

FRICTIONAL BEHAVIOR OF FACING ELEMENTS OF GEOSYNTHETIC
REINFORCED SOIL RETAINING STRUCTURES

by

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ABSTRACT

FRICITIONAL BEHAVIOR OF FACING ELEMENTS OF GEOSYNTHETIC REINFORCED SOIL RETAINING STRUCTURES

Geosynthetic reinforced soil retaining walls (primarily those with precast concrete block facing elements) reinforced by geogrids or geotextiles are in a period of enormous growth. The construction of geosynthetic reinforced soil retaining walls has gained popularity throughout the world and is becoming widespread in Turkey. Geosynthetic reinforced soil retaining walls with precast concrete block facing elements provide an aesthetically pleasing, cost-effective alternative that is easy to construct. In the construction, geosynthetic reinforcements are placed between the precast concrete block facing elements.

This study investigates the friction between dry-stacked concrete blocks and different types of geosynthetic reinforcements used in geosynthetic reinforced soil retaining walls. Three types of polymer geosynthetic reinforcements are compared to one another with respect to their frictional properties with empty dry-stacked concrete blocks, sand used as infill, and gravel used as infill in the tests. The angle of friction was calculated for each case. The results were comparatively evaluated.

The friction between concrete blocks and geosynthetic reinforcement is referred as the connection strength. An apparatus was developed to conduct a connection strength test. In this test, the tensile load applied on geosynthetic reinforcements was increased until complete pull-out occurred. The tests were repeated at different normal loads.

ÖZET

GEOSENTETİK DONATILI DUVARLARIN ÖN CEPHE ELEMANLARININ SÜRTÜNME ÖZELLİKLERİ

Geotekstil ve geogrid donatılar kullanılarak güçlendirilmiş olan geosentetik donatılı toprak istinat duvarlarının (öncelikli olarak prekast beton blok ön cephe elemanlı olanlar) sayısında çok büyük bir artış vardır. Geosentetik donatılı toprak istinat duvarlarının kullanımı tüm dünyada popüler hale gelmiştir ve Türkiye’de de yayılmaktadır. Prekast beton blok ön cephe elemanlarına sahip geosentetik donatılı toprak istinat duvarları estetik, ekonomik ve inşası kolay olan bir seçenektir. İnşa sırasında geosentetik donatılar, prekast beton blok ön cephe elemanlarının arasına yerleştirilirler.

Bu çalışma geosentetik donatılı toprak istinat duvarlarında kullanılan harçsız istiflenen beton bloklarla çeşitli geosentetik donatıların arasındaki sürtünmeyi araştırmaya yöneliktir. Üç çeşit polimer bazlı geosentetik donatı, beton blokların boş olduğu halde, kumla veya çakılla dolu oldukları hallerde, sürtünme özelliklerinin birbirleriyle karşılaştırılabilmesi için deneylerde kullanılmışlardır. Her bir durum için sürtünme açısı hesaplanmıştır. Sonuçlar karşılaştırmalı olarak değerlendirilmiştir.

Beton bloklar ile geosentetik donatılar arasındaki sürtünme bağlantı kuvveti olarak değerlendirilebilir. Bağlantı kuvveti deneyini yürütmek için bir düzenek geliştirilmiştir. Bu deneyde geosentetik donatılara uygulanan gerilme yükü göçmeye kadar arttırılmıştır. Bu deney farklı düşey yükler altında tekrarlanmıştır.

TABLE OF CONTENTS

| | |
|---|------|
| ACKNOWLEDGEMENTS | iii |
| ABSTRACT | iv |
| ÖZET | v |
| LIST OF FIGURES | viii |
| LIST OF TABLES | xiii |
| LIST OF SYMBOLS/ABBREVIATIONS | xv |
| 1. INTRODUCTION | 1 |
| 1.1. General | 1 |
| 1.2. Scope and Objective | 2 |
| 2. LITERATURE REVIEW | 4 |
| 2.1. Geosynthetics | 4 |
| 2.1.1. Geotextiles | 5 |
| 2.1.2. Geogrids | 9 |
| 2.2. Precast Concrete Block Facing Elements | 11 |
| 2.3. Geosynthetic Reinforced Soil Retaining Walls | 13 |
| 2.4. Design Methods | 16 |
| 2.4.1. External Stability | 17 |
| 2.4.2. Internal Stability | 18 |
| 2.4.3. Local Facing Stability | 18 |
| 2.5. Segmental Retaining Wall Construction | 31 |
| 3. METHODOLOGY | 33 |
| 3.1. General | 33 |
| 3.2. Connection Strength Test | 33 |
| 3.2.1. Materials | 33 |
| 3.2.2. Apparatus | 41 |
| 3.2.3. Setup and Testing | 43 |
| 3.2.4. Calculations | 46 |
| 4. RESULTS | 47 |
| 4.1. General | 47 |
| 4.2. Combination of Empty Blocks and Geotextile | 47 |

| | |
|--|----|
| 4.3. Combination of Empty Blocks and Geogrid I | 48 |
| 4.4. Combination of Empty Blocks and Geogrid II | 50 |
| 4.5. Combination of Sand as Infill and Geotextile | 51 |
| 4.6. Combination of Sand as Infill and Geogrid I | 53 |
| 4.7. Combination of Sand as Infill and Geogrid II | 54 |
| 4.8. Combination of Gravel as Infill and Geotextile | 56 |
| 4.9. Combination of Gravel as Infill and Geogrid I | 57 |
| 4.10. Combination of Gravel as Infill and Geogrid II | 59 |
| 4.11. Summary of Results | 60 |
| 5. CONCLUSIONS | 63 |
| REFERENCES | 66 |
| APPENDIX A. PICTURES TAKEN DURING EXPERIMENTS | 69 |

LIST OF FIGURES

| | |
|---|----|
| Figure 1.1. Types of geosynthetic reinforced soil wall facing | 2 |
| Figure 2.1. Woven geotextiles | 7 |
| Figure 2.2. Non-woven geotextiles | 8 |
| Figure 2.3. Knitted geotextiles | 8 |
| Figure 2.4. Uniaxial geogrid | 9 |
| Figure 2.5. Biaxial geogrid | 10 |
| Figure 2.6. The interlocking mechanism in geogrid-reinforced soil | 11 |
| Figure 2.7. Examples of modular block wall facing units | 12 |
| Figure 2.8. Generic cross-section of a MSE structure | 14 |
| Figure 2.9. External stability may happen by (a) sliding, (b) overturning, (c) bearing capacity, and (d) deep turning stability | 17 |
| Figure 2.10. Schematic of pullout test apparatus showing typical masonry concrete block units and geogrid reinforcement | 22 |
| Figure 2.11. Example normalized connection strength curves | 24 |
| Figure 2.12. Influence of sample size on connection strength | 26 |
| Figure 2.13. Connection strength test apparatus | 28 |

| | |
|--|----|
| Figure 2.14. Connection strength test apparatus (Plan view) | 28 |
| Figure 2.15. Connection strength vs. normal load | 31 |
| Figure 3.1. Picture of concrete blocks (Tech Block) used in connection tests | 34 |
| Figure 3.2. Picture of the geotextile used in connection tests | 35 |
| Figure 3.3. Picture of Geogrid I (biaxial geogrid) used in connection tests | 36 |
| Figure 3.4. Dimensions of Geogrid II | 37 |
| Figure 3.5. Picture of Geogrid II (uniaxial geogrid) used in connection tests | 38 |
| Figure 3.6. Grain size distribution curve for sand | 40 |
| Figure 3.7. Grain size distribution curve for gravel | 41 |
| Figure 3.8. Testing apparatus | 42 |
| Figure 3.9. Cross-section of the steel clamp | 43 |
| Figure 3.10. The cross-section of connection between concrete blocks and uniaxial geogrid | 45 |
| Figure 4.1. Plot of test results with holes left empty and geotextile used as reinforcement | 48 |
| Figure 4.2. Plot of test results with holes left empty and Geogrid I used as reinforcement | 49 |
| Figure 4.3. Plot of test results with holes left empty and Geogrid II used as reinforcement | 51 |

| | |
|--|----|
| Figure 4.4. Plot of test results with sand used as infill and geotextile used as reinforcement | 52 |
| Figure 4.5. Plot of test results with sand used as infill and Geogrid I used as reinforcement | 54 |
| Figure 4.6. Plot of test results with sand used as infill and Geogrid II used as reinforcement | 55 |
| Figure 4.7. Plot of test results with gravel used as infill and geotextile used as reinforcement | 57 |
| Figure 4.8. Plot of test results with gravel used as infill and Geogrid I used as reinforcement | 58 |
| Figure 4.9. Plot of test results with gravel used as infill and Geogrid II used as reinforcement | 60 |
| Figure A.1. The hooks hanging on the clamp of the test apparatus | 69 |
| Figure A.2. The clamp and the three holes on the clamp | 70 |
| Figure A.3. Picture of the test apparatus and precast concrete block facing elements | 70 |
| Figure A.4. The top view of the test apparatus and precast concrete block facing elements | 71 |
| Figure A.5. Picture taken before conduction of the first experiment | 71 |
| Figure A.6. Picture taken after conduction of the first experiment | 72 |
| Figure A.7. Picture taken before conduction of the second experiment | 72 |

| | |
|---|----|
| Figure A.8. Picture taken before conduction of the third experiment | 73 |
| Figure A.9. Picture taken before conduction of the fourth experiment | 74 |
| Figure A.10. Picture taken before conduction of the sixth experiment | 74 |
| Figure A.11. Picture taken during conduction of the sixth experiment | 75 |
| Figure A.12. Picture taken after conduction of the sixth experiment | 75 |
| Figure A.13. Picture taken before conduction of the eighth experiment | 76 |
| Figure A.14. Picture taken before conduction of the tenth experiment | 77 |
| Figure A.15. Picture taken after conduction of the twelfth experiment | 77 |
| Figure A.16. Picture taken after conduction of the twelfth experiment | 78 |
| Figure A.17. Picture taken before conduction of the fourteenth experiment | 78 |
| Figure A.18. Picture taken before conduction of the sixteenth experiment | 79 |
| Figure A.19. Picture taken before conduction of the seventeenth experiment | 80 |
| Figure A.20. Picture taken before conduction of the nineteenth experiment | 80 |
| Figure A.21. Picture taken after conduction of the nineteenth experiment | 81 |
| Figure A.22. Picture taken after conduction of the twentieth experiment | 81 |
| Figure A.23. Picture taken after conduction of the twenty first experiment | 82 |
| Figure A.24. Picture taken before conduction of the twenty third experiment | 83 |

Figure A.25. Picture taken before conduction of the twenty sixth experiment 83

LIST OF TABLES

| | | |
|------------|--|----|
| Table 3.1. | Physical and mechanical properties of Tech Block | 34 |
| Table 3.2. | Physical and mechanical properties of geotextile used in connection tests | 35 |
| Table 3.3. | Physical and mechanical properties of Geogrid I used in connection tests | 36 |
| Table 3.4. | Physical and mechanical properties of Geogrid II used in connection tests | 37 |
| Table 3.5. | Properties of soils used in tests | 39 |
| Table 3.6. | Data for mechanical grain size analysis of sand | 39 |
| Table 3.7. | Data for mechanical grain size analysis of gravel | 40 |
| Table 3.8. | The classification of 27 tests | 44 |
| Table 3.9. | Average weights of infill soils put in one hole of concrete blocks | 46 |
| Table 4.1. | Connection test results using empty blocks with geotextile used as reinforcement | 47 |
| Table 4.2. | Connection test results using empty blocks with Geogrid I used as reinforcement | 49 |
| Table 4.3. | Connection test results using empty blocks with Geogrid II used as reinforcement | 50 |

| | | |
|-------------|---|----|
| Table 4.4. | Connection test results with sand used as infill and geotextile used as reinforcement | 52 |
| Table 4.5. | Connection test results with sand used as infill and Geogrid I used as reinforcement | 53 |
| Table 4.6. | Connection test results with sand used as infill and Geogrid II used as reinforcement | 55 |
| Table 4.7. | Connection test results with gravel used as infill and geotextile used as reinforcement | 56 |
| Table 4.8. | Connection test results with gravel used as infill and Geogrid I used as reinforcement | 58 |
| Table 4.9. | Connection test results with gravel used as infill and Geogrid II used as reinforcement | 59 |
| Table 4.10. | Summary of results | 61 |
| Table 4.11. | Normalization of results | 62 |

LIST OF SYMBOLS / ABBREVIATIONS

| | |
|--------------------|--|
| a | Distance between two transversal ribs (mm) |
| a_{cs} | Apparent minimum connection strength between geosynthetic reinforcement and segmental retaining wall unit (kN/m) |
| b | Distance between two longitudinal ribs (mm) |
| C_c | Coefficient of curvature |
| C_u | Uniformity coefficient |
| c | Width of longitudinal ribs (mm) |
| c | cohesion (kN/m^2) |
| d | Width of transversal ribs (mm) |
| $E_{(n)}$ | Reinforcement placement elevation (m) |
| FS_{cs} | Factor of safety against connection failure |
| G | Specific gravity of solid constituents |
| t_1 | Thickness of junctions (mm) |
| t_2 | Thickness of longitudinal ribs (mm) |
| T_a | Allowable connection strength of the geosynthetic (kN/m) |
| $T_{c_1(n)}$ | Long-term allowable connection strength (kN/m) |
| T_{conn} | Connection strength (kN/m) |
| T_{conn}/T_{ult} | Normalized connection strength (kN/m) |
| T_{ult} | Ultimate tensile strength of the reinforcement (kN/m) |
| $T_{ultconn}$ | Maximum connection strength between geosynthetic reinforcement and segmental retaining wall unit (kN/m) |
| W_u | Segmental unit width (m) |
| $W_{w(n)}$ | The weight of the stacked SRW units (kN/m) |
| γ | Unit weight of soils used in tests (kN/m^3) |
| λ | Angle of friction (deg) |
| λ_{cs} | Apparent angle of friction for connection of geosynthetic reinforcement to segmental retaining wall unit (deg) |

| | |
|----------|--|
| σ | Normal stress on the failure plane (kN/m) |
| τ_f | Shear strength (kN/m) |
| AASHTO | American Association of State Highway and Transportation Officials |
| ASTM | American Society for Testing and Materials |
| CD | Cross Direction |
| FHWA | Federal Highway Administration |
| GRI | Geosynthetic Research Institute |
| GRS | Geosynthetic-reinforced soil |
| MBW | Modular block wall |
| MB-GRS | Modular-block geosynthetic-reinforced soil |
| MD | Machine Direction |
| MSE | Mechanically stabilized earth |
| MSEW | Mechanically stabilized earth wall |
| NCMA | National Concrete Masonry Association |
| NHI | National Highway Institute |
| PVC | Polyvinyl chloride |
| RD | Research and development |
| RE | Reinforced earth |
| RMC | Royal Military College of Canada |
| RSS | Reinforced soil slopes |
| SRW | Segmental retaining wall |
| SRWU-2 | Test method for determining shear strength of SRW |
| UV | Ultra Violet light ray |

1. INTRODUCTION

1.1. General

Geosynthetic reinforced soil retaining walls with precast concrete block facing elements are very commonly used for about 30 years. They have many advantages compared with conventional reinforced concrete and concrete gravity retaining walls. Mechanically Stabilized Earth (MSE) walls do not require large construction equipment. MSE walls are technically feasible to heights in excess of 25 m [1].

The relatively small quantities of manufactured materials required, rapid construction, and, competition among the developers of different proprietary systems has resulted in a cost reduction relative to traditional types of retaining walls. Mechanically Stabilized Earth (MSE) walls are likely to be more economical than other wall systems for walls higher than about three meters or where special foundations would be required for a conventional wall [1].

One of the greatest advantages of MSE walls is their flexibility and capability to absorb deformations due to poor subsoil conditions in the foundations. Also, based on observations in seismically active zones, these structures have demonstrated a higher resistance to seismic loading than have rigid concrete structures [1].

Precast concrete block facing elements for MSE walls can be made with various shapes and textures (with little extra cost) for aesthetic considerations [1]. Covering the geosynthetic reinforcement with precast concrete block facing elements provides protection against UV light degradation and vandalism at the wall face [2].

In Turkey, geotechnical engineers extensively use geosynthetic reinforced soil retaining walls in the last decades due to many advantages of this method.

Geotextiles and geogrids have been successfully used to construct geosynthetic reinforced soil retaining walls [2].

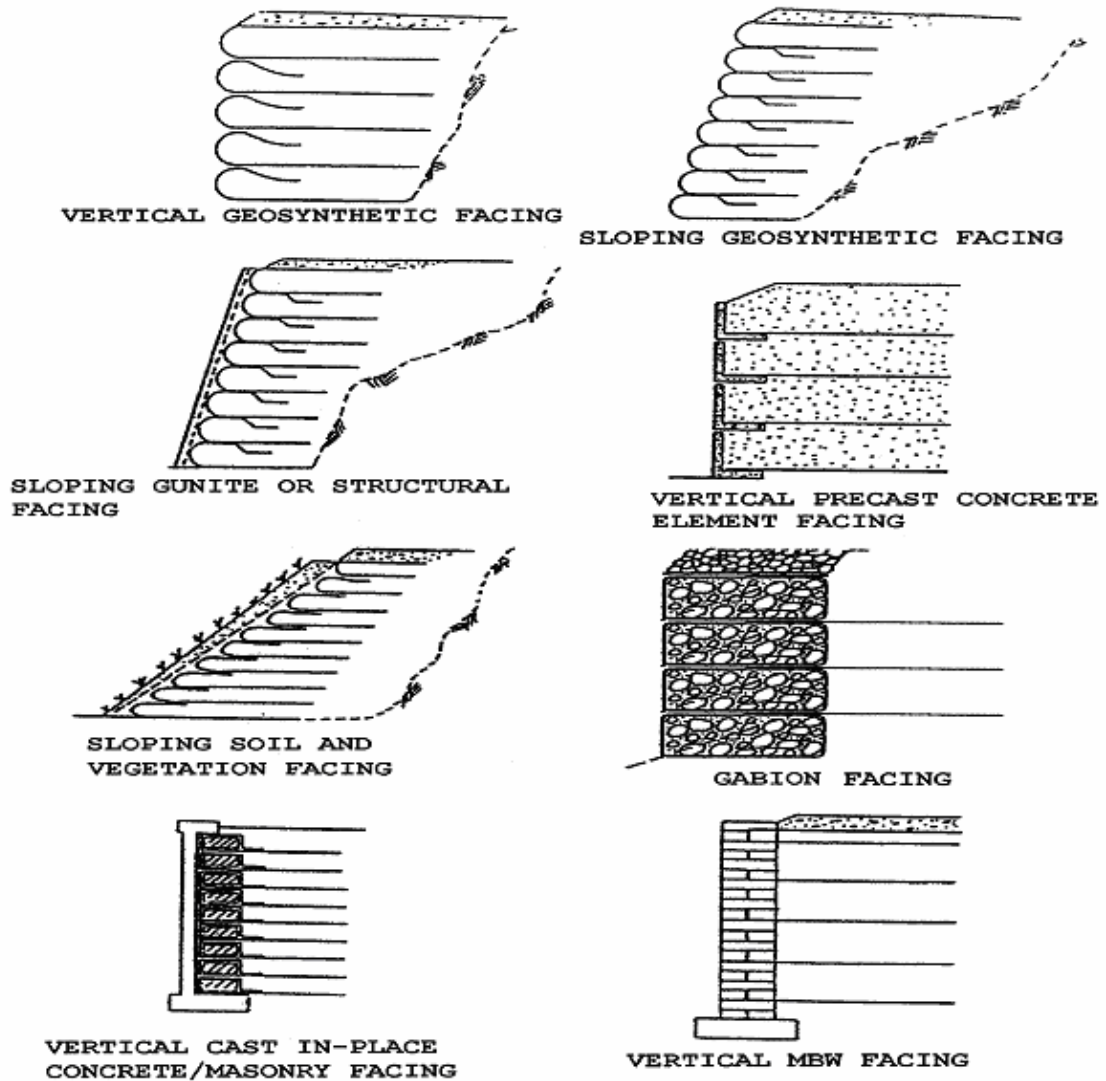


Figure 1.1. Types of geosynthetic reinforced soil wall facing [1]

1.2. Scope and Objective

The friction between precast concrete block facing elements and geosynthetic reinforcements is a vital subject for geosynthetic reinforced soil retaining walls in which any joint elements or stack not employed. In this type of reinforced walls facing stability is

maintained by the friction between precast concrete block facing elements and geosynthetic reinforcements.

Facing stability is related to the interaction between the facing units and the geosynthetic reinforcement. The connection strength between the facing unit and the geosynthetic reinforcement should be sufficient to prevent the rupture of geosynthetic or pullout of geosynthetic through facing units [3].

The objective of this study is to investigate the frictional behavior of precast concrete block facing elements with different types of geosynthetic reinforcements used in geosynthetic reinforced soil retaining walls.

