$ and $\bar{k}_s$), normal and shear strength ($\bar{\sigma}_c$ and $\bar{\tau}_c$), and bond radius ($\bar{R}$). The parallel bond radius ($\bar{R}$) equals a radius multiplier ($\lambda$) times the minimum radius of the two bonded balls, as shown in Figure 3.8.

**Figure 3.7** Parallel bond logic (modified from Itasca, 2008).

**Figure 3.8** Parallel bond depicted as a finite-sized piece of cementation material (adopted from Itasca, 2008).
The total force and moment associated with the parallel bond are denoted by $F_i$ and $M_i$, as shown in Figure 3.8. The force vector can be calculated by the normal and shear component vectors ($F_i^n$ and $F_i^s$) with respect to the contact plane as

$$F_i = F_i^n + F_i^s$$  \hspace{1cm} (3.22)

When the bond is formed, $F_i$ and $M_i$ are initialized to zero. Each subsequent relative displacement- and rotation-increment at the contact results in an increment of elastic force and moment that is added to the current values, that is

$$F_i^n = \{F_i^n\}_{\text{old}} + \Delta F_i^n$$  \hspace{1cm} (3.23)

$$F_i^s = \{F_i^s\}_{\text{old}} + \Delta F_i^s$$  \hspace{1cm} (3.24)

$$M_i = \{M_i\}_{\text{old}} + \Delta M_i$$  \hspace{1cm} (3.25)

The maximum tensile and shear stresses acting on the bond periphery are calculated (via beam theory) as

$$\sigma_{\text{max}} = \frac{-F_i^n}{A} + \frac{|M_i|}{I} \cdot \bar{R}$$  \hspace{1cm} (3.26)

$$\tau_{\text{max}} = \frac{|F_i^s|}{A}$$  \hspace{1cm} (3.27)

where $A$ is the area of the bond cross section;

$I$ is the moment of inertia of the bond cross section.

The values of $A$ and $I$ can be computed as

$$A = \begin{cases} \pi \bar{R}^2 & \text{(SET disk off)} \\ 2\bar{R}t & \text{(SET disk t)} \end{cases}$$  \hspace{1cm} (3.28)

$$I = \begin{cases} \frac{1}{64} \pi (2\bar{R})^4 = \frac{1}{4} \pi \bar{R}^4 & \text{(SET disk off)} \\ \frac{1}{12} t(2\bar{R})^3 = \frac{2}{3} t\bar{R}^3 & \text{(SET disk t)} \end{cases}$$  \hspace{1cm} (3.29)

If the maximum tensile stress exceeds the normal strength ($\sigma_{\text{max}} \geq \sigma_c$), or the maximum shear stress exceeds the shear strength ($\tau_{\text{max}} \geq \tau_c$), the parallel bond breaks. The parallel bond model is used to simulate the geogrid properties in this study.
3.3 Model Development and Calibration for Soil

Due to the limited numbers of micro parameters in the linear contact stiffness model (normal and shear contact stiffness – $k_n$, $k_s$) and in the slip model (friction coefficient – $\mu$), both models have been commonly used in the DEM investigations with PFC (Bhandari and Han, 2010; Stahl and Konietzky, 2011; Zhang et al., 2013). In this study, the soil was modeled as unbonded particles using the above two models. Different approaches for converting porosities from 3D to 2D are reviewed and a new suggestion consisting of a parabolic equation and an iterative criterion is proposed for determining 2D porosities in DEM studies. With further DEM simulations, this new suggestion has been proven to be rational. Finally, the calibration results for sand and gravel are presented at the end of this section by comparing the simulation results with the corresponding experimental data.

3.3.1 Current Approaches for Determination of 2D Porosity

It is well known that DEM simulation results are highly determined by the selected models and the corresponding micro input parameters, e.g. contact stiffness and friction coefficient of the particles and walls, modulus, damping, Poisson’s ratio, etc. Many studies have focused on the influences of those input parameters on the material behavior (Härtl and Ooi, 2008; Abbireddy and Clayton, 2010; Mohamed and Gutierrez, 2010; Zhou et al., 2013). It is possible to draw up a cause-and-effect guideline to rapidly determine the micro parameters for DEM studies, e.g. the macro initial Young’s modulus of the material is linearly related to the micro contact stiffness; the macro peak strength of the material depends on the micro friction coefficient. The effects of all the above parameters, however, are all based on certain particle packing with defined porosities. The porosities used in 3D DEM studies can be determined from laboratory porosities directly. However, an area-based 2D porosity is entirely different from a volume-based 3D porosity so that the porosities used in 2D DEM investigations should be determined with appropriate approaches based on laboratory porosities. Although this additional step of converting porosities from 3D to 2D requires careful engineering judgements, 2D software has been widely used for the investigations of various geotechnical problems due to the advantages of giving insights into key phenomena and mechanisms with less computational time (Wang and Leung, 2008; Bhandari and Han, 2010; Jiang et al., 2011; Jia et al., 2013; Zhang et al., 2013; Ai et al., 2014). Selecting an appropriate approach to determine the 2D porosity in DEM studies is the foremost step for all 2D DEM simulations and further analyses. After fixing the 2D porosity, the calibrations of the models and the input parameters can be proceeded in further DEM studies.

Currently, there are six approaches commonly used to link 2D and 3D porosities for DEM studies. All those methods are summarized and evaluated as follows.

3.3.1.1 The Densest State Method

Assuming particles with identical diameters in both 2D and 3D, Hoomans et al. (1996) derived Equation 3.30 by matching a 2D hexagonal packing structure and a 3D face centered cubic (FCC)
structure (see Figure 3.9). Since the hexagonal packing structure and the FCC structure represent the densest 2D and 3D packing in mono-sized systems, the following equation is named as the Densest State Method in this study.

\[
n_{2D} = 1 - \left[ \frac{\sqrt{\pi/3}}{2} \cdot (1 - n_{3D}) \right]^{2/3} \tag{3.30}
\]

where \(n_{2D}\) and \(n_{3D}\) are the porosities in 2D and 3D, respectively.

The Densest State Method has also been used by Lin et al. (2013) to simulate sandy soil reinforced with horizontal-vertical reinforcing elements in plane strain tests.

van Wachem et al. (2001) modified the Densest State Method by introducing an empirical parameter containing the maximum experimental solids packing in practice.

\[
n_{2D} = 1 - \left[ \frac{\sqrt{\pi/3}}{2v} \cdot (1 - n_{3D}) \right]^{2/3} \tag{3.31}
\]

where \(v = \frac{n_{\text{lab, min}}}{n_{3D, \text{min}}}\); \(n_{\text{lab, min}}\) and \(n_{3D, \text{min}}\) are the minimum 3D porosities in laboratory tests and theoretical calculations, respectively.

3.3.1.2 The Loosest State Method

With a similar derivation process of the Densest State Method, the Loosest State Method is obtained by matching a 2D square packing structure and a 3D simple cubic structure (see Figure 3.10) that represent the loosest 2D and 3D packing in mono-sized systems, as shown in Equation 3.32. Helland et al. (2005) used the identical equation in a numerical study of cluster and particle rebound effects in a circulating fluidized bed, but they used a pseudo-3D concept in which they assumed that the depth of the fluidized bed was equal to the particle diameter.

\[
n_{2D} = \frac{3}{2} n_{3D} - \frac{1}{2} \tag{3.32}
\]
3.3.1.3 Interval Mapping Method

Ouyang and Li (1999) proposed the Interval Mapping Method, as shown in Equation 3.33. Compared with Equation 3.30, the denominator in the fraction of Equation 3.33 is $\sqrt{2}$. They verified their method by comparing a 2D hexagonal packing structure with a 3D hexagonal close packing (HCP) structure (see Figure 3.11). Both structures were based on mono-sized systems.

\[
n_{2D} = 1 - \left[ \frac{\sqrt{\pi \sqrt{3}}}{\sqrt{2}} (1 - n_{3D}) \right]^{2/3}
\]

3.3.1.4 Combination Method

Zhang (2007) combined the Densest State Method and the Interval Mapping Method by introducing the relative density $D_{r}$, which represents different density states in soil mechanics. The Combination Method is illustrated as follows

\[
n_{2D} = 1 - \left( \frac{1 - n_{lab}}{\xi} \right)^{2/3}
\]

where $\xi = \frac{\sqrt{5}}{\sqrt{\pi \sqrt{3}}} + D_{r} \left( \frac{2}{\sqrt{\pi \sqrt{3}}} - \frac{\sqrt{5}}{\sqrt{\pi \sqrt{3}}} \right)$;
\[ D_t = \frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}} \]

\( e_{\text{max}} \), \( e_{\text{min}} \) and \( e \) are the maximum, the minimum and the test void ratios in laboratory tests, respectively.

It should be noted that when \( D_t = 0 \), the Combination Method is the same as the Interval Mapping Method; when \( D_t = 1 \), the Combination Method is identical to the Densest State Method.

### 3.3.1.5 Linear Interpolation Method (LIM)

The definition of density in soil mechanics can also be used to convert porosities from 3D to 2D. The Linear Interpolation Method is based on the assumption that the degree of densities in both 2D and laboratory tests (3D) are identical.

The degree of densities depending on the maximum, the minimum and the test porosities in both 2D and laboratory tests can be expressed by

\[ D = \frac{n_{2D,\text{max}} - n_{2D}}{n_{2D,\text{max}} - n_{2D,\text{min}}} = \frac{n_{\text{lab},\text{max}} - n_{\text{lab}}}{n_{\text{lab},\text{max}} - n_{\text{lab},\text{min}}} \quad (3.35) \]

where \( n_{2D,\text{max}} \), \( n_{2D,\text{min}} \) and \( n_{2D} \) are the maximum, the minimum and the test porosities in 2D;

\( n_{\text{lab},\text{max}} \), \( n_{\text{lab},\text{min}} \) and \( n_{\text{lab}} \) are the maximum, the minimum and the test porosities in laboratory tests.

Transforming Equation 3.35, the test 2D porosity can be computed with the following equation

\[ n_{2D} = n_{2D,\text{max}} - D(n_{2D,\text{max}} - n_{2D,\text{min}}) \quad (3.36) \]

where \( D = \frac{n_{\text{lab},\text{max}} - n_{\text{lab}}}{n_{\text{lab},\text{max}} - n_{\text{lab},\text{min}}} \).

Using the Linear Interpolation Method, Giese (2002) and Hainbüchner et al. (2002) conducted numerical simulations of vibroflotation compactions and shallow foundation stabilities, respectively.

### 3.3.1.6 Quasi 3D Porosity Method

The Quasi 3D Porosity Method was based on the assumption that the diameter and thickness of the “2D discs” were equal to the diameter of the sphere (Bezuijen and van den Berg, 2002). After comparing the volume equations of the disc and the sphere, a quasi 3D porosity equation was proposed to compute the maximum and minimum quasi 3D porosities (\( n'_{3D,\text{max}} \) and \( n'_{3D,\text{min}} \)) with the following equations

\[ n'_{3D,\text{max}} = \frac{1}{3} + \frac{2}{3} n_{2D,\text{max}} \quad (3.37) \]
3.3 Model Development and Calibration for Soil

\[ n'_{3D,\text{min}} = \frac{1}{3} + \frac{2}{3} n_{2D,\text{min}} \]  

(3.38)

Subsequently, the Linear Interpolation Method from Equation 3.36 was used to calculate the quasi 3D porosity \( n'_{3D} \) as follows:

\[ n'_{3D} = n'_{3D,\text{max}} - \frac{n_{\text{lab,\text{max}}} - n_{\text{lab}}}{n_{\text{lab,\text{max}}} - n_{\text{lab,\text{min}}} \cdot (n'_{3D,\text{max}} - n'_{3D,\text{min}})} \]  

(3.39)

Finally, the 2D porosity is obtained with the following equation:

\[ n_{2D} = \frac{3}{2} n'_{3D} - \frac{1}{2} \]  

(3.40)

This method can be regarded as the combination of the Loosest State Method and the Linear Interpolation Method.

Figure 3.12 shows the relations between 2D and laboratory porosities based on the above different approaches, in which the maximum and the minimum porosities of sand in laboratory tests are 0.4571 and 0.3253, respectively. From Figure 3.12 we can see that the 2D porosities obtained with the Combination Method decrease with increasing laboratory porosities, which is against the general relation between 2D and laboratory porosities. The negative values of 2D porosities derived with the Loosest State Method and the Interval Mapping Method demonstrate that both methods cannot be used in dealing with soil mechanical problems. According to the definition of the Densest State Method, this method was proposed based on an extreme packing with monosized systems and thus it cannot be used in a wide range of polydisperse particle systems. The Quasi 3D Porosity Method is only applicable to the case that the diameter and thickness of the “2D discs” are equal to the diameter of the sphere. The Linear Interpolation Method illustrated above is obtained by matching the theoretical maximum and minimum 2D porosities with the practical maximum and minimum laboratory porosities. Hence, the method is regarded as \( \text{LIM}_{2D-\text{lab}} \), as shown in Figure 3.13. However, the theoretical maximum and minimum 2D porosities should correspond to the theoretical maximum and minimum 3D porosities based on monosized systems, i.e. the method should be \( \text{LIM}_{2D-3D} \), as shown in Figure 3.13. The \( \text{LIM}_{2D-3D} \) is independent on the maximum and the minimum porosities from the laboratory tests. Corresponding to a laboratory porosity of \( n_{\text{lab}} = 0.3434 \) (dense packing with a relative density of \( D_r = 0.89 \)), the 2D porosity obtained from the \( \text{LIM}_{2D-3D} \) is 0.14 and the 2D porosity calculated based on the \( \text{LIM}_{2D-\text{lab}} \) is 0.11. The differences of the 2D porosities based on the \( \text{LIM}_{2D-3D} \) and the \( \text{LIM}_{2D-\text{lab}} \) are compared in Figure 3.13.

Moreover, due to the varying particle size distributions of soil in practice, it might be impossible just to use a specific equation to link 2D and 3D porosities for polydisperse particle systems. Therefore, it can be concluded that none of the above approaches can meet the requirements of linking 2D and 3D porosities in a wide range of polydisperse particle systems, especially in dealing
with soil mechanical problems. It is necessary to use an alternative way to link 2D and 3D porosities especially for arbitrary assemblies with various polydisperse particle systems.

![Figure 3.12](image1.png)

**Figure 3.12** Relations between 2D and laboratory porosities based on different methods.

![Figure 3.13](image2.png)

**Figure 3.13** Relations between 2D and 3D/laboratory porosities based on LIM.

### 3.3.2 A New Suggestion for Determining 2D Porosities

In this study, a new suggestion consisting of a parabolic equation and an iterative criterion is proposed for the determination of 2D porosities in DEM studies. Although the $\text{LIM}_{2\text{D}-3\text{D}}$ is based on a mono-sized system, the two endpoints of this linear relationship between 2D and 3D porosities are used as a rough guideline for developing the parabolic equation for linking 2D and 3D porosities.
porosities. Moreover, when the porosity in 3D (or in laboratory) is 0 in a polydisperse particle system, the corresponding 2D porosity should also be 0. Therefore, the initial parabolic equation is obtained based on the above three points, as shown in Figure 3.13. It should be noted that the parabolic equation is independent on the maximum and the minimum porosities in the laboratory and it is only an initial guideline for converting porosities from 3D to 2D. The parabolic equation is shown as follows

\[ n_{2D} = 0.42 \times n_{lab}^2 + 0.25 \times n_{lab} \]  

(3.41)

where: \( n_{2D} \) is the initial 2D porosity;  
\( n_{lab} \) is the laboratory porosity.

\[ n_{2D} = 0.42 \times n_{lab}^2 + 0.25 \times n_{lab} \]

Preliminary simulation and measure \( F_{top} \)

\[ 0 < F_{top} < 1\% \times G_{specimen} \]

Set vertical velocity to top plate

Change vertical velocity

Measure \( F_{top} \)

\[ |n_{2D,new} - n_{2D}| < 0.005 \]

\[ a: F_{top} = 0 \]
\[ b: F_{top} \geq 1\% \times G_{specimen} \]

\[ n_{2D} \text{ used for further simulations} \]

Yes

\[ 0 < F_{top} < 1\% \times G_{specimen} \]

No

\[ a: n_{2D} = n_{2D} - 0.01 \]
\[ b: n_{2D} = n_{2D} + 0.01 \]

\[ a: n_{2D} = n_{2D} - 0.01 \]
\[ b: n_{2D} = n_{2D} + 0.01 \]

Figure 3.14 Flow chart of the iterative criterion for the determination of the final 2D porosity.

The final 2D porosity, which can be used for further DEM simulations, is suggested to be determined through an iterative process according to the flow chart in Figure 3.14. First of all, the numerical specimen is prepared using the initial 2D porosity obtained from Equation 3.41. Then, preliminary DEM simulations are conducted to record the contact force distribution in the
Numerical Modeling and Calibration using PFC2D

3.3.3 Numerical Direct Shear Tests and Calibration Results

In order to verify the new suggestion for determining 2D porosities to be rational for further DEM simulations and to calibrate the models and input parameters for sand and gravel, numerical direct shear tests are carried out in this section based on the corresponding laboratory tests. All the experimental direct shear tests were conducted using the large-scale direct shear apparatus ($W/D/H = 305/305/124$ mm$^3$) at the geotechnical laboratory of RWTH Aachen University. For the numerical simulations of direct shear tests, a direct shear box was created with the same dimensions in width and height ($W = 305$ mm, $H = 124$ mm) as the one used in the laboratory tests. The physical specimens were prepared with four horizontal layers by compacting each layer to the target testing density. In both the experimental and the DEM investigations, a fixed shear rate of 0.384 mm/min was applied during the shearing process, while the normal stress applied on top of the specimen was kept constant.

3.3.3.1 Sand

In the laboratory tests, dry sand with the particle size distribution shown in Figure 3.15 has been used. The sand was compacted into the direct shear box with a compaction degree of 101% proctor density ($\rho_{pr} = 1.725$ g/cm$^3$) so that the testing density of 1.74 g/cm$^3$ could be achieved and the corresponding relative density of the soil sample was $D_r = 0.89$. 

specimen. After the numerical specimen reaches the equilibrium state, the top boundary is fixed to keep the target 2D porosity constant. The vertical force on the top plate of the specimen $F_{top}$ is measured. If $F_{top}$ is greater than 0 but less than 1% of the specimen’s weight, it represents that the specimen area is full of particles with the target 2D porosity and meanwhile the contact force distribution in the specimen is regarded to be rational according to the realistic contact force distribution under gravity. Hence, the initial 2D porosity can be used for further simulations. If not, the top plate moves downwards or upwards until $F_{top}$ is greater than 0 but less than 1% of the specimen’s weight. In this study, the criterion of 1% of the specimen’s weight is used and it could fulfill the requirements of both the target 2D porosity and the realistic contact force distribution. It should be noted that the value of the criterion could be smaller than 1% of the specimen’s weight, but it will cause more iteration steps and increase the computational time. Moreover, if the value of the criterion is larger than 1% of the specimen’s weight, the contact force distribution might not be as rational as that in reality.

The relative displacement of the top plate is measured and the new 2D porosity can be calculated and compared with the initial 2D porosity. If the relative change of the 2D porosities is less than 0.005, the effect of porosity differences on the stress–strain relations of soil is quite small, especially for dense specimens according to the studies by Zeng (2006). Thereby, the effects can be neglected and the initial 2D porosity could be used for further DEM simulations. If not, the initial 2D porosity has to be changed correspondingly and new preliminary simulations have to be conducted with the changed 2D porosity again, as shown in Figure 3.14. The main FISH codes of the iterative process are listed in Appendix A.
Since the computational time in DEM simulations is highly depending on the numbers of particles, the particle “up-scaling” technique has been commonly used in DEM studies to balance the computational cost against the scaling effect on the sample response (Wang and Leung, 2008; Lin et al., 2013; Tran et al., 2013). It can be seen in those studies that the mechanical behavior of the investigated objects with the technique of “up-scaling” has been investigated satisfactorily. Therefore, the particle sizes of sand in the laboratory tests were modified and increased with an up-scaling factor of 10 for DEM simulations in this study, as shown in Figure 3.15.

![Figure 3.15 Particle size distributions of sand in laboratory tests and in DEM simulations.](image)

(1) Determination of 2D porosity and numerical specimen preparation

In this study, the testing laboratory porosity was $n_{lab} = 0.3434$. With Equation 3.41, the initial 2D porosity was obtained ($n_{2D} = 0.14$). The corresponding number of the soil particles in the numerical direct shear box was 2591. The micro input parameters for validating the 2D porosity in the numerical direct shear tests are listed in Table 3.1. It should be noted that the thicknesses of the soil particles along the plane of paper were 8 mm so that the calibration results could be used in further investigations with one geogrid tensile member in Chapter 4.

Identical to the physical specimen preparation process, the numerical specimen was prepared with a multilayer compaction method (four horizontal layers in this study). The procedure of the multilayer compaction method is shown in Figure 3.16, which was similar to the approach proposed by Jiang et al. (2003). For each layer, the standard of the equilibrium state was that the maximum contact force ratio (defined as the ratio of the maximum unbalanced force to the maximum contact force) was smaller than 0.001. In order to prepare a dense specimen, a very small friction coefficient of $f_0 = 0.05$ was set to the particles according to the suggestion by Härtl and Ooi (2008). After the last layer reached the equilibrium state, the friction coefficient of the particles was then increased to a large value ($f_s = 3$ in this study) so as to compensate the lack of angularity for circular particles. Large friction coefficients of particles have also been used in previous studies (Lin et al., 2013; Zhang et al., 2013) for circular particles in DEM studies. The
friction coefficient between the particles and the walls $f_{ws}$ was kept constant during the whole process.

**Table 3.1** Input parameters of sand in numerical direct shear tests.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density of the solids $\rho_s$ [kg/m$^3$]</td>
<td>2650</td>
</tr>
<tr>
<td>Water content $w$ [%]</td>
<td>0</td>
</tr>
<tr>
<td>Particle diameters $d_s$ [mm]</td>
<td>Gradation as in Figure 3.15</td>
</tr>
<tr>
<td>Normal contact stiffness of the walls $k_{n,w}$ [N/m]</td>
<td>$1 \times 10^7$</td>
</tr>
<tr>
<td>Shear contact stiffness of the walls $k_{s,w}$ [N/m]</td>
<td>$1 \times 10^7$</td>
</tr>
<tr>
<td>Normal contact stiffness of the particles $k_{n,s}$ [N/m]*</td>
<td>$4 \times 10^5$</td>
</tr>
<tr>
<td>Shear contact stiffness of the particles $k_{s,s}$ [N/m]*</td>
<td>$4 \times 10^5$</td>
</tr>
<tr>
<td>Friction coefficient of the particles for specimen preparation $f_0$ [-]</td>
<td>0.05</td>
</tr>
<tr>
<td>Friction coefficient of the particles after specimen preparation $f_s$ [-]</td>
<td>3</td>
</tr>
<tr>
<td>Friction coefficient between the particles and the walls $f_{ws}$ [-]</td>
<td>1.5</td>
</tr>
</tbody>
</table>

* The initial normal and shear contact stiffnesses of the particles were chosen according to similar DEM simulations (Wang and Leung, 2008; Jiang et al., 2011; Han et al., 2012; Lin et al., 2013; Zhang et al., 2013) and adjusted according to the calibration of the numerical direct shear test results with the corresponding experimental data in this section.

