Analysis of horizontal deformations to allow the optimisation of geogrid reinforced structures

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Analysis of Horizontal Deformations to allow the Optimisation of Geogrid Reinforced Structures

Ian Scotland
ANALYSIS OF HORIZONTAL DEFORMATIONS TO ALLOW THE ANALYSIS OF HORIZONTAL DEFORMATIONS TO ALLOW THE OPTIMISATION OF

By

Ian Scotland

A dissertation thesis submitted in partial fulfilment of the requirements for the award of the degree Doctor of Engineering (EngD), at Loughborough University

November 2016

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Finally, the main author would like to thank his parents, Dennis and Christine Scotland, partner Jessica Howarth and friends for their eternal support throughout the project.
ABSTRACT

Geogrid reinforced structures have been successfully used for over 25 years. However their design procedures have remained largely focused on ultimate failure mechanisms, originally developed for steel reinforcements. These are widely considered over conservative in determining realistic reinforcement and lateral earth stresses. The poor understanding of deformation performance led many design codes to restrict acceptable soils to selected sand and gravel fills, where deformation is not as concerning.

Within UK construction there is a drive to reduce wastage, improve efficiency and reduce associated greenhouse gas emissions. For geogrid reinforced structures this could mean increasing reinforcement spacing and reusing weaker locally sourced soils. Both of these strategies increase deformation, raising concern about the lack of understanding and reliable guidance. As a result they fail to fulfil their efficiency potential.

This Engineering Doctorate improved the understanding of horizontal deformation by analysing performance using laboratory testing, laser scanning industry structures and numerical modelling. Full-scale models were used to demonstrate a reduction in deformation by decreasing reinforcement spacing. Their results were combined with primary and secondary case studies to create a diverse database. This was used to validate a finite element model, differentiating between two often used construction methods. Its systematic analysis was extended to consider the deformation consequences of using low shear strength granular fills. The observations offered intend to reduce uncertainty and mitigate excessive deformations, which facilitates the further optimisation of designs.

KEY WORDS

Geosynthetics, Geogrid Reinforced Structures, Face Deformation, Laser Scanning, Numerical Modelling
**USED ABBREVIATIONS**

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tbody>
<tr>
<td>BSI</td>
<td>British Standards Institute</td>
</tr>
<tr>
<td>CEN</td>
<td>European Committee for Standardization</td>
</tr>
<tr>
<td>CICE</td>
<td>Centre for Innovative and Collaborative Construction Engineering</td>
</tr>
<tr>
<td>EngD</td>
<td>Engineering Doctorate</td>
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<tr>
<td>EOC</td>
<td>End Of Construction</td>
</tr>
<tr>
<td>EPSRC</td>
<td>Engineering and Physical Research Science Council</td>
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<tr>
<td>FE</td>
<td>Finite Element</td>
</tr>
<tr>
<td>FH</td>
<td>Full Height (Construction Method)</td>
</tr>
<tr>
<td>GRS</td>
<td>Geogrid Reinforced Structures</td>
</tr>
<tr>
<td>GNM</td>
<td>Generalised Numerical Model</td>
</tr>
<tr>
<td>HS</td>
<td>Hardening Soil</td>
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<tr>
<td>LL</td>
<td>Layer by Layer (Construction Method)</td>
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<tr>
<td>LVDT</td>
<td>Linear Variable Differential Transformer</td>
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<tr>
<td>MC</td>
<td>Mohr-Coulomb</td>
</tr>
<tr>
<td>NM</td>
<td>Numerical Model</td>
</tr>
<tr>
<td>PET</td>
<td>Polyester</td>
</tr>
<tr>
<td>PWP</td>
<td>Pore Water Pressure</td>
</tr>
<tr>
<td>RDCW</td>
<td>Recycled Demolition and Construction Waste</td>
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<tr>
<td>SLS</td>
<td>Serviceability Limit State</td>
</tr>
<tr>
<td>TLS</td>
<td>Terrestrial Laser Scanner</td>
</tr>
<tr>
<td>ULS</td>
<td>Ultimate Limit States</td>
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# USED SYMBOLS AND NOTATION

<table>
<thead>
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<th></th>
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<tr>
<td><strong>Reinforcement Properties</strong></td>
<td></td>
</tr>
<tr>
<td>$EA$ (N/m)</td>
<td>Stiffness of numerically modelled geogrid</td>
</tr>
<tr>
<td>$N_p$ (N/m)</td>
<td>Ultimate tensile force in geogrid</td>
</tr>
<tr>
<td>$\varepsilon$ (-)</td>
<td>Strain in Geogrid</td>
</tr>
<tr>
<td><strong>Soil Properties</strong></td>
<td></td>
</tr>
<tr>
<td>$\varphi$ (°)</td>
<td>Plane strain friction angle</td>
</tr>
<tr>
<td>$\psi$ (°)</td>
<td>Dilation angle</td>
</tr>
<tr>
<td>$c$ (N/m²)</td>
<td>Cohesion</td>
</tr>
<tr>
<td>$E_{50}^{ref}$ (N/m²)</td>
<td>Tangent stiffness for primary oedometric loading</td>
</tr>
<tr>
<td>$E_{oed}^{ref}$ (N/m²)</td>
<td>Secant stiffness in standard drained triaxial test</td>
</tr>
<tr>
<td>$E_{ur}^{ref}$ (N/m²)</td>
<td>Unloading reloading stiffness</td>
</tr>
<tr>
<td>$p_{ref}$ (N/m²)</td>
<td>Reference Stress Level</td>
</tr>
<tr>
<td>$m$ (-)</td>
<td>Power Exponent of the Ohde/Janbu law</td>
</tr>
<tr>
<td>$\nu$ (-)</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>$\gamma$ (kN/m³)</td>
<td>Unit weight (unsaturated)</td>
</tr>
<tr>
<td>$R_{inter}$ (-)</td>
<td>Strength reduction factor for interfaces</td>
</tr>
<tr>
<td><strong>GRS Dimensions</strong></td>
<td></td>
</tr>
<tr>
<td>$L$ (m)</td>
<td>Reinforcement length</td>
</tr>
<tr>
<td>$H$ (m)</td>
<td>Height</td>
</tr>
<tr>
<td>$S_v$ (m)</td>
<td>Reinforcement spacing</td>
</tr>
<tr>
<td>$\delta_x$ (m)</td>
<td>Lateral deformation during construction</td>
</tr>
<tr>
<td>$\delta x_{pc}$ (m)</td>
<td>Lateral deformation post-construction</td>
</tr>
<tr>
<td><strong>Others</strong></td>
<td></td>
</tr>
<tr>
<td>$\Delta \sigma_v$ (N/m²)</td>
<td>Additional Vertical Loading</td>
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PAPER 3 (APPENDIX C)
1 INTRODUCTION

1.1 BACKGROUND TO THE RESEARCH

1.1.1 GEOGRID REINFORCED STRUCTURES

The term, geosynthetics, refers to fabricated materials, usually planar, that are used to improve the characteristics of the surrounding soil. Geosynthetic products are typically made of polymeric materials, although natural fibres can be included in this category. Geosynthetics were first applied in earthworks in the late 1950s (Jones and Doulala-Rigby 2014). Examples of their applications as reinforcement in geotechnical engineering are shown in Figure 1.1.

![Figure 1.1 Examples of geosynthetic reinforced soil applications (Huesker Limited 2016)](image)

Geogrid Reinforced Structures (GRS) consist of soil and plastic. The latter is typically in the form of a Geogrid, which is a planar material, consisting of a regular open network of intersecting tensile-resistant elements, called ribs. Soil, commonly sand or gravel, is inherently weak in tension, relying on compression and shear strength for stability. Inserting geogrids into a soil mass forms an enhanced composite material termed ‘Reinforced Soil’.

The mechanically enhanced soil bears most of the tension, interacting with the soil through friction and confinement, thus maintaining the stability of the soil mass. Geogrids are manufactured with typical ultimate tensile strengths ranging from 10 kN/m up to 2000 kN/m, from a variety of polymers including Polyester (PET), polyvinyl alcohol, polypropylene and
Analysis of Horizontal Deformations to allow the Optimisation of GRS

High density polyethylene. These polymers vary in loading behaviour, and materials with ultimate strains of 3% to 20% are typically used in reinforcing structures (Shukla 2011).

Typical components of a GRS are displayed in Figure 1.2, comprising:

- Horizontal layers of geogrid reinforcement, extending back from the face;
- Reinforced soil fill, usually a free draining granular soil like sand or gravel, providing compressive strength and interlocking with the reinforcement;
- A facing preventing localised facing failure, when GRS is steeper than the natural slope angle of soil. This research particularly considers wrapped facing, where the geogrid is returned back in to the fill;
- Retained backfill or natural soil, directly behind the GRS;
- Foundation soil, on which the structure rests.

Figure 1.2: Typical GRS Components
1.1.2 FACING AND CONSTRUCTION

The construction of GRS involves the sequence of filling and laying reinforcement. The thickness of fill layers varies from project to project, based on reinforcement spacing, compaction level and soil properties. Specifically, the construction of wrapped GRS requires some form of lateral restraint or propping, during backfilling and compaction. There are two general temporary propping methods: full height formwork (Figure 1.3.a) or layer by layer formwork (Figure 1.3.b). There are also permanent facing methods such as steel meshes and segmental blocks but these are not considered in this investigation. These methods have implications for the magnitude, distribution and timing of horizontal deformation (Section 2.3.1.2).

![Temporary construction methods for wrapped GRS: a) FH b) LL](image)

In the Full Height (FH) construction method, the GRS is constructed to its full height while laterally restrained behind a full height propped panel (Figure 1.4). This panel is then released in one action, allowing the structure to deform simultaneously.

In the Layer by Layer (LL) construction method, the GRS is built behind localised facing panels, covering one or two layers. These are released locally after a subsequent layer is constructed. This means that deformation occurs throughout the construction process. This form
of construction is favoured in cases where a full height propping solution is not possible due to costs and construction feasibility.

![Figure 1.4: Construction Methods: FH (left) or LL (right)](image)

### 1.1.3 Deformation in Geogrid Structures

Deformation is typically defined as the action of changing shape and is measured relative to an external point of reference. Deformation in GRS is inherent because reinforcement must strain to resist tensile forces. This research focuses on horizontal deformation, also referred to as lateral movement. As GRS are typically analysed as 2-dimensional structures in plane strain, horizontal deformation can be considered as the sum of three components (Figure 1.5):

- Face deformation; specifically bulging in wrapped faced GRS, resulting in the deformation of the facing elements.
- Internal deformation; resulting from reinforcement and soil movement within the reinforced soil body zone.
- GRS displacement, or global deformation; typically caused by the pressure from the retained fill, resulting in the whole structure moving forward.
These deformations occur throughout the construction and service life of the GRS. Deformation during the construction period is the focus of the research. As layers are constructed, lateral stresses build up, which are initially resisted by formwork or facing units, when some deformation may occur. In the case of temporary formwork which is removed after construction of each Layer by Layer (LL) or upon reaching Full Height (FH), deformation needs to occur for the reinforcement to become tensioned.

Post-construction deformation can occur internally as either creep deformation of polymeric reinforcements (Section 2.3.1.3), degradation of the geogrid or the consolidation of poorly compacted low permeability backfill (British Standards Institute 2013). Creep in particular is a phenomenon dependent on a range of variables including reinforcement tension, time and temperature.

Deformation is typically recorded as a set of distance measurements which some researchers (Section 2.1.2) convert in to a height normalised form to simplify comparisons of GRS with different heights. Deformation in wrapped GRS features highly non-uniform distribution which does not lend itself to the use of a simple assumption. Hereafter deformation data presented is presented as absolute distance.
1.1.4 Context of the Research

The design of GRS currently falls outside the remit of the Eurocodes (Section 2.1.1). As a result there are varying design standards applicable across Europe, each with their own partial factors and procedures. In the UK, the recommended design code is BS 8006:2010, *The code of practice for strengthened/reinforced soils and other fills* (British Standards Institute 2010). Its design procedures are discussed in Section 2.1.1 and in Section 2 of Paper 1.

The next revision of Eurocode 7 for geotechnical design (British Standards Institute 2013) is set to include reinforced soil structures (Bond 2014), leading to a search for common ground between the national design codes, in a bid to create a harmonised document. This offers the opportunity to further optimise existing design methods that have largely remained Ultimate Limit State (ULS) focused since they were developed in the 1990s. There are still aspects of GRS about which little is known. Particularly there is uncertainty regarding deformation behaviour (Section 2.3.1), reinforced soil composite behaviour (Section 2.3.2), construction processes (Section 2.3.3) and performance when incorporating weaker soils (Section 2.3.4).

Within the UK construction industry there is a drive to reduce materials wastage, improve design efficiency and reduce associated greenhouse gas emissions (Department for Business, Innovation and Skills 2013). For GRS, this means considering increasing reinforcement spacing and reusing locally sourced soils, which may be weaker. Both of these strategies increase deformation concerns because although the ULS focused methods have proven reasonable for GRS with high strength granular soils, the poor understanding of deformation remains a big limitation, particularly for weak soils (Christopher *et al.* 1998, Raja *et al.* 2012). As a result, only high grade reinforced fills are recommended for reinforced walls according to BS 8006:2010 (British Standards Institute 2010), leaving the reuse of locally sourced fills underutilised.
In place of design codes, there is a range of design guidance for estimating horizontal deformation. A selection are detailed in Section 2.1.2. However there are many variables not currently considered, such as the influence of construction techniques (Objective 3) and low shear strength soils (Objective 4) in wrapped GRS, which are the focus of this research.

1.2 PROJECT STAKEHOLDERS

1.2.1 CENTRE FOR INNOVATIVE AND COLLABORATIVE CONSTRUCTION ENGINEERING

The project has been undertaken as part of an Engineering Doctorate (EngD), which is a programme designed to facilitate joint ventures between the academic and industry research spheres. This EngD project was supported by the Centre for Innovative and Collaborative Construction Engineering (CICE), which is a centre of excellence, committed to advanced training and research in engineering and management. It is funded by the Engineering and Physical Research Science Council (EPSRC) and operates a number of EngD programmes, covering a range of topics across infrastructure.

1.2.2 THE INDUSTRIAL SPONSOR

The sponsoring company of this EngD is Huesker Limited, a geosynthetic supplier based in Warrington, UK. They are a subsidy of the Huesker Group, headquartered in Gescher, Germany, where the majority of their products are produced. The company has strong expertise in geogrids and geotextiles, having pioneered some of the very first products and regularly invests in research and development of the reinforcement application.

Although providing a variety of products for other applications such as separation, filtration and erosion protection, it specialises in the products for the reinforcement applications. As well
as supplying geosynthetic products, Huesker have a dedicated technical team providing advice and design suggestions.

Huesker hope that this research will increase their competitiveness, by improving the quality and reliability of their designs; to better compete against conventional retaining structures systems. The company also wanted to improve their local expertise in reinforcement application.

1.2.3 THE AUTHOR AND PROJECT TEAM

The principal author of the EngD was Ian Scotland, a postgraduate research student at Loughborough University. He started the project following a civil engineering undergraduate degree at the University of Warwick.

The wider project team included three academic supervisors from Loughborough University: Professor Neil Dixon and senior lecturers Dr. Matthew Frost and Dr. Gary Fowmes. Completing the group was an industrial supervisor from the sponsoring company: Graham Horgan, the managing director of Huesker Limited, with a strong technical background in geosynthetic design.
1.3 AIM AND OBJECTIVES

Aim

The overarching aim of the EngD project was to promote the optimisation of Geogrid Reinforced Structures (GRS), by investigating horizontal deformation occurring during and shortly after construction, using conventional and low strength granular soils.

Objectives

Four specific objectives, outlined below, were identified in order to achieve the research aim. These objectives have been mapped to their related academic paper and sections in Table 1.1 and discussed in Section 3 and Figure 3.1.

Objective 1: Review the current state-of-the-art practices and understanding of the SLS design of GRS, specifically for horizontal deformation, as well as comparing approaches by national design codes and highlighting possible improvements for future research.

Objective 2: Obtain detailed horizontal deformation data, occurring during and shortly after construction for a range of wrapped GRS. The data should differentiate between face and internal deformation.

Objective 3: Develop and validate a numerical model representing the construction stages of a wrapped GRS, using a range of case study data. Use the model to compare the deformation performance of wrapped GRS using commonly applied construction techniques.

Objective 4: Extend the validated model to systematically examine horizontal deformation performance of wrapped GRS using low strength soils and evaluate potential optimisations.
Table 1.1: Objective Research Map

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<td>Literature Review</td>
<td>Section 2; Paper 1: Conference Paper at 5th European Congress of Geosynthetics (2012)</td>
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<tr>
<td>Objective 2</td>
<td>Literature Review</td>
<td>Sections 4.1, 4.2 and 4.3; Paper 2: Conference Paper at 10th International Congress of Geosynthetics (2014)</td>
</tr>
<tr>
<td></td>
<td>Laboratory Modelling Surveying</td>
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<tr>
<td>Objective 3</td>
<td>Literature Review</td>
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<td>Objective 4</td>
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<td>Numerical Modelling</td>
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1.4 JUSTIFICATION AND SCOPE

The design of GRS, is a wide-ranging subject, with many areas having the potential to be optimised (Sections 3 and 4 in Paper 1). In order to make a meaningful contribution to knowledge, the scope of this EngD project focused on the design of SLS, particularly horizontal deformation. Increased deformation is a concern for the potential use of marginal soils (Christopher et al. 1998). Improving the general understanding and developing models capable of estimating deformation in GRS with marginal fills will increase their acceptability.

Current design codes do not contain analytical models to assess horizontal deformation, instead a variety of models are used to estimate it, discussed later in Section 2.1.2. These consider a wide-range of parameters but no existing models include construction variables, such as formwork technique.

As highlighted in Section 2.3.1.2, a lot of existing deformation research has focused on segmental block walls, which deform much less than wrapped faced structures. Few of the existing models focus on wrapped GRS, which is a system commonly used by the sponsoring company.
In order to develop new guidance, the project combined three primary investigatory methods: laboratory testing, surveying industry structures and numerical modelling. The creation of multiple large full-scale models alone are inefficient, requiring the development of a numerical model capable of representing both laboratory models and site-surveyed structures. A Finite Element (FE) model was used to consider deformation trends for variables such as construction method and soil strength.

The investigation does not consider horizontal deformation beyond a stage shortly after the end of construction, after which, polymeric creep-induced strain of the geogrid can cause further deformation (Section 2.3.1.3). This investigation centres on face deformation, on which long-term creep has limited influence. Therefore, investigations using long-term testing or accelerated creep tests were considered outside the scope. To this effect, the laboratory and numerical modelling is simplified by simulating loading immediately after construction.

1.5 STRUCTURE OF THE THESIS

This thesis documents the work undertaken during the four year EngD research project. The structure of the thesis is presented below, indicating the content of each section and the appendix. The appendix includes 4 academic papers, which should be read in conjunction to this thesis.

Chapter 1 introduces the background to GRS, and defines the aim and objectives of researching horizontal deformation.

Chapter 2 provides an overview of the literature surrounding the horizontal deformation in GRS. This includes a look at current design codes (Section 2.1.1), empirical models (Section 2.1.2), investigatory techniques (Section 2.2) and current understanding (Section 2.3). The review of horizontal deformation design formed the basis of Paper 1.
Chapter 3 describes the design and philosophy behind the research (Section 3.1) and the adopted methods for each of the work packages (Section 3.2).

Chapter 4 outlines the main processes involved in the research, which can be divided into laboratory modelling (Section 4.1), industry structure monitoring using a laser scanner (Section 4.2), case study database (Section 4.3), numerical modelling (Section 4.4) and systematic analysis (Section 4.5).

Chapter 5 summarises the findings of each of the work packages (Section 5.1) and discusses the implications for the industry (Section 5.2.1) and sponsoring company (Section 5.2.2).

The Appendices contain three published peer-reviewed academic papers, resulting from this research project. These papers are an important part of the EngD thesis and should be read in conjunction with the main sections. A summary of these can be found in Table 1.2.

<table>
<thead>
<tr>
<th>Thesis Ref.</th>
<th>Paper Title</th>
<th>Publication</th>
<th>Status</th>
<th>Synopsis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Appendix B</td>
<td>‘Serviceability limit state design in geogrid reinforced walls and slopes’</td>
<td>Conference Paper at 5th European Congress of Geosynthetics (2012)</td>
<td>Peer-Reviewed</td>
<td>This paper provided an overview of the current state of serviceability design of GRS according to British and German guidance</td>
</tr>
<tr>
<td>Appendix C</td>
<td>‘Measuring Deformation Performance of Geogrid Reinforced Structures using a Terrestrial Laser Scanner’</td>
<td>Conference Paper at 10th International Congress of Geosynthetics (2014)</td>
<td>Peer-Reviewed</td>
<td>This paper outlined a novel methodology to use a laser scanner to monitor horizontal deformation in GRS. The method was then demonstrated for two GRS case studies, featuring different construction methods.</td>
</tr>
<tr>
<td>Appendix D</td>
<td>‘Modelling Deformation during the Construction of Wrapped Geogrid Reinforced Structures’</td>
<td>Geosynthetic International (2016)</td>
<td>Peer-Reviewed</td>
<td>This paper outlined presented the development of the generalised numerical model based on three wide-ranging GRS. An analysis of construction methods, in particular formwork procedure is outlined and initial guidance for estimating deformation</td>
</tr>
</tbody>
</table>
2 LITERATURE REVIEW

This chapter summarises a concise and focused literature review, initially undertaken in the first year and updated throughout the EngD. The review sought to capture the current best practice and understanding of deformation performance in GRS, meeting objective 1. The outcome was used to inform the direction of the subsequent research, as well as to provide suggestions for improvements in current design practice (Section 2.1.1.5 and Paper 1).

Through Loughborough University and the sponsoring company, Huesker Limited, the research had access to wide-ranging resources. The information captured included design guidance documents, numerical and laboratory research, and case studies with performance data. Therefore the scope of the literature reviews included:

- National codes of practice for the design of geosynthetic reinforced walls and slopes, i.e. BS 8006:2010 (British Standards Institute 2010), EBGEO (Deutsche Gesellschaft fur Geotechnik 2011).
- Articles from geotechnical and geosynthetic journals, such as Geosynthetic International, Geotextiles and Geomembranes and Canadian Geotechnical Journal.
- Articles from regional and international geosynthetic and leading geotechnical engineering conference proceedings such as the International Conference of Geosynthetics.

With a general introduction to GRS already given in Section 1.1.1, the findings presented herein focus on the current level of design practice and understanding of horizontal deformation performance of GRS. The following section (Section 2.1) summarises the historical development of GRS design practice, covering both national standards (Section 2.1.1) and alternative deformation guidance (Section 2.1.2). Section 2.2 considers the current
investigatory techniques for horizontal deformation in GRS. This is countered in Section 2.3 with a summary of the current level of knowledge, highlighting the most influential design factors and causes of horizontal deformation. Finally, Section 2.4 summarises the key findings of the review.

2.1 HISTORICAL DESIGN METHOD DEVELOPMENT

Retaining structures utilising geogrids as reinforcements have been constructed since the 1970’s (Jones and Doulala-Rigby 2014). The application’s similarity to other soil reinforcements such as soil nails, anchors and steel strips, initially saw the design procedures of GRS develop concurrently, typically utilising limit equilibrium methods such as the Tie back wedge and Coherent gravity (Barnes 2010), where soil and reinforcing inclusions are treated as individual complementary components, known as the simple method (Allen et al. 2002).

Similarly to wider construction developments (British Standards Institute 2013), design practice has moved from limit equilibrium, where a lumped factor of safety is applied, to a limit state analysis, where partial safety factors are applied to uncertain properties in the analysis, such as soil strength and loading. However both these methods make a distinction between Ultimate Limit States (ULS) and Serviceability Limit States (SLS). ULS are generally associated with total collapse or structural failure, while SLSs correspond to unacceptable levels of deformation or a reduction in service life. Horizontal deformation, as defined in Section 1.1.3, is considered a SLS. In SLS design, the effects of the sustained design actions should not exceed its service requirements. Serviceability limits are more subjective than ultimate limits because so much of it depends on project-specific requirements. These can be assessed directly or indirectly in a number of ways, which the next section shows.
2.1.1 **National GRS Design Practices**

There are now a plethora of design codes available for developing GRS solutions, as outlined in this section. However, these are still mainly ULS focused and most only offer suggestions for SLS, failing to recommend meaningful design processes to assess limits such as excessive deformation. This has been a long-held criticism of the current design codes and is currently addressed by using research-based deformation models, explored in Section 2.1.2.

2.1.1.1 **European Standard Framework**

All structural designs in Europe should abide by the pan-contintental ‘Eurocodes’. These are divided by field. Geotechnical design is addressed in ‘Eurocode 7’, also known as BS EN 1997-1:2004+A1:2013: *Geotechnical Design: Part 1* (British Standards Institute 2013). Unlike the design of concrete or sheet piled retaining structures, the design or execution of reinforced soil structures is not included in the European standard. Instead, each country’s national annex to the code directs designers to the applicable local design codes. As a result, there is a great deal of non-conformity amongst the various national codes throughout Europe.

2.1.1.2 **International Design Codes**

Table 2.1: Horizontal Deformation in International GRS Design Codes

<table>
<thead>
<tr>
<th>Country</th>
<th>Document name (Reference)</th>
<th>Horizontal Deformation design</th>
</tr>
</thead>
<tbody>
<tr>
<td>France</td>
<td>NF P94-270 (Association Francaise de Normalisation 2009)</td>
<td>Internal and external deformation assessment suggested, without method.</td>
</tr>
<tr>
<td>Germany</td>
<td>EBGEO (Deutsche Gesellschaft für Geotechnik 2011)</td>
<td>Suggests empirical, numerical or analytical methods considering reduced horizontal earth pressure (see Section 2.1.1.4).</td>
</tr>
<tr>
<td>Holland</td>
<td>CUR 198 (CUR Building and Infrastructure 2000)</td>
<td>Post-Construction Strain Limit. Similar to BS 8006 (British Standards Institute 2010)</td>
</tr>
<tr>
<td>Scandinavia</td>
<td>Nordic Guidelines for Reinforced Soils and Fills (Nordic Geosynthetic Group 2005)</td>
<td>Equation limiting GRS Height on soft soils to prevent global shear. Also 2% post construction strain limit. Horizontal deformation suggested as typically between 0.1% to 0.3% of GRS height.</td>
</tr>
<tr>
<td>United Kingdom</td>
<td>BS 8006 (British Standards Institute 2010)</td>
<td>Post-Construction Strain Limit (see Section 2.1.1.2)</td>
</tr>
<tr>
<td>United States</td>
<td>Design Manual for Segmental Retaining Walls’ (National Concrete Masonry Association 2002)</td>
<td>Bulging calculations but limited to segmental block faced GRS.</td>
</tr>
<tr>
<td></td>
<td>AASHTO LRFD Bridge Design Specification (2012)</td>
<td>Includes Christopher (1990) chart for estimating construction deformation of GRS (See Section 2.1.1 for further details).</td>
</tr>
<tr>
<td>Hong Kong</td>
<td>Geoguide 6 (Jones 2002)</td>
<td>No method, provides only tolerances and construction advice.</td>
</tr>
</tbody>
</table>

2.1.1.3 BS 8006:2010

BS 8006:2010, The code of practice for strengthened/ reinforced soils and other fills (British Standards Institute 2010), is the recommended design code for the UK, but it is used across the world, in countries such as Australia (Queensland Department of Transport and Main Roads 2015) and India (Ministry of Railways 2005). BS 8006 has been a standard for reinforced soil structures design for 16 years, the most recent version came into effect on 31st October 2010.