2. LITERATURE REVIEW

2.1. Geosynthetics

In the past three decades, geosynthetics have been used successfully world-wide in several areas of civil engineering, and are now a well-accepted construction material. Their use offers excellent economic alternatives to the conventional solutions of many civil engineering problems. Geosynthetics are indispensable components of modern soil stabilizing systems like retaining walls or slopes [4].

Geosynthetics is a generic term for all synthetic materials used in conjunction with soil, rock and/or any other civil-engineering-related material as an integral part of a man-made project, structure or system. It includes a broad range of synthetic products; the most common ones are:

- geotextiles
- geogrids
- geonets
- geomembranes
- geocomposites

These products are almost exclusively polymeric, and those based on natural fibres (jute, cotton, wool, silk, etc.) are generally not included. They are available nowadays in numerous varieties in the market, under different trade names/designations for their use mainly in geotechnical, environmental, hydraulic and transportation engineering applications [4].

The term *geosynthetics* is formed by two words. The prefix “geo” is coming from the all soil- and rock-related activities in the geotechnical engineering within the general scope of the various applications. The fact that they are human-made products gives the second part to the name “synthetic”. In the last 30 years there has been a devastating increase in

the application of geosynthetics in geotechnical, environmental, hydraulic and transportation engineering areas. The reasons for this are numerous, among which are the following:

- They are indeed needed;
- They can be rapidly installed;
- They generally replace raw material resources;
- They generally replace difficult designs using conventional materials;
- Their timing is very appropriate;
- They are being aggressively marketed.

The materials used in their manufacture are primarily from the plastics industry, although rubber, fiberglass, and other materials are sometimes used [5].

In the reinforcement function, the purpose of the geosynthetic is to add tensile properties to soil, much in the same way that reinforcing steel is used with concrete. In both cases, materials with good compressive properties (soil and concrete) are combined with materials with good tensile properties (geogrids, geotextiles, and steel) to construct a structure with adequate compressive and tensile strengths [6].

The specific families of geosynthetics on which we will focus are the following:

- geotextiles
- geogrids

2.1.1. Geotextiles

Geotextiles are thin, flexible, permeable sheets of synthetic material used to stabilize and develop the performance of soil associated with civil engineering works. Correctly designed and installed, geotextiles have the ability to filter, drain, reinforce and separate soil. In many implementations, geotextiles may be designed and selected to perform a

combination of these functions. For example, when placed at the base of a granular fill embankment constructed over soft clay all four functions might operate.

Like so many other techniques employed in civil engineering, the idea of using fabrics is not entirely novel. Over a century ago, sheets of canvas were incorporated in earth fill to decrease lateral thrusts exerted behind retaining walls. Some fifty years ago, cotton duck fabric, similar to denim, was used in the United States to stabilize dirt roads. In the early fifties, synthetic textiles were used by the Dutch as filters in the rapid repair of the North Sea dykes. Many more instances of this novelty can be found stretching back over the years; however, the improvement of modern geotextiles started in the late sixties [7].

Geotextiles form the largest group of geosynthetics. Their rise in growth during the past 30 years has been nothing short of awesome. In fact they are textiles in a traditional sense, but consist of synthetic fibers rather than natural ones like cotton, wool, and silk. Thus biodegradation is not a problem. The fibers are made into a flexible, porous fabric by standard weaving machinery or are matted together in a random, or nonwoven, manner. Some are also knit. The major point is that they are porous to water flow across their manufactured plane and also within their plane, but to a widely varying degree. There are at least 80 specific application areas for geotextiles that have been advanced; however, the fabric always performs at least one of five discrete functions:

- Separation
- Reinforcement
- Filtration
- Drainage
- Moisture barrier [5]

The role of the fabric manufacturer in the stimulation and growth of the geotextile market has been both large and positive. Many fiber types and fabric styles have been developed both for general use and for specific applications. In fact, it seems that these two approaches toward the marketing of geotextiles represent the current situation: manufacturers with one basic fabric style (usually available in different weights) and manufacturers having a variety of fabric types, targeting each to a specific implementation.

Three points are important insofar as manufacturing is concerned: type of polymer, type of fiber, and fabric style.

The fibers used in geotextiles are made from the following materials, listed in order of decreasing use:

- polypropylene
- polyester
- polyamide (nylon)
- polyethylene
- other polymers and glass [5].

Woven geotextiles are made from yarns (made of one or several fibres) by conventional weaving process with regular textile structure. Figure 2.1. illustrates an example of woven geotextiles [4].

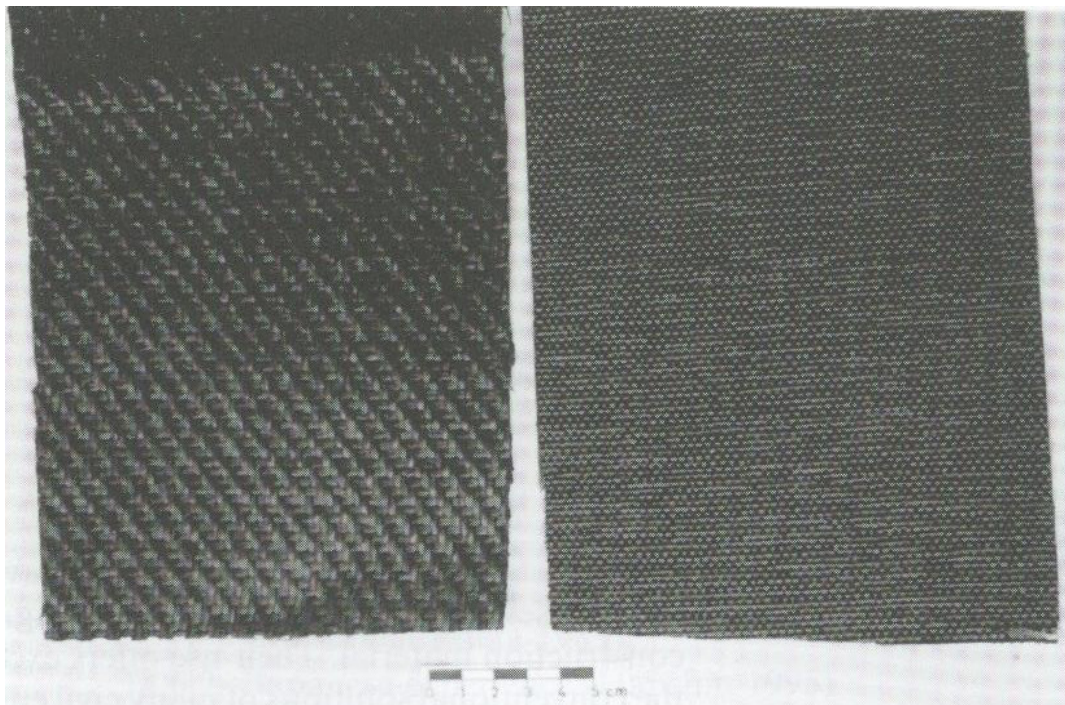


Figure 2.1. Woven geotextiles [4]

Non-woven geotextiles are made from directionally or randomly oriented fibres into a loose web by bonding with partial melting, needle punching or chemical binding agents (glue, rubber, latex). Non-woven geotextiles are shown in Figure 2.2. [4].

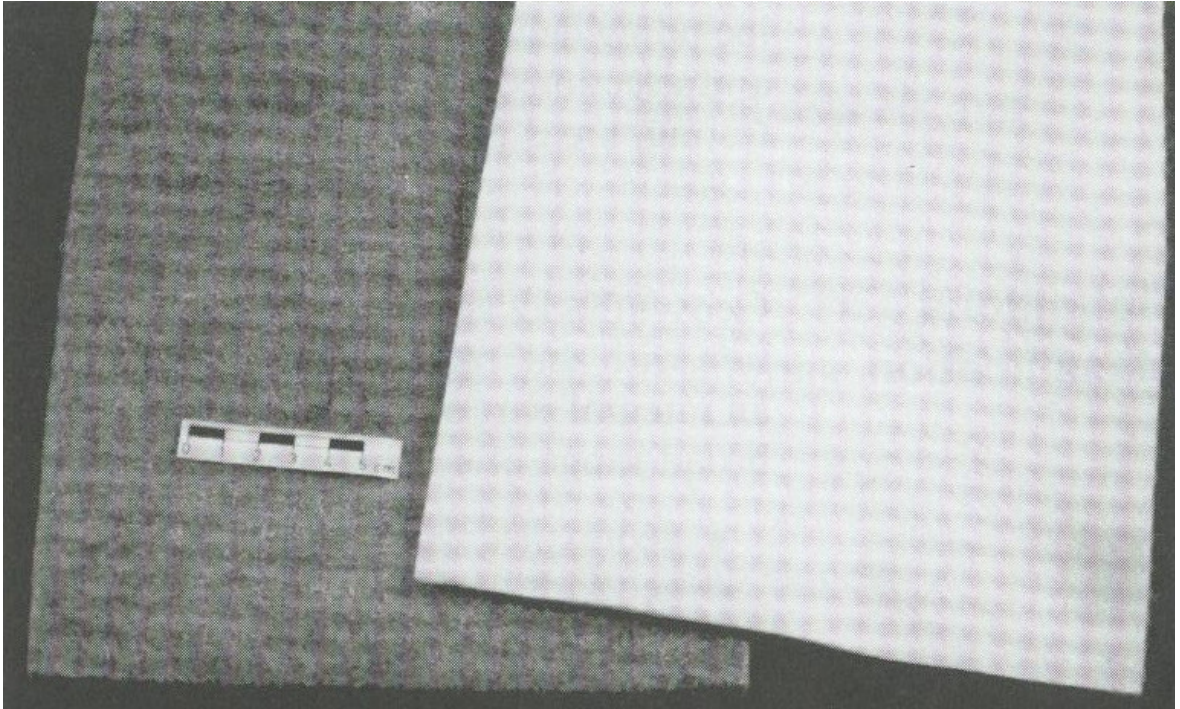


Figure 2.2. Non-woven geotextiles [4]

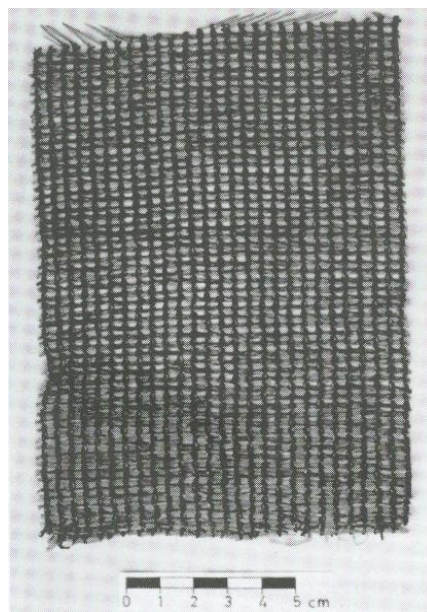


Figure 2.3. Knitted geotextiles [4]

Knitted geotextiles are produced by interlooping one or more yarns together. Knitted geotextiles are shown in Figure 2.3. Stitch-bonded geotextiles are formed by the stitching together of fibres or yarns [4].

2.1.2. Geogrids

Geogrid is a polymeric, mesh-like planar product formed by intersecting elements, called ribs, joined at the junctions. The ribs can be linked by extrusion, bonding or interlacing, and the resulting geogrids are called extruded geogrid, bonded geogrid and woven geogrid, respectively. Extruded geogrids are classified into two categories based on the direction of stretching during their manufacture:

- uniaxial geogrids
- biaxial geogrids

Uniaxial geogrids are made by the longitudinal stretching of regularly punched polymer sheets and, therefore, possess a much higher tensile strength in the longitudinal direction than in the transverse direction. A sample of uniaxial geogrid is displayed in Figure 2.4. [4].

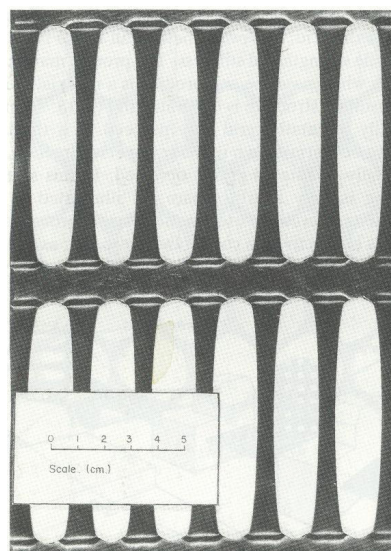


Figure 2.4. Uniaxial geogrid [7]

Biaxial geogrids are made by both the longitudinal and the transverse stretchings of regularly punched polymer sheets and, therefore, possess equal tensile strength in both the longitudinal and the transverse directions [4]. Figure 2.5. shows an example of biaxial geogrid.

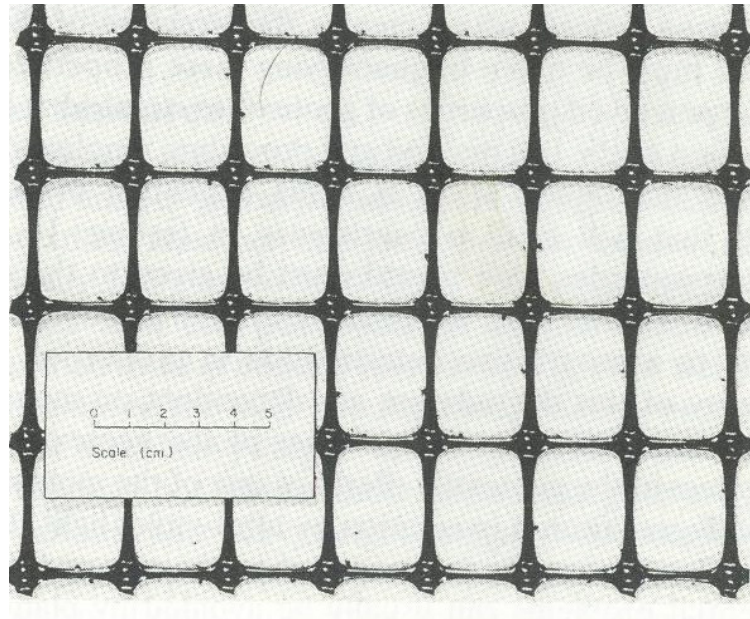


Figure 2.5. Biaxial geogrid [7]

The key feature of geogrids is that the openings between the longitudinal and transverse ribs, called apertures, are large enough to create interlocking with the surrounding soil particles as shown in Figure 2.6. The interaction between soils and geogrids is well because of the open geometry of geogrids. The shapes of the apertures are either elongated ellipses, near-squares with rounded corners, squares or rectangles. The dimensions of the apertures vary from about 2-5 to 15cm. The ribs of geogrids are often quite stiff compared to the fibres of geotextiles. In addition, the junction strength is important in the case of geogrids because, through these junctions, loads are transmitted from one type of rib to the other when placed into the soil [4].

Geogrids are high modulus reinforcement materials, since they are produced by plastics formed into a very open mesh-like configuration. They are used in reinforcement and separation.

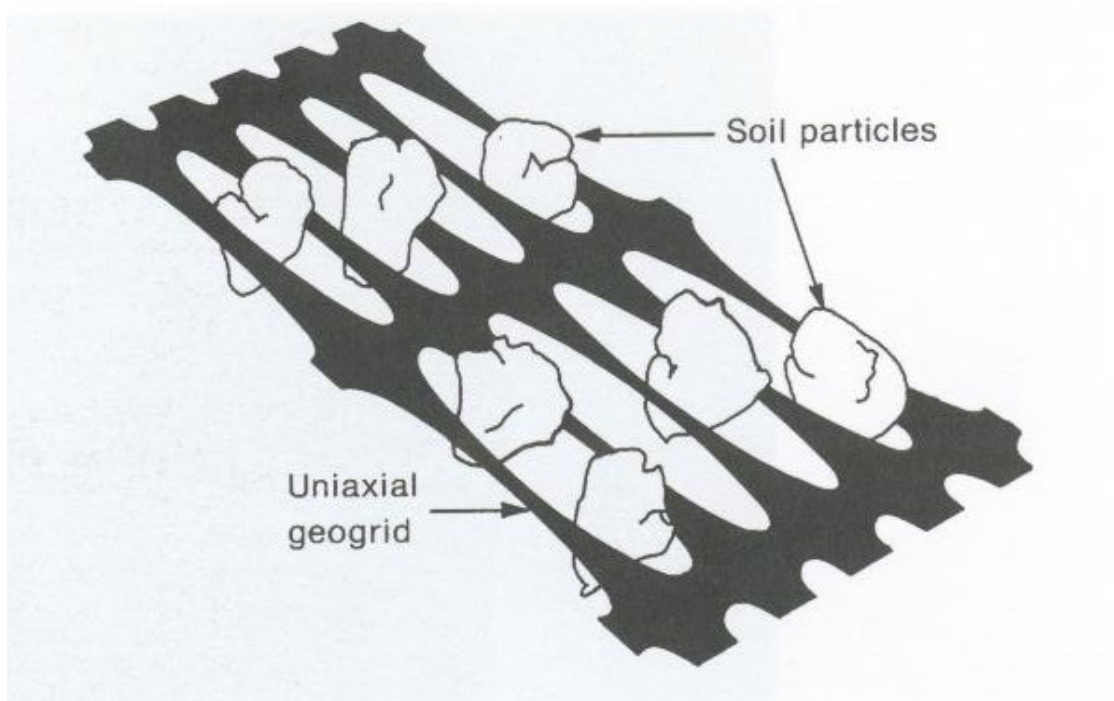


Figure 2.6. The interlocking mechanism in geogrid-reinforced soil [4]

2.2. Precast Concrete Block Facing Elements

Precast concrete block facing elements can be cast in several shapes and supplied with facing textures to match environmental requirements and blend aesthetically into the environment. Figure 2.7. illustrates the various types of retaining wall facing elements. Geosynthetic reinforced soil retaining structures using precast concrete elements as the facings can have surface finishes similar to any reinforced concrete structure [1].

Modular block wall (MBW) units are the most common facing currently used for geosynthetic MSE wall construction. They are approved by the geotechnical engineers because of their aesthetic appeal, widespread availability, and relative low cost.

MBW units may be produced solid or with cores. Full height cores are typically filled with aggregate during erection of retaining wall. Units are normally dry-stacked in a running bond formation.

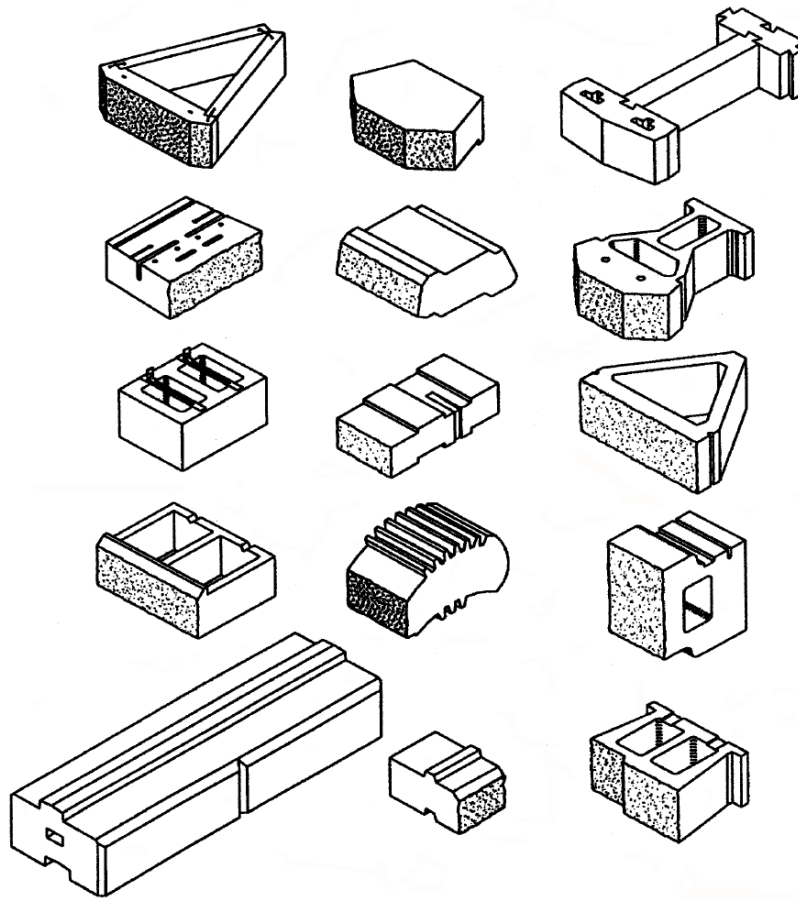


Figure 2.7. Examples of modular block wall facing units [1]

MBW units are relatively small, short and thick concrete units, specially designed and manufactured for retaining wall applications. The units are typically manufactured by a dry casting process and weigh 15 to 50 kg each. The nominal depth (dimension perpendicular to wall face) of MBW units usually ranges between 200 and 600 mm. Unit heights typically change between 100 and 200 mm for the various manufacturers. Exposed face length typically varies between 200 and 600 mm [8].

Retaining structures with metal facings have the disadvantage of shorter life because of corrosion, unless provision is made to compensate for it [1].

