

Determining the Soil-Geosynthetic Composite Stiffness in an Aggregate-Clay System

by

Oliver Dave L. Mag-uyon

A Thesis submitted to the Faculty of Graduate Studies of

The University of Manitoba

In partial fulfillment of the requirements of the degree of

Master of Science

Department of Civil Engineering

University of Manitoba

Winnipeg, Manitoba

Copyright © 2025 by Oliver Dave L. Mag-uyon

ABSTRACT

The stiffness of soil–geosynthetic composites, denoted as K_{SGC} , represents the initial stiffness of the interface between soil and geosynthetics under small displacements. It serves as a performance parameter for evaluating the reinforcing effect of geosynthetics in pavement systems and other geotechnical applications. This study investigates the factors influencing K_{SGC} using large-scale pullout box tests, focusing on the effects of backfill material type, confining pressure, and geosynthetic product selection.

Confining pressure was found to have a notable effect on K_{SGC} . Higher normal stresses generally resulted in increased stiffness due to enhanced confinement and mobilization of interfacial resistance. However, this increase showed a diminishing rate beyond certain pressure thresholds, likely due to stress redistribution and material yielding. Aperture size, rib thickness, and surface roughness were identified as key design parameters influencing the interaction behavior. The analysis also reveals that backfill type and gradation significantly affect soil–geosynthetic interaction. For geogrids, K_{SGC} values are highly dependent on the relationship between soil particle size, aperture dimensions, and rib geometry, as mechanical interlock governs the interface response. In contrast, geotextiles, particularly nonwoven types, demonstrate less sensitivity to particle size due to their extensibility and reliance on friction. The combination of geogrid and geotextile layers yielded higher K_{SGC} values, suggesting a synergistic interaction between frictional and passive resistance mechanisms.

The study underscores the importance of selecting appropriate geosynthetic materials based on specific site conditions and expected stress environments. Consideration of backfill properties and confining stress is critical in maximizing the reinforcing effect and ensuring long-term performance in geosynthetic-stabilized pavement systems.

ACKNOWLEDGEMENT

First and foremost, I would like to express my deepest appreciation to my academic advisors, Dr. James Bartz and Dr. Marolo Alfaro. Your mentorship, encouragement, and belief in my potential have been the driving force behind this research. Thank you for your invaluable guidance, patience, and commitment throughout my master's journey.

I would also like to thank Mitacs for providing the financial support that enabled this research. The opportunity to pursue my graduate studies with the necessary resources is something I will always be thankful for.

To my colleagues and fellow graduate students, thank you for your friendship, support, and for making this journey enjoyable. To our lab technician, Kerry Lynch, thank you for your dependable assistance, technical expertise, and steady support throughout the laboratory phase of this study.

To my parents, thank you for your love and encouragement every step of the way. Special thanks to my sister Olive and brother-in-law Julius for opening their home to me and supporting me throughout my master's program.

To my wife, Kathlyn, your strength and support—despite the distance—meant everything. And to my newborn daughter, Kaitlyn, your arrival gave me new purpose and inspiration.

This thesis is not just the result of academic effort, but a reflection of the love, support, and kindness I have received from so many people. Thank you from the bottom of my heart.

Table of Contents

Abstract	i
Acknowledgement	ii
Table of Contents	iii
List of Figures	vii
List of Tables	xi
1. Introduction	1
1.1 Motivation of the Study.....	1
1.2 Research Objective and Scope	2
1.3 Overview of the Research Paper	3
2. Literature Review	4
2.1 Geosynthetics	4
2.1.1 Functions of Geosynthetics	5
2.2 Applications of Geosynthetic in Roadways	7
2.3 Soil-Geosynthetic Interaction System.....	11
2.3.1 Pullout Test.....	15
2.4 Soil-Geosynthetic Composite Model	16
2.4.1 Background on Analytical Models	16
2.4.2 Development of the SGC Model	18
2.4.3 Formulation	20

2.4.4. Interpreting K_{SGC} value.....	23
2.4.5 K_{SGC} Model Validation	25
2.4.6 Effect of Backfill Materials	26
2.4.7 Effect of Normal Pressure	26
2.4.8 Effect of Test Direction	27
2.4.9 Threshold Value for K_{SGC}	27
3. Methodology.....	29
3.1 Overview	29
3.2 Soil Properties	31
3.3 Geosynthetic Properties.....	34
3.3.1 GT-1.....	35
3.3.2 GT-2.....	36
3.3.3 GT-3.....	37
3.3.4 GG-1	38
3.3.5 GG-2	39
3.3.6 GGC-1	40
3.4 Testing Equipment.....	42
3.4.1 Pullout Box Set-up and Testing	42
3.4.2 Resistance of Embedded Pullout Clamp.....	49
3.4.3 Interpretation of Test Results.....	49

4. Geocomposite Results	51
4.1 Pullout Test Results.....	51
4.1.1 Varying Normal Pressures.....	51
4.1.2 Different Soil Interface	54
4.2 K_{SGC} evaluation using Direct Estimation from Unit Tension Squared versus Telltale Displacement (<i>Method 1</i>)	57
4.2.1 Varying Confining Pressure	57
4.2.2 Varying Backfill Materials	62
4.3 K_{SGC} evaluation using Estimation from Parabolic Regression of Telltale Displacement Data (<i>Method 2</i>)	67
4.3.1 Varying Confining Pressure	67
4.3.2 Varying Backfill Material.....	71
4.4 Comparison of K_{SGC} for GGC-1 for <i>Method 1</i> and <i>Method 2</i>	75
5. Geotextile and Geogrid Results.....	79
5.1 Pullout Test Results.....	79
5.2 K_{SGC} Interpretation and Results.....	84
5.2.1 <i>Method 1</i> : Direct Estimation from Unit Tension Squared versus Telltale Displacement	84
5.2.2 <i>Method 2</i> : Estimation from Parabolic Regression of Telltale Displacement Data	89

5.2.3 Comparison of K_{SGC} Value for Geotextiles and Geogrids for <i>Method 1</i> and <i>Method 2</i>	94
6. Conclusion	97
7. Recommendations	100
8. References	102
Appendix A. Results of Pullout Tests for Titan Environmental Product	108
Appendix B. Results of Pullout Tests for City of Winnipeg Product	113

List of Figures

Figure 2.1 Stress distribution on subgrade layer: (a) unreinforced pavement; (b) geosynthetic-reinforced pavement. (modified from Zornberg & Gupta, 2010).....	9
Figure 2.2 Mechanisms in geosynthetic-reinforced roadways (modified from Holtz et al., 1998): (a) lateral restraint of base; (b) increased bearing capacity; (c) load transfer.....	10
Figure 2.3 Interaction mechanisms in a geosynthetic reinforced soil wall. (modified from Palmeria & Milligan, 1989).	12
Figure 2.4 Soil-geosynthetic interaction mechanism during pullout. (a) plan view of geotextile; (b) cross section of forces on geotextile; (c) plan view of geogrid; (d) cross section of forces on geogrid. (adapted from Elias et al., 2001).....	14
Figure 2.5 Schematic diagram of pullout test. (Alfaro et al., 1995a)	15
Figure 2.6 Unit Tension – Strain Relationship for Confined Geosynthetics. (Zornberg et al., 2017)	19
Figure 2.7 Interface Shear of Soil-Geosynthetic Model. (Zornberg et al., 2017).....	20
Figure 2.8 Boundary Conditions used in the Model. (modified from Zornberg et al., 2017)	21
Figure 2.9 Schematic diagram of steps in estimating the K_{SGC} : (a) frontal unit tension vs telltale displacement; (b) unit tension at point i vs telltale i displacement; (c) unit tension squared at point i vs telltale i displacement. (Zornberg et al., 2017)	24
Figure 3.1 Atterberg Limits (top figure), Hydrometer Analysis (bottom left), Modified Proctor Test (bottom right).	32
Figure 3.2 Compaction curve of clay soil.	33
Figure 3.3 Woven Geogrid from Polypropylene Yarns: GT-1	35
Figure 3.4 Woven Geotextile from Polypropylene Monofilaments: GT-2.....	36

Figure 3.5 Nonwoven Needle Punched Geotextile from Polypropylene Fibers: GT-3	37
Figure 3.6 Biaxial Punched Geogrid from Polypropylene: GG-1	38
Figure 3.7 Biaxial Punched Geogrid from Polypropylene: GG-2	39
Figure 3.8 Biaxial Geogrid Heat-bonded to a Wicking Nonwoven Geotextile: GGC-1	40
Figure 3.9 Clamp setup for: (a) geotextile, (b) geogrid, (c) geocomposite.	43
Figure 3.10 Telltale dimensions (in mm).....	44
Figure 3.11 Setup at the back of the pullout box.	45
Figure 3.12 Pullout box setup (a) compaction of base material, (b) compaction of top material, (c) placement of rubber gasket and rubber membrane, (d) connection of clamp to load cell and servomotor.	47
Figure 3.13 Core sample for measuring unit weight of clay upon compaction.	48
Figure 4.1 Pullout test results of GGC-1 with varying confining pressure.	52
Figure 4.2 Failure of GGC-1.....	53
Figure 4.3 Pullout test results of GGC-1 with varying backfill material.....	56
Figure 4.4 Plot of Unit Tension Squared vs Telltale Displacement for GGC-1 at (a) 7 kPa, (b) 21 kPa, (c) 35 kPa.	60
Figure 4.5 Evaluating K_{SGC} values of GGC-1 from varying confining pressures.	61
Figure 4.6 Plot of Unit Tension Squared vs Telltale Displacement for GGC-1 at 21 kPa with backfill setup of (a) aggregate-clay, (b) pure clay, (c) pure aggregate.	65
Figure 4.7 Evaluating K_{SGC} values of GGC-1 from varying backfill materials.....	66
Figure 4.8 Parabolic Regression of Telltale Displacement Data for GGC-1 at (a) 7 kPa, (b) 21 kPa, (c) 35 kPa.	69
Figure 4.9 K_{SGC} values obtained for GGC-1 from varying confining pressures using <i>Method 2</i>	70

Figure 4.10 Parabolic Regression of Telltale Displacement Data for GGC-1 at 21 kPa with backfill setup of (a) aggregate-clay, (b) pure clay, (c) pure aggregate.	74
Figure 4.11 Summarized K_{SGC} values obtained for GGC-1 from varying backfill materials using <i>Method 2</i>	75
Figure 4.12 Summarized K_{SGC} values obtained for GGC-1 from varying confining pressures using both methods.	77
Figure 4.13 Summarized K_{SGC} values obtained for GGC-1 from varying backfill materials using both methods.	78
Figure 5.1 Net Pullout Load for Geotextiles.	80
Figure 5.2 Net Pullout for Geogrids.	81
Figure 5.3 Net Pullout Load for Geogrids with Nonwoven Geotextile.	82
Figure 5.4 Summary of Net Pullout Load of All Geosynthetics Used.	83
Figure 5.5 Plot of Unit Tension Squared vs Telltale Displacement for geotextiles: (a) GT-1, (b) GT-2.	85
Figure 5.6 Plot of Unit Tension Squared vs Telltale Displacement for the geogrids: (a) GG-1, (b) GG-2.	86
Figure 5.7 Plot of Unit Tension Squared vs Telltale Displacement for geogrids with nonwoven geotextiles: (a) GG-1 with GT-3, (b) GG-2 with GT-3.	87
Figure 5.8 K_{SGC} values obtained for all geosynthetics using <i>Method 1</i>	88
Figure 5.9 Parabolic Regression of Telltale Displacement for geotextiles: (a) GT-1, (b) GT-2. .	91
Figure 5.10 Parabolic Regression of Telltale Displacement for geogrids: (a) GG-1, (b) GG-2...	92
Figure 5.11 Parabolic Regression of Telltale Displacement for geocomposites: (a) GG-1 with GT-3, (b) GG-2 with GT-3.	93

Figure 5.12 K_{SGC} values obtained for all geosynthetics using <i>Method 2</i>	94
Figure 5.13 Summarized K_{SGC} values obtained for all geosynthetics using both methods.	96
Figure A.1 Pullout Test results of GGC-1 at 7 kPa.	108
Figure A.2 Pullout Test results of GGC-1 at 21 kPa.	109
Figure A.3 Pullout Test results of GGC-1 at 35 kPa.	110
Figure A.4 Pullout Test results of GGC-1 at 21 kPa with pure clay interface.	111
Figure A.5 Pullout Test results of GGC-1 at 21 kPa with pure aggregates interface.	112
Figure B.1 Pullout Tests results of GT-1 at 21 kPa.	113
Figure B.2 Pullout Test results of GT-2 at 21 kPa.	114
Figure B.3 Pullout Test results of GG-1 at 21 kPa.	115
Figure B.4 Pullout Test results of GG-2 at 21 kPa.	116
Figure B.5 Pullout Test results of GG-1 with GT-3 at 21 kPa.	117
Figure B.6 Pullout Test results of GG-2 with GT-3 at 21 kPa.	118

List of Tables

Table 2.1 Functions of Geosynthetics. (Jie & Yan, 2013).....	7
Table 3.1 Summary of pullout test materials and setup conditions.	30
Table 3.2 Properties of Coarse Material Adopted (Eissa, 2023).	33
Table 3.3 Index Properties of GT-1.	35
Table 3.4 Index Properties of GT-2.	36
Table 3.5 Index Properties of GT-3.	37
Table 3.6 Index Properties of GG-1.....	38
Table 3.7 Index Properties of GG-2.....	39
Table 3.8 Index Properties of GGC-1.	41
Table 4.1 Summary of constitutive parameters for varying confining pressures.	67
Table 4.2 Summary of constitutive parameters for varying backfill materials.....	72
Table 4.3 Summarized values of K_{SGC} for both conditions and methods for GGC-1.....	76
Table 5.1 Summary of Constitutive Parameters for Different Materials.....	90
Table 5.2 Summary of K_{SGC} for both methods.	95

CHAPTER 1

INTRODUCTION

1.1 Motivation of the Study

Geosynthetics are engineered polymeric materials specifically designed to interact with soil and other construction materials. Their inclusion in pavement structures is a significant advancement in civil engineering due to their ability to enhance performance and longevity, especially in transportation and geotechnical engineering. In geotechnical applications, geosynthetics are typically employed in two main categories: (1) structures designed to meet ultimate limit state conditions, such as reinforced soil slopes and geosynthetic-reinforced retaining walls, and (2) structures designed based on deformation or serviceability limits, such as geosynthetic-stabilized roadways.

Geosynthetics have been widely applied in roadway construction, typically installed either within the unbound base layer or at the subbase-subgrade interface to enhance roadway performance. In particular, their use in base layers help limit the lateral movement of granular materials – thereby preventing loss of base stiffness – through mechanisms of lateral restraint (Holtz et al., 1998). Most existing soil-geosynthetic interaction models have focused on characterizing behavior under limit state conditions. As a result, the geosynthetic properties required for designing and analyzing structures that rely on reinforcement are well established. However, for applications where geosynthetics provide stiffness rather than reinforcement, such as geosynthetic-stabilized pavements, the relevant properties for assessing deformability and performance under small displacements remain insufficiently defined. A parameter known as K_{SGC} , representing the

stiffness of the soil-geosynthetic composite under small displacement conditions, was introduced in a study by Zornberg et al. (2017). Limited research has been conducted to assess the behavior of the soil-geosynthetic composite K_{SGC} across a broad range of confining pressures, geosynthetic types, and especially different backfill materials. This thesis addresses this gap in knowledge by conducting an experimental program to examine how two types of backfill affect the performance of soil-geosynthetic composites under varying loads and with different types of geosynthetics.

1.2 Research Objective and Scope

The main purpose of this study is to evaluate the impact of the backfill material on the K_{SGC} value with varying confining pressures on different geosynthetic materials. A total of eleven soil-geosynthetic interaction tests were conducted as a part of this study. Three types of geosynthetic materials were used in the testing program, three geotextiles, two geogrids and a geocomposite comprised of geogrid-geotextile composite. A hybrid set-up wherein a poorly graded gravel and clay was used to conduct the soil-geosynthetic interaction tests wherein the geosynthetic is placed in between. The geosynthetic materials, placement, and soils are intended to be representative of actual road conditions compared to testing performed in development of the K_{SGC} model. The objectives of the research are:

- (1) Determine and compare the K_{SGC} value of the geocomposite with various confining pressures and backfill materials.
- (2) Determine and compare the K_{SGC} values of the various products and combinations.
- (3) Compare the change in K_{SGC} in geogrids with and without the nonwoven geotextile.

1.3 Overview of the Research Paper

This thesis is structured into six chapters. Chapter 1 serves as the introduction, outlining the study's objective and scope. Chapter 2 presents background information on the application of geosynthetics in geotechnical infrastructure, specifically transportation, including the mechanisms influencing their behavior under traffic loads and the soil-geosynthetic interaction test used. It also provides an overview characterizing the soil-geosynthetic composite parameter. Chapter 3 details the materials utilized and describes the components involved in the soil-geosynthetic interaction testing. Chapter 4 covers the collaborative project with Titan Environmental Containment Ltd. (Titan Environmental) and reports the findings related to the geocomposite material. Chapter 5 highlights the partnered project with the City of Winnipeg and presents the results for the other geosynthetic products in this study. Finally, Chapter 6 concludes with a summary of the key findings of the research.

CHAPTER 2

LITERATURE REVIEW

2.1 Geosynthetics

Geosynthetics has become integral to a wide range of civil engineering applications, particularly in geotechnical engineering. The earliest use of synthetic fibers, initially produced from polyvinyl chloride (PVC) in the early 1900s, led to the development of non-woven geotextiles by the mid-1960s. The first documented application of geosynthetics was in 1958, when woven geotextiles were used to mitigate coastal erosion in Florida, USA—an application that remains in use today (Wilmers, 2002). Following this, the use of geosynthetics expanded rapidly across civil engineering disciplines, fueled by advancements in manufacturing that introduced a wider variety of products such as geomembranes, geogrids, and geocomposites (Van Santvoort, 1994).

As defined by ASTM, geosynthetics are polymeric, planar materials used in combination with soil, rock, or other geotechnical materials within engineering structures or systems. Although their initial use was for soil improvement in geotechnical applications, geosynthetics have since become widespread due to the diversity of available product types and installation methods (Bloise & Ucciardo, 2000). These materials now include geotextiles, geogrids, geomembranes, geonets, geocomposites, and geosynthetic clay liners, among others, and are commonly used alongside conventional construction materials in various infrastructure projects (Han & Thakur, 2015).

2.1.1 Functions of Geosynthetics

Specific types of geosynthetics are engineered to perform distinct functions, each tailored to meet particular engineering demands. The primary functions commonly attributed to geosynthetics, as documented in the technical literature (Koerner et al., 2007; Palmeira et al., 2008; PN-EN ISO 10318-1:2015), include the following:

- **Separation:** involves maintaining the distinctiveness between subsoil and structural layers or between different aggregates with dissimilar properties. By being placed between such materials, geosynthetics preserve their integrity and functional independence, while also potentially offering long-term stress relief. The survivability of the geosynthetic during installation is a key design parameter for this function.
- **Filtration:** water or other fluids to pass between soil layers without disturbing the soil structure. Geosynthetics used for filtration retain fine particles on their upstream side, and their effectiveness is determined by properties such as permittivity and pore-size distribution.
- **Drainage:** refers to the in-plane flow of fluids or gases through the geosynthetic's structure. The design performance is primarily governed by the geosynthetic's transmissivity, which is a function of its in-plane hydraulic conductivity and thickness.
- **Reinforcement:** improves the mechanical behavior of soil and structural components by providing additional tensile strength. This function applies both to the stabilization of soil masses and to roadway applications such as the reinforcement of bituminous layers or unpaved road surfaces. The primary design criterion is the tensile strength of the geosynthetic material. The terms reinforcement and stabilization are often used interchangeably; however, certification bodies such as the International Standards

Organization (ISO) provide distinct definitions. Reinforcement refers to the use of the geosynthetic's stress-strain properties to enhance the mechanical strength of the soil. In contrast, stabilization involves the incorporation of a geosynthetic material to improve the mechanical behavior of unbound materials, primarily to reduce deformation.

- **Surface Erosion Control:** it involves the use of geosynthetics to minimize the displacement of soil or other particles from exposed surfaces, such as slopes. This function helps preserve surface stability in vulnerable environments.
- **Barrier:** this function is aimed at preventing the cross-plane migration of liquids or gases. Geosynthetics designed for this purpose serve as containment layers, and their effectiveness depends on long-term material durability.

Collectively, these functions enhance the utility and performance of roadway systems. In particular, geosynthetics contribute to the stabilization of road bases and subgrades, provide material separation, and enable effective lateral drainage (Zornberg, 2017).

Geosynthetic materials serve multiple functions in civil engineering applications, including separation, drainage, reinforcement, and filtration. Table 2.1 shows different geosynthetics with its function. When considering the substitution of geosynthetics with alternative materials, several critical factors must be evaluated.

Table 2.1 Functions of Geosynthetics. (Jie & Yan, 2013)

Type	Separation	Filtration	Drainage	Reinforcement	Erosion Protection	Barrier
Geotextile	✓	✓	✓	✓	✓	✓
Geogrid				✓		
Geonet			✓			
Geomembrane	✓				✓	✓
Geosynthetic Clay Liner	✓				✓	✓
Geocell				✓	✓	
Geofiber				✓		
Geocomposite	✓	✓	✓	✓	✓	✓

2.2 Applications of Geosynthetic in Roadways

Geosynthetics have been extensively applied in roadway systems to enhance pavement performance under repeated traffic and environmental loading. Their placement typically occurs at key interfaces – such as between the base and subbase, between the subbase and subgrade, or within the unbound base layer – contributing to improved structural response (Al-Qadi et al., 2008; Perkins, 2002; Zornberg, 2011; Zornberg et al., 2012). These materials serve multiple roles in pavement application, including filtration, separation, lateral drainage, stabilization, and reinforcement (Zornberg et al., 2013; Zornberg, Ferreira, et al., 2012). Additional roles include serving as hydraulic/gas barriers and protection layers. Notably, stiffening is now recognized as distinct from reinforcement—focusing on deformation control rather than structural stability and requiring different mechanical properties for design.

As filtration layers, geosynthetics inhibit the migration of fine soil particles while allowing water to flow into coarser aggregates. When used for separation, they prevent the upward movement of subgrade soils into granular base or subbase layers, thereby preserving layer integrity. Their inplane permeability, quantified as transmissivity, facilitates lateral drainage.

One of the most critical functions of geosynthetics in roadway systems is subgrade and base stabilization. Their inclusion distributes vehicular loads over a broader subgrade area, resulting in reduced vertical stress concentration. This load-spreading effect is illustrated in Figure 2.1, which compares stress distributions in unreinforced and geosynthetic-reinforced pavements. Geosynthetics improve pavement performance through three primary mechanisms: (1) lateral restraint, (2) enhanced bearing capacity, and (3) the tensioned membrane effect (Holtz et al., 1998; Perkins & Ismeik, 1997), as schematically shown in Figure 2.2.

The dominant mechanism for base stabilization is lateral restraint (Figure 2.2a), where the geosynthetic layer resists lateral movement of base course particles through interlocking and frictional resistance (Bender & Barenberg, 1978). This confinement effect improves load distribution and reduces permanent deformation. The effectiveness of this interaction depends on geogrid aperture size and shape for geogrids, aggregate particle size, and the geosynthetic's stiffness.

The second mechanism—bearing capacity improvement—is particularly relevant over weak subgrades (Figure 2.2b). Here, the geosynthetic layer redirects potential failure surfaces, effectively increasing the system's overall bearing resistance. In extremely soft subgrades, this mechanism can shift failure modes from deep-seated punching to general bearing capacity failure.

The third mechanism, the tensioned membrane effect (Figure 2.2c), becomes significant under large deformations. As the geosynthetic deforms, tensile forces are mobilized. The vertical component of these forces acts to reduce stresses transmitted to the subgrade beneath loaded areas, further enhancing performance.

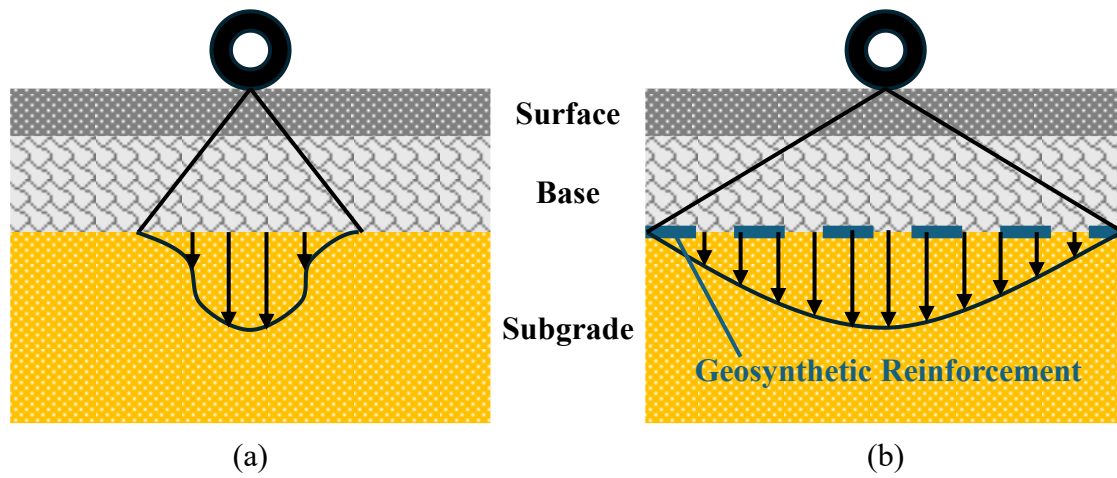


Figure 2.1 Stress distribution on subgrade layer: (a) unreinforced pavement; (b) geosynthetic-reinforced pavement. (modified from Zornberg & Gupta, 2010)

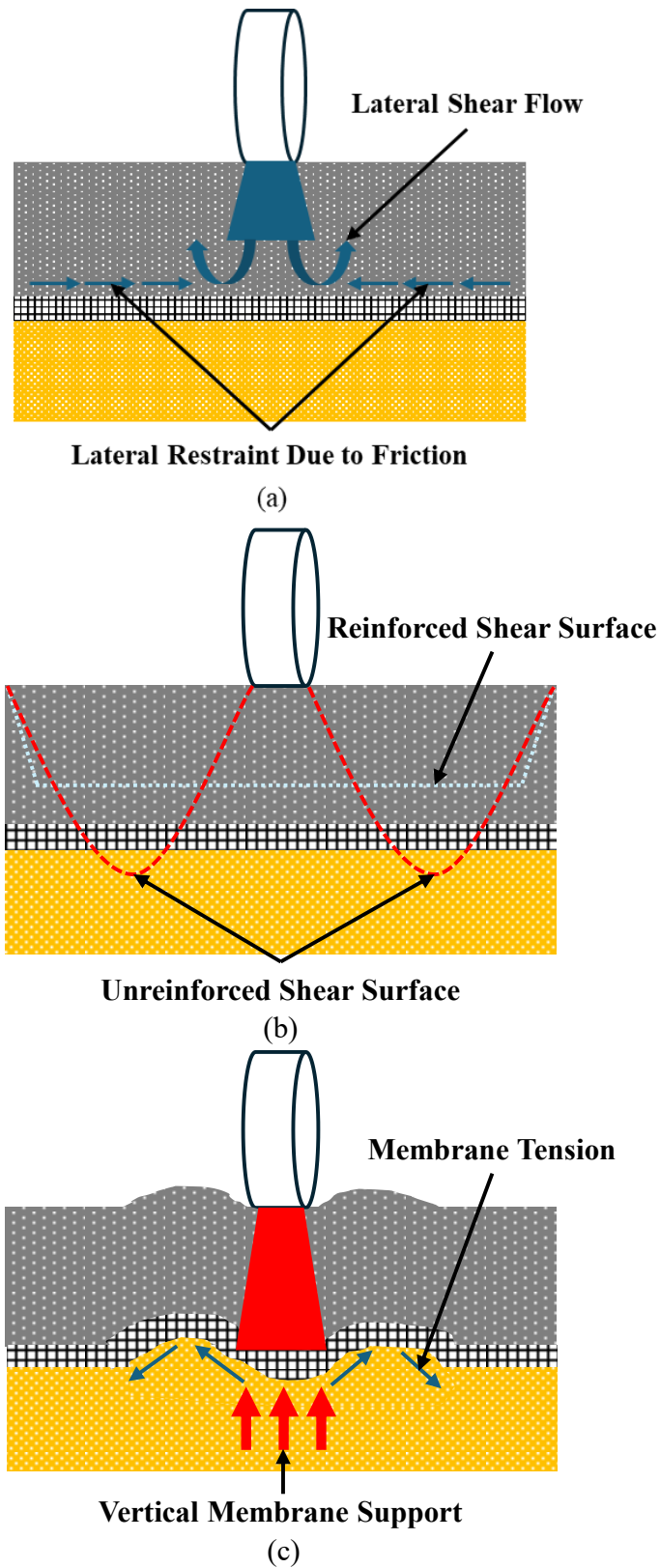


Figure 2.2 Mechanisms in geosynthetic-reinforced roadways (modified from Holtz et al., 1998):
 (a) lateral restraint of base; (b) increased bearing capacity; (c) load transfer.

Related studies are presented in this chapter which aid in the framework of this project. The work of Zornberg et al. (2017) presents the understanding of the concept of formulating the model of the soil-geosynthetic stiffness parameter. Studies conducted by Roodi & Zornberg (2017) and Morsy et al. (2019) provide further understanding of this model which would be used in determining the value of the stiffness coefficient.

2.3 Soil-Geosynthetic Interaction System

Over the past three decades, numerous experimental and analytical methods have been developed to advance understanding of soil–geosynthetic interaction. Purpose-built test devices have been utilized to assess the interface behavior between soil and geosynthetics, helping to explain the mechanisms governing this interaction (Mendes et al., 2007; Palmeria & Milligan, 1989).

According to Palmeria & Milligan (1989), the potential interaction mechanisms in soil reinforced walls vary by structural zone and loading conditions, as illustrated in Figure 2.3. In Zone A, sliding failure along the reinforcement surface is anticipated, and the direct shear test is appropriate to assess this interface. The same test can be applied to Zone C, where the failure surface intersects the reinforcement. In Zone B, where both the reinforcement and surrounding soil experience lateral deformation, the in-soil tensile test is more suitable. In Zone D, where the reinforcement is extracted from the resisting soil mass, the pullout test is the most relevant.

For geosynthetic-stabilized roadway systems, performance enhancement is governed by three primary mechanisms: lateral restraint, improved bearing capacity, and tensioned membrane effect, as depicted in Figure 2.2 (Holtz et al., 1998). Lateral restraint confines the base aggregate, reducing deformation. Enhanced bearing capacity is relevant over weak subgrades, where the geosynthetic

Pullout resistance, a key performance metric, results from two fundamental mechanisms: frictional resistance at the soil–geosynthetic interface, and passive bearing resistance mobilized against transverse reinforcement elements. For geotextiles, pullout resistance is predominantly frictional, as shown in Figures 2.4a and 2.4b. In contrast, geogrids benefit from both friction and passive resistance offered by transverse ribs, as illustrated in Figures 2.4c and 2.4d.

These distinctions highlight the importance of selecting appropriate geosynthetic materials and test methods based on expected field conditions and structural demands.

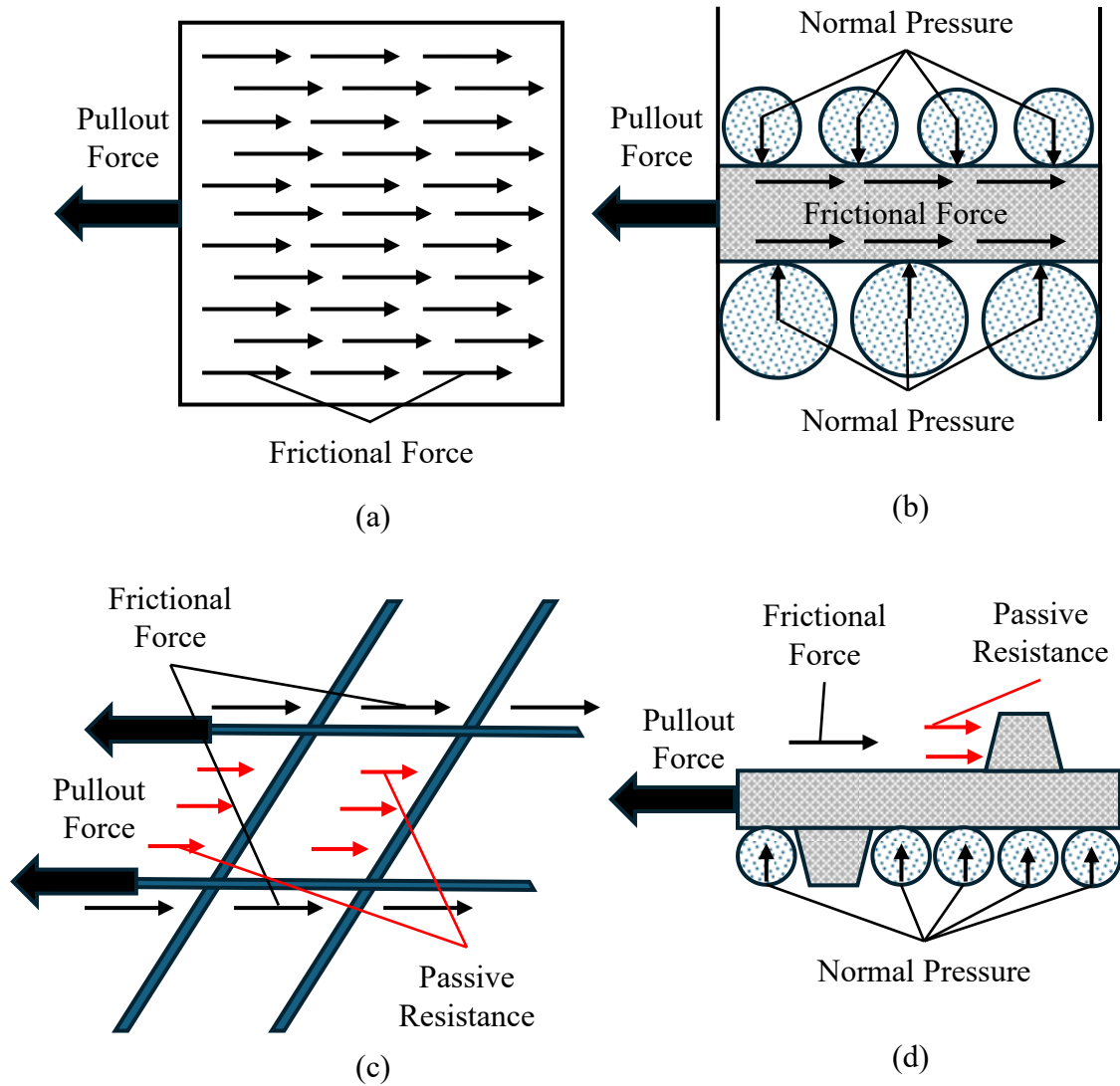


Figure 2.4 Soil-geosynthetic interaction mechanism during pullout. (a) plan view of geotextile; (b) cross section of forces on geotextile; (c) plan view of geogrid; (d) cross section of forces on geogrid. (adapted from Elias et al., 2001)

2.3.1 Pullout Test

Alfaro et al. (1995a) introduced a dual-function test apparatus, as shown in Figure 2.5, capable of performing both pullout and direct shear tests on geogrid reinforcements in dense granular soils. The system also allows measurement of soil dilatancy during pullout, offering insights into how dilative soil behavior contributes to shear resistance. The testing framework offers critical insights into selecting appropriate interaction parameters and models for the design and analysis of reinforced soil systems.