The code is in a partial limit state format. The partial factors have been developed based on empirical evidence. As with reinforced concrete retaining walls, both the external and internal stability should be checked and the overall, rotational or global stability of the reinforced soil mass has to be checked using slope stability procedures as in BS EN 1997-1:2004+A1:2013 (British Standards Institute 2013). BS 8006 is similar to other codes, as GRS are designed according to ULS, like reinforcement rupture and pull-out, before being checked for SLS, such as post-construction strain (Figure 2.1). The code makes an important distinction between the
design of walls, structures greater than 70° of inclination; and slopes, structures less than 70° of inclination. The steeper structures are subject to more stringent post-construction reinforcement strain limits (<1%), compared to the shallower slopes (<5%). There is no limit for construction. Walls are also limited to higher quality fills, such as class 6I and class 6J in the Manual of Contract Documents for Highways Works (Highways England 2016). In most cases ULS design dominates, this is reflected by 40 out of the 48 pages being devoted to ULS design procedures, while only 8 refer to SLS and considerations for serviceability. There is no guidance to estimate construction deformation. Further detail on SLS design in BS 8006 (2010), can be found in Section 2.1 of Paper 1.

![Graph showing post-construction strain limit from BS 8006 (2010).](image)

**Figure 2.1: Post-Construction strain limit from BS 8006 (2010).**

### 2.1.1.4 EBGEO

As a Germanic company, the sponsoring company often refer to the German design code, EBGEO (Deutsche Gesellschaft fur Geotechnik 2011), herein referred to as EBGEO (2011). It is linked to the *German National Standard for Earthworks*: DIN 1054 (Beuth 2005). EBGEO (2011) starts by assessing ULSs, before considering SLSs which it defines as structural deformations resulting from characteristic dead loads and soil parameters. The code highlights the following SLSs: foundation settlement; internal settlement of reinforced fill; horizontal...
Analysis of Horizontal Deformations to allow the Optimisation of GRS

movement of the front of the structure and face deformation. Although no explicit design procedures are detailed, it suggests using numerical analysis, empirical data or observational methods.

Regarding internal deformation, EBGEO (2011) suggests integrating individual reinforcement strains to obtain a total horizontal deformation. The designer can calculate this from the service loading and the load-strain characteristics of an individual geogrid. The code suggests examining the forces present on the face and the subsequent deformation, without giving a detailed design method beyond using the active earth pressure as a reference variable.

The code includes a lateral earth pressure coefficient $K_{agh}$ reduction for flexible faced GRS (Figure 2.2), following research from Pachomow et al. (2007), discussed further in Section 2.3.1.1. In the lower 60% of the structure, a designer is allowed to reduce earth pressure by 30% for partially deformable GRS, like segmental block walls, and 50% for fully deformable structures like wrapped layers.

![Figure 2.2: Earth pressures reduction allowed in EBGEO (2011)](image-url)
2.1.1.5 National Code Comparison

As discussed in Section 1.1.4, in order to include reinforced soil design within Eurocode 7, there is drive to harmonise national design codes. The British, BS 8006 (2010) and German, EBGEO (2011) design codes were directly compared in an initial literature review and published in a conference paper (Paper 1). Both have broadly similar ULS checks, varying mainly in their selection of partial factor values and method flexibility.

Subsequently, this comparison was extended by Hangen et al. (2014), who included the French design code, NF P94-270 (Association Française de Normalisation 2009). Finding only marginal differences in partial factors.

However, as reported in Section 4 in Paper 1, all three have limited or non-existent SLS guidance. Neither have a complete assessment of SLS (Table 2.2). EBGEO suggests a wider range of deformation areas to consider, while BS 8006 is the only one of the codes to provide a procedure and a suggested limit for horizontal displacement. Although this may be unsuitable as Section 2.3.1.3 describes.

<table>
<thead>
<tr>
<th>Deformation Sources</th>
<th>BS 8006 (British Standards Institute 2010)</th>
<th>EBGEO (Deutsche Gesellschaft fur Geotechnik 2011)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation Settlement</td>
<td>Refers to EN 1997 (British Standards Institute 2013)</td>
<td>Refers to DIN 1054 (Beuth 2005)</td>
</tr>
<tr>
<td>Reinforced Fill Settlement</td>
<td>Suggests providing good drainage to prevent, migration of fine-grained soils</td>
<td>Elastic analysis methods are proposed, that are similar to EN 1997 (British Standards Institute 2013)</td>
</tr>
<tr>
<td>Horizontal Displacement</td>
<td>Suggests limiting post-construction geogrid strain based on type of structure. Implemented by using isochronous curves. (See sections 2.1.1.2 and 2.1.1)</td>
<td>Suggests integrating limit equilibrium calculated strains of geogrids.</td>
</tr>
<tr>
<td>Shear Deformation</td>
<td>Not mentioned</td>
<td>No method, but says this can be as much as 30%-50% of Horizontal Displacement</td>
</tr>
<tr>
<td>Facial Deformation</td>
<td>Dependant on face. Does not provide a methodology.</td>
<td>Suggests using reduced active earth pressure acting on back of wall, for facing types (flexible, rigid etc.).</td>
</tr>
</tbody>
</table>
2.1.2 **Horizontal Deformation Guidance**

In the absence of detailed analytical models in the design codes, empirically derived charts and relationships have been developed by various researchers (Giroud *et al.* 1989; Jewell and Milligan 1989; Christopher 1993; Chew and Mitchell 1994; Wu 1994; Lee 2000; Bathurst *et al.* 2002; Allen *et al.* 2003; Wu *et al.* 2013; Allen and Bathurst 2015) for estimating horizontal deformation. Details of the most popular models are listed in Table 2.3 and summarised later. These models cover a range of geosynthetic materials (geotextiles and geogrids) and design variables.

**Table 2.3: Overview of existing empirical and analytical deformation guidance**

<table>
<thead>
<tr>
<th>Reference</th>
<th>Materials Covered</th>
<th>Validation Data</th>
<th>Facing Type</th>
<th>Location of Deformation</th>
<th>Variables Considered</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.a) Allen and Bathurst 2015</td>
<td>Geogrid Geotextile</td>
<td>Case Studies/ NM</td>
<td>Wrapped/ Segmental</td>
<td>Internal</td>
<td>H/qφ/c/EAε/γ/Sv</td>
</tr>
<tr>
<td>1.b) Bathurst <em>et al.</em> 2002</td>
<td>Geogrid</td>
<td>Case Studies</td>
<td>Wrapped/ Segmental</td>
<td>Face</td>
<td>H/q</td>
</tr>
<tr>
<td>2) Chew and Mitchell 1994</td>
<td>Geotextile</td>
<td>NM / Case Studies</td>
<td>Segmental</td>
<td>Face</td>
<td>H/L/EA/Sv/q</td>
</tr>
<tr>
<td>3) Christopher 1993</td>
<td>Geotextile</td>
<td>NM/ Case Studies/ Centrifuge</td>
<td>Segmental</td>
<td>Face</td>
<td>H/L/EA/Sv/φ/ψ</td>
</tr>
<tr>
<td>4) Giroud <em>et al.</em> 1989</td>
<td>Geogrid Geotextile</td>
<td>Analytical</td>
<td>None</td>
<td>Internal</td>
<td>L/ε</td>
</tr>
<tr>
<td>5) Lee 2000</td>
<td>Geotextile</td>
<td>NM/ Case Studies</td>
<td>Wrapped/ Segmental</td>
<td>Internal</td>
<td>H/EA/Sv</td>
</tr>
<tr>
<td>7) Wu 1994</td>
<td>Geogrid Geotextile</td>
<td>Case Studies/NM</td>
<td>Wrapped</td>
<td>Internal</td>
<td>ε/H</td>
</tr>
</tbody>
</table>

2.1.2.1 Allen and Bathurst (2015) and Bathurst et al. (2002)

The most comprehensive, by virtue of the variables considered, is the ‘K-stiffness method’ (Table 2.3, 1.a), first presented by Allen et al. (2003) and later updated by Allen and Bathurst (2008) and Allen and Bathurst (2015). It incorporates a number of empirically calibrated parameters to predict the maximum tension and strain deformation in each layer. These authors relied on data from case studies and instrumented test walls to develop the method. The scope of the main guidance (Table 2.3: 1.a) differentiates between wrapped and segmental block faced walls, by applying a correction factor, $\phi_s$, that suggests internal deformation in a wrapped structure is double that in an otherwise identical block faced wall.

As face deformation data is not included in this method, Bathurst et al. (2002) also provide a supplementary height-normalised chart (Figure 2.3), of measured face deformation, $\delta x$ for three instrumented case studies, including a wrapped faced GRS (GW16). A term was also included to consider surcharges ($q/\gamma$).

![Figure 2.3: Height-normalised deformation chart adapted from Bathurst et al. (2002)](image-url)
2.1.2.2 Chew and Mitchell (1994)

Another deformation model by Chew and Mitchell (1994) is based on segmental block-faced geotextile reinforced structures and includes a range of soil and geogrid parameters. It was developed by accessing a range of numerical models, which were calibrated against monitored structures. The model uses a series of charts (Figure 2.4) to determine height-normalised horizontal deformation compared to a base case, \( (\delta/H)_{\text{Base}} \) for various reinforcement stiffness. This control case is adapted by applying three other factors: \( DI_{LH} \), \( DI_H \), and \( DI_{slope} \) for slenderness ratio \( (L/H) \), height, inclination and loading respectively. Horizontal deformation can then be estimated using the following equation:

\[
\delta/H = (\delta/H)_{\text{Base}} \cdot DI_{LH} \cdot DI_H \cdot DI_{slope}
\]  
(Equation 2.1)
Figure 2.4: Horizontal Deformation prediction charts from Chew and Mitchell (1994):

a) Reference Case b) (δ/H)_Base c) DI_{L/H} d) DI_{H} e) DI_{slope}
2.1.2.3 Christopher (1993)

An alternative is an empirically derived chart (Figure 2.5), originally published in FHWA-RD-89-043 (Christopher et al. 1990) and later updated in (Christopher 1993). It was developed by analysing a range of segmental block wall case studies, which were extended by numerical modelling. As discussed in Table 2.1, the chart is included as guidance in AASHTO LRFD Bridge Design Specification (AASHTO 2012) covering reinforced soil in highways. It suggests construction deformation can be estimated based on GRS slenderness (L/H), but only relating to the segmental block faced GRS and does not consider construction method type.

![Empirically developed chart by Christopher (1993)](image)

**Figure 2.5: Empirically developed chart by Christopher (1993)**

2.1.2.4 Giroud et al. (1989)

The analytical method developed by Giroud et al. (1989) is based on linearly interpolated maximum strain ($\varepsilon$) between both ends of the reinforcement and the location of maximum strain. This maximum strain, suggested as between 1% and 10%, is entered into a simple analytical equation (2.2) to determine the horizontal deformation ($\delta x$) of that layer, where L is reinforcement length:

$$\delta x = \frac{\varepsilon L}{2}$$  
(Equation 2.2)
2.1.2.5 Lee (2000)

Using case studies to validate a numerical model, Lee (2000) primarily considered facing stiffness, and its effect on deformation. Both wrapped and structural facing GRS were assessed against an ‘initial GRS composite modulus’, $E_{comp}$. This is a combination of factors for Reinforcement stiffness ($J$), spacing height ($s_v$) and soil stiffness ($E_s$), as defined by the following equation:

$$E_{comp} = \frac{1}{s_v} + E_s$$  \hspace{1cm} \text{(Equation 2.3)}

This can be used to determine the normalised maximum face ‘deflection’ in Figure 2.6, which is analogous to deformation. There is no indication as to where or when this occurs in the construction process.

![Face Deflection Model by Lee (2000)](imageURL)
2.1.2.6 Jewell and Milligan (1989)

Jewell and Milligan (1989) produced an upper-bound analytically based model considering internal deformation of GRS (Figure 2.7). The model is based on 3 zones defined by soil strength ($\phi$) and dilatancy ($\psi$). In zone 1, the reinforcement strain is assumed to be uniformly mobilised. In zone 3, there is assumed to be no reinforcement strength. Finally in zone 2 is a transition between zones 1 and 3.

![Figure 2.7: Horizontal deformation Model from Jewell and Milligan (1989)](image)

2.1.2.7 Wu (1994)

Also referred to as the ‘CTI method’, Wu (1994) developed a simple semi-empirical equation (2.4) for estimating the horizontal deformation of wrapped GRS, based on instrumented case study data and numerical modelling.

$$\delta = \varepsilon_d \left( \frac{H}{1.25} \right)$$

(Equation 2.4)

Where $\varepsilon_d$ is the reinforcement strain design limit, and $H$ is the GRS height, up to a maximum of 9 m. Wu (1994) suggests designers should consider $\varepsilon_d$ between 1% and 3%.
2.1.2.8 Wu et al. (2013)

Wu et al. (2013) extended the analytical model by Jewell and Milligan (1989) by additionally considering facing rigidity in GRS and the interface friction between component. This model proved useful for analytically determining deformation in GRS with segmental block walls but remains to include wrapped faces.

2.1.2.9 Discussion of Existing Guidance

The review highlighted three analytical and six empirical models for estimating horizontal deformation in GRS. Of these, only four (1b, 2, 3 and 6) account for deformation at the face, not only internal deformations. Although two thirds of the models (1a, 1b, 5, 6, 7, and 8), differentiate for wrapped face GRS, only two (1b, 6) consider deformation at the face. Of these two methods, the former is limited to the performance of an individual case study. While the latter is an analytical model, based on a traditional understanding of horizontal earth pressure, later shown in Section 2.3.1.1 that poorly represents the performance of reinforced soil.

Six of the existing guidance calculate deformation in terms of GRS height. This assumes an inherently linear relationship, which may be excessive when considering lower than expected earth pressures observed in the lower half of wrapped GRS (Section 2.3.1.1). This method also fails to capture the highly non-uniform deformation distribution in wrapped GRS.

Although a wide number of factors are considered in the models, none of them differentiate between construction techniques, such as the formwork. They are limited to high to medium strength soils, typically $\phi$ above 30°.
2.2 INVESTIGATORY TECHNIQUES FOR HORIZONTAL DEFORMATION IN GRS

Aside from design guidance, there is a large body of investigatory research on the deformation performance of GRS. Generally, these investigations fall into 3 groups: Monitored structures (Duijnen et al. 2014; Onodera et al. 2004), Controlled Models (Ruiken et al. 2010; Bathurst et al. 2009; Ehrlich et al. 2012) and Numerical Models (Ehrlich and Mirmoradi 2013; Liu et al. 2009; Kibria et al. 2010; Yang et al. 2012). The state-of-the-art understanding for these three methods is discussed in Section 2.2.1, Section 2.2.2 and Section 2.2.3 respectively.

2.2.1 MONITORING OF GRS CASE STUDIES

In order to measure deformations that can occur as described in Section 1.1.3, monitoring devices can be placed or used in three areas (Figure 2.8): on the face (1), inside (2) or surrounding the GRS (3).

![Figure 2.8: Typical GRS Measurement Locations](image)
Previous monitoring programmes have focussed on implementing instrumentation inside and behind the GRS at positions 2 and 3. This is typically done by embedding inclinometers (Vaslestad et al. 2012), extensometers (Bathurst et al. 2009; Benjamim et al. 2007), strain gauges (Won and Kim 2007) and fibre optics (Yashima et al. 2009). Devices like these are very precise (±0.01 m), although they are limited to a pre-selected location (Bussert 2012).

Monitoring face deformation during construction is difficult because instrumentation can only be attached or observe the face, after it has been released from formwork. Therefore missing a period of its performance. Although photogrammetry through glass panelled sides (Lee et al. 2010) can been used in controlled laboratory models to monitor a pre-selected cross section.

Horizontal deformation on the face, position 1 (Figure 2.8), is often monitored by a mixture of surveying equipment, traditionally using total stations (Benjamim et al. 2007) and Linear Variable Differential Transformer (LVDTs; Bathurst et al. 2009). While total stations have been the traditional method for measuring civil engineering structures for the last 20 years (Busert 2012), surveying multiple points is slow compared to Terrestrial Laser Scanners (TLS). While a total station projects a single beam at a target using a phase shifted laser, a TLS uses a rotating mirror at high speed and moves automatically, allowing the device to scan a large field of view in a short time space. TLS have been used to assess geotechnical structures such as in highway embankment monitoring (Miller et al. 2008). Prior to this research project, they had not been used to survey a GRS (Section 5 in Paper 2).

2.2.1.1 Case Study Databases

The monitoring data from individual case studies is difficult to compare to other GRS, with many variables involved. In order to spot trends some researchers have compiled case study data into databases (Lee and Wu 2004; Pachomov et al. 2007; Mitchell and Zornberg 1995 and Allen and Bathurst 2002).
Although some of these have been analysed to develop semi-empirical design guidance (Allen and Bathurst 2015), none of them currently consider construction effects on deformation.

### 2.2.2 Laboratory Modelling

Unlike case study monitoring, laboratory modelling allows a greater deal of control. Researchers typically design and create representative model GRS, to isolate the most appropriate variables. Much of the same equipment used in monitoring (Section 2.2.1) can be used in a controlled environment. They are often repeated or created in different sections, with slightly varying properties.

Model GRS, can be scaled down, to make the process of constructing multiple models quicker. These smaller models can be used within centrifuges to recreate similar stresses as full sized GRS (Lee et al. 2010). However, scale effects on grain size and geogrid size make it difficult to represent the reinforced soil composite correctly (Viswanadham and König 2004). Although micro models have been used successfully to examine some GRS features, such as failure modes and stresses (Assinder 2004; Rankilor 2006; Lee et al. 2010), they are not particularly suited to considering deformation, spacing height or construction effects, where full-sized models have tended to be preferred (Bathurst and Walters 2000; Ehrlich et al. 2012).

Although laboratory walls have been built outside (Benjamim et al. 2007; Santos et al. 2014) they are open to weather effects, such as rainfall and weathering which are difficult to control. With this in mind, some researchers have created large indoor models (Bathurst and Walters 2000; Bathurst et al. 2009; Ehrlich et al. 2012). However, even these large models have so far been limited to below 4 m high. GRS typically reach up to 10 m high (Duijnen et al. 2014) which is difficult to recreate given the space restrictions.
2.2.3 NUMERICAL MODELLING

As has been demonstrated throughout Section 2.2, research often involves numerical modelling (Hatami and Bathurst 2005; Guler et al. 2007; Alexiew and Detert 2008; Huang et al. 2009; Wu et al. 2013; Mirmoradi and Ehrlich 2015; Yu et al. 2015). One of the main reasons is that laboratory models are time consuming to build. Many researchers choose to extend the range of physical controlled models by using their performance data to validate representative numerical models (Alexiew and Detert 2008; Ehrlich and Mirmoradi 2013). This has been successful in investigating horizontal deformation on the face and within the GRS (Rowe and Ho 1998; Ling and Leshchinsky 2003; Hatami and Bathurst 2005; Kibria et al. 2014), as well as global deformation behind the reinforced soil block (Guler et al. 2007).

Some numerical modelling packages allow staged construction. Mirmoradi and Ehrlich (2014) suggested a procedure to consider compaction related stresses. More information on their research and method is reported in Section 2.3.3 and in Paper 3.

Numerical modelling has not been restricted to high-quality fills. Researchers have previously considered granular soils using relatively simple elastic-plastic models, discussed further in Section 3.2.3 of Paper 3 and Section 4.4.1.2. Some researchers already considered cohesional soils (Guler et al. 2007), but they have so far only been undertaken behind segmental block faced walls, where deformation is much smaller than wrapped GRS (Section 2.3.1.2).

2.2.3.1 Future Developments

All of the work described previously has been based on two dimensional FE modelling. However there are modelling programs such as FLAC 3D, that have been used to access 3D effects (Bhattacharjee and Krishna 2012). In additional new finite particle programs such as PFC²D are set up to model individual soil particles. These have so far only been capable of
modelling parts of structures, rather than full sized GRS, due to limited computed power (Wang et al. 2014).

### 2.2.4 Discussion on Investigating GRS

Throughout this section, a range of techniques have been highlighted. Monitored deformation data is very useful and can involve a number of monitoring devices, inside and outside the GRS. However traditional methods of surveying are slow and lack spatial resolution. Particularly with wrapped GRS, it is not possible to monitor deformation, particularly at the face, throughout the construction stages, because formwork restricts the view of wrapped faced structures. Although there are a wide-range of case studies with performance data available (Section 2.2.1.1), it is difficult to compare two or more case studies, as there are many variables involved, which are difficult to control on real construction sites.

A popular solution amongst other researchers is to isolate single variables in controlled laboratory modelling (Section 2.2.2). There a number of advantages over monitoring, such as the ability to consider a cross-sectional view using a glass panel, allowing the profile of the GRS to be viewed throughout and after construction. However it is still slow and time consuming to build large physical models, which the complex reinforced soil mechanism requires. Therefore some researchers have used numerical models to extend studies, by systematically assessing of variables. However these numerical models need to be validated against measured performance data.
2.3 CURRENT UNDERSTANDING OF HORIZONTAL DEFORMATION

Although there is an appreciation for SLS checks in existing design standards (BS 8006, EBGEO), procedures to assess them are poorly defined (Section 2.1.1), leaving designers to consider other design guidance (Section 2.1.2). Some of this is now over 25 years old and recent advances in understanding offer potential improvements. This section covers the latest understanding in the following areas: Monitored performance of case studies (Section 2.3.1); Reinforced composite and reinforcement spacing (Section 2.3.2); Construction effects (Section 2.3.3) and Marginal fills in GRS (Section 2.3.4).

2.3.1 MONITORED PERFORMANCE OF GRS

Monitored case studies, regularly show horizontal deformation typically ranges from 10 mm to 200 mm (Bathurst and Walters 2000, Bathurst et al. 2006, Santos et al. 2014). While monitored maximum strain levels in geogrids tend to be in the region of 1% to 2% (Allen and Bathurst. 2002). This is far from the post-construction strain limits applied in some design documents such as BS 8006 (Section 2.1.1.2; British Standards Institute 2010). Part of this may stem from designer’s often using conservative estimates for soil and material strength, whilst considering extraordinary loading. However research suggests there are other aspects of GRS, responsible for their improved performance, which remain to be codified.

2.3.1.1 Existing Earth Pressure Theory

One of the main criticisms with the current analytical models for predicting deformation (Table 2.3: Giroud et al. 1989; Jewell and Milligan 1989; Wu et al. 2013), are that they are based on traditional soil mechanics stresses, such as Rankine (1857) earth pressure theory (Barnes 2010). According to this theory, for a linear increase in vertical stress with depth, there is a linear
increase in active earth stress, \( K_{agh} \). The horizontal earth pressure data from a collection of case studies (Figure 2.9, Pachomow et al. 2007), appears to show it curtailed, in lower half of most of the GRS.

![Horizontal Earth Pressure Graph](image)

**Figure 2.9: Horizontal Earth Pressure in GRS (Translated from Pachomow et al. 2007)**

This observation is echoed by four monitored GRS (Yang et al. 2012). Using earth pressure pads, they revealed the current design theory was excessive below the highest layer, with horizontal pressure reducing towards the base of the GRS. Following this body of research, a reduction in lateral earth pressure in the lower structure has been adopted Germany’s reinforced soil design code, EBGEO (2011), as reported in Section 2.1.1.4.

The differences in earth pressure, have been explained in-part by Ruiken et al. (2010), who also observed geogrids reduce the horizontal pressure in GRS. Beginning with a barrel shaped model, without reinforcement, the authors demonstrated a reduction in lateral earth pressure by adding multiple layers of reinforcement. This effect has been attributed the reinforced soil composite, discussed further in Section 2.3.2.

### 2.3.1.2 Facing Rigidity

The facing type and propping method is an important characteristic of a GRS. It has implications for the shape, magnitude and timing of deformation. In wrapped face GRS,
deformation is often higher than in segmental block faced GRS, although both are considered equal in current analytical design. Onodera et al. (2004) compared the geogrid strain distributions of four full sized walls over a period of 12 years. Generally, higher strain was measured in the flexible GRS than in the stiffer GRSs. The geogrid in the most flexible walls, featuring wrapped faced walls, exhibited a trapezoidal distribution, with maximum strain appearing in the middle of the structure, reducing to a nominal strain at the face. Other strain gauge data confirms this result (Bathurst et al. 2006), even though current design codes do not consider facing type.

This difference is also highlighted by a comparison of two 3.6 m high full scale GRS by Bathurst et al. (2006). While the segmental block faced wall deformed by 30 mm, its wrapped faced equivalent deformed by up to 200 mm. Most (65%) occurred in between reinforcement layers in the form of bulging, which was resisted by the rigidity of the segmental blocks or steel mesh. A similar study by Ehrlich and Mirmoradi (2013) came to the same conclusion, with much higher displacement measured at the face of the wrapped GRS (60 mm), than the segmental block wall (20 mm).

2.3.1.3 Long Term Performance

According to current design codes (Section 2.1.1), deformation occurs throughout the service life of a GRS due to creep. However, strains in the full sized walls monitored by Onodera et al. (2004) were seen to either stabilise or decrease over 12 years. This observation is backed by strain gauge and survey data from three wrapped GRS case studies (Benjamim et al. 2007; Alexiew and Detert 2008; Ehrlich and Mirmoradi 2013) that showed no significant increase in strain (<1%) over the long-term, compared to strain and face deformation occurring during and shortly after construction.
Current codified SLS design procedures focus on long-term creep strength and stiffness (Section 2.1.1.2). However confined reinforcement creep tests (Franca and Bueno 2011) suggests reinforcement creep strain is not as significant when confined as when tested in-air. Confined geogrids and geotextiles were observed to reach creep failure in double the time of conventional creep testing.

