The Failure of Concrete Retaining Block (CRB) Walls

by
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Declaration

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Abstract

Concrete retaining block (CRB) walls have been identified by ECSA (*the Engineering Council of South Africa)* as a class of structures prone to failure. In South Africa, four major CRB wall failures occurred in Gauteng in one month alone. By identifying trends in the failures, engineers obtain a better understanding of how a CRB wall system works and how these retaining walls fail. This insight will assist engineers in designing more reliable retaining walls that will satisfy all the foreseen structural, environmental and construction demands.

This study focuses on failed gravity and reinforced soil CRB walls in provinces throughout South Africa, including the Eastern Cape, Kwa-Zulu Natal and Gauteng. Through an extensive review of 18 case histories of failed CRB walls, common trends and aspects that typically cause problems with the walls are identified.

The case histories were obtained from ECSA and private consulting engineering companies. The outcomes of the case histories focus on a description of the failure, identification of the problem and any design-related issues. To further review the case histories effectively, the walls are classified and described according to specific criteria, and the basic failure mechanism(s) are identified. The information collected from this study is compared to the outcomes of previous studies. Furthermore, the outcomes of this study are described in such a manner as to be added to the database of the previous studies. The previous studies form part of the GSI (*Geosynthetic Institute*) database and focus on failed reinforced soil CRB walls on a global scale. Variations in the different studies are highlighted as the methods of classification, specifically regarding the backfill and the basic failure mechanism(s), are unique to each of the studies.

An overall look into the previous and present studies allows the author to make recommendations to improve the current shortcomings in the design and construction of CRB walls, as well as the manufacturing of CRB wall components. The major design and construction-related issues identified in both studies are very similar, with a few variations.

The current study recognises 11 major design and construction-related issues pertaining to gravity and reinforced soil CRB walls. These issues specifically focus on the components of the system including the type and placement/compaction of the backfill, an adequate drainage system and the placement thereof, construction drawings and specifications, performance monitoring, disruption of the system and the design itself.

Opsomming

Betonblok keermure (CRB mure) is deur ECSA (*die Ingenieurswese Raad van Suid-Afrika*) geïdentifiseer as ŉ tipe struktuur wat geneig is om te faal. In een enkele maand was daar vier gevalle in Gauteng, Suid Afrika waar blok keermure gefaal het. Deur ooreenkomste in falings te identifiseer, kan ingenieurs ŉ beter begrip kry van hoe hierdie keermure werk en hoe hulle geneig is om te faal. Met hierdie insig kan ingenieurs meer bertroubare blok keermure ontwerp wat al die voorspelde strukturele, konstruksie en omgewingseise bevredig.

Hierdie studie fokus op swaartekrag en versterkte grond blok keermure wat gefaal het in verskillende provinsies in Suid Afrika, insluitende die Oos-Kaap, Kwa-Zulu Natal en Gauteng. Tipiese tendense en aspekte wat probleme veroorsaak in die ontwerp en konstruksie van blok keermure kan geïdentifiseer word deur ŉ uitgebreide analise van 18 gevallestudies van falings van blok keermure.

Die 18 gevallestudies was by ECSA en private konsultasie ingenieursfirmas verkry. Die uitkomstes van elkeen van die gevallestudies fokus op ŉ beskrywing van die faling, identifisering van die probleem en enige ontwerp verbonde probleme. Die gevallestudies is geklassifiseer en beskryf volgens spesifieke kriteria en die basiese falings meganismes is geïdentifiseer vir verder ondersoek. Die uitkomstes van hierdie studie kan nou vergelyk word met die uitkomstes van vorige studies. Die uitkomstes van hierdie studie is op ŉ soortgelyke manier beskryf as vorige studies sodat dit by die databasis van die vorige studies bygevoeg kan word. Vorige studies is deel van die GSI (*Geosynthetic Instituut*) databasis en fokus op versterkte grond mure wat wêreldwyd gefaal het. Daar is variasies tussen die twee studies omdat daar ŉ verskil is in die metodes wat gebruik word om die mure te klassifiseer. Die klassifikasie van die grond en basiese falings meganismes is uniek vir elke studie.

Deur middel van ŉ algehele ondersoek van vorige en huidige studies kan voorstelle gemaak word om die konstruksie- en ontwerp-verbonde tekortkominge, asook kwessies met die vervaardiging van die komponente van blok keermure te verbeter. Tydens die studie en in vergelyking met vorige studies is daar ŉ beduidende tendens in konstruksie- en ontwerp-verbonde probleme opgemerk.

Hierdie studie erken 11 beduidende konstruksie- en ontwerp-verbonde probleme met betrekking tot swaartekrag en versterkte grond blok keermure. Die bogenoemde konstruksie- en ontwerp-verbonde probleme fokus op komponente van die sisteem, kompaksie en plasing van die grond, ŉ voldoende

dreinering stelsel, die uitleg van die dreinering stelsel, konstruksietekeninge en spesifikasies, prestasiekontrole van die konstruksie van die blok keermuur, ontwrigting van die blok keermuur sisteem asook die ontwerp van die blok keermuur.

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Chapter 1

Introduction

1.1. Problem Statement

Concrete retaining block (CRB) walls have been identified by ECSA (*the Engineering Council of South Africa)* as a class of structures prone to failure. In general, there are two types of CRB walls, namely gravity walls and reinforced soil walls. CRB walls are sometimes referred to as segmental retaining walls (SRW) and reinforced soil walls as mechanically stabilized earth (MSE) walls. Although the failure of CRB walls is a major problem globally, this research project focuses on CRB walls in South Africa. These failures can be the result of various causes which often occur in combination. In most cases, the primary cause of the failure is accompanied by one or more secondary causes, making it difficult to determine the primary cause. As stated by the CMA (*Concrete Manufacturers Association (Pty) Ltd.*) in their publication "*Concrete Retaining Block Walls: Code of practice for Gravity walls"* (Clark, 2005), CRB walls can provide the ultimate slope stability when they are properly erected.

1.2. Motivation

The use of CRB walls is rapidly increasing due to numerous advantages for the architect, engineer and the contractor. A wide range of aesthetically pleasing CRB walls are available and often provide the most economic means of retaining soil. CRB walls are quick and easy to construct, the facing block units are easily transportable and the blocks can be vegetated contributing to greening of the environment. As these walls become more popular and the height to which they are constructed increases, failures become more common and the consequences of these failures become more severe. Four major CRB wall failures occurred in Gauteng in one month alone (Day, 2014). When CRB walls collapse, they pose a threat to human lives and can cause significant damage to property.

By identifying trends in the failures, engineers can gain a better understanding of how a CRB wall system works and how these retaining walls fail. This information will assist engineers in designing more reliable retaining walls that will meet all the foreseen structural, environmental and construction demands. Furthermore, this research project will enhance the use of CRB walls by identifying commonly overlooked or underestimated components when designing, constructing and manufacturing all the constituents of these types of retaining walls. This study seeks to improve the design and construction of CRB walls resulting in more reliable retaining wall structures.

1.3. Aim

The aim of this research project is to review case histories of failed CRB walls to discern common trends and aspects that typically cause problems with the retaining walls. In addition, the designs are reviewed as many of them are flawed, and the major oversights in the designs must be identified. The case histories studied were obtained from ECSA and a private consulting engineering company.

A further aim of this research is to contribute to the GSI (*Geosynthetic Institute*) database. It is important to note that the GSI database only considers reinforced soil CRB walls (or MSE Walls), and not gravity CRB walls. Moreover, this research is aimed at assessing whether the designs themselves were erroneous or whether the existing codes used in the design adequately cover reinforced soil and/or gravity CRB walls or not. If the codes are inadequate, a list of design specifications and/or topics (problem areas) that should receive particular attention in any new design code will be recommended. If a new code, specifically for reinforced soil and gravity CRB walls, is deemed necessary, a work item should be proposed to SABS TC98 SC06 committee to draft a code dealing specifically with this topic. The proposal of a work item to SABS TC98 SC06 is beyond the scope of this research project. Furthermore, if there are significant construction issues regarding CRB walls, proposals will be made to amend SABS 2001 to include standard specifications and prescribed procedures for the construction of CRB walls.

1.4. Overview

There are seven main aspects of this study. The study commences with an extensive literature review of all topics pertaining to CRB walls. This is followed by the analysis of the case histories obtained from ECSA and a private engineering firm. The methods used for collecting and analysing the data are then described. Relevant outcomes from the case studies are presented including trends in the failures. The information obtained is then summarised in an appropriate format to enable it to be added to the GSI database.

Through an overall examination of the information presented in the study, recommendations are made which aim at improving the current shortcomings in the design, specification and construction of CRB walls and related components.

The seven main aspects of the study are discussed briefly:

1.4.1. Literature review

A brief discussion on the history of CRB walls and the state-of-the-art review are included, followed by an extensive literature review on the components of the system, namely the soil, facing, reinforcement and drainage. Various design methods used in South Africa and engineering considerations are discussed before an extensive study into the design of gravity and reinforced CRB walls is presented.

Previous studies on CRB walls were investigated, specifically focusing on the noteworthy findings from these studies, reasons for the failures of the walls and recommendations made in the literature to prevent future failures.

1.4.2. Data collection and processing

A method of collecting and processing data was established in order to review the case histories effectively. This included the collection of the data, classification of the wall, description of the failure and identification of the basic failure mechanism(s).

1.4.3. Outcomes of the present case studies

For each case study, an outcome is presented focusing on a description of the failure, identification of the causes of the problem and any design-related issues. The walls are classified and described according to their modes of failure and basic failure mechanisms. The case histories are reviewed to identify common trends of aspects that typically cause problems with CRB walls.

1.4.4. Trends in the failures

Through an extensive review of the case studies, an assessment is made as to whether or not the reinforced soil and/or gravity walls were adequately designed, and significant construction-related issues are identified. These design and construction-related issues are recognised through the identification of the reasons for the failures. Subsequently, recommendations are made to prevent the reoccurrence of the failures.

1.4.5. Recommendations to improve current shortcomings

An overall look at the previous and present studies enables the proposal of recommendations to improve the current shortcomings pertaining to the design and construction of CRB walls, as well as the manufacturing of CRB wall components.

1.4.6. Add case studies to the GSI database

The information collected from the present studies is compared to the outcomes of previous studies. By classifying the information from the case studies in a similar manner, the statistical data can be added to the GSI database.

1.5. Limitations

The study mainly focuses on gravity and reinforced soil CRB walls and does not consider other forms of soil retaining methods.

A further limitation is the completeness of the data available. Although over 28 cases of CRB wall failures are examined only 18 contain sufficient information to warrant inclusion in this study. In many cases, essential information including design calculations, drawings, photos, reports etc. are missing from the case study files.

1.6. Report Layout

Chapter 1: Introduction

A problem statement is presented and the motivation for the study is explained. In addition, the aim and objectives of the study are discussed, as well as the limitations and report layout.

Chapter 2: Literature Review

A discussion on gravity and reinforced soil CRB walls is presented in this chapter. The discussion focuses on previous publications on CRB wall structures. It includes a problem statement, history of the development and state of the art use of CRB walls as well as components of the system.

Chapter 3: Design

A discussion on the design of gravity and reinforced soil CRB walls is presented in this chapter. The discussion focuses on previous publications relating to the design of CRB wall structures. It includes the different design methods used in South Africa, engineering considerations for CRB walls, typical design procedures, a general description and functioning of each type of CRB wall, modes of failure as well as comments on the CMA (*Concrete Manufacturers Association*) design manuals and design examples.

Chapter 4: Previous Studies

Previous studies on CRB wall failures are presented in this chapter. An overview of the studies is presented and the noteworthy findings as reported in the literature are discussed, as well as the reasons for failure and the recommendations contained in the literature to prevent reoccurrence of these failures. Most of the studies form part of the GSI database.

Chapter 5: Research Methodology

The methodology provides a discussion on the overall approach followed to obtain the outcomes of this study. It discusses the method by which the data was collected and examined, as well as the method by which the information was classified and failures were described.

Chapter 6: Case Studies

The methodological approach, as discussed in Chapter 5, is applied to each case study. The outcomes for each case study are attached in Appendix B at the end of this report. The walls are classified according to the wall type, wall configuration, type of reinforcement, type of retained soil and other factors relating to the wall, followed by a description of the failures and basic failure mechanisms for each case study.

Chapter 7: Failure Trends

The data is examined to identify trends in the failures, specifically focusing on the soil, reinforcing, facing, drainage, disruption of the CRB wall system, environment, construction, design and any other failure trends which contributed to the failure of the walls.

Chapter 8: Discussion of Findings and Recommendations

The main reasons for the failures of the CRB walls in the current study are identified, and recommendations are made to improve the current shortcomings in the design and construction of CRB walls. The main findings are compared to the reasons for failure as discussed by Koerner in the previous studies of 171 MSE wall failures, which form part of the GSI database (Koerner & Koerner, 2013).

Chapter 9: Comparison with Previous Studies

The outcomes of the South African case studies are compared to the case studies in the GSI database in terms of wall ownership, location, facing type, maximum wall height, reinforcement, service life, backfill, compaction, person(s) responsible for the failure and the basic failure mechanism. The information is classified in a similar manner as to be added to the GSI database. Similarities and differences are highlighted and discussed.

Chapter 10: Conclusion

Important aspects are discussed and the interpretations are highlighted to consolidate the findings of the research study.

Chapter 11: Recommendations for Future Studies

Recommendations are made about possible future research to be conducted on CRB walls.

Chapter 12: References

A list of references is presented in alphabetical order.

Chapter 2

Literature Review

2.1. Problem Statement

CRB walls are not typical retaining walls as commonly understood in the industry. CRB walls are rather walls that consist of components, part of a system, to retain a filled or cut embankment. The embankment is constructed primarily of earth. Stabilization of the filled embankment behind the wall in the form of mechanical stabilization, such as fabric reinforcement, and cement stabilization may be incorporated into the system for additional stability. Walls with no reinforcement are called gravity walls and walls with reinforcement are known as MSE (mechanically stabilised earth) walls or simply as reinforced walls.

Gravity walls primarily rely on the strength of the backfill material and on the self-weight and batter of the facing units for their stability. The facing units are filled with soil to form a stable wall. Often the infill material in the lower two block courses incorporates concrete for additional stability. By cement/lime stabilization of a strip of soil behind a gravity wall, the effective thickness and self-weight of the wall can be increased.

Reinforcement may be provided by layers of fabric reinforcement sandwiched between the layers of compacted backfill material. The fill and wall facing are built up simultaneously. The spacing of the layers of fabric reinforcement is chosen to correspond with the height of block courses and the fabric passes through the wall allowing blocks below and above to clamp/anchor the reinforcement (confidential source, n.d.).

The wall facing primarily consists of blocks and serves to protect the face of the embankment. The facing accommodates lateral movement and in this regard differs fundamentally from rigid reinforced concrete retaining wall structures.

2.2. History of the Development of CRB Walls

The concept of CRB walls is not new and dates back 2500 to 3000 years ago when soil reinforcing methods, similar to those of CRB walls were used to construct the Tower of Babel. During the 14th through to the 17th centuries, the Great Wall of China was built. This historic structure was constructed using an early version of a MSE wall (NCMA, 2015).

The British inventor Joseph Gibbs initiated the trend to use hollow core building products in 1850. By 1876, Harmon S Palmer introduced dry-cast concrete block products in the United States. Interlocking concrete units were introduced in the 1960s as a concrete cribbing retaining wall system. These precast concrete panels retained the soil at the face of the wall while metallic strips were used to reinforce the soil (NCMA, 2015).

Koerner and Koerner (2013) explain in their publication "*A database, statistics and recommendations regarding 171 failed geosynthetic reinforced mechanically stabilized earth (MSE) walls"* that H. Vidal of France wrote the first paper on reinforced earth in 1966. Vidal's paper explained how long, closely spaced 100mm wide steel strips, connected to a metallic facing, extended back into the soil mass to provide adequate frictional anchorage, could be used to reinforce and retain the soil mass.

In the mid-1970s, welded grids were introduced as a reinforcing medium. Geosynthetic reinforcing was only introduced into the Civil Engineering market in the 80s. Significant use of CRB walls for conventional structures and soil-reinforced CRB walls commenced in the 1980s. In the 1990s the use of geosynthetic reinforced walls increased dramatically with the introduction of the Segmental Retaining Wall units (Hossain, et al., 2009). These segmental units are typical of the blocks used today.

2.3. The State of the Art use of CRB Walls

Concrete Retaining Block walls are versatile structures which are available in numerous styles and designs. They provide an economic means of retaining earth providing both slope stability as well as erosion protection if correctly installed. Some typical applications of CRB walls include:

- Bridge abutments and culvert wing walls;
- Plant supportive walls, also known as "live" or "green" walls;
- Stabilization and terracing of cut/fill slopes;
- River and lake embankments;
- Beach protection;
- Sharp or wide convex and concave walls;
- Light retaining stairs and seating;
- Landscaping;
- Retaining walls with various finishes and;
- Vertical and plantable section combinations.

2.4 Components of the System

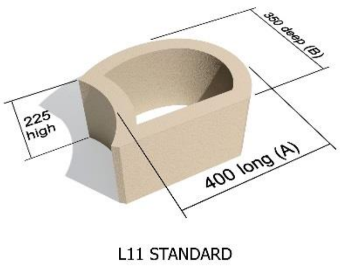
Basic components of a reinforced CRB wall system include the facing, the soil, the reinforcement and the drainage system. Section 5 of SANS 207:2006 explains the interaction between the reinforcement, facing and backfill material in a reinforced soil CRB wall system and may be referred to for additional information on the topic.

2.4.1 Facing

Facing units are available in a wide range of shapes, sizes and finishes. Gravity walls are typically constructed using segmental concrete blocks. Various types of facings can be used for soil reinforced walls including concrete panels, wire panel baskets, wrap-around geosynthetic reinforcement and full height or segmental concrete blocks (Parrock, 2003. James, 2006). This research focusses mainly on case studies using segmental concrete blocks.

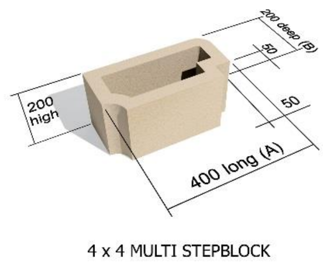
According to the NCMA, early block manufacturing equipment was designed around a standard concrete block unit of 203mm x 203mm x 406mm (NCMA, 2015). The typical depth for a modern facing unit is between 279mm and 305mm to allow for ease of construction, structural stability and economy (NCMA, 2015). The length of a facing unit is typically less than 610mm long, but larger machines can produce longer blocks.

Terraforce, Loffelstein, Cape Brick and Remblock, to mention a few, manufacture and supply CRB wall facing units in South Africa. Examples of some of the available facing units from these block manufacturers are illustrated in Figure 1 and Figure 2.



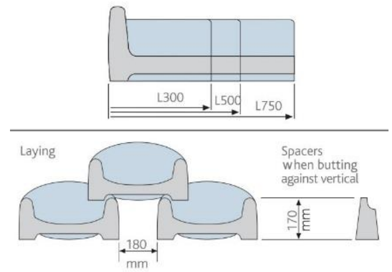
Source: Terraforce ®. Image by Terraforce ®.

(a)



Source: Terraforce ®. Image by Terraforce ®.

(b)



Source: AVENG INFRASET. Image by Unknown.

(c)

Figure 1: Dimensions of standard facing units (a) Terraforce L11 (b) Terraforce 4x4 multi step block (c) L300, L500 and L750 Loffelstein retaining blocks

Source: Terraforce ®. Image by Terraforce ®.

(a)

Source: Terraforce ®. Image by Terraforce ®.

(b)

Source: Deranco (Pty) Ltd. Image by Unknown

(c)

Figure 2: Completed retaining block walls implementing the following facing units (a) Smooth face Terraforce L11 (b) Terraforce 4x4 multi step block (c) Loffelstein retaining blocks

Terraforce and Lofflestein brochures and websites have indicated that most of the commercial facing units in South Africa are typically 100mm to 250mm in height, 200mm to 450mm in width and 300mm to 750mm in length, with a wall thickness between 40mm and 50mm (Terraforce, 2016; BUS Digital, 2016). These facing units vary in weight from 13kg to 82kg per unit (Deranco (Pty) Ltd, 2013).

The type of facing units selected is generally governed by aesthetics. The completed wall should be aesthetically pleasing and blend in with its surroundings. Ease of maintenance, initial cost and resistance to accidental impact from traffic also play a role in the selection of the block type. The required length of the block is determined by the design of the wall. High walls typically require longer blocks or multiple layers of blocks over the lower sections of the wall.

The functions of the facing are to protect the face of the fill, to secure the front ends of the layers of reinforcement and to transmit forces in the plane of the face to the foundations. Two parameters which control the suitability of the facing units include the ability of the blocks to resist the axial loads down the face of the wall, as well as the shear resistance between consecutive courses of blocks. Standard laboratory tests are specified by the CMA to determine the block-on-block friction and the crushing strength of the blocks.

The facing should further accommodate the deflection of the reinforcement during and after construction, without overstressing the facing which results in excessive deformation. To control the movement, James (2006) suggests three approaches:

- Design the reinforced fill and facing to be self-supportive with a void between the two structures as to prevent interaction;
- Allow for the redistribution of movement between the two structures through iterative stiffness design analyses of the two structures; or
- Design a reinforced fill with a facing which is flexible enough to accommodate the movement of the fill.

According to the CMA, a designer should know the following properties of the block prior to design of the CRB wall system:

- The length, width and height of the block;
- The estimated weight per square meter of the block plus infill soil;
- The coefficient of block-on-block friction which is obtained from laboratory tests namely the block-on-block friction test;

- The nib shear strength per meter run of wall obtained from laboratory tests namely the Nib Shear Strength test; and

- The crushing strength of the block obtained from the Crushing Strength laboratory test.

The designer often has a choice regarding the length of the block into the face, for a particular block type. For example, Loffelstein supplies three basic block sizes. Their width and height are identical, but the approximate mass per block differs due to the different lengths of the blocks. The lengths available for the L300, L500 and L750, blocks are 300mm, 500mm and 750mm long respectively. The concrete foundations to these facings are case-specific, but generally a simple mass concrete strip foundation will suffice. The wall system obtains its inclination through the block offset, generally with mechanical interlocking such as shear/concrete keys. In the case of Loffelstein, the block has a front up-stand or nib (a mechanical connector) to ensure face-to-face contact, providing additional stability. Bathurst and Simac (1994) explain that the principle purposes of these mechanical connectors are to assist with unit alignment and to control the wall facing batter during construction. If these mechanical connectors do not provide sufficient wall inclination, the top of the concrete foundation is used to achieve the desired inclination.

It is good practice to provide concrete footings for all CRB walls. Parrock (2003) states that the provision for a footing is dependent on the loads exerted by the facing on the base, the point of application and direction of the load, and the condition of the founding soils. Parrock recommends a factor of safety of at least 4 to limit deflections. If a concrete footing is not provided, Parrock suggests that the blocks should be set into a concrete/mortar layer.

The laboratory tests conducted during the manufacturing process are essential, as the concrete blocks have to comply with certain specifications. With non-standard blocks, these specifications are often difficult to determine. The non-standard blocks could pose a threat to the stability of the CRB wall system if the uncertainties are not accounted for with acceptable factors of safety.

The designer should confirm that the blocks are of sufficient strength and within acceptable tolerances for the specific CRB wall system under consideration. Shear transfer between the unit layers is primarily developed through shear keys and interface friction (Bathurst & Simac, 1994). These shear keys should be able to resist the shear forces between the blocks. If these blocks are subjected to aggressive water or chemicals, the CMA's code for gravity structures suggests that block units with higher cement content should be used (Clark, 2005).

2.4.2 Soil

After the blocks themselves, the soil is probably the most important component of the CRB wall system. It is normally specified in terms of its grading, plasticity and strength after compaction as measured by the CBR (*California Bearing Ratio*) test. Electro-chemical tests are also conducted where there is a possibly that the soils may be aggressive towards concrete (Pequenino, et al., 2015). For design purposes, it is necessary to determine the shear strength of both the compacted backfill and any in situ soils behind the wall. These shear strength properties are determined by shear strength tests and expressed using the Mohr-Coulomb failure criterion.

For economic reasons, soil from the site is often used as backfill material. Although this is convenient, the properties of the soil are often not acceptable. Inappropriate backfill materials include expansive clays, organic soils, poorly graded sands and soils with a plasticity index (PI) larger than 20 or a liquid limit (LL) larger than 40. Pequenino et al. (2015) recommend that the soil on-site should be investigated during the planning or design phase of the project, and should only be used if it has been shown by testing to meet the design requirements.

As further explained by Pequenino et al. (2015), the preferred soil to use as backfill material for a CRB wall is a high quality, granular material of sound durability, drainage, constructability and soil-reinforcement interaction characteristics. These granular soils should have an internal angle of friction between 32° and 36° depending on the degree of compaction. Fine grained, cohesive soils with an internal angle of friction less than 31° are acceptable on condition that the material is adequately compacted and appropriate water management is provided (Block, 2010). According to Parrock (2003), the most appropriate material for construction of a CRB wall with is a G6 type material according to the TRH14 soil classifications. Nevertheless, walls can also be constructed using even G10 type in-situ soil provided the design takes account of the poor quality of the backfill, the material is compacted to specification and adequate drainage is provided.

The material should be placed in relatively thin layers, typically between 150mm and 200mm thick, to ensure that the desired degree of compaction is obtained. This is important in order to limit the settlement of the compacted fill (Day, 2015). Furthermore, proper compaction is required to achieve the maximum friction between the soil and the layers of reinforcement in reinforced CRB wall structures. To achieve the specified compaction, the material should be placed at or near its optimum moisture content (OMC). After each layer has been placed, field density and moisture content determination tests should be carried out at relevant positions and compared with the maximum dry density and optimum moisture content of the material established by means of compaction tests in the laboratory. These test results should be recorded as part of the construction records for quality assurance and for future reference.

James (2006) mentioned in the half-day seminar "*How Safe is That Concrete Retaining Block Wall? Geosynthetics in Reinforced Soils, Polymers, Products, Properties and their Behaviour*" that theoretically any type of soil can be used as backfill material for reinforced CRB walls, but it is important to remember that the soil properties and the state that the soil is in has a huge impact on the behaviour of the system. The choice of soil backfill material is mainly dependant on the type of reinforced structure and the type of reinforcing, namely strips, sheets or grids, being used. James (2006) continues to state that if reinforcing strips or grids are used in a CRB wall system, granular type material should be used as backfill as the dilatant nature of such soils enhances the pull-out resistance of the reinforcement. Material with high fines content is unsuitable for use with strips or grids as the bond between the reinforcing and soil is poor and reduces if positive pore pressure develops. If geotextile sheets are used, the backfill material could contain high fines content, as the geotextile sheets do not utilize the dilating effect in the soil. These sheets could further act as horizontal drainage layers which drain the water from the backfill.

The two most common natural soil types encountered in this study were the Berea Red soils in the Durban area and residual granites to the North of Johannesburg.

The Berea Formation occurs along the Kwa-Zulu Natal coast stretching up to 80km inland in the North. The oldest sands, which are found furthest inland, are the deepest red in colour and contain the highest clay content. The properties of this soil vary over short distances laterally and vertically, mainly due to the large variation in the clay content and moisture status of the soil. A large range of cohesion is present in the area of the Berea Formation and the consistencies of the soil range from loose to very dense, indicating that the soil is collapsible in the more sandy, highly compressible areas (Brink, 1985).

As the moisture content of the soil is often above optimum, the soil may need to be dried out prior to compaction. Furthermore, the shear strength and compressibility is extremely sensitive to changes in moisture content. Compaction of the soil improves both the shear strength and stiffness of the compacted soil and reduces its sensitivity to changes in moisture content. The Berea sands with higher clay content respond well to lime stabilization. The addition of only 4% road lime was found to increase the CBR value of the soil at 95% MOD ASSHTO from 5% to 95% (Brink, 1985).

Residual granite soils are typically friable, fine to coarse grained sands that form from weathering of the granitic rocks of the Johannesburg-Pretoria granite inliers. These residual granites, which can be up to 20m thick in places, underlie the Northern suburbs of Johannesburg and the Midrand area. The geotechnical characteristic of the soil varies according to the degree of weathering. Residual granites exhibit high strength when dry due to the colloidal coatings of the individual quartz grains (Brink, 1979). These colloidal bridges between the quartz particles become lubricated when the soil is saturated under a load and loose strength instantaneously. Brink (1979) explains that the grains become more densely packed and may lead to sudden settlements.

In areas of high rainfall, and in situations susceptible to leaching, the fine-grained particles of colloidal kaolinite are largely removed by circulating groundwater, leaving behind silty sand. This material often exhibits collapsible grain structure. Brink (1979) explains that the cracking in many of the buildings in the Northern suburbs of Johannesburg, Randburg and in Sandton occurred as a result of the residual granites in the area which possess a collapsible grain structure.

In South Africa, the soils which possess this phenomenon are all found to fall within, or in close proximity to areas of annual water surplus. Brink (1979) explains that this emphasises the role played by thorough leaching in the development of these soils. The residual granites are particularly susceptible to the washing out of finer particles from between coarser particles under a sufficient hydraulic gradient. As explained by Brink (1979), the process is known as suffusion. Suffusion is responsible for the development of collapsible grain structure in residual granite soils.

Brink (1979) adds that collapsible grain structure in the soils can be predicted from field evidence. The most obvious and significant field test includes the observation of the failures in existing surrounding structures.

The consistency of the soil depends on its moisture content varying from very stiff in a dry soil to very loose in a saturated soil. The high void ratio and porous structure which characterises the collapsible condition will usually be clearly evident while recording the soil profile. A simple field test using a hand lens can easily recognise the colloidal coating around the quartz grains and clay bridges between them.

Furthermore, Brink (1979) explains that the execution of the Jennings "sausage test" can confirm collapsible grain structure in the soil. This is a simple field test in which two identical cylindrical soil samples are carved out of the undisturbed soil as neatly as possible. The one cylindrical sample is saturated, kneaded and remoulded into a cylindrical shape of similar diameter as the original. An obvious decrease in length to the undisturbed twin sample confirms collapsible grain structure.

An alternative test involves backfilling a pit with the original soil excavated from it. If the backfill material fails to fill the pit completely, the soil possesses collapsible grain structure. The shortfall of material is particularly evident when the backfill material is saturated.

Laboratory tests can also be used to detect a collapsible grain structure in soils (Brink, 1979). The most popular of these tests is the collapse potential test in which a sample is placed in the oedometer at natural moisture content, loaded incrementally to 200kPa and then saturated. The resulting settlement expressed as a percentage is known as the collapse potential of the soil. Similar observations can be made using the double oedometer test.

Visual examination is also an indicator of collapse potential. Soils with a collapsible grain structure frequently exhibit a pin-hole voided structure which can be observed in the field with the naked eye or using a hand lens. Alternatively, one can examine thin sections of the soil under a microscope. The soil specimen must be impregnated with a liquid epoxy resin before microscopic tests can be conducted.

The collapsible nature of these residual granite soils is the result of the open (voided) grain structure of the in-situ soil. This structure is destroyed during excavation and compaction of the material. As a result, the residual granites particularly those with a gravelly sand texture, can provide good quality backfill (G7 - G5 materials according to TRH14) when adequately compacted.

2.4.3 Reinforcement

CRB walls may be reinforced or unreinforced. According to the CMA design check list for CRB wall design (CMA, 2013), geosynthetic reinforcement should be considered for walls higher than two meters or walls with an inclination of more than 70° to the horizontal. Pequenino et al. (2015) explains that the process followed in the design of reinforced soil CRB walls is highly dependent on the type of reinforcement used.

James (2006) describes geosynthetic reinforcement as being a non-linear, visco-elastic plastic material which consists of geotextiles, geogrids and geocomposites. These geosynthetic strips, sheets or grids are known as extensible reinforcement products. Their rupture strains are larger than the maximum tensile strain in the soil without reinforcement, subjected to similar operational stress, and their properties are time and temperature dependant (James, 2006). Inextensible products include steel bars or rods and steel mesh. The rupture strains of inextensible reinforcement products is less than the maximum strain in the soil without reinforcement, subjected to similar operational stress, and their properties are independent of time and temperature. Pequenino et al. (2015) warns that the operational strain of the reinforcement should be thoroughly understood before the reinforcement is specified for use, as failure to do so is likely to result in excessive and undesirable deformations.