![Figure 3.16](image)

**Figure 3.16** Specimen preparation process with the multilayer compaction method.

The contact force distribution with the initial 2D porosity of $n_{2D} = 0.14$ is shown in Figure 3.17. The contact forces distribute all over the specimen. Moreover, the vertical force measured on both the top and the bottom plates were quite large ($F_{top} = 3854$ kN/m and $F_{bottom} = 3855$ kN/m), which were much larger than the self-weight of the specimen ($G_{specimen} = \rho_s \cdot (1 - n_{2D}) \cdot W \cdot H \cdot g = 846$ N/m).
Hence, further steps were conducted and the contact force distribution after loosening the top plate is shown in Figure 3.18. The contact force distribution in Figure 3.18 is more realistic compared with that in Figure 3.17, i.e. the contact forces were increasing with depth of the specimen under gravity and the contact forces at the lower parts of the specimen were larger than those at the upper parts of the specimen. However, the relative change of the 2D porosities was 0.008, which was greater than the required value. Hence, further DEM simulations with a new 2D porosity $n_{2D} = 0.15$ were carried out.

![Fixed top boundary](image1)

**Figure 3.17** Contact force distribution in the specimen with $n_{2D} = 0.14$ (thickness of lines proportional to magnitude).

![Initial position of the top plate](image2)

**Figure 3.18** Contact force distribution in the specimen after loosening the top plate (thickness of lines proportional to magnitude).

Figure 3.19 shows the contact force distribution in the specimen with the new 2D porosity of $n_{2D} = 0.15$, in which $F_{\text{top}}$ meets the requirement of the iterative criterion with a reasonable value of $F_{\text{top}} = 2.7 \text{ N/m}$. The vertical force measured on the bottom plate was $F_{\text{bottom}} = 809 \text{ N/m}$, which was quite close to the self-weight of the specimen ($G_{\text{specimen}} = \rho \cdot (1-n_{2D}) \cdot W \cdot H \cdot g = 836 \text{ N/m}$). Moreover, the contact force distribution was similar to the realistic contact force distribution under gravity. Therefore, the selected 2D porosity of $n_{2D} = 0.15$ was regarded reasonable for conducting further DEM simulations in this case. The corresponding number of the soil particles was 2552 ($n_{2D} = 0.15$).
To sum up, large contact forces existed in the specimen without any normal stresses on the top of the specimen in Figure 3.17 with the 2D porosity of $n_{2D} = 0.14$. In order to reduce the large contact forces without changing the target 2D porosity, the unique way is to decrease the contact stiffnesses of the particles and the walls since the friction coefficient of the particles for the specimen preparation was already very low according to Härtl and Ooi (2008). However, it is a general rule that the macro initial Young’s modulus of the material is linearly related to the micro contact stiffnesses of the particles and walls (Itasca, 2008), i.e. the contact stiffnesses should be adjusted to accord with the experimental data after the specimen generation. Therefore, the large contact forces in the specimen at the initial stage might be caused by the unreasonable 2D porosity. The parabolic equation provides a general relation between 2D and 3D porosities, which can be used as a rough guideline to get an initial 2D porosity. Then the initial 2D porosity was adjusted to meet the requirements of the criteria in the iterative process.

Figure 3.19 shows a realistic contact force distribution in the specimen, in which the contact forces at the lower parts are larger than those at the upper parts. It represents that the 2D porosity obtained based on the suggestion in this study is reasonable. After fixing the 2D porosity, further DEM simulations of numerical direct shear tests are carried out, in which the micro input parameters (i.e. contact stiffnesses and friction coefficients of the particles and walls) are calibrated by comparing the DEM simulation results with the corresponding experimental data.

(2) Calibration results of direct shear tests for sand

The numerical simulations were conducted at three different confining stresses: 50, 100 and 200 kPa, which were identical to those in the laboratory tests. The micro input parameters of the DEM investigations are listed in Table 3.1 and the calibration results of the direct shear tests for sand are shown in Figure 3.20. Although the up-scaling factor influences the numerical results (Achmus and Abdel-Rahman, 2002), the up-scaling factor for sand is constant throughout the study and therefore the numerical results, which have been calibrated with the experimental data, are believed to be reliable.
3.3.3.2 Gravel

In the laboratory tests, dry granular soil with high degree of sphericity has been used. The shapes of most grains were subrounded according to the classification of coarse-grained soils (Holtz and Kovacs, 1981). The sizes of the gravel, ranging from 2 to 8 mm, were strictly controlled by sieving the material according to the particle size distribution, as shown in Figure 3.21. The specimen was prepared with a compaction degree of 95% proctor density ($\rho_{pr} = 1.644$ g/cm$^3$) so that the testing density of 1.56 g/cm$^3$ could be achieved and the corresponding relative density of the soil sample was $D_r = 0.85$.

The granular soil in the numerical direct shear tests was modeled as circular particles. Again, large friction coefficients for particles and walls were used to compensate the lack of angularity for circular particles. In contrast to using the up-scaling technique for sand, the particle size
distribution of gravel in the DEM investigations was identical to that in the laboratory tests, as shown in Figure 3.21. According to the suggestion for determining 2D porosities in the above section, a 2D porosity of \( n_{2D} = 0.17 \) (the corresponding laboratory testing porosity was \( n_{\text{lab}} = 0.41 \)) was used in the numerical direct shear tests of gravel and the number of particles was 2390. The specimen preparation process for the numerical direct shear tests of gravel were similar to that of sand.

The numerical direct shear tests of gravel were conducted under four different confining stresses (25, 50, 100 and 200 kPa) with the micro input parameters listed in Table 3.2. Figure 3.22 compares the DEM simulation results with the corresponding experimental data. Despite the differences of the shear stress–displacement relations mainly in the post-peak phase between the laboratory tests and the DEM simulations, the internal friction angle based on the DEM investigations (\( \phi_{\text{DEM}} = 40.6^\circ \)) is approximately equal to the value obtained in the laboratory tests (\( \phi_{\text{Lab}} = 40.9^\circ \)). Thus, it can be concluded that the model and the corresponding input parameters in the DEM simulations can satisfactorily represent the gravel properties.

The models and input parameters, which lead to good calibration results for sand and gravel, are used for further DEM investigations of numerical compound tensile tests (Chapter 4) and numerical pullout tests (Chapter 5), respectively.
Table 3.2  Input parameters of gravel in numerical direct shear tests.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density of the solids $\rho_s$ [kg/m³]</td>
<td>2650</td>
</tr>
<tr>
<td>Water content $w$ [%]</td>
<td>0</td>
</tr>
<tr>
<td>Particle diameters $d_g$ [mm]</td>
<td>Gradation as in Figure 3.21</td>
</tr>
<tr>
<td>Normal contact stiffness of the walls $k_{n,w}$ [N/m]</td>
<td>$1 \times 10^7$</td>
</tr>
<tr>
<td>Shear contact stiffness of the walls $k_{s,w}$ [N/m]</td>
<td>$1 \times 10^7$</td>
</tr>
<tr>
<td>Normal contact stiffness of the particles $k_{n,g}$ [N/m]</td>
<td>$6 \times 10^5$</td>
</tr>
<tr>
<td>Shear contact stiffness of the particles $k_{s,g}$ [N/m]</td>
<td>$6 \times 10^5$</td>
</tr>
<tr>
<td>Friction coefficient of the particles for specimen preparation $f_0$ [-]</td>
<td>0.05</td>
</tr>
<tr>
<td>Friction coefficient of the particles after specimen preparation $f_g$ [-]</td>
<td>5</td>
</tr>
<tr>
<td>Friction coefficient between the particles and the walls $f_{wg}$ [-]</td>
<td>2.5</td>
</tr>
</tbody>
</table>

![Shear stress–displacement relations.](image1)

(a) Shear stress–displacement relations.

![Peak strength–normal stress relations.](image2)

(b) Peak strength–normal stress relations.

Figure 3.22 Calibration results of the direct shear tests of gravel.
3.4 Model Development and Calibration for Geogrid

With the parallel bond model provided by Itasca (2008), it is easily possible to represent the tensile strength properties of a geogrid with a linear force–strain relationship (e.g. Zhang et al., 2008; Han et al., 2012; Ngo et al., 2014). However, the tensile test results of the used geogrid showed a nonlinear relationship between the tensile force and strain. Therefore, in order to illustrate the real nonlinear tensile properties of the geogrid in this study, a piecewise linear model was developed based on the parallel bond model. With the piecewise linear model, numerical tensile tests were carried out to calibrate the micro input parameters for only one geogrid tensile member and for a normal geogrid.

3.4.1 Numerical Tensile Test of One Geogrid Tensile Member

The piecewise linear model was firstly developed to represent the nonlinear tensile strength behavior of only one geogrid tensile member in the numerical tensile test. Since the particle sizes in the numerical direct shear tests of sand were 10 times of the real sizes, the thickness of the geogrid particles was also increased into 10 times of the real thickness of the geogrid tensile member (1 mm). Hence, the ratio of geogrid thickness to sand particle size was identical to that in the laboratory tests. The width of the numerical geogrid tensile member was 8 mm, which was identical to that in the laboratory tests. In the numerical tensile tests, the geogrid tensile member was modeled as bonded particles with a length of 200 mm as in the laboratory test and 20 particles were generated in one row without any overlaps between the particles, as shown in Figure 3.23. The tensile rate was 1 mm/min.

![Figure 3.23](image)

Figure 3.23 Numerical tensile test of one geogrid tensile member.
The piecewise linear model is expressed by the following equation

\[
F = \sum_{j=1}^{N} \left( a \cdot \bar{k}_{i,j}^{n} \cdot \varepsilon_{j} \right), \quad \bar{k}_{i,j}^{n} = \begin{cases} 
\bar{k}_{1,j}^{n} & 0 \leq \varepsilon_{j} < \varepsilon_{1} \\
\bar{k}_{2,j}^{n} & \varepsilon_{1} \leq \varepsilon_{j} < \varepsilon_{2} \\
\bar{k}_{3,j}^{n} & \varepsilon_{2} \leq \varepsilon_{j} < \varepsilon_{3} \\
\bar{k}_{4,j}^{n} & \varepsilon_{3} \leq \varepsilon_{j} < \varepsilon_{4} \\
\bar{k}_{5,j}^{n} & \varepsilon_{j} \geq \varepsilon_{4} 
\end{cases} \quad (3.42)
\]

where \( F \) is the total tensile force measured at the clamp;

\( N \) is the number of the parallel bond contacts in the geogrid tensile member;

\( a \) is a coefficient relates to the overlap and the relative angular velocity between any two neighboring particles;

\( \bar{k}_{i,j}^{n} \) is the parallel bond normal stiffness at each parallel bond contact;

\( \varepsilon_{j} \) is the strain at any two bonded particles in the geogrid tensile member;

\( \varepsilon_{1}, \varepsilon_{2}, \varepsilon_{3}, \varepsilon_{4} \) are the corresponding critical strains in this study.

The main FISH codes of the developed piecewise linear model are listed in Appendix A. The micro parameters, especially the parallel bond normal contact stiffnesses, have been varied iteratively until the simulation results matched the laboratory test results, as shown in Figure 3.24. The laboratory tensile test was carried out by Schluroff (2012) at RWTH Aachen University with a tensile rate of 1 mm/min. The final micro parameters for the conducted numerical tensile test are listed in Table 3.3.

![Figure 3.24 Calibration results of the tensile test of one geogrid tensile member.](image)
Table 3.3  Micro parameters of one geogrid tensile member in the numerical tensile test.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle diameters $d_{gm}$ [mm]</td>
<td>10</td>
</tr>
<tr>
<td>Parallel bond radius multiplier $\lambda$ [-]</td>
<td>0.005</td>
</tr>
<tr>
<td>Parallel bond normal stiffness $\bar{k}_{ngm,i}$ [$\times 10^{12}$ Pa/m]</td>
<td>9.85, 8.56, 4.28, 3.3, 3.16</td>
</tr>
<tr>
<td>Parallel bond shear stiffness $\bar{k}_{sgm}$ [$\times 10^{12}$ Pa/m]</td>
<td>9.85</td>
</tr>
<tr>
<td>Parallel bond normal strength $\bar{\sigma}_{ngm}$ [$\times 10^{10}$ Pa]</td>
<td>4</td>
</tr>
<tr>
<td>Parallel bond shear strength $\bar{\tau}_{ngm}$ [$\times 10^{10}$ Pa]</td>
<td>4</td>
</tr>
<tr>
<td>Contact normal stiffness $k_{ngm}$ [N/m]</td>
<td>$4 \times 10^5$</td>
</tr>
<tr>
<td>Contact shear stiffness $k_{sgm}$ [N/m]</td>
<td>$4 \times 10^5$</td>
</tr>
</tbody>
</table>

The calibration results of the numerical tensile test of the geogrid tensile member show good agreement with the experimental data, which demonstrates that the developed model and the corresponding micro parameters in the DEM simulations can satisfactorily represent the properties of the geogrid tensile member. In the numerical tensile tests, the geogrid tensile member was stretched evenly along its length in air and therefore, the parallel bond normal stiffness $k_{nij}$ at any parts of the geogrid tensile member are identical. However, when the geogrid tensile member interacts with soil, different parts of the geogrid tensile member may have different strains. With the piecewise linear model in this study, such behavior can also be easily described since $k_{nij}$ is changed automatically according to Equation 3.42 with increasing $\varepsilon_j$ at any parts of the geogrid tensile member.

3.4.2  Numerical Tensile Test of Normal Geogrid

In the numerical tensile test of a normal geogrid, the real 3D biaxial geogrid SG-30 was simplified into a 2D model, as shown in Figure 3.25. The number in the product name, i.e. 30 for SG-30 indicates the tensile strength (unit: kN/m) in machine and cross machine directions as a 95% confidence value. The geogrid SG-30 was made of polypropylene (PP) with rigid thermally welded junctions. The aperture size of the geogrid is approximately 32 mm $\times$ 32 mm and its thickness is around 1 mm with a tensile stiffness of $J_{0.2\%} = 700$ kN/m. The geogrid longitudinal member was simulated with one row of bonded particles and the knots were simulated with five rows of bonded particles on the longitudinal member and four rows of bonded particles beneath the longitudinal member (see Figure 3.25c). High knots were used in the 2D DEM investigations so that similar bearing resistance caused by the transverse members as in the laboratory tests could be simulated when the 2D geogrid model interacted with soil. The bearing resistance was then transferred to the longitudinal member via the knot particles. The influence of knot height on the geogrid tensile behavior is discussed in the later part of this section.
In the DEM studies, the thickness of the longitudinal member was 1 mm and the width of the knot was 8 mm (see Figure 3.25c), which were identical to those in the laboratory tests. The piecewise linear model developed in Section 3.4.1 based on the parallel bond model was used to simulate the nonlinear tensile behavior of the geogrid. The tensile rate in the numerical investigations was 20 mm/min, which was quite similar to that in the German standard (DIN EN ISO 10319) with a tensile rate of 20 %/min. The micro parameters for the geogrid have been varied iteratively until the DEM simulation results ($v = 20$ mm/min and $H_{\text{knot}} = 9$ mm) matched the experimental data, as shown in Figure 3.26. The experimental tensile test result was provided by the geogrid manufacturer. The final input parameters of the normal geogrid in the numerical tensile test are listed in Table 3.4.

Figure 3.25 Simplification of the tensile test of normal geogrid ($H_{\text{knot}} = 9$ mm).

Figure 3.26 Calibration results of tensile tests of normal geogrids.
Table 3.4  Micro parameters of normal geogrid in numerical tensile tests.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle diameters $d_{gg}$ [mm]</td>
<td>1</td>
</tr>
<tr>
<td>Parallel bond radius multiplier $\lambda$ [-]</td>
<td>0.0005</td>
</tr>
<tr>
<td>Parallel bond normal stiffness $k_{n\text{gg},i}$ [$\times10^{14}$ Pa/m]</td>
<td>76, 47, 38, 33, 0</td>
</tr>
<tr>
<td>Parallel bond shear stiffness $k_{s\text{gg},i}$ [$\times10^{14}$ Pa/m]</td>
<td>20, 0</td>
</tr>
<tr>
<td>Parallel bond normal strength $\sigma_{\text{gg}}$ [$\times10^{14}$ Pa]</td>
<td>4</td>
</tr>
<tr>
<td>Parallel bond shear strength $\tau_{\text{gg}}$ [$\times10^{14}$ Pa]</td>
<td>4</td>
</tr>
<tr>
<td>Contact normal stiffness $k_{n\text{gg}}$ [N/m]</td>
<td>$4\times10^5$</td>
</tr>
<tr>
<td>Contact shear stiffness $k_{s\text{gg}}$ [N/m]</td>
<td>$4\times10^5$</td>
</tr>
</tbody>
</table>

Good agreement of the numerical simulation results with the corresponding experimental data in Figure 3.26 demonstrates that the piecewise linear model with the chosen micro parameters in the DEM investigations can satisfactorily represent the geogrid properties in the laboratory tests. Since the pullout rate in further pullout tests is 1 mm/min, the numerical tensile test of the geogrid was also conducted with a tensile rate of 1 mm/min. Identical micro parameters listed in Table 3.4 have been used in the numerical tensile test. The DEM simulation results accord well with the experimental data, as shown in Figure 3.26. The tensile force–strain curves under different tensile rates (i.e. 20 mm/min and 1 mm/min) in the DEM simulations almost coincide with each other. Hence, the effect of tensile rates on the geogrid tensile behavior is neglected in the DEM investigations. Such observations agree with the experimental results of tensile tests conducted by the manufacturer, i.e. tensile tests with displacement rates from 20 mm/min down to 2 mm/min have not shown obvious rate effects since the investigated geogrid products were manufactured by highly prestressed tensile members. For punched and drawn PP geogrids, however, the influence of tensile rates on the load–strain behavior cannot be neglected, as reported by Ezzein et al. (2015) based on the laboratory tests.

In order to investigate the influence of knot heights on geogrid tensile behavior, another numerical tensile test was carried out based on the 2D geogrid with a knot height of 3 mm, i.e. two rows of bonded particles on the longitudinal member and one row of bonded particles beneath the longitudinal member. As a comparison, the numerical tensile test of only one geogrid longitudinal member without knots was also conducted in this section. The same micro parameters listed in Table 3.4 were used for the above mentioned two geogrid samples and the tensile rate was 1 mm/min. It can be seen from Figures 3.26 that the 2D geogrids with different knot heights (i.e. 9 mm and 3 mm) show almost the same tensile behavior in air and the tensile forces of geogrids with knots are slightly larger than those of the geogrid longitudinal member without knots. Such results can be explained by the special structures of 2D geogrids used in this study. Each knot of the 2D geogrids was simulated with a mass of bonded particles, which leads to high tensile stiffness of the knots. Hence, for the 2D geogrid with knots (either $H_{\text{knot}} = 9$ mm or $H_{\text{knot}} = 3$ mm), the tensile behavior of such geogrid is mainly determined by the longitudinal member. Moreover, such high tensile stiffness of knots also result in the difference of tensile behavior between geogrids with knots and the geogrid longitudinal member without knots.
In the numerical large-scale biaxial compression tests (see Chapter 6), the sizes of the granular soil were increased with an up-scaling factor of two so that the DEM investigation results could be obtained within reasonable computational time. In order to maintain the ratio of geogrid aperture size to soil particle size identical to that in the laboratory tests, the length and thickness of the 2D geogrid were also increased into twice of the normal 2D geogrid, which had the knot height of \( H_{\text{knot}} = 9 \) mm. Thereby, the length, thickness and knot height of the up-scaled 2D geogrid were 200 mm, 2 mm and 18 mm, respectively. With the micro input parameters listed in Table 3.5, good agreement of the DEM simulations results with the corresponding experimental data can be found Figure 3.26. The tensile rate was 1 mm/min. The particle diameter of the up-scaled geogrid was twice of the normal 2D geogrid, while the parallel bond radius multiplier was identical to that of the normal 2D geogrid. Other micro input parameters of the up-scaled geogrid listed in Table 3.5 were 1/4 of the corresponding micro parameters of the normal 2D geogrid.

Table 3.5  Micro parameters of up-scaled geogrid in the numerical tensile test.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle diameters ( d_{\text{gg}} ) [mm]</td>
<td>2</td>
</tr>
<tr>
<td>Parallel bond radius multiplier ( \lambda [-]</td>
<td>0.0005</td>
</tr>
<tr>
<td>Parallel bond normal stiffness ( \bar{k}_{\text{ngg,i}} \times 10^{14} \text{ Pa/m} )</td>
<td>19, 11.75, 9.5, 8.25, 0</td>
</tr>
<tr>
<td>Parallel bond shear stiffness ( \bar{k}_{\text{sgg,i}} \times 10^{14} \text{ Pa/m} )</td>
<td>5, 0</td>
</tr>
<tr>
<td>Parallel bond normal strength ( \bar{\sigma}_{\text{cgg}} \times 10^{14} \text{ Pa} )</td>
<td>1</td>
</tr>
<tr>
<td>Parallel bond shear strength ( \bar{\tau}_{\text{cgg}} \times 10^{14} \text{ Pa} )</td>
<td>1</td>
</tr>
<tr>
<td>Contact normal stiffness ( k_{\text{ngg}} ) [N/m]</td>
<td>1x10^5</td>
</tr>
<tr>
<td>Contact shear stiffness ( k_{\text{sgg}} ) [N/m]</td>
<td>1x10^5</td>
</tr>
</tbody>
</table>

3.5 Summary

In this chapter, the basic concept and knowledge of PFC$^{2D}$ has been described as well as the numerical modeling and calibration for soil and geogrid samples based on numerical direct shear tests and numerical tensile tests, respectively.