2.3.2 REINFORCED SOIL COMPOSITE AND REINFORCEMENT SPACING

Current analytical methods only consider reinforced soil using the Simple Method (Allen and Bathurst 2002). This uses geogrid or soil properties individually, rather combined composite properties because they are easier to measure. When used in design, these properties underestimate the performance of GRS which have been seen to exhibit improved mechanical behaviour than predicted (McGown et al. 1993, Bussert 2010, Wu et al. 2013). Reinforced soil features increased shearing resistance and young’s modulus, creating a stiffer material featuring lower deformation than suggested by the Simple Method.

In order to include the effect of these enhanced mechanical properties, Wu et al. (2013) adapted suggested adding an ‘apparent cohesion’ component for ULS analysis. This is predicated on reinforcement providing additional confinement to the soil mass, which is theoretically inversely proportional to reinforcement spacing. Wu et al. (2013) also reports that optimum reinforcement spacing for the greatest apparent cohesion is a function of maximum particle size. The verification of ULS analyses using this apparent cohesion concept are encouraging but have yet to be considered for horizontal deformation analysis.

2.3.2.1 Reinforcement Spacing and Deformation

The properties of this reinforced soil composite are thought to be dependent on reinforcement spacing. Bathurst, et al. (2010) compared deformation in a series of full-scale lab tests and
monitored GRSs, of a range of variables. Heavier compaction and increasing reinforcement spacing were both equally significant in increasing construction deformation but the most influential factor on post-construction was reinforcement spacing.

### 2.3.2.2 Scaled Modelling Reinforced Soil Composite

Assinder (2004) used micro-sized GRS models reinforced by wrapped layers of tissue paper, primarily to assess failure mode. Although he observed deformation more than doubled by similar increase in spacing from 44 mm to 88 mm. Similarly a comparison of two other scaled wrap-around GRS (Palmeira and Lanz 1994), found doubling reinforcement spacing led to a 40% increase in maximum deformation, occurring near the top of the wall. Although these hint at the sensitivity of deformation to spacing height, as discussed in Section 2.2.2, modelling at this scale has its limitations.

### 2.3.2.3 Numerical Modelling of Reinforcement Spacing

Spacing has also been studied numerically by Liu (2009), suggesting short-term horizontal movement is dependent on reinforcement spacing and stiffness. The study indicated that at larger spacing, soil strength ($\phi, c$) has a greater influence as soil properties begin to dominate the composite; on the other hand, soil stiffness begins to influence deformation when reinforcement spacing is small. Reinforcement spacing was linearly related to short-term deformation. Increasing height and hence vertical stress, was shown to increase deformation and therefore it can be suggested that deformation is a function of vertical stress and spacing, as also shown by the empirical guidance (Section 2.1.2).
2.3.3 CONSTRUCTION EFFECTS

Existing empirically derived models of deformation consider a wide range of variables. However construction is not currently explicitly considered in published guidance. There are a number of construction related factors including compaction, formwork and technique.

2.3.3.1 Compaction

Research by Ehrlich et al. (2012) considered two levels of compaction, ‘heavy’ (73 kPa) and ‘light’ (8 kPa) for two model GRS. The heavily compacted model featured higher reinforcement stress during construction, but was less susceptible to further deformation under post-construction loading.

Including compaction in analytical or numerical models is difficult. However Ehrlich and Mitchell (1994) proposed a simple model, based on empirical data, where compaction pressure is distributed with depth, so much that it is only capable of affecting a 0.3 m zone under the compactor. This method does not take in to account instances where heavier compaction induces additional compactive effort in the lower layers, or where lighter compaction is achieved near the face. Mirmoradi and Ehrlich (2015) included this compaction model to improve the accuracy of numerical models.

2.3.3.2 Construction Formwork

In order to construct wrapped faced GRS, some form of formwork or facing is required to prevent the localised failure, as described in Section 1.1.2. Permanent formwork (Section 1.1.2), typically in the form of segmental blocks or steel mesh is extensively covered by existing research (Christopher 1993; Chew and Mitchell 1994; Lee 2000; Bathurst et al. 2006; Mirmoradi and Ehrlich 2015). However there have been no similar studies considering different methods of installation for wrapped GRS.
2.3.4 Weak Reinforced Fills

Most of the research mentioned throughout Section 2.3 is focused on high quality granular fills, \( \varphi' > 30^\circ \). Although poorer fills have been used in many monitored structures (Mitchell and Zornberg 1995; Yang et al. 2012; Portelinha et al. 2013; Santos et al. 2014), the inclusion of weaker fills presents a number of challenges to the conventional design of GRS (Section 2.1). The main concerns are typically low strength, poor permeability causing excess pore water pressures (PWP), long term soil particle movement and low interface strength between soil and reinforcement (Christopher et al. 1998). As a consequence of these characteristics, deformation performance is highlighted as a major concern, leading to design codes like BS 8006(2010) currently reinforced fills to high quality classes such as 6I and 6J (Raja et al. 2012). For the purpose of this thesis, marginal fills refer to low quality granular fills, \( \varphi' < 30^\circ \).

Current theory suggests reducing frictional shear strength, \( \varphi \), causes a decrease in interface strength and increase in horizontal active earth pressure, leading to the need for longer reinforcement length, \( L \) and greater tensile strength, \( N_p \) (Bilgin and Kim 2010).

Although numerical modelling has been used widely used to model and investigate the performance of GRS (Section 2.2.3). Many authors have limited their analyses to high strength soils, or in the cases where low shear strength or cohesive soils have been analysed, these have been based on internal performance and did not include face deformation (Allen et al. 2003; Guler et al. 2007).
2.4 SUMMARY OF THE EXISTING RESEARCH

As the review of design practice and knowledge has shown, there are gaps in the understanding of horizontal deformation. This is, in part, due to a preference in the majority of design codes, for a limit-state philosophy, which only assesses the initial stresses in a structure (e.g. reinforcement rupture). Although most design codes acknowledge SLS, there are currently no detailed analytical models outlined (Table 2.1). Section 2.1.1 highlighted a number of alternative analytical and empirical deformation models. Many of the models, outlined in Section 2.1.2, consider similar variables such as soil properties, geometry and reinforcement characteristics. Construction techniques or low-strength soils are not factors currently considered.

Part of the improved performance of GRS, can be explained by the improved characteristics of the reinforced soil composite. The reinforcement spacing is an important variable in reinforced soil. There is a lack of research on the relationship between reinforcement spacing and deformation, particularly in wrapped GRS as highlighted in Section 2.3.2.1. While face rigidity has been observed to have a strong influence on deformation but is not currently considered by models (Section 2.3.1.2).

When investigating the reinforced soil composite, full sized modelling or case study monitoring are often utilised in order to avoid complex scale effects (Section 2.3.2.2). Most of the monitored case studies and modelling have previously focused on post construction deformation. In wrapped GRS, some form of formwork is required, to prevent local failures, and this restricts the ability to monitor deformation at release. However, some researchers have used a glass-sided face, to be able to observe a section of the structure throughout its construction period, although not at full scale.
There are many published case studies monitoring deformation in GRS. Of the ones that monitor face deformation, all of them use traditional surveying equipment which has poor spatial resolutions, relying on pre-determined points to monitor. In recent years, laser scanning technology has begun to be used on geotechnical projects, however they have yet to be used to monitor deformation in GRS.

Numerical modelling has often been used to extend time-consuming laboratory modelling in order to view trends. It has been successfully used to consider GRS with granular reinforced fills and rigid facings (i.e. segmental blocks, panel facing etc.). Although Section 2.2.3.1 highlights the future potential of finite particle modelling was highlighted, these analyses are still in their early stages and no one has yet investigated a full GRS using them.
3 THE RESEARCH METHODOLOGY

The purpose of this section is to outline the research philosophy and methodology adopted for the project in order to achieve the aims and objectives as set out in Section 1.3. The aim of the research is to promote the optimisation of GRS. This is primarily achieved by creating, validating and analysing a new horizontal deformation model. This improves on existing models, highlighted in the literature review (Section 2.1.2), by considering deformation occurring during and after construction using multiple formwork methods.

Research methodology combines research methods and techniques. Methods refer to how the research is undertaken, while techniques refer to the tools used to achieve them. This section discusses the latter, while Section 4 details the work undertaken. Section 3.1 describes the philosophy underpinning the research, while Section 3.2 breaks down the techniques used for each of the work packages.

3.1 RESEARCH PHILOSOPHY

Like most engineering or scientific research, the development of models and measuring of deformation belongs to the positivism paradigm, where reality is singular, not affected by the act of investigation and knowledge is built on the understanding of what is already known (Collis and Hussey 2009).

There are two possible lines of reasoning in engineering research. Inductive research is concerned with the establishing of new theory from the findings of the research. Whereas deductive research is concerned with the verification of proposed theories by observation. The project primarily involves data collection and validation of a numerical model representing a GRS and therefore is deductive.
The purpose of the research is basic as it aims to develop and establish a better understanding of the phenomena, in this case horizontal deformation. In contrast, applied outcomes intend to improve technology using existing knowledge. In some instances, like research and development, both outcomes are used in a cyclical process, however it is beyond the scope of the research to suggest improvements for GRS.

The process followed in order to improve this understanding can either be quantitative or qualitative. While, quantitative research is based on the measurement of definite quantities like height or force, qualitative research considers non-numerical correlations (e.g. eye colour). When investigating elements of the natural world in the positivism paradigm, quantitative methods are typically used.

The data utilised can be classified as either primary or secondary data. Where new data obtained by the researcher is termed ‘primary’ research (e.g. Laser Scanning), while the use of existing data (e.g. literature case study) is said to be ‘secondary’. Although the project involves the collection of primary data, secondary data in the form of case studies were used for other work packages (Table 3.1), to diversify the database for validation of the numerical model.

Due to the multi-faceted nature of this project, the research was divided into a number of work packages, following specific designs. The literature review followed historical design, as it looked at existing research and case studies and presented the current state of practice and understanding. An experimental design was followed to investigate and produce deformation data in laboratory tests. Alongside, additional deformation data was obtained using a terrestrial laser scanner, following a descriptive design. Once deformation data was collected, a numerical model was established and validated considering a simulation design. Finally this device was used to systematically investigate construction technique and reinforced soil quality, analysing the results by means of correlation.
It has been suggested that each work package falls neatly into categories. However, in reality, the distinction between them is often blurred, like the example of research and develop, that often contains both applied and basic research.

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### 3.2 ADOPTED METHODOLOGY

The first task of the research was to break down the research into ‘work packages’ associated with achieving each of the objectives. This section summarises the methods and tools used for the main work packages, which are: Literature review (Section 3.2.1), Laboratory modelling (Section 3.2.2), Laser Scanning (Section 3.2.3), Numerical Modelling (Section 3.2.4) and Systematic Analysis (Section 3.2.5).

#### 3.2.1 LITERATURE REVIEW

The literature review (Section 2.0) was the first stage of the research programme in order to meet Objective 1, which was to reveal the current understanding and state of the art in deformation in GRS and highlight areas of potential investigation. Given the regular publication of research in the field, the literature was reviewed throughout the process in order to keep it up to date.

The review’s findings were used to direct the later work packages, particularly the investigatory experiences of previous researchers in Section 2.2. Numerical modelling was highlighted as a useful option to create and develop a new model, as long as it was validated against extensive
deformation data. Existing deformation (Secondary) data in the form of case studies, collected as part of Section 2.3.1, was used to bolster the performance data for the validation of numerical model. This data however had poor spatial resolution and did not show deformation throughout the construction process, leading to need for work packages involving laboratory testing (3.2.2) and laser scanning (3.2.3), to meet objective 2.

3.2.2 LABORATORY MODELLING

The review of existing case studies revealed a lack of deformation data occurring during construction. In order to validate the representative numerical model, primary deformation data was sought (Objective 2), following a process similar to previous research (Ehrlich and Mirmoradi 2013). A representative section of a wrapped GRS, was built and instrumented, to investigate deformation using photogrammetry through a glass panel. Setting the models in laboratory conditions allowed greater control of the soil and geogrid parameters, as well as the surcharge loading applied via a hydraulic jack.

3.2.3 LASER SCANNING

The laboratory models were limited in height (<0.8 m), and although full scale, on their own could not be used to validate the numerical model. Additional primary deformation data was sought (Objective 2), using a laser scanner to monitor deformation during and after the construction stages. Unlike deformation data found in existing literature (Section 4.3), laser scanning attains data with high spatial resolution, and does not require monitored locations to be pre-specified. To the author’s knowledge laser scanning had not previously been used on GRS but had been for similar deformation analyses on geotechnical structures (Miller et al. 2008).
3.2.4 Numerical Modelling

The deformation data obtained from laboratory modelling (Section 4.1), laser scanning monitoring (Section 4.2) and case studies (Section 4.3), was not comprehensive enough to compare and investigate specific factors such as construction technique. Alternatively, a numerical model, validated by this deformation data, was developed and used to systematically investigate factors controlling wrapped face deformation (Objectives 3 and 4). This is similar to the numerical modelling procedures used by Mirmoradi and Ehrlich (2014).

3.2.5 Systematic Analysis

Once validated, the numerical model was used to evaluate two important factors, not currently considered by other research (Section 2.4). The first, in order to meet Objective 3, assessed the impact on deformation of two construction techniques: Layer by Layer (LL) and Full Height (FH). While the second, in order to achieve Objective 4, considered low strength reinforced soils ($\varphi<30^\circ$), which are not typically included in design guidance (Section 2.1.1), let alone specifically for wrapped GRS.

3.3 Summary

The literature review highlighted an absence of any analytical or empirical models capable of estimating construction deformation in wrapped GRS. In order to develop a specific model considering construction, numerical models need construction deformation data to validate them. Existing deformation data for wrapped GRS case studies was partly used, although these typically have poor spatial resolution and do not show deformation throughout the construction process. This was addressed by obtaining high resolution deformation data using a laser scanner.
during and after the construction stages. This still did not capture deformation during the whole construction process. Therefore laboratory modelling of representative section of a wrapped GRS was used to investigate deformation using photogrammetry. Once validated, the numerical model was used to systematically investigate deformation for two construction methods and low strength reinforced soils. These interlocking activities have been mapped towards their respective objectives and outputs in Figure 3.1.

Figure 3.1: Research Map: Research, Objectives and Context
4 THE RESEARCH UNDERTAKEN

With objective 1 being met with the literature review (Section 2), this chapter summarises the research undertaken to achieve the remaining objectives (objectives 2, 3 and 4) set out in Section 1.3, and highlights the main findings of the project. The literature review (Section 2) and methodology (Section 3), along with Papers 1, 2, and 3 are referred to throughout this section and should be read in conjunction. This section is divided into 5 sections (Table 4.1), in line with the work packages undertaken.

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This chapter has the following structure: Section 4.1 discusses the development of GRS laboratory model testing, featuring a range of reinforcement spacing. Section 4.2 outlines the development and use of laser scanning to measure deformation in GRS. Section 4.3 details the creation of a database containing case studies including deformation data. Section 4.4 discusses the development and validation of a numerical model for the study of deformation during construction. Finally Section 4.5 details analyses of construction methods and low strength soils in GRS, using data obtained from laser scanning, existing literature and numerical modelling.
4.1 LABORATORY MODELLING

The aforementioned literature review (sections 2.1.1), revealed deformation during construction is not considered by current models (Section 2.1.2.9) or literature (Section 2.3.3). Similarly to other researchers (Bathurst and Walters 2000; Bathurst et al. 2009; Ehrlich et al. 2012), it was decided to construct repeatable full-scale models, under controlled conditions, in order to satisfy objective 2, regarding the collection and examination of GRS deformation data. An investigation into geogrid spacing is well suited to a full-scale investigation because the reinforcement mechanism, by its nature, depends heavily on the interaction between geogrid and soil, which is too complex to replicate on smaller scales (Viswanadham and Koenig 2004; Section 2.2.2). These scale effects on grain size and geogrid size would detrimentally complicate the study and limit the transferable value of the work.

Building scaled laboratory models is useful to assess individual variables, as long as other variables can be controlled. The literature review was unable to find suitable deformation data of GRS throughout the construction period (Section 2.2.1). Undertaking part of the study under laboratory conditions, allows the models to be more accurately measured and scrutinised than by only monitoring case studies on live construction sites.

4.1.1 MODEL DEVELOPMENT

Ideally, a full-sized GRS, up to 7 m tall and covering a plan area up to 5 m by 5 m, would be built inside a controlled laboratory environments, similar to Bathurst (2009). However due to the considerable propping required and the time required to assemble and dissemble each model, this was deemed impractical. As a consequence, many of the elements of the rig were sourced locally at Loughborough University.
4.1.1.1 Steel Tank

The main element of the model test rig, was the reinforced steel tank (Figure 4.1), measuring 1.5 m by 1.5 m by 1.0 m (height; running length; width). This tank was particularly useful because it featured a glass panel, allowing the deformation of the model’s cross-section to be tracked throughout construction. Given the box was not capable of housing a 7.0 m high GRS, the model was designed to replicate a small section, using a pressure applied by a rigid steel plate, to simulate that from overlying layers (Ehrlich et al. 2012).
4.1.1.2 Model Design and Simplifications

The maximum height of the GRS model was restricted to 0.8 m to allow for a hydraulic jack to be placed within the frame of the box. Additional load was applied through surcharging, similar to Ehrlich and Mirmoradi (2013). Each model was representative of the lowest 0.8 m of a 7.0 m high GRS (Figure 4.2). It was also necessary for there to be a 0.73 m space at the front of the model to house the measuring devices (Section 4.1.1.6) and formwork (Section 4.1.1.7). Given the length of the tank (1.5 m), the length of each model was restricted to 0.77 m.

![Figure 4.2: Laboratory Model within GRS](image)

The design of each model was deliberately simplified to improve constructability. Their total dimensions for all the models were standardised, so that the height, width and length were 0.8 m, 1.00 m and 0.77 m respectively, to ensure similar vertical stresses.

With total height of all models restricted to 0.8 m, where full layer spacing could not be accommodated, a reduced upper layer was considered. This did not compromise the design of the model as the primary purpose of the upper layer was to create realistic stress conditions, by redistributing the vertical stress, from the stiff load plate to the layer(s) below.
4.1.1.3 Load Plate

To simulate a 7 m high wall, a surcharge pressure of up to 100 kN/m², approximately equivalent to a further 6.2 m of wall, was applied to the highest layer of each model. A hydraulic jack acting against a cross-beam attached to the frame of the box (Figure 4.1) applied the surcharge on to a specifically created rigid load plate. Measuring 0.6 m wide by 1.0 m long, this was placed flush to the back of the tank. The maximum force required by the hydraulic jack to create the correct pressure was 60 kN, which was distributed over the steel beam reinforced load plate. To ensure the correct force was applied by the hydraulic jack, a hydraulic load cell was placed between the jack and the load plate. The load plate is further discussed in Section 4.1.1.6.

4.1.1.4 Sand

To achieve consistency between models, it was necessary to use a uniformly graded (coefficient of uniformity \( C_u = 1.6 \)) sand for repeatability, which was available in large quantity (>2 m³). On this basis, the soil for all the GRS models was a medium sized granular soil called Leighton Buzzard sand. Samples of the sand were mechanically tested by an external laboratory (Environmental Scientifics Group), to determine shear strength, density and particle size.

The particle size distribution of the soil samples (Figure 4.3) was assessed to have an average particle size, \( D_{50} \), of 1.2 mm, in accordance with BS 1377:2 (British Standards Institution 1990a). The dry density of the soil, \( \gamma_d \) measured 16.2 kN/m³ and 14.6 kN/m³ for its hand tamped and uncompacted conditions respectively; this density is relatively low because the single-sized soil contains a greater proportion of voids, even in a dense state, than typical well graded soils. The soil remained dry throughout the study to improve repeatability.

The shear strength of the host sand was independently tested in accordance with BS 1377:7 (British Standards Institute 1990b). Using a direct shear analysis it had a peak frictional shear strength, \( \varphi_p \) of approximately 43° (Figure 4.4). The test samples were initial compacted using
light compaction (5 kPa), in line with the tamped compacting used in the models and subjected to a three normal stresses during shearing 10 kPa, 40 kPa and 100 kPa.

Figure 4.3: Soil Particle Size Distribution Test Results

Figure 4.4: Soil Shear Strength by Direct Shear Test Results
4.1.1.5 Geogrid

To reinforcement requirement for each model was determined according to GRS design code, BS 8006-1 (British Standards Institute 2010). Each model was designed with wrapped layers of Huesker’s Fortrac 35T geogrid (Figure 4.5). This was tested to BS EN ISO 10319:2008 (British Standards Institute 2008) to have short-term tensile strength of 35.0 kN/m and 20.0 kN/m in machine and cross-machine directions respectively, and an average tensile modulus, $EA$ of 350 kN/m.

![Figure 4.5: Geogrid used in Laboratory Model](image)

A comparatively weak ($EA = 20$ kN/m) geotextile layer, Huesker’s TechnoTex 60.6106 BBS DG/T (Figure 4.6), was utilised in the area behind the face, to prevent sand from escaping through the apertures in the primary geogrid. This geotextile was deliberately cut to only cover the face area of each wrapped layer, to limit its contribution to reinforcement.

![Figure 4.6: Geotextile Preventing Face Erosion in Laboratory Model](image)
4.1.1.6 Instrumentation

As the literature review (Section 2.2.1) revealed, there are many devices available to measure deformation of GRS. Confined space and budget limitations resulted in the use of Linear Variable Differential Transformers (LVDTs) and basic photogrammetry.

As discussed in 4.1.1.2, in order to replicate a higher GRS, the model incorporated a hydraulic jack system to increase vertical stress. The force applied by the hydraulic jack was monitored by a load cell was placed between the jack and the load plate. This force was monitored to within ±1 kN of the target force (50 kN).

Each model was also measured by four linear variable differential transformers (LVDTs), recording outward movement of pre-selected single positions on the face of each model. LVDTs provide extremely accurate, capable of measuring movement to a resolution of 0.01 mm, but were limited to four locations, placed after construction. Three LVDTs were arranged to register loading deformation along, or near to, the centre line of the lowest layer of the model (Figure 4.7), where stresses most resembled plane-strain conditions. A further LVDT was located within 100 mm of the glass face to evaluate edge effects and calibrate the photogrammetry.

![Figure 4.7: LVDT Monitoring Locations during Tests](image-url)
In addition to the LVDTs, face deformation was also recorded using a photogrammetry technique, where a series of photos were taken through the glass panel. This measurement technique is unobtrusive and used to monitor the profile and geogrid elements of the model. Using a 14 megapixel GE X400 camera, positioned 2.0 m from the glass panel, images were taken throughout construction and surcharging. With control targets, placed on the glass, images were merged using the engineering drawing program Vectorworks, to track relative changes in profile, and hence deformation between frames. Limiting errors to the resolution of camera, which had a resolution of approximately 1.0 mm². Perspective distortion was also reduced by applying tape measures to the glass panel.

4.1.1.7 Construction

Each model was constructed using a process mirroring that in industry. The construction of a wrapped GRS (Above 45°) requires formwork to restrain the model laterally. Formwork sets the slope angle and prevents failure until the structure becomes self-supporting. Wooden formwork was created specifically for this project in order to fit precisely into the gap between GRS and the tank. The formwork was removed when each structure became self-supporting. The filling of each model followed a stepwise procedure (Figure 4.8) to ensure high quality and consistency. Each 200 mm thick layer was compacted by a hand tamper. Once formwork was removed, the LVDTs were set to monitor further deformation occurring during controlled phases of loading, simulating the construction of a further 5.6 m of structure on top. Once these phases had been concluded, the assembly was dismantled back to its foundation layer and two further GRSs, with different reinforcement spacing, were assembled, monitored and loaded.
Figure 4.8: Construction Process
4.1.2 REINFORCEMENT SPACING VARIABLE

Following the development of a full-scaled modelling procedure, three physical GRS models were constructed and tested for the project: GRS200, GRS400 and GRS600. The only parameter changed between each model was reinforcement spacing height, $S_v$. The base model, GRS400, had a spacing height of 400 mm, which was varied to 200 mm and 600 mm, in models GRS200 and GRS600 respectively (Figure 4.9).

![GRS200, GRS400, GRS600](image)

**Figure 4.9: Design of Laboratory Models: GRS200, GRS400 and GRS600**

4.1.2.1 Measured Deformation

The results from the three laboratory model tests (GRS200, GRS400 and GRS600) are presented in this section. A comparison of photogrammetry measurements (Figure 4.10) revealed the majority of deformation occurred during construction for each of the spacing heights, immediately after the release of formwork. The greatest construction deformation occurred in GRS600 (134 mm), followed by GRS400 (63 mm) and then GRS200 (33 mm). These maximum values do not describe the highly non-uniform distribution as seen in monitored wrapped GRS. The deformation peaks occur approximately at mid-layer, suggesting the majority of deformation occurred as face deformation (Figure 1.5). In comparison deformations at reinforcements layers are small in each model.
The extent of face deformation is seen to increase with reinforcement spacing, reaffirming similar findings by other researchers (Section 2.3.2). Deformation can also be seen to increase marginally with height in two of the three models, as is typical when releasing formwork simultaneously (FH). Only in GRS600 can this not be seen. This most likely due to the lack of a full sized layer above ($S_v = 200$ mm).

![Photogrammetry Profiles for GRS200, GRS400 and GRS600](image)

Figure 4.10: Photogrammetry Profiles for GRS200, GRS400 and GRS600

Once self-supporting, these models were very resistant to the onset of additional vertical loading. The maximum increase in deformation, due to the application of a 100 kN/m$^2$ overburden pressure was much smaller in each of the models: 14 mm in GRS600, 15 mm in GRS400 and 7 mm in GRS200. This was also confirmed by the LVDTs, where post-construction deformation was 6 mm, 3 mm and 1 mm respectively. These low values are the result of pre-determined locations (Figure 4.7) that did not necessary measure the most deformed areas.
The collection of detailed deformation data, particularly the timing of it in construction, meets objective 2 and later used in the validation of the numerical modelling (Section 4.4.2), where the study was extended.
4.2 LASER SCANNING GEOGRID REINFORCED STRUCTURES

In order to meet objective 2, there was a need to obtain further deformation data, in particular from real work GRS. For this to be done, the authors developed a methodology for surveying using a Terrestrial Laser Scanners (TLS). This section and Section 4 of Paper 2 provide details of its development (Section 4.2.1) and use to monitor 3 case studies (Section 4.2.2).