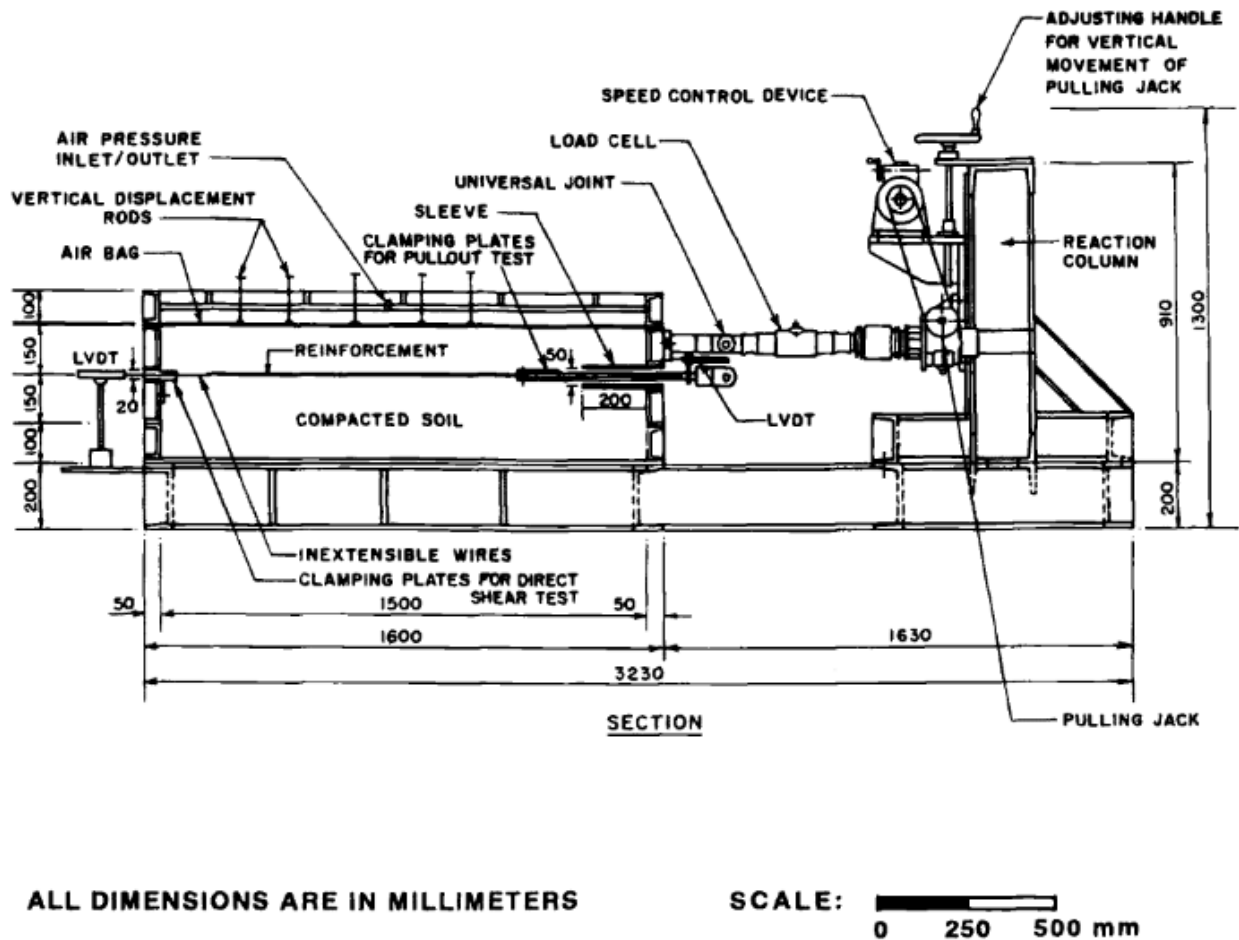


Figure 2.5 Schematic diagram of pullout test. (Alfaro et al., 1995a)

Pullout testing has become a fundamental method for assessing the shear stress transfer between soil and geosynthetic reinforcements. It not only helps characterize the interaction behavior between the reinforcement and surrounding soil but also provides critical data on the stiffness of the resulting soil–geosynthetic composite system. A substantial body of research has employed pullout tests to evaluate the interaction of geogrids with a variety of traditional soil types (Hayashi et al., 1985; Jewell, 1980; Koerner, 1986; Palmeria & Milligan, 1989) and alternative materials such as industrial waste byproducts (Hanumasagar et al., 2014; Kondo et al., 2016).

Several researchers have also focused on experimental conditions that may influence pullout test results. These factors include soil sample preparation techniques, procedural protocols, pullout box dimensions, and friction between the box walls and the soil (Jewell, 1980; Juran et al., 1988; Palmeria & Milligan, 1989). Farrag et al. (1993) conducted an extensive testing program to examine the influence of variables such as geosynthetic type, specimen width, sleeve length, soil thickness and density, displacement rate, and applied confining pressure. Additional studies have emphasized the effects of geogrid geometry, particle size distribution, and compaction on pullout resistance.

2.4 Soil-Geosynthetic Composite Model

2.4.1 Background on Analytical Models

Previous studies examining the behavior of geosynthetic-reinforced systems can be categorized into two primary approaches. The initial approach, developed in the 1970s, treated the reinforced soil mass as a homogenized material, attributing either an equivalent cohesion (Yang, 1972) or an anisotropic behavior based on the Mohr-Coloumb theory (Schlosser & Long, 1974). Though this

method led to simplified models on reinforced soil systems, it offered a limited depiction of the interaction between the soil and the geosynthetics. Notably, this approach could not differentiate between stresses and strains in the soil and those in the geosynthetic inclusions. In contrast, the second approach, adopted since the 1990s, considers soil and geosynthetics as distinct materials. This method necessitates detailed characterization of the soil, the geosynthetic material, and the interface between them, providing a more comprehensive understanding of their interaction.

Using a discrete model requires defining the appropriate relationships for three components: (1) the backfill soil, (2) the geosynthetic material, and (3) the soil-geosynthetic interface. Simple models are sufficient for structures with inextensible reinforcements, as the interaction can be approximated as rigid movement of the reinforcement without elongation. However, for extensible reinforcements, more advanced models are necessary to account for both soil-geosynthetic displacement and geosynthetic elongation. This interaction leads to a gradual buildup of tension in the extensible reinforcement and mobilization of interface shear along its length (Zornberg et al., 2018).

Modeling soil-geosynthetic interaction can incorporate both the local mechanism and the specific geometry of the geosynthetic. There are two main types of geosynthetic inclusions: (1) geogrids, which consist of a structured network of longitudinal and transverse member forming apertures that inlock with the surrounding soil, and (2) sheet reinforcements, which are flat, planar materials without any openings. The transverse member in geogrids is known to activate bearing resistance mechanisms, whereas sheet reinforcements primarily rely on shear resistance developed along their surface. As a result, modeling sheet reinforcements typically focus only on the frictional resistance at the interface (Zornberg et al., 2018).

2.4.2 Development of the SGC Model

The Soil-Geosynthetic Composite (SGC) model is an analytical framework designed to characterize the stiffness of soil–geosynthetic systems under small displacements. It introduces an index parameter known as the Stiffness of the Soil-Geosynthetic Composite (K_{SGC}), which reflects both the tensile behavior of the geosynthetic material and the shear resistance at the soil–geosynthetic interface. The model provides a closed-form solution for predicting geosynthetic displacement and tensile load response in pullout tests under confined conditions, based on two primary input parameters: the yield interface shear stress (τ_y) and the confined geosynthetic stiffness (J_c). This model was developed by Zornberg et al. (2017) to offer a unified approach for capturing the coupled interaction mechanisms of soil and geosynthetics through a single measurable parameter.

The constitutive law considered in the geosynthetic is the relationship between the unit tension (T) and its tensile strain (ϵ). In this model, a linear relationship is assumed to represent the correlation between the geosynthetic unit tension (T) and tensile strain (ϵ). The slope known is characterized as the confined geosynthetic stiffness (J_c) as shown in Figure 2.6 and described in Equation 2.1.

$$T = J_c \cdot \epsilon \quad \text{Equation 2.1}$$

The unit tension (T), defined as the applied force per unit width of the geosynthetic, is commonly utilized to describe the load carried by the geosynthetics. The confined stiffness (J_c) is expressed as $E \cdot t$, where E is the elastic modulus and t is the thickness of the geosynthetic.

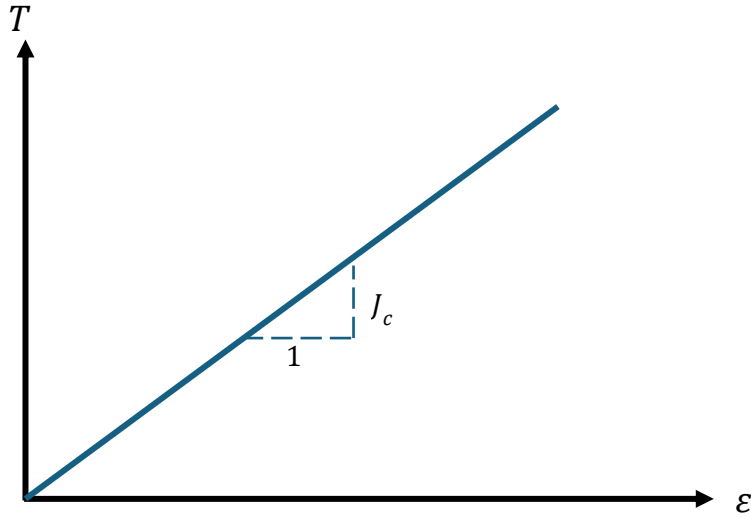


Figure 2.6 Unit Tension – Strain Relationship for Confined Geosynthetics. (Zornberg et al., 2017)

In modelling the interaction between soil and geosynthetics, a constitutive relationship was established to link the shear stress (τ) at the interface to its relative displacement (u) between two materials. The study adopted a rigid-perfectly plastic model for this relationship, meaning that once the shear stress reaches a certain yield point or yield shear stress (τ_y), it remains constant regardless of further displacement. This assumption simplifies the governing differential equations for soil-geosynthetic interaction, allowing for straight forward analytical solutions. In essence, the interface shear stress does not vary with displacement beyond the yield point, maintaining a constant value (τ_y) for any nonzero displacement as shown in the equations below and illustrated in Figure 2.7. Any relative displacement ($u > 0$) results in fully mobilizing the yield shear stress (τ_y). If there is no displacement ($u = 0$), then shear stress is zero.

$$\tau(u) = 0 \quad \text{for } u = 0 \quad \text{Equation 2.2}$$

$$\tau(u) = \tau_y \quad \text{for } u > 0 \quad \text{Equation 2.3}$$



Figure 2.7 Interface Shear of Soil-Geosynthetic Model. (Zornberg et al., 2017)

2.4.3 Formulation

The differential equation for the soil-geosynthetic interaction was obtained by analyzing the local force equilibrium within a controlled differential segment of the geosynthetic as shown in Figure 2.8 (Zornberg et al., 2017). The equation is as follows:

$$\frac{dT}{dx} = -2\tau \quad \text{Equation 2.4}$$

The length of the differential segment is dx , T is the geosynthetic unit tension and τ is the interface shear stress between soil and geosynthetic.

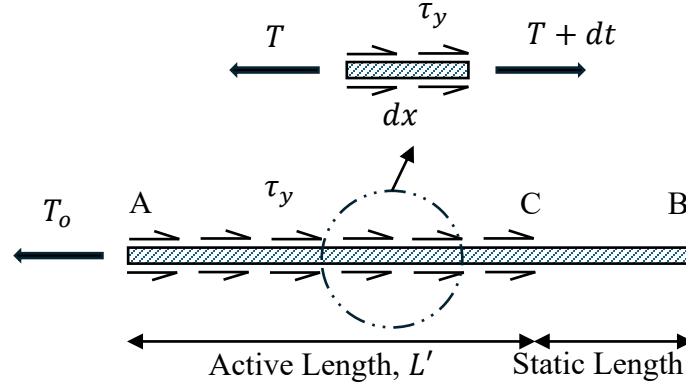


Figure 2.8 Boundary Conditions used in the Model. (modified from Zornberg et al., 2017)

The soil-geosynthetic interface shear is constant along the active length of the geosynthetic (L') where the shear has been mobilized. Thus, it had been assumed that it would not be a function of displacement. The strain (ϵ) at location x can be represented as the derivative of the geosynthetic displacement (u) at that location. J_c is considered constant under normal pressure with small displacements. By combining Equation 2.1 and 2.3 to Equation 2.4 and performing integration twice, a solution for the displacement u is obtained and expressed as follows:

$$u = \frac{d^2u}{dx^2} x^2 + c_1 x + c_2 \quad \text{Equation 2.5}$$

Two boundary conditions were considered in determining the constants. Looking back at Figure 2.8, for a specific confining pressure and applied tension, shear is mobilized only within a segment of the geosynthetic, referred to as the active length (L'). As the applied load increases, this active length progressively increases as well. The interface shear resistance continues to mobilize incrementally until it is fully engaged along the entire length of the geosynthetic or until geosynthetic fails in tension, whichever occurs first. At point A, which is the loading front of the

geosynthetic, a force boundary condition can be applied. On the other hand, at point C, which is the end of the active length, and the start of the station section begins, a displacement boundary condition can be assumed. This incorporates a moving boundary condition to solve the differential equation. Thus, the boundary conditions are,

$$T = T_0 \text{ at } x = 0 \quad \text{Equation 2.6}$$

$$u = 0 \text{ at } x = L' \quad \text{Equation 2.7}$$

Solving and substituting the constants gives the explicit solutions for u and du/dx :

$$\frac{du}{dx} = \frac{2\tau_y}{J_c}x - \frac{T_0}{J_c} \quad \text{Equation 2.8}$$

$$u = \frac{\tau_y}{J_c}x^2 - \frac{T_0}{J_c}x + \left(\frac{T_0}{J_c}L' - \frac{\tau_y}{J_c}L'^2\right) \quad \text{Equation 2.9}$$

Expressions for the geosynthetic strain and unit tension can be obtained below:

$$\varepsilon = \frac{T_0}{J_c} - \frac{2\tau_y}{J_c}x \quad \text{Equation 2.10}$$

$$T = T_0 - 2\tau_yx \quad \text{Equation 2.11}$$

The model presumes that the interface shear along the active length of the geosynthetic is uniform. Beyond the active length, there is no interface shear. As tension increases, the equivalent interface shear remains constant at τ_y while the active length of the geosynthetic expands. The unit tension along the geosynthetic decreases linearly from a maximum value of T_0 to zero at the end of the active length. There is no tension in the stationary part of the geosynthetic. According to Equation 2.9, displacement in the tensioned geosynthetic follows a parabolic pattern, reaching a maximum value u_0 at the loading front and tapering to zero at the active length's end. The coefficients of this

parabola depend on the frontal unit tension and the active length. Tensile strain along the geosynthetic follows a linear pattern within the active length, directly proportional to the unit tension with a slope of $1/J_c$, whereas beyond the active length, the strain remains zero.

Incorporating Equation 2.9 and Equation 2.11 and simplifying yields an expression that relates the unit tension at any location along the active length to its corresponding displacement, resulting in the following:

$$T(x)^2 = (4\tau_y J_c) \cdot u(x) \quad \text{Equation 2.12}$$

This indicates that there is linear relationship defined between the displacement of the geosynthetic and the square of the unit tension at any portion on the active length. Thus, since the confined stiffness and yield shear stress are constant, their product also remains constant. This value becomes a key parameter for characterizing soil-geosynthetic interaction under small displacements and is referred to as the stiffness of the soil-geosynthetic composite (K_{SGC}), which is expressed:

$$K_{SGC} = 4\tau_y J_c \quad \text{Equation 2.13}$$

2.4.4. Interpreting K_{SGC} value

Zornberg et al. (2017) proposed multiple methods for obtaining the K_{SGC} value. The procedures for the methods are described in 2.4.4.1 and 2.4.4.2.

2.4.4.1 Linearity between Unit Tension Squared and Displacements (*Method 1*)

The frontal unit tension at the moment when displacements first appear in each telltale must be carefully recorded, as it is essential for determining the unit tension at each telltale's location. As illustrated in Figure 2.9, the frontal unit tension is a function of displacement for telltale i . It is

then shifted by the amount of $T_{0,i}$, defined as the frontal unit tension when displacement at telltale i is first triggered, to derive the unit tension versus displacement curve at location i . According to the model, this curve follows a parabolic shape for small displacements. The unit tension versus displacement data can then be transformed to T^2 versus u space by squaring the unit tension values. The parabolic shape then becomes a linear function. In line with the model predictions, the slope gives the value of the K_{SGC} .

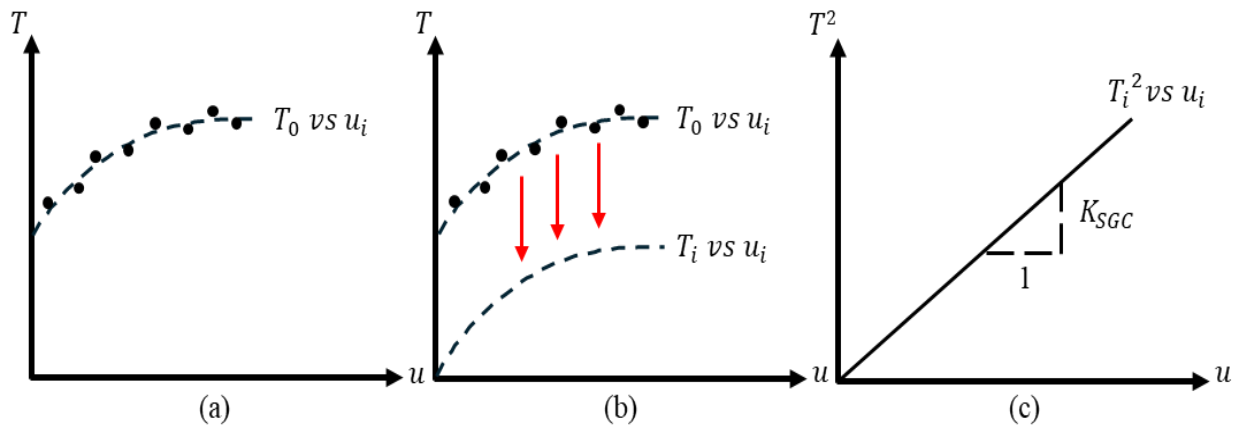


Figure 2.9 Schematic diagram of steps in estimating the K_{SGC} : (a) frontal unit tension vs telltale displacement; (b) unit tension at point i vs telltale i displacement; (c) unit tension squared at point i vs telltale i displacement. (Zornberg et al., 2017)

2.4.4.2 Displacements along Embedment Length (*Method 2*)

An alternative method to determine the K_{SGC} is to use the geosynthetic displacements along the active length directly. Since the model predicts that displacements follow a parabolic function based on τ_y and J_c , polynomial regression can be applied to estimate the equation coefficients. By optimizing the values of τ_y and J_c , the best fit to the experimental data will be determined. This optimization is achieved using the least-squares method, which minimizes the sum of squared errors between the predicted and observed displacement values.

$$S_i = \sum_{j=1}^n r_{ji}^2 \quad \text{Equation 2.14}$$

$$r_{ji} = u_{ji,e} - u_{ji,p} \quad \text{Equation 2.15}$$

The total error sum, S , is obtained by integrating S_i over the range of $T_{0,i}$ values corresponding to the displacements used for estimating K_{SGC} . Here, n represents the number of telltales, r_{ji} is the residual between the predicted displacement ($u_{ji,p}$) and measured displacements ($u_{ji,e}$) at telltale j .

2.4.5 K_{SGC} Model Validation

This section describes the validation of the SGC model done by Roodi & Zornberg (2017). The consistency of the parameters predicted by the model was assessed against the anticipated variability, which was established through a thorough evaluation of the variation in soil and geosynthetic properties. In addition, the model's applicability was validated through a comprehensive series of large-scale soil-geosynthetic interaction tests. The model's constitutive relationships and predictive outcomes were critically examined under a range of experimental conditions, including variations in confining pressure, geosynthetic length, reinforcement type, and backfill material.

The model's core assumptions and constitutive relationships, specifically the linear relationship between the square of unit tension (T^2) and displacement (u), were verified across various conditions. These included changes in confining pressure, reinforcement length, geosynthetic type (such as geogrids and geotextiles), and backfill soil type (including sandy and gravelly materials). The model consistently produced linear and unique $T^2 - u$ relationships, regardless of material or test setup, and the values of K_{SGC} remained stable across different scenarios.

Overall, the findings indicated that the confined stiffness of the soil–geosynthetic composite, represented by the K_{SGC} parameter, can be reliably determined through experimental procedures that demonstrate consistency and repeatability.

2.4.6 Effect of Backfill Materials

According to Roodi et al. (2017), the interaction between geosynthetic and base course can be best represented by gravel-geosynthetic interaction. However, K_{SGC} values were comparatively higher when tested in sand than in gravel.

The stiffness of the soil-geosynthetic composite (K_{SGC}) is significantly influenced by the properties of the backfill material used in reinforcement applications. Key characteristics of backfill, such as particle size, shape, gradation, and moisture content, play a critical role in the mobilization of interface shear strength and, consequently, the magnitude of K_{SGC} values.

Granular backfills with well-graded and angular particles provide better mechanical interlock with geosynthetics, resulting in higher K_{SGC} values due to improved load transfer and confinement. In contrast, uniform or rounded particles reduce friction and interlock at the soil-geosynthetic interface, leading to lower stiffness and diminished K_{SGC} .

2.4.7 Effect of Normal Pressure

The K_{SGC} value has been observed to vary with the level of applied normal stress (Roodi & Zornberg, 2017). At higher normal stresses, the geosynthetic specimen may fail in tension before full mobilization of the soil-geosynthetic interaction within the embedded length. Conversely, lower normal stresses may not generate sufficiently large K_{SGC} values to effectively distinguish between different geosynthetic products. Based on this consideration, Roodi et al. (2017) selected a nominal stress of 3 psi (21 kPa) as a suitable normal pressure in pullout tests to calculate K_{SGC} .

because it adequately represents the permanent stress experienced by geosynthetics in stabilized roadway applications and allows for clear differentiation of K_{SGC} values across various conditions.

2.4.8 Effect of Test Direction

The physical and mechanical properties of geosynthetics can differ between the machine and cross-machine directions, variations in K_{SGC} values are expected when evaluating along these two orientations. Roodi et al. (2017) found that geogrids have similar K_{SGC} values for both directions depending on its geometry. However, the cross-machine direction is typically the primary orientation of interaction influencing the performance of geosynthetic-stabilized roadways.

2.4.9 Threshold Value for K_{SGC}

Based on the evaluation of the field performance and findings of the experimental tests, the limits of acceptable values of K_{SGC} were created by (Roodi et al., 2017). An extensive evaluation was carried out to assess the field performance of over 80 pavement test sections constructed on expansive clay subgrades across 10 different sites. The assessment utilized three primary data sources: (1) visual inspections of pavement surface conditions, (2) transverse profile measurements conducted using total station surveys, and (3) historical pavement data records. The findings demonstrated that geosynthetic stabilization significantly enhanced pavement performance under the environmental stresses associated with expansive soils. Notably, test sections incorporating geosynthetics exhibited a markedly lower frequency of longitudinal cracking compared to adjacent control sections without geosynthetic reinforcement, highlighting the effectiveness of geosynthetics in mitigating pavement distress caused by subgrade movement.

To support the design of geosynthetic-stabilized roadways, fifteen geosynthetic products were selected for further evaluation. These products had previously been implemented in the

experimental test sections, allowing for the direct comparison between laboratory test results and observed field performance. This methodology facilitated a comprehensive validation of laboratory findings through correlation with in-situ data, thus increasing the credibility and applicability of the design recommendations.

Furthermore, the performance trends identified in the field were aligned with the corresponding stiffness values K_{SGC} obtained from soil-geosynthetic interaction tests. This alignment enabled the establishment of performance thresholds based on the K_{SGC} parameter, providing a quantifiable benchmark for the selection and design of geosynthetic reinforcements in roadway applications. Geosynthetics with K_{SGC} values higher than 10 were found to have better field performance.

CHAPTER 3

METHODOLOGY

3.1 Overview

This chapter reports on the experimental testing program aimed at determining the soil-geosynthetic composite parameter, K_{SGC} .

Six geosynthetics were tested: two geotextiles, two geogrids, one non-woven geotextile and a geocomposite. The selected geosynthetics were approved geosynthetics and meet the current requirements of the City of Winnipeg Standard Construction Specifications, specifically CW 3130 and CW 3135. Two backfill materials with different ranges of particles sizes were also identified.

The two backfill materials were characterized in accordance with the AASHTO Soil Classification System and the Unified Soil Classification Systems (USCS). The aggregate used meets the City of Winnipeg Standard Construction Specification CW 3110 for Class A base material sourced from a local material supplier. The clay used is Lake Agassiz clay that was sourced locally in Winnipeg, Manitoba. The grain-size analysis, Atterberg limits and the compaction curve of the soil were determined to classify it.

Pullout tests were conducted using combination of the geosynthetics and the backfill materials to characterize the stiffness of the soil-geosynthetic composite. Additional testing programs were also conducted to evaluate the effect on the results of the normal stress (confining pressure) with respect to the geocomposite and of the geosynthetic variations (geotextile, geogrids and geocomposite) with a constant normal pressure. Overburden pressure was considered and obtained as the product

of the backfill unit weight and a depth of 0.15 m. A total of 11 pullout tests were conducted. Table 3.1 shows the summary of the pullout test materials and setup conditions. The numbering notation shown represents the test number and the objective number connected to it (e.g. Test 1 – Objective 1). Tests 1-1 to 5-1 were done to determine the K_{SGC} of various confining pressures and backfill materials. Test 6-2 to 11-2 were conducted to determine the K_{SGC} of various geosynthetics. The results of Test 8-2 to 11-2 will be utilized to answer objective 3. The results obtained in the testing program were used to establish the stiffness of soil-geosynthetic composite.

Table 3.1 Summary of pullout test materials and setup conditions.

Test Number	Geosynthetic	Normal Pressure (kPa)	Backfill Material (Below geosynthetic)	Backfill Material (Above geosynthetic)	Target Objective
1	GGC-1	7(10.5)*	Clay	Aggregate	1
2	GGC-1	35(38.5)	Clay	Aggregate	1
3	GGC-1	21(24.5)	Clay	Aggregate	1
4	GGC-1	21(23.4)	Clay	Clay	1
5	GGC-1	21(24.5)	Aggregate	Aggregate	1
6	GT-1	21(24.5)	Clay	Aggregate	2
7	GT-2	21(24.5)	Clay	Aggregate	2
8	GG-1	21(24.5)	Clay	Aggregate	2/3
9	GG-2	21(24.5)	Clay	Aggregate	2/3
10	GG-1 + GT-3	21(24.5)	Clay	Aggregate	2/3
11	GG-2 + GT-3	21(24.5)	Clay	Aggregate	2/3

Note: *The bracketed number under normal pressure includes the applied normal pressure plus the overburden pressure from 0.15 of the material above the geosynthetic.

3.2 Soil Properties

The clay used in this study was taken from the South Pointe neighborhood in Winnipeg. Index tests were made to classify the soil as shown in Figure 3.1. The grain size distribution curve of the sampled clay is 100% fines. Only one trial was carried out to determine the grain size as all the samples passed through sieve no. 200. The Atterberg limits of the soil were also determined (ASTM D4318). The soil has a plastic limit of 38%, a liquid limit of 110%, and a plasticity index of 72. Based on the Unified Soil Classification System (USCS) or ASTM D2487, the soil was classified as high plastic clay (CH). The modified proctor test (ASTM D1557) was conducted to determine the optimum moisture content (w_{opt}) and the maximum dry density (γ_{dmax}). The w_{opt} is 18% and the γ_{dmax} is 16.24 kN/m³ determined from the compaction curve shown in Figure 3.2. The soil has poor workability during compaction at w_{opt} .

The coarse-grained soil used in this study consisted of crushed limestone. This limestone, meeting the City of Winnipeg's standards for Class A base material, was sourced from a local supplier and reflects typical base materials used in the city based on CW 3110. This material was also used by Eissa (2023) and they summarized properties of the material shown in Table 3.2.



Figure 3.1 Atterberg Limits (top figure), Hydrometer Analysis (bottom left), Modified Proctor Test (bottom right).

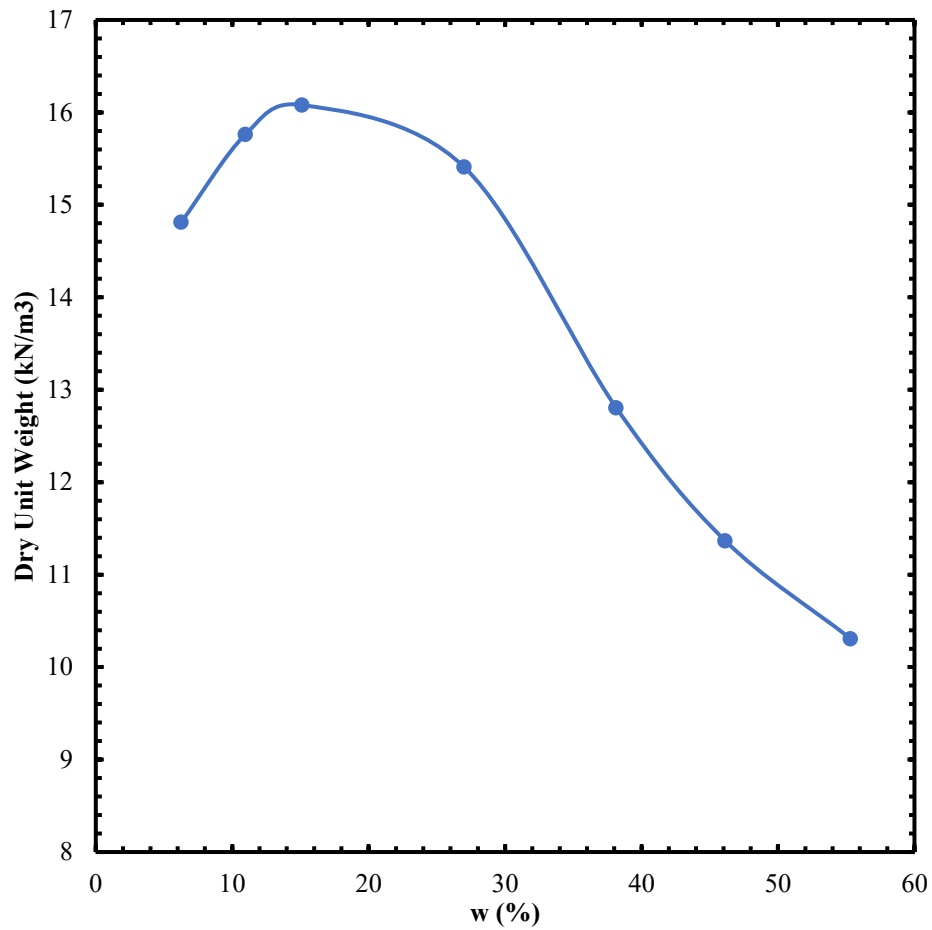


Figure 3.2 Compaction curve of clay soil.

Table 3.2 Properties of Coarse Material Adopted (Eissa, 2023).