Current approaches for converting porosities from 3D to 2D for DEM studies have been summarized and theoretically evaluated. All the current approaches tried to link the 2D and 3D porosities just using specific equations, which might be impossible for arbitrary assemblies with various polydisperse particle systems. Therefore, a new suggestion consisting of a parabolic equation and an iterative criterion has been proposed for determining 2D porosities in DEM studies. The new suggestion was proven to be rational with the realistic contact force distribution in the specimen and the good agreement of further DEM simulation results of the numerical direct shear tests with the corresponding experimental data. Moreover, the contact models and the micro input parameters for sand and gravel have been calibrated for further DEM investigations (i.e. numerical compound tensile tests and numerical pullout tests).
In the numerical tensile tests, a piecewise linear model has been developed based on the parallel bond model provided by PFC$^2$D. The piecewise linear model and the corresponding micro input parameters for only one geogrid tensile member and for normal geogrid samples have been calibrated for further DEM studies. Good calibration results demonstrate that the piecewise linear model could satisfactorily represent the nonlinear tensile strength behavior of the geogrid tensile member and the normal geogrid samples in this study.
4 Compound Tensile Tests

4.1 Introduction

As it is well known, a geogrid is composed of longitudinal and transverse tensile members. The geogrid interacts with its surrounding soil through two mechanisms: (1) frictional resistance between geogrid surface and soil particles; (2) bearing resistance caused by the geogrid transverse members when a relative displacement between geogrid and soil occurs. In order to investigate the first mechanism, i.e. the frictional behavior between geogrid and sand, a simple test with only one geogrid tensile member without transverse members is conducted in this chapter. Due to the small surface area of the one geogrid tensile member, it could be easily pullout of the sand box under usual pullout boundary conditions. Therefore, in contrast to usual pullout tests, the far end of the one geogrid tensile member is fixed and the test is regarded as a compound tensile test conducted in sand.

Based on the experimental compound tensile tests, the corresponding DEM investigations have been carried out in this study. The models and the corresponding micro input parameters for the geogrid tensile member and sand have been described and calibrated in Chapter 3. In this chapter, the force distributions along the geogrid tensile member and in the specimen, the displacement distributions along the geogrid tensile member and the rotations of soil particles in the vicinity of the geogrid tensile member are investigated at different clamp displacements.

4.2 Experimental Compound Tensile Test

The experimental compound tensile test of one geogrid tensile member embedded in sand was carried out at RWTH Aachen University (Schluroff, 2012). Figure 4.1 shows the plan view of the laboratory compound tensile test, in which the inside dimensions of the compound tensile box were $L/W/H = 435/300/200$ (unit: mm). The experimental parameters of the geogrid tensile member and sand are listed in Table 4.1. More information about the properties of the geogrid tensile member and sand has been illustrated in Chapter 3. In this test, a constant confining stress of 100 kPa was applied on the top of the specimen. The far end of the geogrid tensile member (left) was fixed and the geogrid tensile member was stretched via the clamp at the front end (right) with a fixed tensile rate of 1 mm/min, as shown in Figure 4.1. During the tensile process, the tensile force and the clamp displacement were recorded automatically. Moreover, two strain gauges were attached on the geogrid tensile member so that the strains at the corresponding positions could be obtained. The resulting force measured at the clamp over the average strain of the geogrid tensile member is shown in Figure 4.2 together with the DEM calibration results in Section 4.3. It should be noted that the peak tensile strength of the geogrid tensile member was
not achieved in the laboratory test due to the failure of the strain gauges. Detailed illustration about the experimental compound tensile test can be found in Schluroff (2012).

![Plan view of the laboratory compound tensile test](image)

**Figure 4.1** Plan view of the laboratory compound tensile test.

<table>
<thead>
<tr>
<th>Geogrid tensile member</th>
<th>Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width $W_{gm}$ [mm]</td>
<td>Height $H_{gm}$ [mm]</td>
</tr>
<tr>
<td>Length $L_{gm}$ [mm]</td>
<td>Dry density $\rho_d$ [g/cm$^3$]</td>
</tr>
<tr>
<td></td>
<td>Water content $w$ [%]</td>
</tr>
<tr>
<td></td>
<td>Internal friction angle $\phi_{S\text{and}}$ [°]</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
</tr>
</tbody>
</table>

**Table 4.1** Experimental parameters of geogrid tensile member and sand.

![Tensile force vs. average strain of geogrid tensile member in experimental and DEM investigations](image)

**Figure 4.2** Tensile force vs. average strain of geogrid tensile member in experimental and DEM investigations.
4.3 Numerical Investigation with DEM

4.3.1 Numerical Sample Preparation and Calibration Results

Figure 4.3 shows the side view of the numerical compound tensile test with DEM. The length and height of the numerical compound tensile box were 435 mm and 200 mm, respectively, which were identical to those in the physical tests. A geogrid tensile member with a length of 490 mm was used and 49 circular particles were generated in one row without any overlaps between the particles. During the tensile process, a constant confining stress of 100 kPa was applied on the top of the specimen and a fixed tensile rate of 1 mm/min was applied on the clamp.

![Side view of the numerical compound tensile test.](image)

In the numerical investigations, the linear contact stiffness model, the slip model and the piecewise linear model have been used to characterize the properties of the sand and geogrid, respectively. The input parameters of the sand and the geogrid tensile member were identical to those in the numerical direct shear tests and in the numerical tensile test, as well as the up-scaling factor for the particle sizes, which can be found in Chapter 3. The 2D porosity of the numerical specimen was 0.15 and the number of particles was 5872. For the preparation of the numerical specimen, similar processes as in the numerical direct shear tests were conducted for the first two layers. Then the geogrid tensile member was generated on the top of the first two layers. After that, the third and the fourth layers of soil particles were generated again similar to the numerical direct shear tests. After iteratively adjusting the friction coefficient of the geogrid tensile member particles $f_{gm}$, the final simulation results of the compound tensile test with $f_{gm} = 5$ are compared with the experimental results in Figure 4.2. It should be noted that the friction coefficient of the geogrid tensile member particles was much larger than that of actual geogrid tensile members; this is to ensure the simulation results fit the laboratory results. Large friction coefficients of reinforcement have also been used for DEM simulations by Lin et al. (2013) and Zhang et al. (2013). Although the curve of the numerical force–strain relation fluctuates to a small degree when the strain of the geogrid tensile member is larger than 2.5 %, the calibration results still demonstrate that the numerical compound tensile test can represent the laboratory test, and therefore, the calibration results are used for further analyses.

Figure 4.4 summarizes the tensile force–strain relations of the numerical compound tensile test (in soil) and the tensile test (in air). In both tests, the tensile forces increase with increasing average
strains of the geogrid tensile member. Due to the frictional resistance between the geogrid tensile member and soil, the tensile forces at clamp in the compound tensile test are larger than those in the tensile test. The maximum tensile forces at the clamps in both tests are supposed to be the same since the peak tensile strength is the inherent property of the geogrid tensile member, as shown in Figure 4.4.

**Figure 4.4** Tensile force–strain relations of the numerical compound tensile test and the tensile test.

### 4.3.2 Force Distributions along Geogrid Tensile Member

Due to the low tensile rate at the clamp during the tensile test (in air), each state of the tensile process can be regarded as a static equilibrium state. Therefore, the tensile forces at the clamp and the fixed end are exactly the same for all states. In the compound tensile test (in sand), however, the induced force at the clamp cannot be totally transferred to the fixed end due to the frictional resistance between the geogrid tensile member and sand.

Figure 4.5 shows the parallel bond force distributions along the geogrid tensile member at different clamp displacements. Obviously, the bond forces along the geogrid tensile member increase with increasing clamp displacement. The bond forces at the clamp are always larger than those at the fixed end and the bond forces decrease gradually from the clamp to the fixed end. The small fluctuations of bond forces between neighboring particles of the geogrid tensile member do not influence the global trend. Similar force distributions along the geogrid positions have been reported by Ziegler and Timmers (2004), Sieira et al. (2009), Tran et al. (2013) and Jacobs et al. (2014).
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4.3 Numerical Investigation with DEM

The parallel bond forces along the geogrid tensile member at different clamp displacements are illustrated together with the contact forces in the specimen in Figure 4.6. The parallel bond forces between the particles of the geogrid tensile member are represented by the red lines, while the contact forces in the specimen are represented by the black lines. The thickness of the lines is proportional to the reported magnitude. Moreover, the bond forces along the geogrid tensile member are also quantitatively illustrated by the curves in Figure 4.6. The bond forces and the contact forces have varied during the tensile process which visualizes the load transfer behavior between the geogrid tensile member and sand.

At the beginning of the test, the clamp displacement $u_{\text{Clamp}}$ is 0 mm (Figure 4.6a) and the bond forces between the particles of the geogrid tensile member are quite small caused only by the gravitational load of the upper half specimen and the normal force $\sigma_v$ on the top of the specimen. As illustrated above, the bond forces along the geogrid tensile member increase with increasing clamp displacement and the bond forces at the clamp are always larger than those at the fixed end, as shown in Figure 4.6b–d.

Moreover, the distributions and the orientations of the contact forces in the specimen have changed with increasing clamp displacement, as shown in Figure 4.6. When the clamp displacement $u_{\text{Clamp}}$ is 0 mm (Figure 4.6a), the contact forces are relatively small and distributed almost all over the specimen. The orientations of the contact forces are nearly vertical despite some irregularities caused by the random packing of the soil particles. During the tensile process, the load is

\[ F_{\text{Bond}}(x) \]

\[ F_{\text{Contact}}(x) \]
transferred from the geogrid tensile member to the soil by frictional resistance, which causes the changes of the contact force distributions and orientations in the specimen. The contact forces gradually concentrate more and more in the right part of the specimen, since the geogrid tensile member is being pulled towards the right direction, and accordingly, the orientations of the contact forces change from nearly vertical to diagonal. These observations agree well with the research results conducted by Dyer (1985) and Tran et al. (2013).
4.3 Numerical Investigation with DEM

4.3.3 Displacement and Strain Distributions along Geogrid Tensile Member

Figure 4.7 shows the numerically obtained displacement distributions of the geogrid tensile member along its positions at different clamp displacements. It can be seen that the displacements of the geogrid tensile member increase with increasing clamp displacement. Smoothly decreased displacement distributions along the geogrid tensile member are observed from the loaded clamp to the fixed end. The elongation of the geogrid tensile member, which is caused by the tensile force within the sample, is not uniform along the tensile member. Hence, the displacements of the geogrid tensile member do not linearly distribute along the geogrid tensile member during the tensile process. The DEM simulations results have been confirmed by similar studies such as Sugimoto and Alagiyawanna (2003), Ziegler and Timmers (2004), Moraci and Recalcati (2006), Teixeira et al. (2007), Palmeira (2009), Tran et al. (2013) and Jacobs et al. (2014).

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Based on the displacement distributions along the geogrid tensile member in Figure 4.7, the corresponding strain distributions along the geogrid tensile member at different clamp displacements are shown in Figure 4.8. The shapes of the strain distribution curves in Figure 4.8 are quite similar to those of the tensile force distribution curves in Figure 4.5, which demonstrates that the large strains along the geogrid tensile member were caused by the corresponding large tensile forces.

Moreover, the tensile strains obtained with the strain gauges from the laboratory test at the average geogrid tensile member strains of $\varepsilon_{gm} = 1\%$ and $\varepsilon_{gm} = 2\%$ are also marked with triangular and square solid points in Figure 4.8. The average strains of $\varepsilon_{gm} = 1\%$ and $\varepsilon_{gm} = 2\%$ in the laboratory test correspond to the clamp displacement of $u_{Clamp} = 5\mm$ and $u_{Clamp} = 10\mm$ in the DEM investigations, respectively. Despite the small differences of strains between the DEM simulation results and the corresponding experimental data, it can still be concluded that the DEM investigations show reasonable tensile strain distributions compared with the strains obtained with the strain gauges from the laboratory test. Therefore, the DEM simulation results are considered to be reliable.
4.3.4 Rotations of Soil Particles

In this study, the rotations of the soil particles within the specimen, especially in the vicinity of the geogrid tensile member, have been recorded to visualize the load transfer behavior between the geogrid tensile member and soil additionally in another way. Figure 4.9 shows the soil particle rotations with different colors at different clamp displacements. When the clamp displacement is 0 mm (Figure 4.9a), the colors of all soil particles are set to yellow, which represents the initial states of the particles without any rotations. Once the soil particles rotate clockwise or counterclockwise to different degrees, the colors of the soil particles will be changed automatically from yellow to the corresponding colors (Figure 4.9b–d). As expected, the rotations of the soil particles in the vicinity of the geogrid tensile member increase gradually with increasing clamp displacement. The soil particles above the geogrid tensile member mainly rotate counterclockwise, while the soil particles below the geogrid tensile member mainly rotate clockwise, as shown in the enlarged partial view of Figure 4.9c. The area that is activated during the tensile process increases gradually as more and more load is transferred from the geogrid tensile member to the soil. In order to keep the normal pressure constant, the horizontal plate loaded on the top of the specimen might move upwards or downwards, which provides more chances for the soil particles in the upper half specimen to rotate. It should also be noted that due to the dragging effects caused by the geogrid tensile member on the soil particles, more soil
particles have accumulated in the right part of the specimen, which provides more spaces for the soil particles in the left part of the specimen to rotate, as shown in Figure 4.9d.

![Figure 4.9 Soil particle rotations in the compound tensile box.](image)

### 4.4 Limitations

The real 3D problem was simplified into a 2D investigation using 2D software (PFC\textsuperscript{2D}). Since the 2D plane system can only dilate in one direction, the dilation behavior of 2D models is different from that of 3D models (Rothenburg and Bathurst, 1992). Moreover, instead of the real geometry of the geogrid tensile member, a string of bonded particles without any overlaps has been used in the DEM simulations. As a result, the contact areas between the geogrid tensile member and soil in the DEM simulations are different from that in the laboratory tests. In order to obtain the fundamental interface behavior between the geogrid tensile member and soil in a reasonable computation time, the technique of “up-scaling” has been used and all the soil particles have been modeled with circular discs without considering the irregular shapes of real materials. All the above simplifications in this study will surely lead to a difference between the DEM simulations and the laboratory testing; however, the numerical results in this study are still helpful to understand the fundamental load transfer behavior between the geogrid tensile member and soil.

### 4.5 Summary

In this chapter, the frictional interaction between only one geogrid tensile member and soil has been investigated with a numerical compound tensile test, in which the models and the input parameters of the geogrid tensile member and soil were used from the tensile tests and the direct shear tests (described in Chapter 3), respectively. The piecewise linear model, which was
developed in Chapter 3 to characterize the nonlinear tensile strength behavior of geogrid products, has been successfully used in the numerical compound tensile test.

The load transfer behavior between the geogrid tensile member and soil has been visualized by the force, displacement and strain distributions along the geogrid tensile member, the contact force changes in the specimen and the rotations of soil particles in the vicinity of the geogrid tensile member at different clamp displacements.

The visualization results show that PFC$^{2D}$ can be used as a practical tool to investigate the interaction between the geogrid tensile member and soil. The DEM simulation results provide researchers more insights into the interface behavior between the geogrid tensile member and soil at a microscopic scale.
5 Pullout Tests

5.1 Introduction

The geogrid pullout test has been regarded as a simple and direct way to investigate the geogrid–soil interface behavior among all the experimental approaches. Hence, both experimental and numerical pullout tests of geogrids embedded in granular soil are carried out in this chapter. In order to investigate the effects of geogrid transverse members on the total pullout resistance, the tested geogrid specimens have been modified with different numbers of geogrid transverse members. In the laboratory tests, two different failure modes have been observed depending on different numbers of geogrid transverse members in this study.

The numerical pullout tests were conducted using PFC^2D and the micro input parameters for the geogrid and the granular soil have been calibrated in Chapter 3. In this chapter, the geogrid–soil interaction under pullout loads is described not only by the qualitative force distributions along the geogrid and in the specimen but also by the quantitative geogrid force, displacement and strain distributions along the geogrid with different numbers of geogrid transverse members. The numerically obtained contributions of the geogrid transverse members to the total pullout resistance can be used to explain the two different failure modes in the laboratory pullout tests. Based on the Fourier Series Approximation (FSA) method, the reorientations of contacts and forces in the specimen are presented at different clamp displacements. Moreover, the normal stress distributions in the geogrid plane, which is a decisive parameter that can only be evaluated indirectly in the experimental pullout tests, are obtained directly using the FSA method in the numerical modeling.

5.2 Laboratory Pullout Tests and Analyses

5.2.1 Testing Apparatus

Figure 5.1 shows a sketch of the pullout apparatus used in this study. The pullout box was made of steel with the inside dimensions of $L/W/H = 435/300/200$ (unit: mm). Different from the flexible surcharge loading systems recommended in the ASTM test standard (ASTM D6706-01, 2001; Huang and Bathurst, 2009), the vertical load was applied on top of the specimen through a rigid plate in this study. It should be noted that the vertical forces on both the top and the bottom plates were recorded with load cells in this study (as shown in Figure 5.1) and the mean value was regarded as the normal pressure in the geogrid plane. The normal pressure in the geogrid plane was maintained approximately constant during the test by adjusting the applied vertical load on the top plate. The clamp was made up of two pieces of steel plates that were bolted together.
order to minimize the relative displacement between geogrid and clamp during the pullout process, two layers of sandpapers were placed between the geogrid sample and the two steel plates, respectively. The clamp displacement rate was 1 mm/min in this study according to ASTM D6706-01 (2001), as this value has been commonly used in most pullout tests (Huang and Bathurst, 2009). In order to obtain the displacement distributions along the geogrid, flexible stainless steel wires (i.e. tell-tails) were used to connect the different geogrid positions and the LVDTs outsides of the pullout box, as shown in Figure 5.1. The pullout force and the clamp displacement are recorded during the whole pullout process as well as the geogrid displacements along the geogrid. In this study, the pullout failure was defined by the maximum pullout load.

![Figure 5.1 Sketch of the pullout test apparatus.](image)

### 5.2.2 Testing Materials

#### 5.2.2.1 Granular Soil

In this study, dry granular soil with high degree of sphericity has been used. The shapes of most grains were subrounded according to the classification of coarse-grained soils (Holtz and Kovacs, 1981). The sizes of the gravel, ranging from 2 to 8 mm, were strictly controlled by sieving the material according to the particle size distribution, as shown in Figure 3.17 in Chapter 3. The testing density of the soil sample was 1.56 g/cm$^3$ with a relative density of $D_r = 0.85$. The internal friction angle of the soil sample was $\phi_{Lab} = 40.9^\circ$ based on large-scale direct shear tests ($W/D/H = 305/305/124$, unit: mm).
5.2.2.2 Geogrid

In order to investigate the geogrid frictional and bearing resistance separately, four types of modified geogrids (S0 – geogrid without transverse members; S1 – geogrid with only one transverse member in the middle; S3 – geogrid with three transverse members; SV – geogrid with regular transverse members) were used in this study, as shown in Figure 5.2. The first three types of geogrids (S0, S1 and S3) were prepared by carefully removing the corresponding transverse members from normal welded biaxial geogrid products, which were manufactured by flat prestressed tensile members with rigid thermally welded junctions. Moreover, in order to investigate the influence of geogrid tensile stiffness on the pullout behavior, two types of welded biaxial geogrid products with the tensile stiffness of \( J_{0.2\%} = 700 \text{kN/m} \) (SG-30) and \( J_{0.2\%} = 1350 \text{kN/m} \) (SG-60) have been used. The geogrid tensile stiffness was calculated by the average tensile strength at the geogrid strain of 2\%. The mentioned number in the product name, e.g. 30 for SG-30 indicates the tensile strength (unit: kN/m) in machine and cross machine directions as a 95\% confidence value. Both biaxial geogrids were made of polypropylene (PP) with an approximate aperture size of 32 mm \( \times \) 32 mm and a thickness of 1 mm.

(a) S0 – geogrid without transverse members.  
(b) S1 – geogrid with one transverse member.  
(c) S3 – geogrid with three transverse members.  
(d) SV – geogrid with regular transverse members.

Figure 5.2 Four types of geogrid samples used in this study (dots on the geogrid samples represent the positions for the displacement measurements).

5.2.3 Testing Procedure and Program

The soil specimen was prepared in the pullout box with four horizontal layers and each layer was controlled to reach the target density of 1.56 g/cm\(^3\). After preparing the first two layers of soil, the geogrid was laid on the bottom half of the soil specimen and the front end of the geogrid was firmly connected to the loading device by the clamp. Flexible stainless steel wires were connected to the geogrid at different positions (as shown in Figure 5.1) to record the displacement distribution.
along the geogrid. The last two layers of soil were then placed on the geogrid, followed by the top plate on the specimen. Then the vertical pressure was applied on the rigid top plate. During the loading process, the vertical forces on both the top and the bottom plates were recorded with load cells (as shown in Figure 5.1) and the mean value was regarded as the normal pressure in the geogrid plane. The normal pressure in the geogrid plane was maintained approximately constant during the test by adjusting the applied vertical load on the top plate. The geogrid was pulled out of the box at the front end with a constant rate of 1 mm/min until one of the following conditions was first reached: (1) failure of the geogrid longitudinal tensile members; (2) the maximum clamp displacement of 80 mm.

A detailed testing program is listed in Table 5.1. The contributions of different numbers of geogrid transverse members on the total pullout resistance were investigated in this study as well as the influence of the geogrid tensile stiffness \( J_{0.2\%} \) on the pullout behavior.

<table>
<thead>
<tr>
<th>Geogrid products</th>
<th>Geogrid types</th>
<th>Normal pressure ( \sigma_N ) [kPa]</th>
<th>Number of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>SG-30 ((J_{0.2%} = 700 \text{ kN/m}))</td>
<td>S0, S1, S3, SV</td>
<td>20, 50, 100</td>
<td>12</td>
</tr>
<tr>
<td>SG-60 ((J_{0.2%} = 1350 \text{ kN/m}))</td>
<td>S0, S1, S3, SV</td>
<td>50</td>
<td>4</td>
</tr>
</tbody>
</table>

5.2.4 Laboratory Test Results and Analyses

5.2.4.1 Pullout Resistance

Figure 5.3 shows the pullout force against the clamp displacement of the geogrids SG-30 and SG-60 for various types of geogrid samples in the laboratory tests. For normal geogrids with transverse members, the total pullout force consists of frictional resistance caused by the longitudinal members and bearing resistance caused by the transverse members. The S0-tests in Figure 5.3 present the frictional behavior between the longitudinal members and soil. It can be seen that the pullout forces caused only by the frictional resistance of longitudinal members in the S0-tests are quite small compared with those tests with additional bearing resistance of transverse members in the S1, S3 and SV-tests. As expected, with increasing number of transverse members, the total pullout forces increase. The contributions of different numbers of transverse members on the pullout resistance of the geogrid SG-30 are similar to those of the geogrid SG-60, correspondingly. In the S1 and S3-tests, sudden drops of the pullout forces are caused by the failure of the welded junctions between longitudinal and transverse members. According to the studies by Ziegler and Timmers (2004) on welded geogrids, the mobilized soil area, which causes bearing resistance in front of geogrid transverse members, increases gradually with increasing clamp displacement. In the S1-test, a very large soil area can be activated in front of the single transverse member. Similar behaviour can also be observed in the S3-test since the spacing between neighbouring transverse members is very large (112 mm). In the SV-test, however, such area is naturally limited by the next transverse member with a spacing of 32 mm only. Therefore, failure of junctions only occurred within S1 and S3-tests due to very large junction loads. In the
SV-tests, these junction loads were limited to values considerably below the junction strength and a different failure mode has been observed, i.e. rupture of the longitudinal members. It should be noted that in the SV-tests with the geogrid SG-30 under normal pressures of 50 kPa and 100 kPa no pullout failure but material failure of the longitudinal members occurred. The pullout force increases with increasing normal pressure. However, when the normal pressure increases from 50 kPa to 100 kPa, the increments of the pullout forces are not significant in this study due to the failure of welded junctions or the failure of geogrid longitudinal members. The above two different failure modes have been observed in the experimental pullout tests based on the geogrid samples with different numbers of transverse members.