As discussed in Sections 4.3 and 2.2.1, there are many published case studies with deformation data measured from traditional surveying equipment like total stations. Their main drawback is there are relatively few datasets measuring GRS with distinctions between construction and post-construction. Horizontal deformation of GRS, as defined in Figure 1.5, has been typically measured using: surveying techniques, extensometers, inclinometers and strain gauges (Section 2.2.1). A new method was required, that would record the position of a large number of points, during many phases of construction and post-construction.

Terrestrial Laser Scanners (TLS) are of immense potential because of their unobtrusive nature, requiring no markings on the face. Although, TLS have been used to assess geotechnical structures such as in highway embankment monitoring (Miller et al. 2008), to the author’s knowledge, before the project, they had not been used to survey a GRS.

TLS are essentially advanced total stations. Whereas a total station projects a single beam at a target using a phase shifted laser, a TLS uses a rotating mirror at high speed that rotates automatically, allowing the device to scan a large field of view in a short time space, with minimal effort from the operator. Modern TLSs also contain on-board data loggers, human interfaces and on-board cameras, for ease of use and for post-processing visualisation.

As with other surveying devices, accuracy and repeatability are dependent on a number of external factors such as weather, tamping and most importantly reliable control points, which
are outside the area of influence of the engineering structure. High Definition Scanner (HDS) targets were chosen to locate these control points, as they are more accurate (±3 mm) for the Scanstation 2, than black and white (±5 mm) or spheres targets (±5 mm), over a distance of less than 50 m (Kersten et al. 2008).

4.2.1 PROCEDURE DEVELOPMENT

A procedure for use with GRS, was fine-tuned using a field trial on Loughborough University campus. The extent of the field trial, involved a Terrestrial Laser Scanner measuring the position of a timber crib retaining wall on the university’s campus.

The procedure involved the establishment of at least 3 controlled targets placed outside the area of influence of the GRS, that must remain in position throughout the monitoring period. These targets are located by the TLS, before scanning the face of the GRS. A typical TLS, like the one used, is capable of recording the position of 10,000 points per minute. For each stage of construction, the coordinate data of each scan was extracted to a point cloud viewer, *Cyclone*.
7.1, where it was processed before cross-sections were extracted for comparison in Excel. Further details of the procedure can be found in Section 4 of Paper 2 in the Annex.

**Figure 4.12: Laser Scanned Point Cloud of Test Structure, seen in Figure 4.11.**

### 4.2.2 REAL GRS MONITORING

Throughout the project, three real GRS were surveyed, meeting Objective 2 to obtain primary GRS deformation data from a range of structures. This section provides a brief overview of each case study, with further information on case studies 1 and 2 found in Paper 2 in the appendix. All three of these case studies had wrapped faces. While two of them, case studies 1 and 3, built with temporary formwork, case study 2 utilised permanent steel mesh formwork.

#### 4.2.2.1 Laser Scanned Case Study 1

Laser Scanned Case study 1, was a 3.6 m high sloped GRS, running for 40 meters abutted to an ancient wall (Paper 2: Section 5.2). Its structure consisted of seven, 0.6 m thick layers, each 3.6 m long, of Polyester geogrid, with short-term tensile strength of 35 kN/m. A fine-aperture geomesh was incorporated behind the wrap-around to retain the imported gravel fill used for the embankment. The wrapped face was constructed at an angle of 60 degrees, behind a moving
Analysis of Horizontal Deformations to allow the Optimisation of GRS

single layer formwork system. Construction began in September 2013 and was completed in October 2013. Further details of this case study can be found in Section 5.2 in Paper 2.

Figure 4.13: Laser scanned Case Study 1: Detail (Left), Deformation Data (right)

4.2.2.2 Laser Scanned Case Study 2

Laser Scanned Case study 2 was a 6.5 m high GRS, consisted of thirteen geogrid layers of varying length with a spacing of 0.5 m (Paper 2: Section 5.3). Two grades of polyester geogrid were used in the GRS, with short-term tensile strengths of 35 kN/m in the upper layers and 55 kN/m in the lower. Extending for 135 m along a quarry face, it was created from locally-sourced high quality sand and gravel soil with a frictional shear strength, $\phi$ of approximately 35°. Unlike case studies 1 and 3, this GRS was constructed using sacrificial steel mesh formwork, to create the 80 degree slope required. The GRS was constructed between January and February 2014. Further details of this case study can be found in Section 5.3 in Paper 2.
4.2.2.3 Laser Scanned Case Study 3

Finally, a third case study was laser surveyed towards the conclusion of the EngD project. As part of an infrastructure redevelopment of a former forge site, in northern England, reinforced slopes and walls were constructed alongside a river. The embankments on both sides of the river were constructed with uniaxial polyester geogrids, ranging in ultimate tensile strength from 35 kN/m up to 110 kN/m (Figure 4.15). The lower part of the embankment consisted of reinforced slopes with a wrapped face detail, constructed using the layer by layer method. At its highest, the slope consists of ten, 0.4 m thick layers of varying reinforcement lengths. A locally sourced, recycled demolition and construction waste (RDCW), was used as reinforced soil. This graded material had an effective frictional shear strength, $\phi$ of approximately, 38° and a unit weight, $\gamma$ of 20 kN/m$^2$.

For most of its length, a 4.0 m high reinforced segmental block wall, sits above the slope, with similar reinforced soil and arrangement. However for a short 10 m stretch, a bridge seat was cast on top of the slope to house a cast in-situ concrete bridge spanning the river. The bridge was not in place at the time of laser scanning.
A comparison of the two scans, taken when the slope was complete showed the limited deformation that had taken place (Figure 4.16), even as a result of the increase in vertical pressure due to the placement of the reinforced segmental block wall.
4.3 ADDITIONAL GRS CASE STUDIES

In addition to the primary sourced deformation data obtained from laboratory testing (Section 4.1) and laser scanning (Section 4.2), published data from secondary sources was also collected in order to satisfy objective 2 and provide justification for the observed trends in the numerical modelling analysis (Section 4.4.2).

4.3.1 SELECTION CRITERIA

A large number of published GRS case studies were identified by the literature review (Section 2.3.1). However most did not include detailed deformation data or have non-wrapped faces, necessary to facilitate comparison with numerical models. In order to select the most useful and informative case studies, selection criteria were established. The criteria for acceptable GRS case studies were:

- Constructed using geogrid or geotextile as soil reinforcement. Case studies utilising strips or metallic mesh as reinforcement were excluded.

- Each case study had to contain quantitative deformation data, ideally measured externally or close to the face, to enable a comparison of face deformation, according to the definition in Section 1.1.3.

- Should feature a flexible facing detail, such as a wrapped face, which typically feature greater deformation than segmental block wall. A small number of structurally-faced GRS were included for comparison.

- Only steep slopes, with slope angles above 45° were considered. Horizontal Earth pressure acting on the face of a retaining structure reduces for shallower slopes (Wesley 2001).
- GRS models were allowed as case studies as long as they were at full scale. The scaling effects of geogrid reinforced soil make a comparison of different scale models difficult (Viswanadham and Koenig 2004).

Case studies meeting all these criteria were collected using a standard input form (Figure 4.17), containing information on soil, geogrid and geometrical properties. A standardised cross-section for each case study was also created for comparison.

![Figure 4.17: GRS Database Input Form](image)

Case studies were collected and analysed as a component of the literature review (Section 2.2.1) from a range of peer-reviewed academic papers. This review also highlighted many existing case study databases (Section 2.2.1.1), by many different authors (Allen and Bathurst 2002, Lee and Wu 2004, Mitchel and Zornberg 1995). These contain a large collection of monitored GRS with a wide range of properties and interpretations of deformation data. In order to comply with the interpretation in Section 1.1.3, data from some case studies was reinterpreted.
4.3.2 DATABASE OF WRAPPED GRS CASE STUDIES

To understand the performance of GRS in reality, deformation data from as many published case studies was organised into a database. Data was obtained from multiple sources, including major conference proceedings and journal publications, as well as primary data from the laboratory models (Section 4.1.2) and laser scanned GRS (Section 4.2.2). Deformation data complying with the criteria set out in section 4.3. This generated 16 datasets for GRS with wrapped reinforcement faces, from 9 sources (Table 4.2).

<table>
<thead>
<tr>
<th>#</th>
<th>Case Study Name</th>
<th>GRS Type</th>
<th>Source Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-3</td>
<td>GRS200 – GRS400</td>
<td>Test</td>
<td>Section 4.1.2</td>
</tr>
<tr>
<td>4-6</td>
<td>Case Studies 1 – 3</td>
<td>Case Studies/Tests</td>
<td>Scotland et al. 2014, Section 4.2.2</td>
</tr>
<tr>
<td>7</td>
<td>LGA Test</td>
<td>Test</td>
<td>Detert &amp; Alexiew 2010</td>
</tr>
<tr>
<td>8-9</td>
<td>Santos’ RCDW1 and RCDW2</td>
<td>Tests</td>
<td>Santos et al. 2014</td>
</tr>
<tr>
<td>13</td>
<td>RMC – Wall 12</td>
<td>Test</td>
<td>Bathurst &amp; Walters 2000</td>
</tr>
<tr>
<td>14</td>
<td>COPPE – Wall 2</td>
<td>Test</td>
<td>Ehrlich &amp; Mirmoradi 2013</td>
</tr>
<tr>
<td>15</td>
<td>Benjamin – Wall 1</td>
<td>Test</td>
<td>Benjamin et al. 2007</td>
</tr>
<tr>
<td>16</td>
<td>IP3 GRS, Portugal</td>
<td>Case Study</td>
<td>Mendonça et al. 2003</td>
</tr>
</tbody>
</table>

The deformation data was converted to the definitions established in Section 1.1.3 (Face deformation, internal deformation and global deformation etc.) and stored in an excel spreadsheet. The data was recorded as absolute values, rather than normalised with height, as discussed in Section 1.1.3. These case studies cover a wide range of properties and variables, which make direct comparisons difficult.
4.4 NUMERICAL MODELLING

The deformation data obtained from laboratory modelling (Section 4.1), laser scanning monitoring (Section 4.2) and case studies (Section 4.3), was not comprehensive enough to compare and investigate specific factors such as construction technique. A numerical model, validated with deformation data, was developed and used to systematically investigate factors controlling wrapped face deformation (Objectives 3 and 4).

4.4.1 NUMERICAL MODEL AND SET UP

There are a wide range of examples of using numerical models to investigate factors in GRS as discussed in Section 2.2.3. Following these examples, a numerical model was required to replicate the range the case studies used to validate it. Importantly it had to be capable of modelling a staged construction process. This section describes some of the theory of numerical modelling, before Section 4.4.2 details the validation of the model.

4.4.1.1 Finite Element Modelling

This numerical models were created using 2D Finite Element (FE) modelling code, PLAXIS 2D Anniversary Edition v.2 (Plaxis B.V. 2014), herein referred to as Plaxis 2D. The program is specifically adapted for modelling linear geotechnical structures, such as retaining walls, tunnels and embankments, in plane strain or axisymmetric conditions. The program features pre-programmed constitutive models for soil, geogrid and allows staged-construction, where clusters of finite elements can be activated or deactivated to simulate a construction stages.

FE modelling works by discretising a continuum into a number of finite elements, typically triangular in shape, defined by nodes. A simplified example of discretization is displayed in Figure 4.18. At each node, a series of constitutive differential equations are formulated, based on the principles of minimum kinetic energy, equilibrium and compatibility, to explain the
behaviour of the nodes in the primary variable (displacement in geotechnical engineering).

These equations are solved using iterative methods, to calculate the primary variable at each node in the mesh. Values are interpolated between each node and secondary variables, such as stress and strain, are calculated from these values. Modern FE programs, like Plaxis, offer the ability to consider additional nodes within each element to improve interpolation accuracy.

FE is based on four principles: **Equilibrium, compatibility, constitutive behaviour** and **boundary conditions**. Each of these must be true for the model to be valid.

The Equilibrium condition requires a structure to be in balance with internal and external forces acting upon it, at every iteration.

The compatibility condition is the main difference between FE and Finite difference modelling. In FE, the elements must move relative to each other and cannot form voids or overlap, requiring deformations to be consistent across elements. This is not the case in finite difference modelling.

The constitutive behaviour is the mathematical model used for each element. This function can be simple or complex but should be representative of the body it is modelling. As discussed in the literature review (Section 2.2.3) and in the following section, there are a wide range of soil
models available ranging in complexity, but given the complexity of other factors such as geometry, simple soil models are often used.

The boundary conditions are the settings given to the edges of the domain, to represent its interaction. These conditions include fixed or free displacements and should accurately reflect the nature of the modelled structure.

4.4.1.2 Modelling Soil in FE Programs

The behaviour of soil is complex, and is simplified for modelling purposes. A number of potential models exist, each with varying accuracy and complexity. The most widely used for granular soils are: *Mohr-Coulomb (MC)* and *Hardening Soil (HS)*. There are also more complex models such as the *HS model with small strain stiffness (HS Small)*, considered too complex for the current study, given the large deformations expected in wrapped GRS (Section 4.2.2).

In the MC model, the soil behaves as a linear-elastic perfectly plastic material. The initial stiffness, $E_{50}^{ref}$, is elastic, until reaching the MC failure level, $q_f$, determined by shear strength properties ($\phi$ and $c$), upon which it deforms plastically. These properties are ideally determined from direct shear strength testing which better reflect plane strain. It is not currently possible within the program to consider peak and constant shear strength. Although after checking for each model, working strains are typically <1% and hence peak shear strength can be used.

The HS model additionally considers hyperbolic stiffness, through parameters: $E_{50}^{ref}$, $E_{oed}^{ref}$, $E_{ur}^{ref}$, $p^{ref}$ and $m$, which represent secant stiffness, oedometric stiffness, unloading-reloading stiffness, reference stress and a power factor respectively (Figure 4.19). The extra parameters allow unloading/reloading to be taken into account, similar to Mirmoradi and Ehrlich (2015).

The data for $E_{50}^{ref}$ and $E_{oed}^{ref}$ are derived from compaction testing and assumed equal for granular soils, while $E_{ur}^{ref}$ follows the relationship: $E_{ur}^{ref} = 3 \cdot E_{50}^{ref}$ as recommended by Plaxis B.V.
The laboratory soil’s initial stiffness ($E_{50}^{ref}, E_{oed}^{ref}$) was measured during compaction prior to direct shear box tests (Section 4.1.1.4). Meanwhile for the other two validation cases, the NM incorporated stiffness data reported for compacted gravel (Alexiew and Detert 2008). For all cases $p^{ref}$ was taken as 100 kN/m2, following the programmes recommendation and its successful use for similar investigations (Guler et al. 2007; Alexiew and Detert 2008; Huang et al. 2009; Mirmoradi and Ehrlich 2015; Yu et al. 2015). Similarly $m$ was assumed to be 0.5 for granular soils. The sensitivity of these assumptions was evaluated (Section 3.4 of Paper 3) and found to be insignificant compared to that of the parameters investigated. Further details on numerical modelling soil can be found in Section 3.2 of Paper 3.

![Figure 4.19: HS model with small strain stiffness Soil Model (Plaxis B.V. 2014)](image)

4.4.1.3 Geogrid Modelling

Geogrid is a complex planar material with relatively little thickness (<5 mm), compared to the reinforcement spacing (typically 300 – 1000 mm), and feature non-linear stiffness (Figure 4.20). In the numerical model it has been simply modelled using a planar element with perfect elastic-plastic linear stiffness, defined by two parameters: combined area and elastic modulus, $EA$, averaged per meter width and can be obtained solely from tensile test data; as well as $N_p$, representing the plastic threshold. The properties used in each model, appropriate
geogrid selection should ensure that this threshold is not reached, as this would lead to rupture failure (ULS). For each case study this was checked using conventional analysis to BS 8006:2010 (British Standards Institute 2010).

The values adopted for the models were taken from constant loading tensile test data for the products in the case studies. These tests provide stress-strain curves that can be used to derive $E_A$. These traditional ‘in-air’ tests also underestimate shear strength of the geogrid when confined, but they are a widely accepted method currently used by practice (British Standards Institute 2008).

Figure 4.20 highlights the typical non-linear form of short-term geogrid stiffness, along with attempts to find a suitable secant stiffness. Extensive sensitivity study of working strain levels in geogrid, resulted in selecting a 1% secant stiffness modulus for each PET geogrid modelled. PET feature relatively linear stiffness curves (Figure 4.20), where tangential stiffness is similar to secant stiffness. The use of secant stiffness is not always appropriate for modelling geogrid, as it is dependent on the shape of the curve. For polypropylene geogrids, the tangential stiffness may vary widely with the secant stiffness.
Plaxis does not allow relative movement of soil and geogrid elements. It relies on the use of interface elements (Figure 4.21), which simulate how stress and deformation is transferred between soil and geogrid. The interface between soil and geogrid elements, was modelled with a rigid interface element, with no reduction in interface strength (i.e. $R_{inter} = 1.0$), as suggested for geogrids by Mirmoradi and Ehrlich (2015). The rigid interfaces still allow relative movements between soil and geogrid elements, although the contact is not considered a weaker zone than the fill.
This follows the assumption that interface shear resistance is sufficient, so that geogrid pull-out or soil-geogrid sliding does not occur. This assumption is not valid for geotextile reinforcements which typically have lower soil-interface shear strength, as they do not benefit from interlocking (Shukla 2011). All numerical model examples were checked to ensure maximum reinforcement stress levels were below the ULS of pull-out.

4.4.1.4 Geometry and Boundary Conditions

The geometry of each numerical model was created to within 0.1 m of each structure. Identical geogrid numerical elements are used for the reinforced sections and wrapped face feature. In reality this feature extends back in to the fill, by 1.5 m typically. As Plaxis does not allow geosynthetic to geosynthetic contact, a single layer represented the wrap back and overlying reinforced length. This was activated with the infilling of the layer below, creating somewhat more reinforcement than in reality.

The geometry of the models was restricted to the reinforced soil section only, to highlight deformation occurring internally and on the face of each structure. A fixed boundary condition, in both the horizontal and vertical directions, was modelled directly below the base of the GRS.

The compressibility of weak foundation has been shown to influence facing deformation in GRS (Rowe and Skinner 2001), however, in this analysis, all 3 case studies used to validate the model were founded on incompressible or firm ground. To restrict deformation to the reinforced section, a horizontal (x-direction) constraint was added immediately at the back of the reinforced soil zone. Trial modelling of case studies 2 and 3, including backfill and embedment, revealed no noticeable difference (<5%) in deformation at the face. Further discussion on the variables in the development of the numerical model can be found in Section 3.2 of Paper 3.
4.4.1.5 Construction Method Modelling

One of the areas that the NM was innovative, was its use of a staged construction methodology to simulate the stages of construction, found in real GRS.

Due to the limited compaction depth, achieved by compactive equipment, GRS are typically constructed in full or half layers. Upon completion of each layer, compaction is modelled by applying a two-stage load-unload cycle, of opposing vertical distributed loads above and below each layer, as shown in Figure 3 in Paper 3. This method is based on the assumption by Ehrlich and Mitchell (1994), that each wrapped layer is compacted in thin increments (<0.3 m) and that compactive effort throughout the layer is equal, similarly to Mirmoradi and Ehrlich (2014). This method does not take into account instances where heavier compaction induces additional compactive effort in the lower layers, or where lighter compaction is achieved near the face.

Another development of the NM, is the simulation of propping formwork, used to create the wrapped face on GRS (Section 1.1.2). This propping can be either temporary during construction or permanent, as well propping the full structure, or an individual layers. The NM was set up to mimic this process, where deformation is allowed to occur at each of these stages. It is common for a wrapped face GRS to experience most of its deformation during the construction period (Paper 3).

Both full height (FH) and layer by layer (LL) construction methods were modelled using the NM. These were modelled using horizontal constraints on the wrapped face, which were deactivated differently, to simulate the removal of formwork. In FH construction, these are all deactivated simultaneously upon reaching total height. Whereas LL modelling, considers the restraint for each layer is deactivated after the in-filling of an overlying layer.
4.4.1.6 Wrapped Face Deformation

Due to its inherent flexibility, deformation for wrapped faced GRS is generally greater in between reinforcement layers (Section 4.1.2.1). This is sometimes also called bulging. Its zone of its influence reduces with distance from the face and top of the structure (Figure 4.22). Nevertheless, at maximum this zone appears to be approximately equal to the spacing height. Behind this zone the internal deformation is much smaller (<10 mm).

Figure 4.22: Typical Face Deformation Distribution (H: 5 m, L: 3.5 m) showing bulging

Within the reinforced soil block, reinforcement strains are significantly below ULS failures design limits as observed by Allen and Bathurst (2002). When numerically modelled, the typical reinforcement tensile distribution displays a triangular-like distribution, with an exaggerated tail. The maximum tensile force occurs in the lower region of the structure, with maximum strain approximately 0.6% in the example in Figure 4.23, dwarfing average strain (0.1%). Similarly tensile stresses and strain in the wrapped reinforcement facing elements are smaller (< 0.4%) than the reinforced lengths.
4.4.2 NUMERICAL PROCEDURE VALIDATION

To validate the performance of the numerical model, three differing GRS were simulated using the FE program. Their calculated deformations were compared to measured deformation behaviour assessed in the field. The maximum and average magnitudes, distribution and staging of deformations were compared for each case study. These case studies included: the laboratory models (Section 4.1), laser scanned case study 2 (Section 4.2.2) and database case study 7 (Section 4.3). These case studies were selected to represent a variety of geometries and construction methods. Their results were compared by assessing the numerical model’s capability at assessing maximum horizontal deformation and average deformation with height, as well as a qualitative shape assessment. Brief details of each of these case studies is included below as well as in more detail in Paper 3.

4.4.2.1 Validation Case Study 1: Laboratory Model 1

A NM was created to replicate the deformation performance of GRS400, undertaken at Loughborough University, as discussed in Section 4.1. The properties of the reinforcement and soil in the model, were determined from independent testing and used for the model (Table 2 in Paper 3).
The geometry of GRS400 was created to within 0.1 m of the intended geometry, utilising 2648 triangular elements. Following the FH method, all the layers were added in individual steps behind an immovable face. At the completion of each layer, a uniformly distributed load, equal to 5 kN/m², was applied across the crest of the GRS to simulate the small hand tamping equipment. With the addition of the formwork removal phase, as well as one for post-construction loading, this NM consisted of 6 phases.

After running the simulation, the deformation of the NM was compared to the measured results from the laboratory test (Figure 7 in Paper 3). The shape of the NM after construction, was broadly in line with the measured performance of GRS400, with measured average horizontal deformation, 0.024 m, underestimated by the numerical model by 13%; while maximum deformation, 0.046 m, was underestimated by 30%.

4.4.2.2 Validation Case Study 2: Laser Scanned GRS

The second case study to be used to check the performance of the NM method, was the laser scanned case study, reported in Section 4.2.2, Section 5.2 in Paper 2 and Section 3.2 in Paper 3. The FE model of this structure featured a fine triangular mesh with 6795 elements and consisted of 18 construction stages to represent the layer by layer construction. This 3.6 m high GRS case study was useful because it was constructed against an existing masonry wall as a facade, featuring no recorded global deformation. The GRS was founded on a firm sandy clay foundation that in the numerical model was assumed to be immovable for simplicity. An additional horizontal constraint was also added behind the reinforced soil block to prevent global deformation.

The surveyed profile was compared to the modelled deformation profile at the end of construction (Figure 8 in Paper 3). Both profiles showed similar deformation patterns, with higher deformation in the lower half of the GRS, where average deformation was 0.075 m and
0.066 m for the measured and modelled profiles respectively. This is in contrast to the top half, where it was 0.030 m and 0.013 m respectively. Although the cross section of the case study was complex, the NM modelled the deformed profile realistically.

**4.4.2.3 Validation Case Study 3: Database Case Study 7**

A further NM was set up to replicate the deformation performance of case study 7, first reported by Alexiew and Detert (2008) and included in the project’s case study database, discussed in Section 4.3.2. The case was particular relevant because it featured a laboratory controlled GRS, with a vertically wrapped face, consisting of nine 0.5 m thick layers.

Case study 3 was modelled using 20 construction stages using the full height method and was founded on a concrete floor to prevent global movement. Each 0.5 m thick wrapped reinforcement layer was compacted during construction, based on Mirmoradi and Ehrlich (2014), a compacting force of 40 kN/m$^2$ was assumed in the numerical model. The geometry of the numerical model is shown in Figure 6 in Paper 3, and featured a fine mesh with 10797 triangular elements.

After its full height construction, it was surcharged by up to 500 kN/m$^2$. Further details of this case study can be found in Section 3.3 in Paper 3, and in a technical paper by Detert and Alexiew (2010). No measured construction deformation data was given by Alexiew and Detert (2008), so only post-construction deformation data is compared (Figure 7 in Paper 3). The FE model reproduced a deformation pattern similar to the measured, with measured average horizontal deformation, overestimated by 140%, while measured maximum deformation was overestimated by 92%. Further analysis can be found in Section 3.3.5.4 in Paper 3.
4.5 SYSTEMATIC ANALYSIS

In this section, the numerical modelling method, established in Section 4.4, was utilised to create an evaluative FE model, symbolising a typical GRS. This was utilised to evaluate two important factors, not currently considered by other research (Section 2.4). The first of these, discussed in Section 4.5.2, assesses the impact on deformation of two construction methods: Layer by Layer (LL) and Full Height (FH). While the second, discussed in Section 4.5.3, considers low strength reinforced soils (<30°), which are not typically included in design guidance (Section 2.1.1).

4.5.1 GENERALISED NUMERICAL MODEL

Similarly to previous numerical modelling work (Mirmoradi and Ehrlich 2014), with a validated NM process (Section 4.4.2), a Generalised Numerical Model (GNM) was created and systematically assessed, by isolating single variables. This GNM was formed as a simplified version of the validation NM for Database Case Study 7, (Section 4.4.2.3). Its reference properties remained similar (Table 4.3), while geometry was slightly modified for simplicity. The evaluative model was 5.0 m high, consisting of ten 0.5 m spaced reinforcement layers. The reinforcement length, \( L \), was pre-determined to ensure collapse according to the ULS of design standard, BS 8006 (British Standards Institute 2010), would not theoretically occur.