Properties	Unit	Value
Cobbles	%	20
Gravel	%	62
Sand	%	13.3
Fines	%	4.7
Uniformity Coefficient (Cu)		27.8
Coefficient of Curvature (Cc)		7.5
Natural Water Content	%	2.17
Maximum Dry Unit Weight	kN/m ³	22.8
Optimum Moisture Content (OMC)	%	5.7
California Bearing Ratio at OMC	%	52.6
ASTM Classification		GP

3.3 Geosynthetic Properties

A total of six geosynthetic materials were tested: two geogrids, two woven geotextiles, one non-woven geotextile and one geocomposite. The two geogrids have different dimensions with different corresponding ultimate strengths. The woven geotextiles have comparable tensile strength to each other. The non-woven geotextile was used as a separator and was used in a unique setup together with the geogrids to mimic the current methodology of installation by the contractors in field applications. The geocomposite is a geogrid bonded with a nonwoven geotextile. The materials included in the testing method are approved by the City of Winnipeg in accordance with CW 3130 and CW 3135 and are examples of geosynthetics used for stabilization. A naming convention was created for the geosynthetics: GT for geotextiles, GG for geogrids and GGC for geotextile-geogrid composite. Characteristics of each geosynthetics are discussed next.

3.3.1 GT-1

GT-1 is manufactured from high-tenacity polypropylene yarns woven into a stable network that maintains the relative positioning of the yarns. A photo of GT-1 is shown in Figure 3.3. Material properties are provided in Table 3.3



Figure 3.3 Woven Geogrid from Polypropylene Yarns: GT-1

Table 3.3 Index Properties of GT-1.

Characteristics		Units	Machine Direction (MD)	Cross-Machine Direction (CD)
Mechanical Properties				
Tensile Strength (ASTM 4595)	at 2% Strain	kN/m (lb/ft)	14 (960)	19.3 (1,320)
	at 5% Strain	kN/m (lb/ft)	35 (2,400)	39.4 (2,700)
	at 10% Strain	kN/m (lb/ft)	70 (4,800)	70 (4,800)
	Ultimate	kN/m (lb/ft)	70 (4,800)	70 (4,800)
Flow Rate (ASTM D4491)		1/min/m ² (gal/min/ft ²)	1,222 (30)	
Permittivity (ASTM D4491)		sec ⁻¹	0.40	
Apparent Opening Size (ASTM D4751)		mm (US Sieve)	0.60 (30)	
Polymer Type		Polypropylene		
Manufacturing Process		Woven Yarns		

3.3.2 GT-2

GT-2 is a woven geotextile made from high-strength polypropylene monofilaments. It is manufactured through a weaving process that aligns the monofilament yarns in a structured pattern, providing consistent mechanical and hydraulic properties. A photo of GT-2 is shown in Figure 3.4. Material properties are provided in Table 3.4



Figure 3.4 Woven Geotextile from Polypropylene Monofilaments: GT-2

Table 3.4 Index Properties of GT-2.

Characteristics		Units	Machine Direction (MD)	Cross-Machine Direction (CD)
Mechanical Properties				
Tensile Strength (ASTM 4595)	at 2% Strain	kN/m	14	21.9
	at 5% Strain	kN/m	35	43.8
	at 10% Strain	kN/m	-	-
	Ultimate	kN/m	70	70
Flow Rate (ASTM D4491)		l/min/m ²	1,222	
Permittivity (ASTM D4491)		sec ⁻¹	0.50	
Apparent Opening Size (ASTM D4751)		mm (US Sieve)	0.60	
Polymer Type		Polypropylene		
Manufacturing Process		Woven Monofilament		

Both GT-1 and GT-2 have consistent properties but are from different manufacturers.

3.3.3 GT-3

GT-3 is nonwoven needle-punched geotextile made from polypropylene fibers, bonded into a stable network that maintains fiber alignment. These geotextiles are resistant to UV degradation, rotting, biological attack, and chemically inert in typical soil environments. These are for separation and are often used in conjunction with geogrids in local construction practice. A photo of GT-3 is shown in Figure 3.5. Material properties are provided in Table 3.5.



Figure 3.5 Nonwoven Needle Punched Geotextile from Polypropylene Fibers: GT-3

Table 3.5 Index Properties of GT-3.

Characteristics	Units	Value
Mechanical Properties		
Tensile Strength (ASTM 4632)	N (lbs)	911 (205)
Flow Rate (ASTM D4491)	l/min/m ² (gal/min/ft ²)	3,870 (95)
Permittivity (ASTM D4491)	sec ⁻¹	1.40
Apparent Opening Size (ASTM D4751)	Mm (US Sieve)	0.180 (80)
Polymer Type	Polypropylene	
Manufacturing Process	Nonwoven Needle Punched Fibers	

3.3.4 GG-1

GG-1 is a biaxial polypropylene geogrid made by stretching a punched sheet in both directions to form a strong, uniform grid. It features thick ribs and nodes for mechanical interlock with soil. A photo of GG-1 is shown in Figure 3.6. Material properties are provided in Table 3.6.

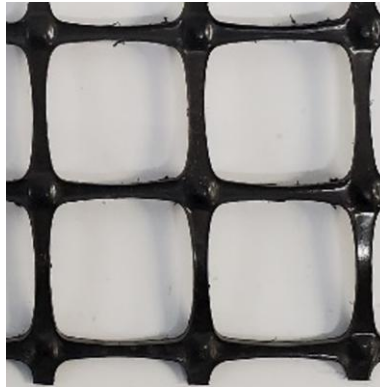


Figure 3.6 Biaxial Punched Geogrid from Polypropylene: GG-1

Table 3.6 Index Properties of GG-1.

Characteristics		Units	Machine Direction (MD)	Cross-Machine Direction (CD)
Mechanical Properties				
Tensile Strength (ASTM D6637)	at 2% Strain	kN/m (lb/ft)	10.5 (720)	10.5 (720)
	at 5% Strain	kN/m (lb/ft)	19.6 (1,343)	19.6 (1,343)
	Ultimate	kN/m (lb/ft)	24 (1,645)	24 (1,645)
Junction Efficiency (ASTM D7737)		kN/m (lb/ft)	22.8 (1,563)	22.8 (1,563)
Flexural Stiffness (ASTM D7748)		mg-cm	1,000,000	
Geometric Properties				
Aperture Dimensions (Nominal)		mm (inch)	34 (1.33)	34 (1.33)
Minimum Rib Thickness (Callipered)		mm (inch)	1.7 (0.07)	1.7 (0.07)

3.3.5 GG-2

GG-2 is a biaxial polypropylene geogrid made by stretching a punched sheet in both directions, forming a strong, uniform structure with thick ribs and nodes. A photo of GG-2 is shown in Figure 3.7. Material properties are provided in Table 3.7.

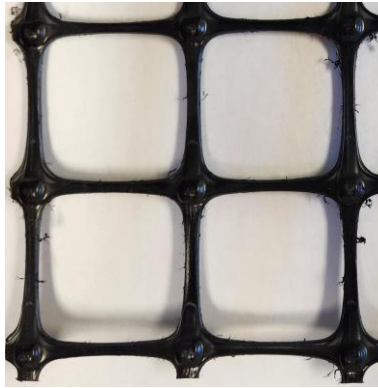


Figure 3.7 Biaxial Punched Geogrid from Polypropylene: GG-2

Table 3.7 Index Properties of GG-2.

Characteristics		Units	Machine Direction (MD)	Cross-Machine Direction (CD)
Mechanical Properties				
Tensile Strength (ASTM D6637)	at 2% Strain	kN/m (lb/ft)	12 (822)	12 (822)
	at 5% Strain	kN/m (lb/ft)	22 (1,508)	22 (1,508)
	Ultimate	kN/m (lb/ft)	30 (2,056)	30 (2,056)
Junction Efficiency (ASTM D7737)		kN/m (lb/ft)	-	-
Flexural Stiffness (ASTM D7748)		mg-cm	1,750,000	
Geometric Properties				
Aperture Dimensions (Nominal)		mm (inch)	38 (1.50)	38 (1.50)
Minimum Rib Thickness (Callipered)		mm (inch)	2.3 (0.09)	2.3 (0.09)

GG-1 and GG-2 look similar but GG-2 has a higher strength, is stiffer and has slightly larger apertures than GG-1.

3.3.6 GGC-1

GGC-1 is a high-stiffness composite geogrid made of biaxial polypropylene, heat-bonded to a wicking non-woven geotextile. A photo of GGC-1 is shown in Figure 3.8. Material properties are provided in Table 3.8.



Figure 3.8 Biaxial Geogrid Heat-bonded to a Wicking Nonwoven Geotextile: GGC-1

Table 3.8 Index Properties of GGC-1.

High Stiffness Bi-Axial Geogrid			
Characteristics	Units	Machine Direction (MD)	Cross-Machine Direction (CD)
Mechanical Properties			
Radial Stiffness at 0.5% Strain	kN/m (lb/ft)	550 (37,697)	550 (37,697)
Secant Stiffness EA at 5% Strain	kN/m (lb/ft)	440 (30,158)	440 (30,158)
Junction Efficiency (GRI-GG ₂ ASTM D7737)	%	>95	>95
Flexural Stiffness (ASTM D7748)	mg-cm	2,000,000	
Geometric Properties			
Aperture Dimensions (Nominal)	mm (inch)	34 (1.34)	34 (1.34)
Minimum Rib Thickness (Callipered)	mm (inch)	2.5 (0.1)	2.5 (0.1)
Nonwoven Wicking Geotextile			
Characteristics	Units	Value	
Mechanical Properties			
Tensile Strength (ASTM 4632)	N (lbs)	740 (166)	
Flow Rate (ASTM D4491)	l/min/m ² (gal/min/ft ²)	4,500 (111)	
Permittivity (ASTM D4491)	sec ⁻¹	1.59	
Apparent Opening Size (ASTM D4751)	mm	0.19	
Polymer Type	Polyester		
Manufacturing Process	Nonwoven Continuous Filament		

The geogrid component of GGC-1 is stiffer compared to GG-1 and GG-2. This is solely based on the flexural stiffness of these products.

3.4 Testing Equipment

The pullout testing box has an internal dimension of 1200 x 600 x 400 mm (L x W x H). The box's bottom section is 250 mm thick, and the top section measures 150 mm. Originally designed for geogrid pullout testing as recommended by others (Alfaro et al., 1995b; Alfaro et al., 1995; Mosallanezhad et al., 2016), the box was modified slightly by adding steel sleeves based on recommendations, as the pullout clamp was embedded within the mixture. During testing, communication cables from strain gauges, the load cell, along with the air pressure gauge for pullout tests, were connected to the data acquisition system.

3.4.1 Pullout Box Set-up and Testing

The preparation details for the testing box and soil are explained here. To reduce the side friction and prevent restrained dilatancy, all four sides of the box were greased and lined with two polyethylene plastic layers (Alfaro et al., 1995a; Hayashi et al., 1996; Ghionna et al., 2001)).

The geosynthetic was anchored to the pullout clamp. For the geotextile, it was wrapped around an aluminum rod and bonded with marine epoxy before a metal bar was screwed onto the pullout clamp (Figure 3.9a). The epoxy was left to cure under compression for at least 12 hours to ensure a strong bond. The aluminum rod enhanced bearing resistance and prevented slippage under pullout forces. For the geogrid and geocomposite, they were screwed onto the clamp with an excess length going behind the clamp as shown in Figure 3.9b and Figure 3.9c, respectively.



(a)



(b)



(c)

Figure 3.9 Clamp setup for: (a) geotextile, (b) geogrid, (c) geocomposite.

Five telltales (T1 to T5) were installed along the geosynthetic, with an additional telltale (T6) on the pullout clamp. As shown in Figure 3.10, the telltale positions used 50 lb (23 kg) tension cables, which were tied to the fabric with plastic bonder epoxy for stability. The telltales were spaced 100 mm perpendicularly to the load direction, and T1 to T5 were positioned away from the pullout clamp edge. These telltales were connected to string potentiometers at the back of the box to measure displacement. The geosynthetic measures 840 mm in length, with an extra length hanging over the end of the box (see Figure 3.11), and 530 or 580 mm in width for geotextiles or geogrids, respectively.

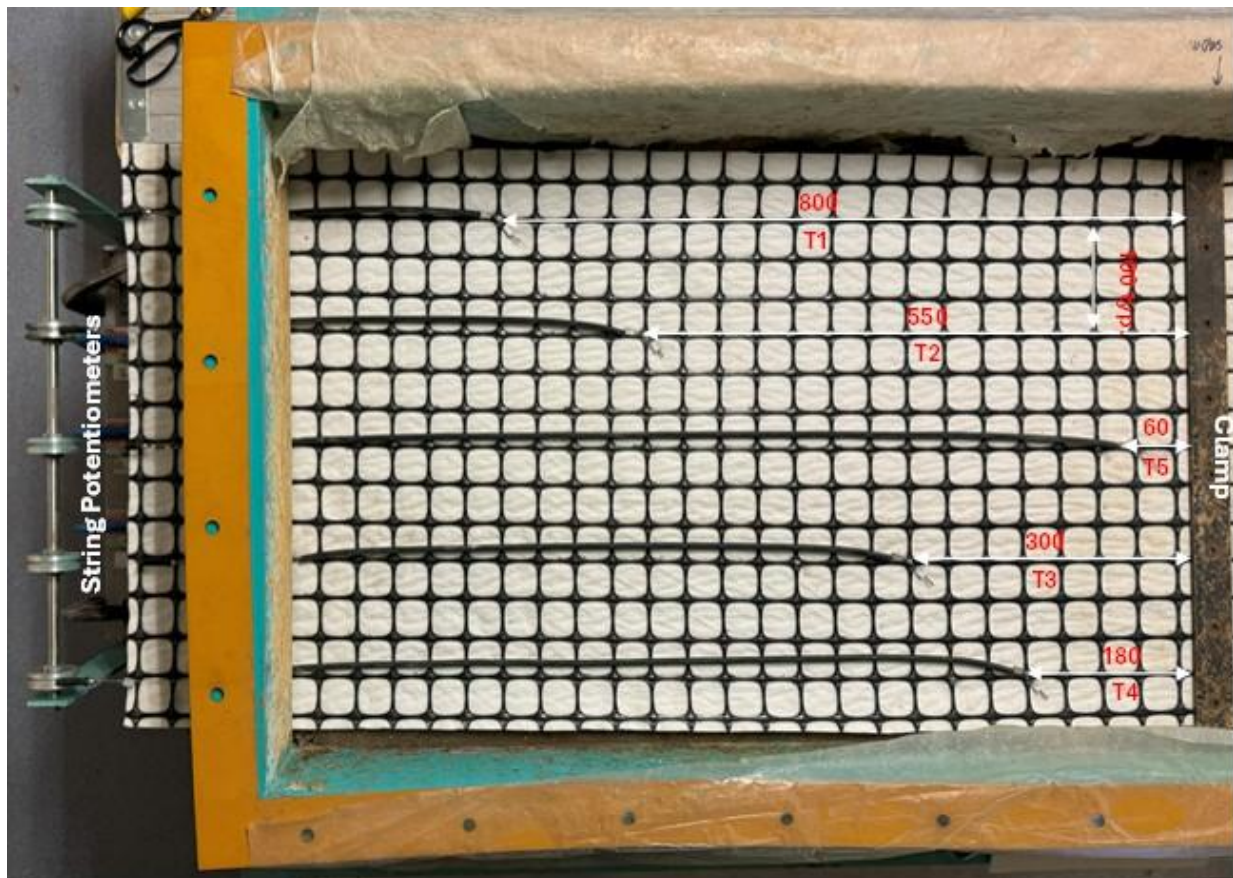


Figure 3.10 Telltale dimensions (in mm).

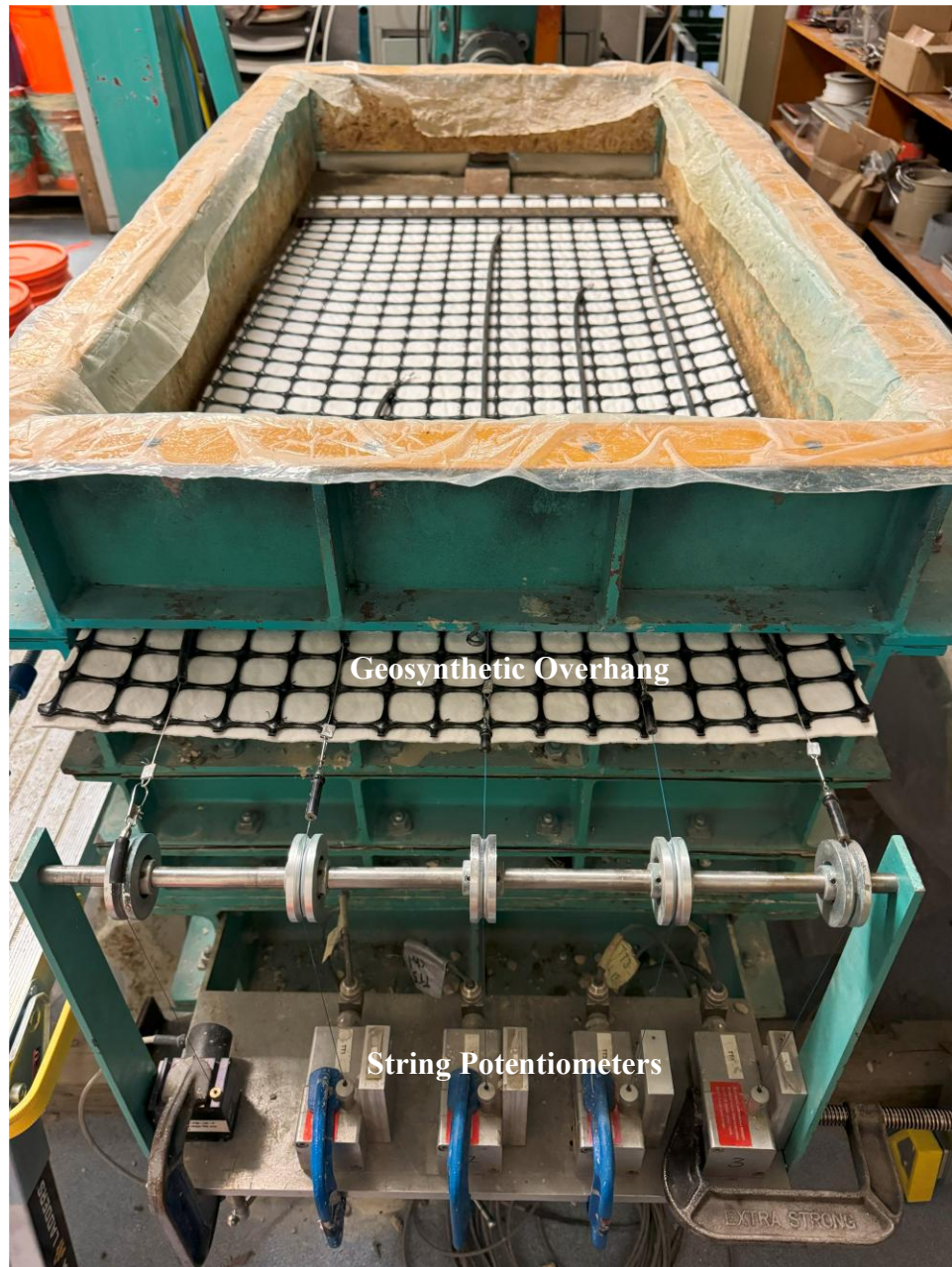


Figure 3.11 Setup at the back of the pullout box.

Using a hammer drill, a 150 mm thick clay base layer was compacted from the bottom (Figure 3.12a). The compacted clay had a natural moisture content of approximately 40%. It was compacted in layers of 50 mm thickness each above the base up until the bottom section was filled (elevation that matches the level of the horizontal load cell). The pullout clamp with the geosynthetic was positioned on top of this surface and connected to the load cell with a servomotor. The servomotor allowed manual positioning adjustments of the clamp. The top section of the testing box was filled with 150 mm of aggregate (Figure 3.12b), placed in three successive 50 mm lifts, each compacted according to the previously described procedure. The pullout clamp connected to the load cell and geosynthetic embedded within the soil. To ensure an airtight seal during the application of normal stress, the rubber gasket was lubricated using a high-vacuum grease, and the rubber membrane was subsequently rolled over its top surface (Figure 3.12c). The top cap was then installed to seal the upper section of the box. All data cables were connected to the data acquisition system, and air pressure was applied, as depicted in Figure 3.12d. Before initiating the pullout test, the soil-geosynthetic system was allowed to consolidate under the specified normal stress for a duration of 30 minutes, while checking for potential air leaks. The displacement rate of 1 mm/min was employed throughout the test.



(a)



(b)



(c)



(d)

Figure 3.12 Pullout box setup (a) compaction of base material, (b) compaction of top material, (c) placement of rubber gasket and rubber membrane, (d) connection of clamp to load cell and servomotor.

During subsequent tests, the aggregates were excavated and about 100 mm of clay soil was excavated and remixed to achieve the target moisture content for optimum condition. The unit weight of the clay was rechecked by getting core samples (Figure 3.13) and dividing the weight with volume of the sample. The average unit weight was around 15.87 kN/m^3 which is within the range of the 95% compaction rate. Then, the aggregate was added with an appropriate amount of water for compaction. Different setups for the subsequent tests are summarized in Table 3.1.



Figure 3.13 Core sample for measuring unit weight of clay upon compaction.

3.4.2 Resistance of Embedded Pullout Clamp

To assess the resistance of the clamp in the pullout test, tests were conducted under various normal stresses corresponding to the pullout test with geosynthetics. This involved a pullout test without any material attached to the clamp. The preparation method determining the resistance of the pullout clamp followed the same procedure as the pullout tests with geosynthetics. The test used varying normal stresses (7, 21, and 35 kPa) and a displacement rate same as the main pullout test. In this context, gross results refer to test data from pullout tests where both the geotextile and pullout clamp were included. Clamp results refer to data from tests conducted with only the pullout clamp, excluding the geosynthetics. Net results are obtained by subtracting the clamp results from the gross results, isolating the contribution of the geotextile in the pullout test.

3.4.3 Interpretation of Test Results

The pullout test results were analyzed using the method proposed by Alfaro et al. (1995a). As this method has been extensively discussed in their study, only a brief summary is provided here. Prior to applying this approach, it is necessary to account for the influence of the pullout clamp. The gross pullout force (P_G), calibration pullout force (P_C), and net pullout force (P_N) were defined accordingly. To ensure that the subtracted P_C values corresponded to the same global displacements (δ_G), a hyperbolic function was employed as the fitting function. The mathematical formulation of the hyperbolic function is presented in Eq. 3.1, along with a brief outline of the procedure for its application.

$$P_C = \frac{\delta_G + \sigma_n}{a + b\delta_G} \quad \text{Equation 3.1}$$

In Equation 3.1, the parameters a and b represent the hyperbolic intercept and slope, respectively, and are derived from the normalized relationship between $\delta_C/(P_C/\sigma_n)$ and δ_G , where δ_C represents

the global displacement during the calibration test. The displacement values were obtained directly from calibration pullout tests, which do not necessarily correspond to the displacements recorded during pullout tests conducted with the geosynthetic attached. The term P_C/σ_n represents the recorded calibration pullout force normalized by the normal stress applied at the top of the pullout box. The surcharge was also taken into consideration by adding it up to σ_n . A plot of $\delta_C/(P_C/\sigma_n)$ against δ_G yielded an approximately linear trend, allowing for the determination of the hyperbolic parameters, which are subsequently used in Equation 3.1 to compute P_C corresponding to δ_G . The net pullout force P_N is then obtained by subtracting P_C from P_G at the corresponding displacement. This force is normalized by dividing by the geosynthetic width to express the net pullout force per unit width.

The K_{SGC} parameter, as discussed in Section 2.4 and 2.5, can be evaluated by analyzing the relationship between the net pullout force squared and telltale displacement measured at multiple telltale locations. Also, the K_{SGC} value can be defined using Equation 2.13 by estimating the constitutive parameters through the parabolic regression of the telltale displacement data.

CHAPTER 4

GEOCOMPOSITE RESULTS

This chapter primarily focuses on the evaluation of GGC-1, a novel geosynthetic product, as part of a collaborative research initiative supported by Titan Environmental. The investigation focuses on analyzing the stiffness of GCC-1 to gain a deeper understanding of its performance in soil stabilization and reinforcement applications. Given its unique composite structure, which combines biaxial polypropylene geogrid with a wicking non-woven geotextile, this product represents an innovative solution in geotechnical engineering. The results presented in this chapter addresses objective (1) which aim to provide insight into the mechanical behavior of GGC-1 under various loading conditions and backfill materials.

4.1 Pullout Test Results

4.1.1 Varying Normal Pressures

The results of the pullout tests conducted under the applied confining pressure of 7, 21, and 35 kPa are presented in Figure 4.1. As expected, the pullout tension versus frontal displacement results indicated that ultimate resistance increases with higher confining pressure when comparing 21 and 35 kPa to 7 kPa. The maximum pullout forces for the normal pressures of 7, 21, and 35 kPa were 6.69, 35.43, and 36.85 kN/m, respectively. The gross, calibration and net data of each confining pressures are shown in Appendix A. The failure mode at normal pressures of 7 and 21 kPa involved the pullout of the specimen. In contrast, at a confining pressure of 35 kPa, the specimen failed under tension. It can also be noted that although the maximum pullout forces for 21 and 35 kPa were relatively close, their failure points differed. The higher confining pressure resulted in failure

with a shorter displacement. Since the specimen is a composite geogrid (GGC-1), it was observed that the geogrid broke first while the fabric stretched as illustrated in Figure 4.2.

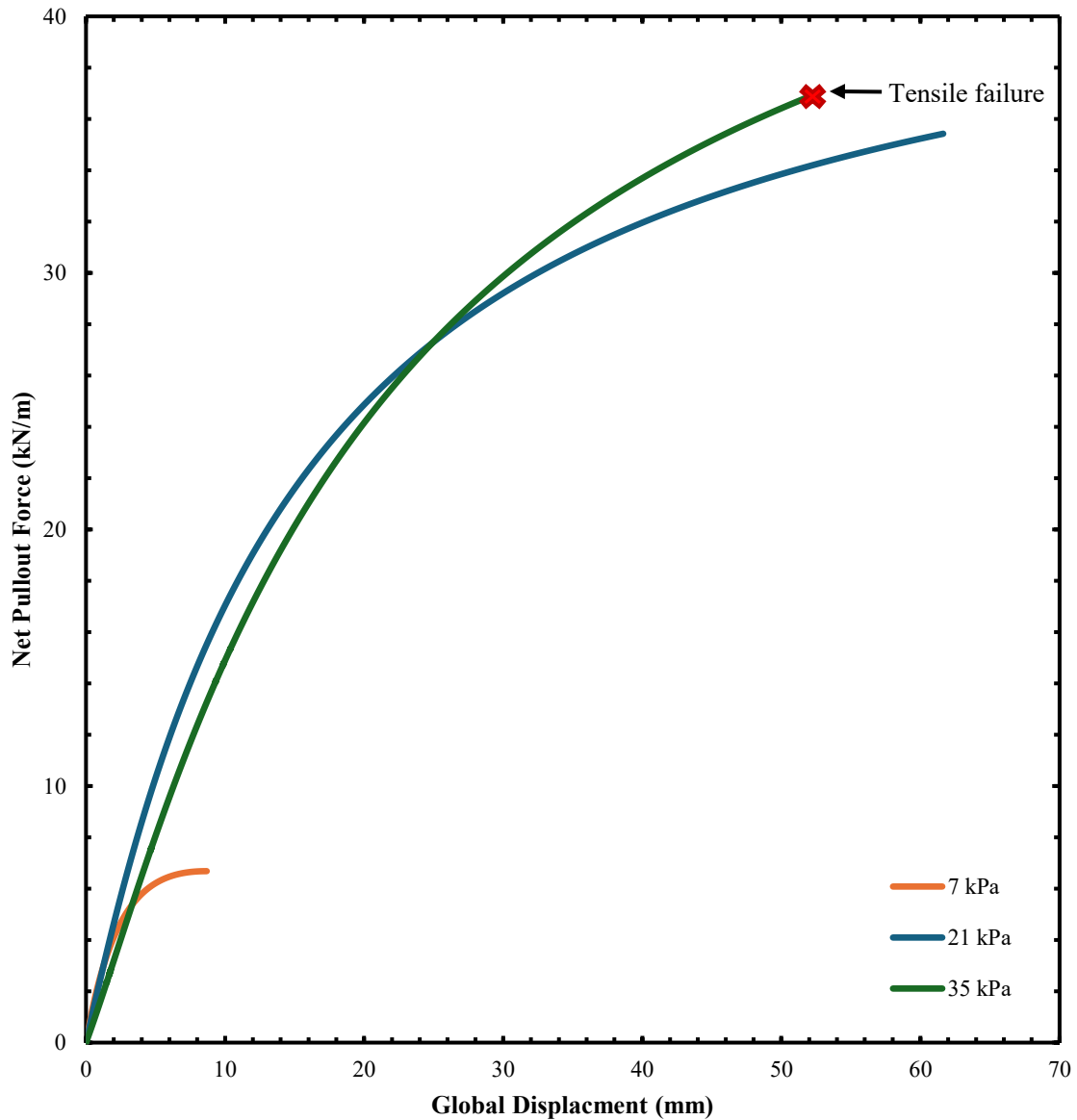


Figure 4.1 Pullout test results of GGC-1 with varying confining pressure.



Figure 4.2 Failure of GGC-1.

4.1.2 Different Soil Interface

The results of the pullout tests conducted using varying backfill materials are shown in Figure 4.3. The varying backfill materials include: aggregate at both above and below the geocomposite (Aggregate-Aggregate), clay at both above and below the geocomposite (Clay-Clay) and aggregate above and clay below the geocomposite (Aggregate-Clay). A constant confining pressure of 21 kPa was used in all tests. The surcharge load above the geocomposite were also taken accounted for in determining the pullout forces. Significant differences were observed in the peak pullout forces and the associated displacement behavior.

The aggregate-clay configuration yielded the highest peak pullout force of 35.43 kN/m, mobilized at a displacement of 61.64 mm. This performance can be attributed to a dual mechanism: the soil beneath provides a cohesive contact surface that increases adhesion and resists initial sliding, while the aggregate above introduces the additional vertical confinement. It should be noted that, aside from the pressure applied to the box, the overburden pressure exerted by the material above the geosynthetic contributes additional stress to the interface. This is because the weight of the overlying material applies a vertical load onto the geosynthetic layer, which increases the normal stress at the interface. As normal stress increases, frictional resistance between the soil and geosynthetic is also enhanced, thereby influencing the overall shear interaction behavior. Furthermore, compacted aggregates exhibit higher density, which further increases the confining pressure.

The pure clay backfill configuration achieved a pullout force of 27.09 kN/m at a displacement of 48.97 mm. The soil's cohesive nature facilitated early bond mobilization and relatively high initial stiffness. However, due to its limited ability to generate additional friction under increasing displacement, the total mobilized force remained lower than the aggregate-clay setup. Such

behavior is consistent with prior observations of strain-softening in cohesive soils during interface shearing. Although peak force was lower, the reduced displacement to failure suggested a more brittle response relative to the previous setup.

The pure aggregate configuration exhibited the lowest pullout force of 19.45 kN/m at a displacement of 59.53 mm. The absence of cohesive fines and the dominance of particle-to-particle contact in granular soils result in lower contact adhesion and delayed mobilization resistance. Furthermore, particle rearrangement and dilation under shear can reduce interface continuity and increase slippage.

Aggregates tend to perform significantly better than clay due to their high shear resistance as a material. But in this case, the adhesion between the nonwoven material aspect of the geocomposite and clay contributed to the apparent higher shear resistance resulting in a higher pullout force.

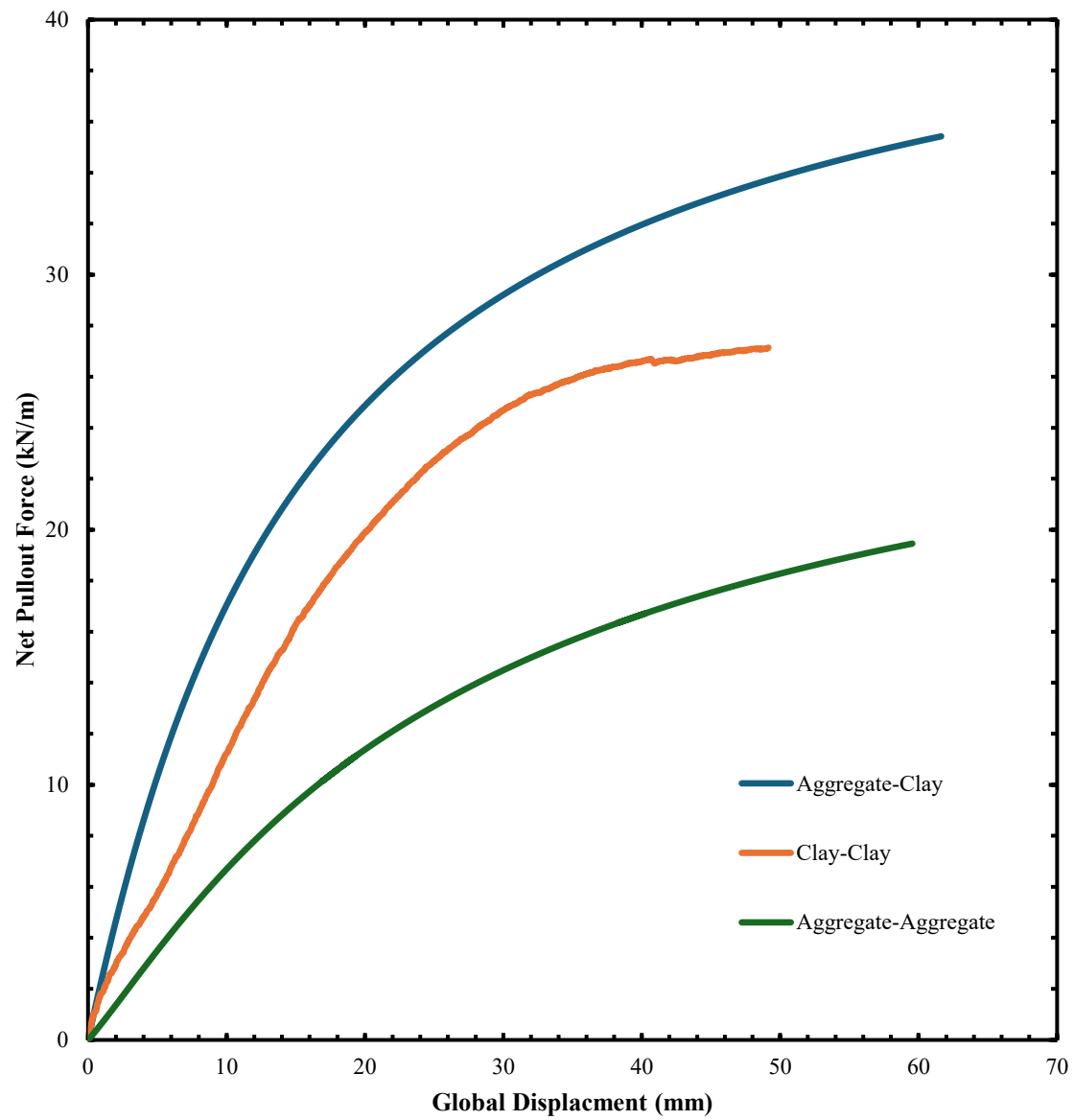


Figure 4.3 Pullout test results of GGC-1 with varying backfill material.