![Pullout force vs. clamp displacement of geogrids SG-30 and SG-60 for various types of geogrid samples and normal pressures in laboratory tests.](image)

**Figure 5.3** Pullout force vs. clamp displacement of geogrids SG-30 and SG-60 for various types of geogrid samples and normal pressures in laboratory tests.
Figure 5.4 summarizes the maximum pullout force against the number of geogrid transverse members in the laboratory tests. The maximum pullout force increases significantly with increasing number of geogrid transverse members, e.g., compared with the maximum pullout force in the S0-test under the normal pressure of $\sigma_N = 20$ kPa, the increments of the maximum pullout force are 237 % for the S1-test, 503 % for the S3-test and 1250 % for the SV-test, respectively. The contributions of transverse members to the total pullout forces in this study are much larger than those conducted by Alagiyawanna et al. (2001) since the full effects of the geogrid transverse members with large tensile stiffnesses have been mobilized in this study. Due to the failure of welded junctions between longitudinal and transverse members or the failure of geogrid longitudinal members, the influence by increasing the normal pressure from 50 to 100 kPa on the maximum pullout force for the geogrid SG-30 is not significant. However, the maximum pullout force can be increased considerably by using geogrids with higher tensile stiffness provided the same normal pressures, as shown in Figure 5.4.

![Figure 5.4 Maximum pullout force vs. number of geogrid transverse members of geogrids SG-30 and SG-60 in laboratory tests.](image)

### 5.2.4.2 Displacement Distributions along Geogrid

In the laboratory tests, the displacement distribution along the geogrid can be obtained with the displacement measurements fixed at different positions along the geogrid. Figure 5.5 shows the displacement distributions along the geogrid at the clamp displacement of $u_{\text{clamp}} = 2$ mm of the geogrids SG-30 and SG-60. In each test, the maximum displacement occurs at the loaded end of the geogrid and decreases gradually to the free end due to the flexibility of the geogrid. In the S0-tests, the geogrids can be easily pulled out of the specimen with only the frictional resistance caused by the longitudinal members. The elongations of the whole S0-geogrids (from the loaded end to the free end) are quite small. With increasing number of transverse members, the displacements along the geogrid decrease more rapidly due to the mobilization of the bearing resistance caused by the transverse members. In the SV-tests, large elongations mainly exist in the front parts of the geogrids. Similar displacement distributions along the geogrids have been
reported in related studies based on geogrids with regular transverse members (Sugimoto and Alagiyawanna, 2003; Ziegler and Timmers, 2004; Moraci and Recalcati, 2006; Teixeira et al., 2007; Palmeira, 2009; Tran et al., 2013; Jacobs et al., 2014). By comparing the displacement distributions in corresponding tests of geogrids SG-30 and SG-60, smaller elongations can be observed in the tests with geogrid SG-60, which has higher tensile stiffness.

![Graph](image)

**Figure 5.5** Displacement distributions along the geogrid in laboratory tests.

### 5.3 DEM Investigations and Analyses

In order to gain further detailed insights into the geogrid–soil interaction under pullout loads, discrete element modeling using PFC\(^2\text{D}\) has been carried out in this study. Despite the limitations of 2D numerical modeling, which will be described in the later part of this chapter, the fundamental interface behavior between geogrid and soil can be obtained using PFC\(^2\text{D}\) with reasonable computational time.
Figure 5.6 shows the numerical models used in the DEM investigations. The granular soil was modeled with unbonded particles using the linear contact stiffness model and slip model and the geogrid was modeled with bonded particles using the piecewise linear model developed in Chapter 3. The microscopic input parameters for the geogrid and the granular soil in the DEM simulations have been calibrated in Chapter 3 as well. It should be noted that the numerical modeling and calibration of the geogrid in this study were based on the geogrid SG-30.

5.3.1 Numerical Modeling and Calibration Results

In the numerical pullout tests, the width and height of the pullout box were identical to those in the laboratory tests \((W = 435 \text{ mm}, \ H = 200 \text{ mm})\). The 2D porosity for the numerical pullout test was 0.17, identical to the value used in the numerical direct shear tests and the corresponding number of soil particles in the numerical pullout test was 5500. Similar to the laboratory test procedure, the numerical specimen was again prepared using the multilayer compaction method (four horizontal layers), as already described in Chapter 3. The bottom two layers of soil were firstly generated in the pullout box, followed by the geogrid layer on the bottom half of the soil specimen. Then the top two layers of soil were generated on the geogrid layer. For each layer, the equilibrium state with a maximum contact force ratio of 0.001 was reached. After the final layer of soil had reached the equilibrium state, the vertical pressure was applied on top of the specimen via bonded particles. The parallel bond model provided by PFC\(^{2D}\) was applied to the bonded particles and the micro input parameters for those particles are listed in Table 5.2. A constant normal pressure in the geogrid plane could be approximately reached by recording the vertical pressure at the bottom plate and adjusting the applied pressure on the top bonded particles, which was similar to that in the laboratory tests. The geogrid particles were pulled out of the box at the front end with a constant rate of 1 mm/min, which was identical to that in the laboratory tests. The maximum clamp displacement in the DEM investigations was set to 20 mm since almost
all the maximum pullout forces were achieved within the clamp displacement of 20 mm in the laboratory tests. The tensile forces and displacements at different geogrid positions were recorded as well as the vertical pressures on the top bonded particles and on the bottom plate. The numerical pullout tests were conducted under the normal pressure of 50 kPa with four types of modified geogrids (S0, S1, S3 and SV) based on the geogrid SG-30.

<table>
<thead>
<tr>
<th>Table 5.2</th>
<th>Input parameters for the bonded particles as loading plate in pullout tests.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle diameter $d_m$ [mm]</td>
<td>1</td>
</tr>
<tr>
<td>Parallel bond radius multiplier $\lambda$ [-]</td>
<td>0.0005</td>
</tr>
<tr>
<td>Parallel bond normal stiffness $\bar{k}_{nm}$ [$\times 10^{14}$ Pa/m]</td>
<td>76</td>
</tr>
<tr>
<td>Parallel bond shear stiffness $\bar{k}_{sm}$ [$\times 10^{14}$ Pa/m]</td>
<td>20</td>
</tr>
<tr>
<td>Parallel bond normal strength $\bar{\sigma}_{cm}$ [$\times 10^{14}$ Pa]</td>
<td>4</td>
</tr>
<tr>
<td>Parallel bond shear strength $\bar{\tau}_{cm}$ [$\times 10^{14}$ Pa]</td>
<td>4</td>
</tr>
<tr>
<td>Contact normal stiffness $k_{nm}$ [N/m]</td>
<td>$4 \times 10^5$</td>
</tr>
<tr>
<td>Contact shear stiffness $k_{sm}$ [N/m]</td>
<td>$4 \times 10^5$</td>
</tr>
<tr>
<td>Friction coefficient of particles $f_m$ [-]</td>
<td>2.5</td>
</tr>
</tbody>
</table>

The friction coefficient of the geogrid particles is an important micro input parameter, which represents the frictional interface behavior between the geogrid and soil in DEM investigations. In the numerical S0-tests, it has been adjusted iteratively until satisfactory results were obtained compared with the corresponding experimental data. The friction coefficient of the geogrid particles was found to be $f_{gg} = 0.3$ in this study and this parameter has been used in further numerical S1, S3 and SV-tests. As mentioned in the numerical tensile tests, the contributions of transverse members were simulated with high knots. Thus, the bearing resistance caused by the transverse members in those 2D investigations can be transferred to the longitudinal member via the knot particles. The average knot height was adjusted in the numerical S1-test until the numerically obtained contribution of the single transverse member was similar to that in the laboratory test. Then identical knots as used in the numerical S1-test were applied in the numerical S3 and SV-tests.

Figure 5.7 presents the pullout force against clamp displacement of laboratory tests and DEM investigations. The experimental results of the S3 and SV-tests have shown much larger increases of the pullout forces, compared with the corresponding DEM simulation results. Similar observations of the differences between the experimental and the DEM investigations have been reported in the pullout tests of geogrids embedded in ballast (Ferellec and McDowell, 2012). Although the pullout forces at large clamp displacements in the numerical S3 and SV-tests are quite small compared with those in the corresponding laboratory tests, the DEM simulation results at small clamp displacements still show reasonable agreement with the experimental data. Therefore, further analyses of the current DEM simulation results are carried out at small clamp displacements.
5.3.2 Force Distributions along Geogrid and in Specimen

Figure 5.8 shows the tensile force distribution along the geogrid with four different types of modified geogrids at the clamp displacement of \( u_{\text{clamp}} = 2 \text{ mm} \). The maximum tensile forces in all the above tests occur at the geogrid loaded ends and the values reduce to zero at the geogrid free ends. In the S0-test, the tensile force decreases gradually from the loaded end to the free end since the pullout force is caused only by the frictional resistance between the geogrid longitudinal members and soil. With increasing number of transverse members, the tensile forces at the loaded ends increase, which clearly shows the contributions of transverse members to the total pullout resistance. In the S1, S3 and SV-tests, sudden jumps of the tensile forces, which are caused by the transfer of bearing resistance of transverse members, occur at the positions of transverse members, as shown in Figure 5.8. The DEM simulation results in this study show good agreement with the FEM simulation results by Wilson-Fahmy and Koerner (1993) and the results also verify the mechanical model proposed by Ziegler and Timmers (2004), Ziegler et al. (2007) and Jacobs et al. (2014).
Figure 5.8  Tensile force distributions along the geogrid in DEM investigations ($u_{\text{Clamp}} = 2$ mm).

Figure 5.9 visualizes the contact force distribution in the specimen together with the tensile force distribution along the geogrid at the clamp displacement of $u_{\text{Clamp}} = 2$ mm. The black lines represent the contact forces in the specimen, while the red lines represent the tensile forces along the geogrid. The thickness of the lines is proportional to the magnitude. Moreover, the curves in Figure 5.9 also illustrate the tensile force distribution along the geogrid quantitatively. In the S0-test, the pullout force is transferred gradually from the geogrid longitudinal member to the granular soil, which causes slight orientation changes of the contact forces (i.e. from nearly vertical to diagonal), as shown in Figure 5.9a. Similar observations have been reported in the numerical compound tensile tests with one geogrid tensile member in Chapter 4. Furthermore, in the S1, S3 and SV-tests, the geogrid transverse members have been mobilized and large contact forces can be observed in front of the transverse members, which results in the sudden jumps of the tensile forces along the geogrid as shown in Figures 5.9b-d. The contact force distributions observed in the DEM investigations agree well with the experimental studies conducted by Dyer (1985) and the finite element simulation results carried out by Abdi and Zandieh (2014).
5 Pullout Tests

(a) S0 – geogrid without transverse members.

(b) S1 – geogrid with only one transverse member in the middle.
5.3 DEM Investigations and Analyses

Figure 5.9  Force distributions along the geogrid and in the specimen in DEM investigations ($\mu_{\text{clamp}} = 2 \text{ mm}$) (thickness of lines proportional to magnitude: red – tensile forces along the geogrid; black – contact forces in the specimen).
In order to better understand the development of force chains within the specimen, the contact and force distributions can be visualized with polar histograms. The polar histograms of the contact and force distributions were obtained by statistically collecting the information of contacts and forces within an angular interval $\Delta \theta$, as shown in Figure 5.10. The angular interval was selected as $10^\circ$ in this study according to the suggestions in similar studies (Han et al., 2012; Lai et al., 2014). The polar histogram of the contact distribution was normalized by the total number of the contacts. Moreover, the polar histograms of the normal and the tangential contact force distributions were normalized by the average normal contact force over all contacts. Figure 5.10 shows the normalized histograms of the contact and force distributions in the SV-test at the clamp displacements of $u_{\text{Clamp}} = 0$ mm and $u_{\text{Clamp}} = 2$ mm, respectively. It should be noted that only the bottom half of the specimen was selected for investigation since the contact forces distributed almost symmetrically above and below the geogrid layer. Figure 5.10 also shows the Fourier series approximations of the normalized contact and force distributions, in which the microstructural parameters can be achieved, e.g. the principal directions of contacts and forces. More detailed illustrations about the Fourier Series Approximation (FSA) method can be found in Rothenburg and Bathurst (1989) and Bathurst and Rothenburg (1990, 1992). The corresponding mathematical expressions of the FSA method are as follows:

$$E(\theta) = \frac{1}{2\pi} \left[ 1 + a_c \cos 2(\theta - \theta_c) \right]$$

$$\bar{f}_n(\theta) = \bar{f}_0 \left[ 1 + a_n \cos (\theta - \theta_n) \right]$$

$$\bar{f}_t(\theta) = \bar{f}_0 a_t \sin (\theta - \theta_t)$$

$$\bar{f}_0 = \frac{1}{2\pi} \int_0^{2\pi} \bar{f}_n(\theta)\,d\theta = \frac{1}{N} \sum_{k=1}^{N} f_n^k$$

where $E(\theta)$ represents the contact normal distribution function; $\bar{f}_n(\theta)$ and $\bar{f}_t(\theta)$ represent the average normal and tangential force distributions; $\bar{f}_0$ represents the average normal contact force over all contacts; $a_c, a_n$ and $a_t$ represent the second-order coefficients of the contact normal anisotropy as well as the average normal and tangential contact force anisotropy; $\theta_c, \theta_n$ and $\theta_t$ represent the corresponding principal directions of the above anisotropy; $N$ represents the number of orientation intervals used in the approximation; $f_n^k$ represents the contact normal force in each orientation interval.
Figure 5.10 Orientations of contacts and forces in DEM investigations.
At the clamp displacement of $u_{\text{Clamp}} = 0$ mm, the contact distribution is isotropic with an approximate circular shape. The principal directions of the normal and the tangential contact forces are 88.9° and 90°, respectively. With increasing clamp displacement, the pullout force is transferred from the geogrid to the granular soil, which causes the reorientation of the major principal stresses. At the clamp displacement of $u_{\text{Clamp}} = 2$ mm, the contact distribution is also isotropic. However, the principal directions of the normal and the tangential contact forces have changed into 129.2° and 127.8°, respectively. These observations quantitatively illustrate the rotations of the contact forces in this study. It should be noted that the normalized polar histograms shown in this study are not spatially invariant and they are highly dependent on the selected investigation area.

### 5.3.3 Contributions of Geogrid Transverse Members

Based on the tensile force distribution along the geogrid in the S1, S3 and SV-tests, the contributions of the transverse members to the total pullout resistance in those tests are presented in Figure 5.11. In the S1-test, the single transverse member contributes to almost 30% of the total pullout resistance, while the whole contributions of the transverse members in the S3 and SV-tests are around 50% and 75%, respectively. In the S3 and SV-tests, the first transverse member (counted from the loaded end to the free end) contributes to the largest bearing resistance and a reduced trend on the contributions of transverse members has been observed from the loaded end to the free end, as shown in Figures 5.11b-c. Moreover, the bearing resistance caused by the transverse members amounts to the force applied on the junctions between the longitudinal and transverse members. The average applied forces on the junctions in the S1, S3 and SV-tests decrease with increasing number of transverse members, which can be used to explain the different failure modes in the experimental S1, S3 and SV-tests.

![Contribution of transverse member to total pullout resistance](image)

(a) S1 – geogrid with only one transverse member in the middle.
5.3.4 Displacement and Strain Distributions along Geogrid

Figure 5.12 shows the displacement distributions along the geogrid with four different types of modified geogrids at the clamp displacement of $u_{\text{Clamp}} = 2\, \text{mm}$. Although the curves in the DEM investigations do not fit very well with the corresponding experimental results in Figure 5.5a due to the simplification of the real 3D problem into a 2D modeling, the DEM investigations still provide a reasonable trend for the displacement distributions along the geogrid. Among all the numerical investigations in this study, the smallest geogrid elongation has been observed in the S0-test and the values increase with increasing number of transverse members. These observations in the DEM investigations are similar to those obtained in the laboratory tests and they also verify the mechanical model proposed by Jacobs et al. (2014). According to the suggestion of Bathurst and Ezzein (2015), the geogrid displacement normalized by the clamp displacement against the distance from the front clamp normalized by the specimen length is shown in Figure 5.13. The exponential function described by Bathurst and Ezzein (2015) fits the measured data from the DEM simulations of this study very well.
Based on the displacement distributions along the geogrid in Figure 5.12, the corresponding tensile strain distributions along the geogrid with four different types of modified geogrids at the clamp displacement of $u_{\text{Clamp}} = 2$ mm are shown in Figure 5.14. It should be noted that the elongations at the junctions were quite small due to the large knot height in the DEM studies. Hence, the tensile strains at the junctions have been removed from the curves in Figure 5.14. The shapes of the tensile strain distribution curves in Figure 5.14 are quite similar to those of the tensile force distribution curves in Figure 5.8, which demonstrates that the large tensile strains along the geogrid are caused by the corresponding large tensile forces. Moreover, the tensile strain distributions along the geogrid also illustrate the influence of geogrid transverse members on the pullout behavior. The strain distributions along the geogrid in this study show similar shapes with the measured and modelled geogrid strain distributions by Jacobs et al. (2014).
5.3 DEM Investigations and Analyses

5.3.5 Normal Stress Distributions in Geogrid Plane

The normal stress distribution in the geogrid plane is regarded to be constant when calculating the relevant design parameters for most of geogrid reinforced earth structures (Sugimoto and Alagiyawanna, 2003; Huang and Bathurst, 2009; Miyata and Bathurst, 2012; Yang et al., 2012). Therefore, the normal stress in the geogrid plane has been controlled to be constant in this study. In order to verify whether the normal stress distribution in the geogrid plane is approximately constant, the method of collecting the information of contacts and forces illustrated in Section 5.3.2 has been used to collect the normal and tangential contact forces in 11 measuring areas, as shown in Figure 5.15. Since the minimum particle size of the granular soil (2 mm) was twice of the geogrid particle size (1 mm) in the DEM investigations, the contact orientations between soil and geogrid particles were larger than 70° and smaller than 110°, as shown in Figure 5.15. The contact orientations were determined with the positive x-axis. Thin measuring areas were selected to assure that most of the contacts between the soil particles were not included in the measuring areas. Some contacts between soil particles might be included in the measuring areas; however, their contacts are mainly in the horizontal direction, i.e. those contact orientations could not be larger than 70° or smaller than 110°, as shown in Figure 5.15.

The main FISH codes of obtaining the normal stress distribution in geogrid plane are listed in Appendix A.

Figure 5.14 Strain distributions along the geogrid in DEM investigations ($u_{\text{Clamp}} = 2$ mm).
Figure 5.15 Measuring areas in geogrid plane.

Figure 5.16 presents the normal stress distributions in the geogrid plane along the geogrid at different clamp displacements in the numerical SV-test. Before pulling the geogrid out of the specimen \(u_{\text{Clamp}} = 0 \text{ mm}\), the normal stress distribution along the geogrid varies approximately \(\pm 10\%\) around the target value of \(\sigma_N = 50 \text{ kPa}\). With increasing clamp displacement, the normal stresses in the front part of the geogrid plane increase sharply, while the normal stresses in the rest part of the geogrid plane remain almost constant with slight reductions in the back part of the geogrid plane, as shown in Figure 5.16. At the clamp displacement of \(u_{\text{Clamp}} = 5 \text{ mm}\), the maximum normal stress is up to around \(600\%\) of the required normal stress. These results are quite similar to the observations of the vertical stresses recorded at a distance of 100 mm above the geogrid layer under pullout loads conducted by Tran et al. (2013). They explained that the stress increase at the front part of the pullout box might be caused by using a rigid horizontal plate for loading. However, Palmeira and Milligan (1989) and our own comparative calculations showed that there was no considerable difference in pullout mobilization at small clamp displacements when using a rigid top boundary or a flexible top boundary. It is rather assumed that the high normal stress in the front part of the geogrid plane close to the front wall of the pullout box is mainly caused by the horizontal forces induced by the geogrid.
5.4 Limitations

The numerical simulations in this study were carried out using 2D software, which has inherent limitations in investigating the real 3D problems, e.g. a 2D model dilates with a much higher rate than a 3D model since the 2D plane assembly can only dilate in the vertical and horizontal directions but not in the cross-plane direction (Rothenburg and Bathurst, 1992). The granular soil was modeled with circular discs in this study and large frictional coefficients have been used to compensate the lack of angularity of circular particles (Lin et al., 2013; Zhang et al., 2013). The irregular shapes of real particles can be modeled by overlapping spheres to form clumps (Ferellec and McDowell, 2010a, 2010b). However, due to the high requirement of computing power, this technique is currently only applied for simulating medium/coarse gravel and ballast (Lackner, 2012; Chen et al., 2014a; Ngo et al., 2014; Stahl et al., 2014). The real 3D geogrid in this study was simplified into a string of continuously connected particles and relative high knot particles at the corresponding positions were simulated to investigate the contributions of geogrid transverse members. Although the above simplifications in this study surely cause differences between the experimental and the DEM investigations, the numerical investigations still provide helpful results to illustrate the fundamental interface behavior between geogrid and soil within reasonable computational time.

5.5 Summary

This study investigated the pullout behavior of geogrids embedded in granular soil with both experimental and numerical approaches. In the laboratory tests, geogrids with different tensile
stiffnesses have been used and those geogrid products have been modified into specimens with
different numbers of transverse members. In the discrete element modeling, the models and the
input parameters of the granular soil and the geogrid were used from the direct shear tests and the
tensile tests in Chapter 3, respectively. The numerical pullout tests were also carried out with
geogrids modified with different numbers of transverse members. The main conclusions of this
chapter can be drawn as follows:

1. In the laboratory tests, two different failure modes were observed based on the geogrids with
different numbers of transverse members in this study, i.e. failure of the welded junctions
between the geogrid longitudinal and transverse members and failure of the geogrid
longitudinal members. The maximum pullout resistance increased with increasing number of
geogrid transverse members or with increasing geogrid tensile stiffness.