Recently, modular block dry cast facing elements have gained acceptance due to their lower cost and worldwide availability. These small concrete units are generally mated with geosynthetic reinforcement, and the wall system is referred to as modular block wall

(MBW). It has been reported that more than 200,000 m² of MBW walls have been constructed yearly in the United States when considering all types of transportation related applications. The current yearly usage for transportation-related applications is estimated at about 50 projects per year [1].

The precast concrete block facing elements may be held together by interface friction, concrete keys or mechanical connectors [3]. The goal of this study is to examine interface friction between the concrete blocks and geotextile or geogrids which are commonly used in geosynthetic reinforced soil retaining wall applications in Turkey.

2.3. Geosynthetic Reinforced Soil Retaining Walls

The modern methods of soil reinforcement for retaining wall construction were pioneered by the French architect and engineer Henri Vidal in the early 1960s. His research led to the invention and development of Reinforced Earth[®], a system in which steel strip reinforcement is used. The first wall to use this technology in the United States was constructed in 1972 on California State Highway 39, northeast of Los Angeles. In the last 25 years, more than 23,000 Reinforced Earth structures representing over 70,000,000 m² of wall facing have been completed in 37 countries. More than 8,000 walls have been built in the United States since 1972. In course of time geosynthetic reinforcements replaced steel strip reinforcements in the construction of retaining earth structures since they offer flexibility, cost effectiveness and high performance [1].

Geosynthetic reinforced soil retaining walls consist of geosynthetic reinforcements, mechanically stabilized earth mass, retained backfill, leveling pad and facing elements as shown in Figure 2.8. The constitution of these components offers a cost-effective, aesthetically pleasing, high performance retaining wall option to the geotechnical engineers. Geosynthetic reinforced soil retaining walls are given increased attention all over the world like in Turkey.

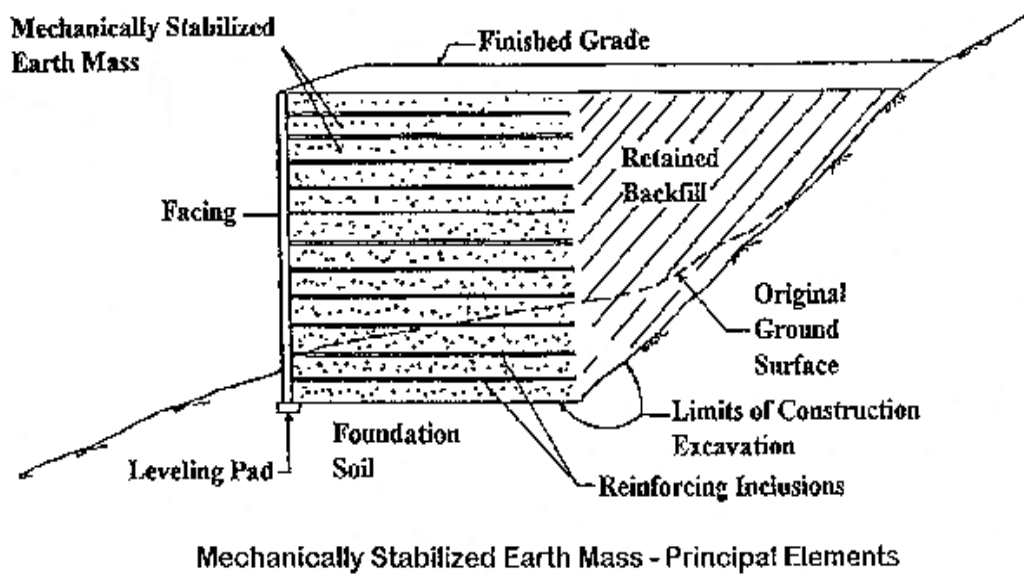


Figure 2.8. Generic cross-section of a MSE structure [1]

Modern reinforced soil technology started with the development of the proprietary reinforced earth (RE) wall. In RE walls, metallic strips tie the concrete panel facing at the front end into a stable soil mass in the rear. An alternative system made of polymeric geogrids combined with modular dry cast concrete blocks as the facing is increasingly being used in recent years to form modular-block geosynthetic-reinforced soil (MB-GRS) walls. The MB-GRS walls are aesthetically pleasing and can be more economical compared to traditional GRS walls and RE walls. Modular-block wall systems are used frequently in the private sector; their acceptance in the public sector, mainly in transportation applications, is steadily increasing [9].

In modular-block walls, the geosynthetic layers are placed between stacked concrete blocks at regular vertical spacings. The integrity of the wall facing is maintained through the interaction between the blocks, the blocks and backfill, as well as between the blocks and confined geosynthetic layers. In a few wall systems, the blocks and reinforcement are connected by mechanical means, such as the pins. In most of the wall systems the geosynthetic layers are connected to the facing via friction developed between the stacked blocks and the embedded geosynthetic [9].

In geosynthetic reinforced soil retaining walls polymeric reinforcements are generally geotextiles and high-modulus geogrids. Because of the lower stress-strain modulus of fabrics as compared to steel, geotextile walls likely deform more than Reinforced Earth (reinforced by metallic strips) walls, and an analysis of short-term and long-term strains is deemed necessary [2].

However the metal strips and the connections of strip reinforcement is prone to corrosion and therefore should be used with precaution. Geosynthetic reinforced soil retaining walls are either constructed with geogrids or geotextiles as the reinforcing element. Geotextiles are more frequently used in the construction of geosynthetic reinforced soil retaining walls in Turkey because of economic considerations [3].

The use of geotextiles in MSE walls and Reinforced Soil Slopes (RSS) started after the beneficial effect of reinforcement with geotextiles was noticed in highway embankments over weak subgrades. The first geotextile reinforced wall was constructed in France in 1971, and the first structure of this type in the United States was constructed in 1974. Since about 1980, the use of geotextiles in reinforced soil has increased significantly [1].

Geogrids for soil reinforcement were developed around 1980. The first use of geogrid in earth reinforcement was in 1981. Extensive use of geogrid products in the United States started in about 1983, and they now comprise a growing portion of the market [1].

Mechanically Stabilized Earth (MSE) walls and Reinforced Soil Slopes (RSS) are economic soil-retaining structures that can tolerate much larger settlements than reinforced concrete walls. By placing tensile reinforcing elements (inclusions) in the soil, the strength of the soil can be improved significantly such that the vertical face of the soil/reinforcement system is essentially self supporting. Use of a facing system to prevent soil raveling between the reinforcing elements allows very steep slopes and vertical walls to be constructed safely. In some cases, the inclusions can also withstand bending from shear stresses, providing additional stability to the system [1].

It is estimated that more than 700,000 m² of MSE retaining walls with precast facing are constructed on average every year in the United States, which may represent more than half of all retaining wall usage for transportation applications [1].

2.4. Design Methods

Since the development of soil reinforcement concepts and their application to MSEW structure design, a number of design methods have been proposed, used, and refined. Current practice consists of determining the geometric and reinforcement requirements to prevent internal and external failure using limit equilibrium methods of analysis [1].

External stability evaluations for MSEW structures treat the reinforced section as a composite homogeneous soil mass and evaluate the stability according to conventional failure modes for gravity type wall systems. Differences in the present practice exist for internal stability evaluations which determines the reinforcement required, principally in the development of the internal lateral stress and the assumption as to the location of the most critical failure surface [1].

Internal stability is treated as a response of discrete elements in a soil mass. This suggests that deformations are controlled by the reinforcements rather than total mass, which appears inconsistent given the much greater volume of soil in such structures. Therefore, deformation analyses are generally not included in current methods [1].

During designing geosynthetic reinforced soil retaining walls which are covered with modular block dry cast facing elements geotechnical engineers pay attention to four general modes of failure.

- external stability
- internal stability
- local facing stability
- global stability

2.4.1. External Stability

External stability calculations consider the reinforced zone or infill soil and the dry-stacked column of facing units to act as a monolithic gravity mass. The four modes of failure shown in Figure 2.9, base sliding, overturning, bearing capacity, and deep turning failure, are considered separately and independent of the properties of facing units and the connection between the facing units and the geosynthetic. The whole design procedure for external stability may be found in NCMA Manual [10].

Due to the flexibility and satisfactory field performance of MSE walls, the adopted values for the factors of safety for external failure are in some cases lower than those used for reinforced concrete cantilever or gravity walls. For example, the factor of safety for overall bearing capacity is 2.5 rather than a higher value, which is used for more rigid structures [1].

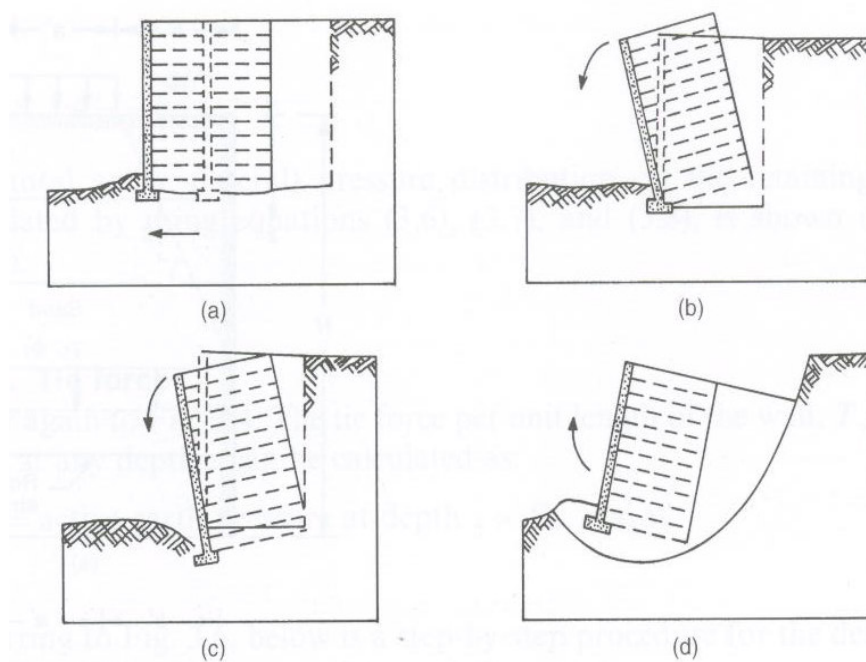


Figure 2.9. External stability may happen by (a) sliding, (b) overturning, (c) bearing capacity, and (d) deep turning stability [4]

2.4.2. Internal Stability

Internal stability calculations are carried out to evaluate the integrity of the reinforced zone as a monolithic composite comprised of geosynthetic reinforcement, soil and facing units. The tensile forces to be resisted by horizontal reinforcement layers are calculated using Coulomb lateral earth pressure theory. Tensile overstress, pullout, and internal sliding are considered separately and independent of the properties of facing units and the connection between the facing units and the geosynthetic. The whole design procedure for internal stability may be found in NCMA Manual [10].

Internal failure of a MSE wall can occur in two different ways:

- The tensile forces (and, in the case of rigid reinforcements, the shear forces) in the inclusions become so large that the inclusions elongate excessively or break, leading to large movements and possible collapse of the structure. This mode of failure is called failure by elongation or breakage of the reinforcements.
- The tensile forces in the reinforcements become larger than the pullout resistance, i.e., the force required to pull the reinforcement out of the soil mass. This, in turn, increases the shear stresses in the surrounding soil, leading to large movements and possible collapse of the structure. This mode of failure is called failure by pullout [1].

The process of sizing and designing to preclude internal failure, therefore, consists of determining the maximum developed tension forces, their location along a locus of critical slip surfaces and the resistance provided by the reinforcements both in pullout capacity and tensile strength [1].

2.4.3. Local Facing Stability

The local stability of facing units which is influenced by construction procedures largely determines the visual identification of the structural performance of the composite system of facing units, soil and geosynthetic reinforcement. Local facing stability is related to the interaction between the facing units and the geosynthetic reinforcement. Local stability is controlled by the specific engineering properties of facing units [3].

The connection strength and the stiffness between the facing unit and the geosynthetic reinforcement should be sufficient to prevent the rupture of geosynthetic or pullout of geosynthetic through facing units [3].

The connection strength at each reinforcement placement elevation, $E_{(n)}$, must be sufficient to prevent rupture or slippage of the reinforcement due to applied tensile force. Limit state strength of the connection at failure and the serviceability state strength of the connection, which is the strength at a specified deformation are two criteria to be addressed. Using the limit state strength, the ultimate connection strength is evaluated as:

$$T_{c_l(n)} = T_{ultconn(n)} / FS_{cs} \leq T_{a(n)} \quad (2.1)$$

where:

| | | |
|------------------|---|--|
| $T_{c_l(n)}$ | = | Long-term allowable connection strength |
| FS_{cs} | = | Factor of safety against connection failure, typically equal to 1.5 |
| $T_{ultconn(n)}$ | = | Maximum connection strength between geosynthetic reinforcement and segmental retaining wall unit |
| $T_{a(n)}$ | = | Allowable connection strength of the geosynthetic |

NCMA recommends the maximum allowable movement of the connections to minimize wall face deformation to be 0.75 inches between two consecutive blocks. In this case, the connection strength at 0.75 inches must be lower than the allowable strength of the geosynthetic. The results of laboratory connection testing is also difficult to interpret, particularly with friction connections where sliding of the reinforcement is observed. This typically occurs at low normal stresses, i.e., near the top of the wall. NCMA limits this slippage to 19 mm (0.75 inches) at which point the corresponding load is assumed for T_{conn} (the connection strength). The maximum connection strength, $T_{ultconn(n)}$, of any specific combination of geosynthetic reinforcement and height of facing units can be determined by laboratory testing. To relate connection strength and the applied load, a_{cs} (apparent minimum connection strength between geosynthetic reinforcement and SRW unit) and λ_{cs} (apparent angle of friction for connection of geosynthetic reinforcement to SRW unit) need

to be determined. Then the connection strength at each geosynthetic reinforcement elevation $E_{(n)}$ can be calculated as:

$$T_{ultconn(n)} = a_{cs} + W_{w(n)} \tan \lambda_{cs} \quad (2.2)$$

where: $W_{w(n)}$ = The weight of the stacked SRW units

The serviceability connection strength may be established using the parameters obtained at 0.75 inch deformation.

The connection must be able to resist the tensile force transferred from the reinforced soil mass to the connection. The tensile force to be resisted is a function of the vertical spacing of reinforcement and proximity of the internal failure surface to the connection. The facing connection between the geosynthetic reinforcement and facing units at each reinforcement placement elevation $E_{(n)}$ must have sufficient connection strength to prevent rupture or slippage of the reinforcement due to the applied tensile force [3].

For modular concrete facing blocks (MBW), sufficient inter-unit shear capacity must be available, and the maximum spacing between reinforcement layers shall be limited to twice the front to back width, W_u , of the modular concrete facing unit or 0.8m (32 inches) whichever is less. The maximum facing height above the uppermost reinforcement layer and the maximum depth of facing below the bottom reinforcement layer should be limited to the width, W_u , of the modular concrete facing unit used.

The inter-unit shear capacity as obtained by testing (Traditional test methods were described by SRWU-2, NCMA) at the appropriate normal load should exceed the horizontal earth pressure at the facing by a Factor of Safety of 2.

On the other hand, geosynthetic materials should not be left exposed to sunlight (specifically ultraviolet radiation) for permanent walls. If geosynthetic elements must be left exposed permanently to sunlight, the geosynthetic shall be stabilized to be resistant to ultraviolet radiation. Furthermore, product specific test data should be provided which can

be extrapolated to the intended design life and which proves that the product will be capable of performing as intended in an exposed environment. Alternately a protective facing shall be constructed in addition (e.g., concrete, shotcrete, etc.) [1].

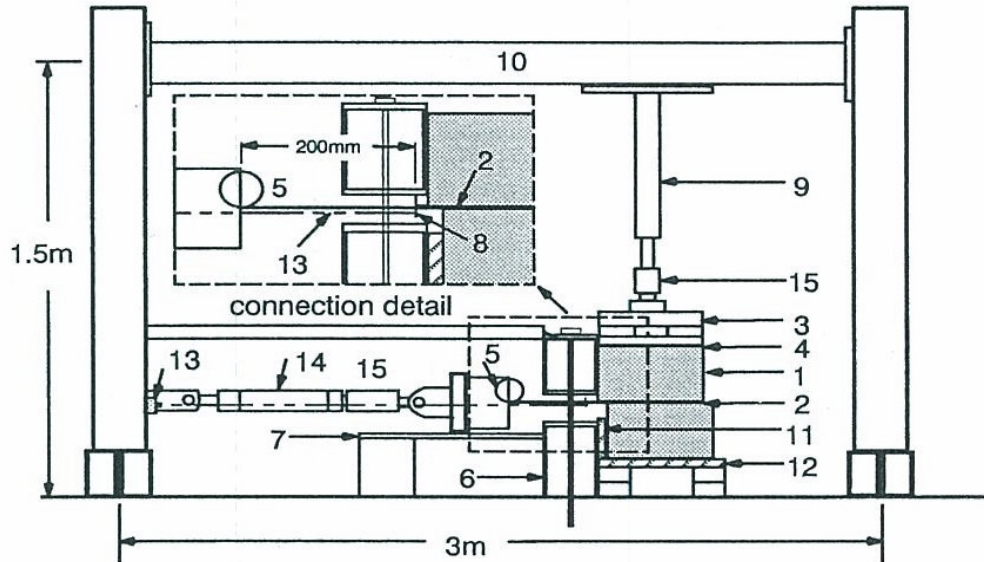
Geosynthetic-soil interface sliding and pullout of reinforcement within anchorage zones are potential failure mechanisms in reinforced walls. Geosynthetic-reinforced soil retaining walls comprise dry-stacked columns of modular concrete blocks which may be solid or infilled with granular soil. The connection between the facing column and reinforced soil mass is typically formed by extending the reinforcement layers between facing units to the front face of the wall. This connection must carry greater loads during seismic shaking and additional shear forces may be transmitted between modular block units [4]. Interface shear capacity developed between soil and geosynthetic reinforcement and connection capacity developed between the facing concrete blocks and the geosynthetic reinforcement should be known for the assessment of facing stability. The normal stress at the interface should be estimated since these two are dependent on the normal stress. The magnitude of normal stress acting at any specific interface depends on the height of the wall above that interface and the inclination of the wall [3].

Conventional design and analysis methods such as those recommended in guide-lines published by AASHTO [12], [13], the Federal Highway Administration [14] and the National Concrete Masonry Association (NCMA) [15], [16] recognize that the internal stability of the reinforced soil wall structure may be controlled by the mechanical performance of the modular unit-geosynthetic reinforcement connection [11].

The tensile load-deformation properties are influenced by as follows:

- geometry and type of geosynthetic-facing unit interface (i.e. continuous keys, lips, dowels or pins)
- properties of concrete
- whether the facing units are hollow or solid
- whether the hollow core is left empty or infilled with a granular soil
- tolerances on block dimensions
- quality of construction

- thickness, structure and polymer type of the geosynthetic [11].



- | | | |
|--------------------------|------------------------------|---|
| 1 masonry concrete block | 6 lateral restraining system | 11 spacers |
| 2 geogrid | 7 guide rail | 12 platform |
| 3 loading platen | 8 extensometer clamp | 13 wire-line extensometer |
| 4 gum rubber mat | 9 surcharge actuator | 14 computer-controlled hydraulic actuator |
| 5 roller clamp | 10 loading frame | 15 load cell |

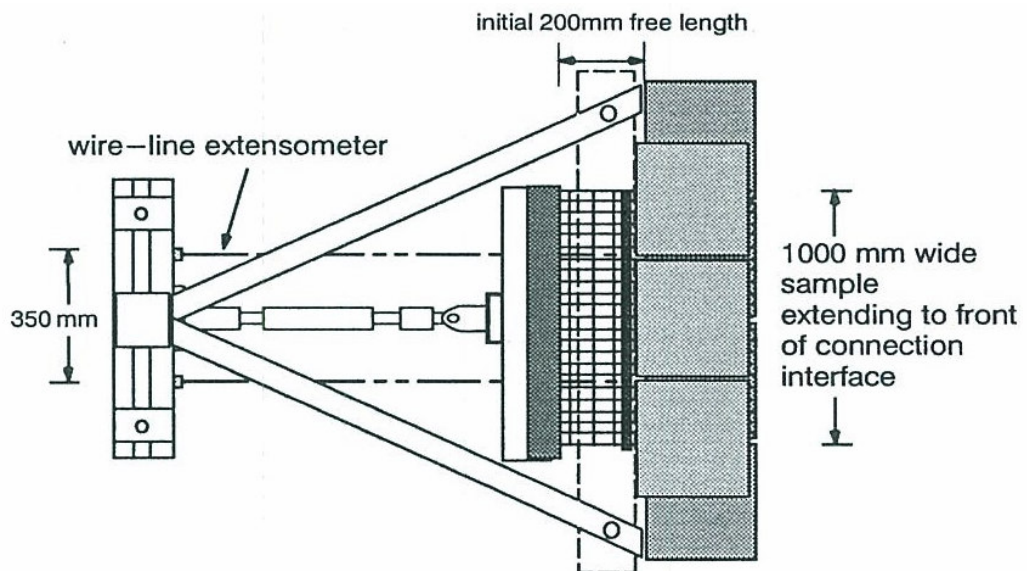


Figure 2.10. Schematic of pullout test apparatus showing typical masonry concrete block units and geogrid reinforcement [11]

At the Royal Military College (RMC) of Canada a large-scale apparatus has been constructed that allows connections up to 1m in width to be tested under stimulated wall heights as great as 10m. The test apparatus allows geotextile and geogrid reinforcement materials to be used. The test apparatus is illustrated in Figure 2.10. The method of test was adopted by the NCMA [15] as the protocol for concrete block-geosynthetic facing connection testing [11].