As with the validating case studies, the deformation was restricted to internal and face deformation. Horizontal constraints behind and vertical constraints below ensured the model simulated only deformation within the GRS. More details on the formation of the Evaluative NM can be found in Section 4.1 Paper 3. This base model, was used to consider deformation trends and implications of construction methods (Section 4.5.2) and reinforced fill strength (Section 4.5.3).
Unlike other researchers (Section 2.1.2.9), no link between height and deformation was assumed by normalising measured values. This section compares only absolute results to evaluate the complex relationship.

Table 4.3: Generalised Numerical Model: soil and geogrid parameters (Paper 3)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Soil HS-model</th>
<th>Geogrid Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi$ (°)</td>
<td>Plane strain friction angle</td>
<td>40</td>
<td>1600</td>
</tr>
<tr>
<td>$\psi$ (°)</td>
<td>Dilation angle</td>
<td>10</td>
<td>80</td>
</tr>
<tr>
<td>$c$ (kN/m²)</td>
<td>Cohesion</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>$E_{sg0}^{ref}$ (kN/m²)</td>
<td>Secant stiffness in standard drained triaxial test</td>
<td>110,000</td>
<td></td>
</tr>
<tr>
<td>$E_{oed}^{ref}$ (kN/m²)</td>
<td>Tangent stiffness for primary oedometric loading</td>
<td>110,000</td>
<td></td>
</tr>
<tr>
<td>$E_{ur}^{ref}$ (kN/m²)</td>
<td>Unloading reloading stiffness</td>
<td>330,000</td>
<td></td>
</tr>
<tr>
<td>$p^{ref}$ (kN/m²)</td>
<td>Reference Stress Level</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>$m$ (-)</td>
<td>Exponent of the Ohde/Janbu Law</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>$R_f$ (-)</td>
<td>Failure Ratio</td>
<td>0.9</td>
<td></td>
</tr>
<tr>
<td>$\nu$ (-)</td>
<td>Poisson’s ratio</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>$\gamma$ (kN/m³)</td>
<td>Unit weight (unsaturated)</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>$R_{inter}$ (-)</td>
<td>Strength reduction factor for interfaces</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>$EA$ (kN/m)</td>
<td>Stiffness of PVA geogrid at 1%</td>
<td>1600</td>
<td></td>
</tr>
<tr>
<td>$N_p$ (kN/m)</td>
<td>Ultimate tensile force in geogrid</td>
<td>80</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.24: Generalised Numerical Model: Geometry (Paper 3)
4.5.2 **CONSTRUCTION METHOD COMPARISON**

The literature review (Section 2.1.2) discussed several deformation guides (Table 2.3) that are used in the absence of codified design guidance. However none explicitly consider the choice of construction method and its effect on deformation. This is particularly relevant for wrapped GRS, where the face of the structure has no significant stiffness and must rely on tension to resist lateral earth pressures. These structures require temporary formwork, which can be utilised layer by layer (LL), or full height (FH), as discussed in Section 1.1.3.

The GNM was established to differentiate between procedures used in both methods (Section 4.4.1.5). The GNM was adapted to compare two identical GRS in all but their construction method. In direct comparison (Figure 4.25), the two models display dissimilar profiles at the End Of Construction (EOC).

![Graph showing construction method comparison in the GNM](image)

*Figure 4.25: Construction Method Comparison in the GNM (H = 5.0 m, L=3.5 m)*
Maximum construction deformation in the numerical model using the LL was 50% greater than in the model where formwork was released simultaneously. The location and distribution of these maximum deformation was also different in both models. In the GRS constructed using FH formwork, this maximum occurred in the upper layers (0.044 m), reducing with depth by up to 50% in the lowest layer. While in contrast, the maximum construction deformation in the layer by layer model (0.062 m) occurred in the lowest layers. This reduced approximately linearly by up to 80% in the highest layer. The distribution of construction method can be simplistically summarised by Figure 4.26.

![Figure 4.26: Simplified Construction Deformation Distributions for FH and LL](image)

Both versions of the GNM were deformed non-uniformly where face deformation dominated, with deformation peaks occurring between layers of reinforcement. This is similar to the observed profiles in the laboratory models (Section 4.1.2). When split, face deformation contributed significantly to horizontal deformation in GRS, accounting for 93% and 68% for the LL and FH respectively.

Both models also considered deformation occurring under post-construction loading. This was simulated by subjecting both models to a surcharge of 100 kN/m², acting over the crest of the structure. Further details in Section 4.1 of Paper 3 show over 50% less additional deformation occurred when using the layer by layer method (0.018 m), than using the full height model.
(0.024 m). This provides evidence that the construction method plays an important role in determining face deformation. The layers that underwent greater deformation during construction featured the smallest increases in deformation during loading. Total cumulative (construction and loading) deformation was higher in the layer by layer model (0.074 m) than for the full height structure (0.053 m). More information available in Section 4 in Paper 3.

4.5.2.1 NM Sensitivity to Height and other Variables

Following the simple comparison, a second variable GRS Height (H) was introduced in to the analysis. GRS height was the most considered factor amongst the existing design guidance (Section 2.1.1), inferring a large influence on deformation. The GNM was modified to consider GRS height between 2.5 m and 10 m, by adding or subtracting complete reinforcement layers, 0.5 m thick. Although considered later in a sensitivity analysis, the reinforcement length L did not change throughout. For each height changed variant of the GNM, the maximum horizontal deformation was recorded, resulting in Figure 14 in Paper 3.

Following extensive investigation of construction method, a representational equation for the observed trend was suggested for each method, based on the height of the GRS. For wrapped GRS constructed using the FH and LL methods, maximum construction deformation (δxc) can be estimated by Equations 4.1 and Equations 4.2 below. The equations presented are not height-normalised, as this would imply simple linear relationships passing through the origin, an assumption which Figure 14 in Paper 3 contradicts.

- GRS using the FH method (m): \( \delta_{xc} = \frac{H}{250} + 0.030 \text{ m} \) (Equation 4.1)

- GRS using the LL method (m): \( \delta_{xc} = \frac{H}{250} + 0.060 \text{ m} \) (Equation 4.2)

In addition, the post-construction loading of the GNM was modified to consider loading ranging from 50 kN/m² to 200 kN/m², applied over the crest of the GRS. Both construction methods
showed approximately linear relationships between maximum deformation and applied surcharge loading (Figure 15 in Paper 3). In all cases, the maximum deformation occurred in the highest layers of each model. Following extensive investigation of construction method, simple representative equations were devised, based on the additional loading surcharge ($\Delta\sigma_v$).

For wrapped GRS constructed using the FH and LL methods, the post-construction deformation ($\delta x_{pc}$) can be estimated by:

- GRS using the FH method (m): $\delta x_{pc} = 0.00018 \times \Delta\sigma_v$  \hspace{1cm} (Equation 4.3)
- GRS using the LL method (m): $\delta x_{pc} = 0.00012 \times \Delta\sigma_v$  \hspace{1cm} (Equation 4.4)

The data suggests that GRS built using full height formwork, may deform 50% more, under a post-construction surcharge, than using the layer by layer method.

### 4.5.2.2 Existing Model Comparisons

The construction deformation guidance provided (Equations 4.1, and 4.3) was compared with similar existing guidance (Christopher 1993; Lee 2000 and Bathurst et al. 2002) in Section 4.4.2 of Paper 3. The differences in parameters and case studies on which these models are based make direct comparisons difficult. With that said, Equations 4.1, and 4.3 show a similarly positive linear relationship between $H$ and deformation and predict maximum deformation within the range of the three existing models. There is a particularly strong resemblance between the Bathurst et al. (2002) model and Equation 4.1 for FH. Equation 4.3 for LL predicts greater deformation than any other model for smaller structures (<5.0 m), but drops below the Christopher (1993) and Lee (2000) models for high structures (>6.5 m). All existing guidance underestimates construction deformation for low walls as a result of being tied into height normalised relationships. This may be acceptable for internal deformation, but not for face deformation which can feature significantly in small wrapped GRS (Section 4.1.2.1).
4.5.3 **Marginal Fill Analysis**

Although numerical modelling has been used widely used to model and investigate the performance of GRS (e.g. Hatami and Bathurst 2005; Guler *et al.* 2007; Alexiew and Detert 2008; Huang *et al.* 2009; Wu *et al.* 2013; Ehrlich *et al.* 2012; Yu *et al.* 2015). Many authors have limited their analyses to high strength soils, or in cases where low shear strength or cohesive soils have been analysed, these have been based on internal performance did not include face deformation (Allen *et al.* 2003; Guler *et al.* 2007). This section, extends the use of the established GNM, to analyse the deformation consequence of using lower friction shear strength ($\phi$).

The primary parameter investigated was the angle of internal friction, $\phi$ of the reinforced soil. Using the one-at-a-time methodology, a range of values for $\phi$ were considered: 40°, 35° 30°, 25°, 20°. Reducing $\phi$, causes a decrease in interface strength and increase in horizontal earth pressure (Bilgin and Kim 2010), leading to the need for longer reinforcement length, $L$ and tensile strength, $N_p$. In order to prevent pull-out failure, the reinforcement length was extended to satisfy BS 8006 (British Standards Institute 2010). With the reinforcement properties of the GNM (Table 4.3) and the lowest soil strength considered, the reinforcement length was set to 4.1 m for all 5 cases.

Figure 4.27 shows the deformation profile of each $\phi$-adjusted NM at the EOC. All 5 models featured similar deformation distributions, with maximum horizontal deformation in the highest layers, reducing in the lower layers. The evaluation models with lower shear strength reinforced fills, featured more deformation, with the $\phi = 20^\circ$ featuring, more than double the maximum deformation of the base model ($\phi = 40^\circ$).

The difference is more prominent in the top half of the model. A reduction in soil shear strength in the evaluation model, increased horizontal deformation throughout the profile, in an
approximately linear relationship, although there was a greater, non-linear, increase in the lower layer. In the base model, \((\phi = 40^\circ)\), the deformation profile is approximately triangular, whereas it is visibly more trapezoidal with lower strength reinforced fills.

Figure 4.27: Frictional Shear Strength Evaluation \((H = 5 \text{ m}, \gamma = 20 \text{ kN/m}^3)\)

4.5.3.1 Marginal Reinforced Fills with Excess Pore Water Pressure

Finer soils typically have low hydraulic conductivity, \(k < 10^{-7} \text{ m/s}\), compared to well graded granular soils, \(k < 10^{-4} \text{ m/s}\) (Nowak and Gilbert 2015), which can lead to Pore Water Pressure (PWP) build up, following heavy rain or flooding, increasing instability. The simulation for each of \(\phi\)-adjusted numerical models was extended to consider a draw-down scenario, where the soil within the GRS hadn’t sufficient time to dissipate the PWP inside the GRS, leaving
high excess PWP, acting on the wrapped face. This analysis assumed, although fully saturated, the soil can still be described with effective frictional shear strength parameter, $\varphi'$. Although a highly unlikely scenario, the extreme scenario was used to highlight the importance of drainage in GRS.

Figure 4.28: Pore Water Pressure Evaluation ($H = 5 \text{ m}, \gamma = 20 \text{ kN/m}^3$)

Figure 4.28 shows the deformation profiles of two $\varphi$-adjusted numerical models, one at the End Of Construction (EOC) and another considering additional excess PWP. The averaged deformation in the models with extra PWP is 127% and 56% higher than at the end of construction, when $\varphi=40$ and $\varphi=20$ respectively. In contrast to the EOC profiles, both models with excess PWP feature similar deformation distributions, with maximum horizontal
deformation in the lowest layers, reducing in the higher layers, similar to the triangular distribution of excess PWP. Even the model with highest shear strength soil ($\phi = 40^\circ$), featured high deformation under high excess PWP, approximately doubling maximum horizontal deformation and inverting the deformation distribution from EOC to PWP.

Although total deformation of the GRS with low shear strength ($\phi = 25^\circ$), was greater than for the stronger soil, deformation was only increased by 38% by the excess PWP. This is likely due to large deformation that had occurred during the EOC.

This analysis shows soil strength is less sensitive than an increase in PWP. Although an extreme event, this highlights the need to ensuring good drainage in the fill, as recommended in BS 8006 (British Standards Institute, 2010) and by Zornberg and Mitchell (1996).
5 FINDINGS & IMPLICATIONS

This chapter begins by summarising the main outcomes of the research project (Section 5.1) on advancing reinforced soil design and understanding. As a key requirement of the EngD, the project’s success is measured by its implications for the sponsoring company and wider industry which are discussed in Section 5.2. This is followed by a discussion of the methods and achievements of the project in Section 5.3, before Section 5.4 concludes by highlighting further research and potential improvements in understanding.

5.1 THE KEY FINDINGS OF THE RESEARCH

The literature review compared existing design procedures and highlighted deficiencies in our current understanding (Objective 1). This revealed a lack of SLS design guidance in comparison to comprehensive ULS design procedures (Paper 1). There are no clear procedures for checking horizontal deformation in GRS which instead leaves designers to consider a range of non-codified analytical and empirical models (Section 2.1.2.9). However these vary in complexity and lack consideration for face deformation which is critical in wrapped faced GRS. In particular, deformation during construction and the use of weak granular soils is not included. This led the research to improve the knowledge of these parameters in order to facilitate further optimisations in design.

5.1.1 HORIZONTAL DEFORMATION DATA DURING CONSTRUCTION

In developing deformation guidance for the construction period, data was collected for wrapped faced GRS throughout their construction (Objective 2) using three main techniques: Laboratory modelling, monitoring using laser scanning and collection of document case studies.
5.1.1.1 Laboratory Modelling of GRS

Similarly to Mirmoradi and Ehrlich (2014), full-scale laboratory modelling was used to investigate deformation and reinforcement spacing (Section 4.1). Each model represented the lowest layers of a typical wrapped GRS. Similarly to other laboratory modelling, deformation was accurately monitored post construction using LVDTs and photogrammetry through a glass sided panel. The latter captured deformation data before and after formwork release, while also differentiating between facing and internal deformation, partly achieving objective 2.

Three physical GRS models were constructed and tested for the project: GRS200, GRS400 and GRS600 (Section 4.1.2). The only alteration was the reinforcement spacing height, $S_v$. The base model, GRS400, had a spacing height of 400 mm, which was varied to 200 mm and 600 mm, in models GRS200 and GRS600 respectively.

Construction deformation increased exponentially with reinforcement spacing, with most occurring as bulging in the face. While GRS200 featured up to 33 mm of deformation, GRS600 deformed by up to 132 mm. Each model was subject to a post-construction surcharge, equivalent to a further 6.2 m of reinforced soil sitting above. The increase in deformation for each model was less than 25% that during construction.

5.1.1.2 Laser Scanning of Case Studies

Although deformation data was obtained from the laboratory models, these were of limited scale and hence further case study data was needed to validate the numerical model. This was addressed by obtaining high resolution deformation data using a Terrestrial Laser Scanner (TLS) during and after the construction stages. This project was the first recorded use of a TLS to measure deformation in a GRS.

Throughout the project, three real GRS were surveyed, further meeting Objective 2 to obtain primary deformation data from a range of GRS including, a 3.6 m high slope, a 6.5 m high wall
and a site containing both walls and slopes. All three of these case studies had wrapped faces. Two were built with temporary formwork while another utilised permanent steel mesh formwork. The procedural development and two of these case studies were published in a conference paper (Paper 2).

5.1.1.3 Collecting Further Case Study Data

Further to the deformation data obtained through laboratory and laser scanning, additional data was acquired from published case studies from multiple sources, including major conference proceedings and journal publications. The compliant data (Section 4.3) was collated into a wrapped GRS case study database. This generated 16 datasets for GRS, specifically with wrapped reinforcement faces (Table 4.2).

5.1.2 Numerical Modelling of Horizontal Deformation in GRS

The deformation data obtained from laboratory modelling (Section 4.1), laser scanning monitoring (Section 4.2) and case studies (Section 4.3), was not sufficient to compare and investigate specific factors such as construction technique.

Similarly to Mirmoradi and Ehrlich (2015), a numerical modelling process was developed to include construction inputs such as compaction and a staged construction methodology. This process was validated against deformation data from GRS: the laboratory models (Section 4.1), laser scanned case study 2 (Section 4.2.2) and database case study 7 (Section 4.3). These case studies were selected to represent a variety of geometries and construction methods.

As the most typical GRS, the geometry and properties of Database case study 7 were chosen to form the basis of a Generalised Numerical Model (GNM). This was required in order to compare two often-used formwork processes: full height (FH), layer by layer (LL) as well as low shear strength soils.
5.1.3 **Horizontal Deformation During Construction**

Following its validation, the GNM was used to systematically investigate deformation for both two wrapped GRS construction methods (Objective 3): Layer by Layer (LL) and Full Height (FH). Construction techniques were shown to influence deformation shape, producing GRS with markedly different deformation distributions (Figure 4.25).

- Construction deformation is not uniform for either method. It peaks in between reinforcement layers and varies with height.

- Total and construction deformation were greater for LL than FH, with maximum deformation being up to 50% larger when using the former construction method.

- Maximum deformation for LL occurred near the bottom of the wall, with deformation reducing with height in a triangular distribution by up to 80% in the highest layers.

- Maximum deformation for FH was observed near the top of the wall and reduced with depth by up to 50% in the lowest layer.

- Face deformation contributed significantly to horizontal deformation in GRS, accounting for 93% and 68% for the LL and FH respectively.

Both versions of the GNM included stages to consider post-construction loading, where a uniformly distributed load of up to 200 kN/m² was applied over the crown. The construction method was observed to influence further deformation during post-construction loading:

- Post-construction loading deformation was 50% lower in LL than in FH, because the face had stiffened to resist the significant deformation that occurred during construction.
• Post-construction deformation was observed to be applied approximately uniformly in across all layers for LL and FH. As a result the distribution of total deformation tends towards uniformity under surcharge loading.

Along with the construction methods, secondary parameters were isolated and considered using a one-at-a-time procedure: GRS height \((H)\), reinforced fill shear strength \((\phi)\), reinforcement stiffness \((EA)\) and post-construction surcharge \((\Delta\sigma_v)\). GRS height was most sensitive of these parameters and was observed to be approximately linearly correlated to horizontal deformation as other analytical models suggested (Section 2.1.2).

A height dependant chart (Figure 4.25) and equations (Equation 4.1, 4.2, 4.3 and 4.4) were produced to estimate construction and post-construction deformation in GRS for the given set of parameters. The modelling procedure and analysis were published (Paper 3) in a leading geotechnical journal, *Geosynthetic International*.

**5.1.4 Horizontal Deformation Considering Low Strength Soils**

The GNM was adapted to systematically investigate deformation for lower strength granular soils than currently recommend by design codes (Objective 4) in order to promote a potential optimisation. The model considered frictional shear strengths \((\phi)\) ranging from \(30^\circ > \phi > 20^\circ\) are typical of site won fills. These are much lower than typical strengths for engineered fills \((\phi > 30^\circ)\). Section 4.5.3 reported the sensitivity of deformation to these lower shear strengths:

• Deformation in wrapped GRS was inversely related to frictional shear strength. A decrease in the shear strength of the reinforced fill resulted in an approximately linear increase in maximum construction deformation. The GNM adapted to consider \(\phi = 20^\circ\) featured more than double the maximum deformation of the base model \((\phi = 40^\circ)\).
• The deformation distribution for each $\varphi$-adjusted NM was approximately similar, with maximum deformation occurring in the upper layers of the GRS, reducing in the lower layers. This deformation profile becomes more uniform for lower shear strength fills. In addition to lower shear strength, site-won fills, may also exhibit lower permeability ($k < 10^{-7}$ m/s). This extends the time for pore water pressure to dissipate following a flood. In order to illustrate the influence of this feature on deformation, a post-flood situation was considered by the GNM (Section 4.5.3.1):

• The excess PWP acting on the face, increased the deformation throughout the structure (Figure 4.28). The averaged deformation in the models with excess PWP was 127% and 56% higher, than at the end of construction, considering $\varphi' = 40$ and $\varphi' = 20$ respectively.

• The deformation distributions for both high and low strength soils was altered as a result of the excess PWP. In order to resist the additional pressure, the deformation increase was so great in the lowest layers, that the triangular distribution was inverted, with a minimum occurring in the upper layers.

This analysis suggests deformation is less sensitive to soil shear strength than the excess PWP associated with poorly draining site won fills. Although an extreme event was considered, this highlights the need to ensuring good drainage in the fill, as recommended in BS 8006 (British Standards Institute, 2010) and by Zornberg and Mitchell (1996).
5.2 INDUSTRIAL IMPLICATIONS AND IMPACT

The EngD differs from a traditional PhD as much of it takes place with strong connection with the construction industry. As a result it is expected to have an immediate applicable impact on the wider industry (Section 5.2.1) and sponsoring company (Section 5.2.2).

5.2.1 IMPLICATIONS FOR WIDER INDUSTRY

The UK government is leading a sustainability drive to reduce materials wastage, improve design efficiency and reduce associated greenhouse gas emissions in the construction industry (Department for Business, Innovation and Skills 2013). One strategy is to reuse local soils as fills in GRS, replacing the need for high quality granular fills such as classes 6I and 6J. This change would reduce excavated soil going to landfill, negate the need for high quality fill and lower associated greenhouse gases emitted in its transportation (Raja et al. 2015; Waste and Resource Action Programme 2010). However this strategy is currently limited by clauses in most design codes due to a perceived uncertainty regarding increased deformation (Christopher et al. 1998; Raja et al. 2012), which is allied by a lack of deformation guidance in design codes (Section 2.1).

The research reported herein has helped to quantify these concerns by enhancing the understanding of deformation in wrapped GRS. The development of relationship equations (Section 4.5.2) provide industry with methods of estimating magnitudes and distributions of deformations in wrapped GRS. These can be used as a supplement to existing GRS deformation models, mostly intended for segmental block faced walls. By investigating the sensitivity of reinforcement spacing, construction method and reinforced fill strength, industry can be reassured that deformations are predictable. Their effects can also be mitigated by design by
reducing reinforcement spacing. Although this investigation was limited to laboratory models, this was seen to reduce deformation, particularly in the zone close to the wrapped face.

During its course, the research innovatively featured tools such as laser scanning (Section 4.2) and FE modelling (Section 4.5) which can be adopted by industry. The laser scanner procedures reported in Paper 2 is an improvement on existing site surveying. It has proved capable of rapidly collecting high spatial density measurements for large geotechnical structures. Meanwhile, Paper 3 established a NM procedure that can be followed to realistically replicate the construction deformation in wrapped GRS, given quality soils and reinforcement data.

The critique and comparison of existing design methods in Paper 1 can be used by industry to highlight areas of compatibility before the proposed harmonisation of European geosynthetic design and its inclusion in the next version of Eurocode 7. The paper also discusses further areas of design optimisation for GRS.

### 5.2.2 Implications for Huesker Limited

As part of the geosynthetic industry Huesker Limited are also in line to benefit from the implications raised in Section 5.2.1. In addition this research project has also provided Huesker with access to guidance, data and expertise to assess a key SLS condition of GRS. Although GRS design is currently dominated by ULS design procedures. SLS is set to become increasingly important, as deformation becomes more critical after optimisations to meet the government’s efficiency and environmental targets (Section 1.1.4). This research project provides Huesker UK with expert knowledge of the implications of optimising reinforcement spacing or reusing weaker soils.

With the formation of a wrapped database, there is now a body of deformation data that can be used to estimate deformation. This database is not exhausted and can be updated following the
publication of further case studies. The existing data has been used to reinforce client confidence in their performance.

The analysis and relationships studied in Section 4.5, have already helped the lead author to win the Young Member’s GeoPrediction competition at the 3rd Pan-American Conference on Geosynthetics in Miami, USA. The worldwide competition tasked entrants to estimate the observed deformation (Figure 5.1), given only geometrical and characteristic details of a monitored case study. The main author’s entry used a combination of similar case studies, previous models and the equations in Section 4.5.2 to realistically predict its deformation. The success of the entry generates confidence in the competency gained.

![Figure 5.1: Prediction of construction deformation for GeoAmericas:](image)

- a) Case study geometry
- b) Prediction and measured deformation

As an off-chute of the project, a design tool was developed, in the program *MathCad*, to analyse GRS strictly according to BS 8006 (2010). This program enhanced the quality of Huesker Limited’s design suggestions, ensuring they are more accurate and require fewer revisions. This was only possible with the additional competency gained during the project.
5.3 CRITICAL EVALUATION OF THE RESEARCH

With all research, there are limitations such as limited resources to explore all variables with significant depth. In turn, this project had inherent simplifications, focused on the most sensitive factors, to maximise the impact and effectiveness of the research.

5.3.1 COLLECTION OF DEFORMATION DATA

In order to investigate deformation in GRS, performance data was collected from laboratory tests, laser scanning and existing case studies (Objective 2).

Given the need to use full scale (Section 3.2.2), although the models were limited to 0.8 m high, an additional 6.2 m of structure above, was modelled using a hydraulic jack applied over a stiff loading plate. The models were built with single sized sand, from which samples were mechanically tested by an external laboratory (Environmental Scientifics Group), to determine shear strength, density and particle size. The laboratory was instructed to undertake only light compaction on the samples, in line with the tamping used in the model, and keeping the sample representative.

The monitoring of construction deformation using photogrammetry, only provided one cross-section of the model, but actions were taken to mitigate the edge effects, such as cleaning the glass panel, and monitoring deformation across the face of the models using LVDTs (Section 4.1.1.6, Figure 4.7). The combination ensured deformation data was captured before, during and after the release of formwork.

Although monitoring real GRS was the most representative form of deformation data collected, there was little control over the parameters used in the GRS, and the number of sites available to scan during the project period was limited to three, however these covered a range of GRS, that met Objective 2. In contrast to the photogrammetry through the glass panel, deformation
was only observed after the release of formwork for each layer, but it was much more detailed with data recorded for every 1 cm$^2$ of the face.

The need to bolster the deformation data for the NM validation, saw the inclusion of existing case studies into a new wrapped GRS database (Section 4.3). This contained a variety of case studies, which individually, with many variables, were difficult to compare. These studies that included deformation data for wrapped GRS, were used as secondary data, which the research took in good faith, but accepts may feature cautious design values.

5.3.2 DEFORMATION MODELLING AND ANALYSIS

The collected data used to validate the numerical model contained simplifications and potential inaccuracies but their effect was limited by using a variety of methods and diverse data sources in the validation of deformation model. The constitutive models used for the reinforced soil and geogrids could add complexity, given more resources. The geogrid and soil were modelled using separately derived parameters (Section 4.4.1.3), known as the simple method (Section 2.3.2). The geogrid was numerically modelled using tensile stiffness properties determined in accordance with current standard practice (British Standards Institute 2008), whereby products are tested in-air. This overlooks geogrid’s improved behaviour when confined in soil (Section 2.3.2; Wu et al. 2013) and hence may over-estimate deformation.