2. In the discrete element modeling, the geogrid–soil interaction has been visualized by the force
distributions along the geogrid and in the specimen with different numbers of transverse
members. The quantitative force, displacement and strain distributions along the geogrid with
different numbers of transverse members have been obtained, which also illustrated the load
transfer behavior between geogrid and granular soil. The bearing resistance caused by the
transverse members was reduced from the loaded end to the free end. The average
contributions of geogrid transverse members decreased with increasing number of transverse
members, which could be used to explain the different failure modes in the laboratory pullout
tests.

3. Based on the Fourier Series Approximation (FSA) method, the reorientations of contacts and
forces in the specimen have been obtained at different clamp displacements as well as the
normal stress distributions in the geogrid plane. The principal contact direction in the SV-
test was changed from nearly vertical (θ ≈ 90°) at the initial state to diagonal (θ ≈ 128°) at the
clamp displacement of $u_{\text{Clamp}} = 2 \, \text{mm}$. The normal stress distribution in the geogrid plane was
not constant and the maximum normal stress near the loaded end was up to around 600 % of
the required normal stress. The DEM simulation results provide researchers more detailed
insights into the geogrid–soil interaction at a microscopic scale under pullout loads.
6 Large-scale Biaxial Compression Tests

6.1 Introduction

Since most geosynthetic reinforced soil (GRS) structures (e.g. retaining walls and embankments) are close to being in a plane strain condition, biaxial compression tests are regarded as the most appropriate testing methodology for the investigation of the compound stress–strain behavior of geogrid reinforced soil (Ketchart and Wu, 2001). Hence, large-scale biaxial compression tests are carried out in this chapter both experimentally and numerically.

In the laboratory tests, the stress–strain behavior of the used unreinforced soil is investigated under six different confining stresses. For the reinforced specimens, six types of geogrid samples with different numbers of longitudinal and transverse members are used to study the contributions of geogrid longitudinal and transverse members to the compound stress–strain behavior of reinforced soil. Additionally, based on the Digital Image Correlation (DIC) method, the kinematic behavior of both unreinforced and reinforced specimens is visualized with particle displacement distributions and particle rotations. Moreover, the geogrid strain distributions are obtained using attached strain gauges on the geogrid samples.

In the numerical investigations, the micro input parameters for the geogrid have been calibrated in Chapter 3, while the micro input parameters for the granular soil are calibrated with the biaxial compression test results of unreinforced specimens in this chapter. The contributions of geogrid longitudinal and transverse members to the compound stress–strain behavior of geogrid reinforced specimens are discussed in the DEM investigations. The numerically obtained kinematic behavior of both unreinforced and reinforced specimens are evaluated and compared in this chapter. In addition, the load transfer behavior between soil and geogrid is visualized not only by the qualitative force distributions in the specimen and along the geogrid but also by the quantitative tensile force and tensile strain distributions along the geogrid.

6.2 Laboratory Biaxial Compression Tests and Analyses

6.2.1 Testing Apparatus

Figure 6.1 shows the sketch of the large-scale biaxial compression test apparatus, which was developed by Ruiken (2013) at RWTH Aachen University. The frame of the apparatus was made of steel to support the boundaries as well as the loading and measuring systems. Large-scale specimens with inside dimensions of $H/W/D = 800/810/460$ (unit: mm) are located in the middle part of the apparatus. In order to form the plane strain conditions, the positions of both front and
back walls were fixed and therefore, the specimen can only dilate in the lateral directions (i.e. left and right). For the lateral boundaries, flexible latex membranes have been used and the lateral deformations of both sides along the specimen’s height can be recorded with the lateral deformation measuring system using the lateral cameras (see Figure 6.1). The confining stress was applied to each specimen via vacuum so that a constant stress on all sides of the specimen could be achieved, as shown in Figure 6.2.

![Figure 6.1 Sketch of the large-scale biaxial compression test apparatus (after Jacobs and Ziegler 2015).](image)

The vertical stress was applied on top of the specimen via a rigid plate using a hydraulic plunger and the vertical stress distribution can be measured with sensitive foil sensors, which were placed between the specimen and the rigid plate. During the compression process, a series of photos were taken through the transparent glass wall using a high-resolution digital camera. The kinematic behavior, i.e. displacements and rotations of soil particles, was then determined with the postprocessing of the photos based on the DIC method.

In order to minimize the contact friction at the interfaces between the top/bottom rigid plates and the specimen, a lubrication method by means of a thin latex membranes and a silicone grease layer was used according to the suggestion from Tatsuoka and Haibara (1985). Based on the lubrication method, the interface friction angle was determined with laboratory direct shear tests to be less than 2° (Ruiken et al., 2010). It should be noted that such laboratory tests were carried out with sand. The transparent glass wall, however, was not lubricated for the purpose of a high quality...
evaluation of the kinematic behavior. Based on laboratory direct shear tests, the interface friction angle between glass and the investigated gravel was determined to be around 22°. More detailed information about the test apparatus can be found in Ruiken (2013).

Figure 6.2 Specimens used in large-scale biaxial compression tests.

6.2.2 Testing Materials

6.2.2.1 Granular Soil

In this chapter, dry granular soil with particle sizes from 2 to 8 mm has been used, as shown in Figure 6.3. The particle size distribution of the gravel in the large-scale biaxial compression tests was identical to that used in the pullout tests (see Chapter 5). However, the testing density of the soil sample in the biaxial compression tests was 1.60 g/cm³, which was different from the value (1.56 g/cm³) used in the pullout tests. The internal friction angle of the soil sample under plane strain conditions was calculated based on the experimental biaxial compression test results of unreinforced specimens in Section 6.2.4.1.

Figure 6.3 Particle size distributions of gravel in biaxial compression tests and DEM simulations.
6.2.2.2 Geogrid

Figure 6.4 shows a normal biaxial geogrid sample used in the biaxial compression tests with a length and a width of 808 mm and 456 mm, respectively. Such biaxial geogrids were manufactured by flat prestressed tensile members with rigid thermally welded junctions. The breadth and thickness of a single geogrid tensile member were around 8 mm and 1 mm, respectively. The biaxial geogrid samples were made of polypropylene (PP) with an approximate aperture size of 32 mm × 32 mm. Those dots on the geogrid sample represent the positions of strain gauges so that the geogrid strain distributions can be obtained during the loading process.

![Figure 6.4](image)

In order to investigate the contributions of geogrid longitudinal and transverse tensile members to the compound stress–strain behavior of reinforced soil, six different types of geogrid samples (L12-S21, L12-S11, L12-S0, L6-S21, L6-S11, L6-S0) were used in this chapter, as shown in Figure 6.5. The letters “L” and “S” represent the longitudinal member and transverse member, respectively, and the numbers after the letters represent the corresponding numbers of the longitudinal and transverse members, respectively. For example, L12-S21 represents a geogrid sample consisting of 12 longitudinal members and 21 transverse members, as shown in Figure 6.4. The other geogrid samples were prepared by carefully removing the corresponding longitudinal and transverse members from the normal welded biaxial geogrid products. It should be noted that very narrow transverse members were kept at both ends of the longitudinal members for the geogrid samples L12-S0 and L6-S0, which made the installation of the above geogrid samples in the laboratory tests more applicable. All the geogrid samples used in this chapter were prepared from geogrid SG-30 with the tensile stiffness of $J_{0.2\%} = 700$ kN/m and the ultimate tensile strengths of the geogrid in both machine and cross machine directions of 30 kN/m.
6.2 Laboratory Biaxial Compression Tests and Analyses

6.2.3 Testing Procedure and Program

In order to achieve uniform compaction, the biaxial specimen was prepared with 16 horizontal layers using the supporting system for the specimen preparation (see Figure 6.1). After each layer of soil was filled, it was then compacted until the height reached the required value. The required height of each soil layer was 50 mm and the target testing density was 1.60 g/cm$^3$. For the geogrid reinforced specimens, two layers of geogrids attached with strain gauges were placed at the positions of 1/4 and 3/4 of the specimen’s height, respectively (see Figure 6.2b). Such placement has been regarded as the most effective way to activate the reinforcement effects with two layers of geogrids embedded in sand (Ruiken et al., 2012). The strain gauges were used to record the geogrid strain development during the whole loading process. After preparing the biaxial specimen, a constant confining stress was applied to the specimen via vacuum. Then the supporting system for the specimen preparation was removed and the lateral deformation measuring system (including the left and right cameras), the digital camera in front of the specimen for DIC analyses and the sensitive foil sensors for the vertical stress distribution measurement were set up as well as the loading equipment.

Figure 6.5 Six types of geogrid samples used in biaxial compression tests (dots on the geogrid samples represent the positions of strain gauges).
During the loading process, the biaxial specimen was compressed vertically with a constant rate of 1 mm/min until the maximum vertical displacement of 80 mm was reached. All data, i.e. the total vertical load and displacement, the vertical stress distribution, the lateral deformation and the digital photos through the transparent glass wall for further DIC analyses were collected automatically. For the geogrid reinforced specimens, the geogrid strain distributions were also collected with the attached strain gauges. The testing procedure in this study was identical to that carried out by Ruiken (2013) with sand, in which the testing procedure was illustrated in more details.

The testing program of this study is listed in Table 6.1. The internal friction angle of the testing material was obtained based on unreinforced specimens under six different confining stresses. For reinforced specimens, the contributions of geogrid longitudinal and transverse tensile members to the compound stress–strain behavior of reinforced soil were investigated in this study. It should be noted that the very low confining stress ($\sigma_3 = 2.5$ kPa) was selected in order to provide optimum conditions for the activation of geogrids allowing the investigation of even the smallest geogrid reinforcement effects (Ruiken et al., 2012).

<table>
<thead>
<tr>
<th>Test items</th>
<th>Geogrid types</th>
<th>Confining stress $\sigma_3$ [kPa]</th>
<th>Number of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced</td>
<td>Without geogrids</td>
<td>2.5, 5, 10, 15, 20, 25</td>
<td>6</td>
</tr>
<tr>
<td>Reinforced</td>
<td>L12-S21, L12-S11, L12-S0, L6-S21, L6-S11, L6-S0</td>
<td>2.5</td>
<td>6</td>
</tr>
</tbody>
</table>

### 6.2.4 Laboratory Testing Results and Analyses of Unreinforced Specimens

#### 6.2.4.1 Stress–Strain and Volumetric Strain Behavior

Due to the large specimen dimensions (800 mm high) and the relative low confining stresses (from 2.5 kPa to 25 kPa), the self-weight of the specimen should not be neglected when calculating the total vertical stresses. Hence, according to the suggestions from Jacobs et al. (2012) and Ruiken et al. (2012), an additional vertical stress, which was calculated by half of the specimen’s weight at middle height of each specimen, was added to compute the total vertical stress, as illustrated in Equation 6.1.

$$\Delta \sigma_1 = \rho_{\text{soil}} \cdot g \cdot \frac{H}{2}$$  \hspace{1cm} (6.1)

where $\Delta \sigma_1$ is the additional vertical stress;

$\rho_{\text{soil}}$ is the testing density of the specimen, $\rho_{\text{soil}} = 1.60$ g/cm$^3$ in this study;

$H$ is the initial height of the specimen, $H = 800$ mm in this study.
Since the confining stress was applied via vacuum on the specimen, it has been regarded that the stresses on all boundary sides of the specimen were identical, that is, the confining stress was also applied on the top surface of the specimen. During each biaxial compression process, the value of the confining stress was kept constant. Hence, the confining stress was also added to calculate the total vertical stress. To sum up, the total vertical stress was calculated with the following equation.

\[
\sigma_1 = \frac{F_v}{A} + \Delta \sigma_1 + \sigma_3
\]

where \( \sigma_1 \) is the total vertical stress;
\( F_v \) is the vertical load on top of the specimen, including the self-weight of the loading plate;
\( A \) is the average cross-sectional area of the specimen;
\( \sigma_3 \) is the confining stress.

Figure 6.6 presents the total vertical stress–strain relations and the volumetric strain–vertical strain relations of the unreinforced specimens under different confining stresses in the laboratory tests. Obviously, all the unreinforced specimens show strain-softening responses and the maximum vertical stress of each unreinforced specimen increases with increasing confining stress, as shown in Figure 6.6a. It should be noted that the vertical stresses under different confining stresses do not start with zero at the vertical strain of \( \varepsilon_1 = 0 \% \) since the self-weight of the loading plate, the additional vertical stress and the corresponding confining stress have been added to compute the total vertical stress (see Equation 6.2). At small vertical strains, the lateral deformation of the unreinforced specimen decreases with increasing confining stress, which leads to a reduced volumetric strain, as shown in Figure 6.6b.
Large-scale Biaxial Compression Tests

Dissertation Zhijie Wang

The maximum vertical stresses of the unreinforced specimens under different confining stresses in the laboratory tests are summarized in Figure 6.7, in which a good linear relationship between the maximum vertical stress and the confining stress can be observed. Thereby, the internal friction angle of the assembly can be calculated with the following equation.

\[
\sin \phi_{BX} = \frac{\sigma_{1,\max} - \sigma_3}{\sigma_{1,\max} + \sigma_3} = \frac{\sigma_{1,\max}}{\sigma_3} - 1
\]

\[
= \frac{\sigma_{1,\max}}{\sigma_3} + 1
\]

(6.3)

Figure 6.6  Vertical stress and volumetric behavior of unreinforced specimens in laboratory tests.

Figure 6.7  Maximum vertical stress vs. confining stress of unreinforced specimens in laboratory tests.
The internal friction angle of the assembly obtained from the large-scale biaxial compression tests is $\varphi_{BX} = 60.6^\circ$, which is larger than that obtained from the large-scale direct shear tests ($\varphi_{DS} = 40.9^\circ$) (see Chapter 3). Different testing densities of the soil samples and different boundary conditions in both tests surely lead to different frictional angles of the assemblies. Moreover, the lateral friction between soil and glass walls in biaxial compression tests is also a main reason for the large internal friction angle of the assembly. Furthermore, the total vertical stresses in the biaxial compression tests were calculated by including half of the specimen’s weight, which also results in a higher internal friction angle than that in the direct shear tests. Similar observations and explanations have been reported by Ruiken (2013) in the comparison of biaxial compression and direct shear test results with sand.

### 6.2.4.2 Kinematic Behavior

Using the DIC method, the total and horizontal particle displacements as well as the particle rotations of the unreinforced specimen at the vertical strain of $\varepsilon_1 = 10\%$ are shown in Figure 6.8 ($\sigma_3 = 2.5\, \text{kPa}$). It is obvious to find that two pieces of shear zones develop from the middle top to the left and right bottoms of the specimen and thus, two sliding wedges are formed. The two sliding wedges distribute almost symmetrically along the central axis, which leads to the almost symmetrical distributions of total and horizontal particle displacements as well as particle rotations. In Figure 6.8c, different colors represent particles with different rotation directions and different rotation degrees, e.g. red and violet represent the maximum particle rotations clockwise and counterclockwise, respectively. Similar kinematic behavior of unreinforced sand specimens was presented by Ruiken (2013) with the same testing apparatus and the same evaluation method.

The particle displacements and rotations of unreinforced specimens under different confining stresses (i.e. from 5 kPa to 25 kPa) are summarized in Appendix B. The shear zones in all unreinforced specimens under different confining stresses develop through the specimens from top to bottom although slightly different kinematic behavior is observed.

![Figure 6.8](image.png)

**Figure 6.8** Particle displacements and rotations of the unreinforced specimen ($\sigma_3 = 2.5\, \text{kPa}$).
6.2.5 Laboratory Testing Results and Analyses of Reinforced Specimens

6.2.5.1 Stress–Strain and Volumetric Strain Behavior

For each geogrid reinforced specimen, the total vertical stress was also computed with the vertical compression load, the additional vertical stress caused by half of the specimen’s weight and the confining stress, which was identical to that in the unreinforced specimens. The detailed computational process is shown in Equation 6.2.

In this chapter, all reinforced specimens were prepared with two layers of geogrids with different numbers of longitudinal and transverse members. In order to illustrate the influences of the geogrid longitudinal and transverse members on the reinforcement effects, the geogrid samples have been classified into two series, i.e. L12 series (all geogrid samples with 12 longitudinal members) and L6 series (all geogrid samples with 12 longitudinal members). The vertical stress–strain relations of the reinforced specimens with different types of geogrid samples are shown in Figure 6.9. The confining stress for all reinforced specimens was $\sigma_3 = 2.5$ kPa. For the purpose of comparison, the vertical stress–strain behavior of the unreinforced specimen with a confining stress of 2.5 kPa is also presented in Figure 6.9.

Compared with the unreinforced specimen, the compression strengths of the reinforced specimens are greatly increased due to the activation of the geogrid reinforcements. In each series, the maximum vertical stresses of the reinforced specimens increase with increasing number of geogrid transverse members. The initial moduli $E_0$ of the reinforced specimens are slightly larger than that of the unreinforced specimen. Moreover, by comparing the corresponding curves with the same numbers of geogrid transverse members in Figures 6.9a and 6.9b, it is obvious that the compression strength of the reinforced specimen with two layers of 12 longitudinal members is always larger than that with two layers of 6 longitudinal members.
The volumetric strain–vertical strain relations of the unreinforced and reinforced specimens are shown in Figure 6.10. Under the same vertical compressing rate, the lateral deformations of reinforced specimens have been reduced due to the lateral confinement effect of geogrids at small vertical strains. The lateral confinement effect is more obvious with increasing number of geogrid transverse members.
Figure 6.10 Volumetric strain–vertical strain relations of unreinforced and reinforced specimens in laboratory tests.

Figure 6.11 summarizes the maximum vertical stresses of the unreinforced specimen and geogrid reinforced specimens with different numbers of longitudinal and transverse members. The maximum vertical stress of the reinforced specimen with only two layers of 6 longitudinal members (without transverse members) has been increased to more than 250 % of that of the unreinforced specimen. The increase of the maximum vertical stress is caused only by the frictional resistance between the geogrid longitudinal members and soil. With increasing number of the geogrid longitudinal and/or transverse members, the maximum vertical stresses of the reinforced specimens increase. The maximum vertical stress of the reinforced specimen with two layers of normal geogrids (L12-S21) has been increased to approximate 900 % of that of the unreinforced specimen. The huge increase is caused not only by the frictional resistance between the geogrid surfaces and soil but also by the bearing resistance of the transverse members.

Compared with the increases of the peak strengths in the L6 series (i.e. from 104 kPa without transverse members to 209 kPa with 21 transverse members), the increases of the peak strengths in the L12 series are more significant (i.e. from 126 kPa without transverse members to 405 kPa with 21 transverse members). Moreover, the difference of the peak strengths in both L6 and L12 series without transverse members is only 22 kPa, while the difference in both series with 21 transverse members reaches around 200 kPa. The above observations show that the geogrid reinforcement effects, referring to the strengths of the composite material, are caused by the combination of geogrid longitudinal and transverse members.
6.2 Laboratory Biaxial Compression Tests and Analyses

6.2.5.2 Kinematic Behavior

Identical to the evaluation process of the unreinforced specimens, the DIC method has also been used to evaluate the kinematic behavior of the reinforced specimens. Figure 6.12 presents the total and horizontal particle displacements as well as the particle rotations of the reinforced specimens (L12 series) at the vertical strain of $\varepsilon_1 = 10\%$. For the purpose of comparison, the kinematic behavior of the unreinforced specimen with $\sigma_3 = 2.5$ kPa is shown in Figure 6.12 as well.

Due to the activation of different types of geogrid samples, the kinematic behavior of the reinforced specimens in Figure 6.12 has been greatly changed compared with that of the unreinforced specimen. With only two layers of 12 longitudinal members (L12-S0), the compression load applied on the top plate cannot be transferred completely to the bottom part of the specimen since the frictional resistance between the longitudinal members and soil has been activated. The activated frictional resistance leads to the formations of several smaller shear zones in the reinforced specimen. Since the frictional resistance of the longitudinal members cannot fully withstand the transferred loads, two pieces of shear zones are also developed from the top of the specimen to the bottom. It should be noted that those two pieces of shear zones in the reinforced specimen of L12-S0 are not as clear as those in the unreinforced specimen with the same confining stress. With increasing number of transverse members, more and more soil particles are interlocked by the geogrid apertures due to the activation of transverse members. In the reinforced specimen of L12-S21, the total and horizontal particle displacements mainly occur at the corners of the top part and the side areas of the middle part of the specimen. The particle displacements below the lower geogrid layer are quite small. Moreover, the shear zones caused by the particle rotations in the specimen of L12-S21 are not as obvious as those in the unreinforced specimen and in other reinforced specimens despite the smaller shear zones developed in the
middle part of the specimen. Similar kinematic behavior of reinforced sand specimens was presented by Ruiken (2013) with the same testing apparatus and the same evaluation method.

The particle displacements and rotations of the reinforced specimens with 6 longitudinal members (L6 series) are summarized in Appendix B. Similar kinematic behavior is observed.

Figure 6.12 Particle displacements and rotations of unreinforced and reinforced specimens in laboratory tests (L12 series).

6.2.5.3 Geogrid Tensile Strain Distributions

In the laboratory tests, 12 measuring points, which were symmetrically distributed along the specimen center on each geogrid layer, were selected to record the tensile strain distributions along the geogrid. Since the geogrid samples in the specimen might bend upwards or downwards under the biaxial compression loads, two strain gauges attached on and beneath the longitudinal member
at each measuring point were used to record the respective strains. The average value of both strains was used to represent the true tensile strain at each measuring point. The strains at both free ends of the geogrid layers were regarded to be zero since the geogrids were not connected to the lateral boundaries of the specimen.

Figure 6.13 shows the tensile strain distributions along the geogrid in the specimen of both L12 and L6 series at different vertical strains. For both geogrid layers in each specimen, the tensile strains increase with increasing vertical strain. Due to the failure of strain gauges or the failure of connections between the strain gauges and the corresponding connecting cables during the loading process, the maximum measurable tensile strains of geogrids in this study is around 2%. Hence, several points of geogrid tensile strains, mainly in the middle part of the specimen at large vertical strains, are not included in the tensile strain distributions in Figure 6.13. For each geogrid layer, the maximum tensile strain occurs at the middle part of the specimen and decreases gradually from the middle part to the side boundaries of the specimen. By comparing the tensile strains within upper and lower geogrid layers at the same vertical strains, it is obvious that the tensile strains within the upper geogrid layers are larger than those within the lower geogrid layers. The large tensile strains within upper geogrid layer are caused by the large tensile forces, which reveals that more vertical compression loads are transferred from soil to the upper geogrid layer compared with the lower geogrid layer since the location of the upper geogrid layer is closer to loading plate.