4.2 K_{SGC} evaluation using Direct Estimation from Unit Tension Squared versus Telltale

Displacement (*Method 1*)

4.2.1 Varying Confining Pressure

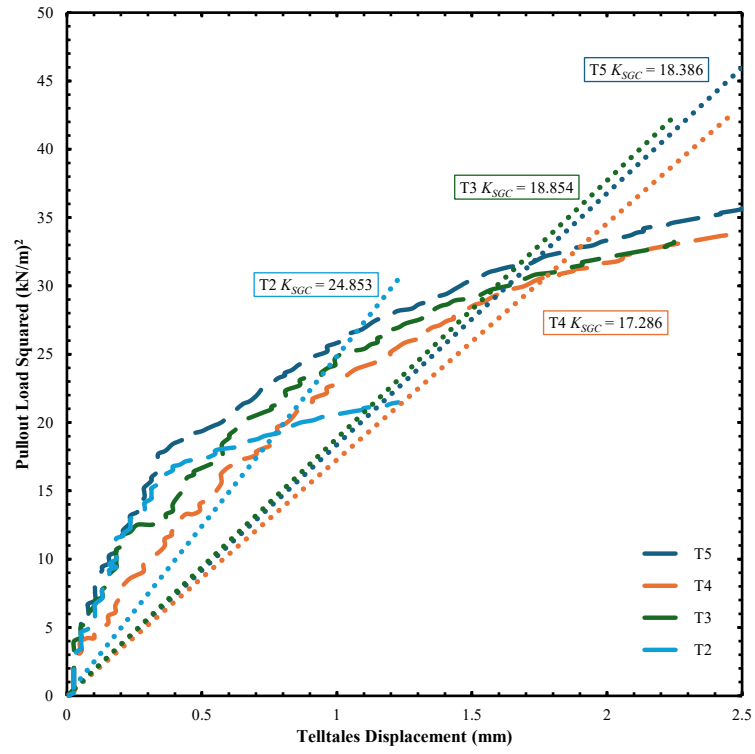
The K_{SGC} was evaluated under confining pressures of 7, 21 and 35 kPa, giving insights into how normal stress influences the stiffness characteristics of the interface. Figure 4.4 shows the different graphs for unit tension squared along with the telltale displacement. Figure 4.5 shows the combined data from Figure 4.4. The K_{SGC} value was determined by taking the average of the linear trendline of the three string potentiometers nearest to the clamp (T3 to T5). The linear trend was evaluated within a 2.5 mm displacement range, as the analysis focused on small displacements.

At the lowest confining pressure of 7 kPa, the K_{SGC} was observed to have a range of 17.29 to 18.85 (kN/m)²/mm with an average value of 18.18 (kN/m)²/mm, consistent with the relatively low pullout resistance and weak confinement. Under such low normal stress, contact friction and interlocking between the soil particles and geosynthetic surface are minimally developed, resulting in low stiffness with limited load-transfer efficiency.

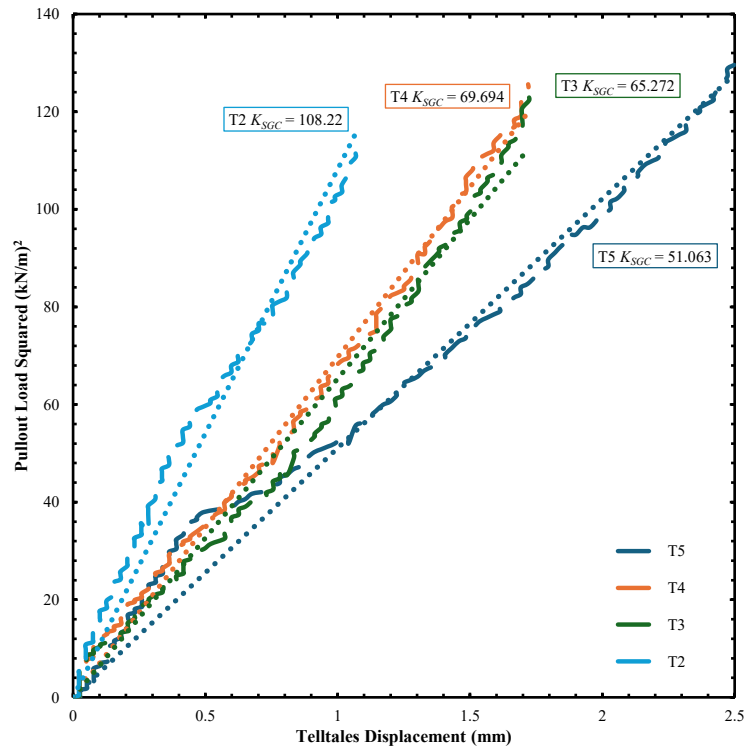
As the confining pressure increased to 21 kPa, the K_{SGC} improved significantly ranging from 51.06 to 69.69 (kN/m)²/mm with an average value of 62.01 (kN/m)²/mm. The increase in normal stress leads to enhanced mechanical interlock and confinement, reducing voids around the geosynthetic and increasing shear transfer capacity. The stiffer response observed under this condition suggests that the system engages more uniformly along the embedment length and resists deformation more effectively.

Further increasing the confining pressure to 35 kPa resulted in an increase in K_{SGC} ranging from 64.18 to 81.90 (kN/m)²/mm with an average value of 73.86 (kN/m)²/mm relative to the value at

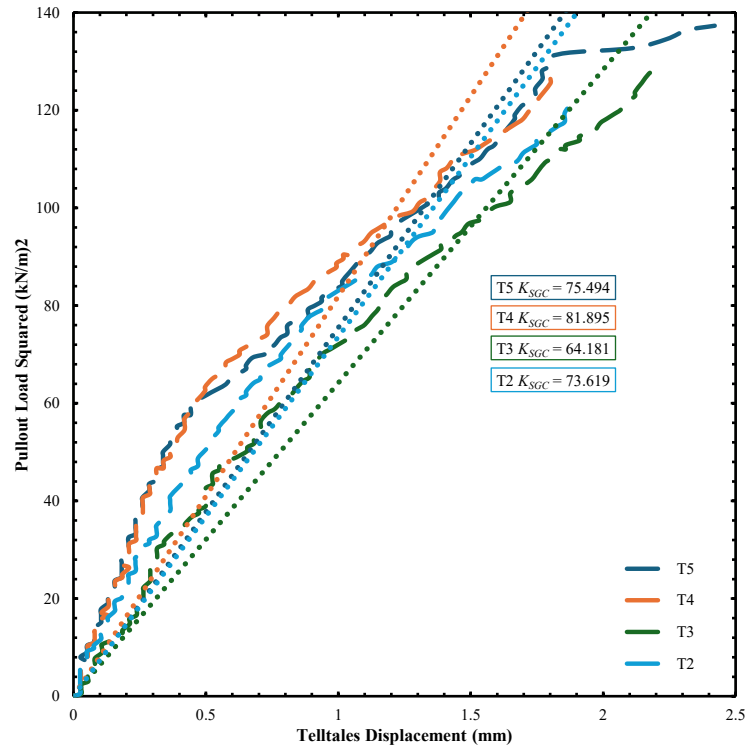
21 kPa. This may be attributed to the maximum engagement of soil particles around the geosynthetic surface, beyond with additional confinement contributes minimally to additional stiffness.



(a)



(b)



(c)

Figure 4.4 Plot of Unit Tension Squared vs Telltale Displacement for GGC-1 at (a) 7 kPa, (b) 21 kPa, (c) 35 kPa.

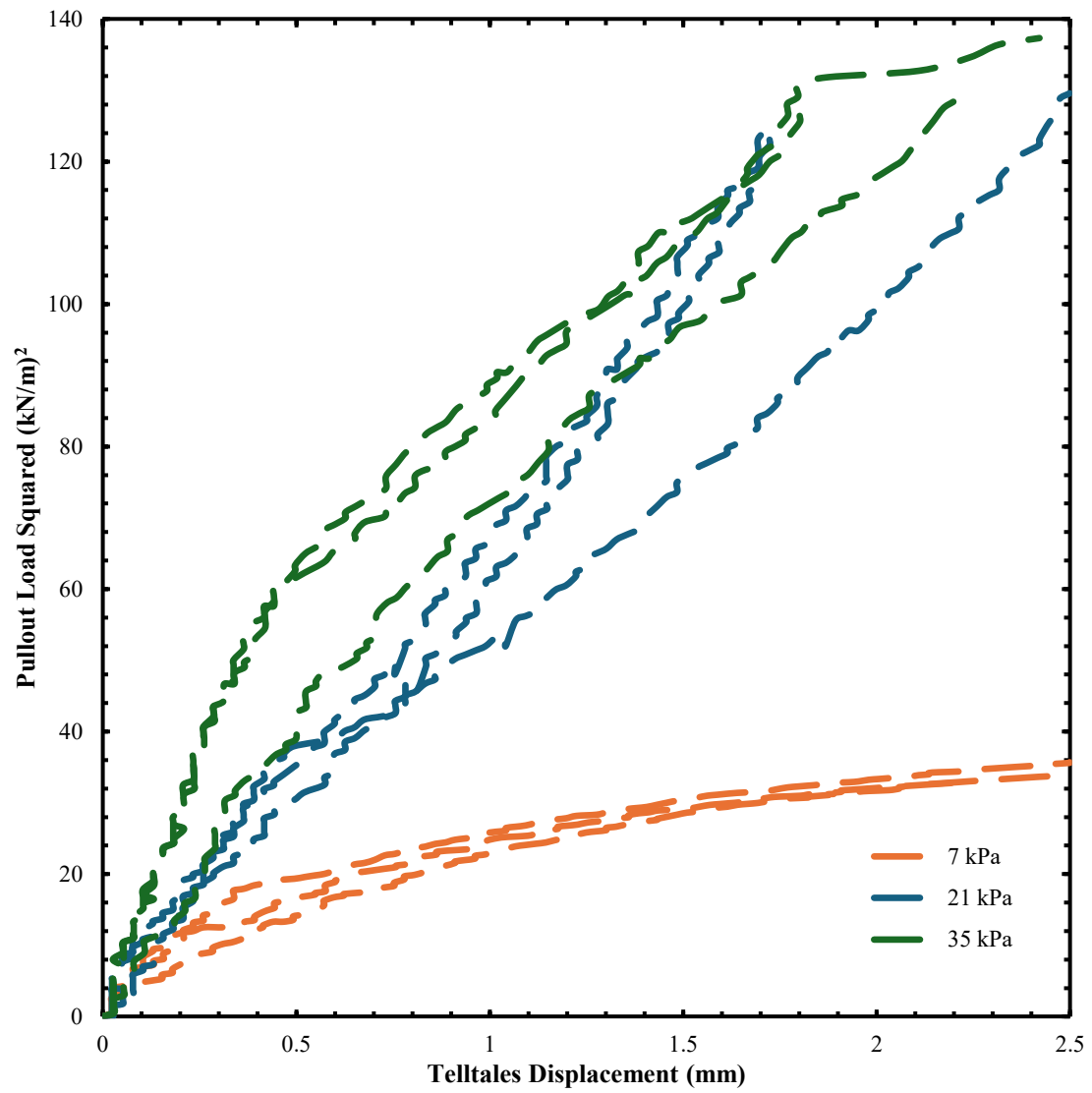


Figure 4.5 Evaluating K_{SGC} values of GGC-1 from varying confining pressures.

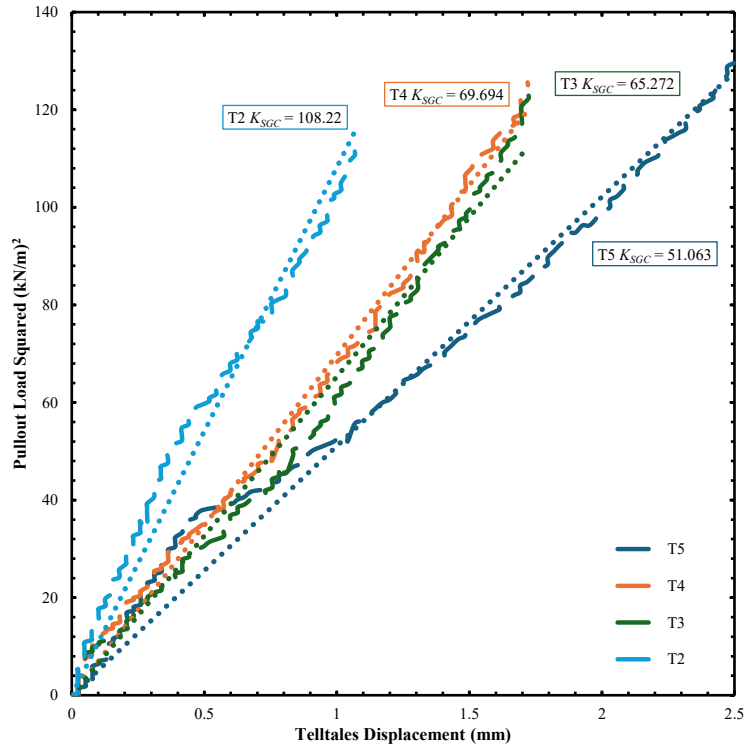
4.2.2 Varying Backfill Materials

The highest average K_{SGC} value of 62.01 (kN/m)²/mm was recorded when an aggregate-clay mixture was used as the backfill material. The results were the same from the 21 kPa test from the previous section. The superior interface stiffness observed configuration may be attributed to the composite interaction mechanism between the geosynthetic and the two distinct materials in contact on either side. This arrangement likely enhances frictional resistance and restricts slippage along the interface, thereby resulting in greater initial stiffness. This configuration presents a hybrid interaction environment that combines the benefits of high contact adhesion with the reinforcing effect of overburden confinement. This suggests that the interface performance is not solely dependent on the material in contact with the geosynthetic, but also on the stress path and confinement conditions induced by the backfill system. Figure 4.6 shows the different graphs for unit tension squared along with the telltale displacement. Figure 4.7 shows the combined data from Figure 4.6.

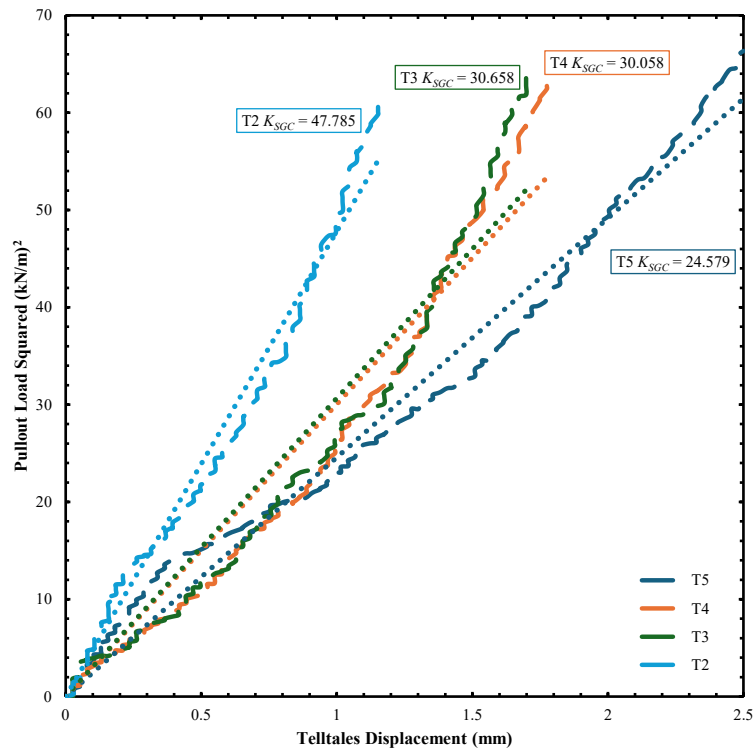
The pure clay backfill yielded a range of 24.58 to 30.66 with an intermediate average value of 28.43 (kN/m)²/mm for its K_{SGC} . The soil's cohesive properties facilitated consistent and adhesive contact with the geosynthetic, which is favorable for shear transfer. However, the absence of a granular component reduces the ability of the system to mobilize high interfacial friction and lateral confinement, particularly under increasing normal pressure. The resulting interface may deform plastically at relatively low strains, limiting stiffness despite the high bonding potential.

The pure aggregate backfill exhibited the lowest K_{SGC} value ranging from 3.04 to 8.68 with an average of 5.54 (kN/m)²/mm. In this setup, the geosynthetic is in direct contact with a coarse, granular matrix that lacks cohesive fines. This condition leads to discontinuous surface engagement and the dominance of point-to-point particle contact, which contributes to lower

frictional resistance and increased likelihood of slippage. Moreover, aggregates tend to rearrange more readily under shear, resulting in delayed mobilization of interface resistance and reduced initial stiffness.



(a)



(b)

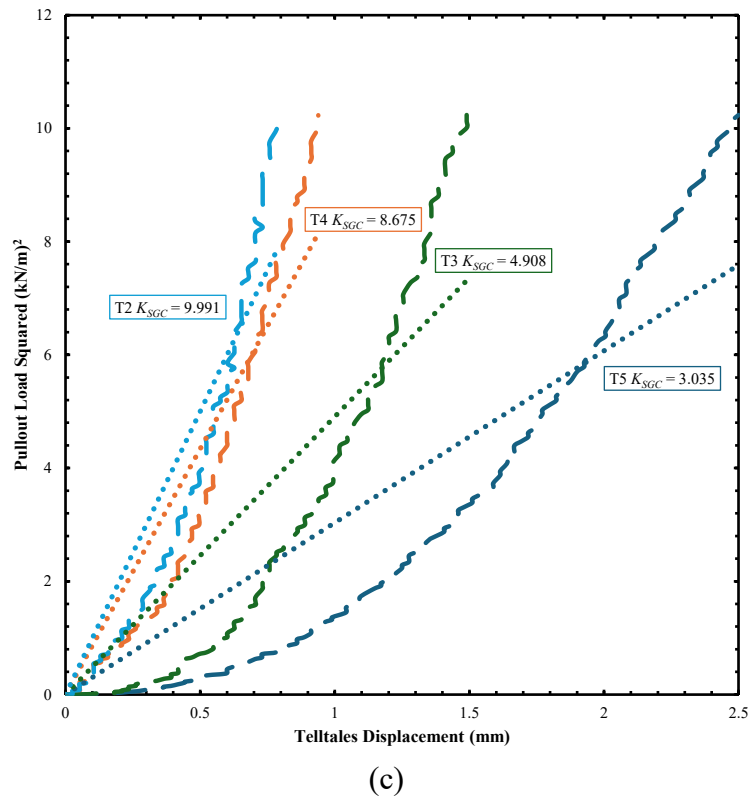


Figure 4.6 Plot of Unit Tension Squared vs Telltale Displacement for GGC-1 at 21 kPa with backfill setup of (a) aggregate-clay, (b) pure clay, (c) pure aggregate.

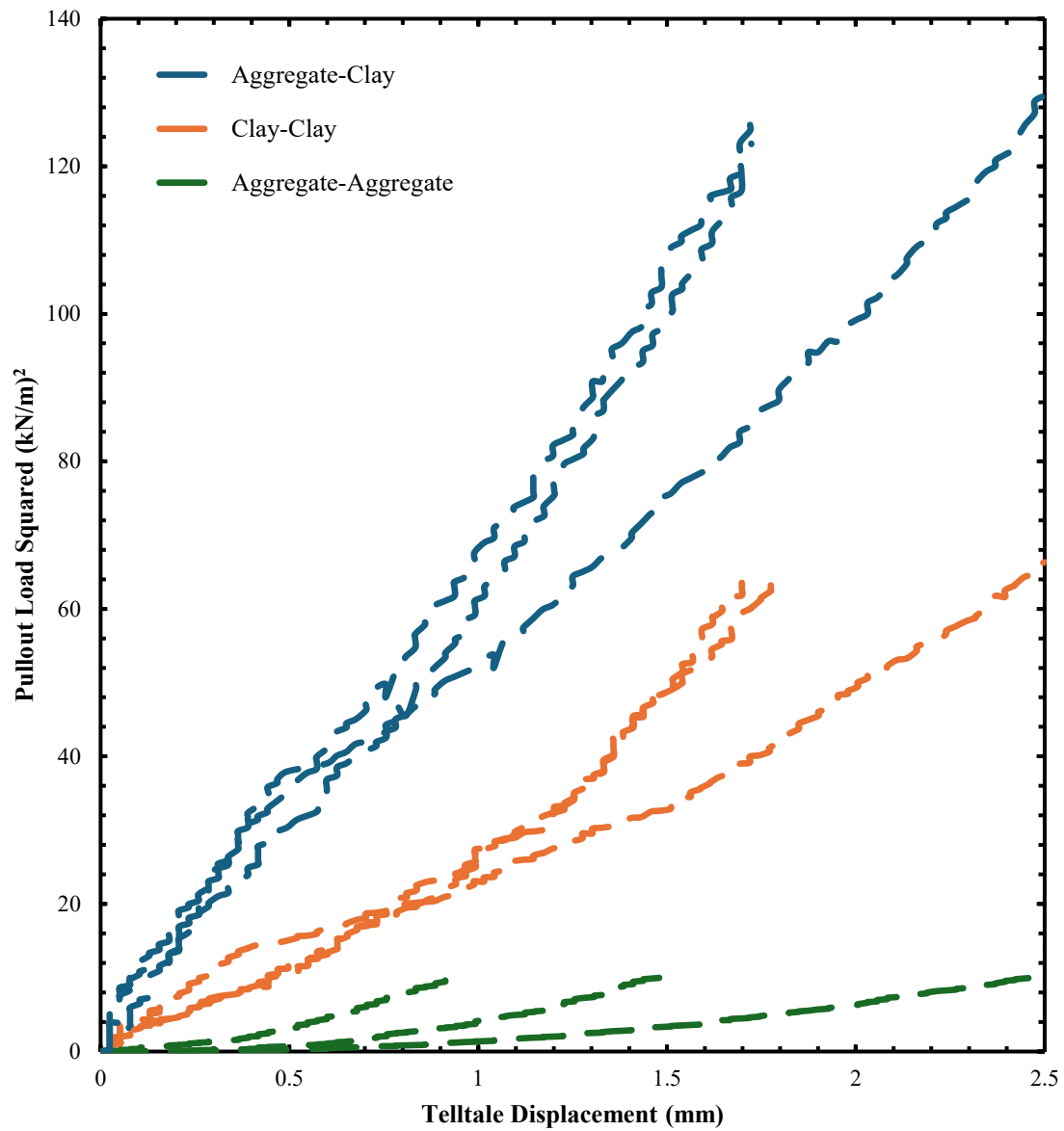


Figure 4.7 Evaluating K_{SGC} values of GGC-1 from varying backfill materials.

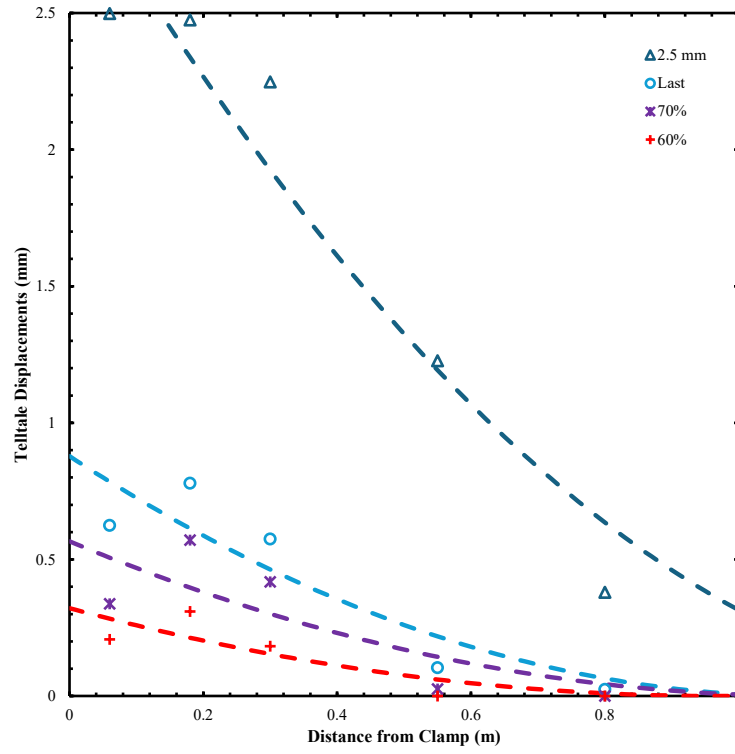
4.3 K_{SGC} evaluation using Estimation from Parabolic Regression of Telltale Displacement Data (Method 2)

4.3.1 Varying Confining Pressure

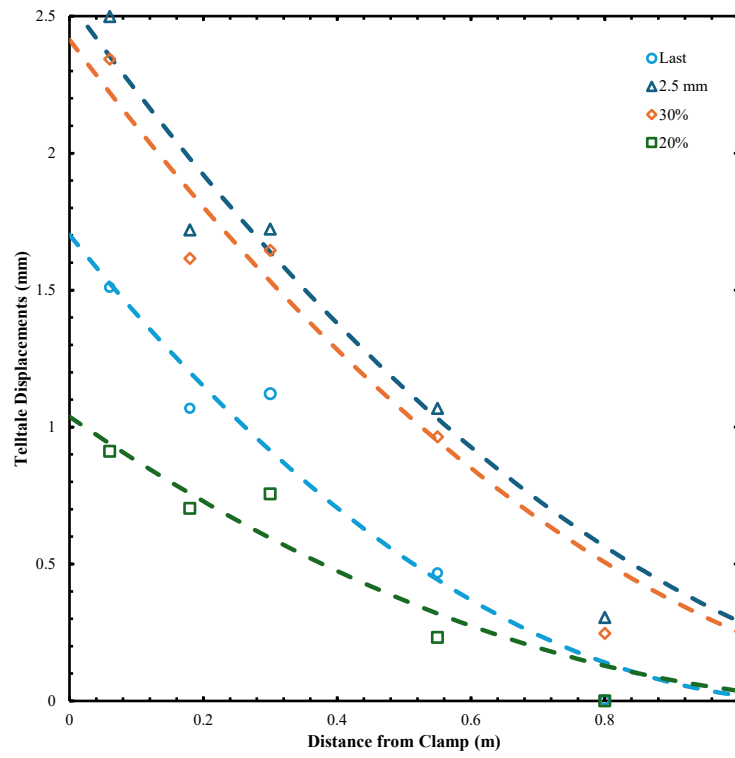
Table 4.1 presents the summary of constitutive parameters derived from the parabolic regression of telltales displacement vs telltales placement. Parameters from the parabolic regression are the shear stress (τ_y), confined stiffness (J_c) and the soil-geosynthetic composite parameter (K_{SGC}). The parabolic regressions were only analyzed up to 2.5 mm telltale displacement or until all the telltales were mobilized since the model only considers small displacements. At 7 kPa, the K_{SGC} value is 21.1 (kN/m)²/mm. Referring to Figure 4.8a, only 60% and 70% of the ultimate pullout load were considered. At 21 kPa, the K_{SGC} value is 42.2 (kN/m)²/mm. For this pressure, only 20% and 30% of the ultimate pullout load were counted (Figure 4.8b). As for the confining pressure of 35 kPa, the K_{SGC} value is 62.5 (kN/m)²/mm which is based on 20% and 30% of the ultimate pullout load (Figure 4.8c). The shear stress of the normal pressures 21 and 35 kPa were similar but differ in confined geosynthetic stiffness (J_c) which is higher on the latter. Figure 4.8 shows the different graphs for unit tension squared along with the telltale displacement. Figure 4.9 shows a summarized comparison of K_{SGC} using *Method 2*.

Table 4.1 Summary of constitutive parameters for varying confining pressures.

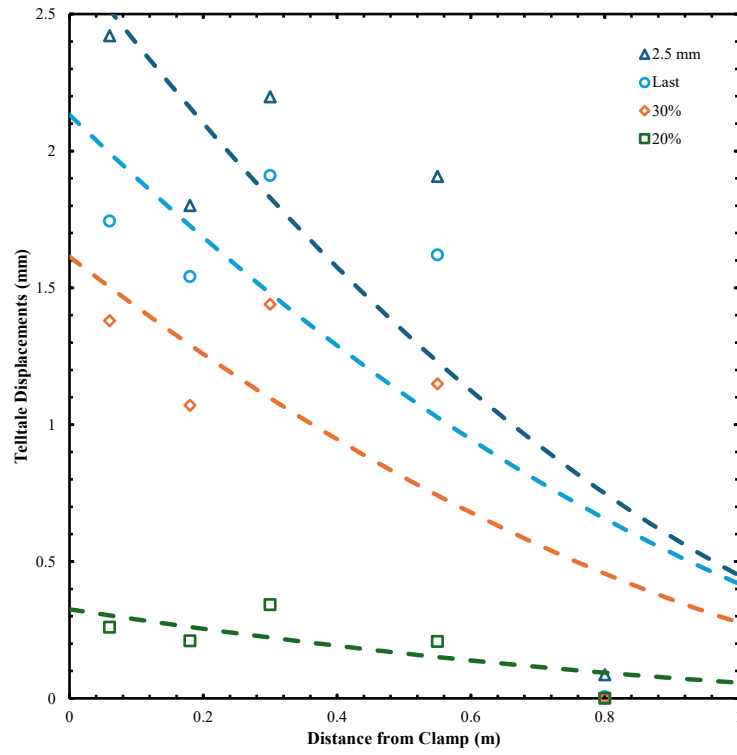
Parameter	Unit	Confining Pressure (kPa)		
		7	21	35
τ_y	kPa	2.2	3.3	3.3
J_c	kN/m	2.3466	3.2251	4.7652
K_{SGC}	(kN/m) ² /mm	21.1	42.2	62.5
S	mm ²	0.18	0.09	0.93



(a)



(b)



(c)

Figure 4.8 Parabolic Regression of Telltale Displacement Data for GGC-1 at (a) 7 kPa, (b) 21 kPa, (c) 35 kPa.

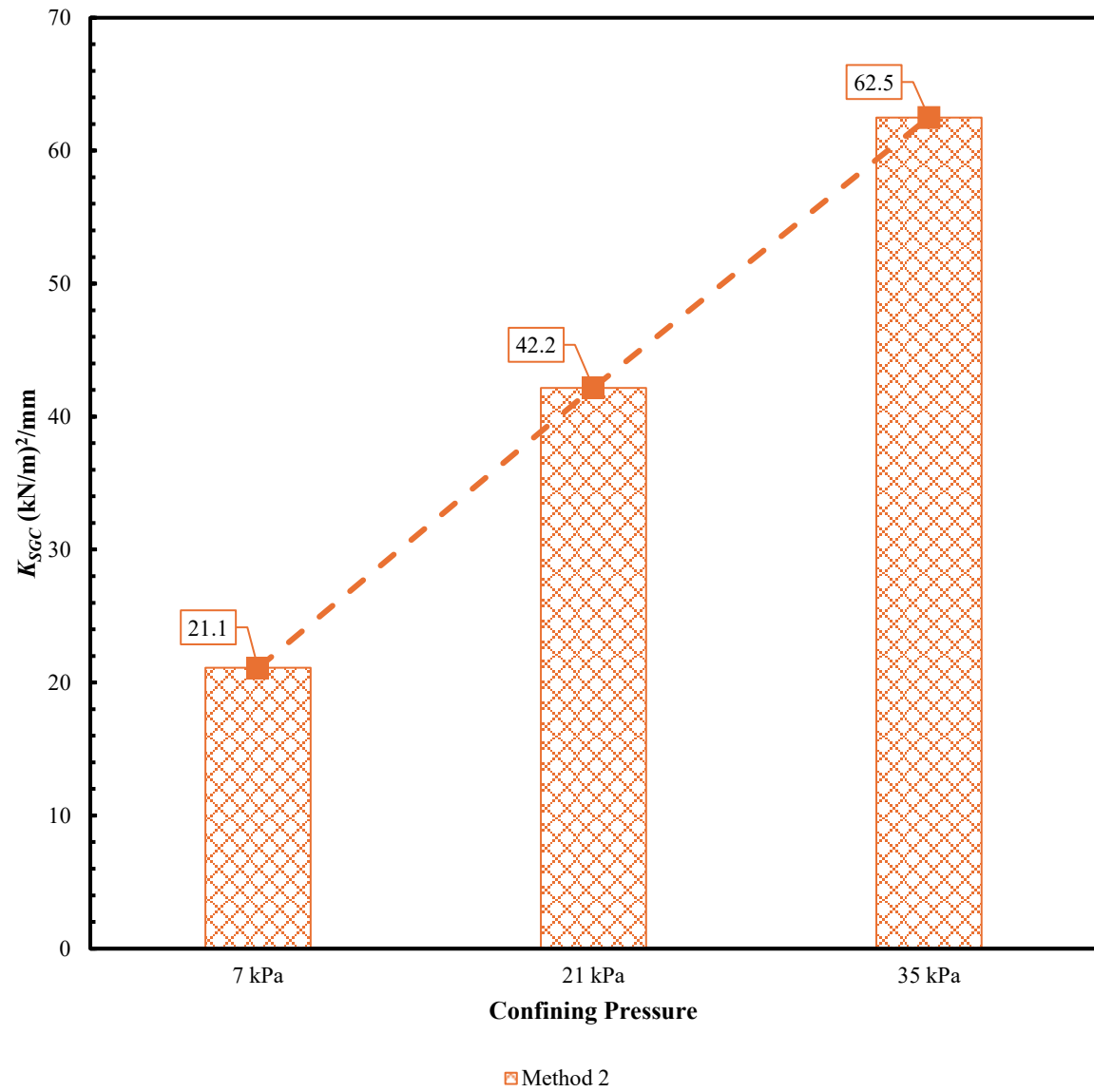


Figure 4.9 K_{SGC} values obtained for GGC-1 from varying confining pressures using *Method 2*.