Further refinement of the mesh size of the model was possible, at the cost of slower computing time, but a range of mesh densities were considered and this was found to have little effect <5%, above 150/m$^2$ for the simple soil model used. Specific discussions on the numerical model can be found in Section 5 of paper 3, which expands on geometrical simplifications (5.1.1 in Paper 3), soil modelling errors (5.1.2 in Paper 3) and validity of numerical approach (5.1.3 in Paper 3).
5.4 RECOMMENDATIONS FOR INDUSTRY AND FURTHER RESEARCH

As the industry ponders further efficiencies and a unification of codes, this research has assessed several interlocking aspects of GRS. Although this has illuminated potential optimisations and provided limited analytical conclusions, there still remains a lot of investigation, before SLS design can be considered as thorough as for the ULS.

Further research is needed on the reuse of weaker fills in GRS, but observations in Section 4.1.2.1 and in existing research (Section 2.3.4) have shown deformations can be managed. Strategies such as reducing reinforcement spacing have decreased deformation levels for otherwise identical structures. Further work could quantity the size of the bulging zone behind the wrapped face, as observed in Figure 4.22. This could be expanded to determine if shorter intermediate reinforcements limited to this zone would have the same effect as a full layers. Even so, it should already be acceptable to remove the limitation on reinforced fills in BS 8006:2010 to only high quality fills, albeit with caveats requiring extensive SLS design and deformation monitoring.

The limitations on resources of the project curtailed its scope to considering only the most sensitive factors. With the GNM developed and validated it could be extended to systematically analyse other potential variables in further detail, such as compaction, soil unit weight and reinforcement stiffness. Despite the extensive numerical modelling undertaken, obtaining real deformation data from laboratory testing or site monitoring is preferable. Although time consuming, further full scale investigations considering alternative soil and geogrid specimens would increase confidence in the use of marginal fills and deformation performance.
The numerical modelling accuracy could be improved by a better understanding of properties of the reinforced soil composite zone and its area of influence. FE numerical models currently utilise parameters for soil and geogrid derived separately. Further research could investigate the accuracy of applying ‘apparent cohesion’ (Wu et al. 2013) to FE models to mimic the increased mechanical behaviour of the reinforced soil composite (Section 2.3.2). This could reduce some of the over-estimation of deformation, associated with unrealistically defined reinforcement and soil characteristics (Section 5.3.2).

As highlighted in Section 2.2.3, improvements in computing power will eventually allow entire GRS to be numerically modelled using finite particle programs, which have showed a lot of early promise in modelling the interaction between geogrid and soil (Wang et al. 2014), which current FE models do not allow (Section 5.3.2).
6 REFERENCES


Analysis of Horizontal Deformations to allow the Optimisation of GRS


Analysis of Horizontal Deformations to allow the Optimisation of GRS


References


APPENDIX A  SERVICEABILITY LIMIT STATE DESIGN IN GEOGRID REINFORCED WALLS AND SLOPES  

(PAPER 1)

Full Reference


Abstract

The design of geogrid reinforced walls and slopes, although a well-established science, still contains many unknowns, particularly around long-term serviceability. Serviceability, for walls and slopes, is associated with excessive deformation or damage affecting appearance, maintenance or service life. In most designs, the serviceability limit state is not considered critical. Currently, most serviceability checks do not attempt to determine or prescribe deformation limits on the built wall or slope, but rather impose limits on the theoretical mobilised strains of geogrid reinforcement, considering the unfactored imposed loads. In many cases, these prescribed post-construction allowable strain limits are based on long-term, or accelerated creep testing, undertaken when the geogrid is not interacting with soil. In some situations, designs are grossly over-conservative. This paper reviews the current state of practice, summarising some of the serviceability design issues around geogrid reinforced walls and slopes, with a particular focus on long-term post-construction deformations. The paper goes on to highlight areas of non-conformity in serviceability design, between the major national codes in Europe, assessing their strengths and weaknesses. Additionally, the paper highlights potential areas of on-going and further work that may offer a better understanding of the serviceability limit state of geogrid reinforced soil walls and slopes.

Keywords – Geogrid, Serviceability, Deformation

Paper type – Conference Paper
1. INTRODUCTION

Soil-retaining structures (SRSs) are a solution to stabilise slopes, where unreinforced slope construction is uneconomical or not technically feasible. SRSs prevent backfill soil from assuming its natural slope angle. Geogrid reinforced soil-retaining structures (GRSRSs) provide an economic alternative to mass concrete and other SRSs. GRSRSs typically consist of several components (Figure 1): Geogrid reinforcement; Reinforced soil fill; Retained backfill soil; Foundation soil and an optional facing component, providing local support to the reinforced soil fill (e.g. segmental blocks, concrete panels, wraparound etc.). Serviceability is often overlooked when designing GRSRSs, with the emphasis on ultimate limit state failures. Conversely a report by Koerner and Koerner (2009) found 23 of 82 reported GRSRS failures were considered to have exceeded their serviceability performance limit, by excessively deforming. GRSRSs are often over-conservative because their internal mechanisms are so poorly understood. This paper reviews the current state of SLS design, comparing the UK’s design code with the German counterpart, summarising issues with current understanding and practice; finally making a number of broad recommendations, to improve design to reflect current understanding.

![Figure 1. Typical Components in a GRSRS](image)

2. SLS DESIGN

When designing structures a number of limits are defined, beyond which the structure no longer satisfies design performance requirements. In design codes these limits are broken down into ultimate limit states (ULSs) and serviceability limit states (SLSs). ULSs are generally associated with collapse or structural failure, while SLSs correspond to unacceptable deformations or other types of damage, increasing maintenance requirements or reducing service life. Deformation in a structure can occur during construction or post-construction. Although the former is not considered in this paper, it is widely acknowledge that quality assurance practices, such as good compaction, help reduce its effects. Examples of post-construction deformation failures are displayed in Figure 2.
There is a great deal of non-conformity amongst the various national codes throughout Europe, as currently the Eurocode for geotechnical design, EN 1997 (British Standards Institute, 2004), does not cover the design and execution of GRSRSs, according to the UK national annex (British Standards Institute, 2007a). Instead, design is determined by individual codes, the most common are BS 8006 (British Standards Institute, 2010) and EBGEO (Deutsche Gesellschaft fur Geotechnik, 2011).

Figure 2. Sources of post-construction deformation in a typical GRSRS according to EBGEO (2011).

Figure 3. Serviceability Limit States according to BS 8006 (2010): a) Wall Deformation; b) Settlement.

2.1. BS 8006 (2010)

In the UK, the principal design code for the design of reinforced soil structures is BS 8006 (British Standards Institute, 2010), herein referred to as BS 8006 (2010). The code defines structures with gradients up to 70° as slopes, while steeper structures are defined as walls, designed as vertical structures. The code provides initial dimension guidelines, before assessing the following external ULSs: bearing and tilt failure, forward sliding and overall slope stability; followed by internal ULSs, using the Tie-back wedge method for walls, or well-established slope stability methods (e.g. slip circle analysis), derived from unreinforced structures, for slopes. BS 8006 (2010) recommends SLSs (Figure 3) are checked to ensure the structure will fulfil its function throughout its design life, without the need for abnormal maintenance. SLS analysis,
considers only characteristic dead loads. BS 8006 (2010) recommends checks are performed on
the following SLSs:

2.1.1. SETTLEMENT OF THE FOUNDATION
This limit involves investigating the consolidation of the foundation over the lifetime of the
structure. This can be calculated using conventional soil mechanics approaches, directing the
designer back to EN 1997 (British Standards Institute, 2004).

2.1.2. POST-CONSTRUCTION CREEP OF SATURATED FINE GRAINED SOILS
Determining post-construction creep of saturated fine grained soils analytically is very
complex, consequently consideration should be given to provide good drainage and/or sealing
of the reinforced zone.

2.1.3. GEOGRID POST-CONSTRUCTION CREEP DEFORMATION
BS 8006 (2010) prescribes a limit on the internal post-construction strain occurring between
the end of construction and the end of the design life. This is limited to 1% in walls (non-
abutments) and 5% in slopes. The restricted tensile capacity of the geogrid, $T_\text{CS}$ is obtained using
isochronous load-strain curves (Figure 4), before reducing this value to the SLS design strength
$T_D$ using equation 1.

$$T_D = \frac{T_\text{CS}}{f_m} = \frac{T_\text{CS}}{RF_{\text{ID}}RF_{\text{W}}RF_{\text{CH}}f_s}$$

Where: $RF_{\text{ID}}$ = reduction factor (RF) for installation damage; $RF_{\text{W}}$ = RF for weathering; $RF_{\text{CH}}$
= RF for chemical and environmental effects; $f_s$ = factor of safety for the extrapolation of data.
These factors are determined in accordance with PD ISO/TR 20432 (British Standards Institute,
2007b). SLS design strength is finally checked against the expected geogrid tensile forces,
derunder service loading conditions.

2.1.4. WALL DEFORMATION
BS 8006 (2010) provides descriptive guidance on wall deformation or horizontal movement
suggesting vertical spacing of reinforcements should be limited to prevent local surface failures
such as bulging.

![Figure 4. Typical isochronous curve used for restricted service tensile stress capacity. Adapted from BS 8006 (2010).](image)
2.2. EBGEO (2011)

The German design code, *Recommendations for Design and Analysis of Structures using Geosynthetic Reinforcements*, EBGEO (Deutsche Gesellschaft für Geotechnik, 2011), herein referred to as EBGEO (2011), is based on the *German National Standard for Earthworks: DIN 1054* (Beuth, 2005). EBGEO (2010) starts by assessing ULSs, before considering SLSs which it defines as structural deformations resulting from characteristic dead loads and soil parameters. The code highlights the following SLSs (Figure 2): foundation settlement; internal settlement of reinforced fill; horizontal movement of the front of the structure and face deformation. Each component may be estimated using numerical analysis, empirical data or observational methods, except for the most trivial structures.

2.2.1 HORIZONTAL MOVEMENT OF STRUCTURE

EBGEO (2011) suggests a general analytical method of integrating individual strains to obtain a total horizontal deformation, for given tensile forces in the geogrid layers. The designer can calculate this from the service loading and the load-strain characteristics of an individual geogrid.

2.2.2. SHEAR DEFORMATION

The horizontal movement of the structure will subsequently cause counter settlement at the surface as material is displaced outward. EBGEO (2011) suggests this can be determined using empirical data.

2.2.3. FOUNDATION SETTLEMENT

As with BS 8006 (2010), the German code directs designers to an additional design code for earthworks, DIN 4019 (Beuth, 2005), suggesting GRSRS may act as a flexible load area on the foundation.

2.2.4. REINFORCED FILL SETTLEMENT

EBGEO (2011) proposes most reinforced fill settlement will take place during construction, at least for granular fill, providing some general empirical data for typical settlements.

2.2.5. FACE DEFORMATION

The German code suggests examining the forces present on the face and the subsequent deformation, without giving a detailed design method beyond using the active earth pressure as a reference variable.

2.3. DESIGN COMPARISON

Comparing the two codes, BS 8006 (2010) offers more prescriptive methods for SLS design. The German code is proficient in conventional designs but becomes more difficult for innovative projects, where less empirical information is available. EBGEO (2011) accounts for the possible sources of deformation, more comprehensively than BS 8006, although in most cases it lacks detailed methodologies.

The most notable contrast between BS 8006 (2010) and EBGEO (2011) is in their assessments of horizontal movement. BS 8006 limits the internal post-construction strain of geogrids, while EBGEO suggests integrating the strains in each layer of reinforcement and calculating a total deformation. This assumes the soil and geogrid deform in unison. Both use theoretical mobilised strain values for reinforcement, as reliable data for reinforced soil compatibility is currently unavailable.

EBGEO (2011) does not currently give any guidance on the use of reduction factors (RFs) for SLS design; therefore not detailing the effect that installation and chemicals have on use of
Serviceability limit state design in geogrid reinforced walls and slopes (Paper 1)

Isochrones and subsequently long-term in-service design strength. BS 8006 (2010) applies arbitrary limits on the post-construction strain of geogrid to its two categories of structures. For example, allowable post-construction strain limits for structures with gradients of 69° and 71° are 5% and 1% respectively. Design should assess SLSs such as differential settlement and bulging of the face, determining deformations in units of length, however current analytical methods make it difficult to do this.

3. ISSUES OF UNDERSTANDING

Several reviews have been compiled, monitoring post-construction deformation of GRSs (Allen et al., 2002; Bussert and Naciri, 2008), revealing grossly over-designed structures, where deformations are much smaller than expected. This suggests problems with our current understanding of GRSs.

3.1. COMPOSITE MATERIAL BEHAVIOUR

Current design codes base their analytical methods on the Simple Method (Allen, et al., 2002): using only geogrid or soil properties of reinforced soil, rather than composite properties because they are more obtainable. The composite material displays different material characteristics than unreinforced soil, such as additional confining stress, contributing to extra load carrying capacity (Bussert 2008). Confinement increases soil shearing resistance and young’s modulus, creating a stiffer material and reducing deformations. Deformation compatibility of reinforced soil is not homogenous and is more complex than current methods suggest. The long-term creep reduction of geogrid strength may also be excessive. Franca and Bueno (2011) used pioneering laboratory equipment to confirm a significant reduction in creep in the composite material, compared to the geogrid alone (in-air).

3.2. VERTICAL STRESS DISTRIBUTION

Although the methods for analysing foundation settlement are well-established throughout geotechnical engineering, there have been studies (Yang et al., 2010) to suggest vertical pressure from the reinforced soil acting on the foundation is more complex than our current understanding, depending significantly on the flexibility of facing in the structure.

3.3. LATERAL EARTH PRESSURE

Corresponding to the observed discrepancies in vertical stress (Yang et al., 2010), variations in observed horizontal stresses have also been observed as non-linear and consistently less than expected by current design. This may be explained in-part by Ruiken et al. (2010), who observed that geogrids reduce the horizontal pressure in the soil, but this has yet to be incorporated into designs. They noticed a reduction in horizontal stress as more layers of geogrid were incorporated. For facing deformation design, only the active earth pressure of soil is considered, without any geogrid reducing effects. Therefore the horizontal design pressure acting on the back of the facing is over-estimated.

3.4. REINFORCEMENT STRAIN DISTRIBUTION

Under current design, strain distribution along the reinforcement is considered to be uniform as a result of uniform vertical stress conditions; however as acknowledged in Sections 3.2 and 3.4, non-linear stresses induce a non-linear strain distribution in the reinforcement as observed by Onodera et al. (2004), Bussert and Naciri (2008) and Yang et al. (2010) amongst others. Integrating strain distribution better accounts total deformation of the geogrid, because it more accurately accounts the whole distribution, unlike the limit on strain as used in BS 8006 (2010).
3.5. REDUCTION FACTORS

In both codes, the understanding of RFs can be improved. Currently they both use partial factors that assume loading starts after the reinforcement has been completely degraded. Additionally, RFs are determined individually and subsequently combined. Work by Kongkitkul et al. (2007) suggests this process underestimates long-term strength, as creep and chemical degradation act simultaneously over the lifetime of the structure, which is affirmed by tests (Onodera et al., 2004) on excavated samples which found higher retained strengths, than are calculated by current design.

4. ISSUES OF PRACTICE

A major source of conservatism in GRSRSs results from simplified designs for specification, manufacture and construction, which result in much more geosynthetic reinforcement than required for acceptable performance. Allen and Bathurst (2002) amongst others have called on designers to adopt a more aggressive approach to the selection of materials and reinforcement spacing; closely matching reinforcement strength to demand. However, in reality this is difficult to achieve as geogrid suppliers offer reinforcement strengths in step changes to obtain economies of scale. Additionally geogrid spacing is often dictated by the height of the facing elements adopted.

5. RECOMMENDATIONS

Throughout the review, it has been established that current SLS design is not as comprehensive as ULS design, highlighting many areas where understanding can be improved. Design codes currently use over-simplified methods to design GRSRSs. For SLS design to be improved, analytical models must be updated to accurately represent the forces developed in the reinforced soil, integrating the full strain distribution, as highlighted in EBGEO (2011). Limiting post-construction geogrid strain is not sufficient to calculating deformation. Any updated method should include the properties of the composite material within current technology limitations. Design should also account for deformable and non-deformable facing types that influence how stress is distributed within the structure. There are of course limitations in the current technology in determining accurate properties for soil/grid composite behaviour. This review has highlighted various opportunities to improve the accuracy of these methods, although in turn, these changes will increase the complexity of designs, making them less accessible. Ultimately, the industry will decide where the balance lies between economy and complexity. However, the use of marginal fills in reinforced design solutions, and the benefits this brings for improved sustainability, will be constrained if agreed analysis methods, which accurately predict deformation behaviour, are not available to assess SLSs.

REFERENCES


APPENDIX B  MEASURING DEFORMATION

PERFORMANCE OF GRS USING A TERRESTRIAL LASER SCANNER (PAPER 2)

Full Reference


Abstract:

Geogrid Reinforced Structures (GRS) are inherently flexible and although the design for ultimate limit state is relatively mature, GRS are often defined by their deformation performance, in the serviceability limit state (Koerner and Koerner, 2013). Currently, serviceability design protocol does not determine or prescribe deformation limits for the built wall or slope, but rather imposes limits on the theoretical mobilised strain of geogrid reinforcement.

Current understanding of the principal mechanisms for GRS deformation is weak and often the only way to assess the serviceability of structures is by the observational method. Typically this has been done with external surveying instruments such as total stations or internally using strain gauges, extensometers and inclinometers. Laser scanning has previously been used to measure the serviceability performance of conventional geotechnical structures and slopes and provided useful information (Mechelke et al., 2007) but has not yet been used on GRS. This paper assesses the potential of a Terrestrial Laser Scanner (TLS) to rapidly survey GRS. This assessment covers a range of structures including a 6.5 m high steel mesh faced retaining wall and a 3.6 m wrap faced structure. The measured behaviour obtained from this range of structures demonstrates the importance of facing stiffness on controlling deformations. Terrestrial laser scanning has potential because it is unobtrusive, only requiring lines of sight to the face and does not use targets located on the GRS. The system can be used to measure the position of the GRS face to within a noise range of ±5 mm (Kersten et al., 2008), across a large surface area from a single observation point in minutes. This paper assesses the application of using TLS to measure deformations during construction and in-service and proposes a standard scanning procedure. It also details experience gained surveying GRS constructed with a range of face systems and discusses accuracy and repeatability issues. It concludes with possible implications of using the TLS method for routine monitoring of GRS.

Keywords – Geogrid, Terrestrial Laser Scanner, Deformation

Paper type – Conference Paper
1. INTRODUCTION

Geogrid Reinforced Structures (GRS) are used as a solution to stabilise slopes. GRS prevent backfill from assuming its natural slope angle of repose, providing a potentially economically beneficial and more sustainable alternative to mass concrete and other conventional retaining structures (WRAP, 2010). GRS typically consist of several key components: geogrid reinforcement; reinforced soil fill; retained backfill soil; foundation soil and can include a range of optional facing components, providing local support to the reinforced soil fill (e.g. segmental blocks, concrete panels, wraparound etc.).

As a result of the need to reducing the excessively conservative nature of commonly used GRS designs (Bathurst et al., 2002), monitoring of GRS structures has been widespread since they started to be increasingly used in the 1990’s. Typically this has been undertaken using conventional geotechnical monitoring techniques such as strain gauges, inclinometers and assessing the face of the structure using conventional survey equipment.

Improvements in the scanning speed and mobility of TLSs have been made in recent years and they have been successfully implemented within several areas of geotechnical engineering. However, the authors are not aware of their use in the monitoring of GRS. There are many advantages of using this advanced form of surveying, not least their ability to measure large swathes of a structure in a short space of time, with minimal effort.

This paper presents the case for utilising TLS to measure deformation of GRS and includes the initial results of two monitored GRS, where the scanner has been used successfully to measure performance over a determined time period. Issues of accuracy and repeatability of measurements are discussed.

2. DEFORMATION IN GRS

GRS are, by their nature, flexible structures and as such they deform during their service life. This deformation can be defined as the action of changing shape and is typically measured relative to an outside point of reference.

Typically GRS are considered as 2-dimensional structures in plane strain, as in most cases these structures are laterally continuous, where strain perpendicular to the sloping face is often insignificant. GRS structures tend to deform outwards horizontally from the face as a result of geogrid strain, and vertically due to settlement, consolidation and the displacement caused by the aforementioned horizontal movement.

This particular paper focuses on the horizontal deformation of GRS. Deformation in GRS can occur horizontally due to three dominant mechanisms (Figure 1): GRS Displacement; typically caused by the pressure from the retained fill, resulting in the whole structure moving forward. Internal deformation; resulting from straining reinforcement, and face deformation; specifically bulging in wrapped faced GRS, resulting from strain within the facing element to resist the lateral earth pressure behind. Understanding this movement is critical in determining performance and improving design.
Analysis of Horizontal Deformations to allow the Optimisation of GRS

3. GRS INSTRUMENTATION

Geotechnical engineering involves the interaction of complex soil and structural properties, which make design and performance measurement difficult and the sub-discipline of geosynthetics adds further complexity. The deformation of GRS has been measured since they were first used over 40 years ago, typically using: surveying techniques, extensometers, inclinometers and strain gauges. Generally, these can be used to measure deformation on the face (1), inside (2) or surrounding the GRS (3) as shown in Figure 2. Previous monitoring programs have focussed on implementing instrumentation at positions 2 and 3, whilst accounting for position 1 with traditional surveying techniques or extensometers. This study presents an innovative use of a TLS, which is a form of surveying to monitor the face profile of two individual GRS. The advantage of profiling the face is that the data measured is the maximum movement acting through the structure, as it is a combination of face deformation, internal deformation and external deformation. Monitoring structures in this way, makes it difficult to trace the source of the deformation. However both GRS case studies featured in the report were monitored only in the short-term and both feature firm foundations and robust global stability, so the face deformation mechanism was expected to be the critical mechanism contributing to deformation observed.

Figure 2. Typical Measurement Positions
3.1. SURVEYING TECHNIQUES AND THE DEVELOPMENT OF LASER SCANNING

Surveying, using total stations, has been the traditional method for measuring civil engineering structures for the last 20 years (Bussert, 2012). However, one of the main limitations of surveying with total stations is that it is time consuming to manually reposition the scope to the next target. Although the data from these individual points can be used to assess individual profiles, it is often not viable to obtain more than a small number of accurate face profiles over a limited portion of the GRS and this is a problem in flexible structures such as GRS.

However, there are two alternative branches of surveying instruments that have developed significantly over recent years with the advance of computer technology. These are TLS and photogrammetry. This paper focuses on the assessment of TLS as a tool to measure GRS. Photogrammetry has not been considered in this piece of work because there are issues with establishing control points and accessing higher areas of tall structures as well as requiring complex photo stitching.

3.2. TERRESTRIAL LASER SCANNING (TLS)

A TLS is essentially an advanced form of a total station. Whereas a total station projects a single beam at a target using a phase shifted laser, a TLS uses a rotating mirror at high speed and moves automatically, allowing the device to scan a large field of view in a short time space, with minimal effort from the operator. Modern TLSs also contain on-board data loggers, human interfaces and on-board cameras, for ease of use and for post-processing visualisation.

Similarly to total stations, the TLS featured in this paper uses a time of flight laser scanner, where the distance to an object is calculated based on the time it takes for the pulse of light to reflect off an object and back to the scanner. The on-board computer logs its position in 3D space relative to the scanner. As it does this for the whole structure, it builds up a 3D representation of this data, termed a point cloud. This point cloud essentially contains thousands of individual coordinates equivalent to those obtained using total station surveys.

To the author’s knowledge these devices have yet to be used to survey GRS but have begun to be deployed in other geotechnical area such as in embankment monitoring. Miller et al. (2008) proved the concept of monitoring deformation of two highway embankments using a TLS. The authors conducted two scans over a period of 6 months allowing them to successfully observe deformation between the two scans.

3.3. TLS EQUIPMENT

The laser scanner used in this assessment was Leica’s Scanstation 2 (see Figure 3). The device, although no longer the state of the art in laser scanners, was chosen to present the concept. A previous study by Mechelke et al. (2007) found this TLS to have a noise range of ±5 mm at a distance of 10 m. This level of accuracy is acceptable for the level of GRS deformation expected (>10 mm) by a typical wrapped faced GRS (Duijnen et al., 2012).

As with other surveying devices, accuracy and repeatability are dependent on a number of external factors such as weather, tamping and most importantly reliable control points, which are outside the area of influence of the engineering structure. High definition scanner (HDS) targets were chosen to locate these control points, as Kersten et al. (2008) had shown them to
be more accurate (±3 mm) for the laser scanner, than black and white (±5 mm) or spheres targets (±5 mm), over a distance of less than 50 m.

Figure 3. Typical Laser Scanning Equipment with HDS target in the background

4. SCANNING PROCEDURE

A scanning procedure was developed in a controlled environment at Loughborough University before applying this to active construction sites. Although based on the use of a particular laser scanner, the methodology is adaptable to most TLS. The work detailed in this paper was carried out between the June 2013 and February 2014.

The first activity to be undertaken when surveying a new structure is the setting up of a minimum of 3 reliable and stationary control points in the area around the proposed section of GRS. This study used the head of a nail in the top of a wooden stake, set in the ground away from the GRS as a control point. These control points should be protected to prevent tamping damage between surveys. These control points need to be accessible to allow tripods to stand vertically above the centre points. HDS targets, set up on these levelled tripods, allow the TLS to locate the control points. The distance of the temporary HDS target above the permanent control point in the ground should be obtained by using a tape measure. This value can then be used by the software to transpose the observed HDS target to the position of the permanent ground control point, removing the variable height of HDS target tripods.