By comparing the tensile strain distributions in both L12 and L6 series with the same numbers of transverse members (e.g. Figures 6.13a and 6.13b), it can be seen that the tensile strains with 6 longitudinal members (L6) are larger than those with 12 longitudinal members (L12) at the same vertical strain. It represents that geogrid samples with fewer geogrid longitudinal members can be stretched easily so that each longitudinal member of those geogrid samples (L6 series) is exposed to larger tensile forces compared with that with more geogrid longitudinal members (L12 series).
Tensile strain distributions along the geogrid in laboratory tests.

Figure 6.13 Tensile strain distributions along the geogrid in laboratory tests.
6.3 DEM Investigations and Analyses

In this section, discrete element modeling using PFC\textsuperscript{2D} has been conducted to get further detailed insights into the geogrid–soil interaction under biaxial compression loads. Using the linear contact stiffness model and slip model, the granular soil was modeled with clumps and each clump was formed by overlapping three circular discs, as shown in Figure 6.3. Since the intragranular bonds in the clump are regarded to be infinitely stiff and strong, the clump cannot break throughout the simulation. The micro input parameters of the clumps were calibrated by matching the numerical simulation results of unreinforced biaxial specimens with the corresponding experimental data. The geogrid was modeled with bonded particles using the piecewise linear model developed in Chapter 3 and the corresponding micro input parameters listed in Table 3.5 of Chapter 3.

6.3.1 Numerical Modeling of Large-scale Biaxial Compression Tests

Laboratory tests showed almost symmetrical results along the central axis, e.g. kinematic behavior and geogrid tensile strain distributions. Hence, in order to obtain the DEM simulation results within reasonable computational time, it is possible to simplify the numerical specimens into half of the specimens in the laboratory tests, as shown in Figure 6.14. Based on the contact bond model, a string of bonded particles was used to simulate the lateral flexible membrane boundary. The height of the numerical specimen was identical to that of the physical specimen ($H = 800$ mm), while the width of the numerical specimen was half of that of the physical specimen ($W = 405$ mm). The thickness of the clumps along the cross-plane direction was 8 mm, identical to the value of numerical soil particles reported in previous chapters. For the numerical reinforced specimens, two layers of geogrids were placed at the positions of 1/4 and 3/4 of the specimen’s height, which was identical to that in the physical reinforced specimens.

![Figure 6.14](image_url) Numerical biaxial specimens in DEM investigations.
Compared to using circular discs, using clumps to represent soil particles increases the computational time. In order to balance the computational cost against reasonable simulation results, the sizes of the numerical clumps were increased with an up-scaling factor of two compared to the physical soil sizes, as shown in Figure 6.3. The size of the numerical 2D geogrid was also increased to twice of the real geogrid size so that the ratio of geogrid aperture size to soil particle size in the DEM investigations was identical to that in the laboratory tests. The 2D porosity in the numerical biaxial compression tests was \( n_{2D} = 0.16 \), which was determined iteratively according to the suggestion proposed in Chapter 3. For each biaxial specimen, the number of the clumps was 5181. Table 6.2 lists the micro input parameters of the soil particle clumps in the numerical biaxial compression tests.

### Table 6.2 Input parameters of soil particle clumps in numerical biaxial compression tests.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density of solids ( \rho_s ) [kg/m(^3)]</td>
<td>2650</td>
</tr>
<tr>
<td>Water content ( w ) [%]</td>
<td>0</td>
</tr>
<tr>
<td>Clump sizes ( d_c ) [mm]</td>
<td>Gradation as in Figure 6.3</td>
</tr>
<tr>
<td>Normal contact stiffness of walls ( k_n, w ) [N/m]</td>
<td>( 1 \times 10^6 )</td>
</tr>
<tr>
<td>Shear contact stiffness of walls ( k_s, w ) [N/m]</td>
<td>( 1 \times 10^6 )</td>
</tr>
<tr>
<td>Normal contact stiffness of clumps ( k_n, cl ) [N/m]</td>
<td>( 2 \times 10^5 )</td>
</tr>
<tr>
<td>Shear contact stiffness of clumps ( k_s, cl ) [N/m]</td>
<td>( 1 \times 10^5 )</td>
</tr>
<tr>
<td>Friction coefficient of clumps for specimen preparation ( f_0 ) [-]</td>
<td>0.04</td>
</tr>
<tr>
<td>Friction coefficient of clumps after specimen preparation ( f_{cl} ) [-]</td>
<td>5</td>
</tr>
<tr>
<td>Friction coefficient between clumps and top/left walls ( f_{wcl,1} ) [-]</td>
<td>0</td>
</tr>
<tr>
<td>Friction coefficient between clumps and bottom wall ( f_{wcl,2} ) [-]</td>
<td>0.2</td>
</tr>
<tr>
<td>Friction coefficient between clumps and membrane particles ( f_{mcl} ) [-]</td>
<td>0</td>
</tr>
</tbody>
</table>

Similar to the specimen preparation process in laboratory tests, the numerical biaxial specimen was prepared using the multilayer compaction method (eight horizontal layers). The equilibrium state for each layer was that the maximum contact force ratio fell below 0.001. After the final layer of soil had reached the equilibrium state, the horizontal confining pressure was applied via bonded membrane particles, while the vertical confining pressure was achieved by adjusting the top plate velocities using a numerical servomechanism. Table 6.3 summarizes the micro input parameters of those bonded membrane particles. Followed the suggestion of Wang and Leung (2008), high values of contact bond strengths were applied to the membrane particles. Thereby, the membrane particle string could behave like the real rubber membrane used in the laboratory tests, which was strong enough not to be torn apart but could be stretched to deform freely. After the whole specimen had reached a new equilibrium state with the required confining stress, the servomechanism of the top plate was stopped and the vertical load was applied on the top plate with a constant displacement rate of 1 mm/min. Each specimen was compressed until one of the following conditions was reached: (1) almost no increase of vertical force on the top plate due to complete failure of the numerical specimen; (2) maximum vertical displacement of 40 mm.
During the loading process, the positions of both the left and the bottom walls were fixed and the vertical pressure and displacement of the top plate were recorded. For geogrid reinforced specimens, the geogrid tensile forces at different geogrid positions were recorded as well. It should be noted that every 1 mm vertical displacement in the current DEM simulations of biaxial compression tests took around 4 hours’ computational time in reality by a PC equipped with an Intel Core i7-4771 Processor (3.50 GHz) and 16 GB memory.

Table 6.3  Input parameters of membrane boundary particles in numerical biaxial compression tests.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density of membrane particle $\rho_m$ [kg/m$^3$]</td>
<td>1000</td>
</tr>
<tr>
<td>Particle diameter $d_m$ [mm]</td>
<td>2</td>
</tr>
<tr>
<td>Normal contact stiffness of particles $k_{n,m}$ [N/m]</td>
<td>$2 \times 10^4$</td>
</tr>
<tr>
<td>Shear contact stiffness of particles $k_{s,m}$ [N/m]</td>
<td>$2 \times 10^4$</td>
</tr>
<tr>
<td>Normal contact bond strength $F_{c,n}$ [N]</td>
<td>$1 \times 10^{300}$</td>
</tr>
<tr>
<td>Shear contact bond strength $F_{c,s}$ [N]</td>
<td>$1 \times 10^{300}$</td>
</tr>
<tr>
<td>Friction coefficient between membrane particles and clumps $f_{mcl}$ [-]</td>
<td>0</td>
</tr>
</tbody>
</table>

6.3.2 DEM Simulation Results and Analyses of Unreinforced Specimens

6.3.2.1 Calibrated Stress–Strain and Volumetric Strain Behavior

Similar to the calculation process of the vertical stress reported in the laboratory tests, the total vertical stress was calculated by adding half of the specimen’s weight at middle height of each specimen, as illustrated in Equation 6.4. Since the vertical confining stress was applied on the top plate via a servomechanism, the confining stress was already included in the vertical load. Hence, the total vertical stress in the DEM investigations was calculated with the following equation.

$$\sigma_1 = \frac{F_v}{A} + \Delta \sigma_1 = \frac{F_v}{A} + \rho_{2D} g \frac{H}{2} \quad (6.4)$$

where $\sigma_1$ is the total vertical stress;

$F_v$ is the vertical load on top of the specimen, including the vertical confining stress;

$A$ is the average cross-sectional area of the specimen;

$\Delta \sigma_1$ is the additional vertical stress;

$\rho_{2D}$ is the 2D density of the numerical specimen;

$H$ is the initial height of the specimen, $H = 800$ mm in this study.
The 2D density of the numerical specimen \( (\rho_{2D}) \) can be computed with the solid density \( (\rho_s) \) and the corresponding 2D porosity \( (n_{2D}) \) as follows:

\[
\rho_{2D} = \rho_s \cdot (1 - n_{2D})
\]  

(6.5)

The numerical biaxial compression tests of unreinforced specimens were carried out under six different confining stresses (i.e. from 2.5 kPa to 25 kPa), which was identical to that in the laboratory tests. The micro input parameters of the unreinforced specimen were firstly calibrated under the confining stress of \( \sigma_3 = 2.5 \) kPa. At the initial calibration stage, several trials have been made with circular disc-shape soil particles. However, due to the lack of interlocking among those circular particles, the numerical assembly collapsed under the low confining stress \( \sigma_3 = 2.5 \) kPa, even a very high micro friction coefficient \( (f = 20) \) was used for the soil particles. In order to simulate the interlocking behavior among soil particles, 3-ball clumps were used in further calibrations. The shape of the 3-ball clumps was defined by the ratio of particle radius \( (r_c) \) to distance of particle centers \( (d_c) \) in the clump, as shown in Figure 6.3. For the initial calibrations with clumps, the ratio of \( r_c / d_c = 1 \) was chosen in this study according to the studies of Rui et al. (2015). The unreinforced specimen could be supported successfully under the low confining stress of \( \sigma_3 = 2.5 \) kPa. However, the numerically obtained internal friction angle of the assembly was still smaller than that obtained from laboratory tests even a very high micro friction coefficient \( (f = 20) \) was used for the clumps. To increase the interlocking among clumps, the ratio of \( r_c / d_c \), which is an additional parameter for clumps, was changed into 0.7 in this study. Other micro input parameters, mainly the normal and shear contact stiffnesses and friction coefficient of the clumps, were adjusted iteratively until the DEM simulation results fitted the corresponding experimental data under the confining stress of \( \sigma_3 = 2.5 \) kPa.

With the same calibrated micro input parameters listed in Table 6.2, further DEM investigations of unreinforced specimens were carried out under other confining stresses (i.e. from 5 kPa to 25 kPa). The DEM simulation results are compared with the corresponding experimental data in Figure 6.15. It should be noted that the DEM simulation results, both the vertical stress–strain curve and the volumetric strain curve, are presented until the maximum vertical stresses occur. In the post-peak phase, the numerical specimens fail much more rapidly than the physical specimens. With increasing confining stress, the differences of the vertical stress–strain relations between the DEM simulation results and the corresponding experimental data become more pronounced, as shown in Figure 6.15a. Due to the inherent limitations of 2D modeling, the volumetric strain behavior develops with a higher rate in the DEM investigations than that in the laboratory tests. However, the development tendency of the volumetric strains in the numerical modeling still show reasonable agreement with that in the experimental investigations but only at the contraction phase.

To sum up, the current DEM models with the corresponding calibrated micro input parameters in this study can represent the general mechanical behavior of the granular soil in the laboratory tests. Therefore, further DEM analyses of the unreinforced specimens and further DEM investigations of geogrid reinforced specimens are carried out based on the current DEM models with the corresponding micro input parameters.
The maximum vertical stresses under different confining stresses in both experimental and DEM investigations are summarized in Figure 6.16. The DEM simulation results show good agreement with the experimental data at relatively small confining stresses (i.e., from 2.5 kPa to 10 kPa). With increasing confining stress, the differences of maximum vertical stresses between experimental and DEM simulation results increase. The internal friction angle of the numerical material considering all confining stresses is $\phi_{BX,DEM} = 56.2^\circ$, which is approximately 4° smaller than the value obtained from the laboratory tests ($\phi_{BX,Lab} = 60.6^\circ$). The slight difference of internal friction angles might be caused by many influencing factors, such as different soil particle shapes and different boundary conditions between 2D modeling and 3D laboratory tests. In particular, the relatively large frictional resistance between soil and glass walls in the laboratory tests was not
considered in the 2D modeling. Nevertheless, the current numerical assembly can still represent the general mechanical behavior of the granular soil in the laboratory tests.

![Figure 6.16 Maximum vertical stress vs. confining stress of unreinforced specimens in laboratory tests and DEM investigations.](image)

**6.3.2.2 Particle Displacement Distributions**

In the DEM investigations, the positions of all soil particles were recorded at every 0.5 mm vertical displacement of each unreinforced specimen. By calculating the position changes of all soil particles at different vertical strains of the specimen, the displacement distributions of soil particles could be obtained. The total and horizontal displacement distributions of soil particles in the DEM investigations are shown in the right part of each figure in Figure 6.17 ($\sigma_3 = 2.5$ kPa). As a comparison, the total and horizontal displacement distributions of soil particles in the laboratory tests are also presented in Figure 6.17 (the left part of each figure). It should be noted that the DEM simulation results were obtained at a vertical strain of $\varepsilon_1 = 1$ % due to the complete failure of the numerical specimen at this compression state. The evaluated deformation behavior of the laboratory tests, however, is not pronounced at $\varepsilon_1 = 1$ % so that the results at compression state of $\varepsilon_1 = 10$ % are presented in the following figure. Therefore, only a qualitative comparison of the particle displacement distributions between the experimental and the DEM investigations is possible in this study.

The total and horizontal particle displacement distributions in the numerical modeling are quite similar to those observed in the experimental studies, i.e. sliding wedges develop from the middle top to the lateral bottoms of the specimen. Such similar kinematic behavior of the unreinforced specimens in both experimental and DEM investigations also demonstrates that the current DEM model is adequate to represent the general mechanical properties of the investigated gravel in this study.
6.3 DEM Investigations and Analyses

6.3.2.3 Qualitative Contact Force Distributions in Specimen

Figure 6.18 presents the vertical stress–strain relation curve of the unreinforced specimen in the DEM investigations with five characteristic points (A-E) under a confining stress of $\sigma_3 = 2.5$ kPa. The vertical strains of the characteristic points are 0 %, 0.25 %, 0.56 %, 0.75 % and 1 %, respectively. The qualitative contact force distributions in the specimen at those vertical strains are shown in Figure 6.19. The black lines represent the contact forces in the specimen and the thickness of the lines is proportional to the magnitude of the contact forces.

At the vertical strain of $\varepsilon_1 = 0$ % (Figure 6.19a), the contact forces between soil particles in the specimen are relatively small. An increasing contact force distribution can be observed with increasing depth of the specimen under the gravitational load and even the low confining stress. With increasing vertical load, the contact forces increase in the specimen and the orientations of the contact forces are mainly vertical, especially at the vertical strain of $\varepsilon_1 = 0.56$ % (Figure 6.19c), which corresponds to the maximum vertical load. Due to further development of shear zones in the post-peak phase, failure occurs in the specimen and the contact force decreases in the specimen and the orientations of the contact forces in the shear zone become diagonal. At the vertical strain
of $\varepsilon_1 = 1\%$ (Figure 6.19e), the contact forces between the top plate and the specimen are quite small, which visualizes the complete failure state of the numerical unreinforced specimen.

![Contact force distributions in specimen at different vertical strains](image)

**Figure 6.19** Contact force distributions in specimen at different vertical strains (thickness of lines proportional to contact force magnitude).

### 6.3.2.4 2D Porosity Distributions in Specimen

With the development of shear zones, more relative particle movements exist within the shear band than that outside of the shear band, which leads to porosity changes in the numerical specimen. In the discrete element modeling, the development of shear zones can also be visualized by the 2D porosity distributions in the numerical specimen at different vertical strains. 200 measuring circles with diameters of 40 mm have been used to record the 2D porosity distributions in the specimen (see Figure 6.14a). The 2D porosity distributions in the specimen at different vertical strains are shown in Figure 6.20. It is obvious to find that the numerical specimen was prepared uniformly at the initial state with the 2D porosity of 0.16 (Figure 6.20a). With increasing vertical load, a shear band develops in the specimen. The 2D porosity in the shear band is significantly larger than that outside of the shear band (Figure 6.20e). Similar 2D porosity distributions in biaxial compression tests have been reported by Gu et al. (2014).

![2D porosity distributions in specimen at different vertical strains](image)

**Figure 6.20** 2D porosity distributions in specimen at different vertical strains.
6.3.3 DEM Simulation Results and Analyses of Reinforced Specimens

6.3.3.1 Calibrated Stress–Strain and Volumetric Strain Behavior

The friction coefficient of the geogrid particles was set to be $f_{gg} = 0.3$, identical to the value used in the numerical modeling of geogrid pullout tests (see Chapter 5). Other micro input parameters of the 2D geogrid were summarized in Table 3.5 of Chapter 3. The calculation process of the total vertical stress of each numerical reinforced specimen was identical to that of the numerical unreinforced specimen, i.e. the total vertical stress was computed with the vertical compression load and the additional vertical stress caused by half of the specimen’s weight.

The numerical biaxial compression tests of geogrid reinforced specimens were carried out under the confining stress of $\sigma_3 = 2.5$ kPa with two types of modified geogrids (S0 and SV) based on the geogrid SG-30. S0 represents geogrids without transverse members and SV indicates geogrids with regular transverse members. Since the size of the 2D geogrid in the DEM investigations has been upscaled with a factor of two compared with that in the laboratory tests, the number of geogrid transverse members in the numerical biaxial specimen (half of the physical specimen) was six for the SV geogrid (see Figure 6.14b). The corresponding numerical simulation results with two layers of normal geogrids (SV) were compared with the experimental data based on the normal geogrid with 11 transverse members, i.e. L12-S11.

Figure 6.21 compares the DEM simulation results of both unreinforced and geogrid reinforced specimens with the corresponding experimental data. It should be noted that the DEM simulation results are presented until the maximum vertical stresses occur. Similar to the calibration results of the geogrid pullout tests reported in Chapter 5, the stress–strain behavior of the numerical modeling shows reasonable agreement with that of the laboratory tests at small vertical strains (see Figure 6.21a). Due to the inherent limitations of 2D modeling, the volumetric strain behavior develops with a higher rate in the DEM investigations than that in the laboratory tests (see Figure 6.21b). However, the development tendency of the volumetric strains in the numerical modeling still show reasonable agreement with that in the experimental investigations. Therefore, further analyses of the current DEM simulation results are carried out at small vertical strains only.
Figure 6.21 Calibration results of large-scale biaxial compression tests of reinforced specimens.

Figure 6.22 summarizes the maximum vertical stresses of the unreinforced specimen and the reinforced specimens with different numbers of transverse members based on the DEM investigations. With only two layers of longitudinal members (without transverse members), the maximum vertical stress of the reinforced specimen has been increased to approximate 250% of that of the unreinforced specimen due to the frictional resistance between geogrid longitudinal members and soil. The maximum vertical stress of the reinforced specimen with two layers of normal geogrids (SV) has been increased to more than 400% of that of the unreinforced specimen due to the frictional resistance between geogrid surface and soil as well as the bearing resistance caused by the transverse members. By comparing the increments of the maximum vertical stresses in both reinforced specimens, it is obvious that the contribution of the transverse members to the peak strength is larger than that of the longitudinal member. Such an observation agrees with the laboratory test results based on geogrids with only longitudinal members (L12-S0) and geogrids with 11 transverse members (L12-S11), as shown in Figure 6.11.

Figure 6.22 Maximum vertical stresses of unreinforced specimen and geogrid reinforced specimens with different numbers of transverse members in DEM investigations (the maximum vertical stress in each case is marked on the figure, unit: kPa).
6.3.3.2 Particle Displacement Distributions

Identical to the evaluation process of the numerical unreinforced specimen, the displacement distributions of soil particles in the numerical reinforced specimens (S0 and SV) were also obtained by calculating the position changes of all soil particles at different vertical strains. The total and horizontal displacement distributions of soil particles in the numerical reinforced specimens (S0 and SV) are shown in the right part of each figure in Figure 6.23. For the purpose of qualitative comparison, the total and horizontal displacement distributions of soil particles in the physical reinforced specimens are also presented in Figure 6.23 (the left part of each figure). It should be noted that the reported experimental results were evaluated at the vertical strain of $\varepsilon_1 = 10\%$, while the DEM simulation results were obtained at the vertical strain of $\varepsilon_1 = 1.25\%$ (S0) and $\varepsilon_1 = 2.5\%$ (SV), respectively, due to the complete failure of the numerical specimens at those compression states.

The total and horizontal particle displacement distributions of the numerical reinforced specimens in Figure 6.23 show similar kinematic behavior to the experimental observations, i.e. the shear zones do not develop from the middle top to the lateral bottoms of the specimen directly. The particle displacements, especially the horizontal particle displacements, have been greatly restricted by the two layers of geogrid reinforcements. The restriction behavior is more obvious in the specimen reinforced with SV-geogrids than that reinforced with S0-geogrids due to the additional bearing resistance of the geogrid transverse members.

**Figure 6.23** Particle displacement distributions of reinforced specimens (in each case: left – laboratory test; right – DEM investigation).

Such similar kinematic behavior of the reinforced specimens in both experimental and DEM investigations also demonstrates that the current DEM model is adequate to represent the general
mechanical behavior of the reinforced soil in this study. Therefore, further DEM analyses of the geogrid reinforced specimens are carried out based on the current DEM simulations with the corresponding micro input parameters.

6.3.3.3 Qualitative Force Distributions in Specimen and along Geogrid

Figure 6.24 shows the vertical stress–strain relation curve of the specimen reinforced with normal geogrids (SV) in the DEM investigations with eight characteristic points (A-H) under a confining stress of $\sigma_3 = 2.5$ kPa. The vertical strains of the characteristic points are $0 \%$, $0.5 \%$, $1 \%$, $1.5 \%$, $2 \%$, $2.1 \%$, $2.2 \%$ and $2.5 \%$, respectively. The qualitative contact force distributions in the specimen and the qualitative tensile force distributions along the geogrid at different vertical strains are presented in Figure 6.25. The black lines represent the contact forces in the specimen, while the red lines indicate the tensile forces within the geogrid. The thickness of the lines is proportional to the magnitude of the contact forces and the tensile forces.

Figure 6.24 Vertical stress–strain relation of reinforced specimen with six characteristic points.