4.3.2 Varying Backfill Material

The summarized K_{SGC} values from interpreting the parabolic curves are presented in Figure 4.10. Table 4.2 shows the constitutive parameters obtained from the parabolic regression from the different backfill materials. It should be noted that different backfill materials were tested in a confining pressure of 21 kPa.

Observing the parameters, the yield shear stress and the confined geosynthetic stiffness gave the largest value in the Aggregate-Clay setup with 3.3 kPa and 3.2251 kN/m, respectively. These were derived from the 21 kPa confining pressure from the 20% and 30% of the ultimate pullout load (see Figure 4.10a). It should be noted that the material is also a geocomposite which could be also a factor in creating different interaction between the top side and bottom side giving it a combined effect.

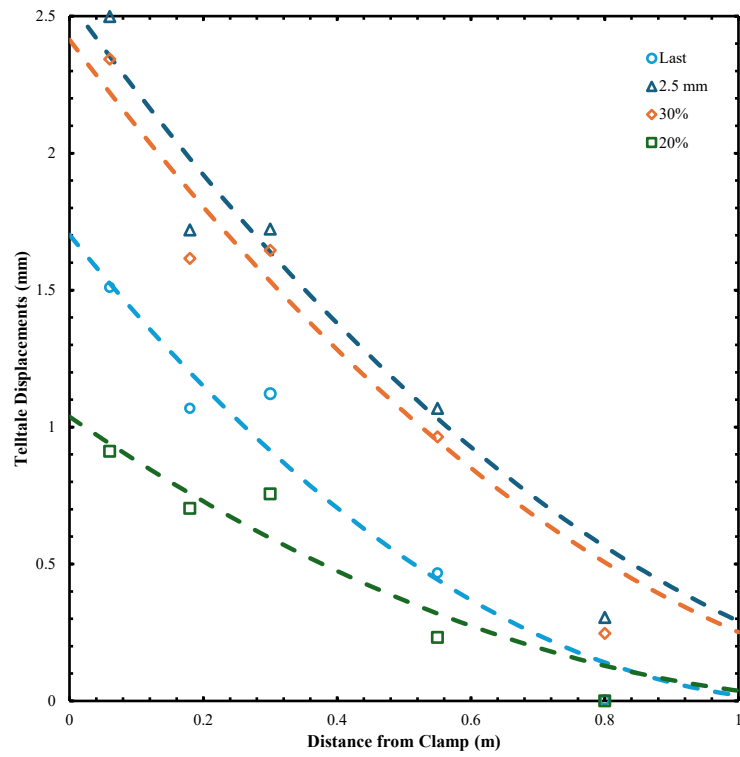
For the Clay-Clay setup, its parameters were determined from the 20% and 30% of the ultimate pullout load. The average yield shear stress and confined geosynthetic stiffness were 2.1 kPa and 2.8637 kN/m.

Lastly, the Aggregate-Aggregate setup had 1.7 kPa and 0.5971 kN/m for its yield shear stress and confined geosynthetic stiffness. This was derived from the boundary of the 2.5 mm displacement parabolic regression curve.

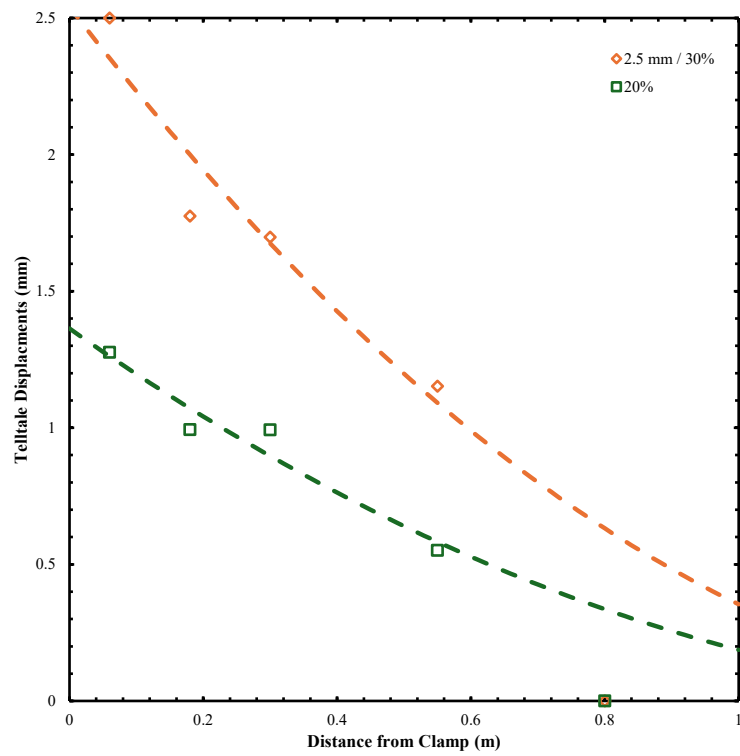
Figure 4.11 shows the summarized K_{SGC} values from the varying backfill materials. The Aggregate-Aggregate material gave a K_{SGC} value of 4.0 (kN/m)²/mm. For Clay-Clay setup, the value of K_{SGC} is 23.9 (kN/m)²/mm. While K_{SGC} for Aggregate-Clay is 42.2 (kN/m)²/mm.

Table 4.2 Summary of constitutive parameters for varying backfill materials.

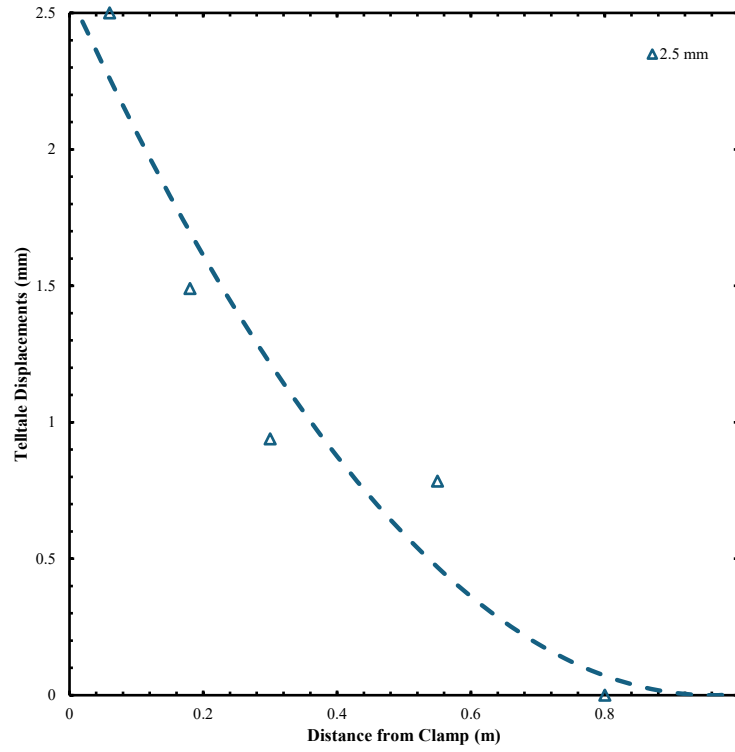
Parameter	Unit	Backfill Material		
		Aggregate	Clay	Aggregate
		-	-	-
		Aggregate	Clay	Clay
τ_y	kPa	1.7	2.1	3.3
J_c	kN/m	0.5971	2.8637	3.2251
K_{SGC}	(kN/m) ² /mm	4.0	23.9	42.2
S	mm ²	0.04	0.05	0.09



(a)



(b)



(c)

Figure 4.10 Parabolic Regression of Telltale Displacement Data for GGC-1 at 21 kPa with backfill setup of (a) aggregate-clay, (b) pure clay, (c) pure aggregate.

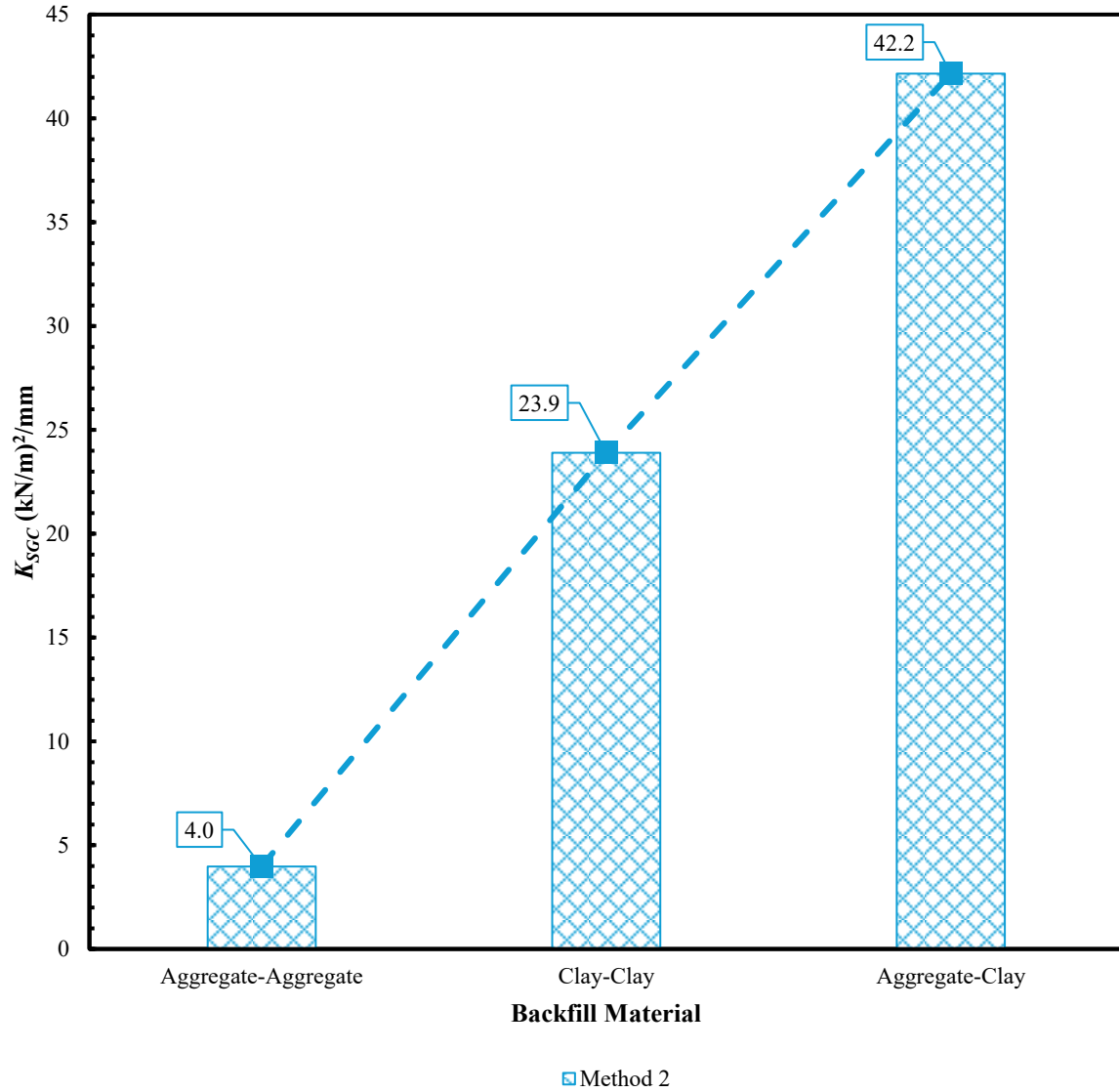


Figure 4.11 Summarized K_{SGC} values obtained for GGC-1 from varying backfill materials using *Method 2*.

4.4 Comparison of K_{SGC} for GGC-1 for *Method 1* and *Method 2*

A summary of the K_{SGC} value for both conditions and methods is presented in Table 4.3. A graphical format showing the comparison of the K_{SGC} is also presented in Figures 4.12 and 4.13 for varying pressures and backfill, respectively. Based on summarized data in Table 4.3, the K_{SGC} value is relatively consistent between the two methods. According to Roodi et al. (2017), a

threshold value of $K_{SGC} > 10$ (kN/m)²/mm was set for any geogrids to be used for base stabilization. Only the Aggregate-Aggregate setup couldn't meet the criterion. This is only a comparison to the threshold value since this study did not include the field performance of the geosynthetics which is essential in making such criterion.

Method 1 did not reflect linear behavior in accordance with the K_{SGC} model. In the study of Zornberg et al. (2017), a uniformly graded backfill was used; however, the results were still nonlinear for both pure clay and pure aggregate setups due to the variability in particle sizes. In contrast, *Method 2* restricted the analysis of the parabolic curves to displacements within 2.5 mm or to the point at which all telltales were mobilized, as the interface shear stress becomes constant beyond the yield point and no longer varies with additional displacement. This explains why the parabolic curves vary from one test to another.

Table 4.3 Summarized values of K_{SGC} for both conditions and methods for GGC-1.

Pressure/Backfill	K_{SGC} (kN/m) ² /mm				Average
	Varying Pressures		Varying Backfill		
	<i>Method 1</i>	<i>Method 2</i>	<i>Method 1</i>	<i>Method 2</i>	
	Direct estimation from T^2 vs u data	Parabolic regression of telltale data	Direct estimation from T^2 vs u data	Parabolic regression of telltale data	
7 kPa	18.2	21.1	-	-	19.7
21 kPa	62.0	42.2	-	-	52.1
35 kPa	73.9	62.5	-	-	68.2
Aggregate-Aggregate	-	-	5.5	4.0	4.8
Clay-Clay	-	-	28.4	23.9	26.2
Aggregate-Clay	-	-	62.0	42.2	52.1

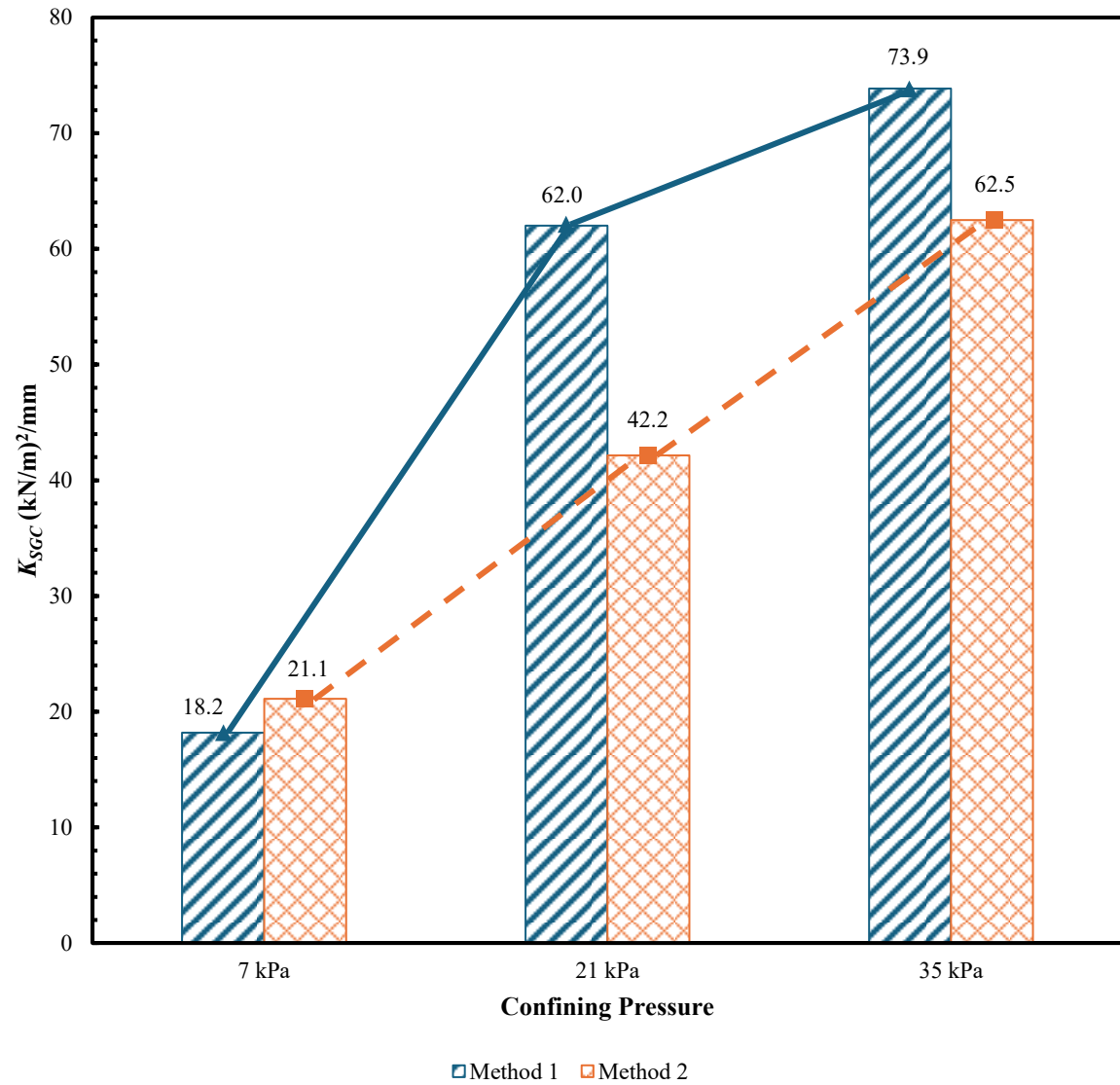


Figure 4.12 Summarized K_{SGC} values obtained for GGC-1 from varying confining pressures using both methods.

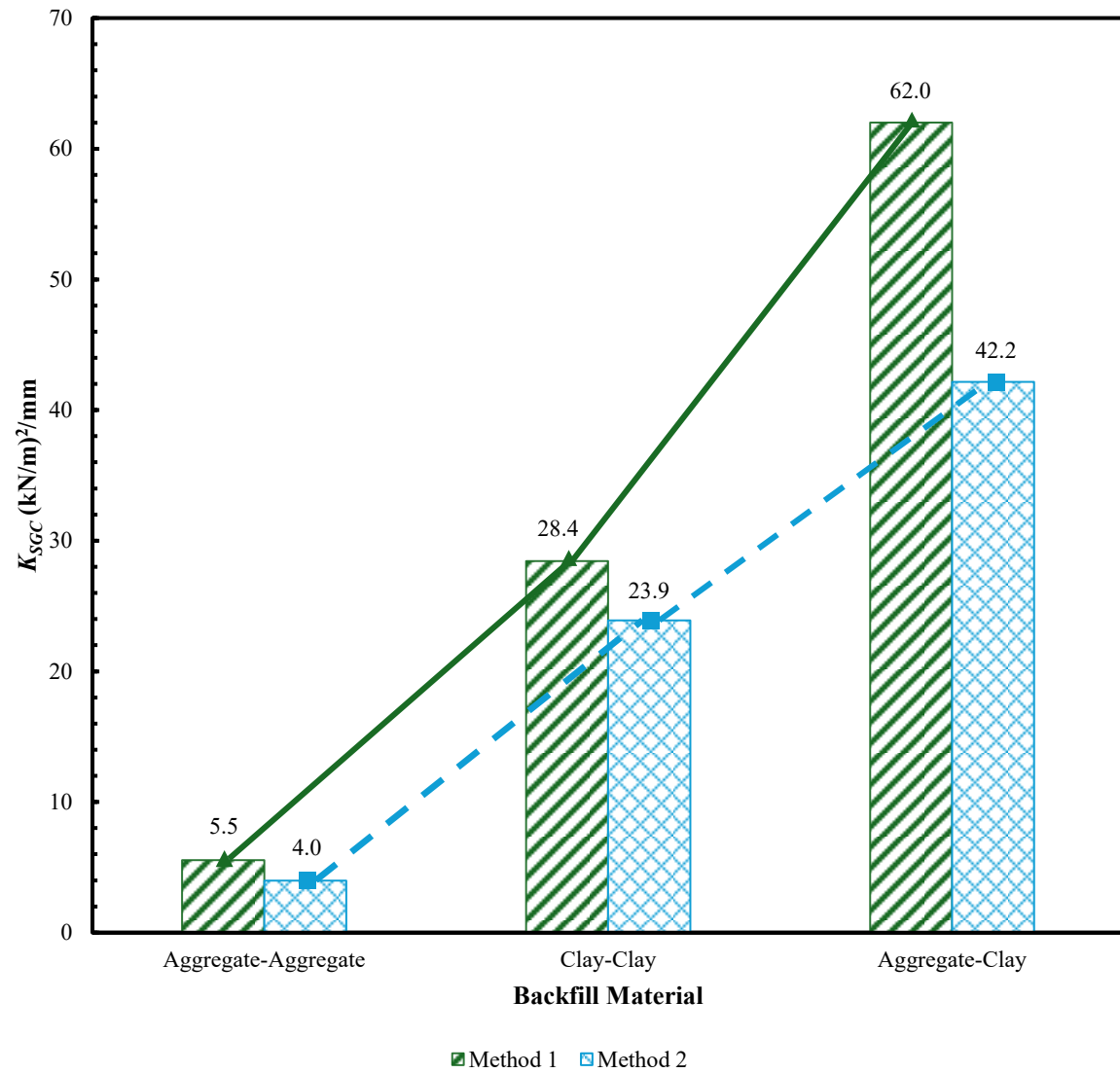


Figure 4.13 Summarized K_{SGC} values obtained for GGC-1 from varying backfill materials using both methods.

CHAPTER 5

GEOTEXTILE AND GEOGRID RESULTS

This chapter presents the results conducted to evaluate K_{SGC} of the geotextiles and geogrids currently used in the specifications for road construction in the City of Winnipeg. The geosynthetics evaluated in this study include geotextiles and geogrids tested within a layered backfill system commonly encountered in Winnipeg roads (Aggregate-Clay setup). The testing was carried out with only a 21 kPa confining pressure for all materials. The surcharge load above the geosynthetics were also taken accounted for in analyzing the pullout forces.

5.1 Pullout Test Results

The net pullout load vs global displacement for all the geosynthetics tested is illustrated from Figure 5.1 to Figure 5.4. The maximum net pullout loads for the geotextiles shown in Figure 5.1 is 19.08 kN/m for GT-1 and 23.05 kN/m for GT-2. The two products have similar specifications in terms of tensile strength; therefore, the results would be expected to be close to each other. For the case of geogrids, two set ups were established. Figure 5.2 shows the results of using only geogrids (Setup 1) while Figure 5.3 illustrates the geogrid with nonwoven fabric set up (Setup 2). In Figure 5.2, GG-1 has a maximum net pullout load of 16.20 kN/m while GG-2 has a maximum net pullout load of 21.38 kN/m. This was an expected outcome since the latter is stiffer than the former. In Figure 5.3, both set ups for GG-1 and GG-2 with GT-3 fabric gave close values which were 23.41 kN/m and 20.53 kN/m respectively. With the introduction of the fabric, the interaction of the materials may have refocused around the fabric which led to a similar result. The aperture size of both geogrids only differs by 4 mm. Figure 5.4 shows that geogrids provide a larger net

pullout load compared to geotextiles with the given set up. The individual results of each material with their gross, calibration and net pullout results are found in Appendix B.

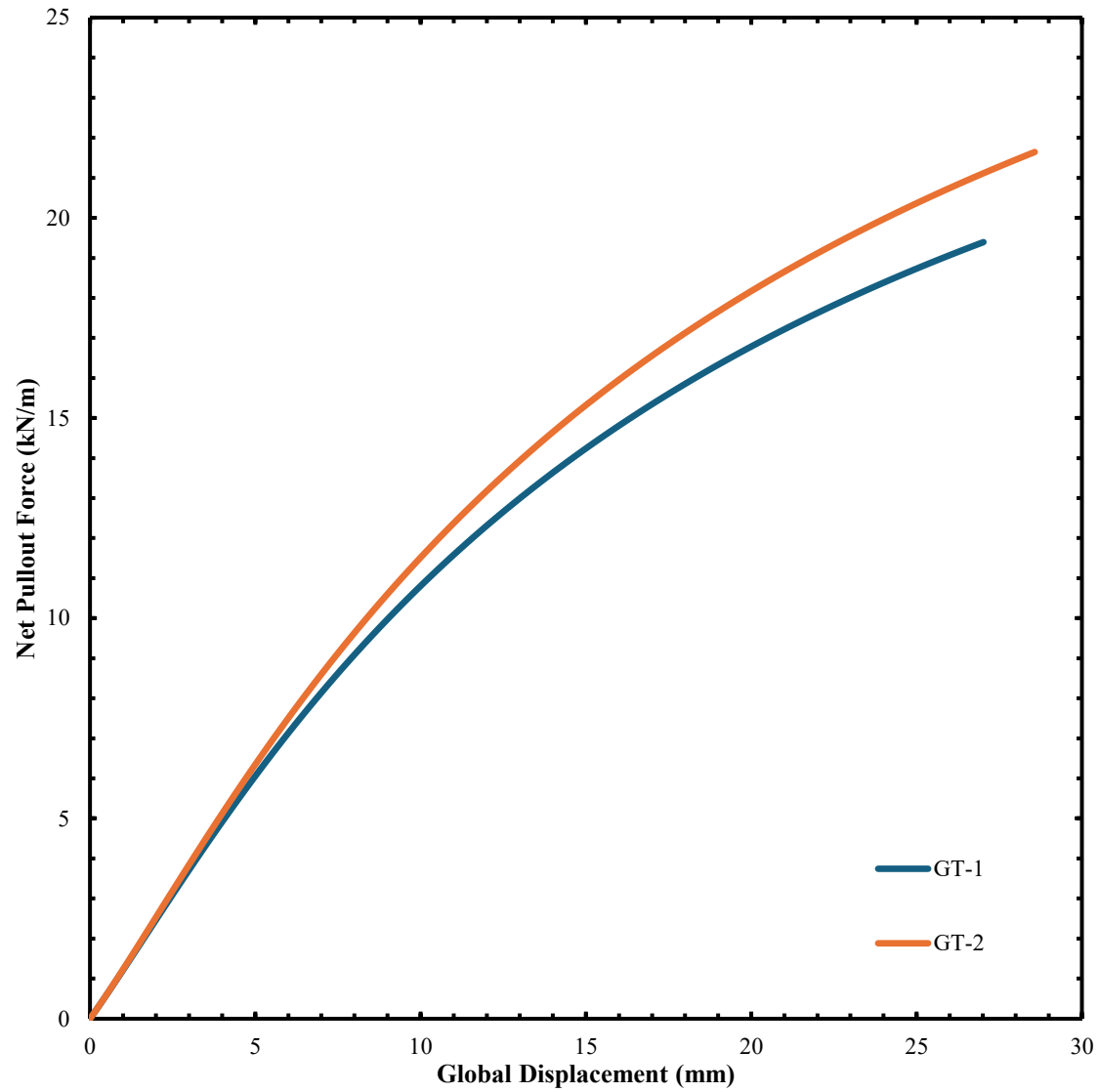


Figure 5.1 Net Pullout Load for Geotextiles.

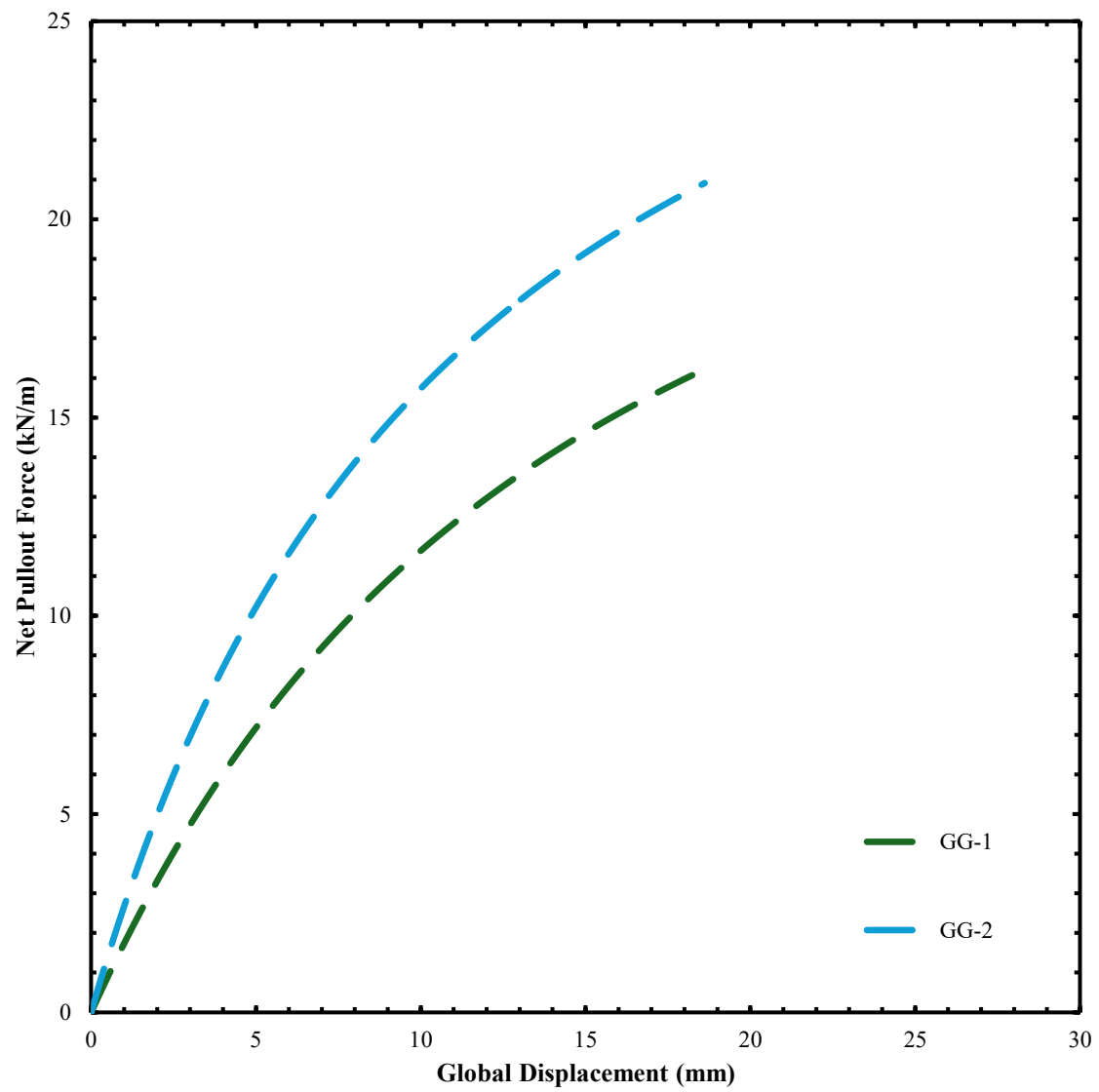


Figure 5.2 Net Pullout for Geogrids.