The laser scanner should be placed on a levelled tripod within sight of the target face of the GRS at a perpendicular distance of at least twice the maximum height of the GRS. Upon start-up most modern scanners will calibrate themselves before measurement scanning. An initial medium resolution image can be taken by the on-board camera to locate and acquire the control points. Once these control points have been acquired the TLS should be set to scan the GRS, making an effort to scan as much of the face as possible in a single scan at a uniform intensity. This is important as this will ensure complete coverage of the structure. The time required to scan a GRS is dependent on size of the target and intensity of the points measured, but the TLS was set to a scanning speed of 40 m²/min, collecting up to 2,500 data points / m². Greater detail or scanning speed can be achieved by adjusting the intensity of points recorded. More data points allow the user to better statistically reduce any noise recorded and improve
accuracy, but a compromise has to be found between detail and speed. At extremes, the TLS used are capable of scanning 500 m²/min or 1,000,000 data points/m². After undertaking the scan, the control points should be checked again to ensure the TLS’s or target positions have not been disturbed during scanning. Once this is confirmed the laser scanning equipment can be dismantled, leaving only the reference points in their position. Subsequent scans should follow the procedure outlined above, locating and acquiring the same control points. The laser scanner does not need to be in the exact position as previous scans, as long as it has a line of sight to the previously established control points. This procedure can be used as often as the surveyor wishes to develop a more complete assessment of changes in profile. Ideally, a GRS could be surveyed every lift height in construction and every month after construction, however, given the time and accessibility constraints of real construction sites, this was not possible for the two case studies featured in this paper.

5.0. SURVEYING GRS

5.1. LOUGHBOROUGH FIELD TRIAL
A field trial was conducted at Loughborough University to determine a suitable methodology for scanning GRS. The extent of the field trial involved a TLS being used to measure the position of a conventional retaining wall on the Loughborough University campus. The work was carried out in May 2013. After development of the procedure (Section 4), the TLS was taken to the two live GRS construction sites.

5.2. 3.6 M HIGH GRS AGAINST HISTORIC WALL

5.2.1. BACKGROUND
A GRS, was built to protect the base of a listed 400 year old harbour wall. Recent neglect required the wall to be repaired and shored to prevent it from collapse. To facilitate these works, a GRS was built in front of the wall, on top of which a light scaffolding frame was erected to allow access to the wall for repair. The GRS has a maximum height of 3.6 m and runs alongside the wall for 40 meters (Figure 4). The structure was built using 7 wrapped layers, each 3.6 m long, of Polyester (PET) geogrid, with a short-term tensile strength of 35 kN/m. A fine-aperture geomesh was incorporated behind the wrap-around to retain the imported gravel fill used for the embankment. The wrapped face was constructed at an angle of 60 degrees, behind a moving single layer formwork system. Construction began in September 2013 and was completed in October 2013.
5.2.2. SCANNING PROGRAMME

At the time of writing, two scans had been completed of this GRS. The first scan was undertaken in September 2013, during construction, when only 3 out of the 7 layers had been constructed. A further scan was undertaken 1 month after completion of construction in November 2013. In both cases, the scanner was positioned approximately 15 meters from the GRS. Figure 5 presents a photograph of the GRS as well as an example of an intensively scanned section of the GRS. At this intensity the scanner is able to observe local deformation occurring across the wrapped layers and highlighted the problem that real deformations are non-uniform due to local soil conditions.

5.1.3. RESULTS

To compare the deformation in the GRS, cross-sections of the Historic Wall GRS were taken from the point clouds collected by the TLS, which had been overlaid using the control points and exported into MS Excel to produce profiles. The location of the specific cross-section was
chosen by the user. Only the first 3 layers of the GRS have been compared. As shown in Figure 4, there is a further buried layer that is not visible to the laser scanner.

MS Excel was used to calculate the horizontal difference between the two profiles, surveyed during and after construction (Figure 6). Between the two scans, an additional 4 layers were constructed on top of the 4 already in place during the first scan. Based on a dry unit weight of 18 kN/m$^3$, these additional layers equate to a considerable additional overburden pressure of more than 40 kN/m$^2$. The largest deformations occurred at mid-height in the lowest layer observed of the structure, with a magnitude of up to 80 mm. A smaller peak deformation was recorded in the wrapped layer above. These strong peaks in deformation around the mid-height of the wrapped face, suggests the primary mechanism is face deformation, occurring due to an increase in vertical pressure. This deformation, is much greater than precision of the TLS (±5 mm). Other deformation mechanisms: GRS Displacement and Internal Deformation were not evident from these cross-sections.

![Figure 6. Sample profiles of historic wall buttress GRS taken from laser scanned point cloud (left) and calculated deformation between September 2013 and November 2014 profiles (right)](image)

## 5.2. 6.5 M HIGH GRS IN A FORMER QUARRY

### 5.2.1. BACKGROUND

The second laser scanned site is situated in a former limestone quarry which is currently in the process of being turned into a large housing estate. The GRS has a maximum height of 6.5 m and consisted of 13 geogrid layers of varying length with a spacing of 0.5 m (Figure 7). Two grades of PET geogrid layers were used in the GRS, with short-term tensile strengths of 35 kN/m and 55 kN/m. The layers were constructed using a sacrificial steel mesh formwork set at 80 degrees, behind which a small aperture geomesh was laid for local stability. The GRS extends against one of the quarry faces for a length of approximately 135 m, and is formed of a locally won high quality sand and gravel soil with a shear resistance of approximately 35°. The GRS was constructed between January and February 2014.
5.2.2. SCANNING PROGRAMME

At the time of writing, 2 laser scanning surveys had been undertaken. The first scan (Figure 8) was undertaken during construction in January 2014. The TLS was positioned approximately 30 meters from the centre of the GRS. At this distance it was possible to scan a 60 m horizontal section of the GRS in a single scan. At this stage 10 of the 12 layers had been constructed. A follow up scan was undertaken in March 2014 approximately a month after the end of construction.

5.2.3. RESULTS

Figure 9 presents user-selected cross-sections through the former quarry GRS (left) and the resulting deformation (right), calculated in *MS Excel* from raw coordinate point data extracted using Leica’s point cloud software, *Cyclone*, which displays the data surveyed using the TLS. The comparison covers the lowest visible 9 layers of the structure, with an additional layer buried and obscured from the laser scanner’s view.

The two profiles, taken from during and after scanning, distinctly show similarity at the bottom of the GRS but gradually become separated with height. Overall the deformations recorded are
much larger than with the smaller historic wall case study. The greatest deformations were featured in the higher layers of the structure, with a change of over 110 mm, recorded at 4.25 m above ground level. This large amount of deformation is likely to have been caused partly by an increase in vertical (approx. 20 kN/m²) and horizontal pressure resulting from the addition of 2 further layers. The profile differences between the January and March scans suggest that a combination of deformation mechanisms are involved as deformation is not limited to mid-heights of reinforcement layers unlike in the previous case study. Further scans are to be undertaken to reinforce this result and monitor any further deformation occurring post construction.

![Figure 9. Sample profiles of former quarry GRS taken from laser scanned point cloud (left) and calculated deformation between the January and February profiles (right)](image)

6. CONCLUSIONS

This paper has presented the case and a methodology for utilising the advanced surveying technique of TLS, to monitor deformations occurring in the facing profile of GRS. The case studies included in the paper have proven its effectiveness; allowing the user to collect extensive profile data about an entire GRS, with up to 20,000 data points measured in under an hour, unlike traditional forms of surveying which are restricted in collecting a number of pre-selected points. Although laser scanning GRS has a precision of ±5 mm (Mechelke, 2008) and features challenges common for all surveying methods such as requiring line of sight and control points, it is a valuable technique to assess deformation in GRS, which, as shown by the examples, can be as much as 100 mm, particularly during construction.

Although the data collected so far is limited to a pair of scans at two GRS, it is the intention of the authors to continue to monitor the case sites over time and to expand laser scanning to a range of other structures with alternative facing types and construction techniques, amongst other variables, in order to create a comprehensive database of GRS deformation, which can be used to consider design methods, with a view to reduce over-conservatism in design in the future.
The authors wish to thank the EPSRC for providing funding through the Centre for Innovative Construction Engineering (CICE) at Loughborough University and Huesker (UK) Limited for funding and supporting this research project.

REFERENCES


APPENDIX C  MODELLING DEFORMATION DURING THE CONSTRUCTION OF WRAPPED GEOGRID REINFORCED STRUCTURES (PAPER 3)

Full Reference


Abstract

Although geogrids and geotextiles have been successfully used for over a quarter of a century to reinforce soil, there are currently no commonly agreed analytical methods to model their deformation behaviour. The Serviceability Limit State is becoming an ever more important design consideration, as structures are built with increasingly tighter tolerances. While there are many deformation databases and design charts available, providing information and guidance on the sensitivity to certain design variables, these are largely focused on facets such as height, shear strength and geogrid ultimate strength and do not consider construction method. Following a review of existing analytical and empirical guidance, this paper presents numerical modelling derived guidance for flexible faced Geogrid Reinforced Structures constructed using cohesionless fill that incorporates installation methods. The modelling approach is validated against measured results from three varied case studies, before analysing the changes in deformation distribution resulting from two different construction methods (layer by layer and full height construction). For the conditions analysed, including height of the structure, the lateral deformation resulting from layer by layer construction, was shown to be consistently greater, than for full height construction. In contrast, an analysis of post-construction deformation, for each of the construction methods, found full height construction to be more sensitive to post-construction loading, for the conditions considered. For low wall height structures constructed using the layer by layer method, <5 metres, this study indicates that horizontal face deformations are underestimated by current guidance.

Keywords – Geosynthetics, Reinforced Soil, Deformation, SLS

Paper type – Journal Paper
1. INTRODUCTION

Deformation in Geogrid Reinforced Structures (GRS) is becoming an ever more important design consideration, as structures are built with increasingly tighter tolerances. First geotextiles, from the 1980s, and later geogrids, from the 1990s, offer major technical, sustainable and cost benefits to civil engineering (WRAP 2010; EAGM 2011; Raja et al. 2012). Their design has been historically linked with that for metallic strips and anchors but their performance has routinely suggested they offer completely different performance (Allen and Bathurst 2002). As a result, new design methods are required that consider this composite effect. As a composite structure, combining the benefits of compressively-strong soil and tensile-resistant polymer-based reinforcement, there are a large number of potential factors that can influence the deformation performance of GRS. These include, but are not limited to, geometrical properties such as structural height and reinforcement length, to the long-term creep characteristics of the polymeric reinforcement.

This paper reports on the definitions of deformation and typical ranges; gives a review of the existing analytical and empirical deformation guidance; explains the proposed numerical modelling procedure that can be used to include construction effects; and presents validation against a range of case studies; The numerical model is used to assess two methods of construction, using full height formwork and layer by layer formwork.

2. DEFORMATION IN GRS

2.1. GENERAL

GRS are, by their nature, flexible structures and as such they deform during their service life. This deformation can be defined as the action of changing shape and is typically measured relative to an outside point of reference. Typically GRS are considered as 2-dimensional structures acting in plane strain, where they are laterally constrained in the out-of-plane direction. These structures tend to deform outwards horizontally from the face as a result of geogrid strain, and vertically due to settlement, consolidation and vertical displacement caused by the aforementioned horizontal movement.

This paper focuses on horizontal deformation of GRS, which can be classified, based on where it occurs in relation to the structure (Figure 1), as:

- Face deformation, occurring in wrapped faced GRS as bulging, resulting from straining of the facing elements under lateral earth pressure and vertical deformation.
- Internal GRS deformation, occurring within the body of the structure, primarily from straining reinforcement, under tensile load, or soil shearing.
- Global GRS displacement, occurring outside the region of reinforced soil and can result in the whole structure moving forward/sliding.

This paper focuses on deformation occurring within the reinforced body (a + b) and does not consider global deformation (c), assuming it can be considered by typical reinforced soil design. Distinction can be made between deformation occurring during and after the construction phase. Published data from three wrapped GRS case studies (Benjamim et al. 2007; Alexiew and Detert 2008; Ehrlich and Mirmoradi 2013), where deformation was monitored during and
after construction, indicate that the ratios of maximum construction to maximum post-construction deformation were 3:1, 4:1 and 7:2 respectively, showing that deformation during construction is the dominant period of internal deformation in GRSs.

Figure 1: Horizontal deformation components in GRS

2.2. DESIGN AND DEFORMATION

In Europe the design of GRS is not covered by the Eurocode for geotechnical design, EN 1997 (British Standards Institute 2004). Instead, it remains the responsibility of individual states to recommend design documents, resulting in a plethora of design approaches with varying procedures, safety margins and philosophies. These include BS 8006:2010 (British Standards Institute 2010) in the United Kingdom, EBGEO (Deutche Gesellschaft für Geotechnik 2011) in Germany, Nordic Guidelines for Reinforced Soils and Fills (Nordic Geosynthetic Group 2005) in Scandinavia, and NF P94-270 (Association Francaise de Normalisation 2009) in France. Globally there are similar documents proposing design methods, such as the ‘Design Manual for Segmental Retaining Walls’ (National Concrete Masonry Association 2002), AASHTO (2012) in the United States and Geoguide 6 (Jones 2002) in Hong Kong, amongst others.

These mainly limit-equilibrium based design methods have been shown to be overly conservative in determining realistic forces and deformations in GRS (Allen and Bathurst 2002, Bathurst et al. 2010). However by considering additional factors such as toe embedment, reinforcement stiffness and compaction it is possible to achieve closer agreement (Ehrlich and Mirmoradi 2012; Ehrlich et al. 2013). Explicit Serviceability Limit State (SLS) design methodologies, considering deformation are not typically included within these design documents (Scotland et al. 2012). These methods have been adapted from theories for traditional retaining walls and do not consider the unique characteristics that the combination of soil and geogrid create, as highlighted in research by McGown and Yogarajah (1993), Bussert and Cavanaugh (2010) and Wu et al. (2013). In the absence of analytical models that can explain this composite effect, designers typically use empirically derived charts and relationships, as guidance. Some of the most popular of these deformation models are reviewed in Section 2.3.
2.3. EXISTING DEFORMATION GUIDANCE

There is a wide range of analytical and empirical deformation guidance available (Allen et al. 2015; Chew and Mitchell 1994; Christopher 1993; Giroud et al. 1989; Jewell and Milligan 1989; Lee 2000; Wu 1994; Wu et al. 2013). These have been briefly summarised in Table 1. The most prominent of these is the ‘K-stiffness method’, first presented by Allen et al. (2003) and later updated by Bathurst et al. (2008) and then Allen et al. (2015). It incorporates a number of empirically calibrated parameters that enable the calculation of maximum tension and strain deformation in each layer. These authors used a wide range of case studies and instrumented test walls to develop the method. The scope of the method covers wrapped and segmental block faced walls, by applying a correction factor for wrapped or block faced GRS. Bathurst et al. (2002) also present a supplementary height-normalised chart, displaying measured face deformation data for three instrumented case studies.

Table 1: Overview of existing empirical and analytical deformation guidance

<table>
<thead>
<tr>
<th>Reference</th>
<th>Materials Covered</th>
<th>Validation Data</th>
<th>Facing Type</th>
<th>Location of Deformation</th>
<th>Variables Considered</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) Allen et al. 2015</td>
<td>Geogrid, Geotextile</td>
<td>Case Studies/ NM</td>
<td>Wrapped/ Segmental</td>
<td>Internal</td>
<td>$H/\Delta \sigma, \varphi/c/EA/\gamma/S_v$</td>
</tr>
<tr>
<td>2) Bathurst et al. 2002</td>
<td>Geogrid</td>
<td>Case Studies</td>
<td>Wrapped/ Segmental</td>
<td>Face</td>
<td>$H/\Delta \sigma$</td>
</tr>
<tr>
<td>3) Chew &amp; Mitchell 1994</td>
<td>Geotextile</td>
<td>NM / Case Studies</td>
<td>Segmental</td>
<td>Face</td>
<td>$H/L/EA/S_v/\Delta \sigma$</td>
</tr>
<tr>
<td>4) Christopher 1993</td>
<td>Geotextile</td>
<td>NM/ Case Studies/ Centrifuge</td>
<td>Segmental</td>
<td>Face</td>
<td>$H/L/EA/S_v/\varphi/c$</td>
</tr>
<tr>
<td>5) Giroud et al. 1989</td>
<td>Geogrid, Geotextile</td>
<td>Analytical</td>
<td>None</td>
<td>Internal</td>
<td>$L/\varepsilon$</td>
</tr>
<tr>
<td>6) Lee 2000</td>
<td>Geotextile</td>
<td>NM/ Case Studies</td>
<td>Wrapped/ Segmental</td>
<td>Internal</td>
<td>$H/EA/S_v$</td>
</tr>
<tr>
<td>7) Jewell &amp; Milligan 1989</td>
<td>Geotextile</td>
<td>Analytical</td>
<td>Wrapped</td>
<td>Face</td>
<td>$H/\varphi/\gamma/EA/S_v/\Delta \sigma$</td>
</tr>
<tr>
<td>8) Wu 1994</td>
<td>Geogrid, Geotextile</td>
<td>Analytical</td>
<td>None</td>
<td>Internal</td>
<td>$\varepsilon/H$</td>
</tr>
<tr>
<td>9) Wu et al. 2013</td>
<td>Geotextile</td>
<td>Analytical</td>
<td>Wrapped/ Segmental</td>
<td>Internal</td>
<td>$H/\varphi/\gamma/EA/S_v/\Delta \sigma$</td>
</tr>
</tbody>
</table>

Christopher (1993) and Chew and Mitchell (1994) offer alternative deformation models in the form of charts. Both were originally based on block faced geotextile reinforced structures and cover a wide range of parameters. However neither method accounts for flexibility of the face, which has been shown to play an important factor in contributing to deformation resistance (Bathurst et al. 2006).

Many of the leading deformation models consider similar variables, such as soil properties, geometry and reinforcement characteristics. The range of factors, displayed in Table 1, does not include compaction and construction technique. As this paper shows (Section 4), these can have a considerable effect on total deformation.
2.4. CONSTRUCTION TECHNIQUES

2.4.1. OVERVIEW

One variable not yet included in GRS charts and design guidance is the construction method. Specifically, the construction of wrapped GRS requires some form of lateral restraint or propping, during backfilling and compaction. Typically, there are 3 general propping methods, which provide the lateral restraint required during placement and compaction of fill. These three are: permanent formwork (Figure 2.a), temporary full height formwork (Figure 2.b) or temporary layer-by-layer formwork (Figure 2.c). Permanent formwork, typically in the form of segmental blocks or steel mesh is extensively covered (Christopher 1993; Chew and Mitchell 1994; Lee 2000; Bathurst et al. 2006; Mirmoradi and Ehrlich 2015b) and has therefore not been analysed in this paper.

![Figure 2: Construction methods for wrapped GRS: a) permanent steel mesh formwork, b) temporary full height formwork, c) temporary layer by layer formwork](image)

2.4.2. FULL HEIGHT TEMPORARY FORMWORK

In this case, the GRS is constructed to its full height while laterally restrained behind a full height propped panel. This panel is then released in one action, allowing the structure to deform.

2.4.3. LAYER BY LAYER FORMWORK

Contrasting with full height construction, the structure is built behind localised facing panels, often covering a single layer. These are released locally after subsequent layers are constructed. This means that deformation occurs throughout the construction process. This form of construction is favoured in cases where a full height propping solution is not possible due to costs and construction feasibility.
3. NUMERICAL MODELLING OF GRS

3.1. PREVIOUS NUMERICAL MODELLING

Numerical Modelling software has been successfully used in modelling GRS by many researchers (e.g. Hatami and Bathurst 2005; Guler et al. 2007; Alexiew and Detert 2008; Huang 2009; Wu et al. 2013; Mirmoradi and Ehrlich 2015a; Yu et al. 2015) to investigate a range of parameters that influence behaviour. However none have considered more than one construction method and their effect on the magnitudes of deformation both during and after construction.

3.2. PROPOSED NUMERICAL MODEL

3.2.1. GENERAL

This study used the 2D Finite Element (FE) modelling code, PLAXIS 2D Anniversary Edition v.2 (2014), herein referred to as Plaxis 2D. It has previously been used by many different researchers (Alexiew and Detert 2008; Anubhav and Basudhar 2011; Damians et al. 2015; Ehrlich and Mirmoradi 2013; Guler et al. 2007; Herold and Wolferdorff 2009; Mirmoradi and Ehrlich 2015a), and is commercially available and used in design practice. Plaxis 2D is a FE program, specifically adapted for modelling geotechnical structures such as retaining walls, tunnels and embankments, in plane strain or axisymmetric conditions. The program features pre-programmed constitutive models for soil, geogrid and allows staged-construction, where clusters of finite elements are activated or deactivated to simulate a particular construction sequence.

3.2.3. SOIL MODEL AND INPUT PARAMETERS

The soil for each case study is modelled using a pre-programmed constitute model called the Hardening Soil (HS) Model with Mohr-Coulomb Failure Criteria (Schanz et al. 1999). Unlike a linear-elastic perfectly plastic model, the HS model is elasto-plastic, featuring a hyperbolic strain-stiffness relationship. It also includes compression hardening, to simulate the irreversible compaction of soil. It is defined by parameters: $E_{s0}^{ref}$, $E_{oed}^{ref}$, $E_{ur}^{ref}$, $p^{ref}$, m which are the secant stiffness, oedometric stiffness, unloading-reloading stiffness, reference stress and a power factor respectively.

More complex models, such as the Hardening Soil model with small strain, that allow varied soil stiffness at small strain (below 0.1%), but given typically strains in GRS range from 1% to 2% (Allen and Bathurst 2002), this was unnecessarily complex, as the later sensitivity analysis shows (Section 3.4).

Following Hatami and Bathurst (2005), the shear strength parameters: $\varphi$ and c, friction angle and cohesion respectively, have been obtained from plane-strain tests. The dilation angle of the soil, $\psi$, has been taken to obey the relationship with friction angle, introduced by Bolton (1986). Although defined as a cohesionless soil, a small cohesion value, <1.0 kN/m2, has been incorporated into each model to prevent the initial stress state from being on the tip of the yield surface. The impact of its inclusion has been assessed in Section 3.4.
3.2.3. GEOGRID

Geogrid is a complex planar material with insignificant thickness and non-linear stiffness. In the numerical model it has been simply modelled using a planar element with perfect elastic-plastic linear stiffness, defined by two parameters: combined area and elastic modulus, ‘EA’, averaged per meter width and can be obtained solely from tensile test data; as well as $N_p$, representing the plastic threshold. In design, appropriate geogrid selection should ensure that this threshold is not reached, as this would lead to rupture failure (ULS).

Determination of EA follows the principle of compatibility, as suggested by McGown and Yogarajah (1993), where soil strain and geogrid strain must be apportioned in selecting an appropriate secant modulus. Extensive sensitivity study of working strain levels in geogrid (Section 3.5), resulted in selecting a 2% secant stiffness modulus for each geogrid modelled, based on available tensile test data. The value selected for stiffness should be suitable to the time period considered in design, to account for the action of creep. The case studies included in this paper only consider polyester and polyvinyl alcohol geogrids, that under short-term (< 1 year) working stress conditions feature approximately linear stiffness and do not exhibit strains greater than 2%, partly due to high reduction and safety factors applied in their design process (Kaliakin et al. 2000; Allen and Bathurst 2002). The rheological behaviour of polymeric reinforcement is more pronounced under high strain levels where stiffness softening can occur. Therefore a detailed evaluation of creep and reducing stiffness with time is beyond the scope of this study.

3.2.4. GEOMETRY, BOUNDARY CONDITIONS AND INTERFACES

The geometry of each numerical model was created to within 0.1 m of each structure. The geometry of the models was restricted to the reinforced soil section only, to highlight deformation occurring internally and on the face of each structure. A fixed boundary condition, in both the horizontal and vertical directions, was modelled directly below the base of the GRS. The compressibility of weak foundation has been shown to influence facing deformation in GRS (Rowe and Skinner 2001), however in this analysis, all 3 case studies selected were founded on incompressible or firm ground. To restrict deformation to the reinforced section, a horizontal (x-direction) constraint was added immediately at the back tip of the geogrids. Trial modelling of case studies 2 and 3, including backfill and embedment, revealed no noticeable difference (<5%) in deformation at the face.

The interface between soil and geogrid elements in each case, is modelled rigidly, with no reduction in interface strength (i.e. $R_{inter} = 1.0$), as suggested for geogrids by Mirmoradi and Ehrlich (2015a). This follows the assumption that interface shear resistance is sufficient, so that geogrid pull-out or soil-geogrid sliding does not occur. This assumption is not valid for geotextile reinforcements which typically have lower soil-interface shear strength. Assuming rigid interfaces, does not allow relative movement between soil and geogrid interfaces, and therefore all numerical model examples, need to be checked to ensure maximum stress levels are below the ULS of pull-out. This can be assessed using established design practices as discussed in section 2.2.
3.2.5. CONSTRUCTION METHOD MODELLING

The construction process is modelled using a staged-construction procedure, where after defining the boundaries, the structure is built in full layers, defined by the geosynthetic vertical spacing, behind a horizontal restraint. Upon completion of each layer, compaction is modelled by applying a two-stage load-unload cycle, of opposing vertical distributed loads above and below each layer, as shown in Figure 3. This method is based on the assumption by Ehrlich and Mitchell (1994), that each wrapped layer has been compacted in thin increments (<0.3 m) and that compactive effort throughout the layer is equal, as shown by Mirmoradi and Ehrlich (2015a). This method does not take into account instances where heavier compaction induces additional compactive effort in the lower layers, or where lighter compaction is achieved near the face.

Both full height and layer by layer construction methods are modelled using horizontal constraints on the wrapped face. These are deactivated, differently for both construction methods, to simulate the removal of formwork. In full height construction, these are all deactivated simultaneously upon reaching total height. Whereas, the restraint for each layer is deactivated after the in-filling of a subsequent layer.

![Figure 3: Compaction modelling in numerical model after Mirmoradi and Ehrlich (2015b)](image)

3.3. VALIDATION OF NUMERICAL METHOD

3.3.1. OVERVIEW

To validate the performance of the construction modelling method, three differing GRS were modelled using the FE program. Their calculated deformations were compared to measured deformation behaviour assessed in the field.

3.3.2. CASE STUDY 1: 0.8 M HIGH MODEL USING FULL HEIGHT CONSTRUCTION METHOD

Deformation data from controlled laboratory tests was used to validate the numerical model. The 0.8 m high tests were undertaken at Loughborough University, UK, and consisted of two 0.4 m thick wrapped layers (Figure 4). They were constructed using uniformly-sized sand, which was tested to have the properties displayed in Table 2. They used wrapped layers of polyester geogrid with a maximum tensile strength of 35 kN/m.
Modelling Deformation during the Construction of Wrapped Geogrid Reinforced Structures
(Paper 3)

Each layer was lightly compacted (5 kN/m$^2$) by a hand-held tamper. As the test was constructed in a confined space, the geogrid tail lengths were limited to 1.0 m and their ends were rigidly fixed to the back of the box, to prevent pull-out failure.
The profile of the GRS was measured using photogrammetry through the glass-sided test box at three stages: at full height behind the full height formwork (during construction), at the end of construction (EOC).
The numerical model of this GRS used a fine mesh size, featuring 2648 triangular elements. Its construction consisted of 6 stages that included: The infilling of each layer followed by a compaction stage, until the total height of the structure (0.8 m) was reached. The final stage of construction modelling was the deactivation of the full height horizontal constraint, causing the structure to deform horizontally (and vertically).