Geotextiles, which are typically supplied in rolls in widths of up to 5,6m, can be woven or non-woven. Geogrids can be uniaxial or biaxial and geocomposites are combinations of two or more of the aforementioned geosynthetics. The products that are available to reinforce soil are categorized into two groups namely directionally structured reinforcement, which has different tensile resistance in long- and cross-directions, and isotropically structured reinforcement, where the strength in both directions is the same (James, 2006). Directionally structured reinforcement includes woven geotextiles, warp knitted geotextiles and geogrids. Isotropically structured reinforcement includes non-woven needle punched geotextiles, non-woven heat bonded geotextiles, non-woven chemically bonded geotextiles and non-woven stitch bonded geotextiles.

Woven geotextiles were identified as the most commonly used geosynthetic for reinforced CRB walls in South Africa. By contrast, Bathurst and Simac (1994) found the majority of CRB walls in Canada and North America incorporated polymetric geogrid materials. Koerner, Soong and Koerner (2005) agree with

Bathurst and Simac as they found over 30 000 SRWs worldwide which generally used geogrids, and only occasionally geotextiles were used as geosynthetic reinforcement.

Woven geotextiles are relatively high in strength and low in extensibility when compared to non-woven geotextiles (James, 2006) making them more suitable for soil reinforcement. Non-woven geotextiles are used primarily for filtration rather than reinforcement.

The four main polymers used as raw materials for manufacturing woven geotextiles consist of polyester, polyamide, polypropylene and polyethylene. These geotextiles are manufactured using a weaving process using two sets of yarns, the warp and the weft, interlaced and running perpendicular to one another (James, 2006). The warp runs along the length of the geotextile, in the strong machine direction, and the weft is the transverse, weaker direction. It often occurs that the reinforcement is rolled out in the wrong direction in the sense that the weaker weft direction runs perpendicular to the facing (Day, 2015). This raises concern as the ultimate strength is lower and extension is greater in the weft direction (James, 2006). Furthermore, James states that the surface friction and adhesion differs in the two directions, and where plane strain conditions occur, the geotextile should be orientated to make best used between the warp and the weft.

In the webinar, "*Geotextiles and Geomembranes: A data base, statistics and recommendations regarding 171 failed geosynthetic reinforced mechanically stabilized earth (MSE) walls*", Koerner states that geotextile reinforcement was originally used to reinforce MSE walls, but recently geogrids are being used as reinforcement for these walls. Manufacturers of these geogrids produce several different types, each having different strengths (Koerner & Koerner, 2013). They are characterised into homogenous, coated yarns or strap/rod geogrids.

When selecting geosynthetics for reinforced CRB wall systems, James (2006) suggests that the following be kept in mind:

- The stiffness of the geosynthetic;
- Strength of the geosynthetic;
- Creep;

- Drainage;
- Design life;
- Susceptibility to change;
- Chemical Stability;
- Effect of heat and moisture;
- Ultraviolet attack from the sun;
- Connection to the facing; and
- Cost of the geosynthetic.

The positioning of the layers of reinforcement within the backfill material and the length of each layer is crucial. The connection of the reinforcement layers to the facing are also of importance. Once the characteristics of the soil and reinforcement are known, the lengths and placement of the reinforcement can be determined by applying the appropriate design methods given in codes and design manuals. If the full strength of a supporting geosynthetic is to be used in combination with the strength of the soil, a geosynthetic with approximately the same magnitude of strain as the soil should be used. If the full strength of the geosynthetic is low compared to the strength of the soil, a geosynthetic with lower extension properties should be used.

2.4.4 Drainage

2.4.4.1 Overview

To ensure successful performance of a CRB wall, a drainage system should be incorporated to adequately deal with any water which might enter the fill either from the surface, from leaking services or from the surrounding ground. This includes preventing the soil from becoming saturated during and after construction, as well as ensuring that the final design routes water away from the system. The former can be accomplished by ensuring that the surface runoff is directed away from the excavation and retaining wall system, through temporary grading of the site. Good construction practice, as suggested by Block

(2006), includes covering the fill material at the end of each day to prevent water saturation during rainy periods.

Drainage redirects water away from all the soil in contact with, or within the retaining wall system, to prevent groundwater pressures from causing failure or excessive deformation. The accumulation of water from rain- and/or groundwater, or other sources such as leaking pipes, etc. negatively affects the stability of the wall. It has a dual impact on the increased earth pressure acting on the wall as well as a decrease in the bearing capacity and resistance of the backfill material to sliding.

The designer should have a thorough understanding of the site; the gradients on the site; surface drainage; direction of flow; type of surface vegetation; seepage; groundwater conditions and where water could originate from. These aspects should be taken into account during the design process in order to determine how the water content can be effectively maintained at acceptable levels. Block (2006) points out specific characteristics of a retaining wall system which would indicate the need for a drainage system:

- If the height exceeds 1.2m;
- Poorly drained/soils with a low permeability;
- Runoff from paved areas in the vicinity;
- Waterlines, mains or fire hydrants in the vicinity;
- Slopes above the wall;
- Multi-tiered or terraced walls;
- All commercial and municipal projects;
- Concentrated water sources are in the vicinity such as:
    - ♦ Driveways;
    - ♦ Excessive grading of the site;
    - ♦ Roof down pipes;

- Sump pump outlets; and
- Irrigation systems.

The GRI Report #38 emphasizes the importance of preventing water from building rooftops, parking areas, vegetated areas, etc. being routed directly into the reinforced soil zone (Koerner & Koerner, 2009).

According to the CMA in their code of practice for gravity walls, conventional gravity CRB walls are naturally free draining if the correct backfill material is used and, therefore, do not require weep-holes or additional drainage systems (Clark, 2005). However, stabilization of the soil behind the wall decreases permeability, requiring additional precautions to be taken. In areas where relatively high groundwater seepage is present, where there is potential for development of water table (perched or permanent) or where stabilised backfill is used, a drainage system such as that illustrated in Figure 3 is necessary. Alternatively, a drainage blanket can be run up the back of the wall at the backfill-retained soil interface and connected to the subsoil drain as seen in Figure 3. If seepage is not expected in the area where a stabilized CRB wall is to be constructed; coarse, clean sand weep-holes at regular intervals would suffice. The coarse, clean sand should be placed behind the stabilized layer and the weep-holes should penetrate through the stabilized material to prevent build-up of water pressure behind this material. The CMA states that the first row of weep-holes should be 200mm above the natural ground level, at 1m lifts up the height of stabilization (Clark, 2005).

The CMA (2005) maintains that most failures occur due to insufficient storm water control above conventional CRB walls. Their code of practice for gravity walls (2005) states that the failures often occur due to ponding at some point behind the wall. To ensure ponding is prevented, the code suggests that a lined surface channel should be installed behind the top of the wall. This surface channel is sized depending on the expected storm water runoff. Irrespective of the presence of storm water runoff to the retaining wall system, the code of practice for gravity walls (2005) suggests that geofabric should run up the back of the wall facing and be tucked underneath the top row of blocks to prevent loss of ground through the gaps between the blocks. This is not necessary if stabilised backfill is used, unless only the bottom portion of the backfill is stabilized. In such instances, a geofabric should be included above the non-stabilized section. Moreover, the code advises that an impermeable geomembrane should be installed approximately 400mm below the top of the wall to prevent the storm water from seeping into the material behind the wall as shown in Figure 7 for both gravity and reinforced soil CRB walls.

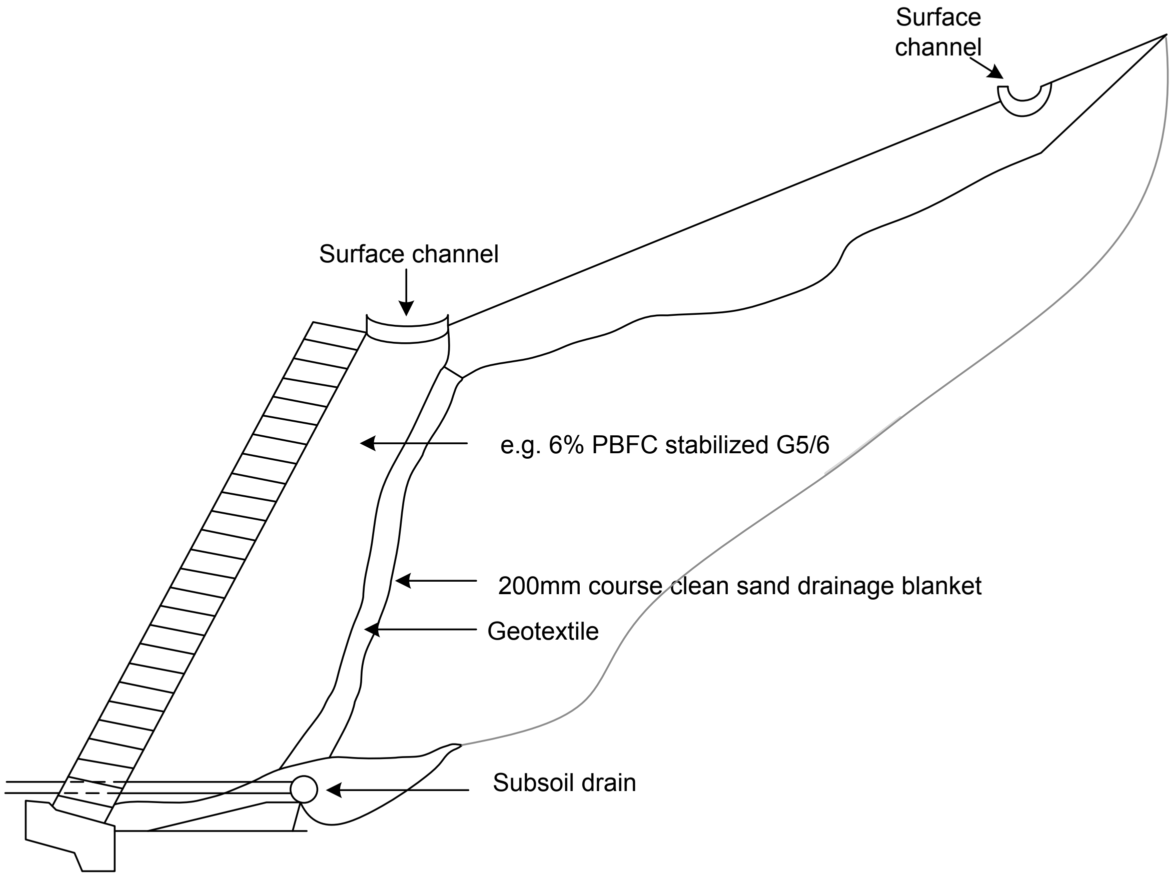


Figure 3: Subsoil drain in a conventional gravity CRB wall system (Clark, 2005)

The standard drainage system for reinforced soil CRB walls includes weep-holes penetrating through the facing with a granular, free draining material (of single size stone) immediately behind the facing, for the full height and length of the wall. The minimum thickness of this granular, free draining soil column and the spacing of the weep-holes are dependent on the size of the wall. Bathurst and Simac (1994) recommend that a geotextile separator be used to prevent loss of the material from the soil column through the facing. Furthermore, the soil column should be connected to a base drain. The base drain is a gravity flowing pipe wrapped in geotextile and connected to outlets or a storm water system which directs the water away from the retaining wall system. When the backfill material of a reinforced soil CRB wall is not granular and free draining, water pressure built up behind the backfill which can be prevented by providing continuous or intermittent geocomposite drains behind the backfill as seen in Figure 5, or chimney drains consisting of a granular, free draining material as seen in Figure 4. The most common drainage system for a reinforced CRB wall is a soil or geocomposite base drain coupled with a geocomposite back drain.

Block (2006) states that the additional chimney drain between the reinforced soil zone and the retained soil zone can be included into the drainage system by connecting the chimney drain to the base drain. Alternatively, a geotextile filter should be provided at the interface. Day (2015) notes that the grading of the filter should be compatible with that of the surrounding soils to prevent fines migration, weep-hole clogging, loss of backfill and caving.

Where water pressure in the in-situ soils behind the backfill threatens the stability of the wall, sub-horizontal drains may be installed into holes drilled into the in-situ soils. A perforated pipe wrapped in a geotextile filter material is inserted into the hole and connected to the drainage system (Day, 2015).

Furthermore, Koerner et al. suggests that a low permeability backfill material in a reinforced CRB wall can be rendered self-draining by using geotextiles or geocomposites along with the geogrid reinforcement (Koerner, et al., 2005).

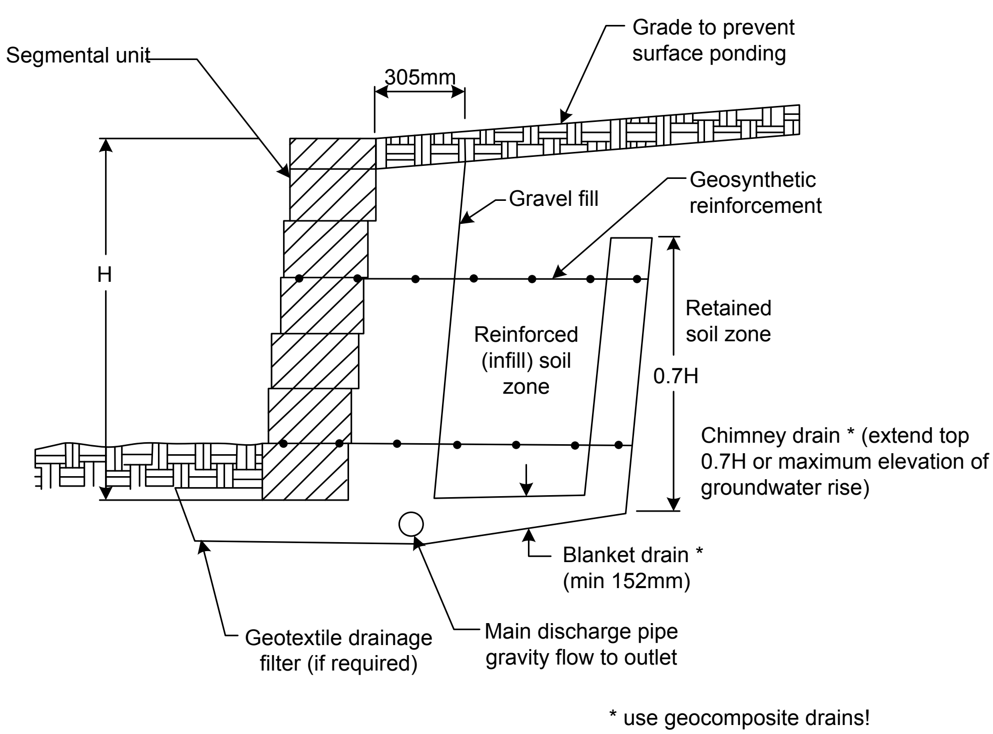


Figure 4: Back drain using soil (Koerner & Koerner, 2011)

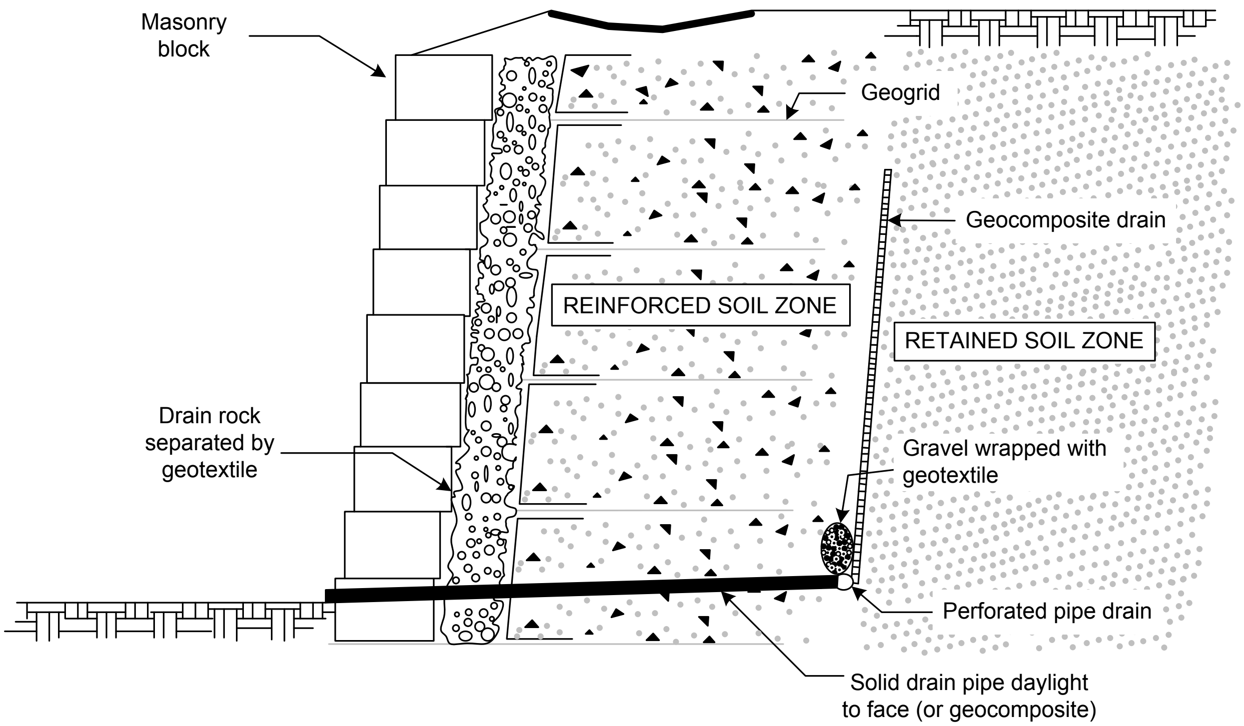


Figure 5: Back drain using geocomposites (Koerner & Koerner, 2011)

Koerner et al. studied the importance of drainage control for reinforced CRB walls with backfill materials of low permeability (Koerner & Koerner, 2011). This study referred to a previous paper where cases of collapsed and excessively deformed reinforced CRB walls were investigated. Poor drainage was the cause of 68% of the failures.

The study highlighted that most CRB wall designs ignore the possibility of groundwater pressures and assume that water will naturally drain away from the facing and reinforced zone. This system can function with no additional drainage measures if a free draining backfill is used. However, in practice, this is not normally the case as Koerner et al. found that 62 of the 82 cases studied had silt or clay as backfill material in the reinforced soil zone (Koerner & Koerner, 2009). In 80% of these cases, the backfill was poorly compacted. Koerner et al. (2005) stated that, if material of low permeability is used as backfill in the reinforced soil zone, drainage behind the reinforced soil zone is essential.

Koerner and Koerner (2011) listed five drainage control measures which should be considered in the design of reinforced CRB walls with backfill material of low permeability. These are discussed below.

2.4.4.2 Drainage control measures

High phreatic surface

The drainage control measures included in this section should be taken into consideration when CRB walls are constructed near or adjacent to standing or flowing water. When a flood occurs, loads from three sources act on the structure, namely hydrostatic loads, hydrodynamic loads and impact loads (Koerner & Koerner, 2011). These load cases subject lateral pressures and/or vertical buoyant forces onto the structure.

Koerner and Koerner (2011) state that failures arising from the aforementioned loads can be prevented by installing base drains beneath the entire reinforced soil zone and full wall length to the discharge outlets. Furthermore, free draining materials should be used as backfill material for a height equal to the maximum water level. The required permeability of this backfill is dependent on the rate of the rise and fall in the water level. Fine grained soil, soil encapsulation and light weight backfill materials should not be used and strong facing elements are required.

Retained soil drainage

If low permeability backfill is used, particularly in cuts with a high water table, hydrostatic pressures can develop behind the backfill.

To prevent this, a back-drain should intercept the water between the retained and reinforced soil zone and form a vertical continuation of the base drain as explained in Figure 4 and Figure 5.

Drainage from paved surfaces and adjacent structures

Rainwater runoff from these surfaces commonly flow towards the wall and is collected by a catch basin, inlet or manhole located in the reinforced soil zone. This water is often transmitted along the wall until it can be released at lower elevations.

Allowing the surface water to enter the reinforced soil zone is dangerous, especially if backfill material of low permeability is present and proper compaction control and inspection has been ignored. Koerner and Koerner (2011) point out that the outward movement of the facing could cause pipe leakage and breakage. Therefore, all piping should be routed away from the reinforced soil zone as explained in Figure 6.

Waterproofing backfilled surface

Water can easily accumulate on the ground surface and infiltrate into the backfill material without it being routed to the drainage system. A simple method of directing surface water away from a CRB wall system and preventing it from infiltrating into the backfill material would be to include a berm or swale.

If a horizontal surface is required making the provision of a berm or swale impractical, the entire reinforced soil zone and an adequate part of the retained soil zone should be provided with an impermeable cover to prevent the surface water from entering the system. Koerner and Koerner (2011) suggest a geomembrane covering should be implemented as waterproofing for the upper surface of the wall as explained in Figure 7. The factors which should be taken into consideration when selecting the waterproofing include the extensibility, flexibility and durability of the geomembrane. All water intercepted by this membrane should be discharged into the drainage system or be allowed to escape through the wall facing.

Tension crack sealing

Tension cracks commonly occur at the end of the reinforcement, especially when backfill material of low permeability is present. These cracks primarily occur when the reinforced soil mass settles or outward deformation of the facing occurs. Water fills the tension cracks and exerts hydrostatic forces onto the reinforced soil CRB wall system. Furthermore, when backfill materials of low permeability form an inherent stable block with the wall facing, translation of the reinforced soil zone could occur as the facing moves forward. As this movement progresses, blocks fall off their supporting reinforcement layer creating a cascading effect.

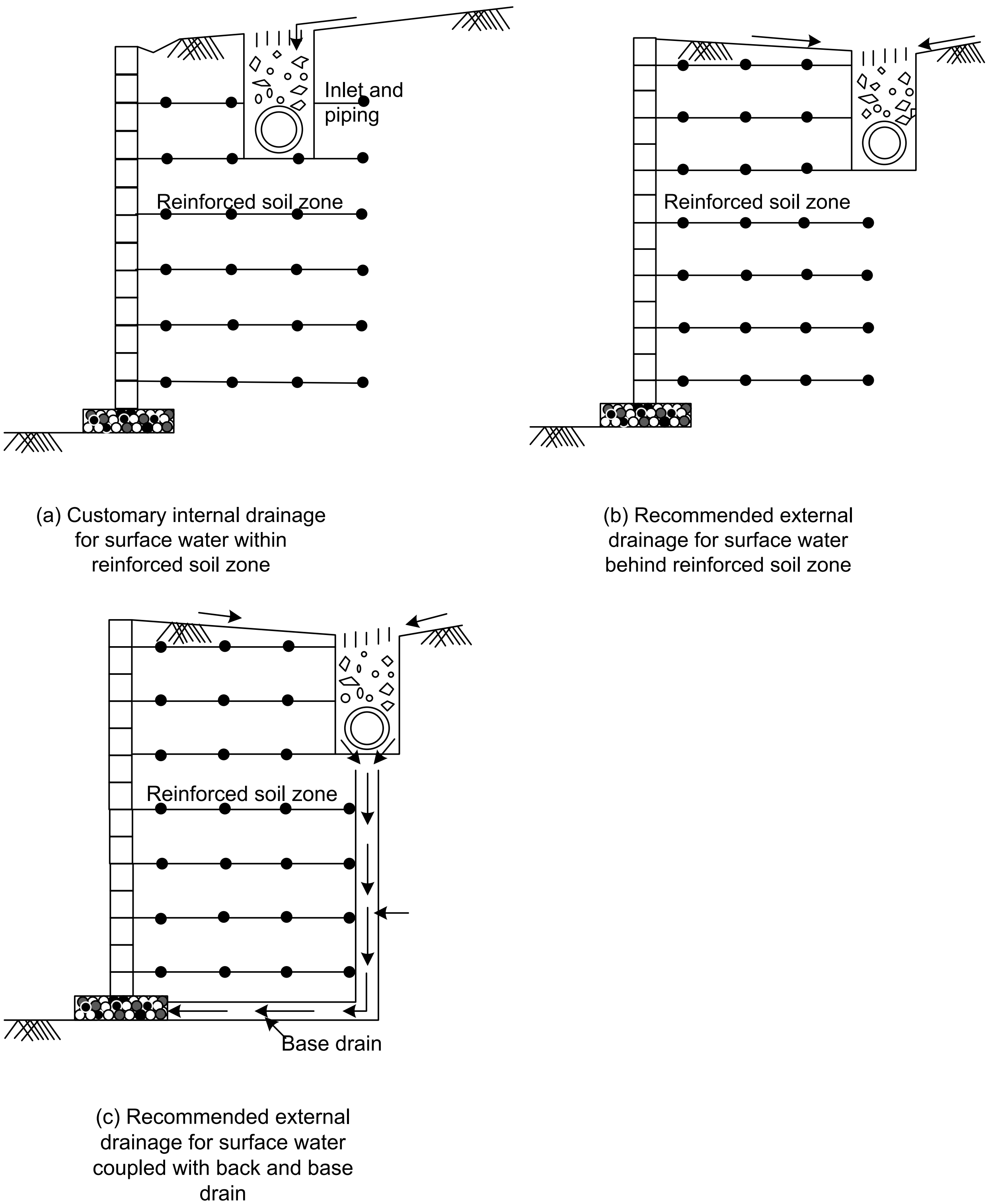


Figure 6: Shifting of the internal drainage system (Koerner & Koerner, 2009)

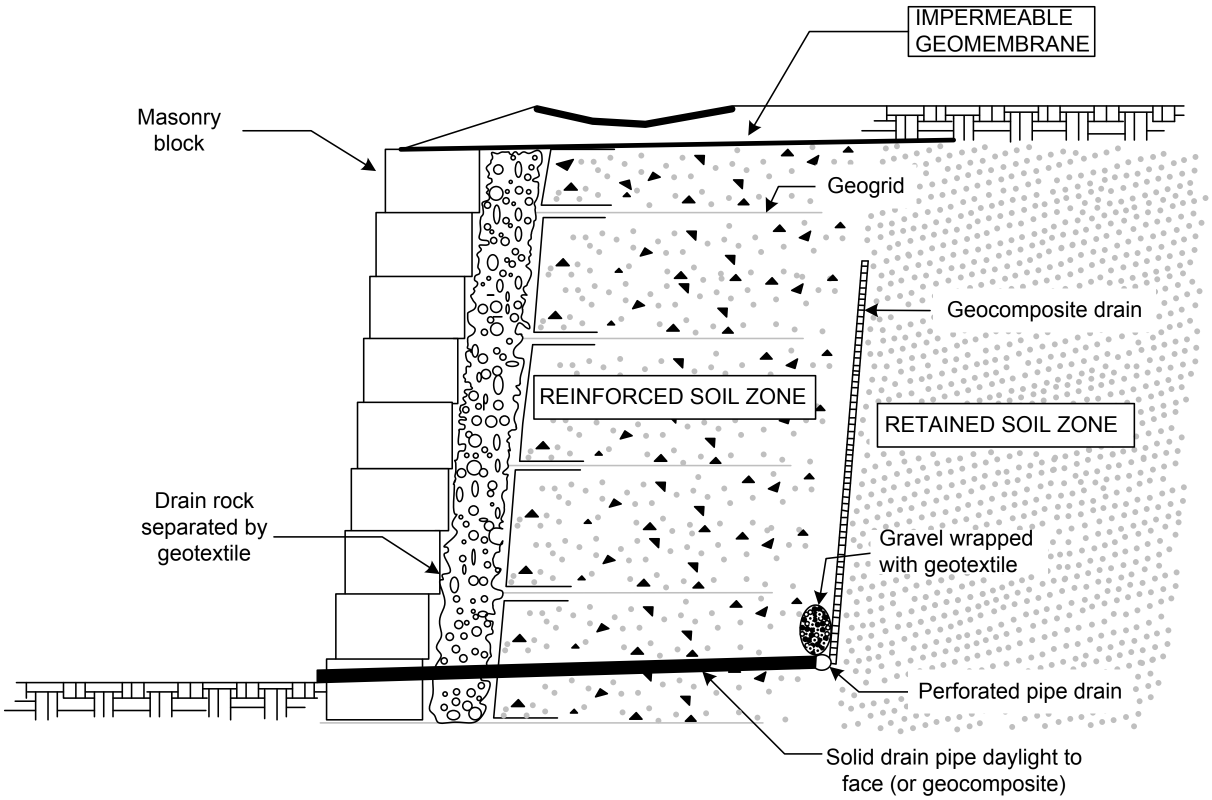


Figure 7: Use of a geomembrane as waterproofing above the reinforced soil zone (Koerner & Koerner, 2011)

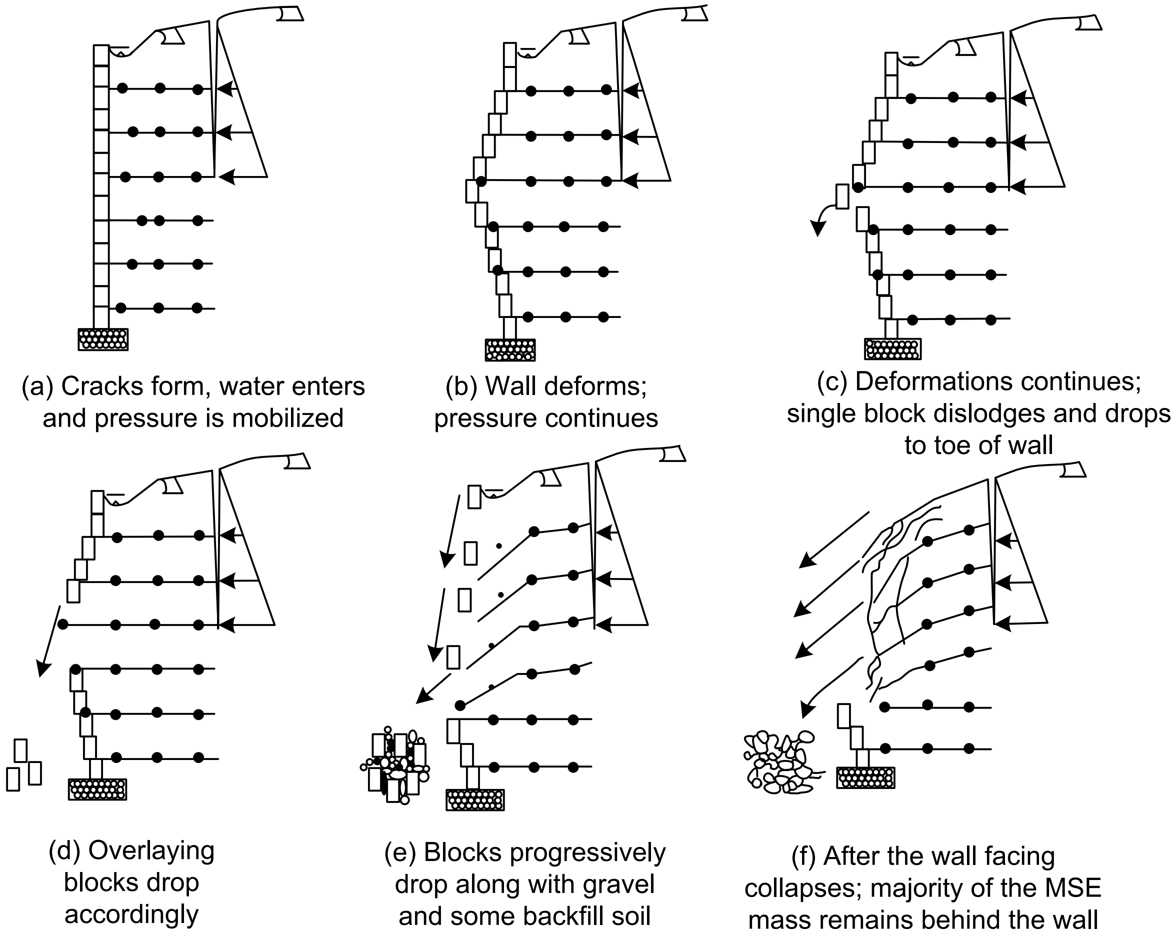


Figure 8: Modular block wall collapse due to hydrostatic pressures in the tension cracks (Koerner & Koerner, 2009)

By waterproofing the retained and reinforced soil zone with a geomembrane as discussed and shown in Figure 7, water will be restricted from entering the tension cracks that might develop and the failure in Figure 8 can be prevented. To effectively prevent water ingress into tension cracks behind the reinforced zone, the waterproofing membrane should extend a short distance beyond the end of the reinforcement.

Koerner and Koerner (2011) concludes that the entire reinforced soil should be encapsulated by waterproofing, above, behind and beneath, when fine grained soils of low permeability are present. Furthermore, granular soils should be used for base drains under the reinforced soil zone and it should extend up behind this zone in the form of a back drain using granular soils or geocomposite drains, to the maximum height of water elevation, or if water emerges from the retained zone.