At the vertical strain of $\varepsilon_1 = 0 \%$ (Figure 6.25a), the contact forces between soil particles in the reinforced specimen are relatively small. The contact forces at the lower part of the specimen are larger than those at the upper part of the specimen under the gravitational load. Due to the higher initial stress state at the lower geogrid caused by the gravitational load, the tensile forces within the lower geogrid are slightly larger than those within the upper geogrid. With increasing vertical strain, the contact forces in the specimen increase and the compression load applied on the top plate is transferred gradually from soil to the geogrids, which activates the geogrid tensile forces (see Figure 6.25b). Due to the geogrid frictional and bearing resistance, soil particles in the vicinity of both geogrid layers are restrained laterally and soil arching is developed in the specimen between two geogrid layers as well as in the specimen between the geogrid and the top/bottom plate, as shown in Figure 6.25c-f. The tensile forces within the upper geogrid are qualitatively larger than those within the lower geogrid. At the vertical strain of $\varepsilon_1 = 2.2 \%$ (Figure 6.25g), sliding failure occurs at the upper geogrid layer, which leads to the failure of the numerical reinforced specimen. However, the lower geogrid still performs well in the lower part of the specimen.
6.3.3.4 Quantitative Tensile Force and Strain Distributions along Geogrid

Figure 6.26 shows the quantitative tensile force distributions along the geogrid with two different types of modified geogrids (S0 and SV) at different vertical strains of each reinforced specimen. It should be noted that the tensile force distributions are presented until the maximum vertical stress of each reinforced specimen occurs. The tensile forces within the lower geogrid samples are larger than those within the upper geogrid at the initial state with $\varepsilon_1 = 0\%$. With increasing vertical strain of each specimen, the tensile forces along the geogrid increase and the tensile forces within the upper geogrid samples are larger than those within the lower geogrid samples due to the applied vertical load. Such quantitative results agree with the qualitative observations in Figure 6.25. The maximum tensile forces occur at the symmetric boundary of the geogrid samples (S0 and SV). For the S0-geogrid, the tensile forces reduce gradually from the symmetric boundary to the right end of the geogrid due to the frictional resistance between the geogrid longitudinal
member and soil. For the SV-geogrid, however, several jumps of the tensile forces are observed at the positions of geogrid transverse members, especially at large vertical strains of the specimen. Such jumps are caused by the bearing resistance in front of the geogrid transverse members.

![Graphs showing geogrid tensile force distributions](image)

**Figure 6.26** Geogrid tensile force distributions at different vertical strains in DEM investigations.

Based on the above geogrid tensile force distributions as well as the nonlinear tensile force–strain relations in Figure 3.26 (Chapter 3), it is easily possible to obtain the geogrid tensile strain distributions along the geogrid, as shown in Figure 6.27. The shapes of the tensile strain distribution curves are almost identical to those of the tensile force distribution curves in Figure 6.26. Moreover, the corresponding experimental data obtained with the strain gauges at different vertical strains of each specimen are also plotted in Figure 6.27. Due to the lack of real angularity of numerical soil particles and the simplification of real 3D geogrids into 2D modeling, the numerically obtained tensile strains of geogrids are smaller than the corresponding experimental results. Nevertheless, the tendency of the geogrid tensile strain distributions in the numerical modeling still shows reasonable agreement with that of the laboratory test results.
6.4 Limitations

The numerical modeling of the biaxial compression tests in this study was carried out using 2D software, which has inherent limitations in investigating the real 3D problems of the geogrid–soil interaction, even though the laboratory tests were conducted under plane strain conditions. The frictional resistance between soil and glass walls in the laboratory tests, which led to a high internal friction angle of the assembly, was difficult to simulate directly in the 2D modeling. Moreover, the real 3D geogrid in this study was simplified into a 2D model, which consists of a string of bonded particles and relative high knot particles at the corresponding positions to simulate the contributions of geogrid transverse members. The granular soil was modeled with clumps by overlapping three circular discs to form an irregular shape. However, such irregular shape was identical for all soil particles and it was still different from the various irregular shapes of real soil particles. Moreover, in order to balance the computational cost against the scaling effect, an up-scaling factor of two has been used for the soil and geogrids in this study. Although the above simplifications of real 3D problems into 2D modeling surely lead to differences between the experimental and numerical investigations, the 2D discrete element modeling still provide satisfactory results to illustrate and to visualize the fundamental geogrid reinforcement mechanisms under biaxial compression loads within reasonable computational time.
6.5 Summary

This chapter focuses on the geogrid–soil interaction under biaxial compression loads with both experimental and numerical approaches. In the laboratory tests, geogrids with different numbers of geogrid longitudinal and transverse members have been used. In the DEM investigations, the models and the micro input parameters of the geogrid and the granular soil have been calibrated based on geogrid tensile tests and biaxial compression tests of unreinforced specimens, respectively. The numerical biaxial compression tests of reinforced specimens were carried out with geogrids modified with different numbers of transverse members.

In the laboratory tests, the compound stress–strain behavior of reinforced soil have been significantly improved with increasing number of geogrid longitudinal and transverse members. The maximum vertical stress of the biaxial specimen increased with increasing number of geogrid longitudinal and transverse members. The geogrid reinforcement effects can be fully activated with the combination of geogrid longitudinal and transverse members. Based on the Digital Image Correlation (DIC) method, the kinematic behavior of both unreinforced and reinforced specimens has been visualized with particle displacement distributions and particle rotations. The visualization results demonstrated that the geogrid reinforcements could greatly reduce the lateral displacements of soil particles. Moreover, using the attached strain gauges on the geogrid samples, the maximum tensile strain has been observed at the central symmetry axis along the geogrid and its value decreased from the center to both free ends of the geogrid.

In the DEM investigations, similar compound stress–strain behavior and similar kinematic behavior of both unreinforced and reinforced specimens to the experimental results have been obtained. Additionally, the load transfer behavior between soil and geogrid under biaxial compression loads has been visualized not only by the qualitative force distributions in the specimen and along the geogrid but also by the quantitative tensile force and tensile strain distributions along the geogrid. The tendency of the numerically obtained geogrid tensile strain distributions showed reasonable agreement with that of the corresponding experimental data, which demonstrates that the current DEM models can satisfactorily reproduce the geogrid–soil interaction under biaxial compression loads, especially at small vertical strains of the reinforced specimens.
7 Application of DEM Investigations for Real Geogrid Reinforced Soil Structures

7.1 Introduction

In the previous chapters, geogrid reinforcement mechanisms have been investigated under single experimental loading conditions, e.g. under pullout loads or under biaxial compression loads. However, the geogrids are experiencing combined loading conditions in real reinforced soil structures. Therefore, based on the previously developed DEM models of this study, this chapter aims to investigate the geogrid reinforcement mechanisms in real geogrid reinforced soil structures under combined loading conditions (i.e. in reinforced base courses or in reinforced slopes loaded with surface strip foundations).

In the numerical modeling of unreinforced and reinforced base courses, the numerically obtained foundation pressure–settlement relations are validated by experimental results from literature. Based on the reasonable numerical modeling, the geogrid reinforcing effects are evaluated by the responses of soil and geogrids, e.g. displacement distributions of soil particles, vertical deformation behavior of geogrids and qualitative contact force distributions in the base courses as well as quantitative tensile force distributions along the geogrids.

In the numerical modeling of unreinforced and reinforced slopes, identical models with the same micro input parameters to the DEM investigations of the base courses are used. The numerically obtained settlement curves of the foundations located on top of the unreinforced and reinforced slopes are presented and compared in this study. The geogrid reinforcement mechanisms are visualized by the qualitative displacement distributions of soil particles and the qualitative contact force distributions in the slopes. Moreover, the geogrid reinforcing effects are also described by the quantitative vertical displacements of the geogrids and the quantitative tensile force distributions along the geogrids.

7.2 DEM Investigations of Reinforced Base Courses

7.2.1 Experimental Background

The experimental results of unreinforced and reinforced base courses under surface strip foundation loads reported by Das et al. (1994) are used to validate the numerical modeling of this study. It should be noted that those experimental results have also been used by Tran et al. (2015) to validate their coupled finite-discrete element framework. For the presented laboratory tests, the inside dimensions of the testing box were $L/W/H = 1100/304.8/910$ (unit: mm). In order to reduce
the friction between soil and walls, the internal wall surfaces were polished as much as possible. The strip foundation was made out of aluminum plates and it had a length of 76.2 mm (noted as \( B \)) and a width of 304.8 mm. Since the width of the strip foundation was equal to the inside width of the testing box, a plane strain condition was generally maintained (Khing et al., 1993). The main experimental parameters of sand and geogrids are listed in Table 7.1. More information about the sand and geogrid properties can be found in the literature (Das et al., 1994).

<table>
<thead>
<tr>
<th>Medium-grained silica sand</th>
<th></th>
</tr>
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<tbody>
<tr>
<td>Average dry unit weight ( \gamma_d ) [kN/m(^3)]</td>
<td>17.14</td>
</tr>
<tr>
<td>Particle sizes ( d ) [mm]</td>
<td>0.2–0.85, ( d_{50} = 0.48 )</td>
</tr>
<tr>
<td>Average relative density ( D_r ) [%]</td>
<td>70</td>
</tr>
<tr>
<td>Internal friction angle ( \phi ) [°] (from direct shear tests)</td>
<td>41</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Biaxial geogrid (Tensar BX1000)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure</td>
<td>Punched sheet drawn</td>
</tr>
<tr>
<td>Polymer</td>
<td>PP/HDPE copolymer</td>
</tr>
<tr>
<td>Aperture size MD/XMD [mm]</td>
<td>25.4/33.02</td>
</tr>
<tr>
<td>Nominal rib thickness [mm]</td>
<td>0.762</td>
</tr>
<tr>
<td>Nominal junction thickness [mm]</td>
<td>2.286</td>
</tr>
<tr>
<td>Tensile stiffness at 2 % strain [kN/m]*</td>
<td>182</td>
</tr>
</tbody>
</table>

* Such a parameter was provided by Tran et al. (2015).

The unreinforced and reinforced base courses were prepared with multilayers and each layer had a thickness of 25 mm. In the reinforced cases, the top geogrid layer was placed at a depth of approximate 25 mm below the soil surface. For the case reinforced with multiple geogrid layers, the vertical distance between two adjacent geogrid layers was also 25 mm. The length of each geogrid layer was 762 mm (10\( B \)). After the final soil layer had been prepared in each base course, a strip foundation was placed on the soil surface. The applied vertical load and the corresponding foundation settlement were measured with a proving ring and two dial gauges, respectively.

### 7.2.2 Numerical Modeling

In this section, discrete element modeling using PFC\(^2\)D has been carried out to gain further detailed insights into the geogrid–soil interaction under surface strip foundation loads. Using the linear contact stiffness model and slip model, the soil was modeled with clumps, which had identical shape, the same particle size distribution and 2D porosity as those clumps used in Chapter 6. Based on the developed piecewise linear model, the geogrid was modeled with bonded particles, which had the same shape and aperture size as the numerical geogrid used in Chapter 6. It should be noted that the sizes of the numerical soil and geogrids in this section were different from those
reported in the laboratory tests (Das et al., 1994). Nevertheless, a qualitatively illustration of the geogrid reinforcement mechanisms in the reinforced base courses at a microscopic scale can still be obtained. In order to obtain the DEM simulation results within reasonable computational time, it is possible to simplify the numerical specimen into half of the physical specimen since the strip foundation was loaded symmetrically on the base course. Figure 7.1 shows the sketch of the reinforced base course loaded with a strip foundation as well as the corresponding DEM model, in which the strip foundation was modeled with bonded particles and the vertical loads were able to be applied on the bonded particles directly. The width of the strip foundation in the numerical analysis was 75 mm \((B/2)\). The length and height of the numerical base course were 500 mm and 600 mm, respectively, which took account of the size of the influencing zone loaded with a strip foundation according to Tran et al. (2015) using finite-discrete element modeling (F-DEM).

![Figure 7.1 Sketch of the geogrid reinforced base course loaded with a strip foundation and the corresponding DEM model.](image)

The DEM investigations of this study focused on three cases: unreinforced \((N = 0)\), reinforced with one geogrid layer \((N = 1)\) and reinforced with two geogrid layers \((N = 2)\). Each numerical specimen was prepared with seven layers (five bottom layers and two top layers), in which the thickness of each bottom layer was 100 mm and the thickness of each top layer was 50 mm. Thereby, for the reinforced cases of \(N = 1\) and \(N = 2\), the geogrid layers with the lengths of 404 mm were able to be placed at corresponding positions, as shown in Figure 7.1. The equilibrium state of each soil layer was that the maximum contact force ratio reached 0.001. After the final soil layer had been prepared in the base course, 40 bonded particles were generated on the soil surface to model the strip foundation. An increase of vertical pressure in load steps of 10 kPa was applied to the bonded particles directly until failure of the numerical specimens.
reached. For each loading step, the equilibrium state was that the ratio of the maximum unbalanced force to the maximum contact force in the specimen fell below 0.001. During the loading process, the applied vertical pressure and the resulting foundation settlement at the equilibrium state of each loading step were recorded. For the geogrid reinforced specimens, the tensile forces and vertical displacements along the geogrids were recorded as well.

7.2.3 Validation of Numerical Simulation Results

Due to the difference between the reported soil from the literature (Das et al., 1994) and the investigated granular soil in Chapter 6, the micro input parameters of the soil clumps in Chapter 6 cannot be applied to the clumps used in this section directly. Hence, in order to provide relatively satisfactory DEM simulation results compared with the reported experimental data, the micro input parameters of the clumps are calibrated briefly in this section. Although the normal and shear contact stiffness are increased 2.5 times as those used in Chapter 6, the ratio of normal to shear contact stiffness is constant in this study. Since the internal friction angle of the reported soil from the literature (41°) is much smaller than that of the granular soil obtained from biaxial compression tests in Chapter 6 (60.6°), the friction coefficient of clumps in this chapter is reduced to 0.4. The micro input parameters of boundary walls and geogrids are identical to those used in Chapter 6. With the final input parameters listed in Table 7.2 and identical micro input parameters of the geogrids used in Chapter 6, the numerically obtained foundation pressure–settlement relations are compared with the experimental results in Figure 7.2.

For the unreinforced case (N = 0) and the reinforced case with one geogrid layer (N = 1), the DEM simulation results show good agreement with the corresponding laboratory test results. For the reinforced case with two geogrid layers (N = 2), however, the bearing capacity of the numerical specimen is about twice of that reported in the literature. The point plotted with dotted lines in the numerical modeling of N = 2 indicates that the defined equilibrium state of the numerical specimen
was not reached under the foundation pressure of 440 kPa. It would take a long computational time to obtain the defined equilibrium state. Nevertheless, sharp settlement has been observed under the current loading state, which represents the failure of the reinforced soil structure. The ultimate bearing capacity of the base course increases with increasing number of geogrid layers. The tendency of the numerical observations agrees with that of the experimental investigation results. Hence, the current DEM models are regarded to be reasonable to reproduce the general interaction between geogrid and soil in such practical loading conditions. Further DEM analyses are carried out to describe and to visualize the geogrid reinforcement effects.

<table>
<thead>
<tr>
<th>Table 7.2</th>
<th>Input parameters of soil particle clumps, foundation particles and boundary walls.</th>
</tr>
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<tbody>
<tr>
<td><strong>Soil particle clumps</strong></td>
<td></td>
</tr>
<tr>
<td>Density of solids $\rho_s$ [kg/m$^3$]</td>
<td>2650</td>
</tr>
<tr>
<td>Water content $w$ [%]</td>
<td>0</td>
</tr>
<tr>
<td>Clump sizes $d_{cl}$ [mm]</td>
<td>Gradation as in Figure 6.3</td>
</tr>
<tr>
<td>Normal contact stiffness of clumps $k_{n,cl}$ [N/m]</td>
<td>$5\times10^5$</td>
</tr>
<tr>
<td>Shear contact stiffness of clumps $k_{s,cl}$ [N/m]</td>
<td>$2.5\times10^5$</td>
</tr>
<tr>
<td>Friction coefficient of clumps for specimen preparation $f_0$ [-]</td>
<td>0.04</td>
</tr>
<tr>
<td>Friction coefficient of clumps after specimen preparation $f_{cl}$ [-]</td>
<td>0.4</td>
</tr>
</tbody>
</table>

| **Foundation particles**                     |                                                                                  |
| Density of foundation particles $\rho_f$ [kg/m$^3$] | 2650                                                                              |
| Particle diameter $d_f$ [mm]                  | 3.75                                                                              |
| Normal contact stiffness of foundation particles $k_{n,f}$ [N/m] | $1\times10^6$                                                                 |
| Shear contact stiffness of foundation particles $k_{s,f}$ [N/m] | $1\times10^6$                                                                    |
| Normal contact bond strength $F_{c,n}^b$ [N]   | $1\times10^{300}$                                                                |
| Shear contact bond strength $F_{c,s}^b$ [N]    | $1\times10^{300}$                                                                |
| Friction coefficient between foundation particles and clumps $f_{fcl}$ [-] | 0.2                                                                            |

| **Boundary walls**                           |                                                                                  |
| Normal contact stiffness of walls $k_{n,w}$ [N/m] | $1\times10^6$                                                                     |
| Shear contact stiffness of walls $k_{s,w}$ [N/m] | $1\times10^6$                                                                    |
| Friction coefficient between walls and clumps $f_{wcl}$ [-] | 0                              |

### 7.2.4 Force Distributions in Specimen and along Geogrid

Figure 7.3 shows the qualitative contact force distributions in the specimens and the qualitative tensile force distributions along the geogrids in the unreinforced and reinforced base courses under a foundation pressure of 80 kPa, which was the ultimate bearing capacity of the unreinforced base course. The black lines represent the contact forces within the specimens, while the red lines indicate the tensile forces within the geogrids. The thickness of the lines is proportional to the
magnitude of the contact forces and the tensile forces. Large contact forces developed beneath the strip foundation. In the unreinforced case \((N = 0)\), the contact forces develop diagonally from the foundation base. In the reinforced cases \((N = 1, \text{ and } N = 2)\), however, due to the activation of geogrid tensile forces, the contact forces between the strip foundation and the geogrid layer(s) are mainly vertical. Below the geogrid \((N = 1)\) or below the lower geogrid \((N = 2)\), the orientations of the contact forces are similar to those observed in the unreinforced case. The DEM visualization results of this study are in accordance with the F-DEM investigation results obtained by Tran et al. (2015).

\[ p = 80 \text{ kPa} \]

\[ p = 80 \text{ kPa} \]

\[ p = 80 \text{ kPa} \]

**Figure 7.3** Contact force distributions in specimens and tensile force distributions along geogrids in unreinforced and reinforced base courses at the same loading state (thickness of lines proportional to magnitude: red – tensile forces along the geogrid; black – contact forces in the specimen).

Besides the qualitative force distributions in Figures 7.3b and 7.3c, quantitative tensile force distributions along the geogrids in the reinforced cases are also obtained, as shown in Figure 7.4. For each geogrid layer, the maximum tensile force is found below the strip foundation near the symmetric boundary of the geogrid and the value decreases from the symmetric boundary to the right free end of the geogrid. The maximum tensile force in the geogrid of \(N = 1\) is larger than that in each geogrid of \(N = 2\). For the reinforced case of \(N = 2\), the tensile forces within the lower geogrid are larger than those within the upper geogrid, especially outside the range of the strip foundation. Such DEM simulation results can be used to explain the load spread mechanism in reinforced soil structures with multiple geogrid layers.
7.2 DEM Investigations of Reinforced Base Courses

Figure 7.4  Quantitative tensile force distributions along geogrid in reinforced base courses.

7.2.5 Displacement Distributions of Soil Particles and Geogrids

Figure 7.5 shows the qualitative displacement distributions of soil particles with and without geogrid reinforcement under a foundation pressure of 80 kPa. It can be seen from Figure 7.5a that a general shear failure mode occurs in the base course and a failure surface develops with a depth and a width of 1.5B and 3.1B, respectively. Those observations are similar to the DEM simulation results obtained by Tran et al. (2015). In the reinforced cases (N = 1, and N = 2), due to the lateral restraint mechanisms of geogrids, the displacements of soil particles are much smaller than those in the case of N = 0. Moreover, because of the load transfer from soil to geogrids, the applied vertical loads are spread and thereby the values of the maximum stresses decrease below the geogrids, which again leads to smaller settlement of the soil particles. By comparing the particle displacements in both reinforced cases, the displacements of soil particles in N = 2 are smaller than those in N = 1.
Figure 7.5 Displacement distributions of soil particles in unreinforced and reinforced base courses at the same loading state (lengths of vector lines proportional to magnitude of particle displacements).

Figure 7.6 presents the quantitative vertical displacements of the geogrid in \( N = 1 \) and the upper geogrid in \( N = 2 \). Both geogrid layers are located at the same depth of the base courses, i.e. 50 mm below the foundation base. For the purpose of comparison, the vertical displacements of soil particles in a horizontal section of the unreinforced base course are also plotted in Figure 7.6. The depth of the soil horizontal section is identical to the depths of the geogrid in \( N = 1 \) and the upper geogrid in \( N = 2 \). The vertical displacements of the soil particles in the horizontal section beneath the strip foundation are much larger than the vertical displacements along the geogrids. Outside the range of the strip foundation, the soil particles have been forced aside and the vertical displacements of soil particles in the horizontal section are also larger than the vertical displacements along the geogrids but in an opposite direction (upwards). By comparing the vertical displacements along the geogrids in both reinforced cases, the vertical displacement of the geogrid in \( N = 1 \) is generally larger than that in \( N = 2 \).
7.2 DEM Investigations of Reinforced Base Courses

With increasing vertical load of the strip foundation on reinforced base courses, rupture might occur along the geogrid, which leads to failure of the reinforced structures. Figure 7.7a shows the displacement distributions of soil particles at a failure state in the reinforced case of \( N = 1 \). The corresponding ultimate bearing capacity of the reinforced structure is 140 kPa. As a comparison, the displacements of soil particles in \( N = 2 \) loaded with 140 kPa is shown in Figure 7.7b. Due to the failure of geogrid reinforcement, large displacements of soil particles can be observed even below the geogrid, as shown in Figure 7.7a. With two geogrid layers, however, the displacements of soil particles are still very small under the same foundation pressure (Figure 7.7b).

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**Figure 7.6** Quantitative vertical displacement distributions along soil section/geogrid in unreinforced and reinforced base courses.

**Figure 7.7** Displacement distributions of soil particles in reinforced cases loaded with the same pressure (lengths of vector lines proportional to magnitude of particle displacements).
7.3 DEM Investigations of Reinforced Slopes

7.3.1 Numerical Modeling

In this section, discrete element modeling using PFC\textsuperscript{2D} has been conducted to investigate geogrid reinforcement mechanisms in soil slopes, which were loaded with surface strip foundations. It should be noted that the purpose of the numerical modeling in this section was to qualitatively describe the geogrid reinforcement mechanisms in a reinforced soil slope at a microscopic scale. Hence, the DEM simulation results were not quantitatively compared with any laboratory test results. The models and micro input parameters of soil particle clumps, geogrids, foundation particles and boundary walls in the DEM investigations of this section are identical to those used in Section 7.2. Figure 7.8 shows the sketch of the reinforced soil slope and the corresponding DEM model loaded with a surface strip foundation. The width of the numerical strip foundation was 75 mm (noted as $B$), which was equal to the distance between strip foundation and edge of the slope crest. Other configuration parameters of the reinforced soil slope can be seen in Figure 7.8a.