![Figure 4: Case study 1 model geometry](image)

Table 2: Case study 1: soil model and geogrid parameters for Plaxis

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Soil HS-model</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varphi$ ($^\circ$)</td>
<td>Plane strain friction angle</td>
<td>43</td>
</tr>
<tr>
<td>$\psi$ ($^\circ$)</td>
<td>Dilation angle</td>
<td>10</td>
</tr>
<tr>
<td>$c$ (kN/m$^2$)</td>
<td>Cohesion</td>
<td>0.5</td>
</tr>
<tr>
<td>$E_{s0}$ (kN/m$^2$)</td>
<td>Secant stiffness in standard drained triaxial test</td>
<td>30,000</td>
</tr>
<tr>
<td>$E_{t0}$ (kN/m$^2$)</td>
<td>Tangent stiffness for primary oedometric loading</td>
<td>30,000</td>
</tr>
<tr>
<td>$E_{u0}$ (kN/m$^2$)</td>
<td>Unloading reloading stiffness</td>
<td>90,000</td>
</tr>
<tr>
<td>$p_r$ (kN/m$^2$)</td>
<td>Reference Stress Level</td>
<td>100</td>
</tr>
<tr>
<td>$m$ (-)</td>
<td>Exponent of the Ohde/Janbu Law</td>
<td>0.5</td>
</tr>
<tr>
<td>$R_f$ (-)</td>
<td>Failure Ratio</td>
<td>0.9</td>
</tr>
<tr>
<td>$\nu$ (-)</td>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
<tr>
<td>$\gamma$ (kN/m$^3$)</td>
<td>Unit weight (unsaturated)</td>
<td>16.4</td>
</tr>
<tr>
<td>$R_{int}$ (-)</td>
<td>Strength reduction factor for interfaces</td>
<td>1.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>EA (kN/m)</td>
<td>Averaged* Axial Stiffness of geogrid at 1%</td>
</tr>
<tr>
<td>$N_p$ (kN/m)</td>
<td>Ultimate tensile force in geogrid</td>
</tr>
</tbody>
</table>

*Value is averaged over 1.0 m out-of-plane width for plane strain calculation
3.3.3. CASE STUDY 2: 3.6 M HIGH GRS USING LAYER BY LAYER CONSTRUCTION METHOD

This GRS was a 3.6 m high structure, consisting of six polyester geogrid layers, constructed at an approximately 65 degree inclination (Figure 5). The GRS used an imported gravel that had properties as displayed in Table 3. It was constructed using a layer by layer method, with each 0.5 m layer being relatively well compacted (40 kN/m2) by a vibrating plate. Each wrapped faced layer was constructed behind temporary wooden panels. (In the numerical model this is simulated by an immovable horizontal boundary). The numerical model of the structure featured a fine triangular mesh with 6795 elements. The GRS was founded on a firm sandy clay foundation that in the numerical model was assumed to be immovable for simplicity. An additional horizontal constraint was also added behind the reinforced soil block to prevent global deformation.

![Figure 5: Case study 2 model geometry (adapted from Scotland et al. 2014)](image)

Table 3: Case study 2: soil model and geogrid parameters for Plaxis (adapted from Scotland et al. 2008)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Soil HS-model</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varphi$ $^\circ$</td>
<td>Plane strain friction angle</td>
<td>35</td>
</tr>
<tr>
<td>$\psi$ $^\circ$</td>
<td>Dilation angle</td>
<td>5</td>
</tr>
<tr>
<td>$c$ (kN/m$^2$)</td>
<td>Cohesion</td>
<td>0.5</td>
</tr>
<tr>
<td>$E^{sec}$ (kN/m$^2$)</td>
<td>Secant stiffness in standard drained triaxial test</td>
<td>30,000</td>
</tr>
<tr>
<td>$E^{tan}$ (kN/m$^2$)</td>
<td>Tangent stiffness for primary oedometric loading</td>
<td>30,000</td>
</tr>
<tr>
<td>$E^{un}$ (kN/m$^2$)</td>
<td>Unloading reloading stiffness</td>
<td>90,000</td>
</tr>
<tr>
<td>$p^{ref}$ (kN/m$^2$)</td>
<td>Reference Stress Level</td>
<td>100</td>
</tr>
<tr>
<td>$m$ (-)</td>
<td>Exponent of the Ohde/Janbu Law</td>
<td>0.5</td>
</tr>
<tr>
<td>$R_f$ (-)</td>
<td>Failure Ratio</td>
<td>0.9</td>
</tr>
<tr>
<td>$\nu$ (-)</td>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
<tr>
<td>$\gamma$ (kN/m$^3$)</td>
<td>Unit weight (unsaturated)</td>
<td>18</td>
</tr>
<tr>
<td>$R_{int}$ (-)</td>
<td>Strength reduction factor for interfaces</td>
<td>1.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Geogrid Model</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$EA$ (kN/m)</td>
<td>Stiffness of geogrid at 1%</td>
</tr>
<tr>
<td>$N_p$ (kN/m)</td>
<td>Ultimate tensile force in geogrid</td>
</tr>
</tbody>
</table>

The profile of the structure was surveyed during and at the end of construction using a terrestrial laser scanner, to survey approximately 10,000 indiscriminate points on the face, inorder to quantify the magnitude and distribution of face deformations, as discussed by Scotland et al.
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(2014). The surveyed profile was compared to the modelled deformation profile at the end of construction.

3.3.4. Case Study 3: 4.5 m High GRS Using Full Height Construction Method

A third case study was modelled using the published details of a laboratory test undertaken by Alexiew and Detert (2008). The structure was a 4.5 m high, wrapped faced geogrid wall and featured 9 layers of 80 kN/m polyester geogrid (Figure 6) and was founded on the laboratory’s concrete floor. The properties of the materials used are displayed in Table 4, which has been adapted from Alexiew and Detert (2008). Outward lateral face deformation of the GRS was monitored using 12 LVDT, as it was tested post-construction, using a load plate. A surcharge of up to 500 kN/m² was applied over a 0.5 m² area, 1.0 m from its face.

![Figure 6: Case study 3 model geometry (adapted from Alexiew and Detert, 2008)](image)

Case study 3 was constructed using the full height method and was founded on a concrete floor to prevent global movement. Each 0.5 m thick wrapped reinforcement layer was compacted during construction, but as no data was available, a compacting force of 40 kN/m² was assumed in the numerical model. The geometry of the numerical model is shown in Figure 6, and featured a fine mesh with 10797 triangular elements.

Table 4: Case study 3: soil model and geogrid parameters for Plaxis (adapted from Alexiew and Detert 2008)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Soil HS-model</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi$ (°)</td>
<td>Plane strain friction angle</td>
<td>40</td>
</tr>
<tr>
<td>$\psi$ (°)</td>
<td>Dilation angle</td>
<td>10</td>
</tr>
<tr>
<td>$c$ (kN/m²)</td>
<td>Cohesion</td>
<td>0.5</td>
</tr>
<tr>
<td>$E_{sg}^{sec}$ (kN/m²)</td>
<td>Secant stiffness in standard drained triaxial test</td>
<td>110,000</td>
</tr>
<tr>
<td>$E_{e0}^{tan}$ (kN/m²)</td>
<td>Tangent stiffness for primary oedometric loading</td>
<td>110,000</td>
</tr>
<tr>
<td>$E_{ur}^{tan}$ (kN/m²)</td>
<td>Unloading reloading stiffness</td>
<td>330,000</td>
</tr>
<tr>
<td>$p_{ref}^{res}$ (kN/m²)</td>
<td>Reference Stress Level</td>
<td>100</td>
</tr>
<tr>
<td>$m$ (-)</td>
<td>Exponent of the Ohde/Janbu Law</td>
<td>0.5</td>
</tr>
<tr>
<td>$R_f$ (-)</td>
<td>Failure Ratio</td>
<td>0.9</td>
</tr>
<tr>
<td>$\nu$ (-)</td>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
<tr>
<td>$\gamma$ (kN/m³)</td>
<td>Unit weight (unsaturated)</td>
<td>20</td>
</tr>
<tr>
<td>$R_{int}$ (-)</td>
<td>Strength reduction factor for interfaces</td>
<td>1.0</td>
</tr>
<tr>
<td><strong>Geogrid Model</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Analysis of Horizontal Deformations to allow the Optimisation of GRS

<table>
<thead>
<tr>
<th>$EA$ (kN/m)</th>
<th>Stiffness of PVA geogrid at 1%</th>
<th>1600</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_p$ (kN/m)</td>
<td>Ultimate tensile force in geogrid</td>
<td>80</td>
</tr>
</tbody>
</table>

3.3.5. NUMERICAL AND MEASURED DEFORMATION COMPARISON

OVERVIEW
GRS profiles from each of the three numerical models, were compared to their measured profiles at the EOC, and under loading (where data was available). These results were compared by assessing each numerical model’s capability at assessing maximum horizontal deformation and average deformation with height, as well as a qualitative shape assessment.

CASE STUDY 1: 0.8 M HIGH MODEL USING FULL HEIGHT CONSTRUCTION METHOD
Comparing both measured and modelled profiles (Figure 7) there are similarities in deformed shape for both layers. Measured average horizontal deformation, 0.024 m, was underestimated by the numerical model by 13%; while maximum deformation, 0.046 m, was underestimated by 30%.

CASE STUDY 2: 3.6 M HIGH GRS USING LAYER BY LAYER CONSTRUCTION METHOD
The profile of the numerical model of Case study 2, was compared to measured results described by Scotland et al. (2014) in Figure 8. Average horizontal deformation, of the face, was underestimated by the numerical model by 32%, while maximum deformation, 0.129 m, was underestimated by 13%. Both profiles also showed higher deformation in the lower half of the GRS, where average deformation was 0.075 m and 0.066 m for the measured and modelled profiles respectively. This is in contrast to the top half, where it was 0.030 m and 0.013 m respectively.

![Figure 7: Case study 1 measured and modelled comparison](image-url)
CASE STUDY 3: 4.5 M HIGH GRS USING FULL HEIGHT CONSTRUCTION METHOD

Figure 9, displays the modelled and measured profiles for case study 3. No measured construction deformation data was given by Alexiew and Detert 2008, so only post-construction deformation data is compared in Figure 9. Measured average horizontal deformation, 0.015 m, is overestimated by the numerical model by 140%, while measured maximum deformation, 0.031 m, is overestimated by 92%.

3.4. LOCAL PARAMETER SENSITIVITY

Numerical models are sensitive to the values given to input parameters. The 9 constitutive parameters of the HS model and geogrid model in case study 3 (Table 4) were independently analysed using the ‘One at a time’ methodology. As discussed in Section 3.2, the 9 parameters
analysed were: $\phi, \psi, c, E_{50}^{ref}, E_{oed}^{ref}, E_{ur}^{ref}, m, \nu, \gamma$ and $EA$. Each variable was altered independently and their effect on horizontal deformation evaluated.

Of these 9 parameters, the most sensitive was the value used to represent the internal friction angle, $\phi$ of the soil. When comparing values for $\phi$ between 30° and 45°, construction and post-construction deformation can be seen to decrease linearly related with increased frictional shear strength, varying over the range by as much as 50% and 30% respectively (Figure 10).

The consequence of including a small cohesive shear strength component ($c = 0.5 \text{kN/m}^2$), to prevent singularity errors within the FE program, is shown in Figure 11. Over a range of 0.1 to 0.5 kN/m$^2$, it has relatively little influence (-1.6% max. construction and +1.3% max. post-construction deformation respectively). Its influence on horizontal deformation disproportionately increases when $c$ is increased to 0.9 kN/m$^2$ (-29.5% and -11.4%).

The sensitivity analysis of tangent soil stiffness for primary oedometric loading, $E_{50}^{ref}$, considered values between 10 kN/m and 110 kN/m. It was found to influence construction deformation by -0.21 $E_{50}^{ref}$ and post-construction deformation by -0.73 $E_{50}^{ref}$. While the sensitivity of the remaining soil parameters ($\psi, E_{oed}^{ref}, E_{ur}^{ref}, m, \nu$) was not significant (i.e. greater than 0.05 $\frac{\partial \delta x}{\partial X}$).

In the selection of the geogrid model parameters, it was assumed there was a simple perfectly elastic plastic relationship. The geogrid stiffness for each model was obtained using wide-width tensile tests and did not include creep, for reasons stated in Section 3.2.3. An analysis of the influence of geogrid stiffness (Figure 12), shows no significant difference in effecting construction deformation between 800 and 2000 kN/m (+10% to -5%), whereas post-construction deformation increases for weaker geogrids by as much as 40%. This highlights the need for careful selection of geogrid stiffness.

Figure 10: Local sensitivity analysis - angle of frictional shear strength. Reference for evaluation model: $\phi = 40^\circ$: $\delta x_c = 0.045 \text{ m}, \delta x_{pc} = 0.075 \text{ m}$. 

<table>
<thead>
<tr>
<th>Change in Horizontal Deformation (%)</th>
<th>Angle of Friction, $\phi$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>28</td>
</tr>
<tr>
<td></td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>32</td>
</tr>
<tr>
<td></td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>36</td>
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<tr>
<td></td>
<td>38</td>
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<tr>
<td></td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>42</td>
</tr>
<tr>
<td></td>
<td>44</td>
</tr>
<tr>
<td></td>
<td>46</td>
</tr>
</tbody>
</table>

Figure 11: Influence of cohesive shear strength component on horizontal deformation.
Figure 11: Local Sensitivity Analysis – Cohesive Shear Strength. Reference for evaluation model: $c = 0.5 \text{kN/m}^2$: $\delta x_c = 0.045 \text{m, } \delta x_{pc} = 0.075 \text{m}$.

Figure 12: Local Sensitivity Analysis – Geogrid Stiffness. Reference for evaluation model: $EA = 1600 \text{kN/m}$: $\delta x_c = 0.045 \text{m, } \delta x_{pc} = 0.075 \text{m}$.

4. CONSTRUCTION METHOD ANALYSIS

4.1. OVERVIEW

In this section, the numerical modelling method was expanded to evaluate the effect of both construction methods on construction and loading deformation. Case study 3, as the most typical GRS of the three case studies examined, was adapted for use as an evaluation model. The reference properties of the evaluation model remained the same (Table 4), while geometry was modified for simplicity. The model’s height, H, was increased to 5.0 m, and reinforcement length, L, restricted to 70% of the total height, forming a GRS consisting of ten 0.5 m layers. Figure 13 displays the profiles of both numerically modelled construction methods: full height and layer by layer.

Comparing both methods at the EOC, the GRS constructed using full height formwork, produces a less deformed structure with maximum deformation occurring in the highest layers (0.044 m). In contrast, the model using a layer by layer construction method, features an approximately linear decrease in deformation towards the top of the structure, with a maximum deformation occurring in the lowest layer (0.062 m).

Both numerical models, were subjected to a surcharge of 100 kN/m$^2$, acting over the crest of the structure. Figure 13 shows over 50% less additional deformation occurred when using the layer by layer method (0.018 m), than using the full height model (0.024 m). This provides
Analysis of Horizontal Deformations to allow the Optimisation of GRS

evidence that the construction method plays an important role in determining face deformation. The layers that underwent greater deformation during construction, also underwent the lowest deformation during loading. Total cumulative (construction and loading) deformation was higher in the layer by layer model (0.074 m) than for the full height structure (0.053 m).

![Figure 13: Construction Method Comparison in the evaluation Numerical Model ($H = 5.0 \text{ m}, L=3.5 \text{ m}$)](image)

4.2. HEIGHT AND SURCHARGE WITH CONSTRUCTION METHOD

The numerical modelling was extended to consider structure heights: between 2.5 m and 10 m (Figure 14). For each height, the model featured equally spaced reinforcement lengths, $L$, remaining equal to 70% of the height, $H$. Maximum construction deformation, for both construction methods, displayed a strongly positive linear relationship with height (0.004H). The maximum deformation occurred in the lowest layers for all of the models. This data suggests that the layer by layer method, would cause 0.030 m more construction deformation in case analysed than the full height method.
Numerical modelling was also extended to include post-construction surcharges: 0 kN/m² to 200 kN/m². Both construction methods showed approximately linear relationships between maximum deformation and applied surcharge loading (Figure 15). In all cases, the maximum deformation occurred in the highest layers of each model. This data suggests that GRS built using full height formwork, may deform 50% more, under a post-construction surcharge, than using the layer by layer method.

4.3. CONSTRUCTION METHOD EFFECT GUIDANCE

Based on the extended analysis of structure height and surcharge (Figure 14 and 15), for wrapped GRS structures, constructed with full height temporary face support:

Max. Construction Deformation (m): \( \delta x_c = \frac{H}{250} + 0.030 \text{ m} \) (Eq. 1)
Max. Post-Construction Deformation (m): \( \delta x_{pc} = 0.00018 \times \Delta \sigma_v \) (Eq. 2)

Alternatively for GRS constructed with a layer by layer method:
Max. Construction Deformation (m): \( \delta x_c = \frac{H}{250} + 0.060 \) m (Eq. 3)
Max. Post-Construction Deformation (m): \( \delta x_{pc} = 0.00012 \times \Delta \sigma_v \) (Eq. 4)

These values are provided for initial guidance only and, where possible, should be validated by further observation and numerical modelling to determine the reliability of these simple expressions.

4.4. VALIDATION OF THE DEFORMATION GUIDANCE

4.4.1. CASE STUDIES

The proposed guidance (Section 4.3) has been compared in (Table 5), to the measured data from the three case studies described earlier in Section 3.3. The maximum measured deformation for case studies 1, 2 and 3, was 0.0462 m, 0.129 m and 0.041 m respectively. Using the new guidance this was estimated as 0.062 m, 0.074 m and 0.084 m respectively. All the case studies used to validate this model are based on GRS with high quality granular soils (\( \varphi > 30^\circ \)). As alluded to in section 3.4., different results will be observed for variations in soil strength, geogrid stiffness and compaction, amongst other properties.

<table>
<thead>
<tr>
<th>Case Study</th>
<th>Height, ( H )</th>
<th>Surcharge, ( \Delta \sigma )</th>
<th>Measured, ( \delta x )</th>
<th>Predicted, ( \delta x )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.8 m</td>
<td>-</td>
<td>0.0462 m</td>
<td>0.062 m</td>
</tr>
<tr>
<td>2</td>
<td>3.6 m</td>
<td>-</td>
<td>0.129 m</td>
<td>0.074 m</td>
</tr>
<tr>
<td>3</td>
<td>4.5 m</td>
<td>300 kN/m²</td>
<td>0.041 m</td>
<td>0.084 m</td>
</tr>
</tbody>
</table>

This guidance has been developed for vertical GRS, the most sensitive condition, and does not account for facing batter in sloping structures. Work by Bathurst et al. (2008), suggest a nonlinear reduction factor between facing batter and stress in the structure, but it is unclear how this translates to construction method deformation, without further detailed analysis.

4.4.2. COMPARISON WITH OTHER DEFORMATION RELATIONSHIPS

The deformation guidance for construction deformation (equations 1 and 3) is in line with similar empirically based guidance (Christopher 1993; Lee 2000 and Bathurst et al. 2002). Figure 16 compares these guides predicted construction deformation for varying heights and the parameters of case study 3 (Table 4). As described in Section 2.3, each deformation guide has been developed by considering differing variables and case studies (Table 1), and as a result the derived deformations do not agree perfectly. The models all show a positive linear relationship, between \( H \) and deformation.
Bathurst et al. (2002) provide a height-normalised case study based reference. Face deformation can be estimated based on comparison with a well instrumented wrapped faced case study (GW16), which is one of 3 case studies. Based primarily on GRS with wrapped facing and employing a range of variables, Lee (2000), predicts the highest deformation, for all but the smallest structures (\(H < 5.0 \text{ m}\)). Closely followed by Christopher (1993), based primarily on the relationship between height \(H\) and reinforcement length, \(L\).

Both models proposed in this paper (equations 1 and 3), predict deformations between the three existing models. The deformations predicted by equation 1 for full height construction closely follows the Bathurst et al. (2002) model. Whereas equation 3, for layer by layer construction, projects the largest deformation than any other model for smaller structures (<5.0 m), but drops below the Christopher (1993) and Lee (2000) models for high structures (>6.5 m). Of particular note is that all existing guidance underestimates the construction deformations for walls lower than 5 metres in height when constructed using the layer by layer method.

5. DISCUSSION

5.1. NUMERICAL MODELLING DISCREPANCIES

5.1.1. GEOMETRICAL SIMPLIFICATIONS

A proportion of the underestimation of deformation for case study 2 (Table 5), can be explained by the simplified initial position of the GRS under construction. The geometry of case study 2 (Figure 5), was not as uniform compared to case studies 1 and 3, and varied along the structure’s running length. For the purpose of this modelling a consistent angle of 65 degrees, was assumed based on the design pre-construction design. This simplification excluded the geometrical imperfections in this structure, and that are common in all GRS, such as variations in inclination and small step backs between layers (Figure 5).
5.1.2. SOIL AND GEOGRID MODELLING ERRORS

Errors between the numerical model and reality can be caused by inaccurate input data for the soil and geogrid models. The sensitivity of the parameters involved has already been assessed in Section 3.4, and showed the most sensitive parameters to be $\phi, c, E_{50}^{ref}$ and $EA$. Other soil models were considered such as a linear-elastic perfectly plastic model or a soil model with different small strain characteristics, but the HS model was considered to replicate the important characteristics of granular soil behaviour that control GRS behaviour, yet requiring parameters that can be reasonably obtained.

The calculation of deformation in FE programs also includes inaccuracies. The staged construction of numerical models in Plaxis can lead to large settlements, due to pre-displacements at the start of each stage. However the program includes a post-calculation option, ‘sum-phase displacement’, to ignore this and present a more accurate model. This feature was considered but it was found to have no effect on horizontal displacements and therefore it was not used in the construction method evaluation. In the calculations, the tolerated error in the partial difference equations in the numerical model are undertaken, to an accuracy of 1%. The mesh size of the model was also considered, but showed no significant variation (<5%) was detected when mesh density was above 200 elements $/m^2$. In FE analysis, the soils model is simulated as a homogenised continuum, which cannot consider the relative movement of individual particles. This may lead to an underestimation of movement internally, with soil particles passing through geogrid apertures. Numerical methods like Discrete Element (DE) modelling, where individual particles are modelled as finite elements, are only beginning to be used to assess this (Wang et al. 2014).

The modelling of simple geogrid elements in FE programs, can result in residual forces in the final nodes as the geogrid is connected to the soil mesh. There are numerical programming tools that can be used to transfer stresses from these final nodes through the geogrid (Teixeira et al. 2007). However for the conditions considered in this particular analysis, such a tool was not considered. In addition, more advanced hyperbolic geogrid stiffness models, could have been considered to more-accurately represent creep and stiffness softening (Kaliakin et al. 2000), although the case studies and analysis considered in this study were short-term.

5.1.3. VALIDITY OF NUMERICAL MODELLING APPROACH

The numerical modelling approach was based on the reinforced soil zone, considering face and internal GRS deformation in the horizontal direction only (section 2.1). As a result it is limited to case studies where there is no movement outside the reinforced soil zone. The numerical model was used to model 3 case studies, where maximum and average wall face deformations were modelled to an accuracy of -30% to 92% and -32% to 140% of measured values respectively. These ranges are a function of the material models selected, material parameter uncertainty, simplifications in the model geometries compared to the case study as constructed geometries and the approaches used to replicate the layer by layer and full face construction processes. However, the magnitudes of deformation, the close agreement between the shapes of surface deformation profiles and the consistency of trends between the modelled and measured behaviours all provide justification for using the presented modelling approach to
investigate the effects of construction method and trends in lateral deformations related to GRS geometry.

5.2. CASE STUDY DATA

The deformation measured in each case study, contain varying degrees of uncertainty, as they used different monitoring devices. The accuracy of LVDTs (±0.1 mm), from case study 3, is typically higher than photogrammetry (±5 mm), used in case study 1 or laser scanning (±5 mm), used in case study 2 (Scotland et al. 2014). However, photogrammetry and laser scanning have very high measured spatial densities, allowing large numbers of cross-sections to be analysed, that do not need to be predetermined.

5.3. DEFORMATION GUIDANCE VALIDITY

The deformation guidance (equations 1 – 4) outlined in Section 4.4, has been developed based on a specific range of high quality (\( \varphi > 35^\circ \)) granular filled case studies and should not be taken out of context. There are many other factors contributing to deformational performance in GRS, such as compactive effort (Bathurst et al. 2009; Ehrlich et al. 2012), global and relative reinforcement stiffness (Christopher 1993; Ehrlich and Mitchell 1994) amongst others. Further work is necessary to adapt the model for these variables and application with granular fills with low shear strength and cohesive fills.

6. CONCLUSIONS

The development of design methods and guidance for geogrid reinforced structures (GRS) has been historically focused on their ultimate limit states, such as pull-out, rupture and global stability failure. However, as these have become more refined, serviceability limits have become more important. While there are many deformation databases and design charts available providing guidance on sensitivity of deformations to a range of variables, methods of construction are not currently included.

This paper presents a simple numerical model methodology (Section 3.2) for modelling two construction methods: GRS with full height temporary formwork or GRS with layer by layer temporary formwork. This modelling approach was validated using 3 granular filled GRS case studies. Following a parametric analysis of deformation and height, simple deformation guidance was outlined (Section 4.3), showing a 0.030 m increase in construction deformation when a layer by layer construction method is used. In contrast, the analysis suggests 50% less deformation under load is predicted after construction using the layer by layer approach, compared to the full height construction technique. Importantly, for low wall height structures constructed using the layer by layer method, <5 metres, this study indicates that horizontal face deformations are underestimated by current guidance.

The use of the results from this study to provide guidance on GRS deformations following construction and under load is limited to the range of cases using high quality reinforced fill. However, the outlined numerical method, coupled with further measured data could be used to extend the guidance to include GRS constructed using marginal soils, where SLS can control design.
ACKNOWLEDGEMENTS
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REFERENCES
Modelling Deformation during the Construction of Wrapped Geogrid Reinforced Structures
(Paper 3)


Analysis of Horizontal Deformations to allow the Optimisation of GRS