Chapter 3

Design

3.1. Overview

Numerous design guidelines are available for gravity and reinforced CRB walls. The design manuals and checklist published by the CMA are well known in South Africa for the design of CRB walls. SABS 508:2008 covers the manufacturing of concrete retaining blocks. The British Code BS8006 focuses on the design of geosynthetic slopes. The South African counterpart of BS8006, SANS 207 was published by the SABS in 2011. A widely used design code for reinforced and soil nailed slopes is Advice Note HA68/94 issued by the Department of Transport of the United Kingdom (Day, 2015). Compliance with recognised codes and standards represent good practice, but many designers use non-standard methods or design methods available, copying design calculations and drawings of existing CRB walls, EXCEL spreadsheets, design charts, design guidelines or computer programmes, many of which are produced by block manufacturers. This chapter examines various aspects related to the design of CRB walls including factors to be considered, failure modes and commonly used design methods.

3.2. Design Methods in South Africa

Methodologies for the design and analysis of Reinforced CRB walls were published by the Federal Highway Administration (FHWA) of the United States of America and the American Association of State Highway and Transportation Officials (AASHTO) in 1989 and 1990 respectively. Following the FHWA and AASHTO design guidelines, a less conservative approach was adopted by the National Concrete Masonry Association (NCMA) in the USA.

The NCMA further included a design approach for the design of Gravity CRB walls. The design guidelines by the NCMA allows for a design approach which assumes that both the Gravity and

Reinforced CRB walls act as gravity structures. Computer Programmes which implement many of the recommendations in the NCMA design guidelines include GEOWALL (ver. 2.0), released in 1994, and SRWall released by the NCMA in 1995 (Bathurst & Simac, 1994).

Koerner et al. (2005) stated in the publication "*Back Drainage Design and Geocomposite Drainage Materials"* that there are numerous approaches to the design of Reinforced CRB walls. Only three of these approaches were studied, namely the modified Rankine Method, Coloumb per the NCMA and Coloumb per the FHWA design guidelines. The paper pointed out that the modified Rankine Method is more conservative than the FHWA design guidelines and the NCMA design guide is the least conservative. The Rankine theory over-estimates the lateral earth pressure acting on the reinforced CRB wall and, therefore, most guidelines adopt the Coloumb earth pressure theory approach (Bathurst & Simac, 1994).

CRB walls were introduced in South Africa by Terraforce (Pty) Ltd. more than 25 years ago. The first local design guidelines for gravity retaining walls were written by Knutton (Johns, 2008). Terraforce (Pty) Ltd. developed their own design manual and set up a number of installers across South Africa (TerraForce, 2015). The company started a catalogue of CRB wall failures which was later adopted by the CMA.

The CMA was established in the early 1970s and describes itself as"… the national co-ordinating body of precast concrete manufacturers in concrete retaining block walls, masonry, suspended floor slabs, paving units and roof tiles." (CMA, 2016). The design guidelines published by the CMA for CRB walls include the "*Code of Practice for Gravity Walls"* (2005), "*Design of Reinforced CRB Walls*" (2005), "*Concrete Retaining Block Wall Design Checklist*" (2013) and the "*Project Review: Engineering Considerations for Concrete Retaining Block Walls"* (1999).

Terraforce is listed as a member of the CMA. The Terraforce design manual, namely the "*Design and Installation Manual for Geosynthetic Reinforced Soil Applications"* by Alston and Bathurst (1996), focuses on 24 different generic designs and design charts for a specific height range of Reinforced CRB walls, subjected to good ground conditions. An example of such a design chart is given in Appendix A. The manual states that walls higher than 1.2m should be constructed with the assistance of a professional engineer. In South Africa, that would mean an engineer (PrEng) or engineering technologist (PrTechEng) registered with the Engineering Council of South Africa. This manual is based on the NCMA design manual for Segmental Retaining Walls.

The design manual should be used with caution as its application is limited by numerous factors such as:

- The blocks must be filled with concrete or well graded gravel;
- The foundations must consist of reinforced concrete and, if applicable, be protected from scour with a protective blanket of rip rap at the toe;
- The design charts do not incorporate a complete drainage system. A drainage system should be incorporated on a site-specific basis to ensure that hydrostatic pressures do not develop in the backfill of the Reinforcement Soil Zone (refer to Figure 17);
- The designs are standardized to use only Mitagrid 2T or equivalent reinforcing and Terraforce L18 and L22 blocks;
- The inclinations are limited to 5°, 10°, 20° and 30°;
- The backfill can be a densely compacted silt, clay, silt/sand mix or a densely compacted sand/gravelly sand with a minimum 300mm drainage fill of well graded sand and gravel or clear crushed stone directly behind the wall;
- The design charts provide the number of layers of reinforcement, vertical spacing and minimum length of the reinforcement for walls ranging from 1.4 to 4.0m;
- The top slopes above the crest of the wall to the horizontal is limited to 5°, 22° and 5° with additional surcharge loads pertaining to parked cars;
- The foundation soil is assumed to have a bearing capacity of 150kPa; and
- The global stability of the wall is not directly addressed in the manual and should be carried out using conventional limit equilibrium slope stability methods.

Furthermore, Alston and Bathurst (1996) refer to other design manuals including "*Guide to Terraforce L13 retaining walls*" and "*Design charts for Terraforce L18 and L22 blocks*" for conventional gravity

CRB wall design. If multiple rows of blocks are required, Alston and Bathurst further refer to design guidelines and spreadsheets (Alston & Bathurst, 1996).

MaxiForce ® 2000, design software capable of designing gravity and reinforced soil CRB walls, was launched by Terraforce (Pty) Ltd. It allows the user to enter various case-specific properties of the retained soil and infill soil, the segmental units, facings, surcharges and geotextile properties of the retaining wall system, and produces a graphical representation of the wall. Furthermore, it provides calculations and detailed information regarding the internal and external instability of retaining wall system under consideration, as well as details on the geosynthetic reinforcement chosen by the designer.

Numerous design programmes are available for the design of gravity and reinforced soil CRB walls. LofGenie® by Damon Clark Associates is one of the design software programs which is particularly relevant in the South African context as the programme is based on the CMA Code of practice for Gravity CRB wall design (2005). This design software produces the results of an optimal gravity CRB wall design in the form of a graphical screen plot. The user has an option between designing a GEOLOCK block wall, Terrace block wall, Loffelstein block wall and designing a wall with custom blocks.

3.3. Engineering Considerations for CRB Walls

3.3.1. Overview

In this section, the structural aspects of CRB walls will be discussed with reference to the CMA's publication "Project *Review – Engineering considerations for Concrete Retaining Block Walls*" (CMA, 1999), before focus is drawn to the design of gravity and reinforced soil CRB walls respectively. A brief discussion emphasizes some of the more important aspects which should be considered in designing CRB walls. Included in this discussion are the design considerations, structural economics, and the nature of the retained material, detailing and installation of gravity CRB walls as well as serviceability considerations in the design of reinforced CRB walls.

3.3.2. Design Considerations

The design of CRB walls must consider the ultimate limit state (ULS), which is associated with collapse, and the serviceability limit state (SLS), associated with excessive deformation. The walls are analysed using standard geotechnical engineering methods for concrete retaining walls with some amendments. Earth pressures acting along the wall is calculated using the Coulomb earth pressure theory. This theory is used as it considers the inclination of the wall, the top slope above the wall and the shear forces between the CRB blocks and in the retained material. Furthermore, the Coloumb theory accommodates the mobilized shear at the retained-reinforced soil interface and the shear at the interface between the retained soil zone and front drainage zone in the lateral earth pressure calculations.

CRB walls can fail due to external instability, internal instability, facing failure and global instability. Gravity CRB walls rely heavily on the strength of the backfill for their stability. Chemical or mechanical soil stabilization methods specifically cement or lime stabilization and geosynthetic reinforcement respectively, can be incorporated for additional stability. Incorporating reinforcement into the CRB wall system includes a few additional modes of failure e.g. pull-out of the reinforcement from the soil or the facing, rupture of the reinforcement and excessive elongation under load.

Bathurst and Simac (1994) identified the modes of failure for CRB walls presented in Figure 9:

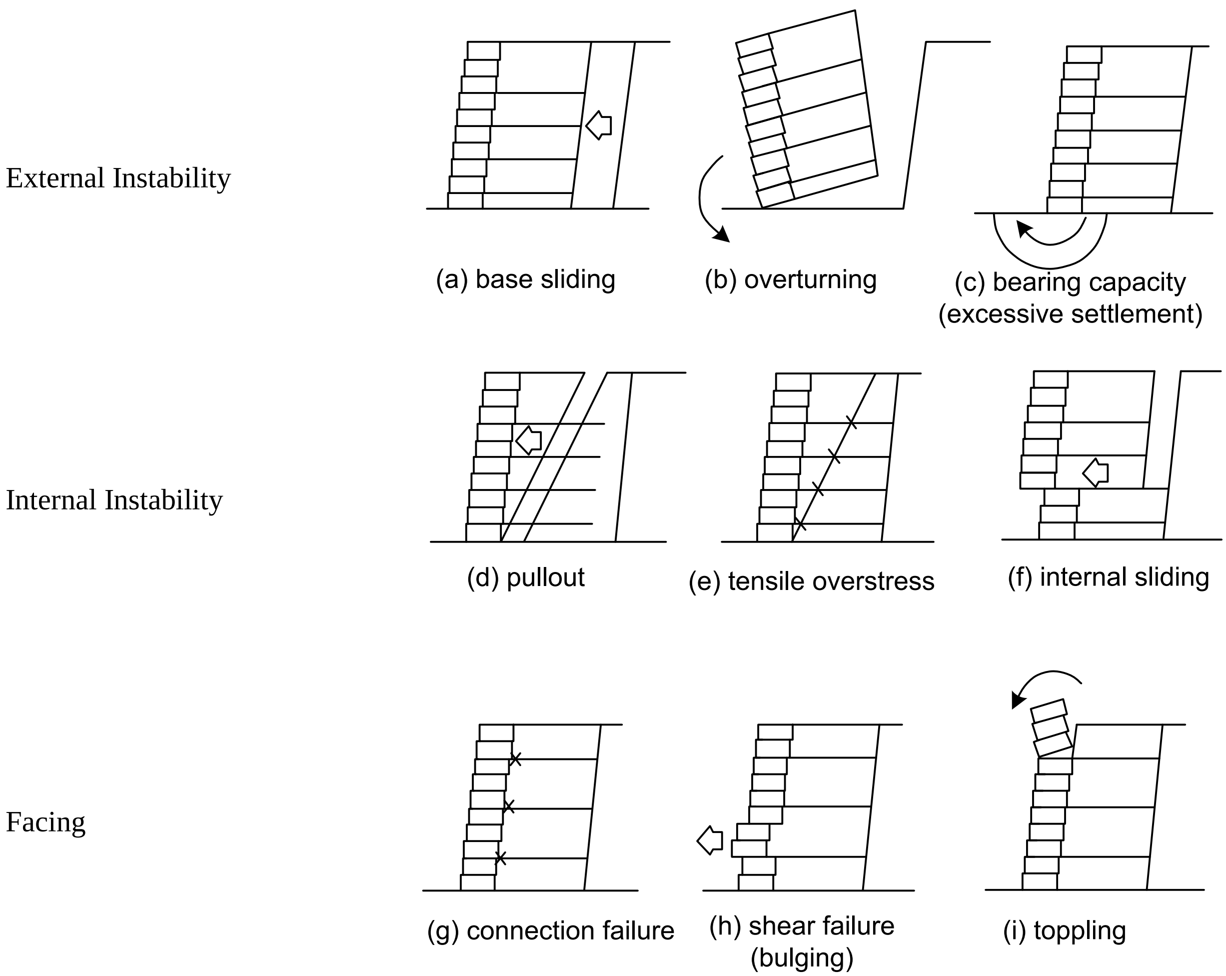


Figure 9: Basic modes of failure (Bathurst et al., 1994)

Global failure occur in the form of linear critical slip planes for gravity CRB walls, while circular slip planes are common for reinforced soil CRB walls. The slip planes for gravity CRB walls often occur at the backfill-existing slope interface, therefore benching into the existing slope is crucial.

As gravity CRB wall systems do not incorporate reinforcement, not all of the above-mentioned modes of failure are applicable. The relevant modes of failure for each type of wall are discussed in the appropriate chapters that follow.

3.3.3. Structural Economics

CRB walls can be an economical means of retaining soil. According to classical soil mechanic theories developed by Rankine and Coloumb, a failure wedge can occur along an inclined failure surface through the soil as seen in Figure 10. Due to the inclined facing of a CRB wall, the mass of the failure wedge is reduced, resulting in the significant reduction of applied forces acting on the wall. Therefore, light and cost effective elements can be used to retain this smaller failure wedge.

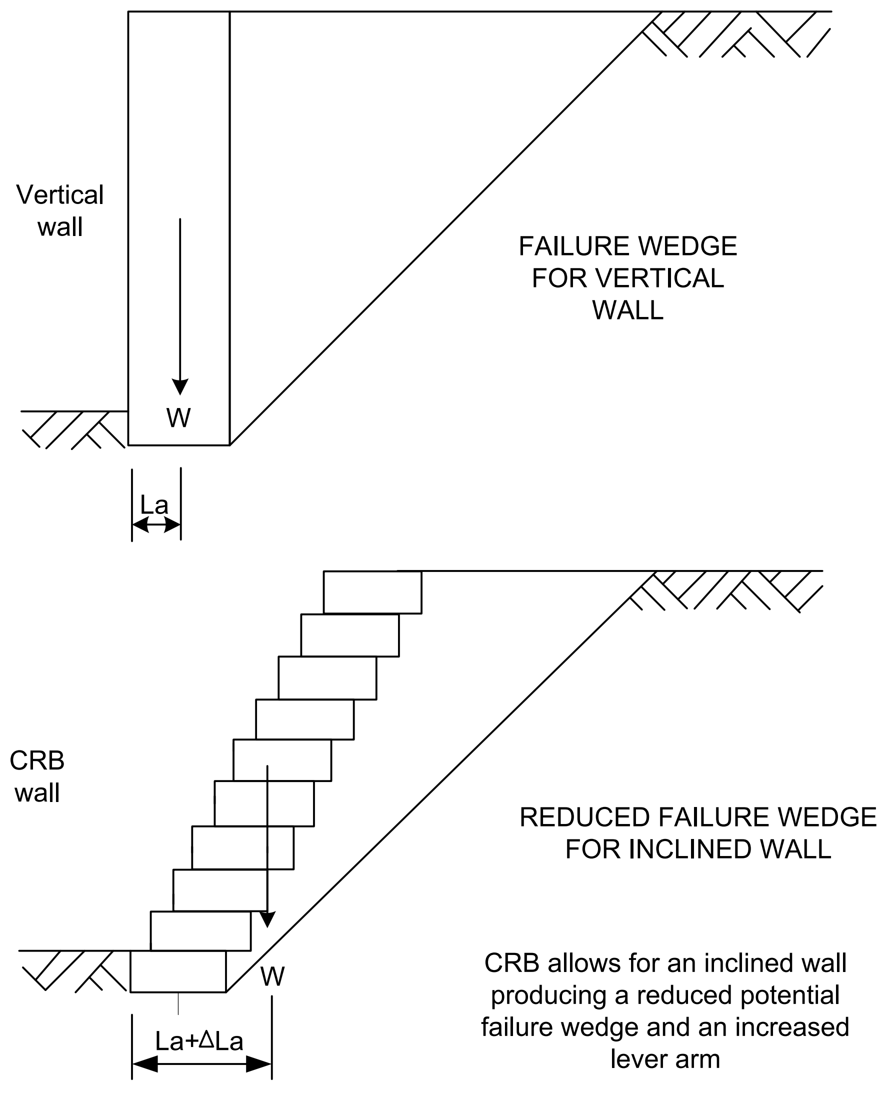


Figure 10: Reduced potential failure wedge (CMA Project Review, 1999)

Furthermore, this inclined facing increases the restraining moment, due to the weight of the wall acting further back from the toe as seen in Figure 11. In combination, these factors increase the effectiveness of the wall and reduce its cost.

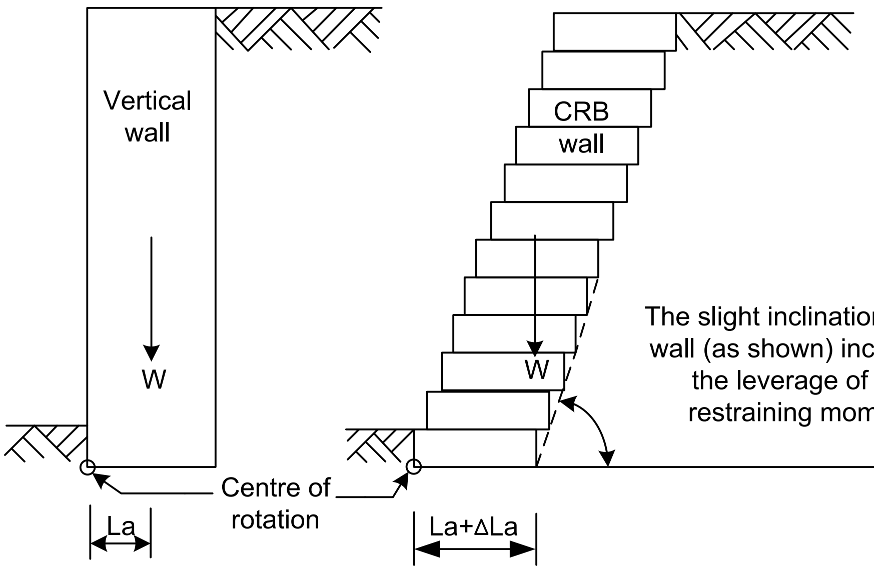


Figure 11: Increase in the leverage of the restraining moment (CMA, 1999)

These flexible retaining walls differ fundamentally from conventional reinforced concrete retaining walls which are more rigid structures. Slight outward movement of the retained material is expected during the construction phase. This movement occurs due to flexibility of the structure. Active earth pressures act on the wall as a result of the movement of the wall and the retained soil. These active earth pressures are substantially less than the at-rest pressure which would act on the wall in the absence of movement, hence the overturning moment is reduced. Furthermore, the weight of the backfill soil in the blocks contributes to the resisting moment.

Due to the flexibility of CRB walls, only flexible structures should be placed on the area immediately behind the wall. The CMA checklist includes an item which queries whether or not any structures are to be built on top of the retained fill, within a distance less than 1.5 times the height of the CRB wall (CMA, 2013).

3.3.4. Nature of the Retained Material

The CMA highlights that special precautions should be taken when expansive or collapsible soils are encountered, especially where groundwater is present (CMA, 1999). If the CRB wall is used to retain slopes cut into hillsides, the engineer has limited control over the properties of the in-situ soils. By reducing the angle of the slope or constructing a heavier wall using longer or multiple layers of blocks, these unfavourable conditions of the soil can be overcome.

Fill conditions are more favourable from a design point of view as the engineer has control over the properties of the backfill. An engineer can import adequate quality backfill material and specify the compaction of this backfill material to a required density. Furthermore, the engineer can incorporate chemical or mechanical stabilization into the system.

The most important engineering consideration, especially when the backfill material is not free-draining, is the incorporation of an adequate drainage system. Many CRB wall systems will fail when additional hydrostatic pressures develop behind the wall or in the backfill. The exact position and method of construction regarding drainage systems require careful consideration. Drainage was discussed in Chapter 2.4.4.

3.3.5. Detailing and Installation of Gravity CRB Walls

Important aspects regarding the foundations, tolerances, compaction, benching and backfill stabilization of gravity CRB walls are discussed in the CMA's Code of Practice for Gravity walls (2005):

3.3.5.1. Foundations

The foundation depth, as indicated in Figure 12, should take into account any excavation that could take place immediately in front of the wall. The depth of the deepest excavation should be accounted for in the design by adding this depth to the foundation depth and it must be confirmed by the engineer on site.

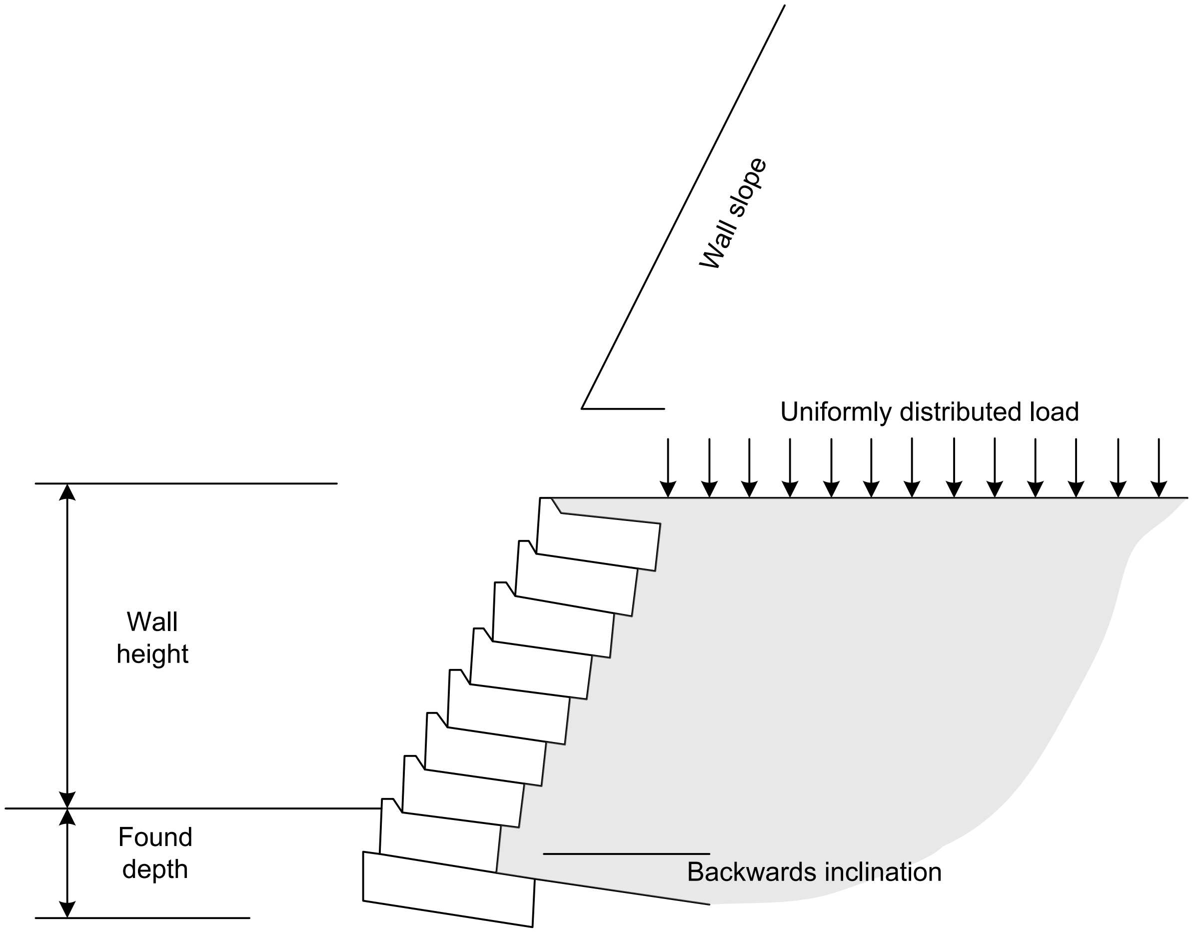


Figure 12: Definition of founding depth for a conventional gravity CRB wall (Clark, 2005)

The minimum depth and thickness for a CRB wall foundation is given in Table 1. The engineer must specify the minimum concrete strength, minimum width and thickness of the foundation. Unless it is suspected that localised weak points could exist below the foundation, foundations are generally unreinforced. If a key is required to assist with sliding, reinforcing should be considered, especially if the key is more than 250mm deep.

The factor of safety recommended by the CMA for foundation sliding resistance should be increased from 1.5 to 2 if buildings are in close proximity to the wall.

Table 1: Minimum allowable founding depth and foundation thickness (Clark, 2005)

CRB wall Height (m)	Minimum Allowable Founding Depth (mm)	Minimum Allowable Foundation Thickness (assuming 20MPa concrete) (mm)
< 1.2	300	100
1.2-2.0	400	150
2.0-3.0	500	200
3.0-4.0	600	200
>4.0	700	250

3.3.5.2. Tolerances

CRB walls should be installed with the rows of blocks laid horizontally and preferably not at an incline. The variation from the line/level should not exceed 20mm in 3m and should not exceed 50mm across the full length of the wall. CRB walls are designed for a specific inclination to the horizontal. This angle changes around corners, therefore the acceptable tolerances in deviation from the specified angles to the horizontal only apply to straight lengths of walls. These tolerances are plus one degree or minus two degrees of the designed wall inclination to the horizontal.

3.3.5.3. Compaction of the backfill

The soil inside the facing units must be compacted to at least 90% of its maximum dry density as determined using the modified ASSHTO compaction test, while the backfill material behind the blocks must be compacted to at least of 93% of its maximum dry density. Compaction of the soil should be done at or near optimum moisture content (typically ± 2%) and in layers not exceeding the block height.

3.3.5.4. Benching of the backfill

The backfill material must be benched into the existing competent material behind the backfill with a minimum bench width of 500mm. Benching reduces the likelihood of a slip plane forming at the interface between the backfill material and existing slope.

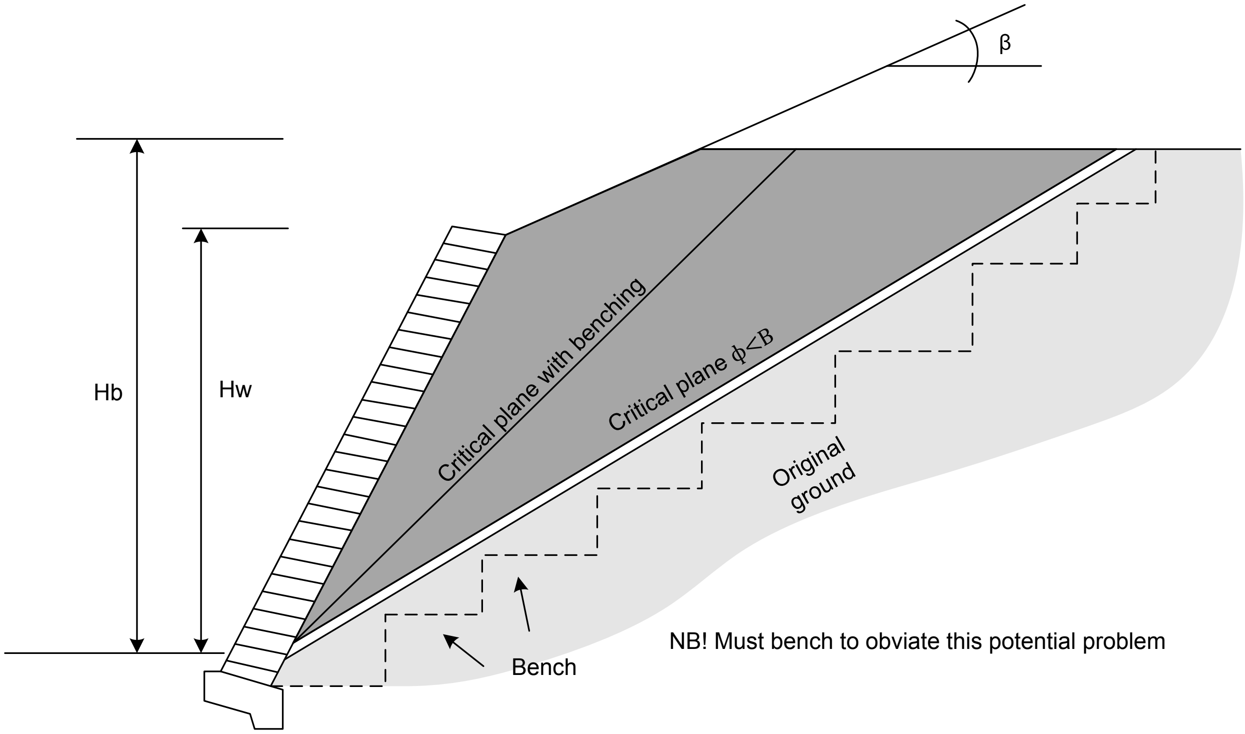


Figure 13: Benching of the backfill material (Clark, 2005)

3.3.5.5. Backfill stabilization

Stabilization of the backfill material increases the effective thickness of the wall and can be used as an alternative to increasing the length of the blocks or using multiple layers of blocks. A stabilized fill contributes to the total weight of a CRB wall.

The percentage of stabilising agent and the type of stabiliser agent used (typically cement or lime) depends on the type of backfill material and is determined by laboratory testing. The stabilised backfill material must form a uniform cemented material, strong enough for the required application. It is often necessary to import a good quality granular material for the stabilized zone. The percentage stabilizing agent should be specified as a percentage by weight. The unconfined compressive strength (UCS) of the stabilised backfill must be at least 2MPa and percentage stabilization should be such that even at the minimum acceptable percentage, the desired UCS of the material is still obtained. The stabilizing agent should be mixed into the backfill material prior to placement behind the wall to ensure adequate mixing.

The minimum degree of compaction of the stabilized backfill must be at least 93% modified AASHTO maximum dry density. Compaction should be done at the material's optimum moisture content and carried out in layers not exceeding the height of the block. Geotextile strips should still be incorporated to link the stabilized backfill to the blocks.

It is essential to incorporate a subsurface drain into the CRB wall system to remove any subsurface water behind the stabilized backfill. If "no-fines" concrete is used as backfill, the high permeability of this material allows it to act as a drain and the need for a subsurface drain is reduced or eliminated.

3.3.6. Serviceability Considerations in the Design of Reinforced Soil CRB Walls

As explained by Gassner (2005), certain serviceability considerations should be taken into account as they have a significant impact on the performance of reinforced soil CRB walls. These serviceability considerations include the following:

3.3.6.1. Saturation of the fill

When the fill becomes saturated, the weight of the backfill is affected, which results in a change in the total load applied to the wall and the reinforcement layers. This change should be assessed taking account of the strain in the reinforcement and the impact this will have on the wall system.

3.3.6.2. Moisture sensitive soils

Moisture sensitive soils should only be considered when an effective seepage and surface water management system is installed. When these soils become saturated, their strength and stiffness decrease substantially, causing the structure to deform. The deformation occurs as a result of the decrease in the stiffness of the fill material and increase in the weight of the fill, which subjects the reinforcement to larger loads. The reduction in volume of the fill material can cause surface settlement behind the wall.

3.3.6.3. Development of a phreatic surface

The development of a phreatic surface in the fill material is acceptable if the wall system has been designed for it. If a phreatic surface develops due to unforeseen circumstances, it can result in excessive deformation or even collapse. When a phreatic surface develops, the pull-out resistance of the reinforcement is decreased and the total load in the structure is increased. This could potentially result in collapse of the wall due to pull-out failure. A reinforced CRB wall would not necessarily collapse, but could undergo excessive deformation if the phreatic surface is located in the retained soil zone. In cases like these, the load exerted onto the CRB wall system is increased, but the pull-out resistance of the reinforced layers is usually not reduced; therefore, a subsoil drain should be placed behind the CRB wall or across the bottom of the reinforced soil zone.

3.3.6.4. Stiffness of the geosynthetic

The strain which occurs upon loading the reinforcement is governed by the stiffness of the geosynthetic. According to Gassner (2005), while some geosynthetics have an ideal linear stress-strain relationship, others have stiffness's which varies significantly over the stress range under consideration. In the latter case, excessive deformation could occur. Gassner (2005) states that it is good practice to limit the stress in the geosynthetic reinforcement to a stress range with an approximate constant stiffness.