The test apparatus allows tensile loads to be applied to the geogrid while it is confined between two dry stacked masonry concrete unit layers. The modular concrete blocks are laterally restrained and surcharged vertically. The wall sections are constructed using the technique that is anticipated in the field [11].

A bottom row of modular block units is placed so that the interface elevation is coincident with the horizontal axis of the hydraulic actuator. Next, a strip of geogrid reinforcement 1000mm wide is attached to a rigid thick-walled 200mm diameter roller clamp and the reinforcement extended to the front of the interface between fascia units. The construction technique that is to be used in the field is adopted in the preparation of the simulated wall sections. A single layer of concrete units is placed over the connection interface. A heavy rigid steel box and plate section is used to transfer the vertical surcharge load to the top row of modular blocks. To ensure an even distribution of load to each concrete block unit a stiff gum rubber mat is placed between the loading platen and masonry units. An electronic load cell is attached to the vertical actuator piston to control applied surcharged loads. In this test apparatus the tensile load in the reinforcement is measured by a load cell located between the roller clamp and actuator piston [11].

Prior to the start of loading the actuator position is adjusted to ensure that the any slackness in the free length of reinforcement is removed. Each test is continued until there is a sustained loss in connection capacity. Failure may be due to:

- pullout of the reinforcement through the interface
- rupture of the reinforcement
- failure of the concrete units
- any combination of these mechanisms.

Following each test, the concrete units are removed and the reinforcement examined to confirm failure modes. A virgin sample of reinforcement is used for each test and damaged concrete units are replaced [11].

The typical variable in a test series is the magnitude of surcharge load (normal stress) applied to the connection interface. The surcharge pressures used in a series of connection tests are selected to cover the range of equivalent connection elevations in the proposed wall design [11].

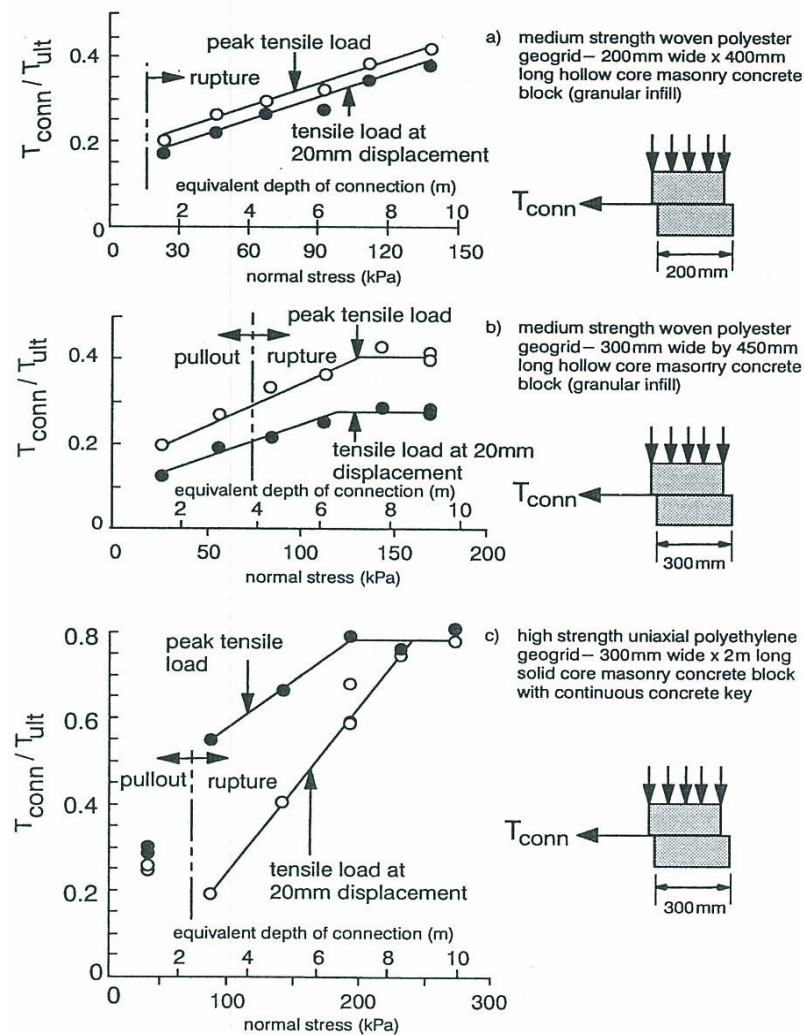


Figure 2.11. Example normalized connection strength curves [11]

The results from a series of tests are presented as connection strength (T_{conn}) versus normal stress envelopes that capture the trend in the data. It is often more useful for design purposes to present the results on a plot of normalized connection strength ($T_{\text{conn}}/T_{\text{ult}}$) versus equivalent depth of connection below the wall crest as displayed in Figure 2.11. [11]. As before, normalization is carried out with respect to the ultimate tensile strength (T_{ult}) of the reinforcement [17]. The experience of Bathurst, R. J., and M. R. Simac (1993) has been that, regardless of the choices of axes, the trend in the data is best represented by a linear or bi-linear curve with each segment of the curve established using a standard first-order linear regression. The magnitude of the ratio $T_{\text{conn}}/T_{\text{ult}}$ provides the designer with a practical relative measure of the efficiency of the connection system that can be used to trace the efficiency of the connection with wall elevation and to compare the efficiency of different connection systems [11].

The 1m wide sample adopted in Bathurst's test procedure has proven necessary to ensure that the reinforcement covers a reasonable number of vertical joints between block units. The majority of block units available in the market at the time of writing are 200 to 450mm in length (i.e. measured perpendicular to the direction of pull) [11].

The irregularities in the interface surface geometry that are introduced by the joints, edges, corners and small variations in dimensions between units have a major influence on the magnitude of tensile capacity available at the connection. Also reinforcement width has great influence on facing connection strength. The influence of sample width on nominal identical test configurations is illustrated on Figure 2.12. for typical dry cast masonry concrete units in combination with three different woven polyester geogrids. Figure 2.12. illustrates that tensile capacities based on either a displacement criterion of 20mm or peak load using a single stack of blocks may be 200 per cent greater than for the identical system constructed with 1m wide courses (i.e. 5 blocks and 4 running joints). The current GRI method of test [18] recommends a sample width of 200mm or the dimension of the facing element perpendicular to the direction of pull, whichever is less. Bathurst's results show that this standard would greatly over-estimate facing connection strengths for the example connections described here [11].

Variability in test results can be anticipated for nominal identical connection tests as a result of small variations in test setup, variations in the dimensions of masonry concrete blocks and laying out of the reinforcement. In addition, the connection capacity is sensitive to the quality of infill placement and compaction in the case of hollow core units [11].

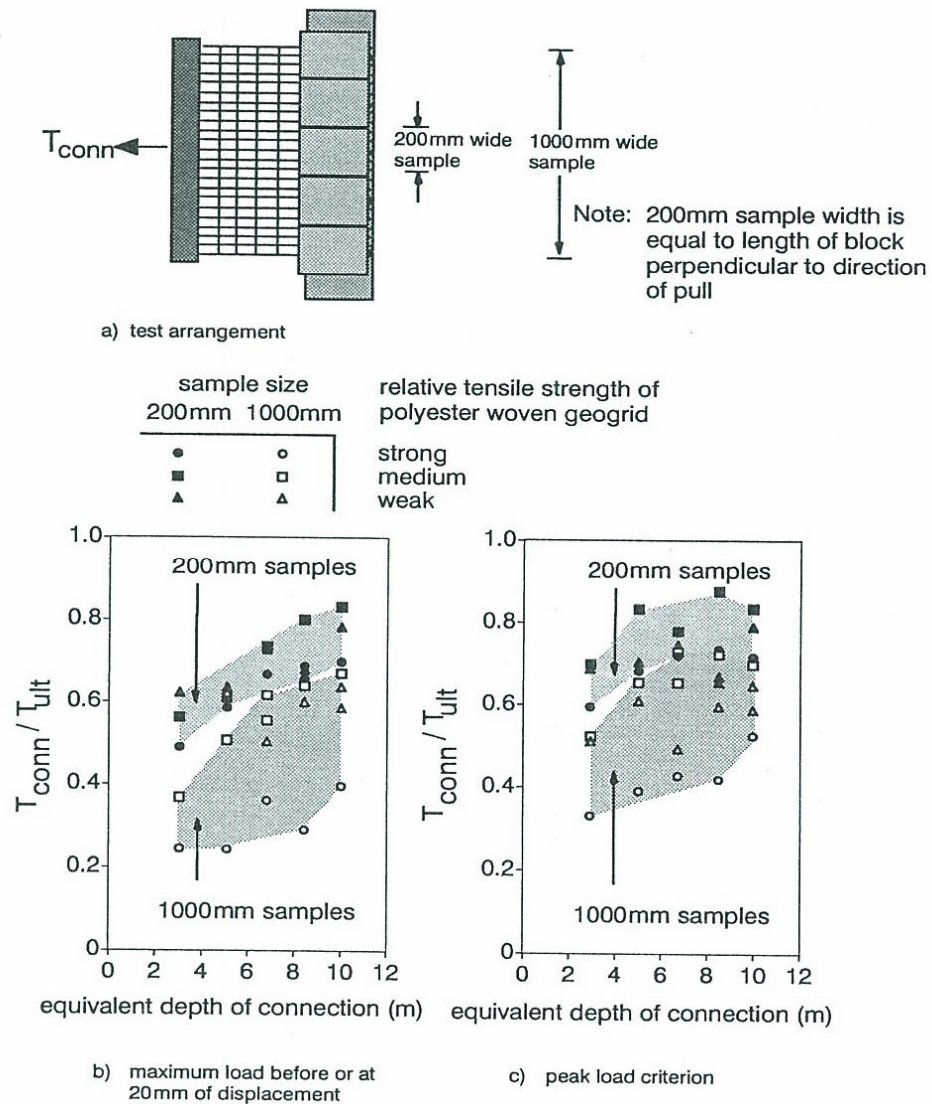


Figure 2.12. Influence of sample size on connection strength [11]

The results of more than 500 pullout connection tests carried out by Bathurst and Simac using a large number of masonry concrete unit types and geogrid reinforcement products has shown that the tests must be carried out over the full range of equivalent

connection elevations anticipated for the wall in the field. The data in Figure 2.11. illustrate that the trend in connection capacity for some block-geogrid combinations carried out with a maximum surcharge equivalent to a 6m deep connection may greatly over-estimate the connection strength at greater depths below the crest of the wall if the data is simply extrapolated. A possible explanation for the bi-linear trend in the data is that the geogrid reinforcement suffers mechanical damage when placed between concrete block units, particularly after large surcharge pressures have been applied. Mechanical damage to the geogrid occurs as a result of (typically) rough concrete surfaces, irregularities in concrete unit dimensions, joints between the blocks, and sharp edges. These conditions lead to pinching and crushing of the reinforcement during construction and result in uneven load transfer within the connection. The initial mechanical damage increases at higher confining pressures and prevents the further increase in connection capacity with depth that might otherwise be anticipated for an essentially frictional connection [11].

ASTM D 6638-07 standard introduces a similar test apparatus and test method referred by Bathurst and Simac (1993). This test method is used to determine the connection properties between a layer of geosynthetic reinforcement and segmental concrete block units used in construction of reinforced soil retaining walls. The test is carried out under conditions determined by the user that reproduce the connection system at full-scale. The results of a series of tests are used to define a relationship between connection strength for a segmental unit-geosynthetic connection system and normal load [19].

This is a performance test used to determine properties for design of retaining wall systems utilizing segmental concrete units and soil reinforcing geosynthetics, either geotextiles or geogrids. The test is performed on a full-scale construction of the connection and may be run in a laboratory or the field [19].

One end of a wide geosynthetic reinforcement test specimen is attached to dry stacked segmental concrete block units assembled as specified by the user. The other end of the test specimen is attached to a clamp, which is part of a constant rate of extension tensile loading machine. The top course of segmental concrete block units is then loaded vertically to a constant normal load and the geosynthetic is then tensioned under constant

rate of displacement until a sustained loss of connection capacity and/or excessive movement (greater than 150 mm) of the reinforcement out from the connection [19].

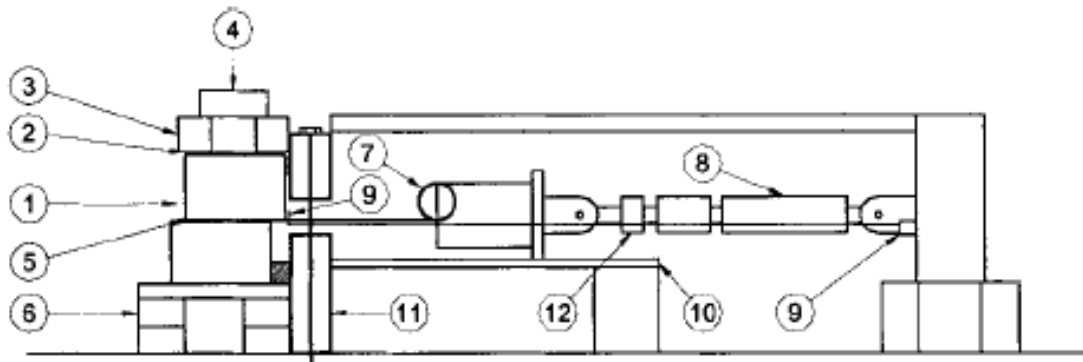


Figure 2.13. Connection strength test apparatus [19]

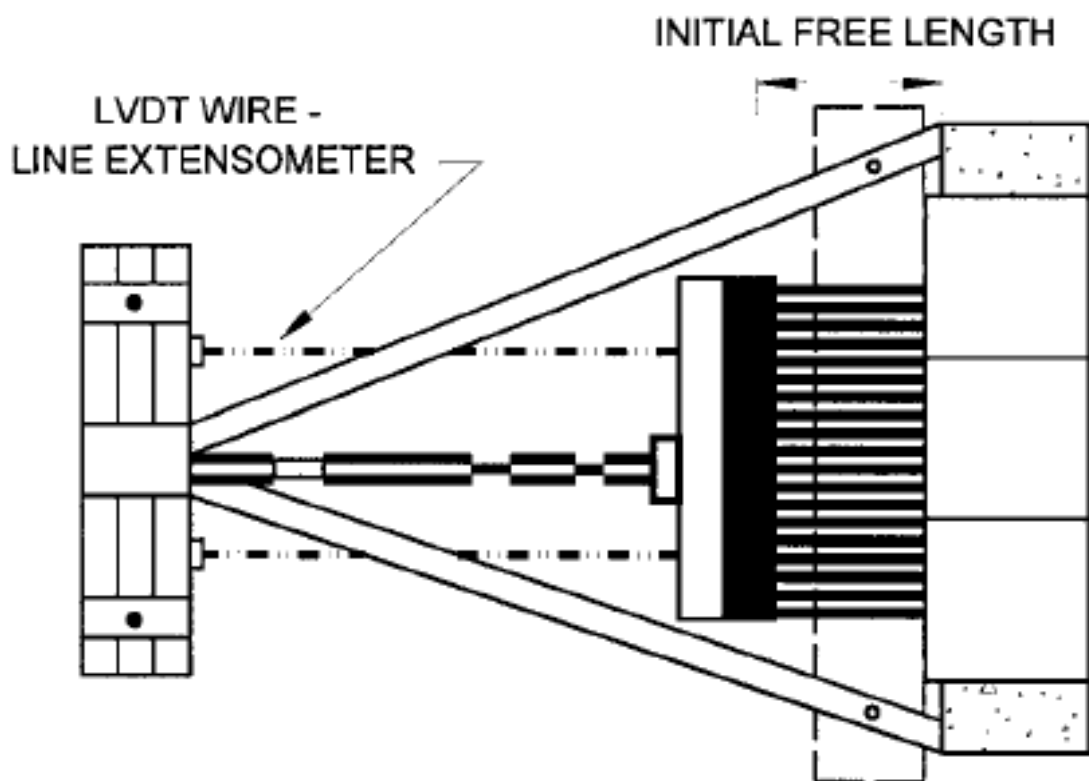


Figure 2.14. Connection strength test apparatus (Plan view) [19]

An example of a test apparatus and setup is illustrated in Figure 2.13. and 2.14. The principal components of the test apparatus are loading frame, normal load piston/actuator, vertical loading platen with stiff rubber mat or airbag, vertical load cell, geosynthetic loading clamp, horizontal piston/actuator, horizontal load cell, and two horizontal displacement measurement devices [19].

The orientation of the tensioning force shall be horizontal and perpendicular to the back of the segmental units and shall be applied at the elevation where the geosynthetic exits the back of the segmental units [19].

Peak connection capacity, and tensile capacity after a user prescribed displacement criteria has occurred, is used to define connection strength based on peak and service state criteria respectively. Both these values may be obtained from each test that measures geosynthetic displacement. Generally a series of tests are performed to establish a mathematical relationship between connection strength and normal load on the connection [19].

The connection strength between geosynthetic reinforcement and segmental concrete block units is used in design of geosynthetic reinforced soil retaining walls [19].

This test is used to determine the connection strength for the design of the connection system formed by segmental concrete block units and geosynthetic reinforcement layers in reinforced soil retaining walls. Performing a series of these connection tests at varying normal loads permits development of a relationship between connection strength and normal load. This relationship may be linear, bi-linear, or some other complex mathematical expression [19].

In this test method the geosynthetic reinforcement test specimen shall be a minimum of 750 mm in width. For tests that use two or more full segmental retaining wall units on the bottom course, the geosynthetic shall be an exact multiple of the segmental retaining wall unit width totaling closest to, but exceeding 750 mm in width. For segmental retaining wall unit widths greater than 500 mm a geosynthetic specimen width of 1000 mm may be used [19].

The wall for testing shall be constructed to a minimum of 750 mm in width and contain at least one typical segmental concrete unit running bond joint. The segmental wall width for testing shall be at least as wide as the geosynthetic test specimen width. Testing of segmental concrete unit widths greater than 500 mm, may be represented in this test by limiting the test wall to 1000 mm in width [19].

The geosynthetic specimen shall have sufficient length to cover the interface surface as specified by the user. The specimen must be trimmed to provide sufficient anchorage at the geosynthetic loading clamp and a free length between the back of the concrete blocks and loading clamp ranging from a minimum of 200 mm to a maximum of 600 mm. The geosynthetic reinforcement specimen shall be placed between the stacked segmental concrete units to cover the same area that will be used in field construction of the connection or as determined by the user. A new geosynthetic reinforcement test specimen shall be used for each test [19].

A single course of segmental units shall be placed on a rigid base. A second course of segmental units will later be placed over the bottom course of units, with the geosynthetic reinforcement located and placed between these courses as described by the user or in the same manner anticipated for field construction. Both courses of segmental concrete units shall be rigidly braced to prevent lateral movement of the units during geosynthetic tension testing [19].

The minimum width of the top course of segmental concrete units shall be 750 mm and shall be fully supported by the bottom course. Reducing the width of segmental concrete units by cutting with a concrete/masonry saw is permissible, provided that the cut (rough) edges are located beyond the edge of the geosynthetic sample [19].

The test shall be carried on until there is a sustained loss of tensile resistance recorded at the loading clamp due to failure of the reinforcement at or within the connection system and/or failure of the blocks. In some cases the failure will be defined as excessive displacement or slippage of the reinforcement in the connection without a sustained loss of tensile resistance. Failure or slippage of the geosynthetic within the loading clamp constitutes an invalid test [19].

The results of facing connection testing shall be summarized on a plot as indicated in Figure 2.15. The triangle points on the plot display connection strength (based on peak load criterion) versus normal load, and the other cross points on the plot show connection strength (based on 0.75 inches displacement criterion) versus normal load.