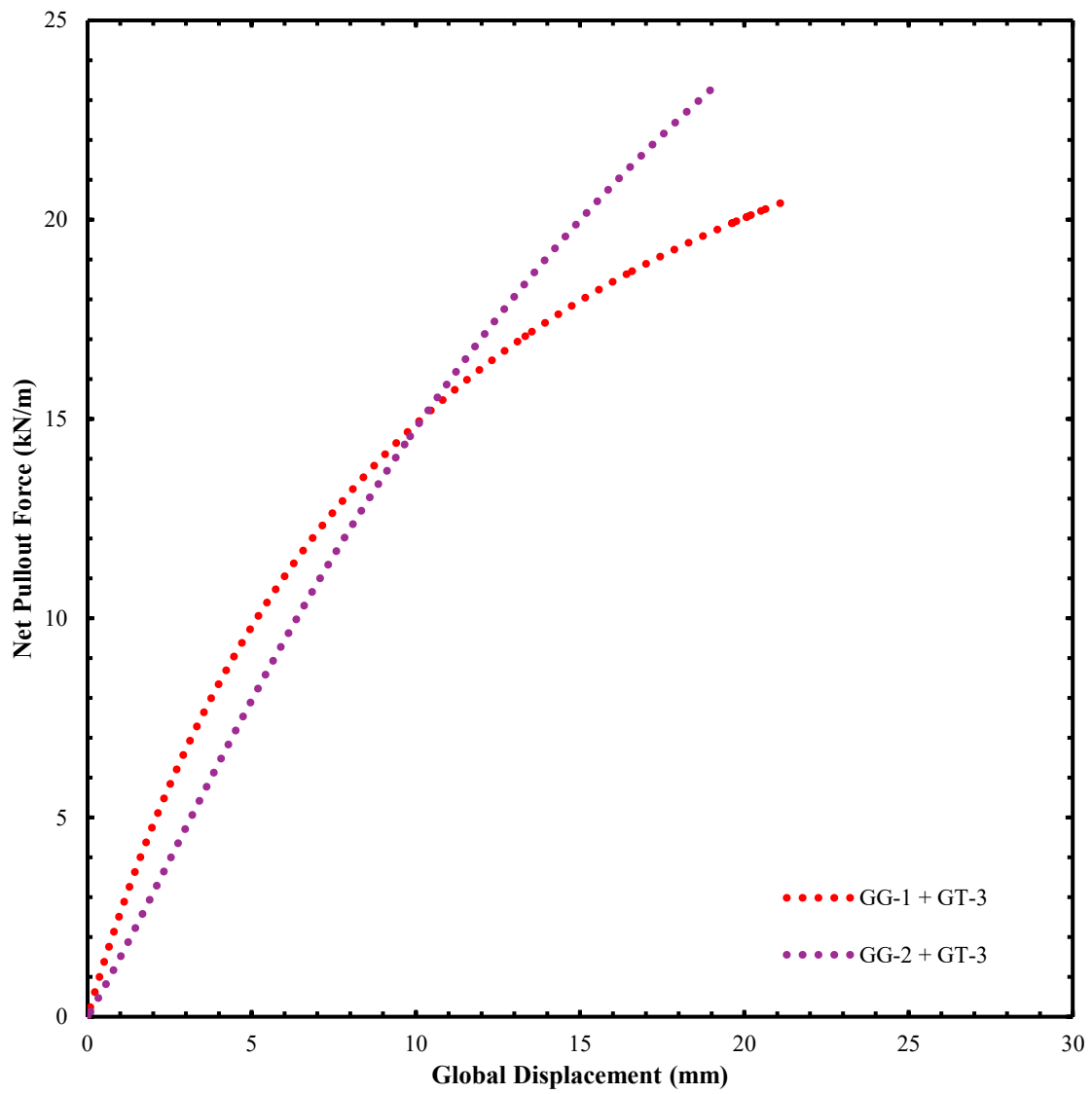


Figure 5.3 Net Pullout Load for Geogrids with Nonwoven Geotextile.

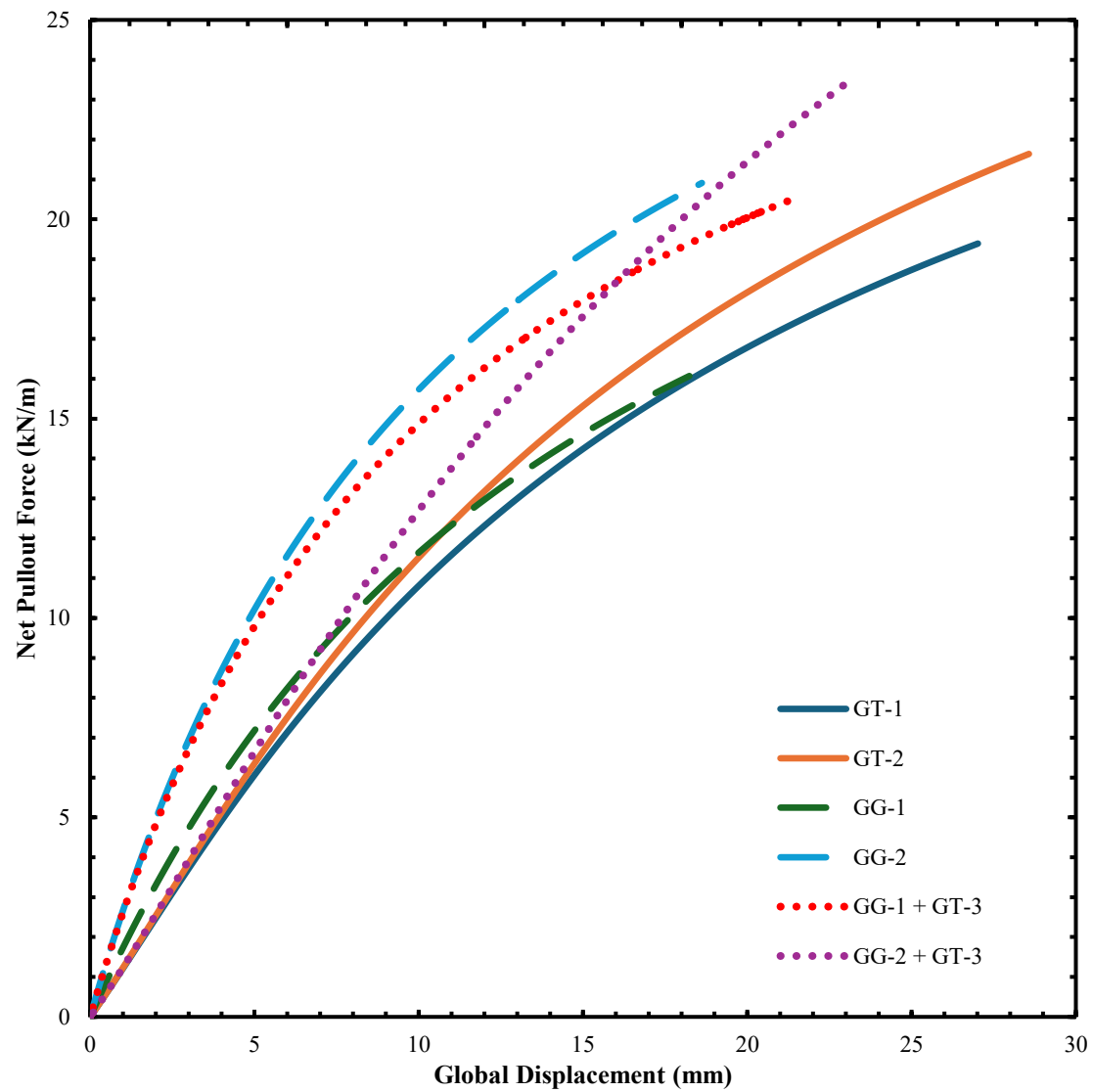


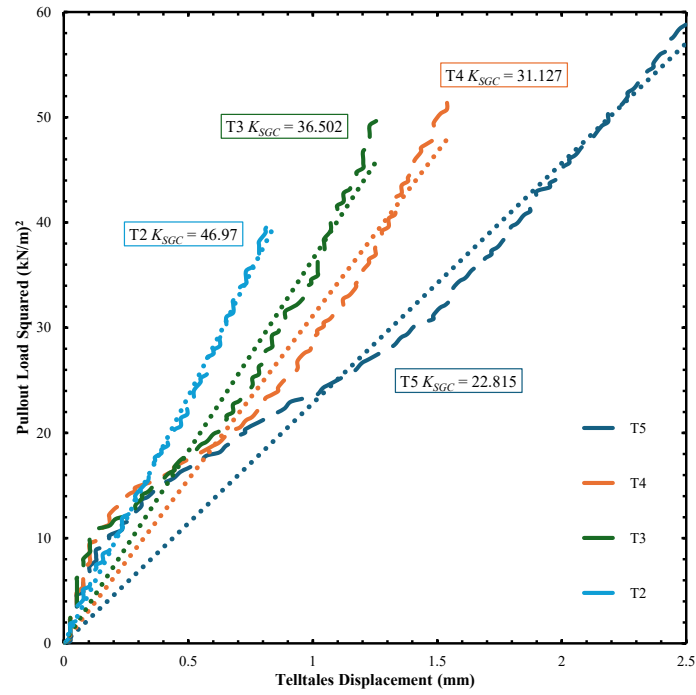
Figure 5.4 Summary of Net Pullout Load of All Geosynthetics Used.

5.2 K_{SGC} Interpretation and Results

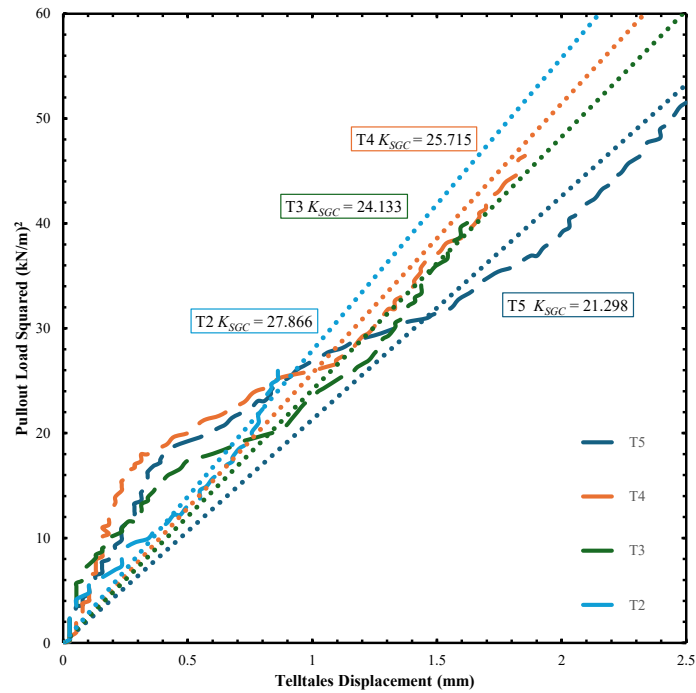
5.2.1 *Method 1*: Direct Estimation from Unit Tension Squared versus Telltale Displacement

The plot between the pullout load squared and the telltale displacement was obtained by taking the average of the three frontal telltales (T3 to T5), with the analysis was limited to the first 2.5 mm of displacement to capture the behavior under small-strain conditions (Zornberg et al., 2017). The K_{SGC} values for GT-1 and GT-2 ranged from 22.82 to 36.50 (kN/m)²/mm and 21.30 to 25.72 (kN/m)²/mm, respectively. The average K_{SGC} values were 30.2 and 23.7 (kN/m)²/mm, respectively (see Figure 5.5). Since the two products have similar specifications, the K_{SGC} values were similar. GG-1 had an K_{SGC} value ranging from 37.61 to 60.14 (kN/m)²/mm with an average of 51.6 (kN/m)²/mm. GG-2 has a K_{SGC} value range of 72.05 to 114.8 (kN/m)²/mm with a mean value of 89.7 (kN/m)²/mm (see Figure 5.6). The K_{SGC} is expected to be higher because GG-2 has higher stiffness. Additionally, the aperture size and the number of transverse members differ between GG-2 and GG-1, which can affect the transfer of shear stress.

The combination of the fabric with the geogrid had an effect in the value of the K_{SGC} . With the introduction of GT-3, the value of K_{SGC} for both geogrids, despite the differences in stiffness, gave similar range of results which were averaged at 61.2 and 61.8 (kN/m)²/mm for GG-1 and GG-2 with the GT-3, respectively. The range of K_{SGC} values for GG-1 with GT-3 is 51.45 to 68.40 (kN/m)²/mm while the range for GG-2 with GT-3 is 53.15 to 72.02 (kN/m)²/mm (see Figure 5.7). It can be noted that the interaction of the materials changed with the introduction of the nonwoven fabric. On the upper part (aggregate), it interacts with the geogrid and a portion of the fabric in between the apertures. While on the bottom part (clay), it is only interacting with the fabric. Figure 5.8 summarizes all the K_{SGC} determined for all products.

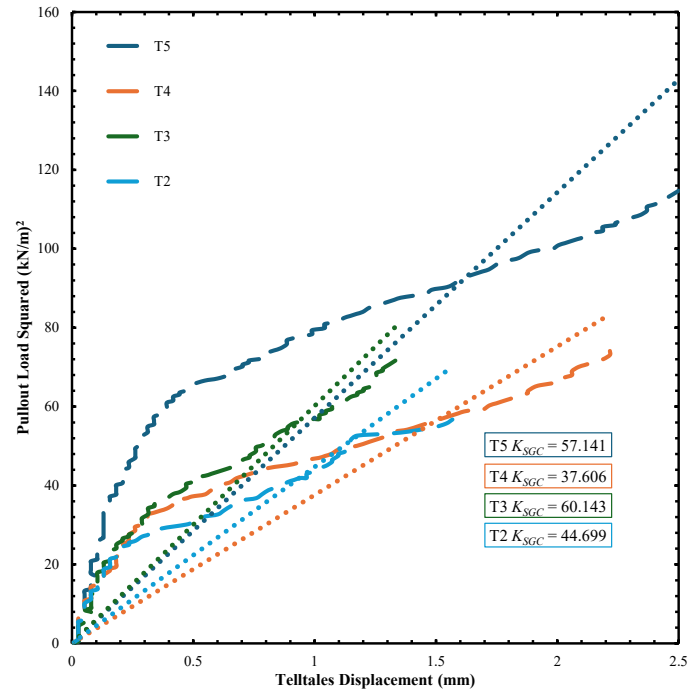


(a)

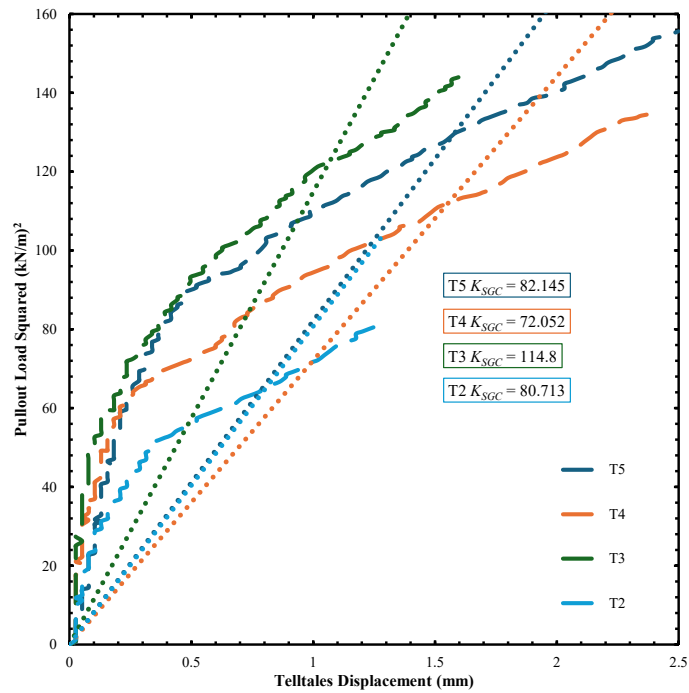


(b)

Figure 5.5 Plot of Unit Tension Squared vs Telltale Displacement for geotextiles: (a) GT-1, (b) GT-2.

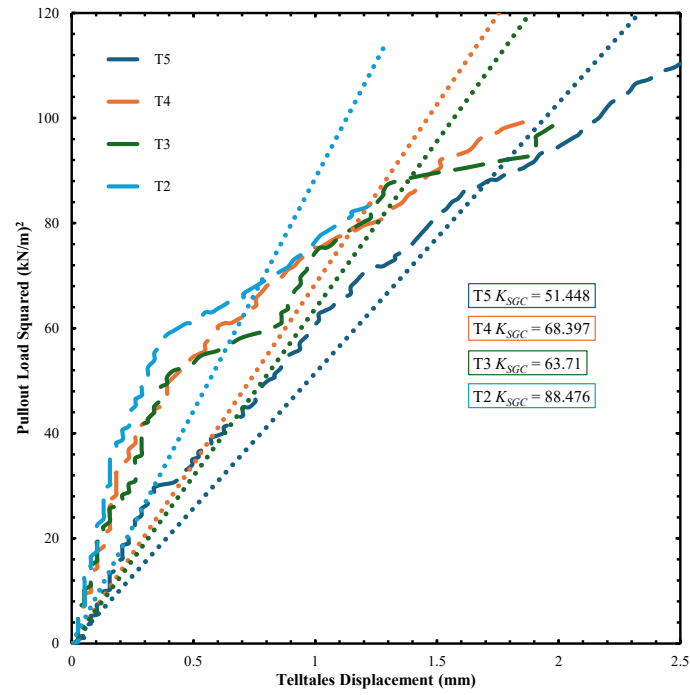


(a)

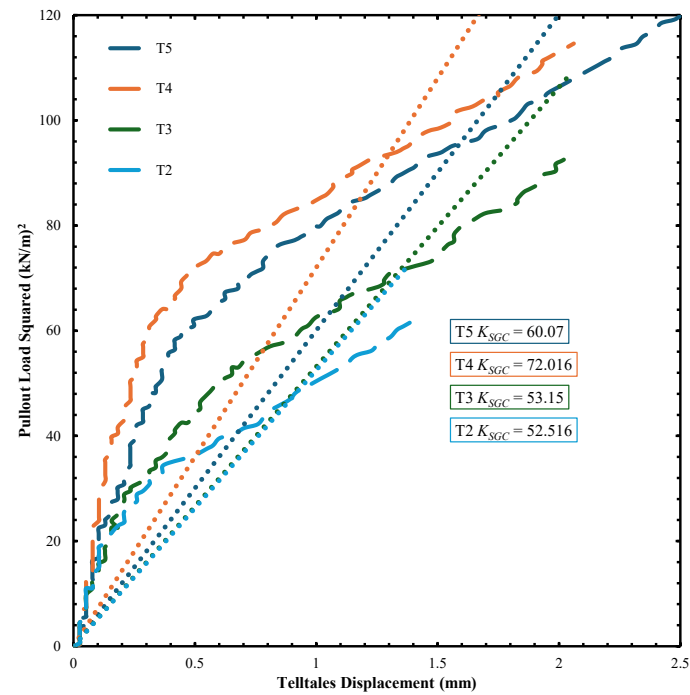


(b)

Figure 5.6 Plot of Unit Tension Squared vs Telltale Displacement for the geogrids: (a) GG-1, (b) GG-2.



(a)



(b)

Figure 5.7 Plot of Unit Tension Squared vs. Telltale Displacement for geogrids with nonwoven geotextiles: (a) GG-1 with GT-3, (b) GG-2 with GT-3.

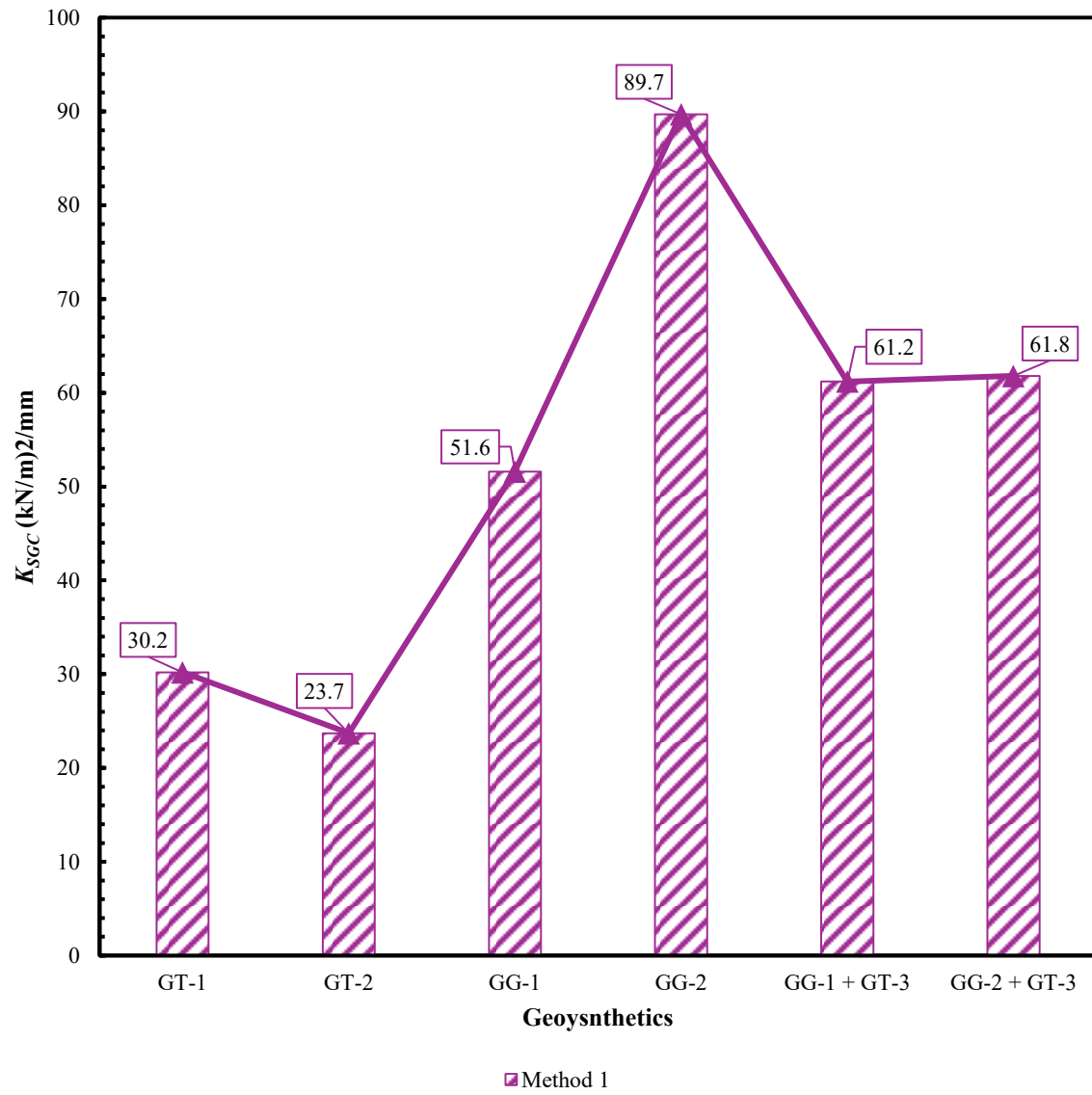


Figure 5.8 K_{SGC} values obtained from all geosynthetics using *Method 1*.

5.2.2 Method 2: Estimation from Parabolic Regression of Telltale Displacement Data

The constitutive parameters, τ_y and J_c , were estimated from the parabolic regression of the telltale displacement data. It should be noted that only displacements below 2.5 mm were considered in the analysis or until all the telltales were mobilized. This is consistent with the procedure by Zornberg et al. (2018). The percentage of pullout loads below this limit were analyzed. Also, the last mobilization of the telltale (T1) was included if it is still below the range of 2.5 mm. The regression analysis parameters are summarized in Table 5.1.

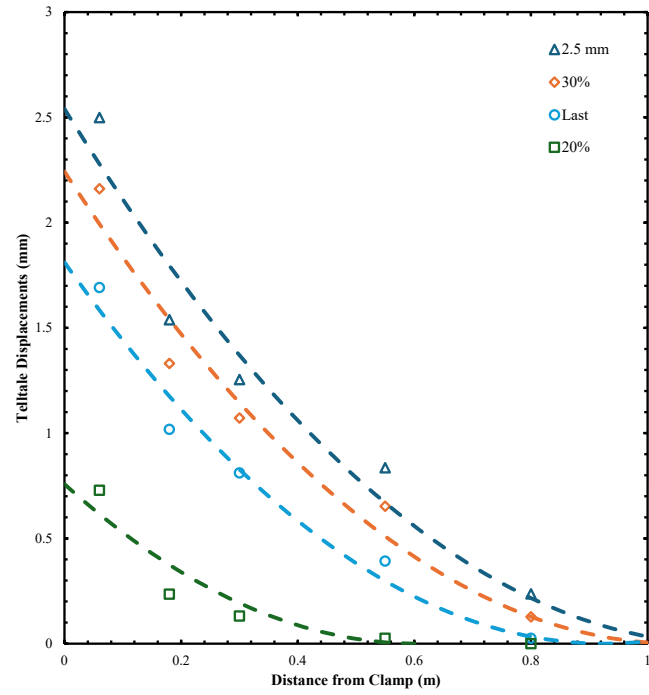
The two geotextiles have a similar range of parameters leading up to the K_{SGC} . GT-1 only utilized 20% of the ultimate pullout load because beyond that all telltales were mobilized. GT-2 utilized the 20% and 30% of the ultimate load to determine the parameters (see Figure 5.9). The regression resulted in a value of 3.2 and 3.1 kPa for its yield shear stress (τ_y) and a confined geosynthetic stiffness (J_c) range of 1.548 and 1.516 kN/m for GT-1 and GT-2, respectively. The K_{SGC} value of GT-1 and GT-2 were 19.8 and 19.0 (kN/m)²/mm, respectively. Similar results are expected since the two fabrics have similar specifications.

The geogrids gave similar results with *Method 1* with GG-2 having a larger value than GG-1. In the regression analysis, the parameters for GG-1 were derived using 40% of its ultimate pullout load, whereas for GG-2, the parameters were based on 50% of its ultimate pullout load (see Figure 5.10). The constitutive parameters verified that GG-2 is stiffer than GG-1 having yield shear stress of 5.6 kPa compared to 4.5 kPa. The confined geosynthetic stiffness (J_c) of GG-2, measured at 3.268 kN/m, is higher than that of GG-1, which is 2.790 kN/m. The K_{SGC} value of GG-1 and GG-2 were 49.7 and 72.9 0 (kN/m)²/mm, respectively.

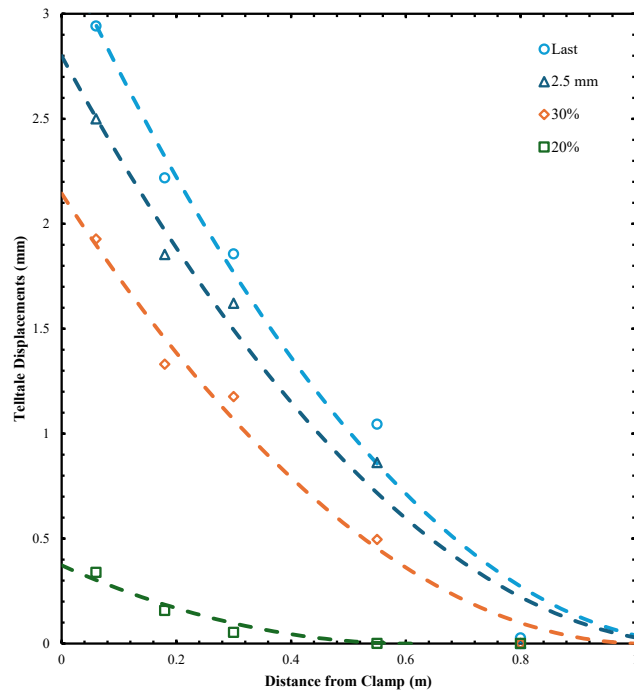
The geogrids with the nonwoven geotextile gave a similar range of parameters. The 40% of the ultimate pullout load was used to determine the parameters of the GG-1 with GT-3 (see Figure 5.11). While the last mobilization of the telltales and 50% of the ultimate pullout load were used and averaged for GG-2 with GT-3. With the introduction of the fabric, both the shear stress and confined stiffness fell within a similar range, resulting in comparable values of K_{SGC} (around 52 (kN/m)²/mm) despite the differing stiffness of the geogrids (GG-2 being stiffer than GG-1). This suggests that the interaction was primarily governed by the frictional mechanism of the nonwoven sheet rather than the geogrid itself, as indicated by the yield shear stress and confined geosynthetic stiffness values ranging from 4.0 to 4.3 kPa and 3.049 to 3.310 kN/m, respectively.

Table 5.1 Summary of Constitutive Parameters for Different Materials in an Aggregate-Clay setup at 21 kPa.

Parameter	Unit	Geogrid and Geotextile Combination					
		Geotextiles		Geogrids		Geogrid and Geotextile	
		GT-1	GT-2	GG-1	GG-1	GG-1 + GT-3	GG-2 + GT-3
τ_y	kPa	3.2	3.1	4.5	5.6	4.0	4.3
J_c	kN/m	1.548	1.516	2.790	3.268	3.310	3.049
K_{SGC}	(kN/m) ² /mm	19.8	19.0	49.7	72.9	52.9	52.0
S	mm ²	0.04	0.04	0.70	0.05	0.06	0.15

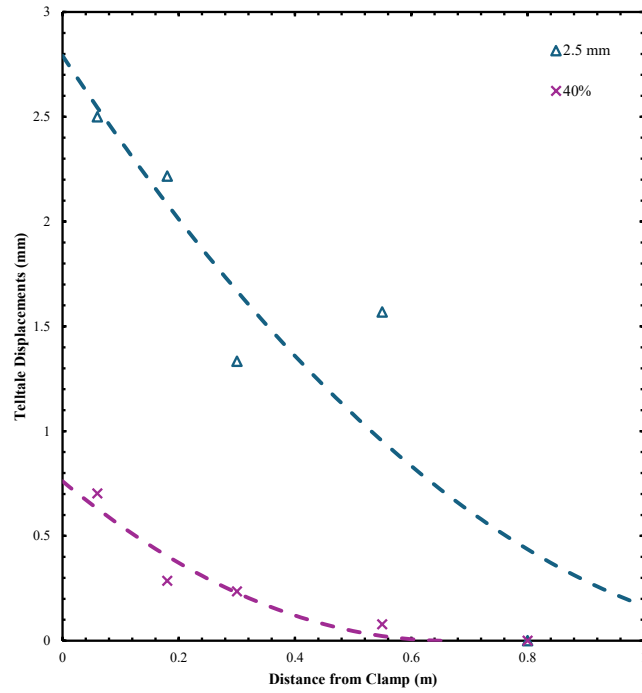


(a)

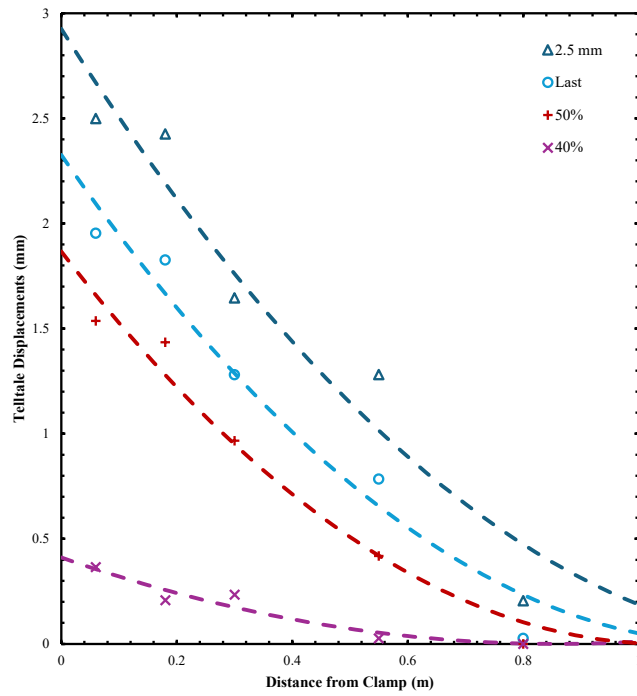


(b)

Figure 5.9 Parabolic Regression of Telltale Displacement for geotextiles: (a) GT-1, (b) GT-2.

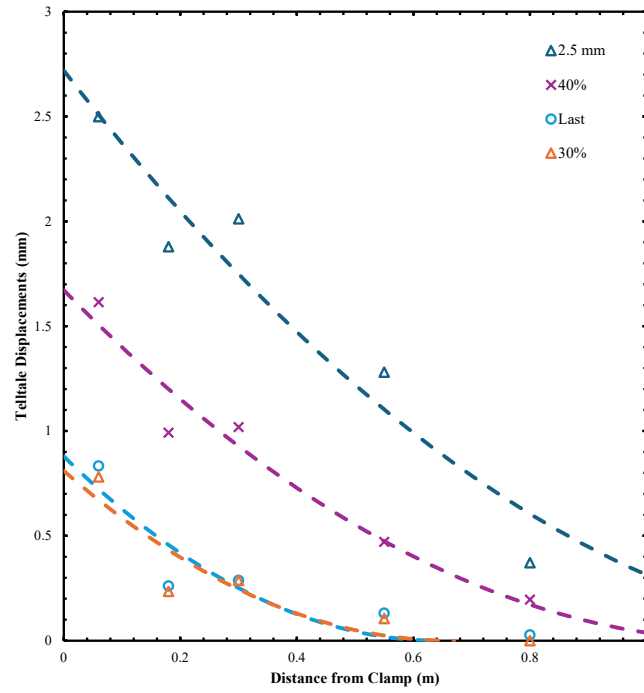


(a)

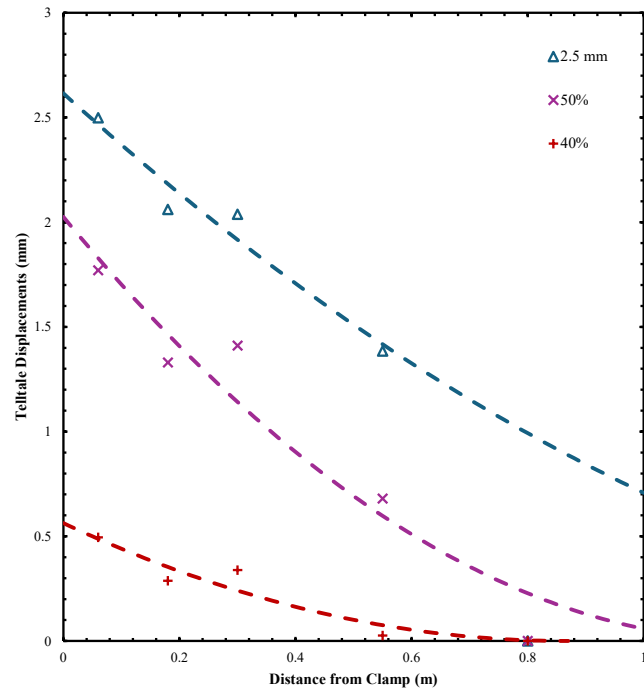


(b)

Figure 5.10 Parabolic Regression of Telltale Displacement for geogrids: (a) GG-1, (b) GG-2.



(a)



(b)

Figure 5.11 Parabolic Regression of Telltale Displacement for geocomposites: (a) GG-1 with GT-3, (b) GG-2 with GT-3.