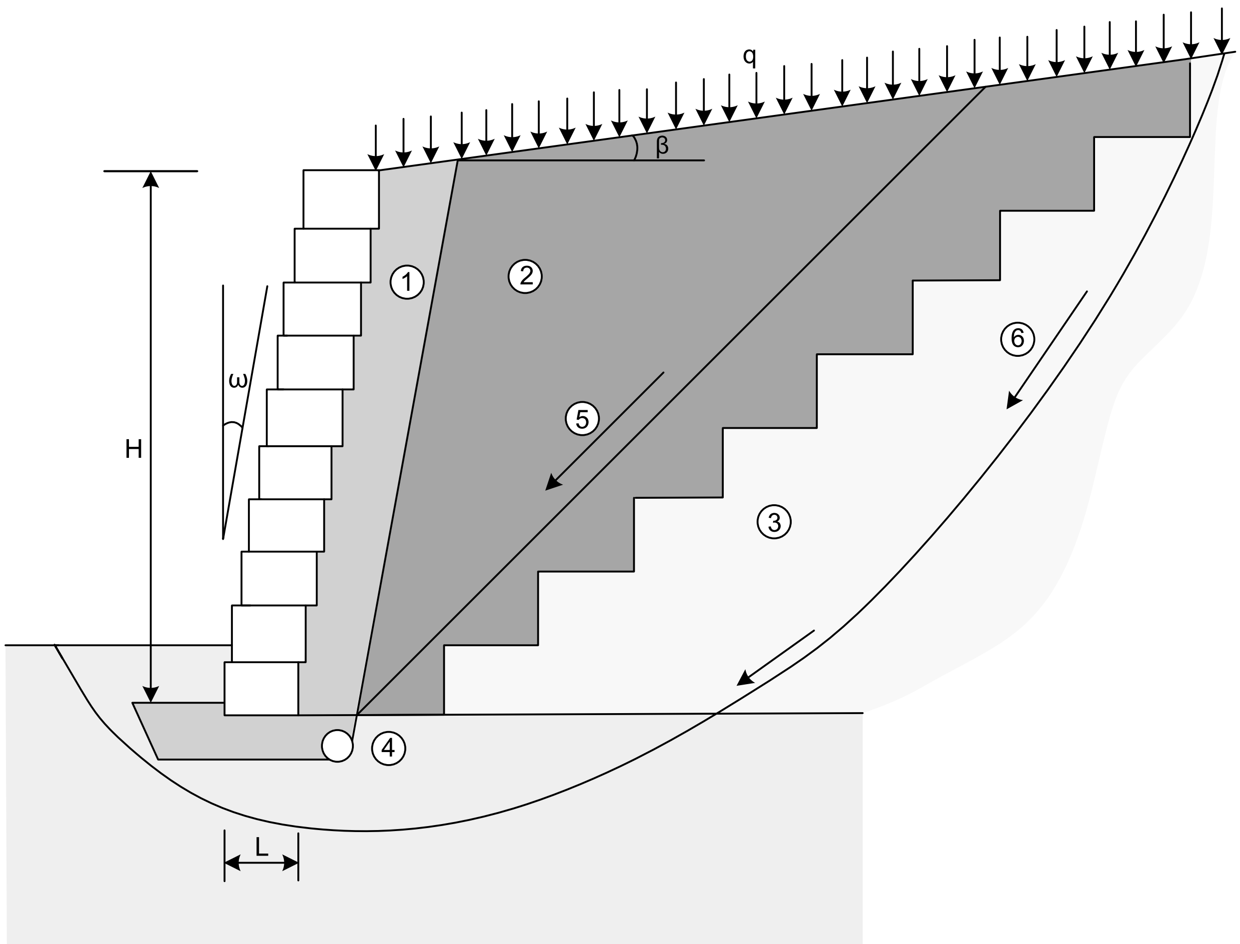
3.3.6.5. Creep of the geosynthetic

Creep of the reinforcement should also be considered as described in Section 3.5.1 which focuses on the general description and functioning of reinforced soil CRB walls.

3.4. Gravity Walls

3.4.1. General Description and Functioning

The second edition of the design guide "*Concrete Retaining Block Walls: Code of Practice for Gravity Walls"* (Clark, 2005) provides guidance to engineers in the design of gravity walls, testing of the concrete blocks, detailing and installation and various retaining conditions for gravity CRB walls.



1 Front Drainage Zone (±300mm wide gravel column)

2 Backfill Zone

3 Retained Soil Zone

4 Foundation Soil

5 Potential linear failure surface

6 Potential circular failure surface

Figure 14:Cross-section of a typical gravity CRB wall, figure adapted from "A data base and analysis of geosynthetic reinforced wall failures" (Koerner & Koerner, 2009)

Gravity walls primarily rely on the strength of the backfill material for their stability and on the self-weight and batter of the facing units. The stacked blocks and infill soil of a gravity CRB wall is assumed to act as a single body in the design.

3.4.2. Modes of Failure

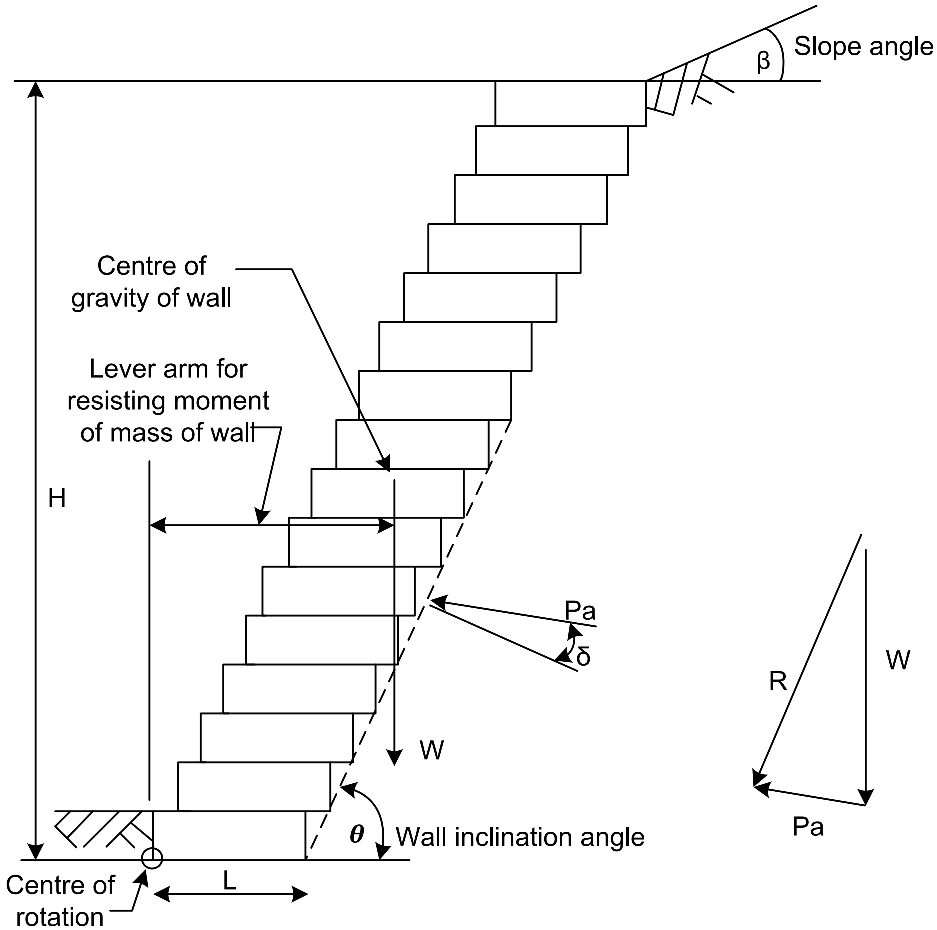


Figure 15: Forces acting on a typical section of a CRB wall (CMA, 1999)

The forces acting on a typical CRB wall system as shown in Figure 15 should be stable against the following modes of failure with a suitable safety factor.

3.4.2.1. External Instability

External instability includes:

- Overturning;
- Base sliding; and
- Bearing capacity failure.

By ensuring that the resistance moment exceeds the overturning moment with a suitable safety factor, the structure is stable against overturning. If the passive resistance at the front of the foundation, in addition to the friction on the underside of the footing, exceeds the horizontal component of the active soil force by a suitable factor of safety of 1.5 or higher, base sliding will be prevented. Furthermore, the structure can fail due to excessive deformation or inadequate bearing capacity of the founding soil.

3.4.2.2. Facing

Failure of the facing includes:

- Movement (sliding) between the block courses;
- Failure (crushing or cracking) of the blocks themselves; and
- Toppling of the upper section of the wall.

The inter-block friction generated by the weight of the blocks (plus infill soil) should exceed the applied sliding force at any height to ensure that the wall is stable against sliding between courses.

3.4.2.3. Global Failure

Global failure includes:

- Linear slip plane or circular slip plane failure which is a critical design aspect.

3.4.3. Typical Design Procedure for Gravity CRB Walls

A typical design procedure for gravity CRB walls is described as follows:

***STEP 1**: Decide on the soil parameters*

The first step when designing CRB walls is to estimate the shear strength and bulk unit weight parameters of the soil. The cohesion (c) and internal angle of friction ($\emptyset$) of the soil can be determined through the direct shear box test or the triaxial compression test for granular and clayey soils respectively. Often these tests are not conducted and the engineers base their designs on typical soil parameters as given in Table 2.

Table 2: Typical soil parameters (Clark, 2005)

Type of material	$\emptyset$ (^{0})	γ (kN/m^3)
Loose, sandy silt or clayey sand	25	18
Very loose, uniformly graded sand/ slightly silty sand	28	17
Loose, uniform sand, round grains or dense sandy silt	30	18
Dense or particularly cemented uniform sand or loose well-graded sand	33	19
Dense, well-graded sand – angular grains	35-40	20-22
Loose sandy gravels	35	19
Dense, sandy gravels	35-40	20-22

Due to the highly variable nature of the cohesive component of shear strength, the cohesion of the backfill is generally ignored and the backfill is designed as a purely frictional material, hence the soil parameters are solely based on the internal friction angle of the soil and the cohesion is assumed to be zero. The cohesion can be incorporated in the design by using the graphical wedge analysis to analyse the soil, but this technique is cumbersome and, therefore, not often used. Cohesion should only be used in the design when the designer is confident of its existence. In most design manuals it is recommended that the cohesion should be ignored.

The wall friction at the rear of the retaining wall (δ) is often assumed to be between 0.8 and 0.9 times the drained shear strength ($\emptyset'$) of the retained material. In the publication *"GEOLOK COMPUTER PROGRAM: A COMPUTER BASED DESIGN APPROACH FOR DRY STACKED RETAINING WALLS"*, Clark (2002) states that when the active wedge behind the wall is mobilized, the shear surface at the rear of the wall will be a full soil-on-soil contact.

According to Clark (2002), if a cast-in-situ foundation is used, the base friction (μ) is taken as being equal to the $\emptyset'$ of the underlying soil. Where precast foundation elements are used, the base friction between the underlying soil and the precast concrete should be taken as between ½ and ⅔$\emptyset'$.

***STEP 2**: Select a trial wall inclination*

As the wall inclination is dependent on numerous factors including the height to be retained, properties of the retained material, ground slope behind the wall, external loads and the block type and size, it is difficult to select an initial trial walls inclination which closely represents the final wall inclination.

An iterative process is used to determine the final design wall inclination, starting with a wall inclination between 65^0 and 70^0 if no space constraints are present. The wall is flattened until a design which meets the desired criteria is achieved.

Where sloping backfill is present, flattening the inclination of the wall increases the maximum height of the wall.

The limit beyond which no significant improvement will be gained by flattening the wall inclination is dependent on the ground slope behind the wall, the shear strength as well as the weight of the retained material. Based on the design requirement in which the effective weight of the wall should be reduced if the line of action of the forces passes behind the bottom row of blocks, no significant improvement will be gained by flattening the wall to an inclination of less than 60^0 for a level backfill, or an inclination of less than 55^0 with a sloping backfill up to 26^0 (Clark, 2005).

If space constraints exist and flattening the inclination of the wall is no longer an option, stabilisation or reinforcement of the backfill material may be required if the blocks alone cannot retain the desired height of fill.

***STEP 3**: Calculate the earth pressure*

The active earth pressures on the wall are calculated based on the assumption that a wedge of retained material bounded by a failure plane behind the blocks and a critical failure plane within the backfill moves downwards (Clark, 2005). According to Clark (2005), the Muller-Breslau method is generally used to calculate the active pressure. This method assumes a purely frictional soil and allows for a sloping backfill, a sloping back face to the wall and friction on the back face of the wall.

The external forces, in terms of point loads, line loads or uniformly distributed loads (UDLs), at various distances from and orientations to the wall, increases the horizontal stresses which act on the wall. This increase can be calculated using standard elastic solutions (Clark, 2005). The effect of a UDL behind the wall (e.g. a road) can be represented as an extra height of soil or as a uniform stress on the back of the wall equal to the coefficient of lateral earth pressure times the surcharge loading.

***STEP 4**: Calculate the resultant force*

The destabilizing forces which act on the wall include the active force due to earth pressures and forces acting on the wall due to external loads. These resultant destabilizing forces act on the back of the wall at an angle from a line perpendicular to the wall equal to the wall friction. This force is split into its horizontal and vertical components as seen in Figure 16. The resisting force is the total/effective weight of the wall which includes the blocks and the infill soil.

Figure 16 shows a wall consisting of three sections, reducing in width from the bottom of the wall. The weight (W) of each of the three sections is shown. It also shows the vertical and horizontal components as well as the resultant of the earth pressures (Q) from the retained soil (Qa), the uniform surcharge from behind the wall (Qu) and the line load surcharge (Ql). Note that in context of retaining walls, “behind” the wall refers to the side of the wall where the retained soil is situated.

$$\boldsymbol{Q_{av}} = \boldsymbol{Q_a \, sin}(\boldsymbol{\delta} + \boldsymbol{\alpha} - \mathbf{90}°)$$

$$\boldsymbol{Q_{ah}} = \boldsymbol{Q_a \, cos}(\boldsymbol{\delta} + \boldsymbol{\alpha} - \mathbf{90}°)$$

$$\boldsymbol{Q_{uv}} = \boldsymbol{Q_u \, sin}(\boldsymbol{\delta} + \boldsymbol{\alpha} - \mathbf{90}°)$$

$$\boldsymbol{Q_{uh}} = \boldsymbol{Q_u \, cos}(\boldsymbol{\delta} + \boldsymbol{\alpha} - \mathbf{90}°)$$

$$\boldsymbol{Q_{lv}} = \boldsymbol{Q_l \, sin}(\boldsymbol{\delta} + \boldsymbol{\alpha} - \mathbf{90}°)$$

$$\boldsymbol{Q_{lh}} = \boldsymbol{Q_l \, cos}(\boldsymbol{\delta} + \boldsymbol{\alpha} - \mathbf{90}°)$$

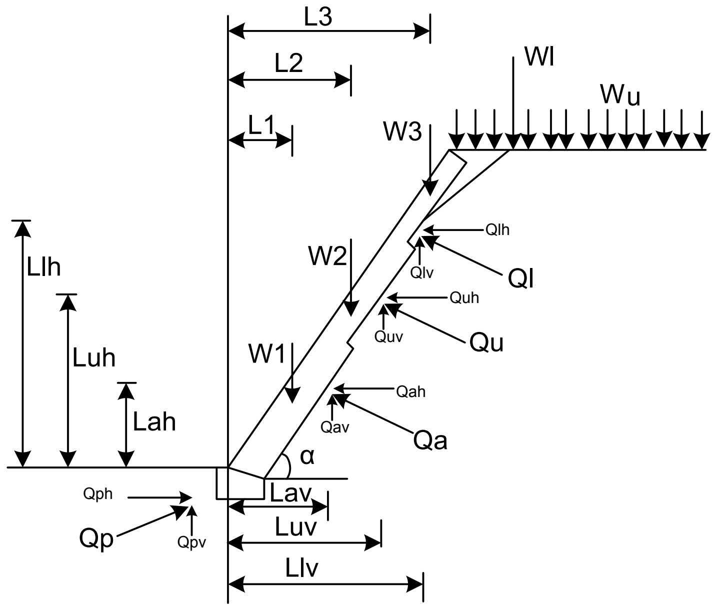


Figure 16: Forces to be considered in the analysis of a conventional gravity CRB wall, figure adapted from CMA design manual for gravity CRB walls (Clark, 2005)

***STEP 5**: Check the line of action of the resultant force*

By taking moments of the horizontal and vertical components of the resultant destabilizing forces, and the effective weight of the wall, about the toe of the bottom row of blocks, and dividing this total moment by the vertical component of the total resultant force, the line of action can be computed. This line of action must pass within the blocks and not behind the back of the bottom row of blocks. If this condition is not satisfied, the effective height and corresponding effective weight should reduce until the line of action passes within the blocks. The design procedure allows the blocks at the top of the wall to lie on top of and be supported by the retained material slope while not contributing to the sliding resistance at the base of the wall. When this line of action passes behind the blocks, the potential exists for the blocks to rotate and for the facing to buckle forward.

This design requirement allows the line of action to extend as much as $\frac{1}{2}L$ behind the centreline and is more of an overturning consideration than one of preventing tension. If the middle third rule is adopted, the line of action is restricted to $\frac{1}{6}L$ in front of the centreline of the wall and $\frac{1}{6}L$ behind the centreline. This ensures that the full length of the blocks is in compression and tension does not develop.

When this line of action passes outside the middle third, the width of the compression block decreases, increasing the compressive stresses on the blocks. Hence, the ability of the block to carry the increased compressive stresses should be checked.

***STEP 6**: Check for overturning*

The resistance and overturning moments about the second to lowest row of blocks can be used to calculate the factor of safety against overturning of the wall. The overturning is considered about the second to lowest row of blocks as it is common practice to set the bottom row in wet concrete to form part of the foundation. The resistance moment is the product of the weight of the wall and the distance from the centre of rotation to the centre of gravity of the wall. The overturning moment is the lever arm of the horizontal destabilizing forces about the centre of rotation as seen in Figure 15. The lever arm of the vertical component of the destabilizing forces about the centre of rotation contributes to the total resisting moment.

When a wall consists of wider units below and narrower units above, Clark (2002) considers whether the weight of the “slither” of soil behind the upper blocks within the projection of the back line of the wider blocks below should be included in the overall weight of the wall when determining the resisting of the wall to overturning and sliding. For a typical CRB wall with a slope of 70°, it is unlikely that a long narrow inclined portion of the soil would act as part of the wall. A relatively small wedge of soil above the top row of the larger blocks may contribute to the top of the wall, but this wedge is small enough to be neglected.

It is common practice to assume that the active force due to the backfill behind the wall is applied at a third of the total height of the wall. The earth pressure due to a uniformly distributed surcharge is applied at half the total height of the wall.

***STEP 7**: Check the mode of failure against block-on-block sliding*

The factor of safety against block-on-block sliding should be calculated between the bottom two rows of blocks. Nib shear strength should only be taken into account if the wall is constructed so that each block is placed hard up against the nibs of the blocks below. By adding the angle of the backward tilt of the blocks to the design inclination of the wall, the maximum inclination of the block wall can be calculated.

It is important to ensure that the nibs interlock. If concrete keys are used instead of nibs, they should be included in the block-on-block sliding resistance in a similar manner to the nibs (Clark, 2005).

***STEP 8**: Determine a suitable founding depth*

Determination of the minimum founding depth is a trial-and-error process, therefore, the initial trial founding depth is either assumed by an experienced engineer or taken as 0,5m. The initial trial founding depth is used in the calculation of the factor of safety against foundation sliding. By dividing the resisting force by the mobilising force, the factor of safety is determined. If this calculated factor of safety is too low, the founding depth should be increased until a suitable factor of safety has been obtained.

The height of the wall to be used in the calculation of active pressure is measured from the base of the foundation and not only the height of the retained soil mass as shown in Figure 15. The passive pressure is produced by the foundation pushing against the soil in front of the wall. The Muller-Breslau method is used to calculate the passive force using the founding depth as the “height” of the wall. For both the active and passive pressure states, the solutions assume that the soil is frictional, rigid and cohesion-less and that the failure occurs on a critical discrete planar shear plane as described by Clark (2002). To allow for cohesion in the soil and for cases where the ground is not horizontal in front of the wall, graphical techniques are available which assumes a combined curved and planar slip surface (Clark, 2005).

***STEP 9**: Check mode of failure against excessive settlement*

The bearing pressure is checked beneath the back and front of the foundation. The foundation is treated as an eccentrically loaded foundation and the applicable standard method to calculate foundation pressure is used. If the foundation pressure beneath either the back or front of the foundation is found to be too high, the foundation width should be increased.

***STEP 10**: Optimize the block mix*

The blocks should be optimized through the incorporation of as many smaller blocks as possible while still meeting the design criteria limits.

***STEP 11**: Repeat if the design criteria limits are not satisfied*

If the design criteria limits are not met, one can do either one or both of the following and repeat the analysis:

- Flatten the walls slope;
- Increase the effective width and weight of the wall.

***STEP 12**: Check global stability*

Assess the possibility of a deep seated slip failure passing beneath the wall and ensure global failure is prevented. This applies particularly where sloping ground is present in front and/or behind the wall or where the in-situ soils are weak. A conventional slope stability analysis is used in this assessment.

3.4.4. Design Example

The gravity CRB wall design example is attached at the end of this report in Appendix C.

3.4.5. Comments on the CMA Design Manual for Gravity CRB Walls

From the design example in Appendix C, certain shortcomings have been identified in the CMA design manual for gravity walls.

As a general comment, the calculation method would have been more transparent and less prone to calculation errors if the forces and moments had been expressed in terms of a defined coordinate system. Instead, the calculations use the vertical and horizontal components of the forces acting on the wall to determine the resultant force at the base of the wall (R) and its angle of inclination (ψ). Then, to determine the eccentricity of the load on the wall foundation, the vertical component of the resultant force is re-calculated in section 2.9 of the manual using an incorrect angle of inclination ($\psi + \omega$) instead of ψ.

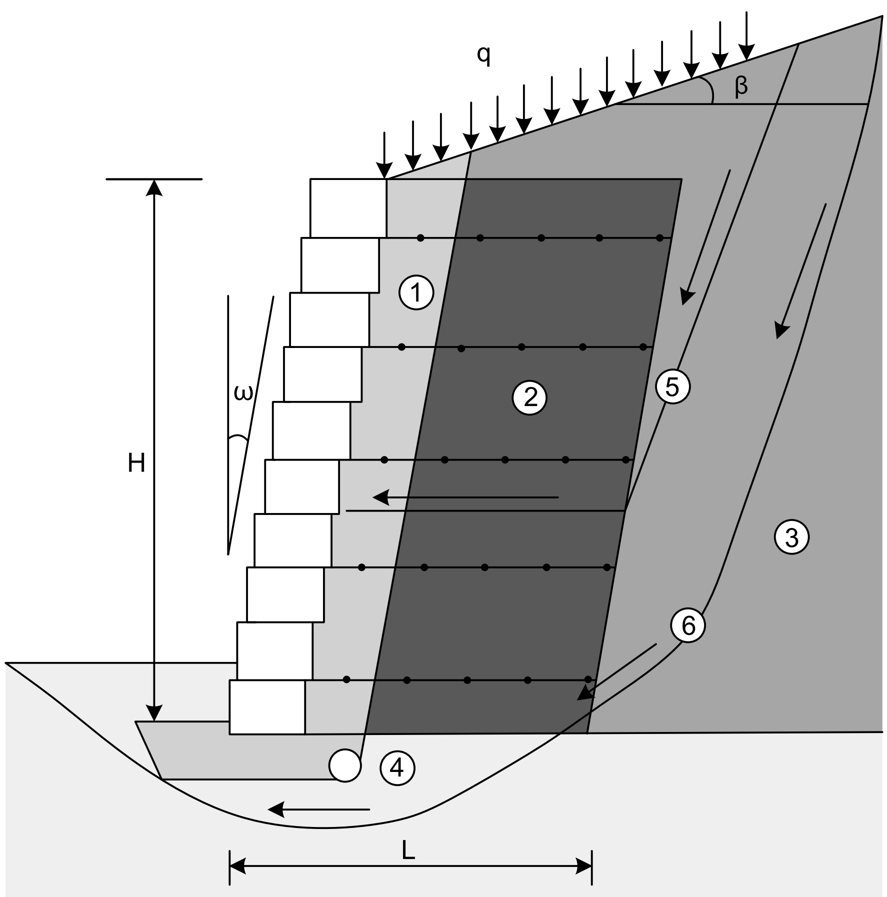
The angle α is used in three different ways in the calculations. In Figure 6, α is the forward inclination of the back of the wall measured from the horizontal. In Figure 9, it is the backward inclination of the back of the wall. Finally, in section 6, α is used as the angle of inclination of the resultant force which was defined in Figure 9 as ψ.

In common with many working stress design methods, the definition of the factor of safety is not necessarily unique. In calculating the factor of safety against overturning in Section 2.6 of the manual, the moment due to the vertical component of earth pressure is treated as a resisting moment and the moment due to the horizontal component as an overturning moment. This is in spite of the fact that the vertical component of earth pressure can act upwards or downwards depending on the angle of inclination of the back of the wall and the angle of wall friction. It is recommended that earth pressure be treated as a single force and that the moments caused by both the vertical and horizontal components of earth pressure should be regarded as overturning moments.

3.5. Reinforced Walls

3.5.1. General Description and Functioning

The second edition of the design guide "*Concrete Retaining Block Walls: Design of Reinforced CRB Walls"* (Gassner, 2005) provides guidance to engineers for the design, construction and serviceability of reinforced soil CRB walls.



1 Front Drainage Zone (± 300mm wide gravel column)
2 Reinforced Soil Zone
3 Retained Soil Zone
4 Foundation Soil
5 Potential piecewise linear failure surface
6 Potential circular failure surface

Figure 17:Cross-section of a typical reinforced CRB wall (Koerner & Koerner, 2009)

Gassner (2005) states, *'A reinforced CRB wall system behaves in such a way that the system is an interaction between the retained soil, backfill, reinforcing elements, foundation and the facing'*. The reinforcement allows the reinforced backfill material to act as a unit, allowing the system to be treated as a large, simple gravity wall in conjunction with the wall facing (Block, 2010). Most of the loads exerted on the CRB wall are resisted by the backfill and the reinforcement in the backfill. The reinforcement in the backfill provides horizontal stability to the system. The facing is usually of flexible construction and lightly loaded. Typically, the backfill reinforcement will develop loads in the facing that are higher than the self-weight of the facing. Vertical stability and support is provided to the facing through its foundation. Usually provision is made for a full vertical component of the active forces from the backfill, in addition to the self-weight of the facing.

Additional considerations regarding foundation settlement and loading of the reinforcement are applicable to reinforced soil CRB walls. These additional considerations are discussed below.

When assessing the settlement of the foundation, the combined effect of the bearing pressure from the facing and the bearing pressures from the reinforced backfill should be taken into account. Gassner (2005) explains that the deflection and settlement of the wall system is primarily influenced by the stiffness of the backfill material and the stiffness of the reinforcement over the range of strains in the backfill material layers due to the service loads on the CRB wall. Other factors that influence the deformation of the structure include the time over which the loads are applied and the type of backfill used. Consolidation of the in-situ soil and creep settlement of the backfill must also be considered.

The reinforcement is placed and becomes incrementally loaded as the backfill is placed and compacted. The tension in the reinforcement varies along the length of the reinforcement with the peak tension occurring near the Coloumb failure surface. The Coloumb failure surface for a vertical-faced CRB wall with a horizontal, granular backfill is $45°+\emptyset'/2$ up from the horizontal, starting at the toe of the fill behind the wall facing.

Polymer based materials have high strengths when the loads are applied for a short time period. However these materials are subject to creep overtime. The creep rate increases at high stress levels. Therefore, geosynthetic polymer based reinforced structures are ideal to resist dynamic loads such as earthquakes loads. Gassner (2005) explains that the reinforcement is installed from the bottom up as the fill is placed. Extension of the reinforcement allows active pressure conditions to develop in the retained material. The tension in the reinforcement at the facing connections varies over the height of the wall, with the highest tensions occurring near the bottom of the wall and reducing towards the top of the wall. Imposed loads near the top of the wall control the tensions in this vicinity.

For long-term loads such as dead loads, the permissible tension in the reinforcement is factored down to limit creep to acceptable levels. These limits are established from long-term tests and aim to restrict deflections to levels similar to those in other engineering structures (Gassner, 2005).

3.5.2. Modes of Failure

All modes of failures as discussed in Section 3.3 are considered for reinforced soil CRB walls. The ULS and SLS conditions explained in 3.3.2 are divided into internal and external modes of failure according to the limit state approach as explained by Pequenino et al. (2015). They explain that external modes of failure include base sliding, overturning and excessive settlement, as well as global failure and are dealt with in Section 9.5 of SANS 207:2006. Similarly, internal modes of failure directly relate to the reinforcement and are dealt with in Section 9.6 of SANS 207:2006.

Pequenino et al. (2005) includes that external modes of failure are generally evaluated by the design engineer, while internal modes of failure are generally evaluated by the supplier.

3.5.3. Typical Design Procedure for Reinforced Soil CRB Walls

***STEP 1**: Check the external stability*

Base sliding and overturning

By treating the wall and reinforced backfill as a rigid stable block, conventional slope stability methods can be used to check the stability of the wall against sliding and overturning.

The active force on the wall is assumed to act at the back of the reinforced soil zone and is broken up into its vertical and horizontal components. The vertical component of the resultant active force is added to the total weight of the wall and multiplied by the coefficient of friction between the reinforced soil mass and the underlying soil (Block, 2010). The vertical component of the total resisting force is then divided by the horizontal component of the resultant active force to calculate the factor of safety against sliding.

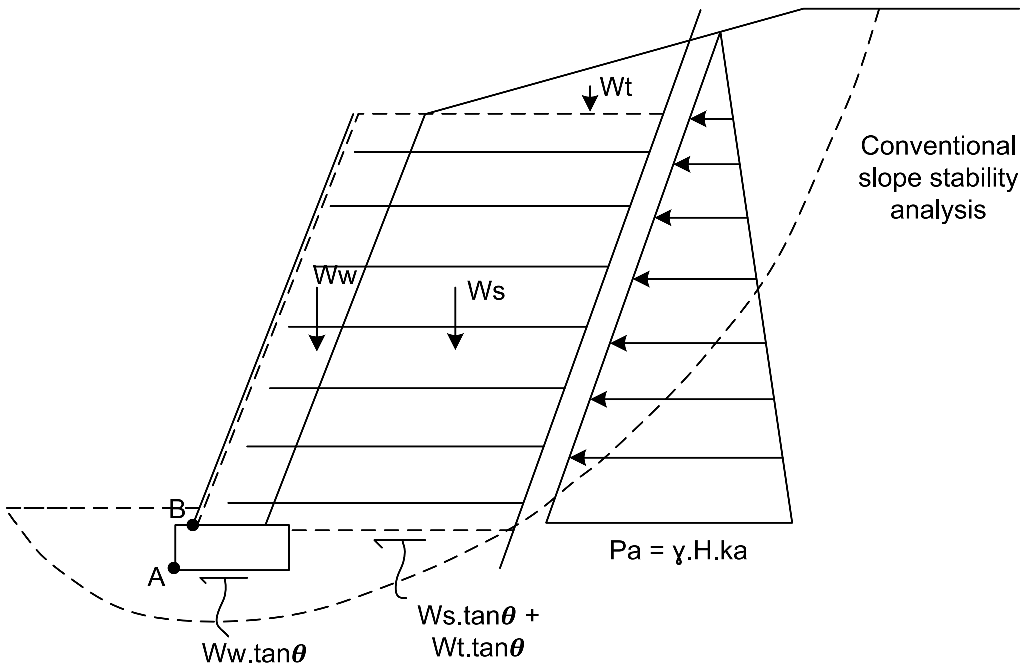


Figure 18: Overall stability of a reinforced CRB wall, figure adapted from CMA design manual for reinforced CRB walls (Gassner, 2005)

To calculate the factor of safety against overturning, moments about point A and point B in Figure 18 should be checked to assess the worst condition. The resisting moments, which include all vertical forces taken about the point A or B, are divided by the overturning moments. These overturning moments include the horizontal component of the active force about the same points. The ratio of resisting moments to overturning moments is the factor of safety against overturning.

The minimum factor of safety against sliding and overturning should be larger than 1.5 for normal applications. The Alan Block Engineering Manual (2010) suggests a more conservative factor of safety against overturning of at least 2 (Block, 2010). If the desired factor of safety is not achieved, the reinforcement length can be increased and the calculations repeated. A rule of thumb, in granular soils with simple loading and no groundwater, is that the width of the wall including the reinforced soil zone should be approximately 80% of the wall height.

Bearing capacity and excessive settlement

Since the CRB wall is treated as a rigid stable block, the applied bearing pressure on the foundation soils below the wall can be calculated using the Meyerhof method for inclined eccentric loading. The foundation pressure is then compared to the allowable bearing pressure of the foundation soil to find the factor of safety. The minimum factor of safety for bearing pressure should be taken as 3.

Settlement of the wall and the block of reinforced soil should be calculated using conventional settlement analysis methods. The compressibility of the founding soils can occur up to a depth of at least 1.5 times the combined width of the wall and the reinforced soil block. The higher bearing pressures near the toe of the wall will cause the wall to tilt. Excessive tilting can lead to the formation of tension cracks behind the reinforced soil block. Creep settlement of the backfill should also be considered for high walls.

***STEP 2**: Check the internal stability*

The internal stability of the CRB wall is the ability of the backfill soil to work in conjunction with the reinforcement to form a stable block. The internal stability is compromised when the permissible tensile force in the reinforcement or the maximum pull-out resistance of the reinforcement in the ground is exceeded.