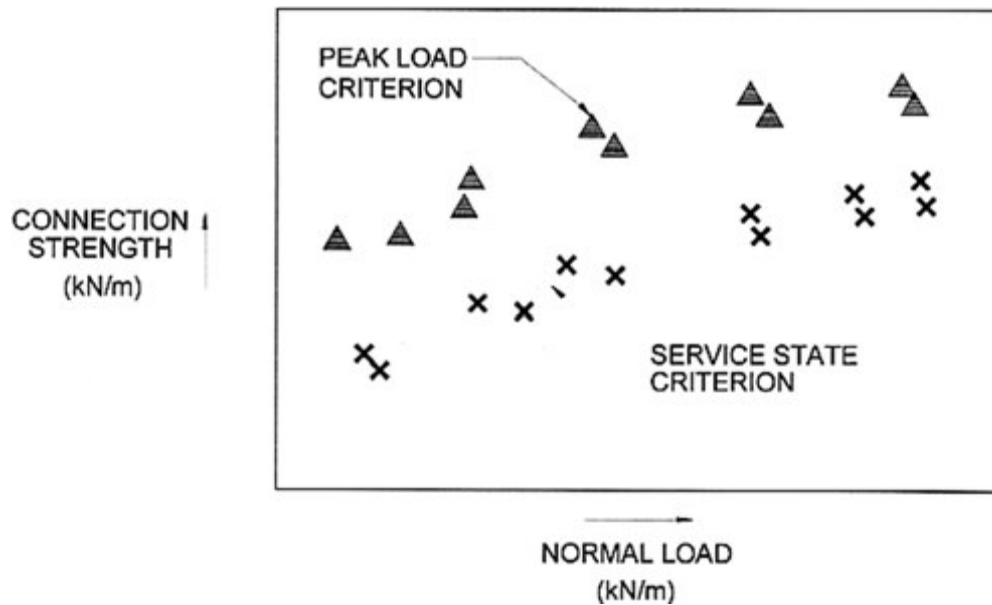


Figure 2.15. Connection strength vs. normal load [19]

2.5. Segmental Retaining Wall Construction

The success of any segmental retaining wall installation depends on complete and accurate field information, careful planning and scheduling, the use of specified materials, proper construction procedures, and inspection. The basic steps taken in the construction of segmental retaining walls are as follows:

- Excavation and leveling pad construction,
- Setting, leveling, and backfilling base course,
- Placement and backfilling of units in succeeding courses,
- Placement, straightening, and backfilling of geosynthetic reinforcement,

- Compaction of backfill to the specified density,
- Capping and finish grading.

As with any structure used to retain soil, careful attention should be paid to the compaction equipment and procedures used during construction. When compacting soil within 0.9m of the front face of a wall, compaction tools should be limited to hand operated equipment, preferably a vibrating plate compactor. Reinforced soil can be compacted with walk-behind or self-propelled riding compaction equipment [3].

The facing blocks generally used in the construction of segmental retaining walls in Turkey are rectangular blocks with two holes inside. These blocks are internationally standardized building blocks. The characteristics of these blocks will be explained in detail in Chapter 3.

Generally a coarse granular soil is poured into the holes of these blocks. The soil inside the holes is manually compacted and the soil remaining on the surface of blocks is cleaned. The area behind the layer of blocks is filled with the soil available at site up to the same height and compacted with a compaction cylinder. Geosynthetic with the appropriate width according to the design is spread over the concrete blocks and the compacted soil. Geosynthetic reinforcements used in geosynthetic reinforced soil retaining walls are usually geogrids and geotextiles. In Turkey the geosynthetic used is nearly always a woven geotextile. The next layer of blocks is placed over the first layer. Sometimes this layer is shifted one centimeter backwards to provide a more pleasing appearance. These blocks are also filled with the same granular material. The fill in the blocks is compacted, the area behind the blocks is filled and compacted and the geotextile is placed as in the first layer. The wall is completed in this way [3].

The holes of building blocks are slightly wider on one side. Sometimes sand is placed between the geotextile and the concrete blocks for adjusting the alignment of blocks. The top two rows of blocks are filled with mortar. Geotextile reinforcement is used for each layer. This facing system utilizes only the frictional forces. No shear key or pin is used for connection [3].

3. METHODOLOGY

3.1. General

In Chapter 2, geosynthetic reinforced soil retaining walls with concrete block facing elements were discussed. In order to investigate the frictional behavior between modular concrete blocks and geosynthetic reinforcements a total of 27 connection strength tests were executed. The modular concrete blocks were either left empty or filled with different types of soil. As reinforcement a woven geotextile and two types of geogrids were used. An angle of friction was calculated in each test from the relationship between the normal load applied to geosynthetic reinforcements and tensile load at failure point. The photographs illustrating these experiments with different conditions can be found in Appendix A.

3.2. Connection Strength Test

3.2.1. Materials

“Tech Block” is widely used in the construction of geosynthetic reinforced soil retaining walls in Turkey as modular concrete blocks. Modular concrete blocks (Tech Block) were used in the connection tests. The properties of the modular concrete blocks (Tech Block) are shown in Table 3.1. as provided by “Eko Modüler Yapı Malzemeleri”.

Tech Block has two holes that are larger on the top surface and get smaller towards the bottom surface. A photograph of Tech Block is shown in Figure 3.1.

Three different geosynthetic reinforcements a geotextile, a biaxial geogrid, and a uniaxial geogrid were used in connection tests. The first reinforcement was woven polypropylene geotextile with 44 kN/m tensile strength. The physical and mechanical properties of geotextile are shown in Table 3.2. A photograph of the geotextile is displayed in Figure 3.2.



Figure 3.1. Picture of concrete blocks (Tech Block) used in connection tests

Table 3.1. Physical and mechanical properties of Tech Block [20]

| Property | Value for Tech Block |
|---|-------------------------------------|
| Type | Tech Block Standart Tam Eleman 1111 |
| Weight | between 17-18.5 kg |
| Density | 2.4 gr/cm ³ |
| Dimensions | 39 x 19 x 19 cm |
| Hole dimensions | 12 x 13.6 cm |
| Sound isolation | 52 dB |
| Impact isolation | 26 dB |
| Water absorption | 5 per cent |
| Heat isolation coefficient | 0.33 m ² K/W |
| Fire resistance | 2 hours |
| 28-day compressive strength (considering the net surface area) | 180 kgf/cm ² |

Table 3.2. Physical and mechanical properties of geotextile used in connection tests

| Product Characteristics | Value for the Geotextile |
|--------------------------------|--|
| Tensile strength | 44 kN/m (-4 kN/m) |
| Elongation | 15 % (± 4.5 %) |
| Dynamic perforation resistance | 9 mm (+2 mm) |
| Resistance to static puncture | 5600 N (-550 N) |
| Opening size | 380 μm (± 95 μm) |
| Water permeability | 34×10^{-3} m/s (-9×10^{-3} m/s) |

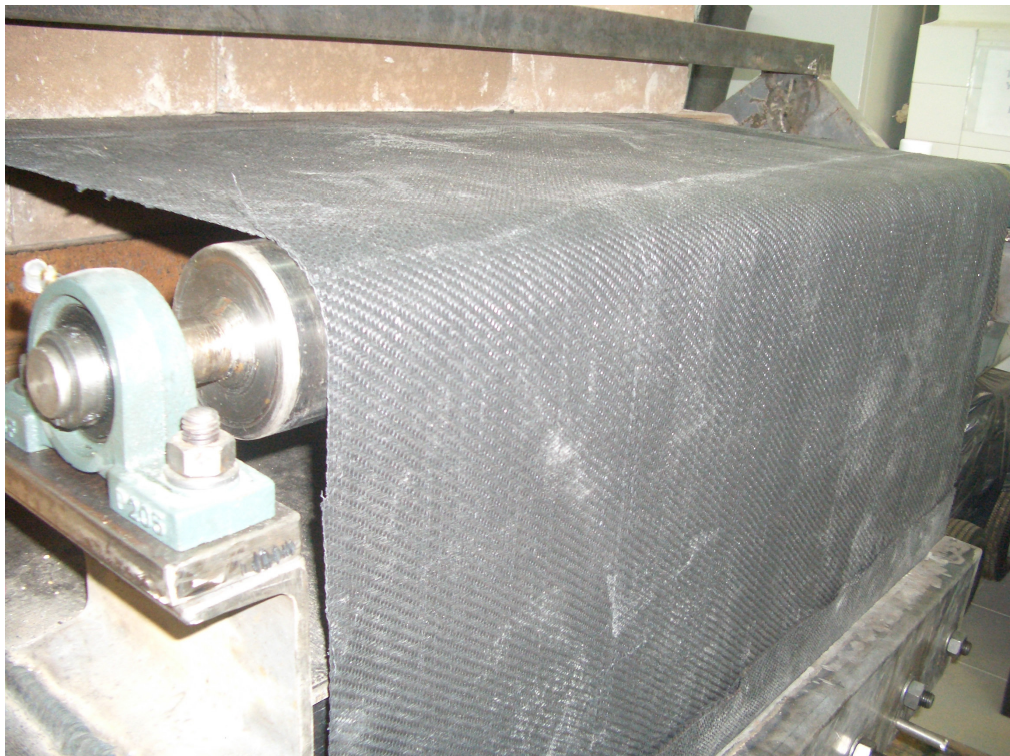


Figure 3.2. Picture of the geotextile used in connection tests

This woven polypropylene geotextile can be used in the construction of roads and other trafficked areas, railways, earthworks, foundations, retaining structures, reservoirs, dams and canals, and erosion control works (coastal protection and bank revetments).

The first geogrid used in the connection tests was a biaxial geogrid with 55 kN/m tensile strength. This geogrid was made from polyester and coated with PVC. The physical and mechanical properties of Geogrid I are exposed in Table 3.3. and a photograph of Geogrid I is shown in Figure 3.3. This biaxial geogrid can be used in the construction of roads and other trafficked areas, earthworks, foundations, and retaining structures.

Table 3.3. Physical and mechanical properties of Geogrid I used in connection tests

| Product Characteristics | Value for Geogrid I |
|--------------------------------|----------------------------|
| Mesh dimensions | 40 x 40 mm |
| Mass per unit area | 300 g/m ² |
| Tensile strength MD | ≥ 55 kN/m |
| Tensile strength CD | ≥ 55 kN/m |
| Elongation at maximum load MD | 12 % (±2 %) |
| Elongation at maximum load CD | 12 % (±2 %) |

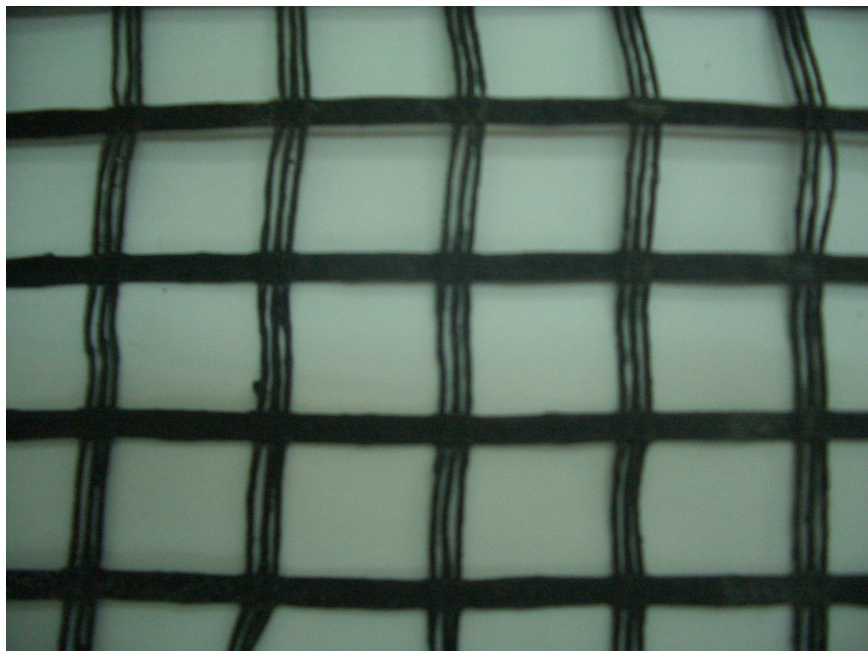


Figure 3.3. Picture of Geogrid I (biaxial geogrid) used in connection tests

The second geogrid used in the connection tests was a uniaxial geogrid with 45 kN/m tensile strength. The raw material of this uniaxial geogrid was polypropylene. The physical and mechanical properties of Geogrid II are represented in Table 3.4. The dimensions of Geogrid II are illustrated in Figure 3.4. and a photograph of Geogrid II is indicated in Figure 3.5.

Table 3.4. Physical and mechanical properties of Geogrid II used in connection tests

| Product Characteristics | Value for Geogrid II |
|--|----------------------|
| Tensile strength (longitudinal) | 45 kN/m |
| Tensile strength (transversal) | 10 kN/m |
| Elongation at maximum load (longitudinal) | 10 % |
| Elongation at maximum load (transversal) | 13 % |
| Unit weight | 390 g/m ² |
| Distance between two longitudinal ribs (b) | 14 mm |
| Distance between two transversal ribs (a) | 80 mm |
| Thickness of longitudinal ribs (t ₂) | 0.90 mm |
| Thickness of junctions (t ₁) | 2 mm |
| Width of longitudinal ribs (c) | 5 mm |
| Width of transversal ribs (d) | 10 mm |

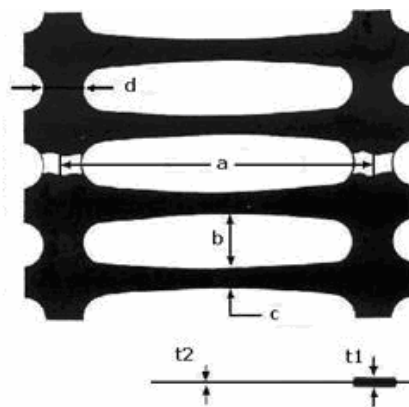


Figure 3.4. Dimensions of Geogrid II [21]



Figure 3.5. Picture of Geogrid II (uniaxial geogrid) used in connection tests

This uniaxial polypropylene geogrid can be used in the construction of roads and other trafficked areas, earthworks, foundations, retaining structures, drainage canals, and erosion control works.

Width of geosynthetic reinforcement specimens were 100 cm in all connection tests. Geogrid I (biaxial geogrid) has 27 longitudinal ribs in 100 cm width while Geogrid II (uniaxial geogrid) has 52 longitudinal ribs in 100 cm width. The length of the all geosynthetic reinforcement specimens were 140 cm. All of the specimens were cut carefully in order not to affect the test results.

Two different soils were used for filling the holes of the modular concrete blocks, sand and gravel. The properties of the soils used in connection tests are given in Table 3.5. Data for mechanical grain size analysis of sand, grain size distribution curve for sand, data for mechanical grain size analysis of gravel, grain size distribution curve for gravel are given in Table 3.6, Figure 3.6, Table 3.7, and Figure 3.7, respectively.

Table 3.5. Properties of soils used in tests

| Properties of Soils | Sand | Gravel |
|--|-------------|---------------|
| Unit weight, γ (kN/m ³) | 15.8 | 16.2 |
| Specific gravity of solid constituents, G | 2.59 | 2.72 |
| Uniformity coefficient, C_u | 3.23 | 3.97 |
| Coefficient of curvature, C_c | 1.32 | 1.04 |

Table 3.6. Data for mechanical grain size analysis of sand

| Sieve diameter (mm) | Weight of sieve | Weight of sieve + soil | Weight retained | Per cent retained | Per cent passing |
|--------------------------------|----------------------------|-----------------------------------|----------------------------|------------------------------|-----------------------------|
| 9.52 | 461.12 | 461.12 | 0 | 0 | 100 |
| 4.75 | 441.02 | 469.60 | 28.58 | 2.77 | 97.23 |
| 2.00 | 405.80 | 580.85 | 175.05 | 17.00 | 80.23 |
| 1.18 | 403.91 | 523.91 | 120.00 | 11.65 | 68.58 |
| 0.43 | 382.15 | 931.75 | 549.60 | 53.36 | 15.22 |
| 0.21 | 368.36 | 492.39 | 124.03 | 12.04 | 3.18 |
| 0.15 | 356.27 | 366.41 | 10.14 | 0.98 | 2.20 |
| 0.075 | 356.83 | 378.84 | 22.01 | 2.14 | 0.06 |
| (Pan) | 262.64 | 267.27 | 4.63 | | |
| TOTAL | | | 1034.04 | | |
| Measured weight = | | | 1030 (OK) | | |

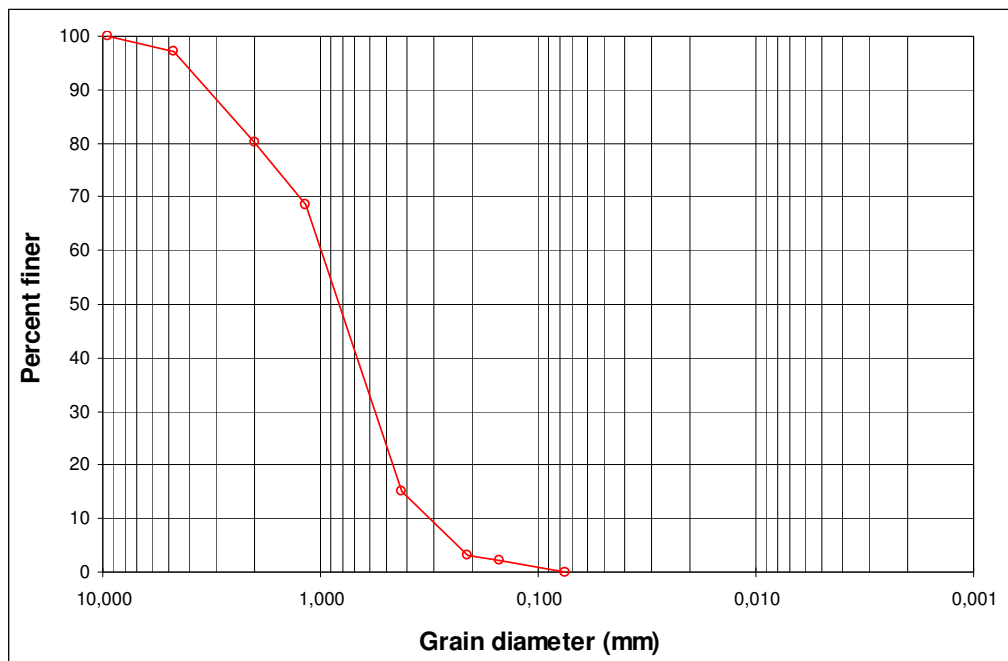


Figure 3.6. Grain size distribution curve for sand

Table 3.7. Data for mechanical grain size analysis of gravel

| Sieve diameter (mm) | Weight of sieve | Weight of sieve + soil | Weight retained | Per cent retained | Per cent passing |
|---------------------|-----------------|------------------------|-----------------|-------------------|------------------|
| 16 | 441.37 | 652.34 | 210.97 | 24.25 | 75.75 |
| 4.75 | 441.21 | 916.87 | 475.66 | 54.67 | 21.08 |
| 2.00 | 406.80 | 568.94 | 162.14 | 18.64 | 2.44 |
| 1.18 | 404.30 | 410.22 | 5.92 | 0.68 | 1.76 |
| 0.43 | 382.73 | 387.61 | 4.88 | 0.56 | 1.20 |
| 0.21 | 370.70 | 372.29 | 1.59 | 0.18 | 1.02 |
| 0.15 | 356.65 | 357.47 | 0.82 | 0.09 | 0.93 |
| 0.075 | 351.58 | 358.54 | 6.96 | 0.80 | 0.13 |
| (Pan) | 259.65 | 263.48 | 3.83 | | |
| TOTAL | | | 872.77 | | |
| Measured weight = | | | 870 (OK) | | |

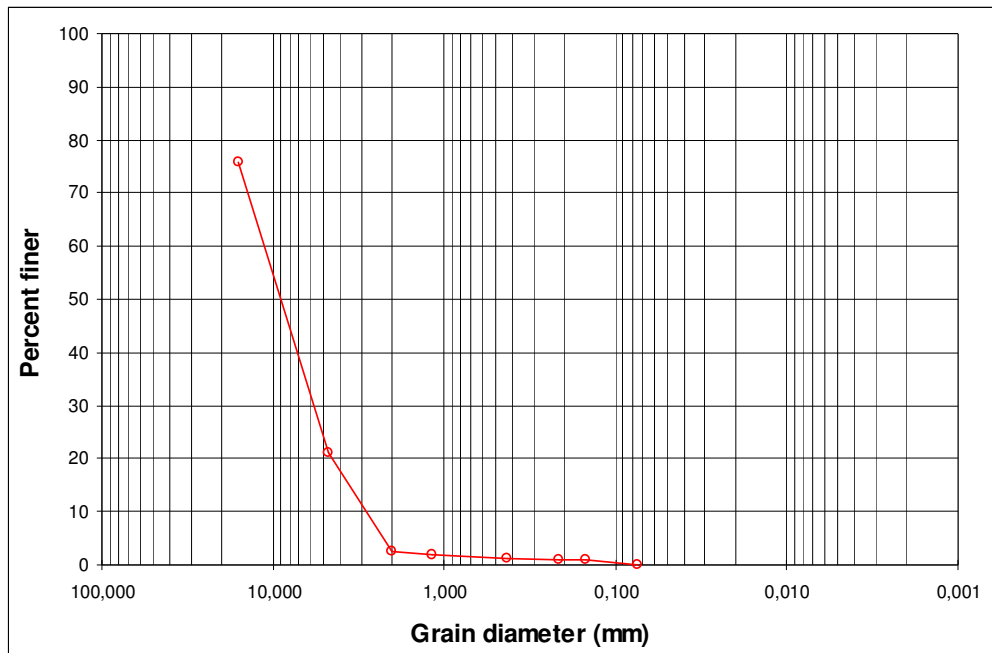


Figure 3.7. Grain size distribution curve for gravel

3.2.2. Apparatus

The angles of friction between concrete blocks and three different geosynthetic reinforcements under different conditions were calculated with the help of the testing apparatus. The lowest part of the apparatus was a special tray fixed on the testing table. The length of the tray was 160 cm while its width was 80 cm. A confining piece was fixed to the tray in order to prevent the movement of the lower layer of concrete blocks. The upper layer of concrete blocks was fixed by another confining piece. These two confining pieces were welded to the tray. The schematic of the apparatus can be seen in Figure 3.8.