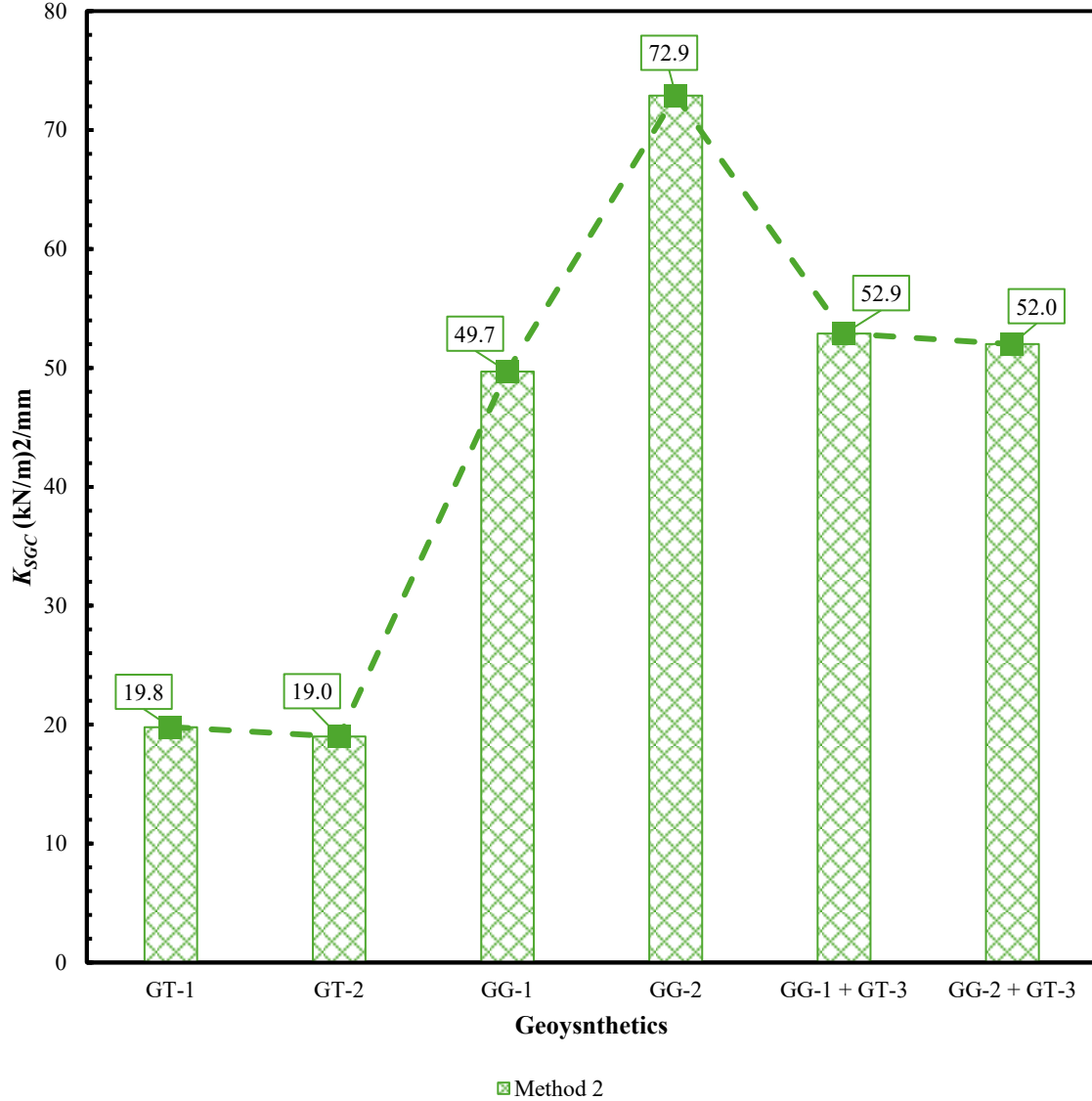


Figure 5.12 K_{SGC} values obtained for all geosynthetics using *Method 2*.

5.2.3 Comparison of K_{SGC} Value for Geotextiles and Geogrids for *Method 1* and *Method 2*

A summary of the K_{SGC} value for both methods from Section 5.2.1 and 5.2.2 are presented in Table 5.2. It was observed that consistency is found with respect to the direct estimation of K_{SGC} using *Method 2* compared to *Method 1* as summarized in Section 5.2.1. The values may not necessarily be in range but the trend in comparing the values was similar for both methods. A comparison of

K_{SGC} from both methods is provided in Figure 5.13. Referring to Section 4.4, the K_{SGC} threshold value of 10 (kN/m)²/mm was considered to evaluate the geogrids for base stabilization. All materials passed this criterion. This serves only as a comparison to the threshold value based on the study of Roodi et al. (2017), since the study did not assess the field performance of the geosynthetics. This is a crucial factor in developing a reliable criterion.

Method 1 did not show linear behavior as expected by the K_{SGC} model. In Zornberg et al. (2017), a single, uniformly graded backfill was used. However, this study used two different backfill materials, which may have caused the nonlinear results. On the other hand, Method 2 limits the analysis to displacements under 2.5 mm or until all the telltales were mobilized, since the interface shear stress no longer changes with additional displacement beyond the yield point. This explains why the parabolic curves vary between tests.

Table 5.2 Summary of K_{SGC} for both methods.

Materials	K_{SGC} (kN/m) ² /mm		
	<i>Method 1</i>	<i>Method 2</i>	Average
	Direct	Parabolic	
	estimation from T^2 vs u data	regression of telltale data	
GT-1	30.2	19.8	25.0
GT-2	23.7	19.0	21.4
GG-1	51.6	49.7	50.7
GG-2	89.7	72.9	81.3
GG-1 + GT-3	61.2	52.9	57.1
GG-2 + GT-3	61.8	52.0	56.9

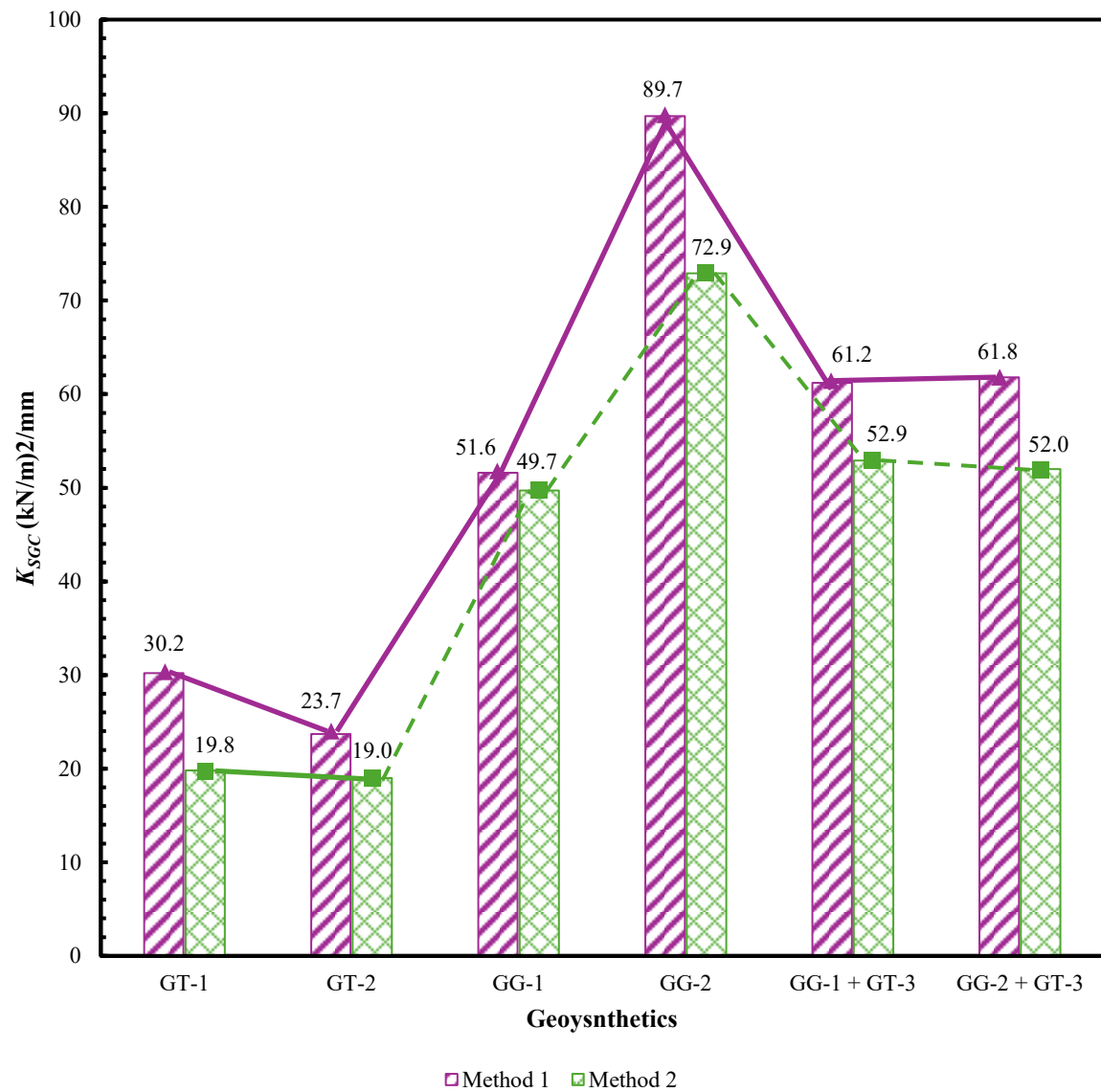


Figure 5.13 Summarized K_{SGC} values obtained for all geosynthetics using both methods.

CHAPTER 6

CONCLUSION

The analysis of K_{SGC} values across different test conditions reveals the significant influence of backfill material type, confining pressure, and geosynthetic product selection on the shear interaction behavior of soil-geosynthetic systems.

Increasing the confining pressure generally leads to higher K_{SGC} values, indicating enhanced interface stiffness. Higher confining stress promotes greater interlocking and frictional resistance at the soil-geosynthetic interface. However, the rate of increase in K_{SGC} may diminish beyond a certain threshold, suggesting a non-linear relationship due to stress redistribution and potential material failure.

The type and gradation of backfill significantly affects the behavior of soil-geosynthetic interface. The aggregates gave a small K_{SGC} value while the aggregate-clay setup gave the highest K_{SGC} value. Aggregates usually perform better than clay because they have higher shear resistance. However, in this case, the uniform contact and cohesion between the nonwoven part of the geocomposite and the clay increased the shear resistance, leading to a higher pullout force.

Geosynthetics show diverse performance depending on their type. Geocomposites gave an average K_{SGC} value of 52.1 (kN/m)²/mm with applied confining stress of 21 kPa. Geotextiles gave the smallest K_{SGC} value with an average value of 21.4 to 25 (kN/m)²/mm. Geogrids generally exhibit higher K_{SGC} than geotextiles because their open apertures allow for better mechanical interlock with aggregate materials. The average range of K_{SGC} values for geogrids in this study were around

50.7 to 81.3 (kN/m)²/mm. Geogrids underlain with nonwoven geotextile provided intermediate results, leveraging frictional and filtration characteristics depending on their structural design. The K_{SGC} values averaged from 56.9 to 57.1 (kN/m)²/mm. The aperture size and number of transverse members of the geogrid are primary factors influencing pullout and shear resistance. With the given setup of aggregate-clay in this study, the woven geotextiles have lower K_{SGC} values than incorporating geogrids. According to Roodi et al. (2017) who conducted an extensive field performance test, geogrids performed better due its gravel-geosynthetic interaction. This suggests that incorporating geogrids is beneficial with our local base materials.

The model of the K_{SGC} was intended only for small displacements which is below 2.5 mm and were derived from analyzing one material. However, this study differed where the pullout tests were performed using two backfill materials. *Method 1* results did not represent linearity as expected for the K_{SGC} model. Also, *Method 2* was limited by a few parabolic regression curves. The parabolic curves were generated using different percentages of the ultimate pullout load. In addition to limiting the analysis to a specific displacement range, only curves in which all telltales had mobilized were considered. This approach was taken because, beyond the yield point, the interface shear stress becomes constant and does not change with further displacement according to the model.

The K_{SGC} parameter is an index parameter that describes the mechanical behavior of soil–geosynthetic systems. It characterizes the composite stiffness of the soil-geosynthetic system under small displacements.

Overall, the results of this study show that K_{SGC} is affected by backfill type, confining pressure, and the geosynthetic product used. Higher confining pressures generally increase K_{SGC} values due to improved interlocking and friction, while different material combinations exhibit distinct interaction behaviors. Thus, while K_{SGC} is not yet a direct measure of field performance, it provides a meaningful benchmark for selecting geosynthetics that enhance the soil-geosynthetic composite stiffness and contribute to better road performance.

CHAPTER 7

RECOMMENDATIONS

While this study has demonstrated the significant influence of backfill material, confining pressure, and geosynthetic type on the shear behavior of soil–geosynthetic systems, several areas warrant further investigation. Future recommendations should be made on the following:

(1) Improved Apparatus and Equipment:

Future studies should improve the pullout apparatus to minimize the error in determining the parameters in the experimentation such as the clamp, the strain potentiometers and the data loggers.

(2) Derive an Equation representing Two-Layered Backfill System:

The work of Zornberg et al. (2017) represents only one material to determine the stiffness parameter. Thus, a new model representing the shear interface of a two-layered backfill system must be generated, and necessary laboratory tests will validate this model. It is therefore recommended from the output of this study that a specific equation be derived to be tailored to the type of backfill setup this research had.

(3) Cyclic Loading Conditions:

Most existing studies, including this one, are based on static or monotonic loading. Future research should investigate the K_{SGC} behavior under cyclic loading to better replicate real-world loading conditions (e.g., traffic, seismic activity, or seasonal ground movement). Cyclic loading can be introduced by using an actuator to apply cyclic horizontal loads to the pullout box and examine how the K_{SGC} parameter is affected.

(4) Field-Scale Validation:

Laboratory pullout tests should be supplemented with field-scale or full-scale testing to validate the applicability of K_{SGC} values derived from controlled tests. Field studies can capture the variability and complexity of in-situ soil conditions, installation effects, and construction quality. Field monitoring may be used to determine movement in the study area such as simple survey of change in geometry (differential movement) or a sensor may also be imbedded to determine movement. A threshold value of K_{SGC} may then be generated by comparing field monitoring results with the laboratory results (Roodi et al., 2017).

(5) Numerical Modeling and Predictive Frameworks:

Development of numerical models (e.g., finite element or discrete element methods) calibrated with experimental data can enhance the predictive understanding of K_{SGC} . Such models could simulate different loading scenarios, soil types, and reinforcement configurations efficiently. Nevertheless, existing modeling platforms currently lack provisions for directly incorporating K_{SGC} as an input parameter or for representing its role within soil–geosynthetic interaction analyses.

REFERENCES

- Alfaro, M. C., Hayashi, S., Miura, N., & Watanabe, K. (1995). Pullout Interaction Mechanism of Geogrid Strip Reinforcement. *Geosynthetics International*, 2(4), 679–698.
<https://doi.org/10.1680/gein.2.0030>
- Alfaro, M., Miura, N., & Bergado, D. (1995a). Soil-Geogrid Reinforcement Interaction by Pullout and Direct Shear Tests. *Geotechnical Testing Journal*, 18(2), 157–167.
<https://doi.org/10.1520/GTJ10319J>
- Alfaro, M., Miura, N., & Bergado, D. (1995b). Soil-Geogrid Reinforcement Interaction by Pullout and Direct Shear Tests. *Geotechnical Testing Journal*, 18(2), 157–167.
<https://doi.org/10.1520/GTJ10319J>
- Al-Qadi, I. L., Dessouky, S. H., Kwon, J., & Tutumluer, E. (2008). Geogrid in Flexible Pavements. *Transportation Research Record: Journal of the Transportation Research Board*, 2045(1), 102–109. <https://doi.org/10.3141/2045-12>
- Bender, D. A., & Barenberg, E. J. (1978). Design and Behavior of Soil-Fabric-Aggregate Systems. *Transportation Research Record*, 671, 64–75.
- Bloise, N., & Ucciardo, S. (2000). On site test of reinforced freeway with high-strength geosynthetics. *EUROGEO 2000: Proceedings of the 2nd European Geosynthetics Conference. Volume 1: Mercer Lecture, Keynote Lectures, Geotechnical Applications*, 369–371.

- Eissa, A. (2023). Stabilizing Soft Clay Soil using Nano-Modified Cementitious Binders, Basalt Fiber Pellets, and a Novel Geosynthetic Composite [Doctoral dissertation, University of Manitoba]. MSpace. <http://hdl.handle.net/1993/37217>
- Elias, V., Christophert, B., & Berg, R. (2001). Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines. 132042.
- Farrag, K., Acar, Y. B., & Juran, I. (1993). Pull-Out Resistance of Geogrid Reinforcements. *Geotextiles and Geomembranes*, 12(2), 133–159. [https://doi.org/10.1016/0266-1144\(93\)90003-7](https://doi.org/10.1016/0266-1144(93)90003-7)
- Ghionna, V. N., Moraci N., & Rimoldi P. (2001). Experimental Evaluation of the Factors Affecting Pull-out Test Results on Geogrids. *Landmarks in Earth Reinforcement*, VOL 1, 31–36. <http://hdl.handle.net/11583/1412340>
- Han, J., & Thakur, J. K. (2015). Sustainable roadway construction using recycled aggregates with geosynthetics. *Sustainable Cities and Society*, 14(1), 342–350. <https://doi.org/10.1016/j.scs.2013.11.011>
- Hanumasagar, S. S., Roodi, G. H., Zornberg, J. G., & Grubb, D. G. (2014). Pullout characterization of geogrids embedded in dredged material and steel slag fines (DM-SSF) blends. 10th International Conference on Geosynthetics, ICG 2014.
- Hayashi, S., Alfaro, M. C., & Watanabe, K. (1996). Dilatancy effects of granular soil on the pullout resistance of strip reinforcement. *Proc. of the Int. Symp. on Earth Reinforcement*, Vol. 1, 39–44.

- Hayashi, S., Ochiai, H., & Son, S. (1985). Mechanism of pullout resistance of polymer grids in soils. *Tsuchi-to-Kiso, JSSMFE*, 21–26.
- Holtz, R. D., Christopher, B. R., & Berg, R. R. (1998). *Geosynthetic Design and Construction Guidelines Report No, FHWA-NHI-07-092, U.S.* (p. 612).
- Jewell, R. A. (1980). Some effects of reinforcement on the mechanical behaviour of soils. Thesis presented to Cambridge University, Cambridge, UK (1980) in partial fulfilment of the requirements for the Degree of Doctor of Philosophy.
- Jie, H., & Yan, J. (2013). Use of geosynthetics for performance enhancement of earth structures in cold regions. *Sciences in Cold and Arid Regions*, 5(5), 517.
<https://doi.org/10.3724/SP.J.1226.2013.00517>
- Juran, I., Knochenmus, G., Acar, Y. B., & Arman, A. (1988). Pull-Out Response of Geotextiles and Geogrids (Synthesis of Available Experimental Data). *Geotechnical Special Publication*, 18, 92–111.
- Koerner, R. M. (1986). Direct shear/pull-out tests on geogrids. In Report No. 1. Department of Civil Engineering, Drexel University USA.
- Kondo, J. R., Roodi, G. H., & Zornberg, J. G. (2016). Evaluation of the Interaction between Dredged Materials and Geosynthetic Reinforcements. *EVALUATION OF THE INTERACTION BETWEEN DREDGED MATERIALS AND GEOSYNTHETIC REINFORCEMENTS*, 2, 1187–1202.
<https://www.eiseverywhere.com/eselectv2/frontend/index/122547>

- Mendes, M. J. A., Palmeira, E. M., & Matheus, E. (2007). Some factors affecting the in-soil load–strain behaviour of virgin and damaged nonwoven geotextiles. *Geosynthetics International*, 14(1), 39–50. <https://doi.org/10.1680/gein.2007.14.1.39>
- Morsy, A. M., Zornberg, J. G., Leshchinsky, D., & Han, J. (2019). Soil–Reinforcement Interaction: Effect of Reinforcement Spacing and Normal Stress. *Journal of Geotechnical and Geoenvironmental Engineering*, 145(12), 1–17.
[https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0002180](https://doi.org/10.1061/(ASCE)GT.1943-5606.0002180)
- Mosallanezhad, M., Taghavi, S. H. S., Hataf, N., & Alfaro, M. C. (2016). Experimental and numerical studies of the performance of the new reinforcement system under pull-out conditions. *Geotextiles and Geomembranes*, 44(1), 70–80.
<https://doi.org/10.1016/j.geotexmem.2015.07.006>
- Palmeria, E. M., & Milligan, G. W. E. (1989). Scale and other factors affecting the results of pull-out tests of grids buried in sand. *Géotechnique*, 39(3), 511–542.
<https://doi.org/10.1680/geot.1989.39.3.511>
- Perkins, S. W. (2002). Evaluation of Geosynthetic Reinforced Flexible Pavement Systems Using Two Pavement Test Facilities Prepared by. Montana Dept. of Transportation, Helena, MT, FHWA/MT-02-008/20040, 120.
- Perkins, S. W., & Ismeik, M. (1997). A Synthesis and Evaluation of Geosynthetic-Reinforced Base Layers in Flexible Pavements- Part II. *Geosynthetics International*, 4(6), 605–621.
<https://doi.org/10.1680/gein.4.0107>

- Roodi, G. H., & Zornberg, J. G. (2017). Stiffness of Soil-Geosynthetic Composite under Small Displacements. II: Experimental Evaluation. *Journal of Geotechnical and Geoenvironmental Engineering*, 143(10), 1–17. [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0001769](https://doi.org/10.1061/(ASCE)GT.1943-5606.0001769)
- Roodi, G. H., Zornberg, J. G., Aboelwafa, M. M., Phillips, J. R., Zheng, L., & Martinez, J. (2017). Soil-Geosynthetic Interaction Test to Develop Specifications for Geosynthetic-Stabilized Roadways (FHWA 5-4829-03-1) (Issue 5). <http://library.ctr.utexas.edu/ctr-publications/5-4829-03-1.pdf>
- Schlosser, F., & Long, N.-T. (1974). Recent Results of French Research on Reinforced Earth. *Journal of the Construction Division*, 100(3), 223–237. <https://doi.org/10.1061/JCCEAZ.0000429>
- Van Santvoort, G. P. (1994). *Geotextiles and geomembranes in civil engineering*. CRC Press.
- Wilmers, W. (2002). The revised German regulations for the use of geosynthetics in road construction. *Geosynthetics: State of the Art – Recent Developments*. Proceedings of the Seventh International Conference on Geosynthetics, 7-ICG, 1401–1404. <https://trid.trb.org/View/645782>
- Yang, Z. (1972). *Strength and Deformation Characteristics of Reinforced Sand*. In ProQuest Dissertations and Theses. <https://uml.idm.oclc.org/login?url=https://www.proquest.com/dissertations-theses/strength-deformation-characteristics-reinforced/docview/302549886/se-2?accountid=14569>
- Zornberg, J. G. (2011). Advances in the Use of Geosynthetics in Pavement Design. *Proceedings of the Second National Conference on Geosynthetics*, 1, 3–21.

- Zornberg, J. G. (2017). Functions and Applications of Geosynthetics In Roadways. *Procedia Engineering*, 189(May), 298–306. <https://doi.org/10.1016/j.proeng.2017.05.048>
- Zornberg, J. G., Ferreira, J. A. Z., & Roodi, G. H. (2012). Experimental Results on Soil-Geosynthetic Interaction Stiffness. FHWA/TX-12, 232.
- Zornberg, J. G., Ferreira, J. A. Z., & Roodi, G. H. (2013). Geosynthetic-Reinforced Unbound Base Courses: Quantification of the Reinforcement Benefits (Report 5-4829-01-2). 7.
- Zornberg, J. G., & Gupta, R. (2010). Geosynthetics in pavements: North American contributions. 9th International Conference on Geosynthetics - Geosynthetics: Advanced Solutions for a Challenging World, ICG 2010, 379–400.
- Zornberg, J. G., Roodi, G. H., Ferreira, J., & Gupta, R. (2012). Monitoring Performance of Geosynthetic-Reinforced and Lime-Treated Low-Volume Roads under Traffic Loading and Environmental Conditions. *GeoCongress 2012*, 30(1987), 1310–1319.
<https://doi.org/10.1061/9780784412121.135>
- Zornberg, J. G., Roodi, G. H., & Gupta, R. (2017). Stiffness of Soil–Geosynthetic Composite under Small Displacements: I. Model Development. *Journal of Geotechnical and Geoenvironmental Engineering*, 143(10). [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0001768](https://doi.org/10.1061/(ASCE)GT.1943-5606.0001768)
- Zornberg, J. G., Roodi, G. H., & Gupta, R. (2018). Closure to “Stiffness of Soil–Geosynthetic Composite under Small Displacements: I. Model Development” by Jorge G. Zornberg, Gholam H. Roodi, and Ranjiv Gupta. *Journal of Geotechnical and Geoenvironmental Engineering*, 144(11), 1–3.
[https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0001972](https://doi.org/10.1061/(ASCE)GT.1943-5606.0001972)

Appendix A. Results of Pullout Tests for Titan Environmental Product

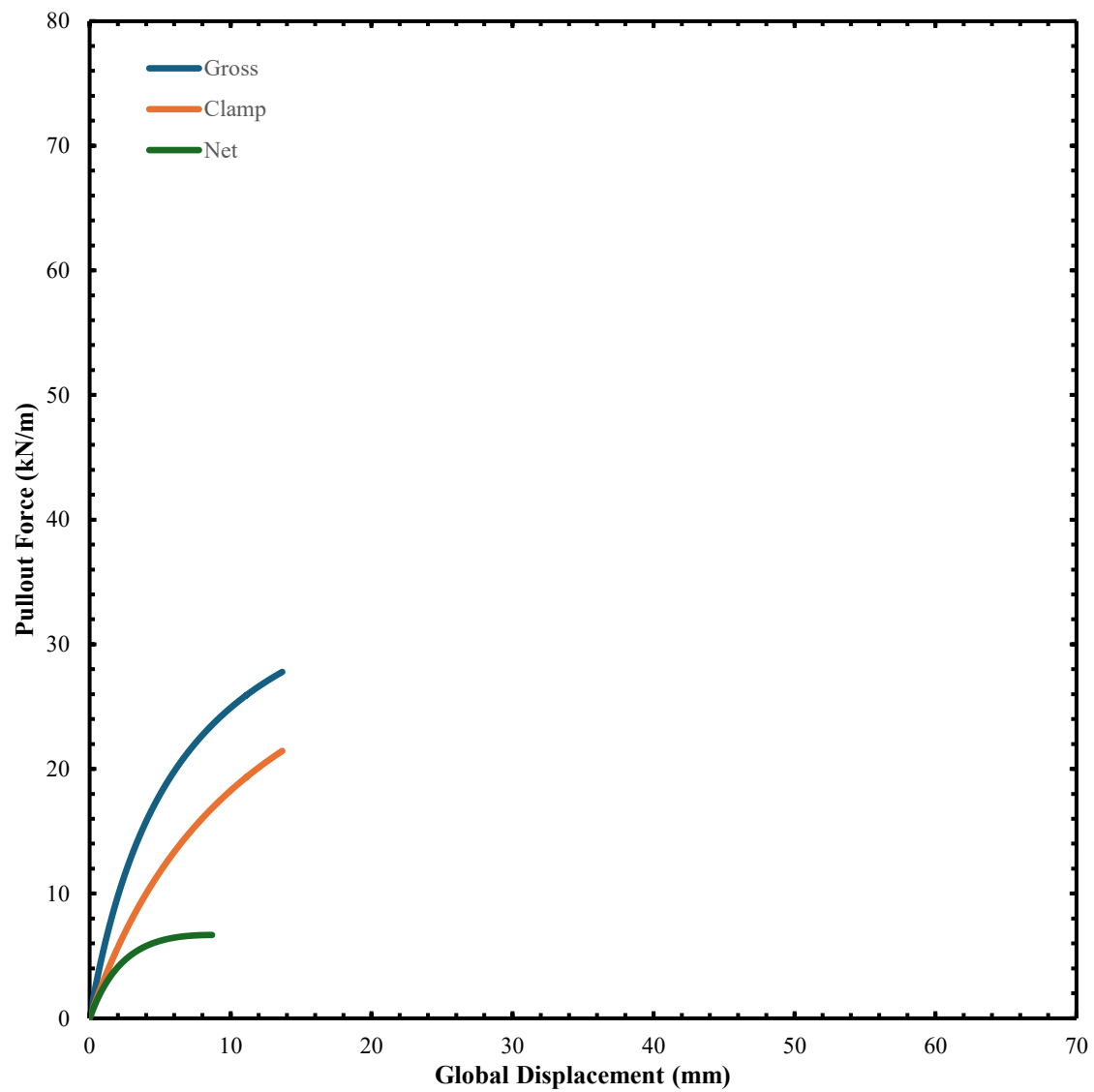


Figure A.1 Pullout Test results of GGC-1 at 7 kPa.

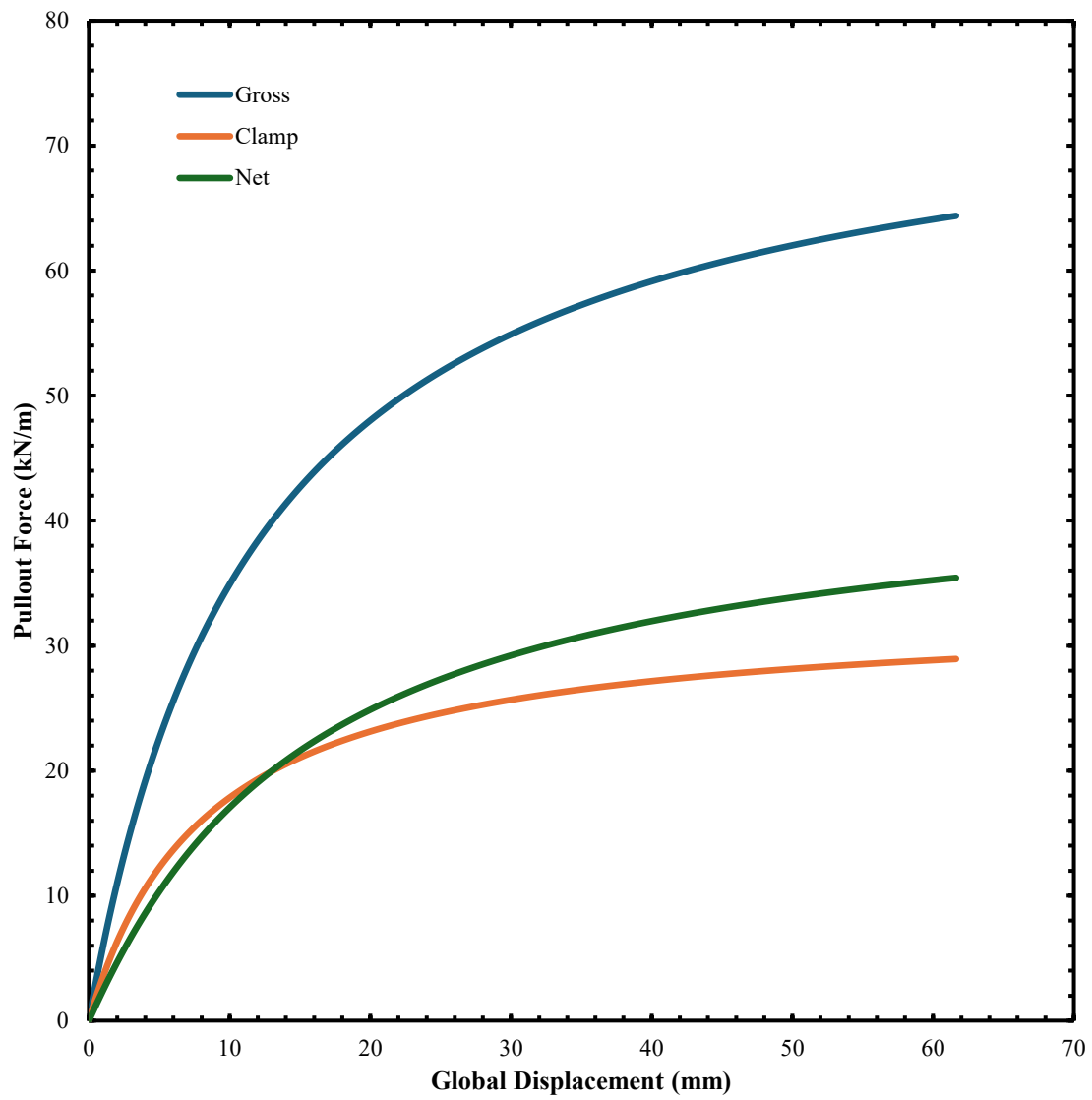


Figure A.2 Pullout Test results of GGC-1 at 21 kPa.