A preliminary layout and strength of the reinforcement is calculated using Rankine's earth pressure theory considering an active failure wedge in the backfill, starting at the toe of the wall behind the blocks. The reinforcement spacing is governed by the height of the blocks and a preliminary vertical spacing is assumed to be 0.3 times the reinforcement length, calculated using the overall stability, bearing capacity and settlement criteria discussed previously. The preliminary load in the reinforcement per unit length is calculated by multiplying the vertical spacing with the calculated horizontal pressure at the level of the reinforcement. The calculated force should not exceed the permissible force in the reinforcement or the pull-out resistance of the embedded length of the reinforcement behind the failure plane.

The internal stability is calculated through a trial-and-error process where the preliminary layout and force in the reinforcement layers are used in a wedge analysis. Two wedge analyses exist. Walls that have a facing slope between 70° and 90° are analysed using the single-wedge method, while walls that have a flatter facing slope are analysed according to the dual-wedge method.

The layout and strength of the reinforcement layers are adjusted until an adequate design has been obtained, using the wedge analyses to check the stability. Both the single and dual wedge methods are used to check the stability of compound failure surfaces. Wedges that extend outside the reinforced soil zone should also be considered. Compound failure surfaces are failure surfaces part inside and part outside the reinforced CRB wall system (Gassner, 2005). The minimum factor of safety for internal stability, as well as pull out resistance, is 1.5.

Pull-out

The tensile capacity of the reinforcement depends both on the strength of the reinforcement and its pull-out resistance. Pull-out resistance should be checked for every layer, for each failure surface under consideration. The anchorage length is taken as the embedded length of the reinforcement behind the failure plane.

The pull-out resistance develops from the friction force generated by the weight of the soil on top of the reinforcement. Any shortfall in pull-out resistance at a particular level is transferred to layers of reinforcement lower down the wall. The combined pull out resistance of the reinforcement at all levels is then compared to the load required to prevent failure which in turn determines the factor of safety against pull-out. If the factor of safety is too low, the anchorage length is increased.

When assessing pull-out resistance, the type of reinforcement should be chosen before-hand as different reinforcements have different friction and interlocking properties. These properties influence the pull-out resistance in different types of soils. After the pull-out resistance has been calculated, the strength of the geosynthetic can be determined at each level of the CRB wall.

Tensile overstress

Geosynthetics have a plastic flow stress of approximately 30% to 50% of their ultimate strength. The plastic flow stress of the geosynthetic depends on the polymer and manufacturing process used. When the geosynthetic reinforcement reaches its plastic flow stress level, the materials will strain over time without adding extra load to it. Furthermore, geosynthetics are often damaged during installation, or are damaged by the surrounding environment due to chemicals, biological ecosystems, temperatures, etc.

The ultimate strength of a geosynthetic is factored down using reduction factors. Gassner (2005) explains that the reduction factors include creep, construction damage, chemical attack, etc. These reduction factors depend on the type of backfill material used, the likely long-term environmental exposure of the geosynthetic and the type of polymer used to manufacture the geosynthetic. Typically the manufacturer of the geosynthetic will provide the designer with the reduction factors required to determine the permissible tensile load to be used in design of the geosynthetic. These factors are combined according to the following formula:

$$F_{\text{design}} = \frac{F_{\text{plastic flow}}}{(\text{factor 1} \times \text{factor 2} \times \text{factor 3} \times \ldots\ldots \times \text{factor n})}$$

If the stress in the geosynthetic is well below the plastic flow stress level, the rate of creep will reduce according to a log-scale. Creep continues for quite some time after installation. The percentage of on-going creep depends on the type of geosynthetic and is a function of the long-term stress in the geosynthetic. To control the amount of lateral deflection that may be expected at the face of the wall, the following rule of thumb may be adopted: For reinforced CRB walls steeper than 70°, the creep limit based on permanent loads should be 0.5%. The creep limit based on permanent loads for reinforced CRB walls with a slope less than 70° should be 1.0%.

Internal sliding

Bathurst and Simac (1994) highlight internal sliding as a mode of failure which requires special consideration in design and analysis of reinforced soil CRB walls. According to Bathurst and Simac (1994), the unit-to-unit interface shear capacity is crucial to prevent internal sliding mechanisms mobilized by the facing.

***STEP 3**: Determine the type of facing*

The local stability of the reinforced CRB wall structure focuses on the facing as well as the facing-reinforcement connections.

Connection failure

The type of facing is restricted by the facing connection needed. Bathurst and Simac (1994) further explain that the horizontal geosynthetic layers should be placed between the facing units to form a frictional connection. The pull-out resistance of the reinforcement from between the facing units can govern the reinforcement spacing and type of reinforcement used.

The force applied to the facing connection is generally less than the maximum load in the reinforcement layer which occurs at a point some distance behind the face. According to the CMA design guide for reinforced CRB walls (Gassner, 2005), when designing a 70° or steeper reinforced CRB walls with continuous layers of reinforcement which are all connected to the facing, the following guide line can be followed: The connection to the facing in the bottom third of the height of the wall should be able to handle 100% of the tensile load in the reinforcement. The face connection in the top two-thirds of the wall needs to handle 50% of the tensile load.

If strip reinforcement is used instead of continuous sheets of reinforcement, the connection of the reinforcement to the facing should be capable of carrying 100% of the force in the reinforcement reducing to 50% over the upper half of the wall.

The minimum factor of safety for the facing connection for steep walls should be taken as 1.5. The pull-out force is governed by the type of reinforcement and facing. If laboratory tests to determine the pull-out force are unavailable, the factor of safety should be increased to account for the uncertainty. For walls at flatter inclinations and not subjected to pore water pressures through flooding or seepage, the factor of safety is generally taken as 1.3.

Shear failure and bulging

Bathurst and Simac (1994) explain that the unit-to-unit interface shear capacity is crucial to prevent to prevent localized bulging. This is accounted for in the design through the block friction and nib shear strength when calculating the factor of safety against block-on-block sliding.

Toppling

Alston and Bathurst (1996) include that the maximum unreinforced height at the top of the structure is obtained through a similar stability analysis and factors of safety as for gravity CRB walls.

***STEP 4**: Check global stability*

Global/Overall stability involves failure mechanisms passing through or beyond the reinforced soil zone. Alston and Bathurst (1996) suggest that global stability of the structure should be satisfied as for all retaining wall systems. The minimum base width and embedment depth to maintain overall stability of the slope or structure being retained is calculated during the analysis when the stability of the wall against sliding and overturning is checked. In addition, the overall stability of the slope should be checked using a conventional slope stability analysis as shown in Figure 18. The conventional slope stability methods have been modified to include the stabilisation contribution from geosynthetic reinforcement. Figure 18 has been adapted from the figure in the CMA design manual for reinforced soil walls due to the errors highlighted in Section 3.5.5.

3.5.4. Design Example

The reinforced soil CRB wall design example is attached at the end of this report in Appendix D.

3.5.5. Comments on the CMA Design Manual for Reinforced Soil CRB Walls

The CMA design manual for reinforced CRB walls uses simplified methods based on approximate earth pressure theories. These simplified methods are acceptable for basic overall stability checks and preliminary checks on the force in the reinforcement, but not for calculating the adequacy of the length of the reinforcement where changing the angle of the failure plane changes both the earth pressure on the block and the embedded length of the reinforcement.

Furthermore, the formula given on Sketch 2 to calculate the applied bearing pressure on the foundation soils is incorrect. If the eccentricity of the load is measured from the front edge of the block of reinforced soil as shown in Sketch 2, the effective width of the foundation is 2e' and not just e', therefore the bearing pressure equation should be as follows:

$$P_s = \frac{W}{2e'}$$

This bearing pressure calculation is based on the vertical load only. This is acceptable as most bearing capacity calculations give the vertical load capacity of the base taking the inclination of the load into account in the calculation. Unfortunately the CMA manual makes no mention of the need to consider the inclination of the applied load in the calculation of bearing capacity.

Similarly, the formula given on Sketch 4 to calculate the tension in the reinforcement based on the depth to the top and bottom layer is incorrect. Both h_1 and h_2 inside the brackets should be squared, therefore the equation should be as follows:

$$F_i = k_a \times \gamma \times \left(\frac{{h_2}^2 - {h_1}^2}{2}\right)$$

Moreover, the calculations in the preliminary load assessment of stresses in the reinforcement and pull-out resistance is done on a layer-by-layer basis which is based on using a simplified Rankine earth pressure distribution. This method breaks down if the angle of the failure plane is varied as required by the manual and where the failure plane passes outside the reinforced soil block. In addition, the calculation of earth pressure takes no account of the inclination of the backfill and wall friction against the back of the blocks. It would be far preferable to use a Coulomb-type analysis in which all the forces acting on the potential failure wedge are determined and the overall stability of the wedge is considered rather than looking at each layer of reinforcement individually. Where the upper layers of reinforcement do not extend beyond the failure wedge, the adequacy of the reinforcement over the upper portion of the wall should be checked by performing similar analyses at intermediate depths.

Chapter 4

Previous Studies

4.1. Overview

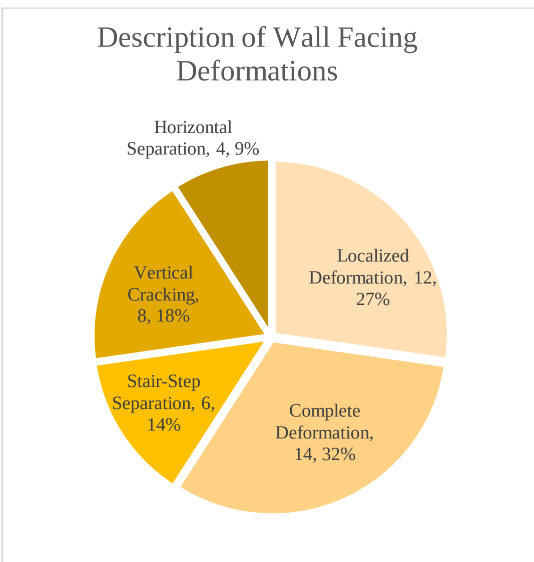
The Geosynthetic Institute (GSI) is an international consortium of organisations interested in, and involved with, geosynthetics. The organisations which form part of the GSI include federal and state governmental agencies, facility owners, resin producers, manufacturers, installers, design consultants, test laboratories and QC and QA organisations (Koerner & Koerner, 2016).

The GSI has compiled a database of failed geosynthetic reinforced, mechanically stabilized earth (MSE) walls. Most of the failures recorded in this database occurred in North America. The GSI's investigations began in the 1980's. Over time, the database has grown and contain 171 failures by 2013 (Koerner & Koerner, 2013).

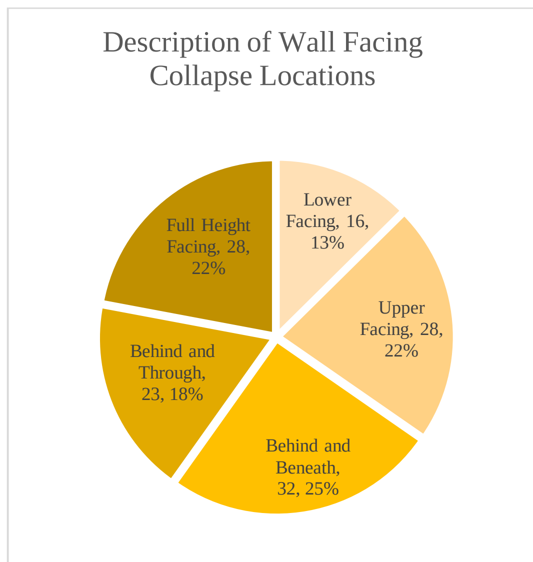
Many case studies have been conducted to investigate the reasons for the failure of geosynthetic reinforced segmental retaining walls. Some of these refer back to the GSI database for verification of their findings. Two of these case studies are of particular interest. The first paper, "*Lessons learned from a failure of a geosynthetic reinforced segmental retaining wall* " by Yoo and Jung, is a comprehensive investigation into the possible causes of the collapse of a 7.4m high segmental retaining wall in Korea (Yoo & Jung, 2006). The failure occurred after a severe rainstorm, immediately after the completion of construction. Two design approaches were used to analyse the failure namely the NCMA and the FHWA. The second paper, "Case *history of a geosynthetic reinforced segmental retaining wall failure"* by Hossain, Omchenko and Mahmood, is an extensive forensic geotechnical investigation into the collapse of a 4.6m high geosynthetic reinforced segmental retaining wall in Rockville. The failure also occurred after the completion of construction. A stability analysis was carried out using the PLAXIS computer software (Hossain, et al., 2009).

4.2. Noteworthy Findings

In 2001, Koerner and Soong (2001) published a report on the failure of 26 MSE walls (Koerner & Soong, 2001). The failures were classified into two groups, excessive deformation and collapse. Twenty of the 26 failures had silt or clay backfill. Most of these walls did not have a continuous quality assessment (CQA) inspection. A further GRI report, Report #38, was published in 2009 (Koerner & Koerner, 2009). This report studied 82 cases of failed reinforced CRB walls, 23 of which failed due to excessive deformation and 59 of which collapsed; only 3 cases resulted in both excessive deformation and collapse. Twenty-seven of the 82 were published cases, 13 came from GSI files, 36 came from colleagues' files and the remaining 6 were obtained from other sources. In January 2013, Koerner and Koerner published a report, "*Geotextiles and Geomembranes: A data base, statistics and recommendations regarding 171 failed geosynthetic reinforced mechanically stabilized earth (MSE) walls"* (Koerner & Koerner, 2013), on 171 total case studies. Forty-four of the 171 cases resulted in excessive deformation and the remaining 127 walls collapsed. These two failure classifications are broken down further in Graph 1 and Graph 2.



Graph 1: Description of wall facing deformations (Koerner & Koerner, 2013)



Graph 2: Description of wall facing collapse locations (Koerner & Koerner, 2013)

The GSI's main statistical findings of the 171 failed MSE walls as discussed by Koerner and Koerner (2013) were as follows:

- 96% were privately owned walls;
- 78% were in North America;
- 71% were masonry block faced (SRWs);
- 65% were 4 to 12 m high;
- 91% were geogrid reinforced (the others were geotextiles);
- 86% failed less than four years after construction;
- 61% used silt and clay backfill soils;
- 72% had poor to moderate compaction;
- 98% of the failures were caused by improper design or construction;
- No failures occurred due to improper manufacturing of the geosynthetic; and
- 60% were caused by internal or external water (the remaining 40% were caused by soil related issues).

4.3. Reasons for Failure as Reported in the Literature

Koerner's statistics show that the primary causes of failures are poor design and construction. The five major design and construction-related issues identified in a webinar presented by Koerner in 2013 are given below.

4.3.1. Reasons for the Failures of the 171 MSE Walls

4.3.1.1. The use of fine grained soil in the reinforced soil zone

Koerner found that 61% of the 171 MSE wall failures had a silt and/or clay soil backfill type. According to Koerner (2013), it could be the in-situ soil which was reused due to the easy availability and negligible cost of the soil. Soft, silty soil was also used in the case study by Hossain et al. (2009).

4.3.1.2. The poor placement and compaction of backfill coupled with lack of inspection

Koerner saw that 72% of the 171 failed MSE walls had backfills which were moderately or poorly compacted. Hossain et al. (2009) found voids under the geogrid reinforcement of the retaining wall, which indicated that the backfill material was inadequately compacted.

4.3.1.3. Placing of drainage in the reinforced soil zone

This includes routing the surface water internally, within and through the reinforced soil zone (Koerner & Koerner, 2013). Forty-eight percent of the internal wall failures occurred as a result of a faulty internal plumbing system (water-bearing services) which was installed in the reinforced soil zone.

4.3.1.4. Poor control of ground water and surface water

Poor control of surface water was found to be one of the main reasons for the failure of CRB walls, accounting for 53% of the external failures contained in the GSI database.

4.3.1.5. Improperly assessed and/or misunderstood design details

Koerner et al. (2009) believe that improper recognition of design details reduces the factors of safety values. These design details include steep slopes at the toe of the wall, wide spacing of reinforcement layers, strength reduction of geosynthetic reinforcement due to holes from the penetration of light poles or posts, poor foundation conditions leading to global/overall instability and a lack of conservative external water level estimates and seismic events.

4.3.2. Reasons for the Failure of CRB Walls as Found by Others

Yoo and Jung (2006) found that an inappropriate design contributed to the failure of a wall they investigated. The stability of the system was compromised by the failure of the designer to consider additional surcharge load due to a 5m high broke-back slope portion in the retained soil zone. In addition, the internal stability of the wall was compromised by an incorrect and unrealistic assessment of the internal soil friction angle of the retained soil zone. This is a common occurrence as in many cases no laboratory tests are done to obtain accurate shear strength parameters of the soil.

Irrespective of the design, Yoo and Jung (2006) established that most available design analysis software cannot fully account for the complex geometries engineers might face; therefore, engineering judgment is often necessary.

Hossain et al. (2009) found that, amongst others, the cause of failure of the CRB wall studied was improper geogrid installation. Other common causes of failure as mentioned by Hossain et al. (2009) include a small offset distance of the blocks which affects the overall inclination of the wall facing; increased height of the CRB wall beyond its design height; insufficient reinforcement length and a sudden draw down of water which affects the overall stability of the wall. Furthermore, a deep seated slip failure occurred as the foundation was weak and the backfill was of a low quality.

Yoo and Jung (2006) concluded that site inspections were not correctly implemented and were not executed on a regular basis.

4.4. Recommendations Contained in the Literature

4.4.1. Recommendations Based on Statistical Findings by Koerner

Pequenino et al. (2015) further analysed Koerner's findings and isolated two fundamental causes for the poor performance of the MSE walls, namely:

- The nature in which the MSE walls are planned, designed and constructed; as well as
- Surface and subsoil drainage issues.

Through the aforementioned identifications, Pequenino et al. (2015) discussed the reasons for the occurrence of the failures and made suggestions to prevent these design and construction-related issues in the future.

Pequenino et al. (2015) found that the manner in which MSE walls are planned, designed and constructed is a collaborative process between the designer and supplier. Unfortunately, a lack of communication exists between the two parties which leads to design assumptions made by the supplier in the internal stability analysis, not being fully grasped by the designer when analysing the external stability of the retaining wall system. Therefore, Pequenino et al. (2015) states that flaws arise in the design and problems arise in the allocation of responsibility during the design and construction phase of the project.

Hence, Pequenino et al. (2015) highlights that the designer should have a thorough understanding of the assumptions made by the supplier and perform regular routine verifications on internal and external stability. Furthermore, regular routine site inspections should be performed to ensure the construction procedures conform to the design specifications.

Pequenino et al. (2015) also concluded that a geotechnical site investigation programme is crucial to ensure an adequate drainage infrastructure. Variations encountered during construction to the flow of groundwater assumed in the design should be communicated to the designer. In addition, surface water should be controlled during construction to ensure that the fill material is not compromised.

4.4.2. Additional Recommendations Contained in the Literature

Koerner and Koerner (2009) found that improper compaction is a common construction issue, but admit that it is often not easy to obtain the desired degree of compaction. The GRI #38 Report (2009) includes Figure 19, which was compiled by Turnbull in 1950, and illustrates water content-vs.-density relationship for fine grained soils under six different conditions.

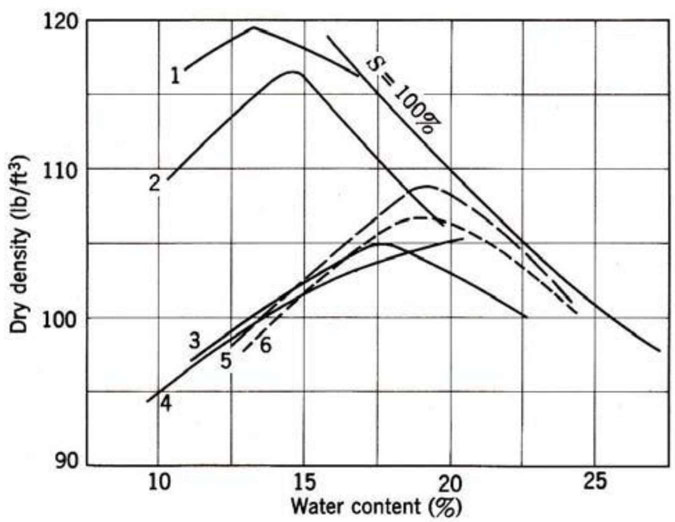


Figure 19: GSI Report #38 - Comparison of field and laboratory compaction compiled by Turnbull in 1950 (Koerner & Koerner, 2009)

They point out that the required density can only be achieved at a narrow range of water contents. Moreover, each fine-grained soil is different and the unique curve of the site-specific soil needs to be determined. Therefore, the report further presents Table 3, which indicates the level of inspection as a percentage of construction time, to assure that the desired degree of compaction is achieved.

Table 3: Suggested levels of construction quality assurance (CQA), or inspection, as a percentage of construction time (Koerner & Koerner, 2009)

SIGNIFICANCE	DURATION	
	TEMPORARY	PERMANENT
NONCRITICAL	33%	67%
CRITICAL	67%	100%

An additional finding was that reinforcement should be properly tensioned to prevent drops or bends and associated slack from occurring. The reinforcement will not achieve its purpose of causing the soil mass to act as a gravity structure if it is not installed properly before placement of the backfill.

The GRI Report #38 recommends that all piping should be routed away from the reinforced soil zone. This is accomplished by raising the height of the facing and sloping the soil above the wall away from the face thereby routing water away from the facing to catch basins located behind the end of the reinforcing. From here, the surface drainage can easily be connected to the back drain without interrupting the reinforcement. Furthermore, if leakage occurs, the water can be intercepted by in the back drain and removed from the system through the base drain (Koerner & Koerner, 2011).

Surface water should be controlled to prevent sequential failure from occurring as shown in Figure 8. According to Koerner (2013), the surface of the fill behind the reinforced CRB wall system should be sealed. If the area behind the wall is paved, the cracks must be filled immediately with joint sealant or asphalt. If the area behind the wall is vegetated, a geomembrane water barrier can be included into the retaining wall system.

If the wall is located next to a river or stream, the design must estimate a maximum service life of 75 to 100 years for the structure. The design should be extremely conservative and no-fines gravel such as GW or GP classified backfill should be used up to the estimated maximum water level.

Yoo and Jung (2006) suggested that pore water pressure in the retained and reinforced soil zones is often underestimated in slope stability analyses, and should be considered with a higher degree of realism. They further state that it would be beneficial for all final designs of a non-routine wall to be checked by a qualified geotechnical engineer with an appropriate background in the field. Such engineers should be able to bring basic geotechnical engineering principles into their CRB wall designs, as neglect of these basic principles can cause catastrophic failures.

Chapter 5

Research Methodology

5.1. Overall Approach

As stated in the introduction, this research analyses South African case histories of failed gravity and reinforced soil CRB walls. In broad outline, the methodology followed was the collection of data, establishment of an outcome for each case study, classification of the CRB walls and describing the failures to be able to identify the mode and prime cause of failure. Each step in this process is described below.

5.2. Data Collection

In this report, data refers to the information extracted from the case history files obtained from the various sources. Of the total 28 case histories obtained, 18 of them were retrieved from the ECSA database. The remaining 10 were obtained from a private engineering firm *Jones & Wagener (Pty) Ltd.* Of the 18 case histories obtained from ECSA, only ten could be used. Similarly, only 8 of the ten case histories from *Jones & Wagener (Pty) Ltd.* could be used. The remaining 10 case histories did not contain sufficient information for the purposes of this study.

The case histories obtained from ECSA are records of complaints of improper conduct against professionals registered by ECSA arising from the failures of CRB walls. Only certain extracts were needed from the case history files namely the complainant's affidavit, the expert or assignee report, and any calculations, photos and drawings available for each case. The case histories obtained from the private engineering firm are from job files assembled when this firm is asked by other professional engineers, contractors or clients to advise on repair works for CRB wall failures and/or provide reasons for the failures of these walls. When studying the case histories obtained from the private engineering firm, the researcher was typically interested in any drawings, design calculations, photos and expert reports on the failures.

A condition imposed by ECSA was that no details of any case history may be revealed. This includes the names of the designer, contractor, block supplier and client and the exact location of the wall. The researcher has therefore kept each case history completely anonymous.

To conclude, the reasons for the failures as well as problems encountered in the design as found by the author, in agreement with the complainant's affidavit, the expert or assignee report, calculations made and any drawings available for each case, are studied and a comprehensive discussion is presented.

5.3. Case Study Outcomes

Each case study is investigated and an outcome for the case study is determined. Specific attention is given to the wall properties and design parameters, the purpose of the wall, a description of the failure, details of the problem and design issues encountered by experienced professionals in assignee and expert reports. The design assumptions made in each case study are studied and any deviation from the design in the as-built wall is assessed as a potential reason for the failure of the wall. A summary of the outcomes of each case study is attached in Appendix B at the end of this report.

5.4. Classifications of CRB Walls

5.4.1. Type of Wall

The failed CRB walls are classified either as gravity walls or reinforced soil walls. The gravity walls may include cement stabilization of the backfill in the form of soilcrete for additional stability.

A study is conducted on the environments in which the retaining walls are located, as the author suggests that it might have had a severe impact on the stability of the structural systems.

In each of these categories, the wall is further classified as described below.

5.4.1.1. Maximum height of the wall

This maximum height is the height for which the wall was designed initially. If this design height was exceeded in the construction and the as-built wall is higher than designed, it is seen as a construction fault as well as a potential reason for the failure of the wall and is described as such. If the design height of the wall is unavailable, the design height can be assessed from photographs of the wall based on a typical block height of 250mm.The maximum height of the wall is used to categorise the wall height into a given category, e.g. a wall that varies in height from 4m to 7,5m would be placed in the 6m to 8m height category.

5.4.1.2. Service life time of the wall

The service life of the wall is the number of years or months it took for the failure to occur after the wall was completed. Where a completion certificate was provided, the date of signing was taken as the completion date. If the wall failed during construction, it is placed in the <1 year category.

5.4.1.3. Wall inclination to the horizontal

The inclination of the wall is the inclination for which the wall was designed. If the wall is described as being steep, it would fall in the 80°- 90° inclination category. If the as-built wall was constructed at a different inclination than what it was designed for, it is seen as a construction fault as well as a potential reason for the failure of the wall and is described as such.

5.4.1.4. Top slope of soil behind wall

The walls have been classified according to the slope of the ground surface behind the wall. If the ground surface behind the wall slopes upwards initially and then becomes horizontal, the retaining condition of the wall is described as a limiting bank height.

5.4.2. Wall Configuration

5.4.2.1. Uniform soil

The wall is described as retaining uniform soil when the backfill behind the CRB wall system has similar properties to the retained material.

In such cases, the position of the critical failure plane is unrestricted and can pass through the backfill and/or the retained soil. Therefore, all possible critical wedges should be considered when assessing the stability of the retaining wall system (Clark, 2005).

5.4.2.2. CRB wall in front of stable rock face

In many instances, CRB walls are constructed in front of a cut face formed in stable (self-supporting) ground at the same angle as the wall facing. Such walls have been classified as CRB walls in front of stable "rock" face.

The main purpose of such walls is to prevent deterioration or erosion of the cut face. In such cases the wall is designed to withstand the destabilizing forces due to the infill material between the cut face and wall rather than to provide support to ground behind the cut face. The destabilizing forces acting on the wall are solely caused by the infill soil. The magnitudes of the forces are dependent on the distance between the cut face and the wall. According to the CMA Code for gravity CRB walls (2005), the best solution would be to stabilize the infill soil, thereby relieving the wall from the destabilizing forces if an effective drainage medium is installed behind the stabilized backfill. The soilcrete should be thick enough to prevent sliding on the backfill/cut interface.

In such cases, the position of the failure surface is confined to the backfill behind the wall. If the backfill is stabilised, the slip plane may form at the backfill-rock face interface.

5.4.2.3. Limiting bank height

The wall is described as having a limiting bank height if the ground surface immediately behind the wall slopes upwards over a short distance and then flattens out. This is a common configuration for CRB walls. The additional height of soil above the top of the wall acts as a surcharge which must be considered in the design.

5.4.2.4. Tiered

A tiered wall is one where the facing is stepped back at regular intervals, i.e. the full height of the wall is not in the same plane, but constructed on benches. Each tier of the wall is surcharged by the tier above unless the step-back is so great that the foundations of the tier above are behind the line of the natural angle of repose, measured from the heel of the lower tier. It is essential that tiered walls be checked for possible global/overall instability.

5.4.3. Type of Reinforcement

If the wall is reinforced soil, the type of reinforcing is categorised as either a geotextile, geogrid or a geocomposite. Geotextile reinforcement is further classified as woven or non-woven.

5.4.4. Type of Retained Soil

The backfill material is classified according to the grading and origin of the material. If the backfill differs from the retained soil, it is classified as imported fill. Where the backfill is similar in nature to the retained soil, it is classified according to its geological origin or formation. For example, many of the walls in Kwa-Zulu Natal are in Berea Red soil and many of the walls in Gauteng are in Residual Granite.

The degree to which this backfill material is compacted is classified to the extent possible as well compacted, moderately compacted or poorly compacted. Often the compaction must be assumed based on descriptions contained in the record of the case history as the degree of compaction is either not stated or the compaction might vary across the site.

5.4.5. Other Details

The CRB wall failures included in this study occurred between 1993 and 2014. The walls have been classified according to the year in which failure occurred.

The failed walls have also been classified according to their ownership (private or public) and according to their location in different provinces of South Africa.

For classification in terms of responsibility for the failure, an attempt has been made to determine who was responsible for the proximate cause of failure. Where there is more than one cause of failure, it is not always clear who bears the responsibility. In such cases, the proximate cause of failure can often be distinguished from the accompanying cause(s) by asking the question whether the failure would still have occurred had the accompanying cause not been present.

5.5. Failure Descriptions

The failures can either be classified as a serviceability problem, where excessive deformation of the wall occurs and the wall remains standing or as full or partial collapse of a section of the wall. These classifications are based on visual descriptions of the failures. Excessive deformations of the walls are taken as deformation beyond the intention of the designer and expectations of the client. Continued deformation can eventually lead to collapse of the structure.

5.5.1. Deformation

5.5.1.1. Localized deformation

Localized deformation of a CRB wall includes local bulging, cracking or toppling of the wall facing units. Localized deformation can visually be recognised when a portion of the wall demonstrates an obvious bowing effect relative to the remainder of the wall (Koerner & Koerner, 2009).

5.5.1.2. Horizontal separation

Horizontal separation is evident when the lower courses of blocks settle away from the upper portion of the wall. Koerner et al. (2009) states that horizontal separation can visually be recognised by gaps between masonry block rows or incremental lifts. It can occur near the middle of a long wall where the blocks above are able to arch over the area where the support is lost due to settlement of the lower blocks (Day, 2015).

5.5.1.3. Vertical cracking

Vertical shear is evidenced by cracking of the blocks, or dislocation of blocks, along a vertical line through the wall.

5.5.1.4. Complete deformation

Koerner (2009) explains that a completely deformed wall leans to such an extent that the original batter of the wall is lost or even reversed.

5.5.1.5. Stair-step separation

Stair-step separation is visible through the separation or gaps between courses of facing block units in the form of a stair. The deformation of the wall facing occurs at the weakest point, which is generally between consecutive block units.

5.5.2. Collapse

5.5.2.1. Full height

Full height collapse is collapse which occurs from the top to the bottom of the wall, i.e. over the entire height of the structure. Full height failure will occur when the reinforcement is not anchored and the facing units do not interlink, causing block-on-block sliding, and the entire wall face collapses. Often the wall collapses completely and the facing units tumble down, but the foundation is still intact and no foundation movement is observed. Full height collapse often occurs during construction.

5.5.2.2. Behind and through

Behind and through collapse occurs by shear movement along a slip surface which passes through the soil and exits above the toe of the wall. The wall often bulges before the slip plane pushes through the face of the wall. The foundation and lower portion of the wall may remain in place.

5.5.2.3. Behind and beneath

Behind and beneath failure occurs by shear movement on a slip surface which passes through the soil and below the foundation of the wall, exiting immediately in front of the wall or a short distance in front of the wall, resulting in collapse or rotation of the full height of the wall. The foundation moves together with the base of the wall and the retained soil.

5.5.2.4. <u>Upper facing</u>

This is a situation where the portion of the upper facing of the wall fails. This failure often occurs when water ponds behind the upper portion of the wall.

5.5.2.5. <u>Lower facing</u>

This is where a portion of the lower facing of the wall fails.

5.6. Basic Failure Mechanism Classifications

The basic failure mechanisms consist of 4 groups as seen in Figure 20. Multiple mechanisms contributed to the failure of most of the walls. The author attempted to categorize the failures based on instability and water issues. It is important to emphasize that water pressure is a fundamental driving mechanism which directly leads to wall failures (Koerner & Koerner, 2009).