The roller was free to rotate around an axle connected to the tray on both sides. The length of the roller was 110 cm. The tension load was applied by placing weights on three hooks hanging from a steel clamp attached to the reinforcements. The clamp was specially designed to ensure a uniform load distribution to the geosynthetic reinforcement. The steel clamp is indicated in Figure 3.9. It consisted of two pieces with channels made on the inside faces for placing a steel rod to fix the geosynthetic reinforcements and screws to

tighten the clamp. The total weight of the clamp, the steel rod, and three hooks was 31.3 kg.

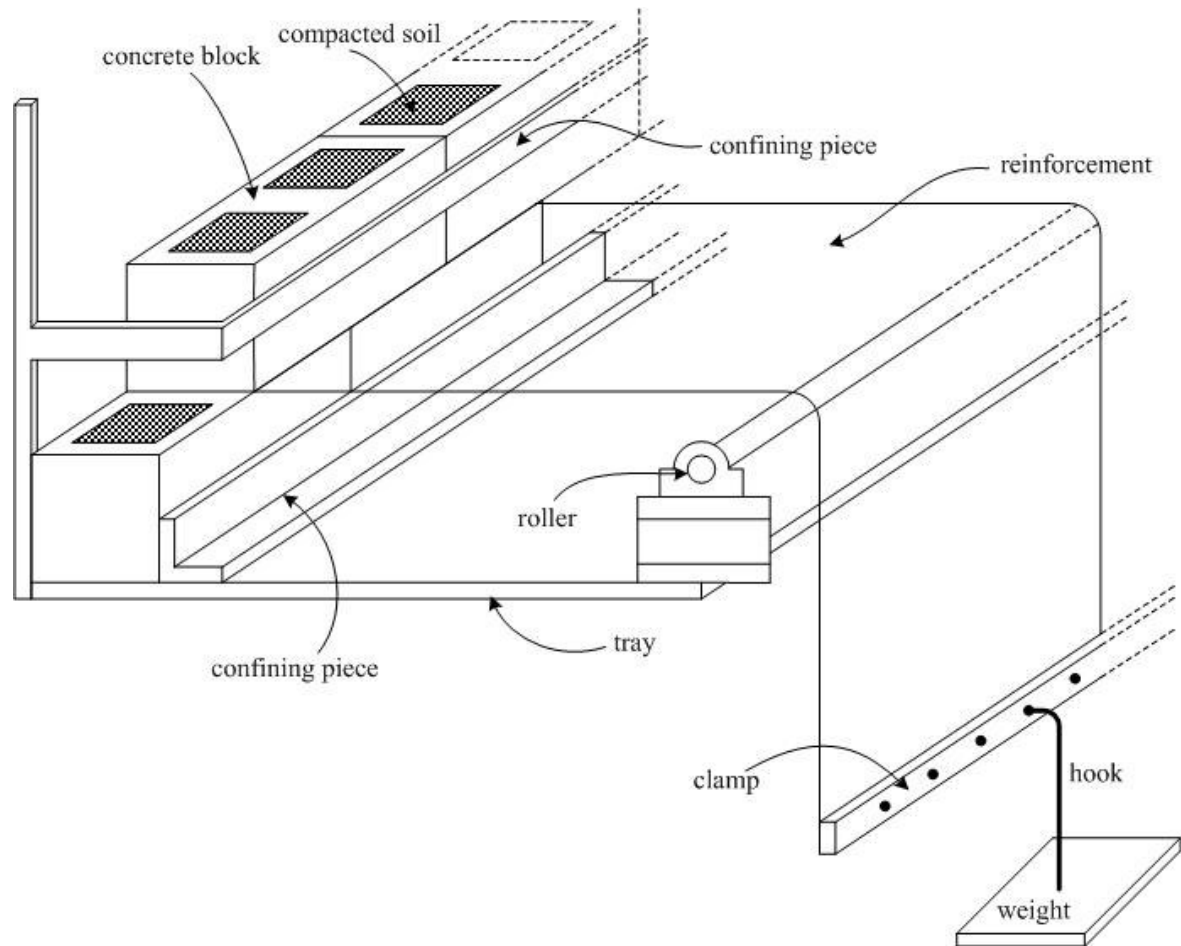


Figure 3.8. Testing apparatus

The length, width and depth of the clamp were 100 cm, 18 cm, and 10 cm respectively. The clamp had seven holes for screws. Screws were tightened after geosynthetic reinforcement was rolled around the steel rod and placed between the two sides of the clamp. The diameter of the screws was 20 mm. Under the alignment of the seven screw holes, there were three holes for hooks to suspend the weights. The diameter of the hooks was 12 mm. There were disc plates at the bottom of the hooks to place the weights and they were welded to the hook in order to tolerate heavy loads. The diameter of the steel rod between the two sides of the clamp was 50 mm.

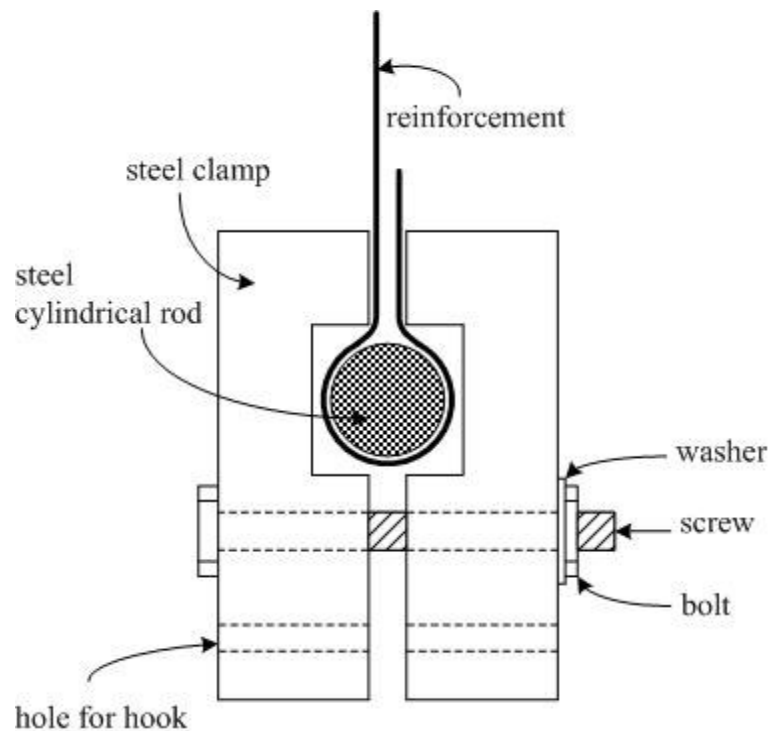


Figure 3.9. Cross-section of the steel clamp

3.2.3. Setup and Testing

27 tests were executed with different types of infill, different types of geosynthetic reinforcements, and number of concrete block layers on the geosynthetic reinforcement. 27 tests were divided into nine groups according to type of infill and type of geosynthetic reinforcement. The classification of 27 tests is shown in Table 3.8.

The four concrete blocks (Tech Block) were placed side by side to constitute the first layer of concrete blocks on the tray. The surface with the wider holes was on the top when all concrete blocks were placed on the testing apparatus. Care was given to place the blocks always in the same direction in order to eliminate the block orientations as a variable. The tests were performed either with the holes left empty or filled with sand, or gravel. The infill was compacted using a hand rammer. One end of geosynthetic reinforcement sample was laid on the block and the second layer of concrete blocks was placed eccentrically as seen in Figure 3.8. There were three blocks at the second layer of concrete blocks.

Table 3.8. The classification of 27 tests

| Test No | Infill | Reinforcement | Number of concrete block layers on reinforcement |
|----------------|---------------|----------------------|---|
| 1 | Empty | Geotextile | 1 |
| 2 | Empty | Geotextile | 2 |
| 3 | Empty | Geotextile | 3 |
| 4 | Empty | Geogrid I | 1 |
| 5 | Empty | Geogrid I | 2 |
| 6 | Empty | Geogrid I | 3 |
| 7 | Empty | Geogrid II | 1 |
| 8 | Empty | Geogrid II | 2 |
| 9 | Empty | Geogrid II | 3 |
| 10 | Sand | Geotextile | 1 |
| 11 | Sand | Geotextile | 2 |
| 12 | Sand | Geotextile | 3 |
| 13 | Sand | Geogrid I | 1 |
| 14 | Sand | Geogrid I | 2 |
| 15 | Sand | Geogrid I | 3 |
| 16 | Sand | Geogrid II | 1 |
| 17 | Sand | Geogrid II | 2 |
| 18 | Sand | Geogrid II | 3 |
| 19 | Gravel | Geotextile | 1 |
| 20 | Gravel | Geotextile | 2 |
| 21 | Gravel | Geotextile | 3 |
| 22 | Gravel | Geogrid I | 1 |
| 23 | Gravel | Geogrid I | 2 |
| 24 | Gravel | Geogrid I | 3 |
| 25 | Gravel | Geogrid II | 1 |
| 26 | Gravel | Geogrid II | 2 |
| 27 | Gravel | Geogrid II | 3 |

The other end of the geosynthetic reinforcement was attached to the steel clamp as seen in Figure 3.9. It was wrapped around the steel rod. This still rod was placed in the channel existing between the right and left parts of the clamp and the screws were tightened carefully. The cross-section of connection between the two layers of concrete block and uniaxial geogrid can be seen in Figure 3.10.

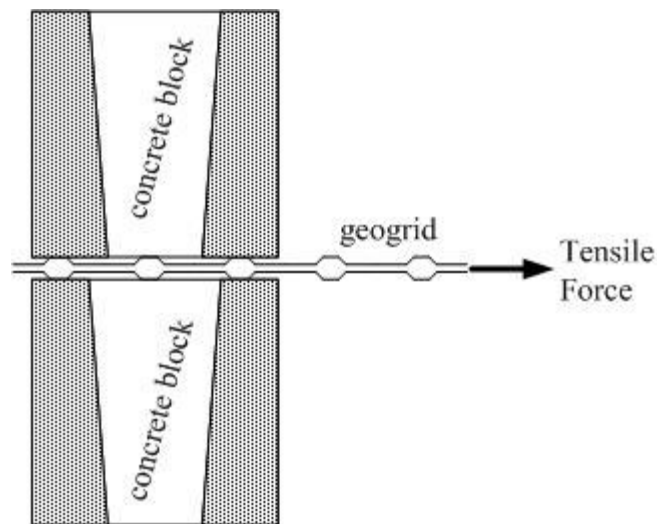


Figure 3.10. The cross-section of connection between concrete blocks and uniaxial geogrid

The infill soil of the second layer concrete blocks was placed and compacted with a hand rammer. The lower confining piece prevented the horizontal movement of the first layer of concrete blocks as the upper confining piece restrained the second layer of concrete blocks. If needed, new layers of concrete blocks were added and the number of concrete block layers on the geosynthetic reinforcement increased. This procedure provided more normal load in order to be able to get more tensile load. Each layer was filled with compacted soil. Also infill soil contributed to get more normal load. The weight of each concrete block was measured individually to find the normal load acting on geosynthetic reinforcement precisely.

The average weights of sand and gravel put in one hole of concrete blocks were calculated before the execution of tests. The average weights of sand and gravel as infill soil are given in Table 3.9.

Table 3.9. Average weights of infill soils put in one hole of concrete blocks

| Infill Soil | Main Hole | Hole between two side by side concrete blocks |
|-------------|-----------|---|
| Sand | 3.825 kg | 1.395 kg |
| Gravel | 3.936 kg | 1.440 kg |

An initial weight of 10 kg was placed on the hook. This weight was increased until failure. The load at failure, type of failure, and any interesting points were recorded.

3.2.4. Calculations

In this study Mohr-Coulomb failure criterion was used to calculate angle of friction for each case. In this criterion normal stress and tensile stress data are needed. Failure from shear will occur when the shear stress on a plane reaches Mohr-Coulomb failure envelope.

$$\tau_f = c + \sigma \tan \lambda \quad (3.1)$$

where:

- c = cohesion
- λ = angle of friction
- σ = normal stress on the failure plane
- τ_f = shear strength

Cohesion was neglected during calculations. The friction on both sides of geosynthetic reinforcements was taken into consideration in calculation of the angles of friction.

$$\arctan\left(\frac{\tau_f}{\sigma}\right) = \lambda \quad (3.2)$$

4. RESULTS

4.1. General

In each test, the failure load on the reinforcement is plotted against the normal load on the geosynthetic reinforcement. An angle of friction is calculated by the help of tangent of slope obtained from the shear stress vs. normal stress graph in each group of tests. The friction on both sides of geosynthetic reinforcements is taken into consideration in calculation of the angles of friction. In this study calculations are made by utilizing Equation 3.2. The results are given in the following sections.

4.2. Combination of Empty Blocks and Geotextile

The first group of tests was the constitution of the first, second and third tests which were conducted with the combination of empty blocks and geotextile. The results are shown in Table 4.1. and these results are plotted in Figure 4.1.

Table 4.1. Connection test results using empty blocks with geotextile used as reinforcement

| Empty Blocks – Geotextile | | | | | |
|---------------------------|--------------------------------|--|---|--|---|
| Test No | Number of blocks on Geotextile | Total normal load on Geotextile (kg/m) | Tensile load applied to Geotextile (kg/m) | Total normal stress on Geotextile (kN/m ²) | Shear stress applied to Geotextile (kN/m ²) |
| 1 | 1 | 45.265 | 77.3 | 2.337 | 1.996 |
| 2 | 2 | 89.060 | 161.3 | 4.598 | 4.164 |
| 3 | 3 | 134.205 | 221.3 | 6.929 | 5.713 |

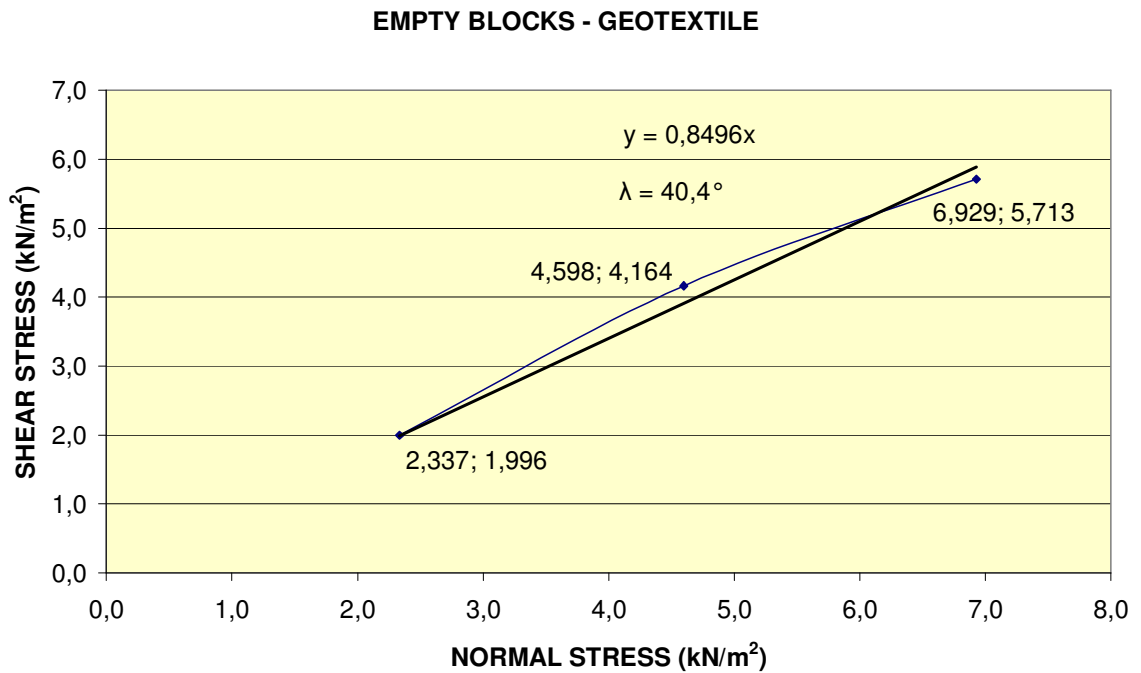


Figure 4.1. Plot of test results with holes left empty and geotextile used as reinforcement

4.3. Combination of Empty Blocks and Geogrid I

The second group of tests was the constitution of fourth, fifth and sixth tests which were conducted with the combination of empty blocks and Geogrid I. The results are shown in Table 4.2. and these results are plotted in Figure 4.2.

Table 4.2. Connection test results using empty blocks with Geogrid I used as reinforcement

| Empty Blocks – Geogrid I | | | | | |
|--------------------------|-------------------------------|---------------------------------------|--|---|--|
| Test No | Number of blocks on Geogrid I | Total normal load on Geogrid I (kg/m) | Tensile load applied to Geogrid I (kg/m) | Total normal stress on Geogrid I (kN/m ²) | Shear stress applied to Geogrid I (kN/m ²) |
| 4 | 1 | 44.735 | 120.3 | 2.310 | 3.106 |
| 5 | 2 | 89.900 | 196.3 | 4.642 | 5.068 |
| 6 | 3 | 135.043 | 229.3 | 6.972 | 5.920 |

EMPTY BLOCKS - GEOGRID I

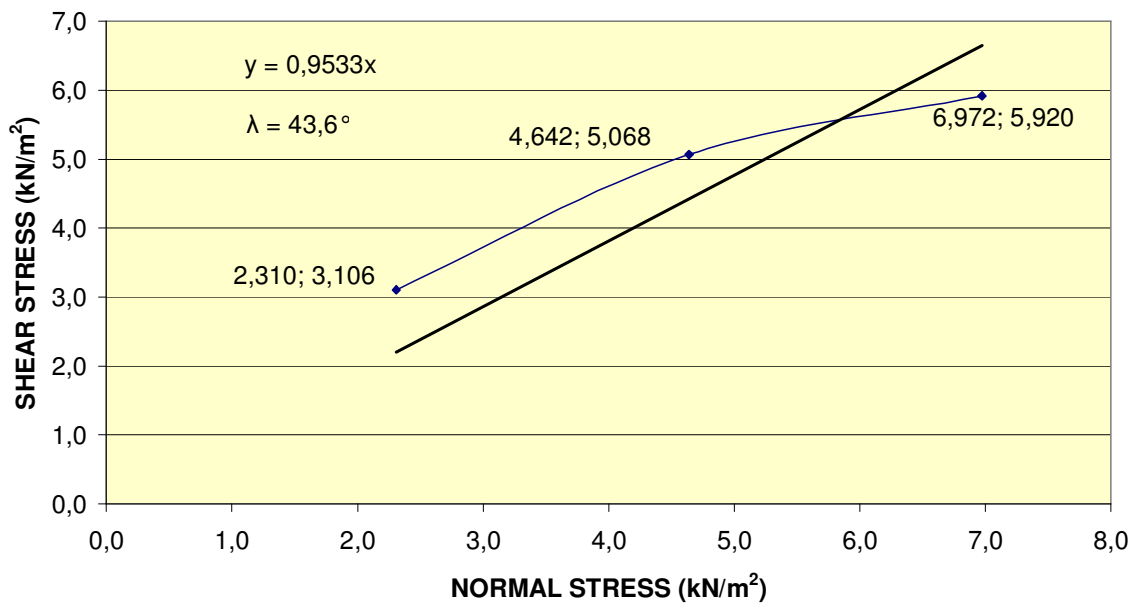


Figure 4.2. Plot of test results with holes left empty and Geogrid I used as reinforcement

4.4. Combination of Empty Blocks and Geogrid II

The third group of tests was the constitution of seventh, eighth and ninth tests which were conducted with the combination of empty blocks and Geogrid II. The results are shown in Table 4.3. and these results are plotted in Figure 4.3.