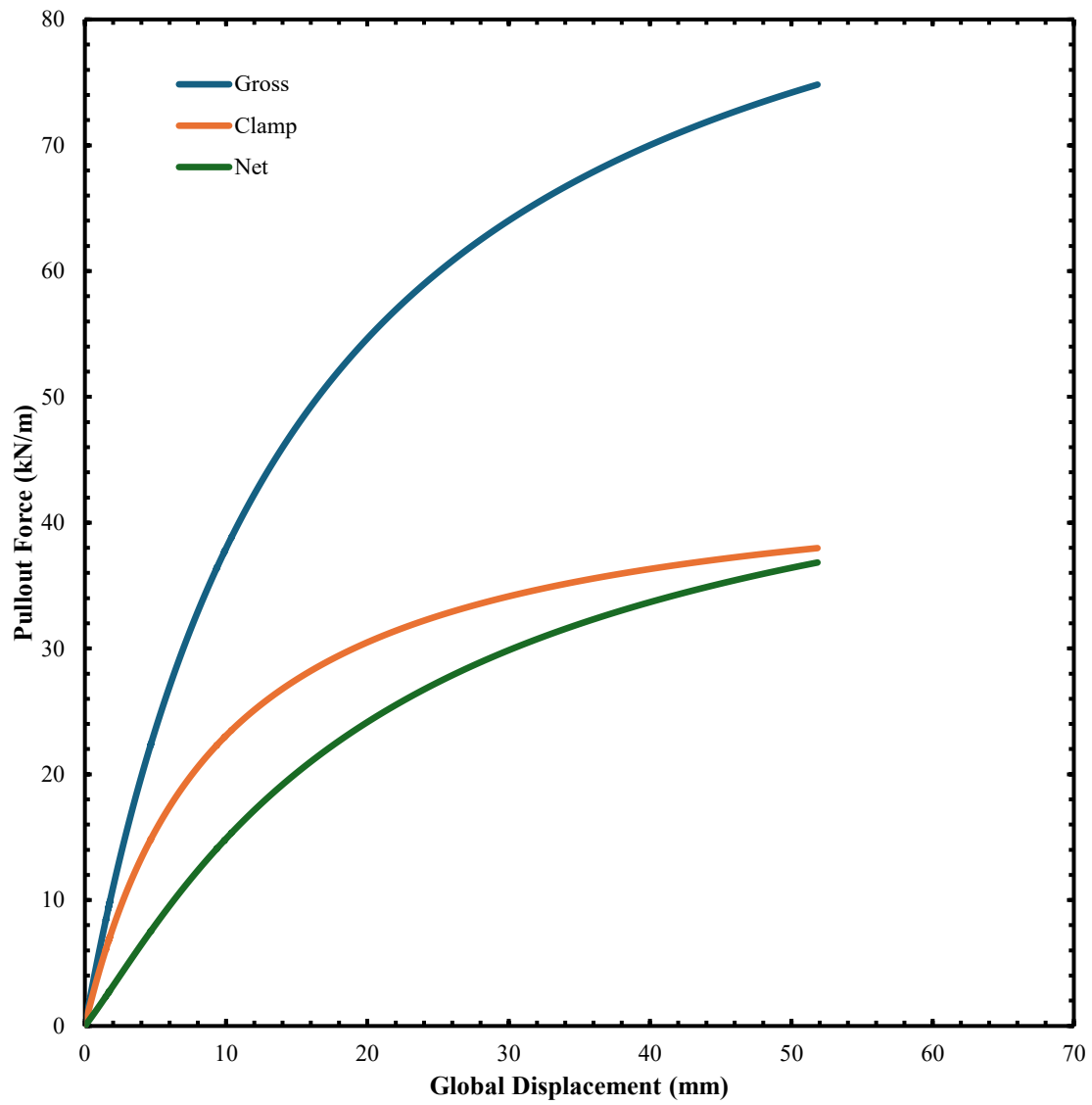


Figure A.3 Pullout Test results of GGC-1 at 35 kPa.

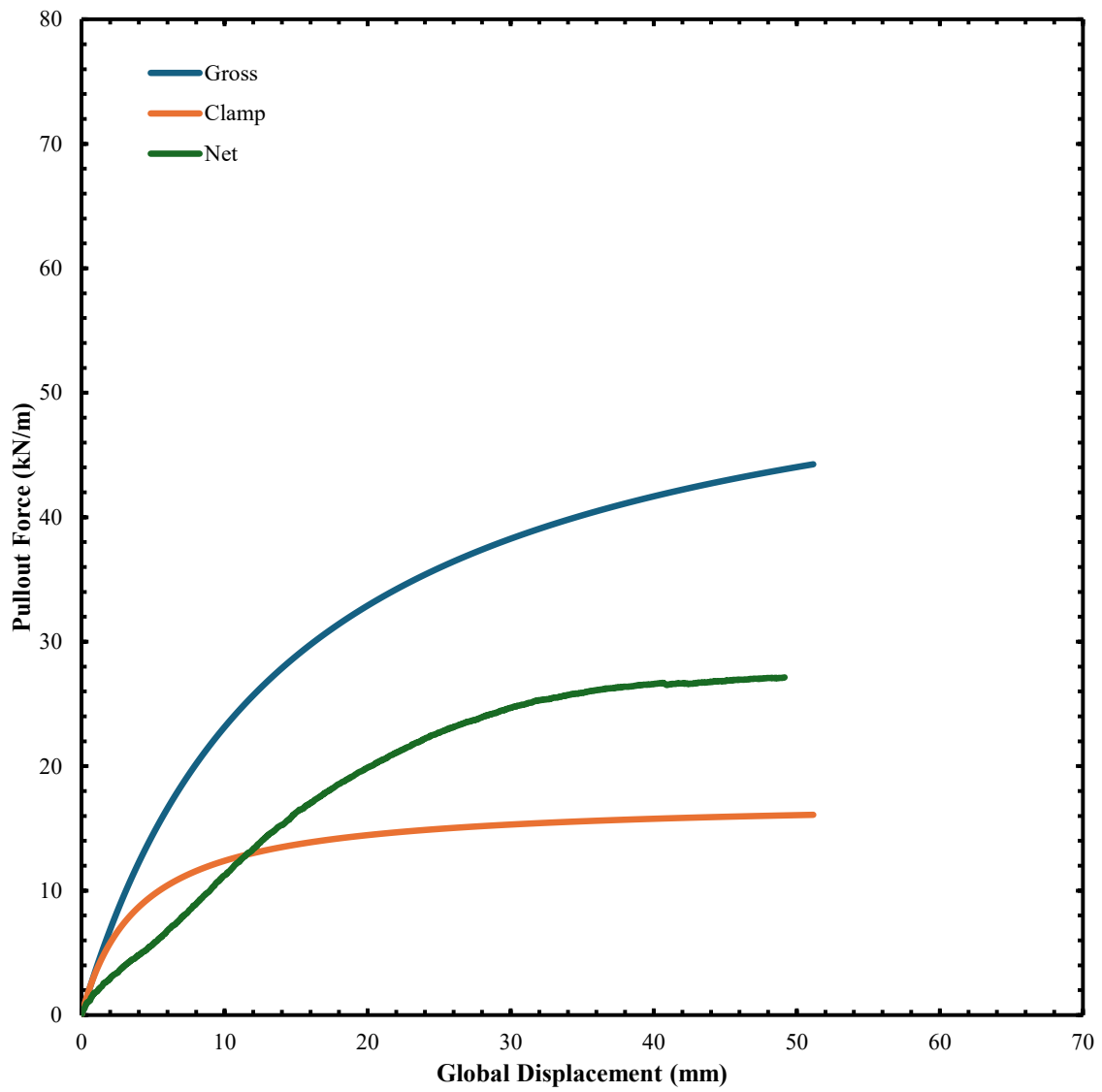


Figure A.4 Pullout Test results of GGC-1 at 21 kPa with pure clay interface.

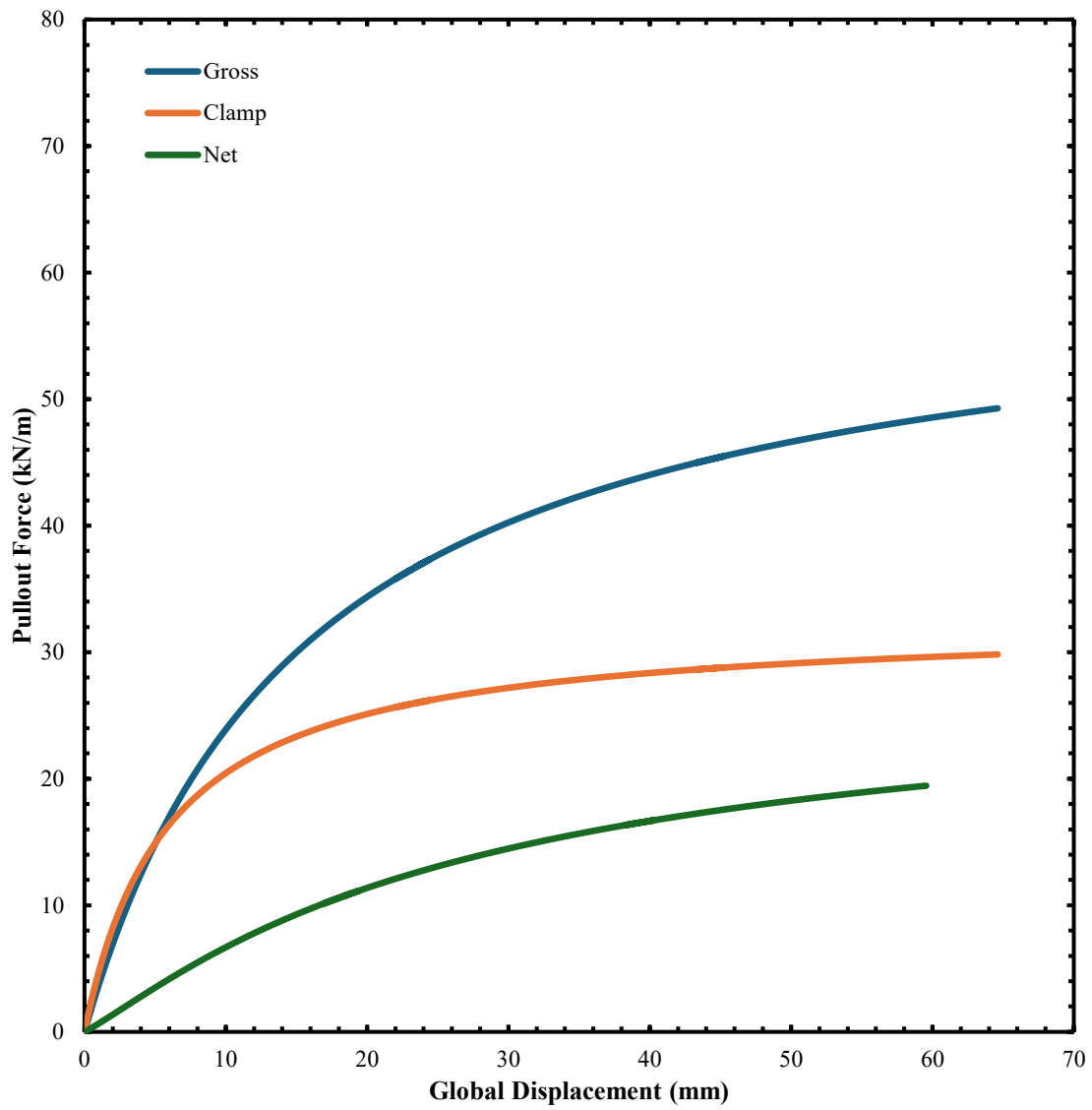


Figure A.5 Pullout Test results of GGC-1 at 21 kPa with pure aggregates interface.

Appendix B. Results of Pullout Tests for City of Winnipeg Product

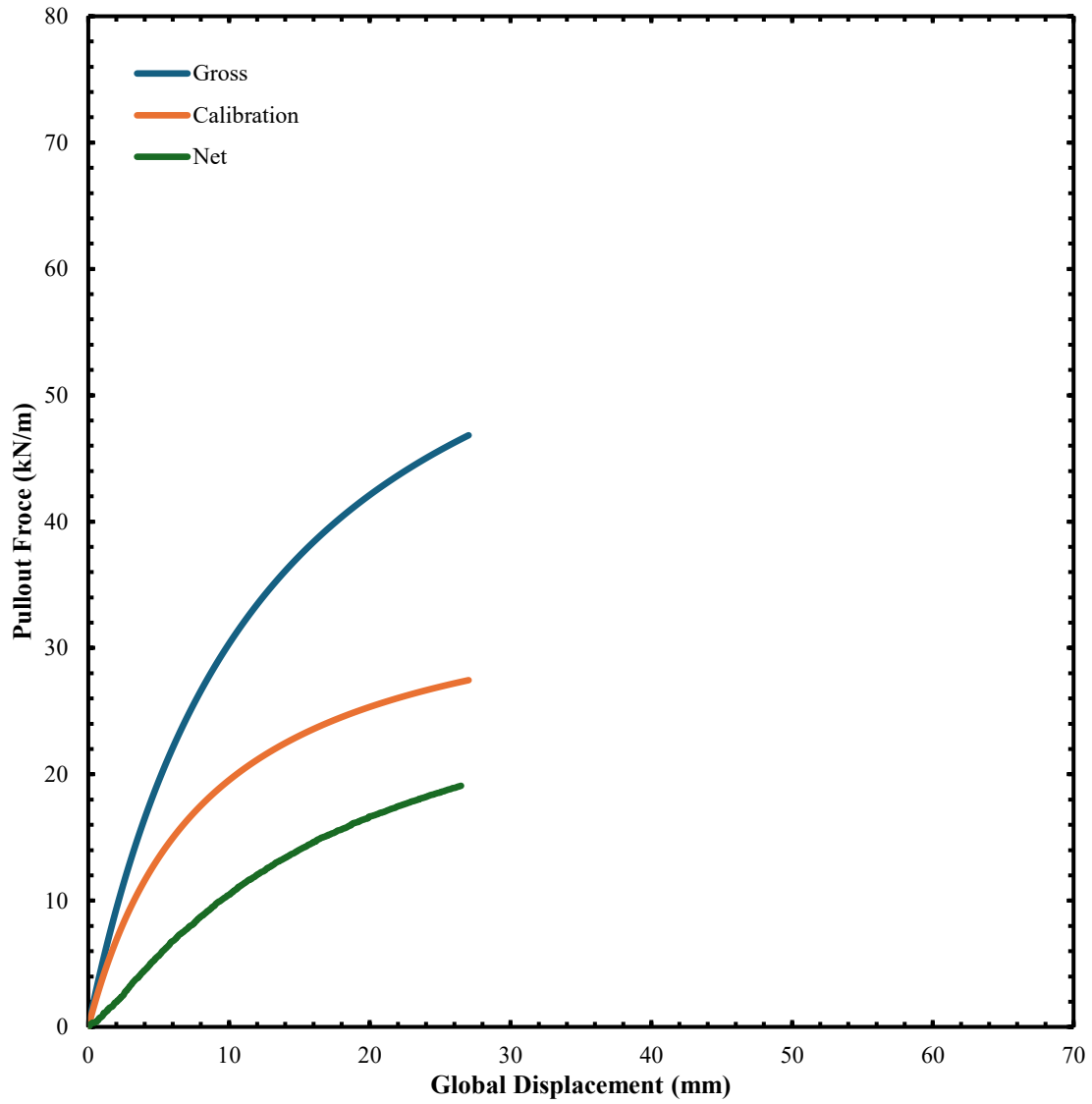


Figure B.1 Pullout Tests results of GT-1 at 21 kPa.

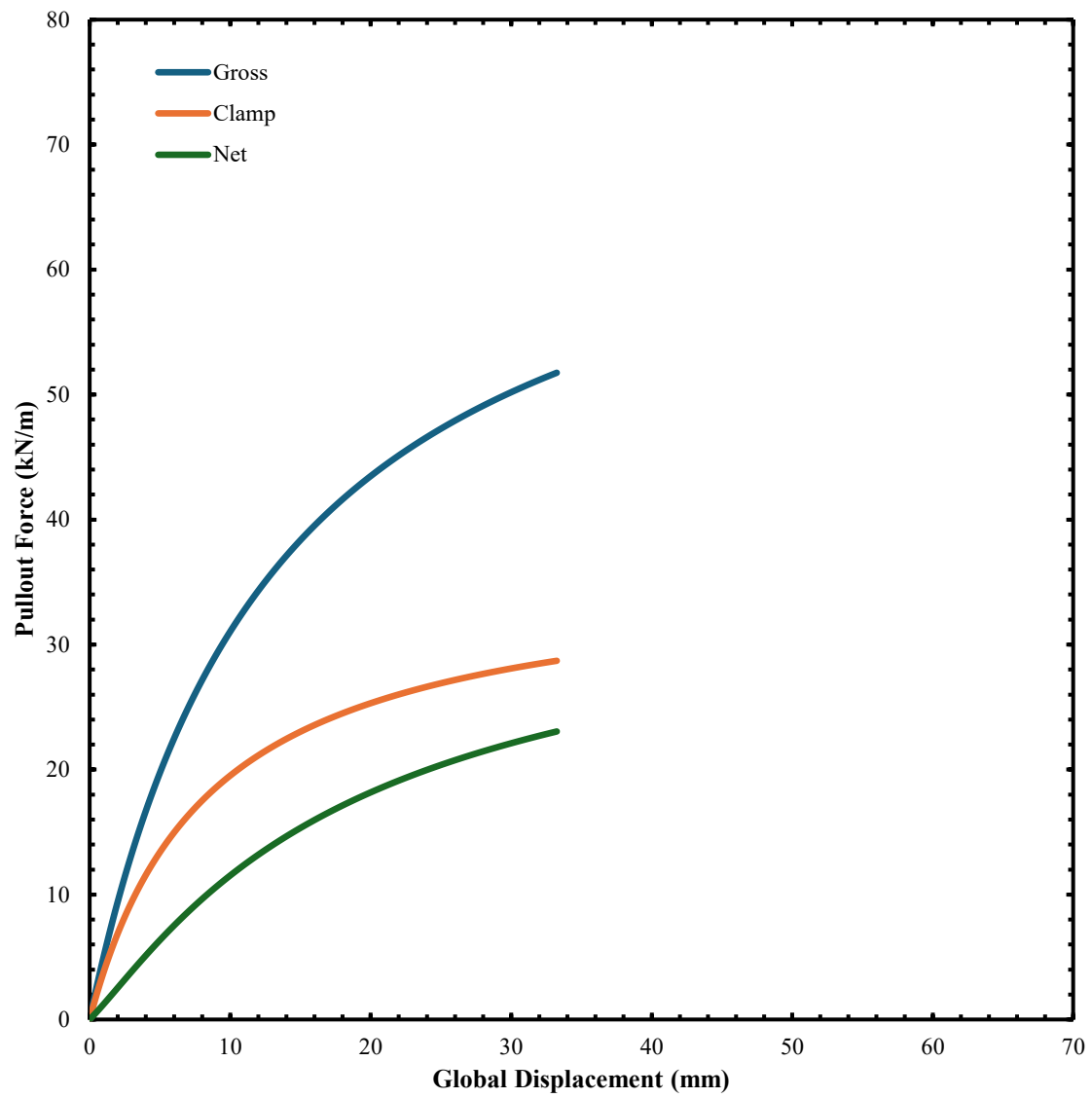


Figure B.2 Pullout Test results of GT-2 at 21 kPa.

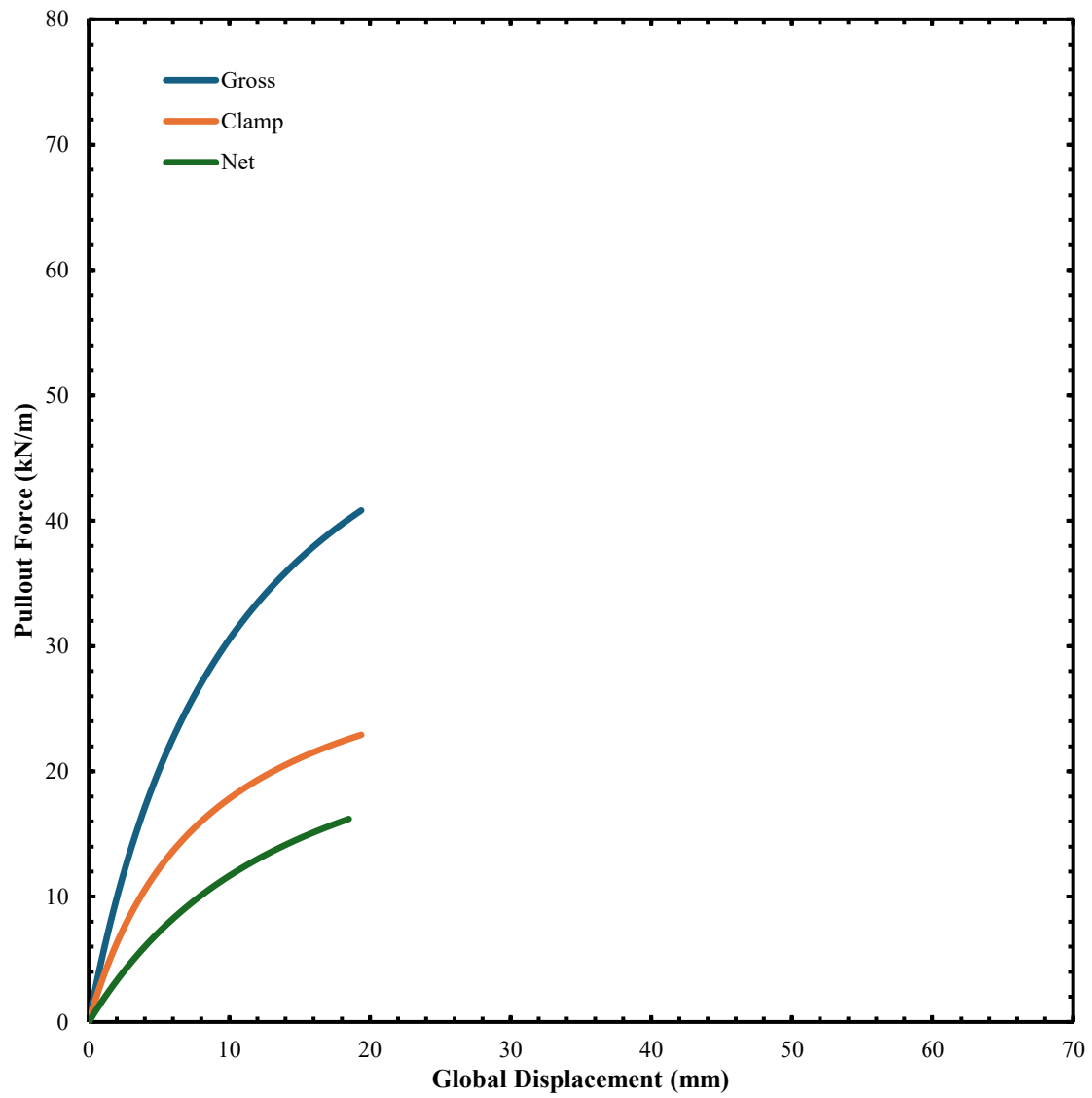


Figure B.3 Pullout Test results of GG-1 at 21 kPa.

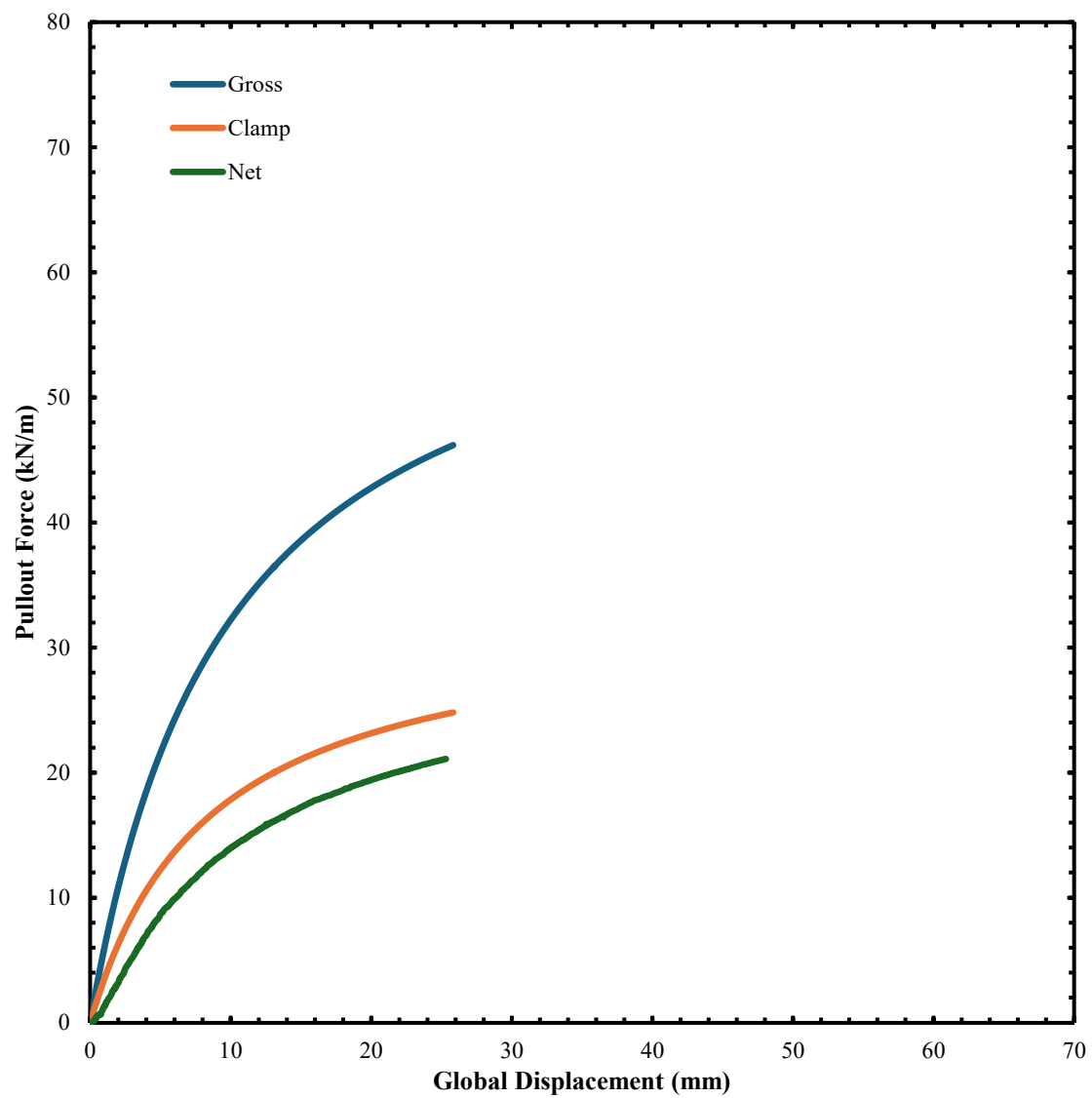


Figure B.4 Pullout Test results of GG-2 at 21 kPa.

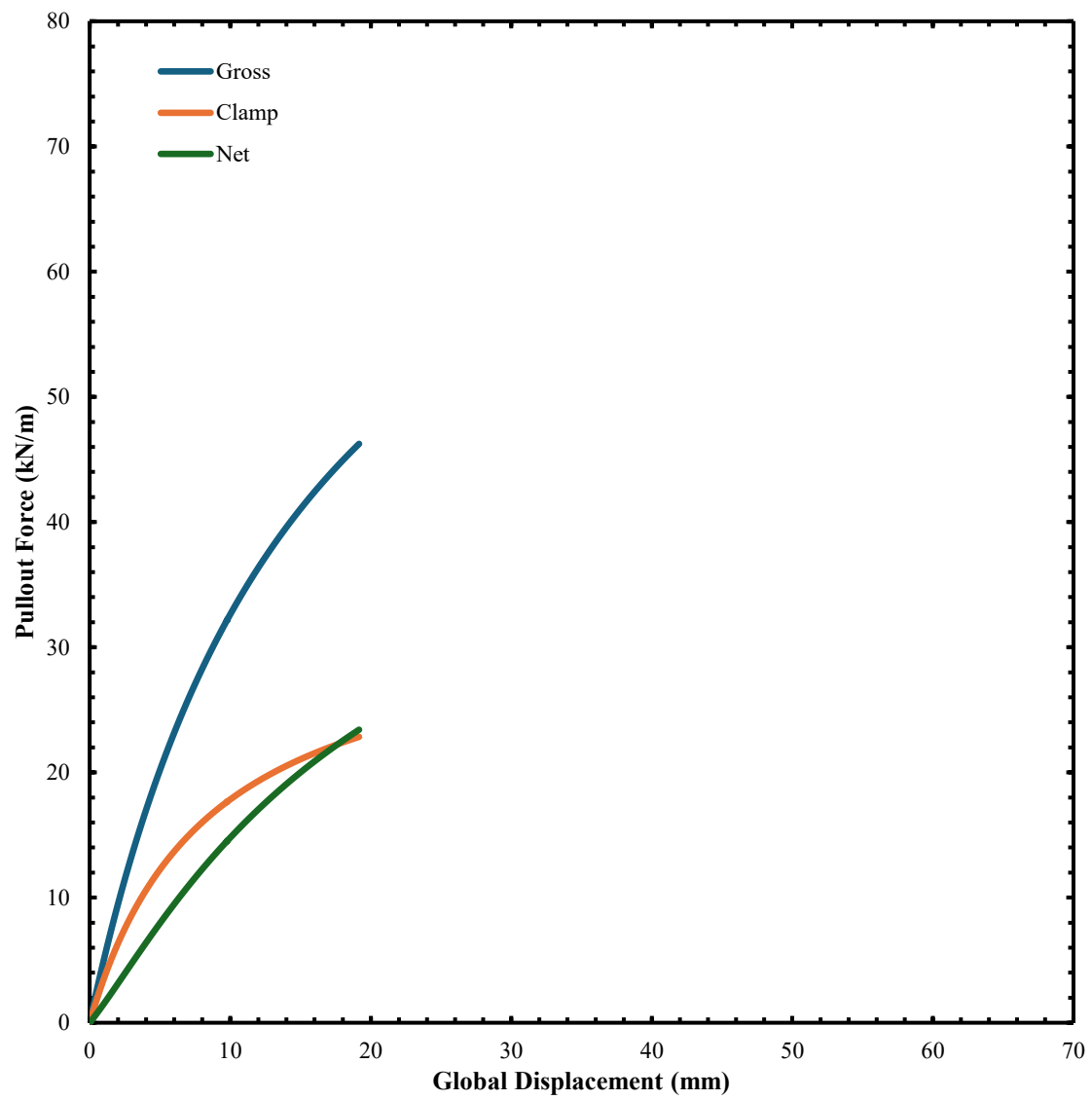


Figure B.5 Pullout Test results of GG-1 with GT-3 at 21 kPa.

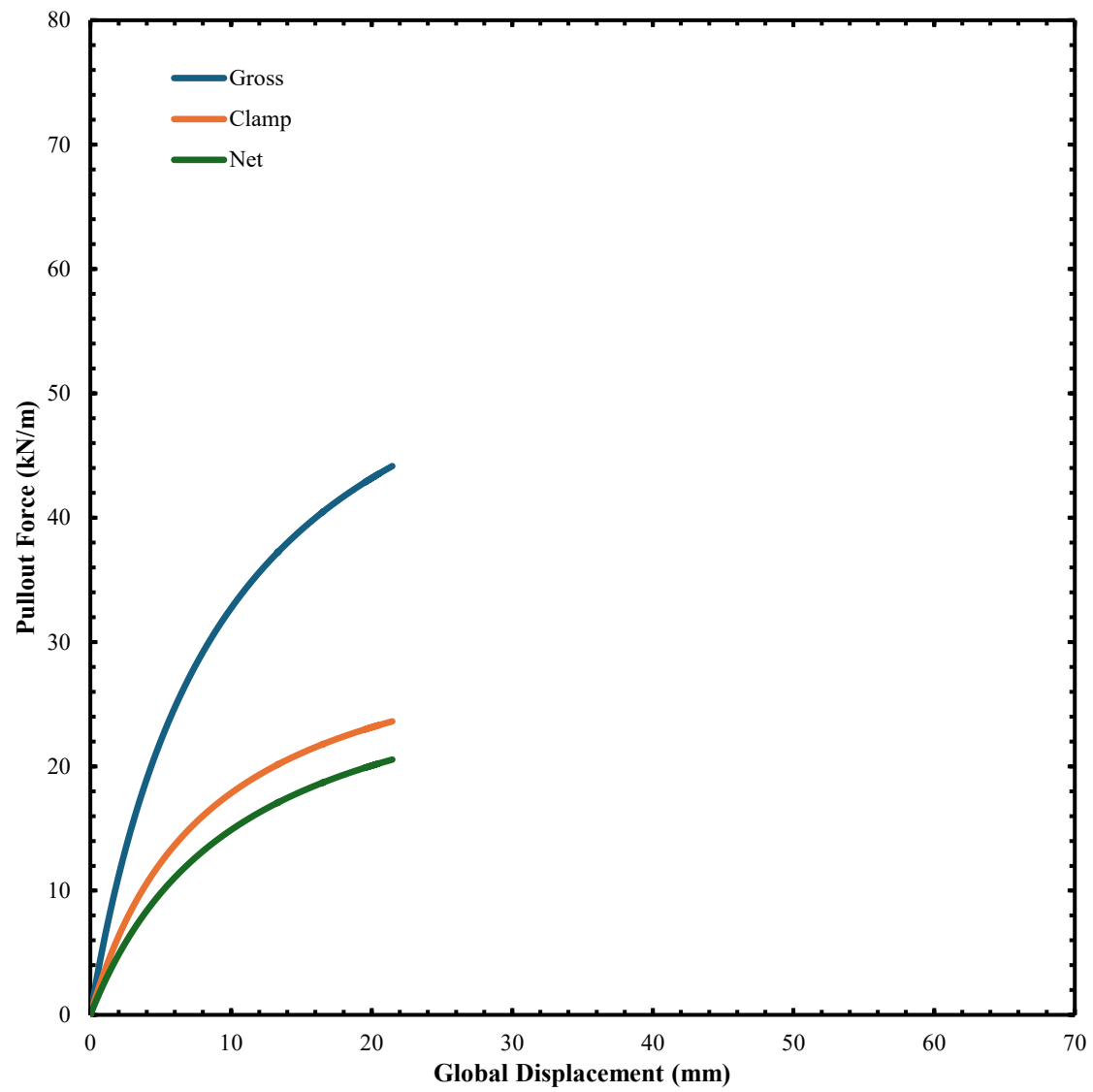


Figure B.6 Pullout Test results of GG-2 with GT-3 at 21 kPa.