The basic failure mechanisms are listed and briefly discussed below.

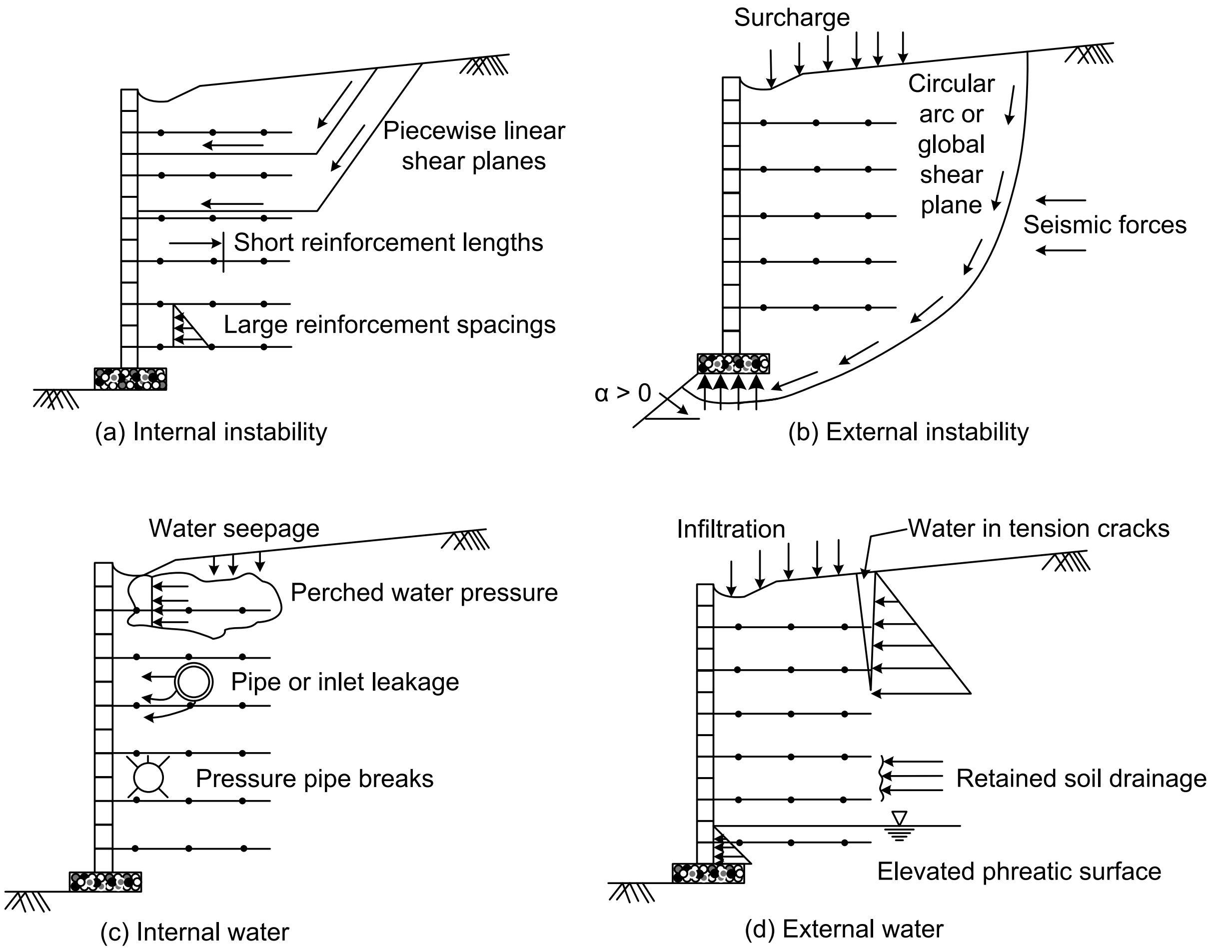


Figure 20:Basic failure mechanisms (Koerner & Koerner, 2013)

5.6.1. Internal Instability Failures

Internal instability is manifest by a failure of the wall due to an inability of the wall and any reinforcement provided to resist the earth pressure exerted by the retained soil.

Factors that contribute to internal instability include:

- Low soil friction;
- Low interface friction;
- Poor quality of backfill material;
- Inadequate compaction;
- Omission of cement stabilization;
- Omission of mechanical stabilization in the form of soil reinforcement;
- Wide reinforcement spacing;
- Short reinforcement length;
- Incorrect reinforcement orientation;
- Failure of the facing units;
- Foundation failure (bearing, sliding or differential settlement); and
- Overturning.

5.6.2. External Instability Failures

External instability occurs when the ground on which the wall is built fails as a mass, taking the wall with it. This type of failure is common on sloping sites or with multi-tiered walls.

Factors that contribute to external instability include

- Sloping ground above and/or below the wall;
- High surcharge behind the wall;
- Low strength of in situ soils behind and below the wall;
- Inadequate benching into existing material; and
- Seismic activity.

External instability failures can also be caused by tampering with the original retaining wall by, for example, increasing the wall height above the design height, filling behind the wall or excavating in front of the wall.

5.6.3. Internal Water Failures

Internal water failure mechanisms occur as a result of erosion or saturation of the backfill due to leakage from water-bearing services behind the wall, pipe bursts, ponding of surface water behind the wall or malfunction of the drainage system.

5.6.4. External Water Failures

External water failure mechanisms arise from seepage from the retained soil behind the wall and infiltration from the retained soil above the wall, water pressure in tension cracks, the development of a perched water table or a rise in the level of the permanent water table.

Chapter 6

Case Studies

6.1. Overview

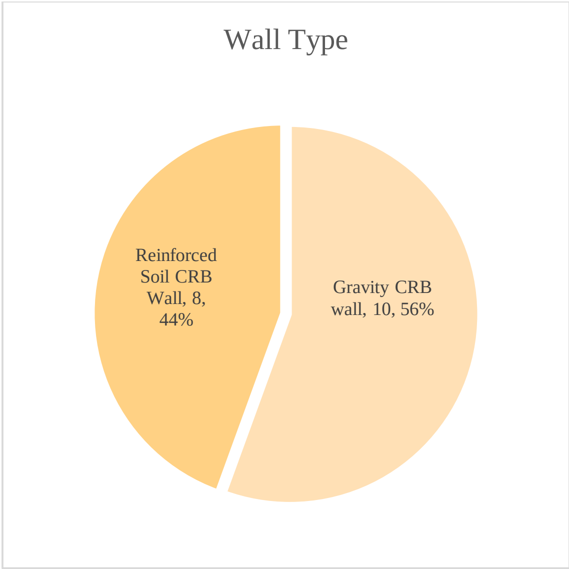
A total of 28 case histories were studied, but only 18 contained sufficient information for the purpose of this research. The failure of the eighteen gravity and reinforced CRB walls is presented in this chapter. An outcome for each case study as well as a summary of the outcomes is attached in Appendix B at the end of this report. In line with the confidentiality agreement with ECSA, specific details of the case histories have been omitted, including the exact location of the site and names of the designer, contractor, block supplier and client.

The following information regarding the wall properties is based on the wall that was designed and not the as-built wall. Any deviation from the design is seen as a potential reason for the failure of the wall and is described as such in the chapters to follow.

6.2. Classification of CRB Walls

6.2.1. Type of wall

Ten of the failed walls (56%) were gravity CRB walls and the remaining eight (44%) were reinforced soil CRB walls (Graph 3). There are sufficient numbers of these two different types of CRB walls to enable observations to be made regarding the failures of both types of CRB retaining wall systems in South Africa.



Graph 3: Types of walls presented in this report

6.2.1.1. Maximum wall height

The CRB walls range from 1.8m to 15m high. All of the walls fully or partially deformed or collapsed at their maximum heights. Most of the walls failed over a significant length of the wall. According to the following data, more than three quarters of the failed walls were between 2m and 8m high.

- 1(6%) wall was less than 2m high;
- 8 (44%) walls fell in the 2-5m height category;
- 3 (17%) walls fell in the 5-6m height category;
- 3 (17%) walls fell in the 6-8m height category;
- 2 (11%) walls fell in the 8-12m height category; and
- 1 (6%) wall was higher than 12m.

6.2.1.2. Service life

The following data indicates that most of the failures occurred soon after construction.

- 3 (17%) walls failed during construction;
- 6 (33%) walls failed in less than a year after completion of construction;
- 1 (6%) wall failed in 1 to less than 2 years after completion of construction;
- 3 (17%) walls failed in 2 to less than 4 years after completion of construction;
- 3 (17%) walls failed in 4 to less than 8 years after completion of construction;
- 1 (6%) wall failed more than 8 years after completion of construction; and
- The service life of one wall was unknown.

6.2.1.3. Wall inclination

As derived from the data, more than half of the failed walls were inclined steeper than 70^0 to the horizontal.

- 1 (6%) of the failed walls was inclined less than 60^0 to the horizontal;
- 6 (33%) of the failures were inclined at 60^0 to less than 70^0 to the horizontal;
- 5 (28%) of the failures were inclined at 70^0 to less than 80^0 to the horizontal;
- 5 (28%) of the failures were inclined at 80^0 to less than 90^0 to the horizontal; and
- 1 (6%) of the failed walls was inclined more than 90^0 to the horizontal.

6.2.1.4. Top slope

The failed walls typically had a top slope of less than 4^0 to the horizontal.

- 16 (89%) walls had top slopes less than 4^0 to the horizontal;
- 0 (0%) walls had top slopes between 4^0 and 10^0 to the horizontal;
- 0 (0%) walls had a top slope between 10^0 and 20^0 to the horizontal;
- 2 (11%) walls had a top slope between 20^0 and 30^0 to the horizontal;
- 0 (0%) walls had a top slope more than 30^0 to the horizontal.

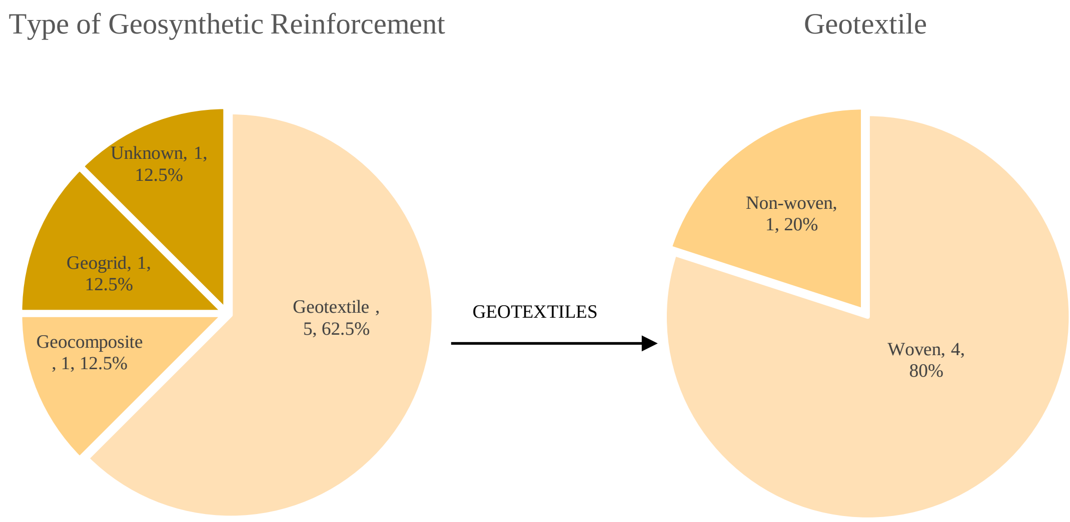
6.2.2. Wall Configuration

The retaining conditions of the failed walls were as follows:

- 10 (56%) of the walls exhibited a uniform soil retaining condition;
- 1 (6%) of the walls was constructed in front of a stable rock face;
- 2 (11%) of the walls exhibited a limiting bank height; and
- The remaining 5 (28%) walls were tiered.

6.2.3. Type of Reinforcement

Five walls incorporated geotextile reinforcement (62.5%), while one wall (12.5%) incorporated geocomposite reinforcing and one wall (12.5%) was affiliated with geogrids. The reinforcement for the remaining wall was unknown, but the failure did not occur due to inadequate reinforcement and, therefore, the type of reinforcement is irrelevant. 80% of the geotextiles were woven and the remaining 20% were non-woven.



Graph 4: Types of geotextile geosynthetic reinforcement in 18 CRB walls in South Africa

6.2.4. Type of Retained Soil

6.2.4.1. Backfill material

The different backfill materials incorporated in the wall systems are described as the following:

- 8 (44%) walls incorporated backfill material of the Berea Red Formation;
- 6 (33%) walls incorporated backfill material described as Residual Granite; and
- 4 (23%) walls belonged to other formations.

6.2.4.2. Degree of compaction

Compaction data was not available in many of the cases studied. Where information on the degree of compaction was available or could be inferred from the available information, the degree of compaction of the backfill material was classified as follows:

- The backfill material of 15 (83%) walls were poorly compacted;
- The backfill material of 2 (11%) walls were moderately compacted;
- The remaining wall (6%) had backfill material that was compacted adequately.

6.2.5. Other Details

6.2.5.1. Year of occurrence

The earliest failure was reported in 1994. The failures trend to approximately one failure per year, with it spiking to five (5) failures in 2007. All except one of these failures occurred in Kwa-Zulu Natal. This could have been a result of the tidal surcharges and storm events experienced by Kwa-Zulu Natal during March of that year (Govender, 2011).

Graph 5: Year of occurrence regarding the 18 CRB wall failures in this report

6.2.5.2. Wall ownership

Gravity and reinforced CRB walls are extremely popular as a retaining structure in the private sector of South Africa. Most of the failed CRB walls were associated with housing developments and apartments. None of the failed walls was owned by the state. The failed walls formed part of the following developments:

- 1 (5.5%) wall was at a hospital;
- 1 (5.5%) wall was constructed at a recreational park;
- 1 (5.5%) wall was constructed at a commercial shopping centre;
- 12 (67%) walls were constructed on a residential property or in a residential development;
- 2 (11%) walls were constructed in a business park; and
- The remaining wall (5.5%) was constructed at a service station.

6.2.5.3. Location by province

The Google Earth image (Figure 21) indicates the locations of the failed CRB walls. From the Google Earth image, it is evident that the failures are relatively spread over South Africa; more focusing on the Eastern side of the country. Most of the failures were located in Kwa-Zulu Natal and in Gauteng.

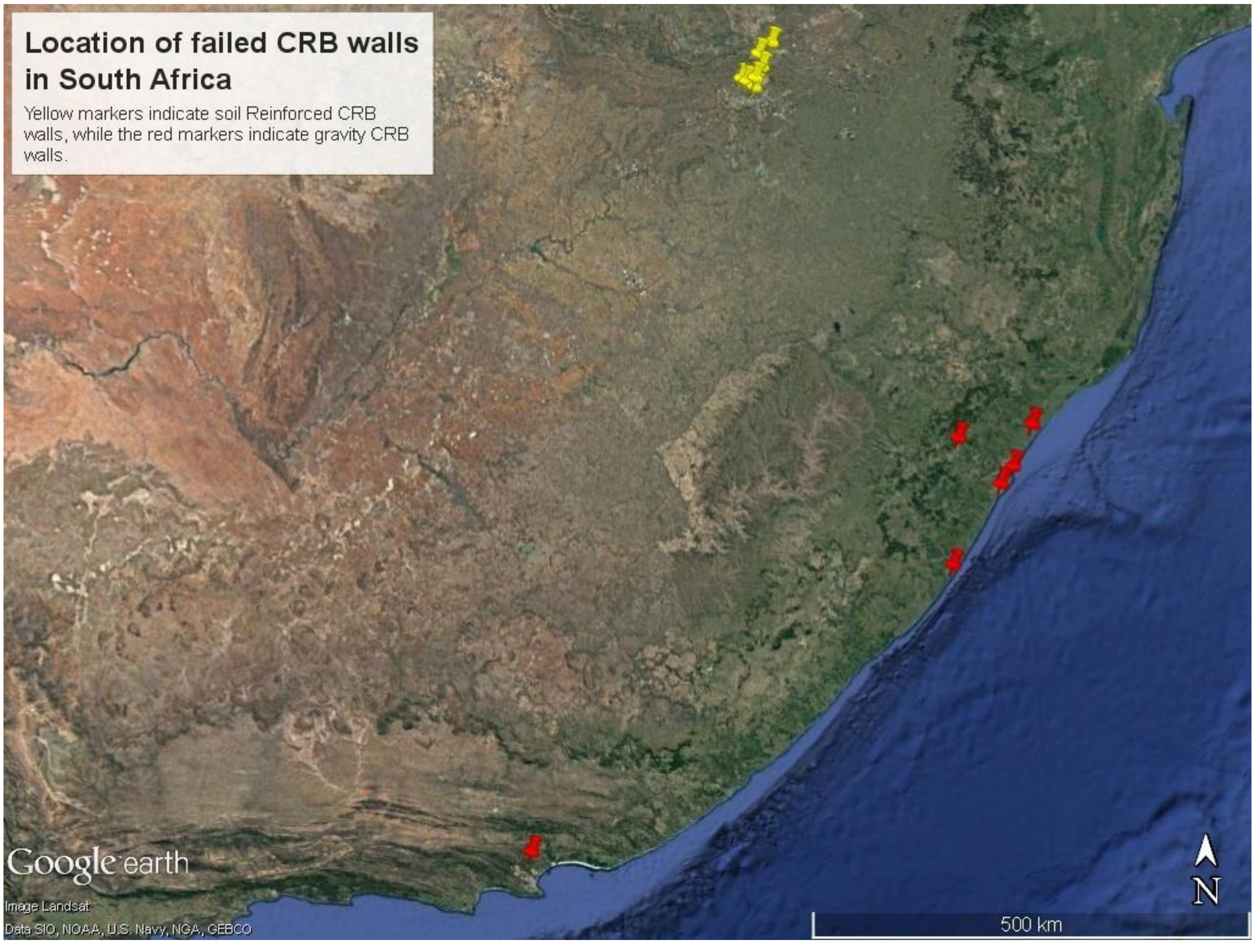


Figure 21: Location of 18 CRB wall failures in South Africa (Google Earth Pro, 2015)

While a large majority of the failed walls were located near the coast, there were other cases reported inland and more to the south of the country as follows:

<u>Kwa-Zulu Natal</u>

In Kwa-Zulu Natal the 9 failures were distributed as follows:

- 4 (44%) walls were located in Ballito;
- 1 (11%) wall was located in Margate;
- 2 (22%) walls were located in Durban;

- 1 (11%) wall was located in Amanzimtoti; and
- 1 (11%) wall was located in Pietermaritzburg.

<u>Gauteng</u>

In Gauteng the 8 failures were distributed as follows:

- 1 (13%) wall was located in Sandton;
- 1 (13%) wall was located in Pretoria;
- 2 (25%) walls were located in central Johannesburg;
- 1 (13%) wall was located in Northcliff;
- 1 (13%) wall was located in Roodepoort;
- 1 (13%) wall was located in Midrand; and
- 1 (13%) wall was located in Centurion.

<u>Eastern Cape</u>

The wall that failed in the Eastern Cape was in Port Elizabeth.

6.2.5.4. Person responsible for failure

In the same way as it is difficult to assess the proximate cause of failure, so too is it difficult to assign responsibility for the failure to a particular party. In many cases, more than one party contributed to the failure. Each categorization was case specific and based on the following principles:

The designer is deemed to have been responsible if the failure occurred as a result of design errors, incorrect specification of backfill material was specified, or omission from the drawings of vital information necessary for construction.

The contractor was deemed responsible for the failure if the failure occurred as a result of poor compaction, the walls were not constructed according to the drawings or the drainage system (or similar vital components of the system) were not built.

If the failures occurred as a result of faulty facing units, the block manufacturer is deemed to be responsible for the failure.

In the author's opinion, the primary responsibility for the wall failures studied should be allocated as follows:

- The design engineer was mainly responsible for 78% of the failures;
- The contractor was mainly responsible for 17% of the failures;
- The block manufacturer was mainly responsible for 6% of the failures.

6.3. Failure Descriptions

As most of the walls deformed prior to collapse, it is evident that failure started before noticeable damage raised concern. The cases of failed CRB walls consist of:

- 6 (33%) walls which deformed excessively; and
- The remaining 12 (67%) walls collapsed.

6.3.1. Excessive Deformation

- 1(17%) wall deformed locally; and
- The remaining 5 (83%) walls deformed completely.

6.3.2. Collapse

- 4 (33%) walls collapsed over the full height of the wall;

- 3 (25%) walls collapsed behind and through the wall;
- 3 (25%) walls collapsed behind and below the wall; and
- An upper portion of the remaining 2 (17%) walls collapsed.

6.4. Basic Failure Mechanisms

The basic failure mechanisms which resulted in the failures, to the best estimate of the author in this regard were as follows:

- 15 (83%) walls failed due to water and instability issues;
- 2 (11%) walls failed solely due to instability issues; and
- 1 (6%) wall failed solely due to water issues.

The 15 walls which failed due to water and instability issues are further categorized as follows:

- 2 (13%) walls failed solely due to external issues;
- 1 (7%) wall failed solely due to internal issues; and
- 12 (80%) walls failed due to a combination of external and internal, water and instability issues.

Further examination of the case studies can assist in determining the reasons for the failures so that solutions can be recommended to prevent the reoccurrence of these failures. Table 4 to Table 7 are summaries of the information discussed in Chapter 6. These tables assist in identifying the trends in the failures.

Table 4: Wall classification and wall configuration of 18 case studies of failed CRB walls in South Africa

Case study	Wall Classification					Wall Configuration
	Wall Type	**Max Height (m)**	**Service Life (years)**	**Inclination (degrees)**	**Top Slope (degrees)**	
CS1	Gravity	5.5	3	69	0	Tiered
CS2	Gravity	3.6	3	70	26	Limiting bank
CS3	Gravity	8	6	65	0	Uniform soil
CS4	Gravity	10	<1	60	0	Tiered
CS5	Gravity	3, 4, 5	DC*	65	0	Tiered
CS6	Gravity	3.7	<1	90	0	Uniform soil
CS7	Gravity	4.5	7	56	0	Uniform soil
CS8	Gravity	7	<1	65	0	Uniform soil
CS9	Gravity	3.4	<1	70	0	Uniform soil
CS10	Gravity	4	4	62	3	Stable rock
CS11	Reinf.	1.8	DC*	85	0	Uniform soil
CS12	Reinf.	3.3	<1	76	0	Tiered
CS13	Reinf.	5.5	<1	80	0	Uniform soil
CS14	Reinf.	5.8	1	75	0	Uniform soil
CS15	Reinf.	15	Unknown	85	0	Tiered
CS16	Reinf.	7.7	>10	80-90	0	Uniform soil
CS17	Reinf.	4.5	3	87	0	Uniform soil
CS18	Reinf.	9.6	DC*	75	27	Limiting bank

* During Construction

Æ Eastern Cape

Table 5: Type of soil and reinforcement used in 18 case studies of failed CRB walls in South Africa

Case study	Soil		Reinforcement
	Backfill	Compaction	
CS1	Berea Red	Poor	-
CS2	Berea Red	Poor	-
CS3	Berea Red	Poor	-
CS4	Berea Red	Poor-Mod	-
CS5	Berea Red	Poor	-
CS6	Berea Red	Poor	-
CS7	Berea Red	Poor	-
CS8	Berea Red	Poor	-
CS9	Residual Granite	Poor	-
CS10	Stable rock	Good	-
CS11	Residual Granite	Poor-Mod	Geocomposite
CS12	Timeball Hill	Poor	Woven geotex.
CS13	Residual Granite	Moderate	Non-woven geotex.
CS14	Residual Granite	Poor	Geogrid
CS15	Residual Granite	Moderate	Unknown
CS16	Mixture	Poor	Woven geotex.
CS17	Unknown	Poor	Woven geotex.
CS18	Residual Granite	Poor	Woven geotex.

Table 6: Relevant details pertaining to 18 case studies of failed CRB walls in South Africa

	Other			
Case study	**Year of failure**	**Ownership**	**Location**	**Responsibility**
CS1	2007	Residential	KZN	Contractor
CS2	2007	Residential	KZN	Contractor
CS3	2012	Residential	KZN	Designer
CS4	2005	Residential	KZN	Designer
CS5	2004	Residential	KZN	Designer
CS6	2011	Residential	KZN	Designer
CS7	2007	Residential	KZN	Designer
CS8	2007	Residential	KZN	Designer
CS9	2001	Service Station	Gauteng	Designer
CS10	2008	Residential	Gauteng	Contractor & Manu.
CS11	2011	Residential	Gauteng	Designer
CS12	2003	Residential	Gauteng	Designer
CS13	1997	Residential	Gauteng	Contractor
CS14	2007	Business park	Gauteng	Designer
CS15	1994	Recreational park	Gauteng	Designer
CS16	2003	Hospital	Gauteng	Designer
CS17	2000	Business park	EC[Æ]	Designer
CS18	2014	Shopping centre	Gauteng	Designer

* During Construction

[Æ] Eastern Cape

Table 7: Failure descriptions and the basic failure mechanisms of 18 case studies of failed CRB walls in South Africa

Case study	Failure Description		Basic Failure Mechanism			
	Collapse	**Deformation**	**External water (EW)**	**Internal water (IW)**	**External Instability (EI)**	**Internal Instability (II)**
CS1	Behind & through	-	EW		EI	II
CS2	-	Localized	EW			II
CS3	Upper section	-	EW	IW		
CS4	Behind & through	-	EW	IW	EI	II
CS5	Full height	-	EW		EI	
CS6	Behind & beneath	-	EW		EI	II
CS7	Behind & beneath	-	EW		EI	II
CS8	-	Complete			EI	II
CS9	Full height	-		IW	EI	
CS10	Full height	-	EW		EI	
CS11	Behind & through	-	EW			II
CS12	-	Complete		IW	EI	II
CS13	Full height	-			EI	II
CS14	-	Complete	EW	IW	EI	II
CS15	Upper section	-	EW			
CS16	-	Complete		IW		II
CS17	-	Complete	EW	IW	EI	II
CS18	Behind & beneath	-	EW	IW	EI	II

Chapter 7

Failure Trends

7.1. Overview

Gravity CRB walls in this study were typically located on steeply sloping sites. The often poorly compacted backfill material formed part of the Berea Red formation. Failures usually occurred as a result of water ingress from external and internal sources, forming a slip plane behind and through/beneath the walls. Initially the walls bulged, which can be seen as a partial state of failure, before the final collapse occurred. Moreover, instability issues resulted in the failure of gravity CRB walls where excessive surcharge loads threatened the stability of the walls. These excessive surcharges typically arose from alterations to design parameters, such as an increase in the height of the wall above the height for which it was designed either during the construction or some time after completion of the wall. Furthermore, many of the designs should have considered stabilization of the backfill material as instability was a common failure mechanism.

Internal instability problems were common among reinforced CRB walls due to inadequate reinforcement design and installation. Furthermore, water ingress was common, especially in the tension cracks which developed at the ends of the reinforcement due to settlement and lateral wall movements. This partial state of failure occurred as a result of the settlement of the underlying foundation soil, omission of a concrete foundation, deformation of the facing, low density and low quality of the fill material as well as additional hydrostatic pressures from saturation of the backfill.

Typical design related issues include the incorrectly assumed soil properties, omission of or failure to design an adequate drainage system, and incompatibility of the design with the true conditions on site. Other issues attributable to the designer include inadequate construction monitoring and poor standard of the construction drawings.

The data obtained from the case studies in Appendix B is examined further by focusing on each component of the system, as well as the environment to which the retaining walls were exposed, methods in which the retaining wall systems were disrupted, problems with the design and construction of the retaining walls as well as other useful information that could contribute to meeting the objectives of this study.

7.2. Data Examination

7.2.1. Soil

7.2.1.1. Backfill materials

The most common backfill material, specifically in case studies 1 to 8, used behind the failed gravity CRB walls under consideration was from the Berea Red formation. This is to be expected as the soil is typically encountered in the areas where most of the failed gravity walls were located.

Berea Red soil is notorious as a moisture sensitive soil and the walls in case studies 1 to 7 encountered water problems. The moisture sensitivity of the backfill behind the wall will be reduced by adequate compaction. However, any poorly compacted backfill and the in situ soil outside the backfilled zone remains susceptible to softening due to moisture ingress. It is therefore debatable whether the failures occurred due to inadequate compaction of the backfill, the poor quality of the retained in situ soil or as a result of the inadequate water control. Theoretically any soil can be used as backfill material and any soil can be retained provided the wall is appropriately designed and constructed, but then an adequate drainage system must be implemented. As Berea Red soil is so susceptible to softening on wetting, the question should be raised whether it is correct to assume in the design that the backfill zone would remain completely drained for the expected service life of the wall. Furthermore, as gravity walls largely rely on the backfill for their stability, the assumptions made in the design with regard to strength of both the compacted backfill and the retained soil should be carefully considered. This includes the value of cohesion assumed for the material. If there is any likelihood of water ingress, the material should be assumed to be cohesion-less.

Similarly, the residual granite backfill material encountered in case studies 9, 11, 13, 14, 15 and 18 is classified as a moisture sensitive soil and the in situ soil exhibits a collapsible grain structure, therefore the considerations mentioned above remains equally important. Furthermore, as the in situ soils and poorly compacted backfill are erodible, residual granites are susceptible to backfill washout as occurred in case study 18. Backfill washout further occurred in case studies 16 and 17 where the type of backfill material was unknown. If these materials are correctly compacted and are adequately drained, they are potentially very good materials to use as backfill.

7.2.1.2. Compaction

Most of the backfill materials were poorly compacted. Poor compaction of the soil was evident in case studies 1 to 9, 11, 12, 14 and 16 to 18, which constitutes 15 out of the 18 case studies of failed CRB walls. If soil is not compacted at its optimum moisture content, it may not be possible to achieve an adequate degree of compaction, regardless of the amount of effort. In practice this tends to be an area of difficulty (Day, 2015).

Inadequate compaction has two main effects. Firstly, poorly compacted backfill is permeable and allows rapid ingress of surface water behind the wall. Secondly, the strength and compressibility of the material are adversely impacted and this impact is exacerbated by the ingress of water. Adequate compaction is therefore as essential as adequate drainage.

7.2.1.3. Disregard for the foundation soil properties

Minimal information was available regarding the founding soil in each of the case studies. It is common knowledge that a structure only is as stable as its foundation. Therefore, the founding soil and its bearing capacity are crucial design considerations in any CRB wall system.

The foundation should be designed and checked for its ability to support the wall system. To ensure the stability of any structure, it must be founded on stable, preferably natural ground with an adequate bearing capacity and not on uncontrolled fill material.

As settlement and movement of the founding soils occurred in case studies 6, 7, 12, 14, 17 and 18, it is suspected that deformation of the walls occurred, hence the stability of the walls was compromised before the final failures occurred.

Foundation sliding often occurred when the walls were founded in fill material or retained a sloped embankment. As in case studies 4 and 7, heavy rains saturated the founding soil(s), leading to the settlement and/or rotation of the foundation(s) under the imposed load of the wall(s). In case study 14, the settlement of the underlying greenstone caused the block of reinforced soil to move laterally and vertically as well as rotate forwards. This movement caused tension cracks in the soil behind the wall at the end of the reinforcement layers, which allowed water to seep into the cracks and exert additional hydrostatic pressures onto the retaining wall system.

7.2.1.4. Cement/ Lime Stabilization

Stabilisation of the backfill material to the conventional gravity CRB wall in case study 3 with cement/lime was specified but the stabilization of the soil was omitted in the construction of the wall. As the literature states, Berea Red soil responds well to lime stabilization and the use of stabilised backfill should be considered more often with this material.

7.2.1.5. Drastic variation in soil profiles

As evident in case studies 7 and 18, zones of weaker material in the backfill were to be expected as the geotechnical soil profiles of Berea Red and Residual Granite soils are known to vary substantially.

7.2.2. Reinforcing

Problems encountered with reinforcement include inadequate anchorage, length and spacing of the reinforcement, omission of the reinforcement during construction or overstressing of the reinforcement due to incorrect design. Furthermore, in certain gravity walls, the shear strength of the backfill and restraining moment generated by the facing units against the inclined face was insufficient to resist the destabilizing forces and the provision of reinforcement could have remedied this situation.

None of the failures in this study occurred due to faults in the manufacturing of the geosynthetic reinforcement.

7.2.2.1. Omission of reinforcement

The provision of geosynthetic reinforcement requires excavation of material for a distance behind the wall up to 0,8 times the height of the wall. In many residential applications, there is not enough space to permit this and gravity walls are provided instead of reinforced walls. The gravity walls specifically in case studies 4, 5 and 8 should have incorporated soil reinforcement for additional stability.