![Figure 7.8](image-url)
The DEM investigations of this study focused on two cases: unreinforced \( (N = 0) \) and reinforced with two geogrid layers \( (N = 2) \) with the length of each geogrid of 416 mm. Each numerical specimen was prepared with multilayers and the equilibrium state of each soil layer was that the ratio of the maximum unbalanced force to the maximum contact force in the specimen fell below 0.001. For the reinforced case, the upper and lower geogrids were placed at 50 mm and 150 mm below the foundation base, respectively, as shown in Figure 7.8. After the final soil layer had been prepared in the slope, 40 bonded particles were generated on the top surface of the slope to model the strip foundation. An increase of vertical pressure in load steps of 2 kPa was applied to the bonded particles directly until failure of the numerical specimens reached. After the foundation pressure for the reinforced case had reached 50 kPa, the increment of the vertical pressure was increased to 5 kPa. For each loading step, the equilibrium state was also that the ratio of the maximum unbalanced force to the maximum contact force in the specimen fell below 0.001. During the loading process, the applied vertical pressure and the resulting foundation settlement at the equilibrium state of each loading step were recorded. For the geogrid reinforced specimen, the tensile forces and vertical displacements along the geogrids were recorded as well.

7.3.2 Numerical Simulation Results

Figure 7.9 shows the numerically obtained pressure–settlement relations of the strip foundations on the soil slopes with and without geogrid reinforcement. Compared with the unreinforced case, the bearing capacity of the strip foundation with two layers of geogrids increases remarkably, which is approximately seven times of that for the unreinforced case. Such significant increase of the bearing capacity demonstrates the reinforcing effects of geogrids on soil slopes. The point plotted with dotted lines in the reinforced case indicates that the defined equilibrium state of the numerical reinforced slope was not reached under the foundation pressure of 130 kPa. It would take a long computational time to obtain the defined equilibrium state. Nevertheless, sharp settlement has been observed under the current loading state, which represents the failure of the reinforced soil slope. Based on the current numerical modeling, further DEM analyses are carried out in this section to provide detailed geogrid–soil interaction at a microscopic scale.
7.3.3 Force Distributions in Specimen and along Geogrid

Figure 7.10 shows the qualitative contact force distributions in the specimens and the qualitative tensile force distributions along the geogrids in the unreinforced and reinforced soil slopes under a foundation pressure of 18 kPa, which was the ultimate bearing capacity of the unreinforced slope. The black lines represent the contact forces within the specimens, while the red lines indicate the tensile forces within the geogrids. The thickness of the lines is proportional to the magnitude of the contact forces and the tensile forces. Similar to the observations in the DEM investigations of base courses, large contact forces developed beneath the strip foundation. In the unreinforced case (N = 0), the contact forces do not symmetrically distribute along the vertical center of the strip foundation; instead, the contact forces in the right part are smaller than those in the left part as a sliding surface develops in the unreinforced slope. In the reinforced case (N = 2), due to the activation of tensile forces within the geogrids, the contact forces beneath the strip foundation distribute almost symmetrically along the vertical center of the strip foundation. Moreover, the activated geogrids help to widen the contact force distribution in the reinforced slope. Those DEM simulation results visualize the load spreading due to the load transfer between soil and geogrids.
Figure 7.10 Contact force distributions in specimens and tensile force distributions along geogrids in soil slopes without and with geogrid reinforcement at the same loading state (thickness of lines proportional to magnitude: red — tensile forces along the geogrid; black — contact forces in the specimen).

Besides the above qualitative force distributions in Figure 7.10b, quantitative tensile force distributions along the geogrids are also obtained, as shown in Figure 7.11. For each geogrid layer, the maximum tensile force within the geogrid is found below the strip foundation and the value decreases from the vertical center of the strip foundation to both ends of the geogrids. Beneath the strip foundation, the tensile forces within the upper geogrid are larger than those within the lower geogrid. However, outside the range of the strip foundation, the tensile forces within the lower geogrid are larger than those within the upper geogrid, which indicates that those geogrid segments in the lower geogrid are activated to widen the vertical stress distribution in the reinforced slope.

Figure 7.11 Quantitative tensile force distributions along geogrid in the reinforced slope.
7.3.4 **Displacement Distributions of Soil Particles and Geogrids**

Figure 7.12 presents the qualitative displacement distributions of soil particles in the unreinforced and reinforced soil slopes under a foundation pressure of 18 kPa. It can be seen from Figure 7.12a that a general shear failure mode occurs in the unreinforced slope \((N = 0)\) and a sliding surface develops from the strip foundation to the toe of the unreinforced slope. Such DEM simulation results are similar to the finite element modeling results of the unreinforced slope by Keskin and Laman (2014). In the reinforced slope \((N = 2)\), the displacements of soil particles are restrained by the geogrids and thus they are much smaller than those in the case of \(N = 0\). Moreover, due to the load spread in the reinforced soil slope, no real slip surface develops in the reinforced structure. Besides several relatively large displacements of soil particles along the slope surface, relatively large displacements of soil particles are only observed beneath the strip foundation and above the upper geogrid, as shown in Figure 7.12b.

![Figure 7.12 Displacement distributions of soil particles in unreinforced and reinforced slopes at the same loading state (lengths of vector lines proportional to magnitude of particle displacements).](image)

Figure 7.13 shows the quantitative vertical displacements of the geogrids in the reinforced slope under a foundation pressure of 18 kPa. As a comparison, the vertical displacements of soil particles in horizontal sections of the unreinforced slope are also plotted in Figure 7.13. The depths of the soil horizontal sections in the unreinforced slope are identical to the depths of geogrids in the reinforced slope. Beneath the strip foundation, the maximum vertical displacement occurs at the upper soil section, followed by the upper geogrid layer. The vertical displacements of the lower soil section and the lower geogrid are the smallest beneath the strip foundation. Outside the range of the strip foundation, the soil particles in the upper section have been forced aside, which leads to the movement of soil particles in an opposite direction compared with those soil particles beneath the strip foundation. Due to the development of the sliding surface in the unreinforced slope, large vertical displacements of the soil particles in the lower section are observed near the toe of the slope. The vertical displacements of both upper and lower geogrids outside the range of the strip foundation are quite small.
7.3 DEM Investigations of Reinforced Slopes

With increasing vertical load of the strip foundation on the reinforced slope, rupture might occur along the geogrid, which leads to failure of the reinforced structure. Figure 7.14 shows the displacement distributions of soil particles with an ultimate loading pressure of 130 kPa. Beneath the strip foundation, large vertical displacements of soil particles can be observed above and below the upper geogrid, which represents a punching shear failure directly beneath the strip foundation. Below the lower geogrid, large displacements of soil particles are observed, which indicates a general shear failure of the slope. According to the main directions of the displacement vectors, the bottom part of the base can be divided into three parts, as shown in Figure 7.14. Such DEM observations show good agreement with the bearing capacity failure mode of the geogrid reinforced structures reported by Ziegler (2012).

![Figure 7.13](image1.png)  
**Figure 7.13** Quantitative vertical displacement distributions along soil layer/geogrid in unreinforced and reinforced slopes.

![Figure 7.14](image2.png)  
**Figure 7.14** Displacement distributions of soil particles at failure state in the reinforced slope (lengths of vector lines proportional to magnitude of particle displacements, maximum displacement = 44 mm).
7.4 Summary

This chapter presents the geogrid reinforcement mechanisms in real reinforced soil structures (i.e. reinforced base courses and reinforced slopes), in which the geogrids are experiencing combined loading conditions. All the DEM investigations were carried out under surface strip foundation loads.

In the numerical modeling of unreinforced and reinforced base courses, the numerically obtained foundation pressure–settlement relations have been validated by the experimental results from the literature. With increasing number of geogrid layers, the bearing capacity of the soil structures has been greatly increased. The geogrid reinforcement mechanisms have been visualized not only by the qualitative force distributions in the specimens and along the geogrids but also by the qualitative displacement distributions of soil particles. Moreover, the quantitative tensile force and vertical displacement distributions along the geogrids also indicate the geogrid reinforcing effects in such practical reinforced soil structures.

In the numerical modeling of unreinforced and reinforced slopes, the bearing capacity of the reinforced slope has been significantly increased due to the contribution of geogrid reinforcement. The load transfer behavior between soil and geogrid has been visualized by the qualitative contact force distributions in the specimens as well as the qualitative and quantitative tensile force distributions along the geogrids. Moreover, the displacement distributions of soil particles in both unreinforced and reinforced slopes have been obtained at different ultimate loading states, which visualizes the different failure modes in unreinforced and reinforced slopes.

Based on the reasonable numerical investigation results in this chapter, it is concluded that the developed DEM models in this study can be successfully used to describe and to visualize the geogrid reinforcement mechanisms in real geogrid reinforced soil structures.
8 Conclusions and Recommendations for Future Research

8.1 Conclusions

As an important geosynthetic material, geogrids have been widely used in practice to reinforce various soil structures. Geogrid reinforcing effects are performed via the interaction of geogrids together with the surrounding soil. In order to improve the understanding of geogrid reinforcement mechanisms from a microscopic perspective, discrete element modeling using PFC2D was carried out in this study. Based on the developed models of soil and geogrids, this work focused on the geogrid–soil interaction under different experimental loading conditions, i.e. compound tensile loads, pullout loads and biaxial compression loads. Moreover, the developed DEM models were also used to investigate the geogrid reinforcement mechanisms in real geogrid reinforced soil structures loaded with surface strip foundations. The main conclusions of this study can be summarized from the following three aspects.

8.1.1 Discrete Element Modeling of Soil

Using the linear contact stiffness model and slip model provided by PFC2D, soil was modeled with unbonded particles in this study. Selecting a reasonable 2D porosity is the foremost step for all 2D DEM investigations. Thus, current approaches for converting porosities from 3D to 2D for DEM studies were summarized and evaluated theoretically in this study. All the current approaches tried to link the 2D and 3D porosities just using specific equations, which might be unsuitable for arbitrary assemblies with various polydisperse particle systems. Hence, a new iterative approach was formulated for determining 2D porosities in this study. The new suggestion was proven to be rational with the realistic contact force distribution in the numerical specimen. Moreover, based on reasonable 2D porosities, the numerical simulation results of direct shear tests and unreinforced biaxial compression tests with different soil samples showed good agreement with the corresponding experimental data by calibrating other micro input parameters. Therefore, the new approach can be used for the determination of 2D porosities in a wide range of polydisperse particle systems.

In the numerical modeling of unreinforced biaxial compression tests, circular disc-shaped soil particles even with a very high micro friction coefficient ($f = 20$) failed to meet the high internal friction angle of the biaxial specimens. The technique of 3-ball clumps with reasonable clump shapes could successfully solve the problem. Hence, it is concluded that besides increasing micro frictional coefficients of soil particles, clumping circular particles to form reasonable shapes provides an alternative way to increase the internal friction angle of the numerical assembly.
8.1.2 Discrete Element Modeling of Geogrids

In this study, the real 3D geogrid was simplified into a 2D model, in which the geogrid longitudinal member was simulated with one row of bonded particles and the knots were simulated with five rows of bonded particles on the longitudinal member and four rows of bonded particles beneath the longitudinal member. Hence, the bearing resistance caused by the transverse members could be transferred to the longitudinal member via the knot particles in the 2D DEM investigations.

Based on the parallel bond model provided by PFC$^{2D}$, a piecewise linear model was developed in this study to represent the nonlinear tensile behavior of geogrids. The calibration results of the numerical tensile tests showed good agreement with the corresponding experimental data, which demonstrated that the developed model and the corresponding micro input parameters in the DEM simulations could satisfactorily represent the geogrid properties. Therefore, the developed piecewise linear model for the geogrid can be used in further DEM investigations of geogrid–soil interaction under different loading conditions.

8.1.3 Discrete Element Modeling of Geogrid–Soil Interaction

The frictional interaction between only one geogrid tensile member and soil was investigated with a numerical compound tensile test. The load transfer behavior between the geogrid tensile member and soil was visualized by the contact force distributions in the specimen, the tensile force distributions along the geogrid tensile member and the rotations of soil particles in the vicinity of the geogrid tensile member at different clamp displacements. Moreover, the numerically obtained geogrid tensile strain distributions showed good agreement with the experimental results. Such reasonable results lead to the conclusion that PFC$^{2D}$ can be used as a practical tool to investigate the interaction between the geogrid tensile member and soil.

In order to validate the numerical modeling of geogrid pullout tests, experimental pullout tests of geogrids embedded in granular soil were performed in this study. Besides geogrid pullout failure, two other different failure modes were observed based on geogrids modified with different numbers of transverse members, i.e. failure of welded junctions and failure of geogrid longitudinal members. The failure of welded junctions occurred only when the tested geogrids were modified with fewer transverse members, i.e. quite large spacing between adjacent transverse members.

In the discrete element modeling, the geogrid–soil interaction under pullout loads was visualized by the force distributions along the geogrids and in the specimens with different numbers of transverse members. The numerically obtained force, displacement and strain distributions along the geogrid with different numbers of transverse members also illustrated the load transfer behavior between geogrid and granular soil. The average contributions of geogrid transverse members decreased with increasing number of geogrid transverse members, which could be used to explain the different failure modes observed in the laboratory pullout tests. Based on the Fourier Series Approximation (FSA) method, the reorientations of contacts and forces in the specimen as well as the normal stress distributions in the geogrid plane were obtained at different clamp displacements. The principal contact direction in the SV-test was changed from nearly vertical
(θ ≈ 90°) at the initial state to diagonal (θ ≈ 128°) at the clamp displacement of \( u_{\text{Clamp}} = 2 \text{ mm} \). The normal stress distribution in the geogrid plane was not constant and the maximum normal stress near the loaded end was up to six times of the required normal stress at the clamp displacement of \( u_{\text{Clamp}} = 5 \text{ mm} \). The DEM simulation results provide researchers more detailed insights into the geogrid–soil interaction at a microscopic scale under pullout loads.

Moreover, the geogrid–soil interaction under biaxial compression loads was also investigated experimentally and numerically. In the laboratory tests, the compound stress–strain behavior of geogrid reinforced soil was significantly improved with increasing number of geogrid longitudinal and transverse members. The maximum vertical stress of the biaxial specimen increased with increasing number of geogrid longitudinal and transverse members. Based on the Digital Image Correlation (DIC) method, the kinematic behavior of both unreinforced and reinforced specimens was visualized with particle displacement distributions and particle rotations. The visualization results demonstrated that the geogrid reinforcement greatly reduced the lateral displacements of soil particles. Using the attached strain gauges on the geogrid samples, the maximum tensile strain was observed at the central symmetry axis along the geogrid and its value decreased from the center to both free ends of the geogrid.

In the DEM investigations, similar compound stress–strain behavior and similar kinematic behavior of both unreinforced and reinforced specimens to the experimental results were obtained. The load transfer behavior between soil and geogrid under biaxial compression loads was visualized not only by the qualitative force distributions in the specimen and along the geogrid but also by the quantitative tensile force and tensile strain distributions along the geogrid. The tendency of the numerically obtained geogrid tensile strain distributions showed reasonable agreement with that of the corresponding experimental data, which demonstrates that the current DEM models can satisfactorily reproduce the geogrid–soil interaction under biaxial compression loads, especially at small vertical strains of the reinforced specimens.

Furthermore, the developed DEM models were applied in the numerical modeling of real geogrid reinforced soil structures, i.e. reinforced base courses and reinforced soil slopes. In both types of the geogrid reinforced soil structures, the bearing capacities of the reinforced cases were much larger than those of the corresponding unreinforced cases. Such DEM observations agreed with laboratory test results reported from the literature. The geogrid reinforcement mechanism was visualized not only by the qualitative force distributions in the specimens and along the geogrids but also by the qualitative displacement distributions of soil particles. At the same loading state, the displacements and settlements of soil particles in the reinforced soil structures were much smaller than those in the unreinforced cases. The quantitative tensile force and vertical displacement distributions along the geogrids clearly indicated the geogrid reinforcing effects in such practical reinforced soil structures. Therefore, it is concluded that the developed DEM models in this study can be successfully used to describe and to visualize the geogrid reinforcement mechanisms in real geogrid reinforced soil structures.
8.2 Recommendations for Further Discrete Element Modeling

8.2.1 Influence of Top Boundaries in Pullout Tests

In geogrid pullout tests, a rigid or a flexible top boundary has been commonly used. Although a flexible surcharge loading system is recommended in the ASTM test standard (ASTM D6706-01, 2001), the experimental pullout tests in this study loaded with a rigid top boundary still showed reasonable results. Thus, it is a controversial issue to decide whether to use a rigid or a flexible top boundary in geogrid pullout tests. Based on the experimental studies of Palmeira and Milligan (1989), a slight difference in the maximum pullout resistance was obtained using either a rigid or a flexible top boundary. However, due to the restriction of testing conditions, detailed insights into the geogrid–soil interaction under different top boundaries were not provided in their study.

In order to investigate the influence of rigid and flexible top boundaries on geogrid pullout behavior, DEM investigations of geogrid pullout tests with different top boundaries are suggested to be carried out in further research. Both top boundaries can be simulated with a string of bonded particles. The rotational degree of freedom of the bonded particles is fixed along the rigid top boundary, while the bonded particles can rotate freely along the flexible top boundary. Other micro input parameters should be set identical in both DEM investigations. The numerically obtained pullout force–clamp displacement relations with both top boundaries can be compared with the laboratory test results of Palmeira and Milligan (1989). Moreover, the influence of rigid and flexible top boundaries on the geogrid pullout behavior can be discussed by comparing the corresponding responses of soil and geogrids in both DEM simulations, such as force distributions in the specimen and along the geogrid, tensile force and strain distributions along the geogrid as well as normal stress distributions in the geogrid plane.

8.2.2 Geogrid–Soil Interaction under Running Wheel Loads

The DEM has particular advantages of modeling geogrid–soil interaction under running wheel loads. Trials have been made based on PFC$^{2D}$ using the previously developed DEM models of soil and geogrids. General interaction between soil and geogrid could be visualized during the movement of the running wheel, as shown in Figure 8.1. However, due to the inherent limitations of 2D modeling, the movement of soil particles in dense specimens along the cross-plane direction could not be simulated using PFC$^{2D}$ under the running wheel. The contribution of geogrids in reducing rut depths of unpaved roads was not obvious based on 2D discrete element modeling. Hence, further numerical investigations of geogrid–soil interaction under running wheel loads are suggested to be carried out using 3D software. The influence of geogrid placement depth, geogrid tensile stiffness, wheel loads and wheel running velocities on the performance of geogrid reinforced soil bases has to be investigated compared with an unreinforced base course. Detailed insights into the geogrid–soil interaction can provide researchers improved understanding of geogrid reinforcement mechanisms under running wheel loads.
Figure 8.1  Contact force distribution in reinforced base course and tensile force distribution along geogrid under a running wheel load (the wheel corresponds to a roller because of 2D modeling) (thickness of lines proportional to magnitude: red – tensile forces along the geogrid; black – contact forces in the base course).
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Partial results from this dissertation have been published in the following scientific journals and conference proceedings:


Other publications related to the topic of this dissertation:


Appendix A  Main FISH Codes

Appendix A1  Iterative Process of Determining 2D Porosity

```plaintext
1  w_001 = w_speimen*0.01
2  wadd1 = find_wall(1)
3  wadd83 = find_wall(83)
4
5  loop while 1 # 0
6     w1_f = w_yfob(wadd1)
7     w83_f = w_yfob(wadd83)
8     wydisp_83_1 = w_y(wadd83)
9     posi_yvel83 = -w1_f * 1e-5
10    nega_yvel83 = w1_f * 1e-5
11  if w83_f = 0 then
12     command
13        wall id 83 yvel = nega_yvel83
14        cyc 1
15        solve max 0.001
16     endcommand
17  endif
18  if w83_f > w_001 then
19     command
20        wall id 83 yvel = posi_yvel83
21        cyc 1
22        solve max 0.001
23     endcommand
24  endif
25  if w83_f < w_001 then
26     if w83_f > 0 then
27        command
28            wall id 83 yvel=0
29        endcommand
30        exit
31     endif
32  endif
33  endloop
```
Appendix A2  Developed Piecewise Linear Model of Geogrid

```
1  ball_a = find_ball(j)
2  ball_b = find_ball(j+1)
3  b_xdisp_a = b_xdisp(ball_a)
4  b_xdisp_b = b_xdisp(ball_b)
5  strain_ball = (b_xdisp_b - b_xdisp_a)/(_rad*2)*100
6
7  if strain_ball >= eps1
8    cp_a = b_clist(ball_a)
9    loop while cp_a # null
10       pbp_a = c_pb(cp_a)
11          if pbp_a # null
12             pb_kn(pbp_a) = pb_stiff1
13          endif
14       b1 = c_ball1(cp_a)
15       if b1 = ball_a
16         cp_a = c_b1clist(cp_a)
17       else
18         cp_a = c_b2clist(cp_a)
19       endif
20  endloop
21  endif
```
Appendix A3  Normal Stress Distribution in Geogrid Plane

```plaintext
1  cp = contact_head
2  loop while cp # null
3       if c_x(cp) >= x1 then
4           if c_x(cp) <= x2 then
5               if c_y(cp) >= y1 then
6                   if c_y(cp) <= y2 then
7                       if c_xun(cp) = 0
8                       angle = -90
9                   else
10                       angle = atan(c_yun(cp)/c_xun(cp))*180/Pi
11                   endif
12           else
13               if angle < -80 then
14                   angle_80 = angle_80 + 1
15                   c_nforce_80 = c_nforce_80 + c_nforce(cp)
16                   c_sforce_80 = c_sforce_80 + c_sforce(cp)
17               else
18                   if angle < -70 then
19                       ...  
20                   else
21                       ...    
22               else
23                   if angle < 90 then
24                       angle90 = angle90 + 1
25                   c_nforce90 = c_nforce90 + c_nforce(cp)
26                   c_sforce90 = c_sforce90 + c_sforce(cp)
27               endif
28           endif
29       endif
30   endif
31  cp = c_next(cp)
32 endloop
```
Figure B.1  Particle displacements and rotations in unreinforced specimens in laboratory tests (vertical strain $\varepsilon_1 = 10\%$).
Figure B.2 Particle displacements and rotations in unreinforced and reinforced specimens in laboratory tests (L6 series) (vertical strain $\varepsilon_1 = 10\%$).