Table 4.3. Connection test results using empty blocks with Geogrid II used as reinforcement

| Empty Blocks – Geogrid II | | | | | |
|---------------------------|--------------------------------|--|---|--|---|
| Test No | Number of blocks on Geogrid II | Total normal load on Geogrid II (kg/m) | Tensile load applied to Geogrid II (kg/m) | Total normal stress on Geogrid II (kN/m ²) | Shear stress applied to Geogrid II (kN/m ²) |
| 7 | 1 | 45.436 | 71.3 | 2.346 | 1.841 |
| 8 | 2 | 90.462 | 127.3 | 4.671 | 3.286 |
| 9 | 3 | 136.085 | 181.3 | 7.026 | 4.680 |

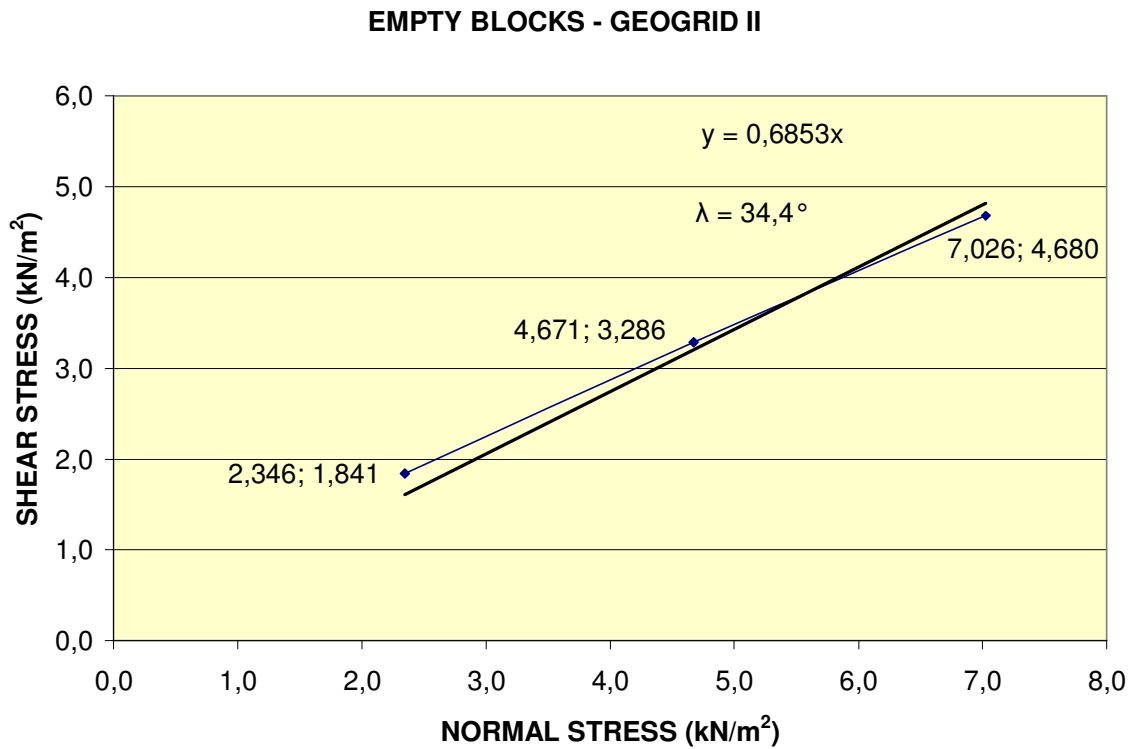


Figure 4.3. Plot of test results with holes left empty and Geogrid II used as reinforcement

4.5. Combination of Sand as Infill and Geotextile

The fourth group of tests was the constitution of tenth, eleventh and twelfth tests which were conducted with the combination of blocks filled with sand and geotextile. The results are shown in Table 4.4. and these results are plotted in Figure 4.4.

Table 4.4. Connection test results with sand used as infill and geotextile used as reinforcement

| Sand – Geotextile | | | | | |
|-------------------|--------------------------------|--|---|--|---|
| Test No | Number of blocks on Geotextile | Total normal load on Geotextile (kg/m) | Tensile load applied to Geotextile (kg/m) | Total normal stress on Geotextile (kN/m ²) | Shear stress applied to Geotextile (kN/m ²) |
| 10 | 1 | 66.051 | 85.3 | 3.410 | 2.202 |
| 11 | 2 | 134.444 | 189.3 | 6.942 | 4.887 |
| 12 | 3 | 202.017 | 273.3 | 10.430 | 7.055 |

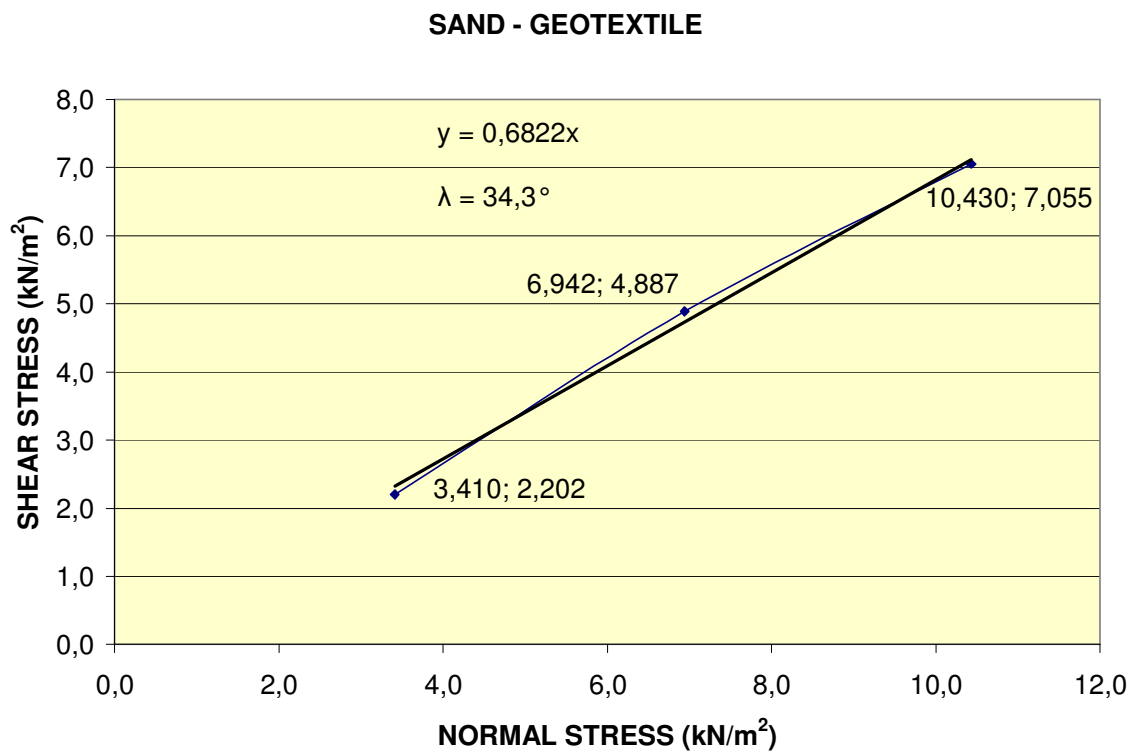


Figure 4.4. Plot of test results with sand used as infill and geotextile used as reinforcement

4.6. Combination of Sand as Infill and Geogrid I

The fifth group of tests was the constitution of thirteenth, fourteenth and fifteenth tests which were conducted with the combination of blocks filled with sand and Geogrid I. The results are shown in Table 4.5. and these results are plotted in Figure 4.5.

Table 4.5. Connection test results with sand used as infill and Geogrid I used as reinforcement

| Sand – Geogrid I | | | | | |
|------------------|-------------------------------|---------------------------------------|--|---|--|
| Test No | Number of blocks on Geogrid I | Total normal load on Geogrid I (kg/m) | Tensile load applied to Geogrid I (kg/m) | Total normal stress on Geogrid I (kN/m ²) | Shear stress applied to Geogrid I (kN/m ²) |
| 13 | 1 | 67.026 | 219.3 | 3.461 | 5.661 |
| 14 | 2 | 134.034 | 337.3 | 6.920 | 8.708 |
| 15 | 3 | 201.607 | 443.3 | 10.409 | 11.444 |

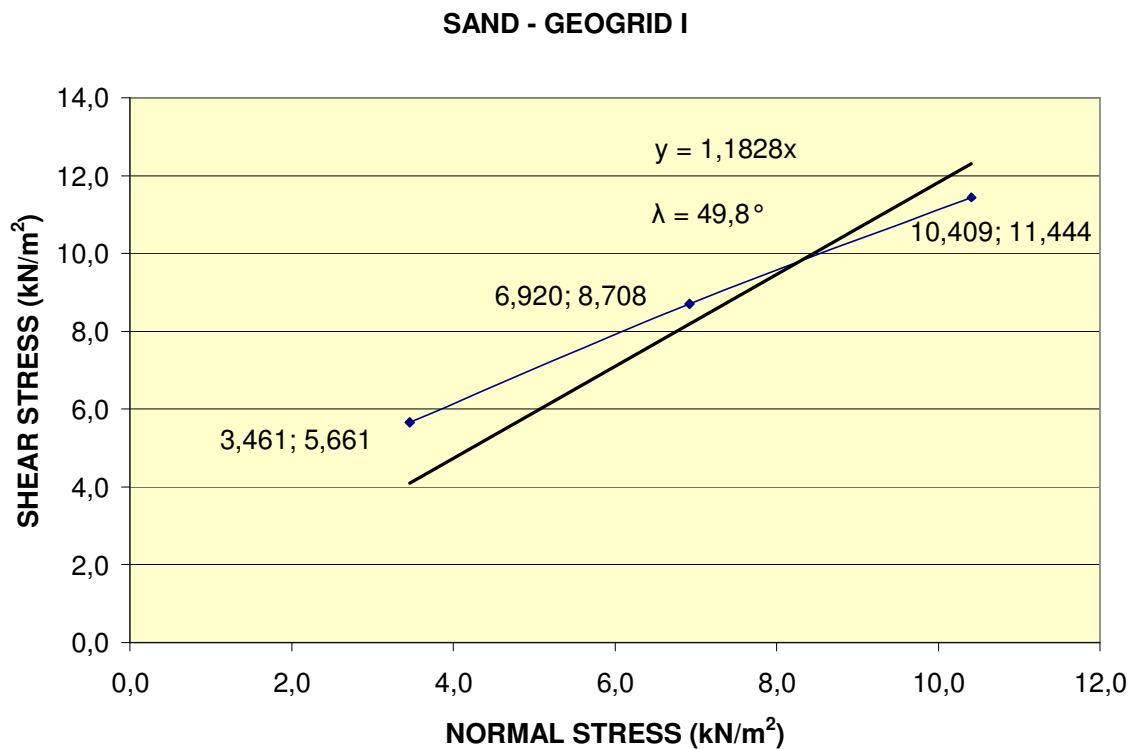


Figure 4.5. Plot of test results with sand used as infill and Geogrid I used as reinforcement

4.7. Combination of Sand as Infill and Geogrid II

The sixth group of tests was the constitution of sixteenth, seventeenth and eighteenth tests which were conducted with the combination of blocks filled with sand and Geogrid II. The results are shown in Table 4.6. and these results are plotted in Figure 4.6.

Table 4.6. Connection test results with sand used as infill and Geogrid II used as reinforcement

| Sand – Geogrid II | | | | | |
|-------------------|--------------------------------|--|---|--|---|
| Test No | Number of blocks on Geogrid II | Total normal load on Geogrid II (kg/m) | Tensile load applied to Geogrid II (kg/m) | Total normal stress on Geogrid II (kN/m ²) | Shear stress applied to Geogrid II (kN/m ²) |
| 16 | 1 | 67.026 | 96.3 | 3.461 | 2.486 |
| 17 | 2 | 133.795 | 241.3 | 6.908 | 6.229 |
| 18 | 3 | 202.923 | 351.3 | 10.477 | 9.069 |

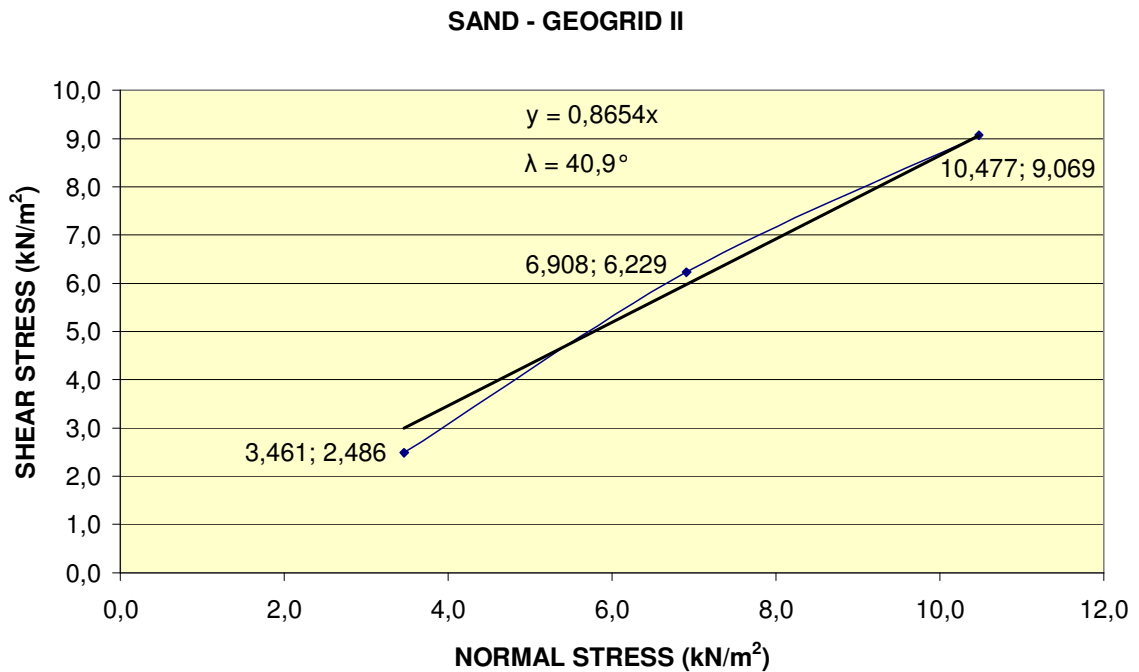


Figure 4.6. Plot of test results with sand used as infill and Geogrid II used as reinforcement

4.8. Combination of Gravel as Infill and Geotextile

The seventh group of tests was the constitution of nineteenth, twentieth and twenty first tests which were conducted with the combination of blocks filled with gravel and geotextile. The results are shown in Table 4.7. and these results are plotted in Figure 4.7.

Table 4.7. Connection test results with gravel used as infill and geotextile used as reinforcement

| Gravel – Geotextile | | | | | |
|---------------------|--------------------------------|--|---|--|---|
| Test No | Number of blocks on Geotextile | Total normal load on Geotextile (kg/m) | Tensile load applied to Geotextile (kg/m) | Total normal stress on Geotextile (kN/m ²) | Shear stress applied to Geotextile (kN/m ²) |
| 19 | 1 | 67.108 | 141.3 | 3.465 | 3.648 |
| 20 | 2 | 135.070 | 271.3 | 6.974 | 7.004 |
| 21 | 3 | 202.862 | 413.3 | 10.474 | 10.670 |

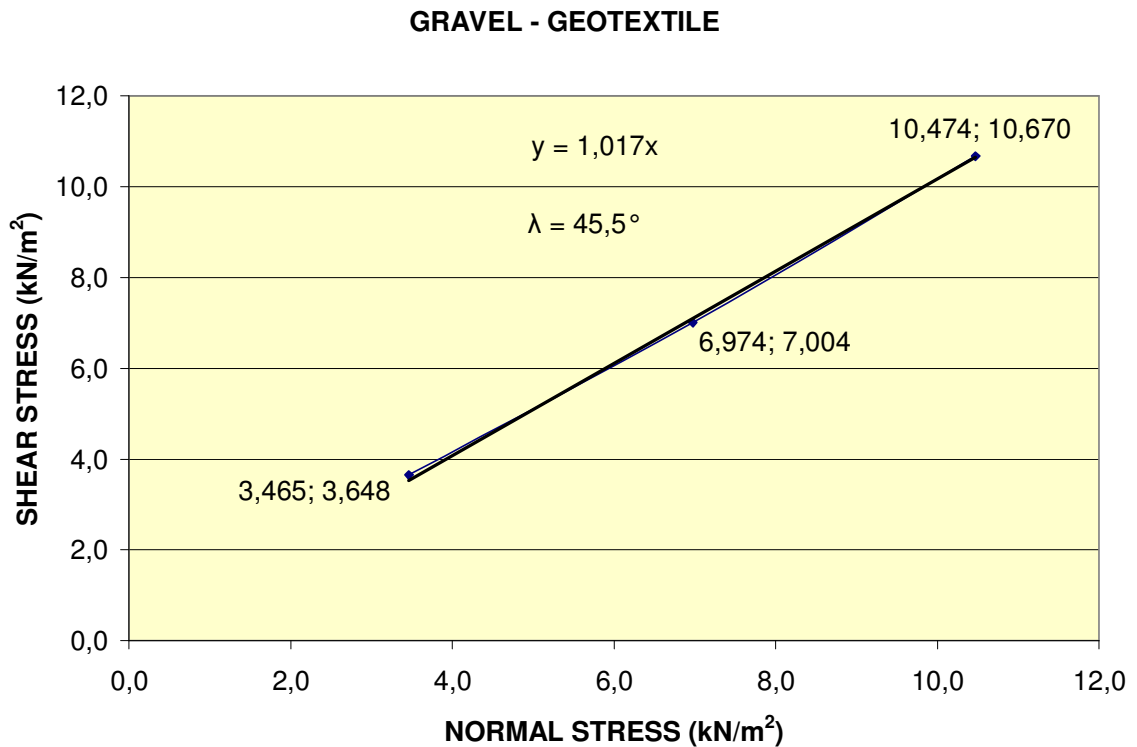


Figure 4.7. Plot of test results with gravel used as infill and geotextile used as reinforcement

4.9. Combination of Gravel as Infill and Geogrid I

The eighth group of tests was the constitution of twenty second, twenty third and twenty fourth tests which were conducted with the combination of blocks filled with gravel and Geogrid I. The results are shown in Table 4.8. and these results are plotted in Figure 4.8.

The weight capacity of the testing apparatus was surpassed during the conduction of twenty fourth test. The weight suspended to the geosynthetic reinforcement by steel clamp was increased up to 600 kg and no failure occurred during the test. No data could be obtained from this test.

Table 4.8. Connection test results with gravel used as infill and Geogrid I used as reinforcement

| Gravel – Geogrid I | | | | | |
|--------------------|-------------------------------|---------------------------------------|--|---|--|
| Test No | Number of blocks on Geogrid I | Total normal load on Geogrid I (kg/m) | Tensile load applied to Geogrid I (kg/m) | Total normal stress on Geogrid I (kN/m ²) | Shear stress applied to Geogrid I (kN/m ²) |
| 22 | 1 | 66.697 | 421.3 | 3.444 | 10.876 |
| 23 | 2 | 135.387 | 599.3 | 6.990 | 15.471 |
| 24 | 3 | 202.284 | | 10.444 | |

GRAVEL - GEOGRID I

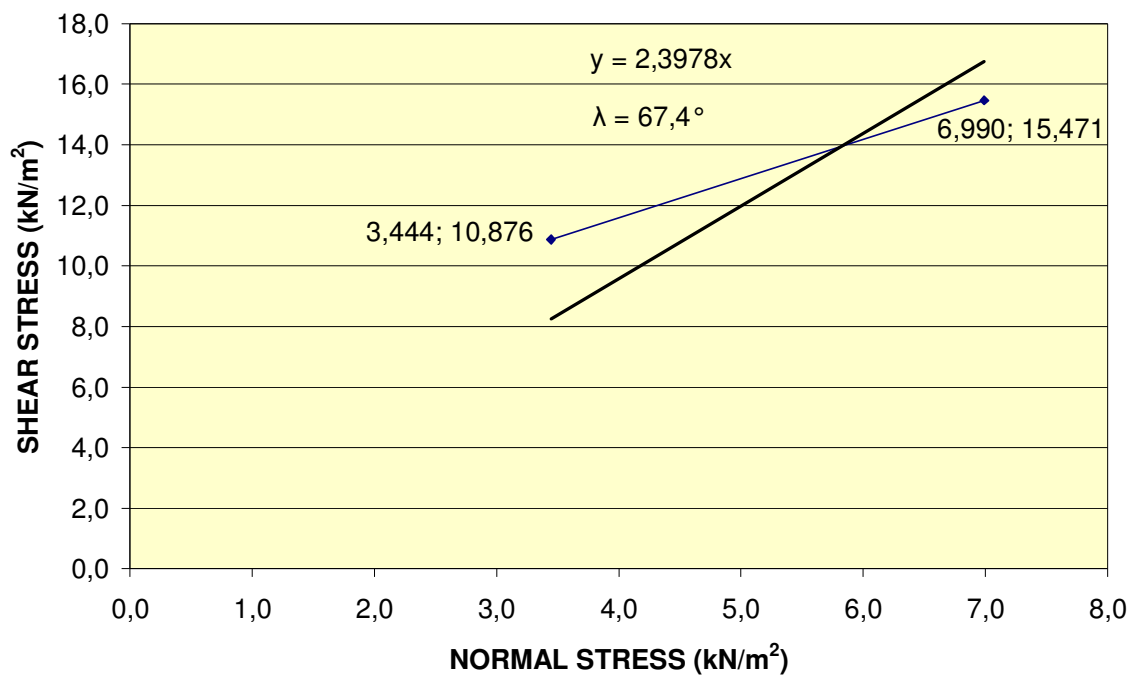


Figure 4.8. Plot of test results with gravel used as infill and Geogrid I used as reinforcement

4.10. Combination of Gravel as Infill and Geogrid II

The ninth group of tests was the constitution of twenty fifth, twenty sixth and twenty seventh tests which were conducted with the combination of blocks filled with gravel and Geogrid II. The results are shown in Table 4.9. and these results are plotted in Figure 4.9.