Unfortunately, all the gravity CRB walls were higher than 2m and the walls in case studies 2, 6, and 9 were inclined at 70^0 or steeper, hence the literature indicates that mechanical stabilization in the form of geosynthetic reinforcement was necessary. Stabilisation of the backfill or alternative forms of reinforcement such as soil nails used in conjunction with a CRB facing can be considered where such space constraints existed.

7.2.2.2. Incorrect type of reinforcement used

Non-woven, needle punched geotextile reinforcement was used in case study 13. This type of geotextile is better suited to separation and filtration applications as it is highly extendible. This is the most likely cause of the excessive wall movements observed in the case study.

In case study 17, the woven geotextile reinforcement was overstressed. This contributed to ongoing creep movement of the wall. This situation could have been avoided by using more layers of geotextile, a stronger geotextile or a geogrid with superior creep properties.

The wall in case study 18 contained a combination of woven geotextile in the lower half of the wall, while the upper half of the wall was reinforced with two layers of geocomposite reinforcing. The geocomposite reinforcement would further have consisted of needle-punched non-woven material. It would be logical to assume that an attempt was made to use the geocomposite not only as reinforcement, but further as a drainage medium.

Geotextile reinforcement is ideal for use with a backfill containing a high percentage of fines. In this instance, it would complement the residual granite backfill in case studies 13 and 18, but the soil properties of residual granite vary rapidly, therefore it is difficult to say. Thus laboratory tests should have been conducted to verify the soil conditions in all the case studies.

7.2.2.3. Inadequate reinforcement length and spacing

The author suspects that the woven geotextile reinforcement in case studies 12 and 16 to 18 was rolled out in the wrong direction in the sense that the weaker weft direction ran perpendicular to the facing. As a result, the reinforcement length was assumed in the initial investigation to have been too short or to have inadequate strength, but in reality it may not have been rolled out correctly.

Moreover, the author noticed that the length of the reinforcement was cut short in case study 13, 14 and 17 as it was restricted by surrounding structures. If the design engineer was aware of the surrounding buildings, this should have been incorporated in the design and a site instruction should have been issued regarding the method of incorporating sufficient reinforcement into the retaining wall system. If the height of the wall is increased, as in case study 14, the reinforcement length must be adjusted to accommodate for the increased wall height. Inadequate length and spacing of reinforcement will affect the movement of a reinforced CRB wall and can lead to overall instability.

7.2.2.4. Inadequate anchorage

Inadequate anchorage of the reinforcement was evident in case study 13. The reinforcement was restricted by a manhole or catch basin in the reinforced soil zone and the reinforcement was not sandwiched between the concertainer baskets. Furthermore, the baskets behind the manhole were omitted. This is a clear indication of shortcomings during construction and inadequate construction monitoring.

7.2.3. Facing

7.2.3.1. Inadequate strength of the blocks

The facing units in case studies 6, 10 and 12 failed due to the inadequate strength of the blocks. The crushing strength tests, namely the back line load and front line load tests indicated that the strength of the facing units in case study 10 was far below the strength for commercial blocks and therefore, the blocks were inadequate.

As the coefficient of block-on-block friction, nib shear strength per meter run of wall and the crushing strength of the blocks in case study 10 were unknown; these uncertainties should have been accounted for with appropriate safety factors in the design. The failure could have been prevented had the blocks been SABS approved and an adequate drainage system was installed.

7.2.3.2. Inadequate shear keys

Since no shear connectors existed between the block courses in case studies 7 and 10, the blocks did not interlock and therefore, a shear failure occurred. Furthermore, shear keys facilitate the stacking of the blocks at a fixed angle and therefore it is expected that many of the walls were constructed at an inclination which deviated from the design. This is clearly a general shortcoming in the design and construction of CRB walls.

7.2.3.3. Inadequate concrete foundation

Failure of the concrete foundation does not appear to have been a significant factor, but an under designed base was recorded in case study 17. As the bearing capacities of the founding soils as discussed previously were weak, adequate foundation compaction would have been necessary and an adequate concrete footing would have assisted with load distribution. In cases where very high retaining walls are constructed, the bottom courses should be filled with concrete to assist the concrete foundations.

7.2.3.4. Other

In case studies 2, 4, 5, 8, 10, 12, 16 and 17, facing units cracked due to stress concentrations, but this was a consequence of failure and not the cause.

7.2.4. Drainage

All CRB wall systems should incorporate adequate drainage mechanisms to safely deal with water as it is likely that water will come into contact with the backfill material. The literature explains that, if applicable, adequate drainage includes the provision for a high phreatic surface, retained soil drainage, drainage from paved surfaces and adjacent structures, waterproofing the backfill and tension crack sealing. It is evident from the case studies that additional hydrostatic pressures exacerbated the problems which led to many of the failures.

7.2.4.1. Inadequate drainage

When drainage is not adequately incorporated to safely deal with water issues, the wall is not designed for the additional hydrostatic pressures it is exposed to and failure is inevitable. The penetration of excessive water behind the wall not only places additional hydrostatic pressure on the wall, but lowers the shear strength of the soil too. The majority of case studies, specifically case studies 1 to 7, 9 to 12 and 14 to 18, raised the issue of an inadequate drainage system. Typically the drainage systems were inadequately designed, installed incorrectly or were partially completed when the walls failed.

As moisture sensitive soils were incorporated as backfill material for most of the walls, an adequate drainage system should have been one of the main priorities in design and construction. Unfortunately, most designs, including the designs for the walls in case studies 14 and 17, assumed that the backfill material was free draining and therefore the drainage system was not adequately designed for the true soil conditions on site.

When walls are exposed to large amounts of water, typically after a high intensity rainfall, it is noted that the engineer cannot possibly foresee the unexpected.

7.2.4.2. Blocked drainage systems

The storm water pipes in case study 18 had been blocked for long periods of time which saturated the material below the pipes and resulted in minor settlement and opening of the pipe joints. The trench settlement resulted in tension cracks developing behind the wall and exposed the backfill material to water from numerous sources. It was recorded that the drainage was blocked by siltation and debris, but it is possible that the drainage system may have been blocked by the fines in the backfill material as well.

Careful consideration should go into the design of a drainage system to ensure compatibility with all other components of the system, including the grading of the backfill, potential settlement, etc. Furthermore, the contractor and designer should ensure that the drainage system works correctly and that it is free of obstructions.

7.2.4.3. Drainage and systems founded in backfill/reinforced soil zones

It is dangerous to place a drainage system in the backfill or reinforced zone, especially if moisture sensitive soils are present. Surfaces should not slope towards the wall and water should not be collected in manholes, kerb inlets or open drains founded in the reinforced soil zone as in case studies 1, 9 and 13. The rupturing of the underground main pipes, buried sprinkler system and leaking fittings and valves directly behind the wall in case study 9 completely waterlogged the soil behind the wall and led to the failure.

The literature emphasises the importance of routing all piping away from the backfill and reinforced soil zones. This is further discussed in the chapter to follow.

7.2.4.4. Surface water directly into backfill/reinforced soil zones

Research focusing on the design, construction and failure of CRB walls emphasises the need to prevent water runoff from other structures, vegetated areas, etc. being routed directly into the backfill or reinforced soil zone. This was evident in case studies 1 to 7 as well as in case studies 14 and 16 to 18.

7.2.5. Disruption of the system

The CRB wall system can be disrupted in numerous ways. The system is disrupted when certain important system components are omitted during the construction of the wall, or after completion of construction when certain acts threaten the stability of the wall. Often walls deform due to vertical and lateral movement, some walls deform to such an extent that the stability of the wall is compromised as in case study 17. This too is regarded as a disruption to the retaining wall system.

The acts which threaten the stability of the wall include the excavation of trenches or cut embankments in front of the wall as in case study 4, as well as the installation of a pool directly behind the wall as in case study 6. It is the responsibility of the contractor and design engineer to warn the client about the risks involved with future construction in the vicinity of the CRB wall as it might threaten the stability of the existing retaining wall system.

The wall system can further be disrupted if a wall is constructed at a lower elevation, in front of an existing wall, so that the lower wall threatens the stability of the upper wall, or the upper walls places a surcharge on the lower wall in excess of the surcharge for which it was designed as in case studies 4 and 5.

If the wall is constructed at an inclination which deviates from the design as evident in case studies 3, 5, 9, 10 and 13, or the wall height is raised above the design height, as was the case for four gravity walls (case studies 5-7 and 9) and two reinforced soil walls (case studies 12 and 14), the walls are exposed to excessive surcharge loads for which they are not designed. Similarly, the system is disrupted when the backfill material is submerged and the wall is exposed to additional hydrostatic pressures as discussed previously. The aforementioned is evident in case study 4 where the brick boundary wall disrupted the tiered wall system and caused the accumulation of muddy water behind the brick wall leading to water ingress into the system.

Due to the sensitivity of these retaining wall structures, variations to the design cannot be made on a "last minute", on-site adjustment basis as in case study 5. Each wall should be designed to take all failure modes into account and to ensure that all elements which might threaten the stability of the wall are accounted for.

7.2.6. Environment

Certain elements impose excessive surcharges onto the wall system, or expose the system to additional hydrostatic pressures above those for which it was designed. This occurs when the design does not incorporate certain environmental influences which threatened the stability of the wall. A perfect example would be case study 11 where a portion of the wall flooded after an intense rainstorm, as the wall encroached into the flood plain of a river. The engineer was not familiar with the requirements for river/stream management and therefore the design did not consider critical failure modes. Similarly, the wall in case study 6 failed as the wall had to deal with an upstream phreatic profile associated with a seepage zone at the toe of the wall.

In numerous case studies as discussed previously, the soil on-site was of too low quality to be used as backfill material, the drainage system was inadequate and residential areas did not allow sufficient space for construction, hence alternative retaining methods should have been investigated.

7.2.7. Construction

Typical construction-related issues include the omission of important elements of the structural system, such as drainage as in case studies 2, 5 to 7, 9, 10 and 12, or backfill stabilization as in case study 3, as well as the inadequate compaction of the backfill material and incorrect installation of reinforcement in case studies 11, 12 to 14 and 16 to 18. Unacceptable construction practice arises from the use of unskilled labour and level of local building practice in South Africa, as well as the speed of construction which largely influences construction quality of the wall.

7.2.8. Design

The walls in case studies 3 to 9, 11 to 12 and 14 to 18 failed due to inadequate design. The design issues encountered in assignee and expert reports for each case study can be seen in the case study summaries attached at the end of this report.

It is cause for concern that a design did not even exist for case study 4 as no structural calculations were prepared. Similarly, the design of the retaining wall in case study 6 did not ensure that the wall was stable against the modes of failure relating to internal stability with a suitable factor of safety. The remaining walls were designed, but fundamental errors existed in some of the calculations. Numerous designs such as in case studies 5, 14 and 16 incorrectly assumed the properties of the backfill materials and not all the surcharges were taken into account as in case studies 4, 5, 6 and 12. Furthermore, additional ground water pressures were not taken into account in the design of the retaining walls as evident in case study 9.

In case study 7, no calculations existed to determine the extent of shear keys required. Furthermore, the safety factors were below 1.5. The design in case study 8 incorrectly calculated the active pressure coefficient, which resulted in a lower active pressure, approximately a quarter of what it should be. In addition, the vertical component of earth pressure was ignored, and the wall friction was not incorporated in the design, which results in unrealistic design assumptions. These significant design errors are unacceptable.

Further examination of the case studies highlight the lack of understanding of CRB wall design through the following design errors:

In case studies 4, 5 and 12, the designers did not check the walls against all failure modes, the design height of the walls were incorrectly calculated and the reinforcement in case study 16 and 18 was incorrectly determined. Furthermore, no calculations existed to determine the structural adequacy of the foundations as in case studies 4 and 17, and no measures were incorporated to prevent the migration of soil particles through the gaps in the facing units in case studies 16 and 17, therefore backfill washout occurred.

In numerous case studies, including case studies 4, 5 and 9, the design did not represent the true conditions on site. The failure of the wall in case study 17 was caused by inherent deficiencies in the fill/wall combination. According to the CMA design manuals:

- The design should have provided for a drainage immediately behind the facing units;
- The bottom facing units should have been founded on an adequately sized concrete footing;
- The base block should not have been installed at an angle larger than 15^{o};

- The design should have incorporated a drain at the base/toe of the fill immediately behind the facing/wall; and
- Existing steep slopes should have been cut back in a stepped fashion.

In addition, the design of the wall in case study 14 had the following shortcomings:

- The founding soil was highly compressible and had a very low bearing capacity when saturated; and
- The material used as backfill behind the wall was not suitable.

The design engineer should ensure that all critical design components, such as the concrete foundation and drainage system, are incorporated during construction, through adequate construction monitoring. If critical information is omitted from the construction drawings, the contractor cannot possibly foresee that the components should be constructed. Often construction drawings lacked the following crucial information, specifically in case studies 3 and 12:

- No specifications for the composition or compaction of the soilcrete behind the blocks;
- No horizontal spacing for the drains extending from behind and through the soilcrete to the front of the wall;
- No dimensions for the concrete storm water drain at the top of the wall;
- No information on the Bidim wrapped sand drains;
- No specifications for the compaction of the soil in and behind the facing units;
- Material properties of the backfill and compaction standards;
- Type and placement of the reinforcing;
- The tensile capacity of the reinforcing;
- No dimensions for the concrete foundation; and
- Benching into the natural cut face.

7.2.9. Other

7.2.9.1. Design height and wall inclination

The average design height of the gravity and reinforced soil CRB walls were 5.4m and 6.65m respectively. The average wall inclination was 73° to the horizontal and the top slope behind the wall was typically horizontal.

7.2.9.2. Service life

The average service life of gravity and reinforced soil CRB walls at the time of failure was 2.5 years and 2.1 years respectively. In this calculation it was assumed that walls which failed during construction had a 0 year service life and walls with a service life of more than 10 years were assumed to have a 10 year service life. It is a common belief that structural defects start to occur 2 to 3 years (two wet and two dry seasons) after construction. This can possibly be ascribed to the consolidation of the supporting soil layers as well as the consolidation of the backfill and retained soil.

7.2.9.3. Tiered walls

The tiered walls were generally constructed higher than 5.5m. The majority of tiered walls were conventional gravity walls.

If tiered walls are used, the literature states that reinforcement should be considered. Only the two tiered walls in case study 12 and 15 were reinforced with geosynthetics. The wall in case study 15 failed due to the saturation of the pea gravel in the upper portion of the wall and the wall in case study 12 failed due to foundation settlement and the poor quality of the backfill material.

All of the conventional gravity tiered walls had a poorly compacted backfill material, nevertheless, they failed as the result of design issues. The majority failed due to global instability which formed a slip plane passing behind and beneath/through the gravity CRB walls.

The tiered wall in case study 5 failed during construction, after an intense rainstorm. Even though the excessive penetration of the rain water catalysed the failure, the design did not account for global stability of the walls as they were not designed as tiered walls, but rather individually standing walls which did not incorporate additional surcharges from surrounding structures or walls.

7.2.9.4. Location and time of failure

Most failures which occurred in Kwa-Zulu Natal in 2007 included external water as a basic failure mechanism, which indicates that Kwa-Zulu Natal possibly experienced high rainfall as well as storm conditions in that specific year.

Through the classification and examination of the information obtained from the case studies, the main findings are presented in the chapter that follows.

Chapter 8

Discussion of Findings and Recommendations

8.1. Overview

The main reasons for the failures of the 18 case studies have been identified in previous chapters. This chapter discusses these findings and makes recommendations to prevent the reoccurrence of such failures in the future.

8.2. Discussion and Recommendations

8.2.1. The use of moisture sensitive soil in the backfill/reinforced soil zone

Backfill material should preferably consist of a free draining, granular material with a low plasticity index. Fine grained materials with a high plasticity index should be avoided as they tend to absorb water and hinder its flow, both of which increase the loads imposed on the retaining wall structure. Furthermore, these materials may swell on wetting.

It is not only fine grained plastic soils that can cause problems. Some sandy soils of low to moderate plasticity can also be problematic if not treated correctly. If the density of the in situ soils is low or if they are used as backfill without adequate compaction, such soils tend to reduce in both strength and stiffness when they are saturated, causing an increased pressure on the wall and settlement of the backfill.

The moisture sensitive soils used as backfill in the case studies, namely the Berea Sands along the KwaZulu-Natal coast and the residual granites north of Johannesburg, were the in-situ soils which were reused due to the availability of the soil. Theoretically these soils are acceptable for use as backfill, but should be properly accounted for in both design and construction. Furthermore, due to the large variation in grading, standard laboratory tests should be carried out to determine the suitability of the material for use as backfill.

8.2.2. The poor placement and compaction of backfill coupled with lack of inspection

The extent of the backfill behind a CRB wall is often too limited to permit the use of large compaction equipment. Furthermore, large equipment cannot be used for compaction immediately behind the wall as this will dislodge the blocks. With smaller compaction equipment, it is essential that the layer thickness be reduced and that the soil be compacted at its optimum moisture content. It should be kept in mind that the Berea Red soil often has to be dried out to reach its optimum moisture content.

All material should be placed in layers and compacted to not less than 93% MOD ASSHTO maximum dry density. Field density and moisture content tests should be executed after the placement of each compaction layer. Samples of the material from the fill should be taken on a regular basis to monitor the quality of the material and determine the appropriate maximum dry density and optimum moisture content against which to assess the compaction test results.

8.2.3. Placing of drainage in the backfill/reinforced soil zone

When any drainage is placed in the backfill/reinforced soil zone, leakage or breakage of any component of the drainage system or water catchment structure, as well as associated transmission piping and/or pressure piping can potentially threaten the stability of the wall, especially if the wall was not designed to accommodate additional hydrostatic pressures. Therefore, the placing of drainage in the backfill/reinforced soil zone directly behind the wall should be avoided.

8.2.4. Poor control of ground water and surface water

Poor control of ground and surface water was observed in many of the case studies.

An effective seepage and surface water management system is crucial to limit water ingress into the backfill, the reinforced soil zone and the retained soils. Where water ingress cannot be avoided, e.g. where it is due to seepage within the retained material, an effective drainage system should be provided or provision should be made in the design for water pressures within the system. In particular, pressures due to water-filled tension cracks should be considered.

8.2.5. Incorrectly assessed and/or misunderstood design details

When certain structural components of the walls as-built deviate from the design, the margins of safety are compromised and dimensional tolerances and aesthetic limits may be exceeded (Pequenino, et al., 2015). Regular construction monitoring is of utmost importance and should be executed on a regular basis. A site inspection checklist or construction monitoring guide should be drafted to ensure that the wall is constructed according to the design. This checklist/guide will further assist in ensuring that no information is omitted from the drawings issued to the contractor.

Recommendations are listed below to be included in the construction monitoring guide. The contractor and the supervising engineer should ensure that the wall is correctly constructed with particular attention to:

- Benching into existing material;
- Casting of the foundation/levelling pad to ensure correct line, grade and offset is achieved (Pequenino, et al., 2015);
- Erection of the facing units and subsequent fill placement and compaction;
- Installation of the drainage system:
    - ♦ Base drain;
    - ♦ Back drain or chimney drain; and the
    - ♦ Geosynthetic separators.

- Mixing and placement of soilcrete; and
- Installation of the reinforcement and waterproofing.

Furthermore, the following should be ensured:

- Rainwater is routed away from the CRB wall during construction and the site is protected from water damage/ingress;
- The backfill is stockpiled and handled in a manner that prevents contamination with other materials;
- The foundation is cast a minimum of 12 hours before the placement of the first course of facing units (Pequenino et al., 2015);
- Backfill material is compacted to the specified percentage of the MOD ASSHTO maximum dry density. Pequenino et al. (2015) recommend a frequency of one density quality assurance test for every 1.5m height per 30m length;
- At no time must any construction equipment be in direct contact with the reinforcement (Pequenino, et al., 2015);
- No heavy machinery must be utilized within 2m from the facing. The area between the facing and 2m behind the facing should be compacted with hand operated machinery (Pequenino, et al., 2015);
- Design height and founding depth is correct;
- Correct batter angle of the first two rows of facing units is achieved; hence the
- Correct inclination is achieved; and the
- Reinforcement is rolled out in the correct direction, with the warp direction perpendicular to the facing when woven geotextiles are used.

A final inspection of the wall as-built should be done to ensure that the design engineer is satisfied with the retaining wall before the completion certificate is signed.

Pequenino et al. (2015) explains that there is a need to specify requirements for contractor experience, as the construction of CRB walls can become quite complex. The complexity of the construction of CRB walls and maintaining lines and levels, particularly for very high walls and especially when ongoing movement of the walls occurs, can become very challenging and complicated. Therefore, they recommend that requirements of contractor experience are made before the construction phase of the project commences.

Pequenino et al. (2015) further states that the issue of shared design responsibility complicates the contractor-supplier relationship. The supplier is responsible for the guarantees and quality of the construction materials delivered to site, while the contractor is responsible for the storage, placement and erection of the wall. Material guarantees are nullified when the construction materials are damaged or placed incorrectly, but it is the supplier's responsibility to ensure his requirements are met. Unfortunately the supplier is hardly ever on site. This is a grey area which should be clarified between the supplier and contractor.

8.2.6. Inadequate performance monitoring

Pequenino et al. (2015) suggest that a performance monitoring programme should be established to ensure the safety and economy of the structure. They suggest that the performance monitoring programme should aim to:

- Confirm design stress levels and monitor safety;
- Allow construction procedures to be monitored based on construction performance;
- Control construction rates;
- Enhance knowledge of the behaviour of CRB walls to verify design assumptions and to establish a basis for future design; and
- Provide insight into maintenance requirements by means of long term monitoring.

8.2.7. Incomplete construction drawings and specifications

Drawings and specifications serve three important purposes. They are the means by which the designer communicates the design requirements to the contractor. They provide the information required for the contractor to price and plan the work. Finally, they form the basis for checking of the contractor's work on site.

Early on in the design process, the designer should inspect the site conditions and assess the need for any special provisions to be made in the design. For example, if the reinforcement length is obstructed by surrounding structures, the founding conditions are poor, or there is a need for special construction procedures, this should be taken into account during the design and should be indicated on the construction drawings. The designer should also ascertain what the site will be used for after the wall has been completed as this could influence the design requirements.

The contractor should also inspect the site to assess any aspects of the site that could affect the execution of the work such as access, working space, need for dewatering, excavation conditions, and so on. If required to do so, the contractor should also determine suitable sources of backfill material or conduct tests on material available on site to ensure compliance with the requirements of the specifications.

The contractor should be issued all plans, drawings and material specifications as well as the construction sequence. The following should be included on the construction drawings:

- Specifications for the facing units:
    - ♦ Type, height, offset and all other block properties; as well as
    - ♦ Specifications for the shear keys.
- Backfill material properties, specified in terms or recognised standards such as SABS 1200M;
- Backfill compaction requirements in and behind the facing units;
- Specifications for any soilcrete required;
- Specifications for the drainage system including:
    - ♦ All dimensions for the concrete components;
    - ♦ Horizontal spacing for the weep holes or drains; and
    - ♦ Information on the bidim wrapped sand drains.
- Specifications for the reinforcement including:
    - ♦ Tensile capacity of the reinforcement;
    - ♦ Length, vertical and horizontal spacing; as well as the
    - ♦ Type of reinforcement and reinforcement properties.

- Dimensions, line, grade and offset of the foundation;
- Concrete specifications for foundations and filling of blocks; and
- All information on benching into the existing material.

8.2.8. Disruption of the retaining wall system

Due to the sensitivity of CRB wall structures, deviations from the design can adversely affect the stability of the walls as seen in the case studies under consideration. If any alterations are to be made to an existing CRB wall or alterations are made during construction, the design of the wall should be verified to ensure these alterations can be accommodated and that the stability of the wall will not be compromised through the execution of these alterations. If the original design does not accommodate the alterations, irrespective of the extent to which the wall should be altered, the CRB wall should be re-designed to ensure the stability of the wall against all failure modes. Furthermore, the client should be made aware of the aforementioned and the consequences which could arise from the disruption or alteration of the CRB wall system.

8.2.9. The use of inadequate facing units

SABS approved facing units should be used to ensure that the block specifications are up to standard and that the quality of the facing units are adequate. Where facing units are not so approved, tests should be carried out to demonstrate compliance with the design requirements relating to the strength and dimensions of the blocks. Strength tests should include block-on-block friction tests, back and front line load crushing strength tests and nib shear strength tests.

Except in minor applications, blocks with nibs or shear keys are preferred to ensure correct alignment of the blocks, the shear stability of the facing and ultimately stability of the wall.

8.2.10. Inadequate incorporation of reinforcement or soil stabilization

For CRB walls with a design height of more than 2m or an inclination of more than 70^0 to the horizontal stabilisation of the backfill or the provision of reinforcement should be considered. Soil reinforcement is often not implemented due to space constraints and the increase in the complexity of the design and construction. In order to construct a reinforced CRB wall, the soil must be excavated out for a distance of between 0,6 and 0,8 times the height of the wall. Where this is not possible due to the proximity of existing development or the slope of the ground behind the wall, alternative methods of construction should be considered.

8.2.11. Inadequate design

Many of the designs reviewed during this study showed lack of understanding of fundamental soil mechanics. Furthermore, there was a lack of appreciation of the sensitivity of the system and the need to ensure the stability of the structure by adequate design of the wall and all its components against all failure modes. Hence, designs should always be reviewed by a geotechnical engineer experienced in CRB wall design.

The following design oversights illustrate the nature of the problem.

8.2.11.1. Incorrectly assessed site conditions

An environmental study or geotechnical site investigation is fundamental to assessing the nature of the soil conditions of the area, environmental influences which might affect the stability of the system as well as the availability of suitable backfill sources. Furthermore, such studies are necessary to establish the feasibility of the project, as well as assist in the selection of the type of CRB wall which would be most appropriate for the site, or whether alternative retaining measures should be implemented (Pequenino, et al., 2015).

The importance of a proper geotechnical investigation cannot be over emphasised. The purpose of the investigation is to provide the information for assessment of:

- The earth pressures that will be exerted on the wall by the retained soil;
- The bearing capacity and compressibility of the founding soils;
- The presence of ground water and the potential for development of a perched water table; and

- The availability of suitable backfill material on the site.

8.2.11.2. Incorrect selection or design of reinforcement

The reinforcement directly influences the stability and deflections of the wall system.

The type of reinforcement required depends on the nature of the backfill and the allowable displacement of the wall over its design life. Geogrids made from low-creep polymers are preferred in applications where movement of the wall is to be limited. Non-woven geotextiles should only be used in low walls where the extension of the geotextile will not have a material effect on the performance of the wall. Account must be taken of factors such as installation damage and environmental conditions that could cause deterioration of the reinforcement.

The strength of the reinforcement required depends on the nature of the height of the wall, surcharge loading, the nature of the retained material, and water pressures behind the wall. The allowable tensile strength of the reinforcement must take account of the design life of the structure, creep characteristics of the polymer, installation damage, and soil material factors, class of the structure as well as environmental factors which include chemical degradation, sunlight degradation, temperature degradation, hydrolysis degradation, biological degradation and polymer ageing. The allowable tensile strength is typically a relatively small fraction (±15% – ±50%) of the ultimate short term tensile strength of the reinforcement. Additional layers of reinforcement (i.e. closer spacing) may be provided in areas of the wall or applications where greater reinforcement resistance is required.

The length of the reinforcement required is governed by the height of the wall, the slope of the surface of the retained material, position and magnitude of surcharges and pull-out resistance. In certain applications including tiered walls, the reinforcement may need to be extended beyond the length required for internal stability of the wall to ensure overall stability of the system.

The designer should implement the correct type of reinforcement, which further complements the backfill material used, and correctly calculate the length, spacing, anchorage and all other components relating to the reinforcement. Furthermore, all crucial reinforcement information and specifications should be included on the construction drawings issued to the contractor.

8.2.11.3. Incorrect foundation design

The foundations for a CRB structure must be designed to prevent bearing failure, sliding, overturning and excessive settlement. All these requirements can be checked using standard geotechnical design methods. If the founding conditions are poor, possible solutions include widening or deepening the foundation or excavation and replacement of material below the foundation with mass concrete, soilcrete or compacted selected fill.

8.2.11.4. Failure to check overall stability

In the case of tiered walls and walls on slopes, the overall stability of the entire system should be checked. This normally involves a slope stability analysis to ensure an adequate factor of safety against shear failure of the ground behind and below the wall rather than failure of the wall itself.

The overall stability of tiered walls is often neglected in design as the walls are designed as isolated structures. If the lower walls cannot support all the load they are exposed to, the overall stability of the wall is compromised and slip circle planes form which pass behind and beneath/through the gravity walls as found in this study. Moreover, the maximum height of tiered walls are much higher than individual standing walls and the maximum soil bearing pressure at the foundation is much higher which would further affect the overall stability of the structure.

Walls on slopes are exposed to higher active soil forces due to the sloping backfill and they are prone to sliding on the critical shear plane. As there is no restriction on the position that the critical plane will develop, various failure planes should be analysed to ensure overall stability is obtained.

8.2.12. Conclusion

The top five major design and construction issues recognised by Koerner include:

- The use of fine grained soil in the reinforced soil zone
- The poor placement and compaction of backfill coupled with lack of inspection;
- Placing of drainage in the reinforced soil zone;
- Poor control of ground water and surface water; and
- Improperly assessed and/or misunderstood design details.

The 11 major design and construction issues recognised in this study include:

- The use of moisture sensitive soil in the backfill/reinforced soil zone;
- The poor placement and compaction of backfill coupled with lack of inspection;
- Placing of drainage in the backfill/reinforced soil zone;
- Poor control of ground water and surface water;
- Incorrectly assessed and/or misunderstood design details;
- Inadequate performance monitoring;
- Incomplete construction drawings and specifications;
- Disruption of the retaining wall system;
- The use of inadequate facing units;
- Inadequate incorporation of reinforcement or soil stabilization; and
- Inadequate design which includes:
    - ♦ Incorrectly assessed site conditions;
    - ♦ Incorrect selection or design of reinforcement;
    - ♦ Incorrect foundation design; and
    - ♦ Failure to check overall stability.

The major design and construction-related issues identified in both studies are very similar. Differences include the type of backfill material, where Koerner focuses on fine grained soils, compared to the use moisture sensitive soils in the current study. Koerner’s study does not focus on gravity CRB walls and therefore it is understandable that the current study would include the inadequate incorporation of soil stabilization above those design and construction-related issues raised by Koerner.

Furthermore, this study focuses on additional construction-related issues including the need for performance monitoring, as well as the submission of incomplete construction drawings. Oversights in design as well as fundamental design errors are highlighted in the current study. Design-related issues focusing on the improper assessment of environmental impacts, reinforcement design and improper assessment of founding soil conditions appear to be a problem in South Africa. Moreover, the implementation of inadequate facing units and the disruption of the CRB wall system, due to variations in the as-built wall from the original design, tend to be a common problem.

Due to the adverse effect of water in the failure of the CRB walls found in both studies, an adequate drainage system is recommended behind the wall. An example of such a system is shown in the sketch below.