The weight capacity of the testing apparatus was surpassed during the conduction of twenty seventh test. The weight suspended to the geosynthetic reinforcement by steel clamp was increased up to 600 kg and no failure occurred during the test. No data could be obtained from this test.

Table 4.9. Connection test results with gravel used as infill and Geogrid II used as reinforcement

| Gravel – Geogrid II | | | | | |
|---------------------|--------------------------------|--|---|--|---|
| Test No | Number of blocks on Geogrid II | Total normal load on Geogrid II (kg/m) | Tensile load applied to Geogrid II (kg/m) | Total normal stress on Geogrid II (kN/m ²) | Shear stress applied to Geogrid II (kN/m ²) |
| 25 | 1 | 67.347 | 291.3 | 3.477 | 7.520 |
| 26 | 2 | 135.771 | 554.3 | 7.010 | 14.310 |
| 27 | 3 | 202.647 | | 10.463 | |

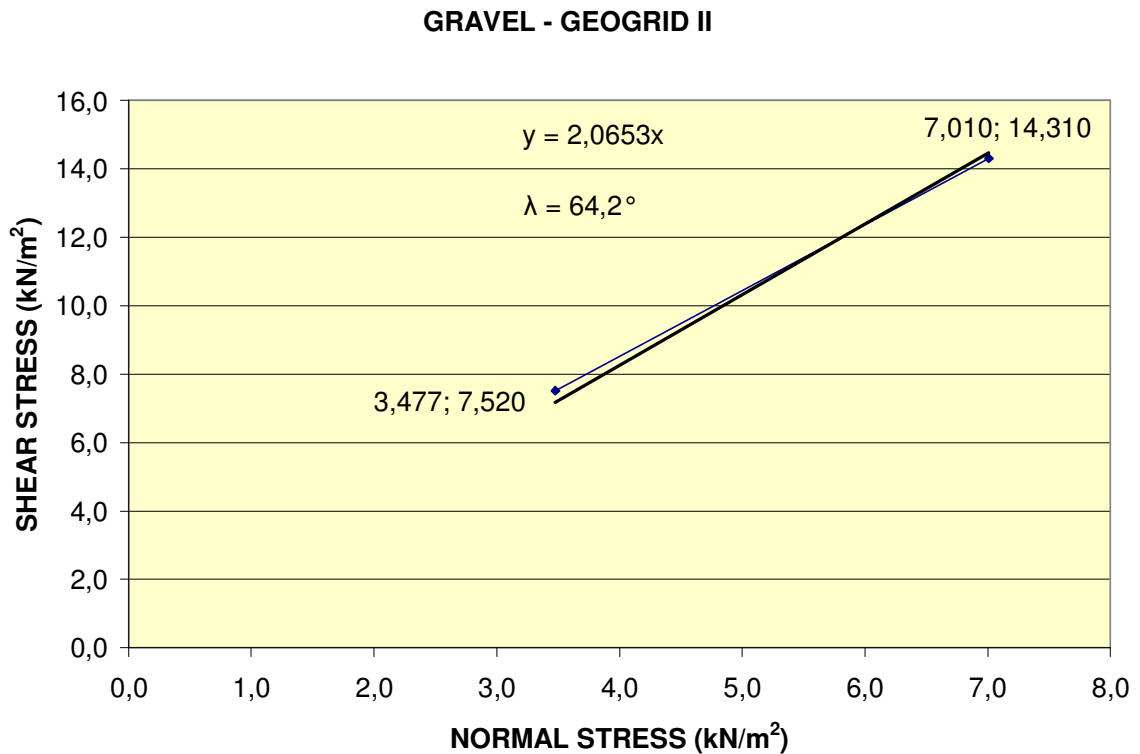


Figure 4.9. Plot of test results with gravel used as infill and Geogrid II used as reinforcement

4.11. Summary of Results

The results are summarized in Table 4.10. The greatest value of angle of friction was obtained from the combination of gravel used as infill and Geogrid I used as geosynthetic reinforcement.

Geogrid I (biaxial geogrid) exhibited the best performance among the other reinforcement types during tests. Geogrid I gave greater values of angle of friction in all infill conditions. In the tests the holes of concrete blocks left empty, geotextile had a greater value of angle of friction than Geogrid II. On the other hand, Geogrid II exhibited better performance than geotextile in the tests sand or gravel was used as infill material.

Table 4.10. Summary of results

| Infill and reinforcement type | Angle of friction (λ) (deg) |
|-------------------------------|---------------------------------------|
| Empty Blocks – Geotextile | 40.4 |
| Empty Blocks – Geogrid I | 43.6 |
| Empty Blocks – Geogrid II | 34.4 |
| Sand – Geotextile | 34.3 |
| Sand – Geogrid I | 49.8 |
| Sand – Geogrid II | 40.9 |
| Gravel – Geotextile | 45.5 |
| Gravel – Geogrid I | 67.4 |
| Gravel – Geogrid II | 64.2 |

The tests conducted with concrete blocks filled with gravel gave best results among the other options. The interlocking effect in two types of geogrid reinforcement and the weight of gravel put forward gravel infill among the other infill situations. Also the combination of concrete blocks filled with gravel and geotextile had a greater angle of friction than other combinations of geotextile.

The combination of concrete blocks filled with sand and geotextile exhibited the worst performance among the other combinations. Sand particles draw into the surfaces of reinforcement and concrete blocks so sand particles existing between the surfaces of geotextile and concrete blocks accelerated the slippage of geotextile. Empty concrete blocks and Geogrid II constitute the other low efficient combination.

Geogrid I and Geogrid II interlocked well with soil particles because of their large apertures. The interlocking mechanism had an increasing effect on frictional behavior of these two reinforcements.

To compare the effect of infill, the $\tan \lambda$ values are normalized with respect to the $\tan \lambda$ values obtained from tests conducted with empty blocks. To compare the effect of

reinforcement, the $\tan \lambda$ values are normalized with respect to the $\tan \lambda$ values obtained from tests conducted with geotextile reinforcement.

Table 4.11. Normalization of results

| Infill and reinforcement type | Angle of friction (λ) (deg) | $\tan \lambda$ | Normalization with respect to infill type | Normalization with respect to reinforcement type |
|-------------------------------|---------------------------------------|----------------|---|--|
| E. Blocks – Geotextile | 40.4 | 0.85 | 1.00 | 1.00 |
| E. Blocks – Geogrid I | 43.6 | 0.95 | 1.00 | 1.12 |
| E. Blocks – Geogrid II | 34.4 | 0.68 | 1.00 | 0.80 |
| Sand – Geotextile | 34.3 | 0.68 | 0.80 | 1.00 |
| Sand – Geogrid I | 49.8 | 1.18 | 1.24 | 1.74 |
| Sand – Geogrid II | 40.9 | 0.87 | 1.28 | 1.28 |
| Gravel – Geotextile | 45.5 | 1.02 | 1.20 | 1.00 |
| Gravel – Geogrid I | 67.4 | 2.40 | 2.53 | 2.35 |
| Gravel – Geogrid II | 64.2 | 2.07 | 3.04 | 2.03 |

Geogrid I was the most efficient reinforcement in all infill conditions among the other reinforcement types during tests. The two combinations of gravel with Geogrid I or Geogrid II were the most efficient options to utilize in the construction of geosynthetic reinforced soil retaining walls as shown in Table 4.11. The utilization of sand with geotextile and empty blocks with Geogrid II had the worst results during tests as shown in Table 4.11.

5. CONCLUSIONS

The construction of geosynthetic reinforced soil retaining structures can be executed rapidly, without large construction equipment. In addition, they are more flexible and economical than conventional retaining structures. In Turkish practice, the front-line of geosynthetic reinforced soil retaining structures are generally constructed with hollow modular concrete blocks. In the construction of these reinforced structures, the facing elements are connected to reinforcements without connection elements. Only the frictional forces maintain the integrity of the wall.

Modular concrete blocks are indispensable members of facing elements of geosynthetic reinforced soil retaining structures. This study investigated the frictional behavior of modular concrete blocks under several conditions. The frictional characteristics of hollow concrete blocks and geosynthetic reinforcement interface were observed with different infill soil types and different geosynthetic reinforcement types. Another variable to classify the tests was the number of concrete block layers on the geosynthetic reinforcement. Greater values of tensile stress were obtained by increasing the values of normal stress acting on the geosynthetic reinforcement. The increment of normal stress was supplied by increasing the number of concrete block layers on the geosynthetic reinforcement.

In this study, a total of 27 connection strength tests were executed in order to investigate the frictional behavior between modular concrete blocks and geosynthetic reinforcements. 27 tests were divided into nine groups according to type of infill and type of geosynthetic reinforcement. As reinforcement a woven geotextile and two types of geogrids were used. The tests were performed either with the holes left empty or filled with sand, or gravel.

The four concrete blocks were placed side by side to constitute the first layer of concrete blocks on the tray. The infill soil of the first layer concrete blocks was placed into the concrete blocks. One end of geosynthetic reinforcement sample was laid on the block and the second layer of concrete blocks was placed eccentrically. There were three blocks

at the second layer of concrete blocks. The other end of the geosynthetic reinforcement was attached to the steel clamp. The infill soil of the second layer concrete blocks was placed. If needed, new layers of concrete blocks were added and the number of concrete block layers on the geosynthetic reinforcement increased. This procedure provided more normal load in order to be able to get more tensile load. Each layer was filled with compacted soil. An initial weight of 10 kg was placed on the hook. This weight was increased until failure. An angle of friction was calculated in each test from the relationship between the normal load applied to geosynthetic reinforcements and tensile load at failure point.

It is obvious that soil filled modular concrete blocks will increase normal load more than empty blocks. Since frictional forces increase linearly with normal load, the geosynthetic reinforced soil retaining walls constructed with infilled modular concrete blocks will be more stable than walls constructed with empty blocks.

It is observed that the angle of friction λ has the highest value with the combination of blocks filled with gravel and Geogrid I. This biaxial geogrid interlocks with gravel particles successfully and protects the integrity of the wall in higher tensile loads. The effectiveness of the interlocking mechanism in geogrid-reinforced soil is evident.

Geogrid I demonstrated the best performance among the other reinforcement types during tests. In all infill conditions, Geogrid I gave greater values of angle of friction. This means that Geogrid I protects the integrity of geosynthetic reinforced soil structures better than the other reinforcements.

In spite of geotextile had a greater angle of friction than Geogrid II in the tests conducted with empty concrete blocks, Geogrid II gave better results than geotextile in the tests which concrete blocks filled with sand or gravel. Geotextile combined with empty concrete blocks better than Geogrid II. The rough surface of the concrete blocks and the continuous texture of geotextile combined successfully. In the tests which concrete blocks filled with sand or gravel, the soil particles interlocked well with the large apertures of Geogrid II. This interlocking mechanism resisted to the high tensile stress values at failure

plane. Geotextile had very small apertures so soil particles remained at the two sides of geotextile and they could not interlock with geotextile.

From all of the reinforcement types the best result was obtained with gravel infilled concrete blocks. The interlocking effect in two types of geogrid reinforcement and the weight of gravel put forward gravel infill among the other infill situations.

On the other hand, the combination of sand infill and geotextile displayed a worse performance than the combination of empty blocks with geotextile in spite of the weight of the sand existing in the holes of concrete blocks. Sand particles draw into the surfaces of reinforcement and concrete blocks so sand particles existing between the surfaces of geotextile and concrete blocks expedited the slippage of geotextile. This may affected the frictional behavior of geotextile negatively. The combination of sand infill and geotextile was the worst option among the other alternatives. This combination could not protect the integrity of the system successfully.

The other factor affecting the test results was the physical characteristics of the reinforcements. Generally, Geogrid II failed suddenly because of its sleek surface. Geogrid I was the most resistant reinforcement because of its slight adherent surface. The rough surface of concrete blocks connected best with the adherent surface of Geogrid I.

In conclusion, the combination of the gravel filled concrete blocks and Geogrid I proved to be the best option to be used in geosynthetic reinforced soil retaining walls. In the same way, the utilization of Geogrid II with the gravel filled concrete blocks was the other efficient option. There were no significant difference between the tests conducted with empty concrete blocks and sand filled concrete blocks.

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APPENDIX A: PICTURES TAKEN DURING EXPERIMENTS

Three hooks were fixed to the clamp with three holes in the clamp. Weights were suspended to the test apparatus by three hooks. The hooks are shown in Figure A.1. The clamp builds up connection between weights hanging on the three hooks and geosynthetic reinforcements. The clamp and the three holes on the clamp are displayed in Figure A.2. The test apparatus and the precast concrete block facing elements are indicated in Figure A.3. and Figure A.4.



Figure A.1. The hooks hanging on the clamp of the test apparatus

Figure A.5. illustrates the picture taken before conduction of the first experiment. The first experiment was conducted with a stratum of modular concrete blocks over the geotextile layer. The concrete blocks were empty during the first experiment. Figure A.6. indicates the condition that geotextile slipped between the block layers and failure occurred after conduction of the first experiment. Figure A.7. illustrates the picture taken before conduction of the second experiment. Two layers of concrete blocks were placed on geotextile. The concrete blocks were empty during the second experiment.



Figure A.2. The clamp and the three holes on the clamp



Figure A.3. Picture of the test apparatus and precast concrete block facing elements



Figure A.4. The top view of the test apparatus and precast concrete block facing elements



Figure A.5. Picture taken before conduction of the first experiment



Figure A.6. Picture taken after conduction of the first experiment



Figure A.7. Picture taken before conduction of the second experiment

In the third experiment, three layers of concrete blocks were placed on geotextile. The picture taken before conduction of the third experiment is displayed in Figure A.8. The fourth experiment was conducted with a single layer of modular concrete blocks placed on Geogrid I (biaxial geogrid). The concrete blocks were empty during the fourth experiment. Picture taken before conduction of the fourth experiment is shown in Figure A.9.



Figure A.8. Picture taken before conduction of the third experiment

In the sixth experiment, three layers of concrete blocks were placed on Geogrid I. The concrete blocks were empty during the sixth experiment. Figure A.10. indicates the picture taken before conduction of the sixth experiment. Figure A.11. shows the picture taken during conduction of the sixth experiment. Weights were hung to the clamp with the help of the three hooks. The picture taken after conduction of the sixth experiment is indicated in Figure A.12. In this figure the last shape of Geogrid I sample is shown after the failure. The sixth experiment was conducted with empty concrete blocks. Some ribs of Geogrid I come face to face with the hollow sections of concrete blocks and some ribs of Geogrid I come face to face with the middle part of the concrete blocks so the sample of Geogrid I was twisted.

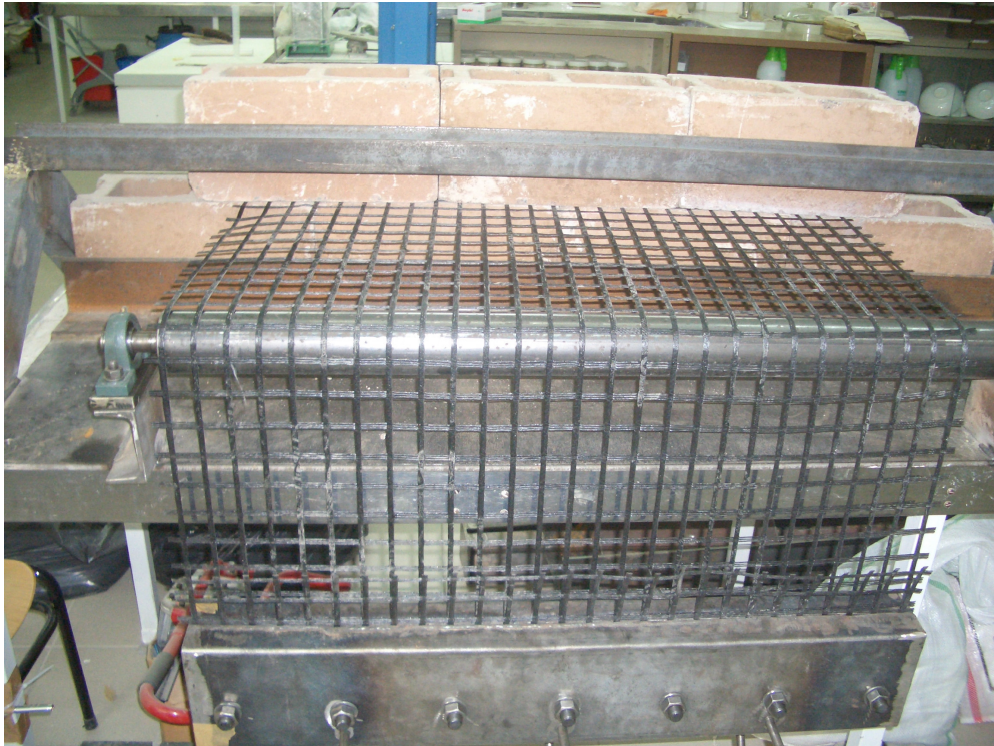


Figure A.9. Picture taken before conduction of the fourth experiment



Figure A.10. Picture taken before conduction of the sixth experiment



Figure A.11. Picture taken during conduction of the sixth experiment



Figure A.12. Picture taken after conduction of the sixth experiment

The picture taken before conduction of the eighth experiment is shown in Figure A.13. In the eighth experiment, two layers of concrete blocks were placed on Geogrid II. The concrete blocks were empty during the eighth experiment.

Figure A.14. indicates the picture taken before conduction of the tenth experiment. The tenth experiment was conducted with concrete blocks filled with sand. A single layer of concrete blocks was placed on geotextile. The picture taken after conduction of the twelfth experiment is displayed in Figure A.15. In the twelfth experiment, three layers of concrete blocks were placed on geotextile and sand was used as infill material. Sand particles draw into the surfaces of geotextile and concrete blocks after the slippage of geotextile. Also Figure A.16. displays the picture taken after conduction of the twelfth experiment. Sand particles draw into the surfaces of geotextile and concrete blocks.



Figure A.13. Picture taken before conduction of the eighth experiment

The fourteenth experiment was conducted with two layers of concrete blocks placed on Geogrid I. The concrete blocks were filled with sand. The picture taken before conduction of the fourteenth experiment is indicated in Figure A.17.



Figure A.14. Picture taken before conduction of the tenth experiment



Figure A.15. Picture taken after conduction of the twelfth experiment



Figure A.16. Picture taken after conduction of the twelfth experiment



Figure A.17. Picture taken before conduction of the fourteenth experiment

Figure A.18. shows the picture taken before conduction of the sixteenth experiment. In the sixteenth experiment, a single layer of concrete blocks was placed on Geogrid II. Sand was used as infill material. The picture taken before conduction of the seventeenth experiment is indicated in Figure A.19. In seventeenth experiment, two layers of concrete blocks were placed on Geogrid II. The concrete blocks were filled with sand.



Figure A.18. Picture taken before conduction of the sixteenth experiment

The picture taken before conduction of the nineteenth experiment is displayed in Figure A.20. In nineteenth experiment, a single layer of concrete blocks was placed on geotextile. The concrete blocks were filled with gravel. Figure A.21. shows the picture taken after conduction of the nineteenth experiment. The gravel particles draw into the surfaces of geotextile and concrete blocks after the slippage of geotextile.

The picture taken after conduction of the twentieth experiment is indicated in Figure A.22. In twentieth experiment, two layers of concrete blocks were placed on geotextile. Gravel was used as infill material. In this experiment the gravel particles draw into the surfaces of geotextile and concrete blocks after the slippage of geotextile.



Figure A.19. Picture taken before conduction of the seventeenth experiment



Figure A.20. Picture taken before conduction of the nineteenth experiment



Figure A.21. Picture taken after conduction of the nineteenth experiment



Figure A.22. Picture taken after conduction of the twentieth experiment

Figure A.23. shows the picture taken after conduction of the twenty first experiment. In twenty first experiment, three layers of concrete blocks were placed on geotextile. The concrete blocks were filled with gravel. Figure A.23. displays the last position of concrete blocks, geotextile and gravel particles. The gravel particles draw into the surfaces of geotextile and concrete blocks after the slippage of geotextile. Because of the upper confining piece, the tensile load acting on concrete blocks and the gravel particles, the initial position of concrete blocks was changed as shown in Figure A.23.



Figure A.23. Picture taken after conduction of the twenty first experiment

Figure A.24. displays the picture taken before conduction of the twenty third experiment. In twenty third experiment, two layers of concrete blocks were placed on Geogrid I. The concrete blocks were filled with gravel. The picture taken before conduction of the twenty sixth experiment is indicated in Figure A.25. In twenty sixth experiment, two layers of concrete blocks were placed on Geogrid II. Gravel was used as infill material.



Figure A.24. Picture taken before conduction of the twenty third experiment



Figure A.25. Picture taken before conduction of the twenty sixth experiment