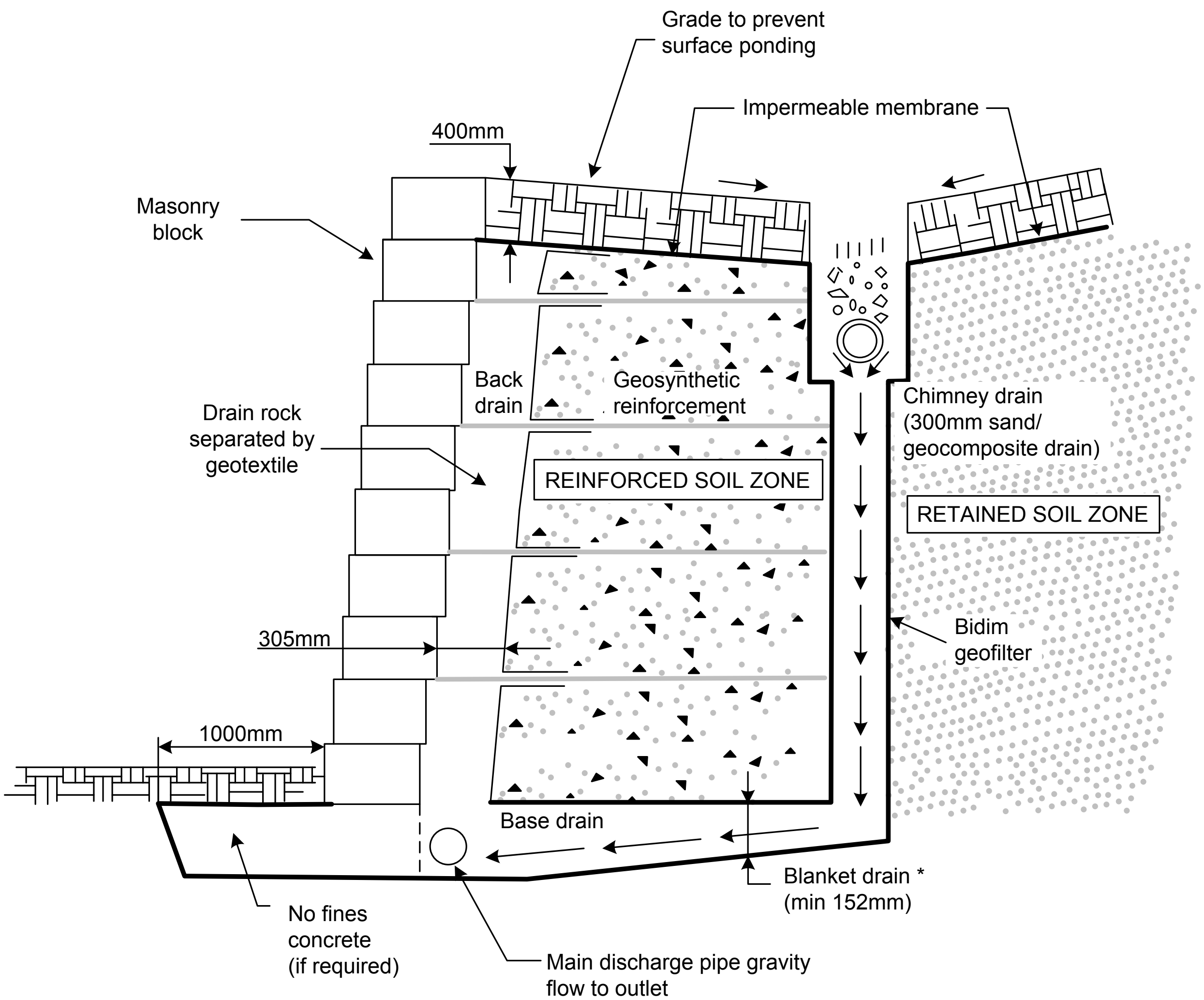


Figure 22: Recommended drainage system. Figure adapted from "A database and analysis of geosynthetic reinforced wall failures" (Koerner & Koerner, 2009) and "The importance of drainage control for geosynthetic reinforced MSE walls" (Koerner & Koerner, 2011)

The recommended drainage system limits water ingress/infiltration and provides proper drainage to the CRB wall system.

Chapter 9

Comparison with Previous Studies

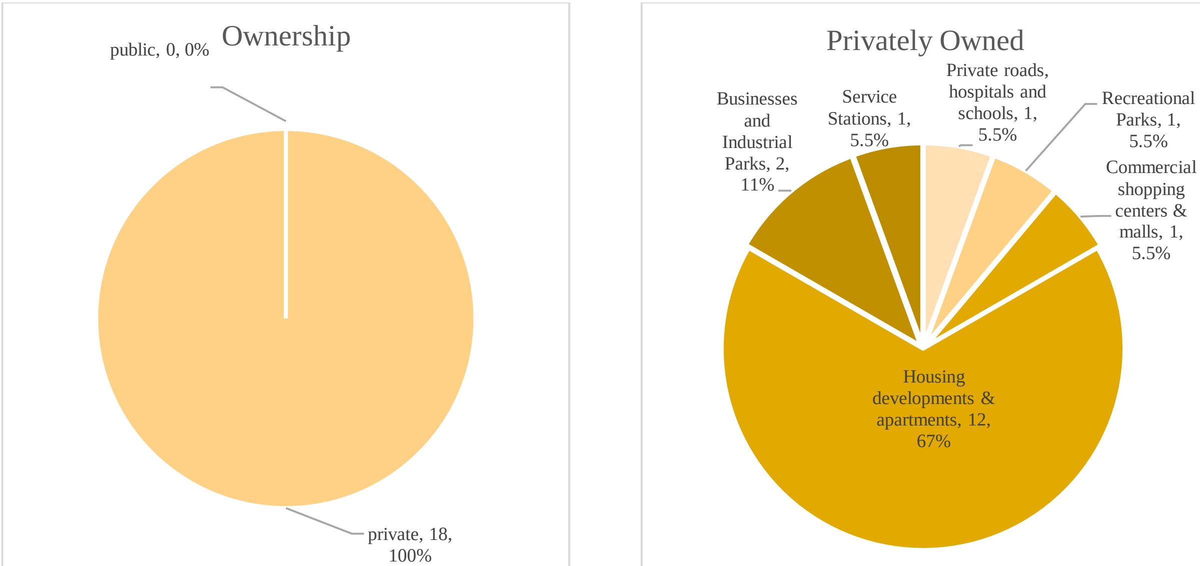
9.1. Comparison with GSI Database

The outcomes from the case studies of the 18 failed CRB walls in South Africa are compared to the 171 case studies of failed MSE walls in the GSI database. Even though the GSI database is much larger than the database of the study in South Africa and the GSI focuses on reinforced CRB walls (or MSE walls) only, the author believes that a comparison is relevant and that the outcomes of the two studies are similar. Moreover, as the classifications are made on the authors' opinions and are case specific in both studies, similarities and differences in opinions are to be expected.

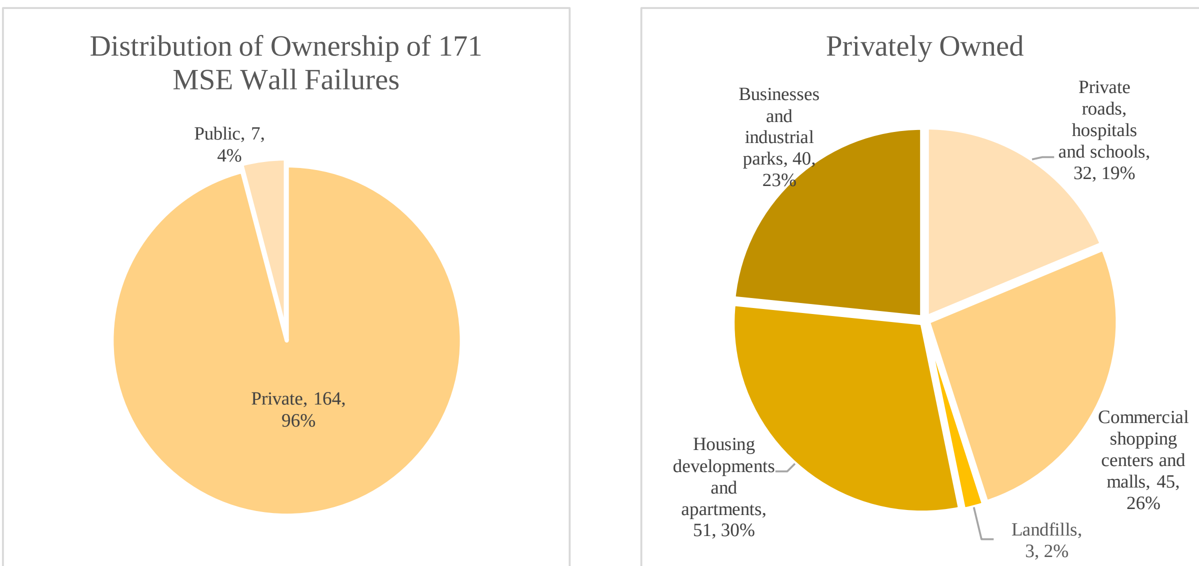
9.1.1. Wall Ownership

Wall ownership draws attention to the parties who financed the walls. The financing which was available is an indication of the quality of the final product and attention to detail in the design and construction phase of the project.

The walls in both studies were mostly privately owned. The privately owned walls in South Africa were typically located in housing developments and apartments, while the privately owned walls in the GSI database were distributed between the different privately owned sectors, specifically between housing developments and apartments, commercial shopping centres and business and industrial parks.



Graph 6: Distribution of ownership of 18 wall failures in South Africa



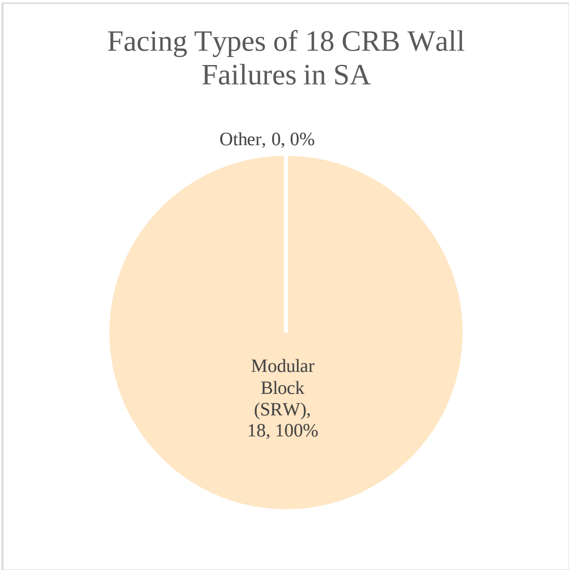
Graph 7: Distribution of ownership of 171 MSE wall failures (Koerner & Koerner, 2013)

9.1.2. Wall Location

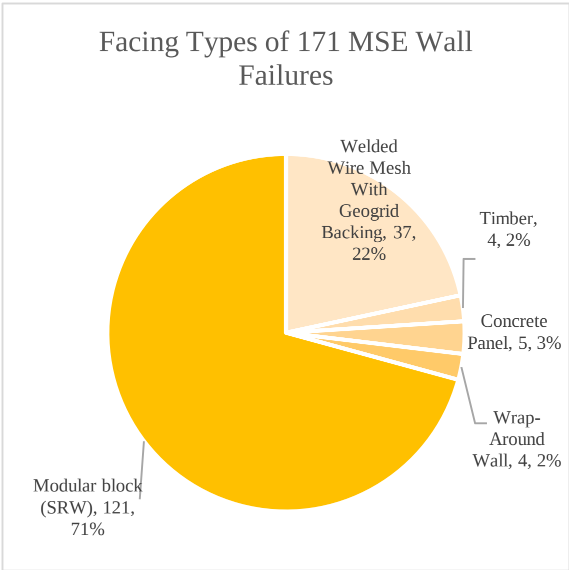
The GSI database is a global study which focuses on failed MSE walls in North America, Asia, Europe, South America, Oceania and Africa, while the failed walls in the current study are located in South Africa, specifically in Kwa-Zulu Natal, Gauteng and the Eastern Cape.

9.1.3. Type of Facing

All of the walls in the current study and the majority of the walls in the GSI database incorporated modular blocks as facing units.



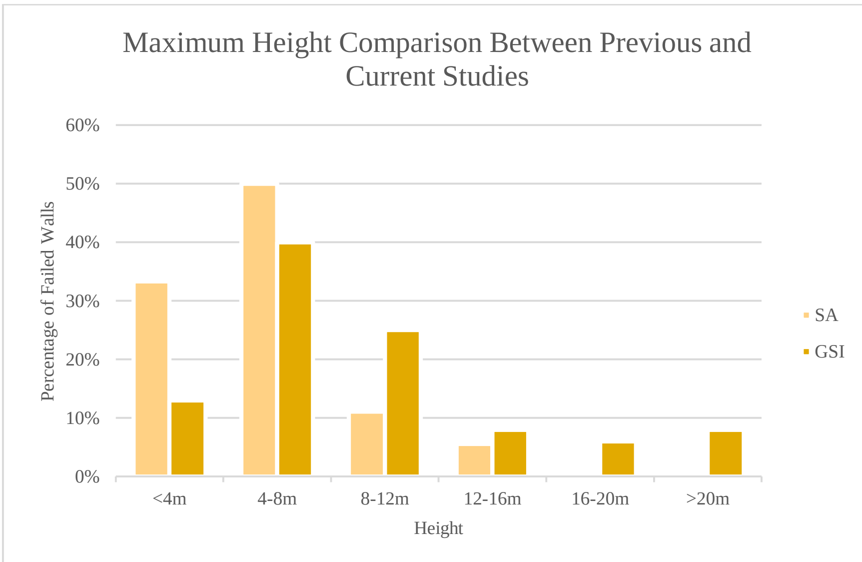
Graph 8: Facing Types of 18 CRB Wall Failures in South Africa



Graph 9: Facing Types of 171 MSE wall failures (Koerner & Koerner, 2013)

9.1.4. Maximum Wall Height

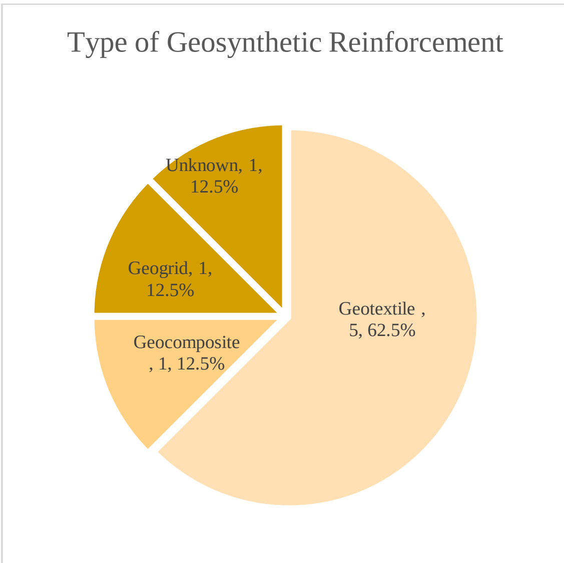
The maximum design heights of the 18 failed walls in the current study are compared to the maximum heights of the failed MSE walls in the GSI database as illustrated in Graph 10. From the bar graph it is apparent that the walls typically fell in the 4-8m height category. Most of the walls in the current study were lower than 8m while most of the walls in the GSI database were much higher with some walls exceeding far beyond the 15m maximum height in the current study.



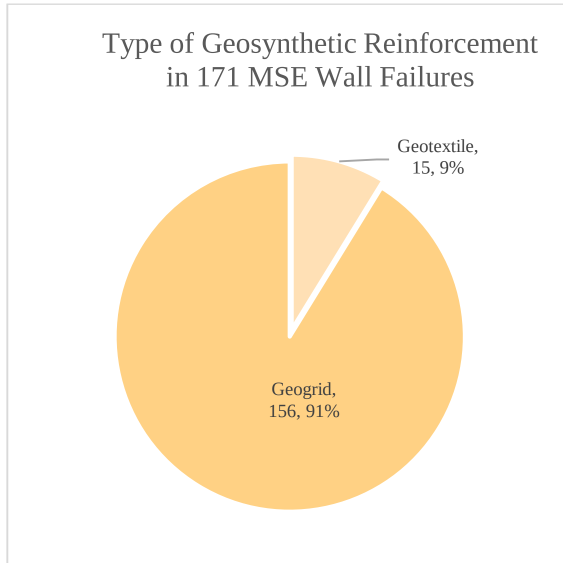
Graph 10: Maximum height of 18 CRB wall failures in South Africa compared to the maximum height of 171 MSE wall failures by (Koerner & Koerner, 2013)

9.1.5. Type of Reinforcement

Most of the reinforced CRB walls in the GSI database implemented geogrid reinforcement while the majority reinforced walls in the current study incorporated geotextile reinforcement.



Graph 11: Types of geosynthetic reinforcement in 18 CRB walls in South Africa

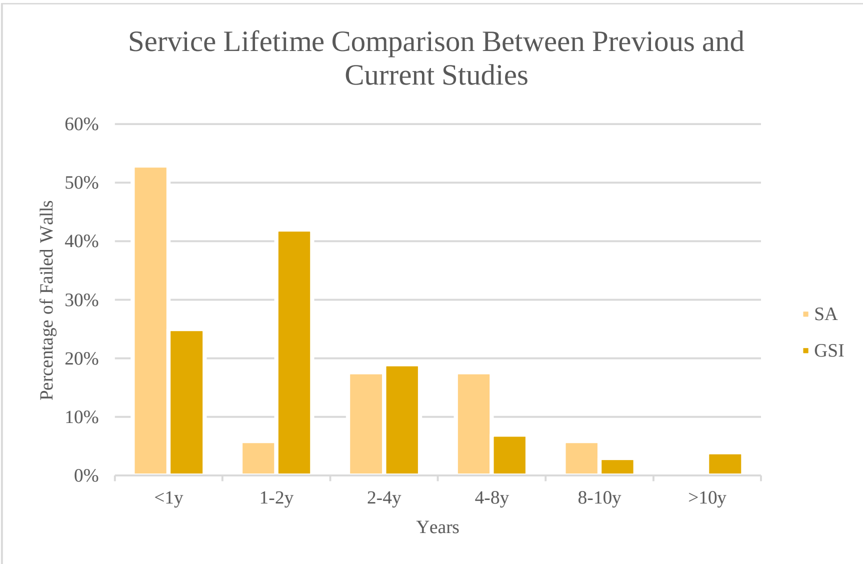


Graph 12: Types of geosynthetic reinforcement in 171 MSE walls (Koerner & Koerner, 2013)

Koerner et al. (2009) did not mention the specific type of geogrid or geotextile, as none of the failures reported in the previous study were reinforcement related. The author notes the importance of fine-grained backfill material which should not be used with the geogrid reinforcement, as James (2006) warns that the bond between grid reinforcing and fine-grained soil is poor.

9.1.6. Service Lifetime of the CRB Walls

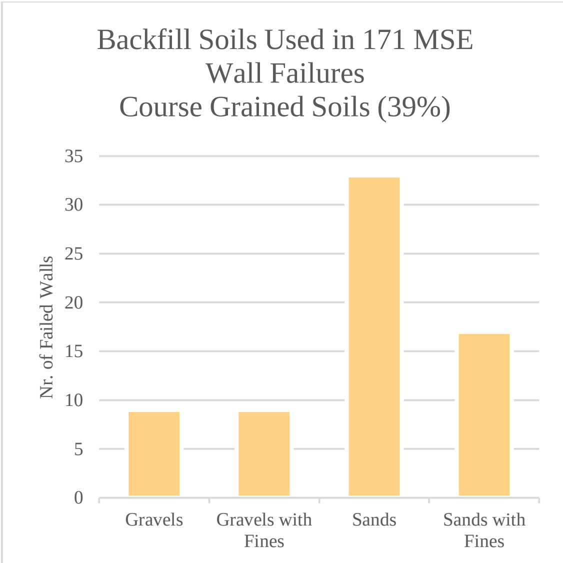
Most of the walls in the current study failed less than a year after completion of construction, while the walls in the GSI database failed in less than two years. The service life trends after 2 years differ slightly between the two studies with the service life in the current study spiking in the 4-8 year category. Few walls in the GSI database exceeded the maximum service life of the walls in the current study, but not to a large extent.

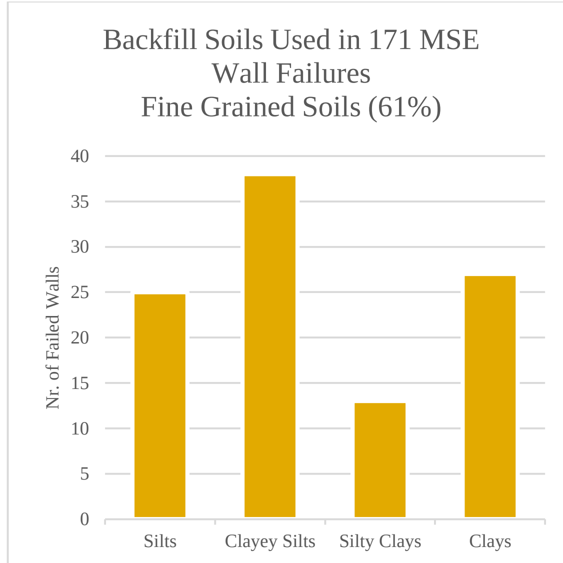


Graph 13: Service lifetime of 18 CRB wall failures compared to the service lifetime of 171 MSE wall failures (Koerner & Koerner, 2013)

9.1.7. Type of Backfill Material

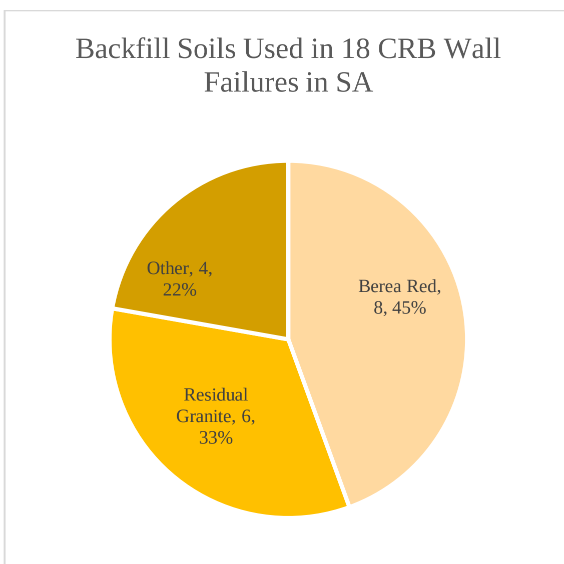
The backfill materials were classified differently in the different studies as the walls in the GSI database encountered mainly fine grained soils, while the current study encountered mostly moisture-sensitive granular soils with a large variation in their grading. The different classification styles for the GSI database and the current study can be seen in Graph 14 and Graph 15 respectively.





Graph 14: Backfill soils used in 171 MSE wall failures (Koerner & Koerner, 2013)

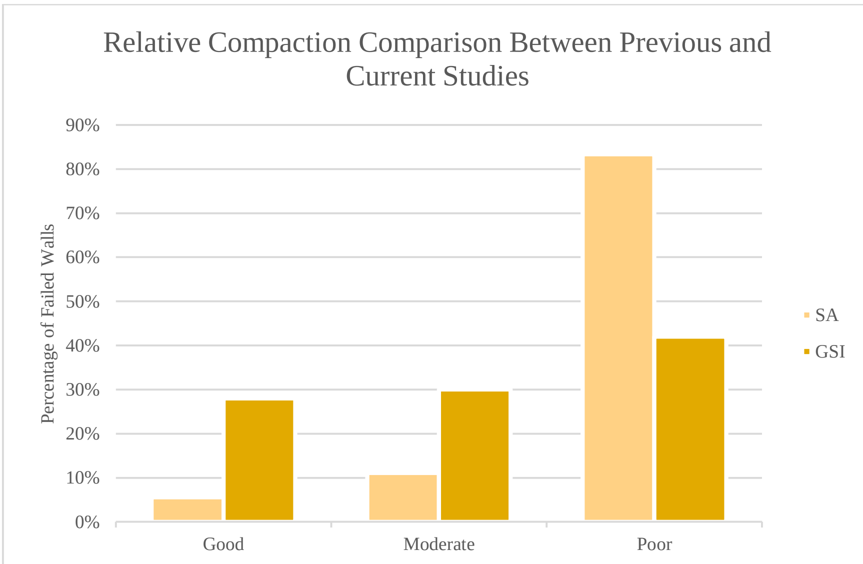
The author struggled to classify the backfill materials in a similar manner, as the main soil groups contain properties which vary widely, both laterally and vertically. Furthermore, the information available regarding the properties of the soils was inconsistent and incomplete, therefore classification of the material in a similar manner as used by Koerner and Koerner (2013) was not possible. The backfill material for the South African case histories is classified as seen in Graph 15.



Graph 15: Backfill soils used in 18 CRB wall failures in South Africa

9.1.8. Degree of Compaction of the Backfill Material

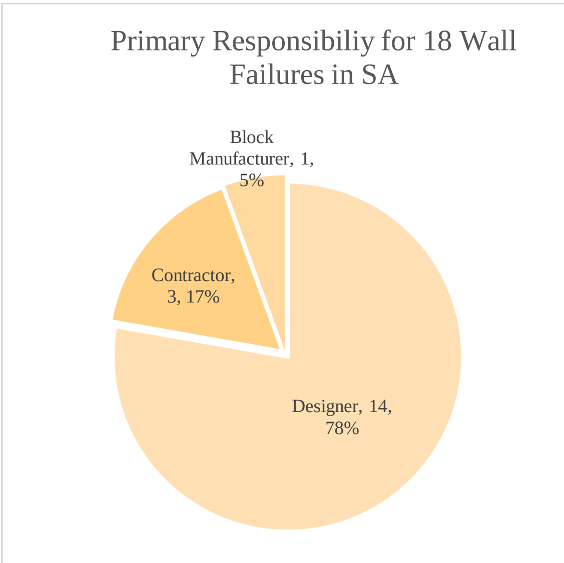
The relative compaction of the backfill material in both studies is quite telling. This is not only a construction quality control (CQC) issue, but insufficient implementation of construction quality assurance (CQA) as well.



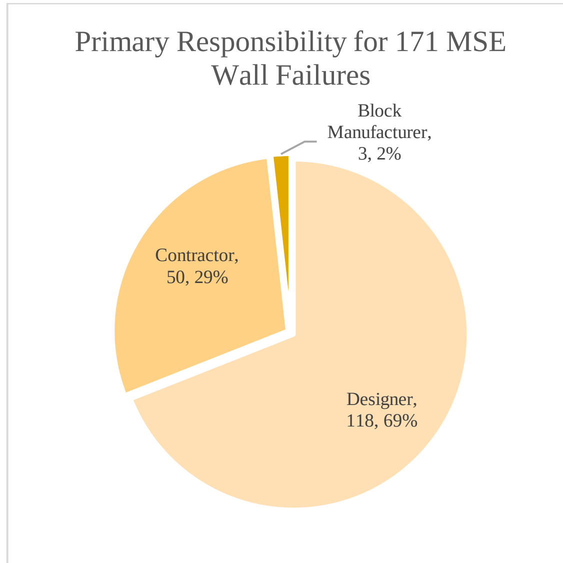
Graph 16: Relative compaction of 18 CRB wall failures in South Africa compared to the relative compaction of 171 MSE wall failures (Koerner & Koerner, 2013)

9.1.9. Person(s) Primarily Responsible for the Failure

The failures typically occurred as a result of design and construction related issues.



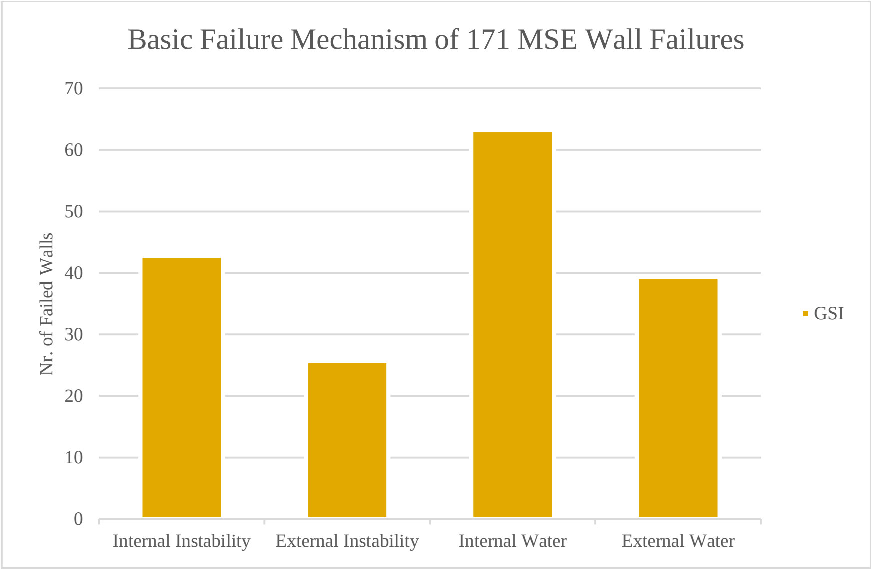
Graph 17: Primary responsibility for 18 CRB wall failures in South Africa



Graph 18: Primary responsibility for 171 MSE wall failures (Koerner & Koerner, 2013)

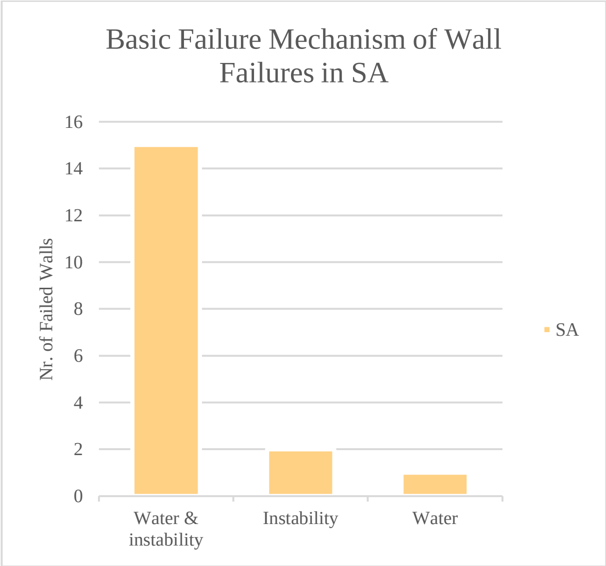
9.1.10. Basic Failure Mechanism

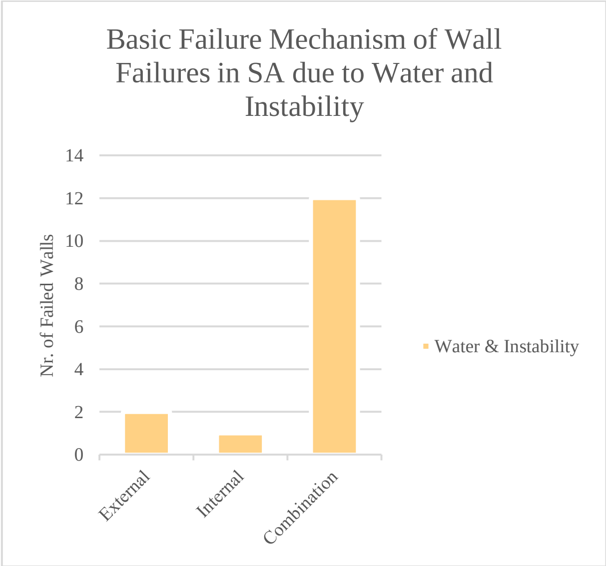
The basic failure mechanisms in the GSI database were classified according to their primary cause of failure, while the current study classified the failures according to multiple failure mechanisms as seen in Graph 19 and Graph 20 respectively. The primary failure mechanisms in the current study were not always evident due to multiple issues encountered in the design and construction of the walls, therefore, multiple problems could have instigated or catalysed the failures.



Graph 19: Basic failure mechanisms of 171 MSE wall failures (Koerner & Koerner, 2013)

The failures in the GSI database typically occurred due to internal water issues, followed by internal stability and external water issues, and lastly due to external instability issues. The failures in the current study mostly failed due to a combination of water and instability issues.





Graph 20: Basic failure mechanisms of 18 CRB wall failures in South Africa

Chapter 10

Conclusion

The main objective of this study was to extensively review as many case histories of CRB wall failures as possible, in order to discern common trends of aspects which typically cause problems with the design and construction of these types of retaining walls. By identifying the trends in the failures, engineers can gain a better understanding of how CRB wall systems work, how they fail and which aspects should receive particular attention during the design and construction stages. Hence, a more reliable retaining wall can be constructed that will meet all the structural, environmental and construction demands.

From a total of 28 case histories obtained from ECSA's Investigating Committee files and a private engineering firm, 18 case histories contained sufficient information for the purpose of this research. An outcome for each case study was determined from the case study files. The outcome of each case study gave a description of the failure, details of the problem and design-related issues identified by others in assignee and expert reports, which potentially led to the failure.

10.1. Reasons for Failure

From the outcomes of the case studies, the walls were classified and the failures and basic failure mechanisms were described according to specific criteria. The data was examined to identify the trends in the failures, which further assisted in identifying the reasons for the failures of the gravity and reinforced soil CRB walls. The 11 major design and construction issues recognised in this study include:

- The use of moisture sensitive soil in the backfill/reinforced soil zone;
- Poor placement and compaction of backfill coupled with lack of inspection;
- Placing of drainage in the backfill/reinforced soil zone;

- Poor control of ground water and surface water;
- Incorrectly assessed and/or misunderstood design details;
- Inadequate performance monitoring;
- Incomplete construction drawings and specifications;
- Disruption of the retaining wall system;
- The use of inadequate facing units;
- Inadequate incorporation of reinforcement or soil stabilization; and
- Inadequate design which includes:
  - Incorrectly assessed site conditions;
  - Incorrect selection or design of reinforcement;
  - Incorrect foundation design; and
  - Failure to check overall stability.

10.2. Recommendations to Improve Current Shortcomings

Recommendations were made to avoid the main reasons for failure in an attempt to reduce the current high failure rate of CRB walls. In conclusion, the following should always be considered in the design and construction of gravity or reinforced soil CRB walls:

Backfill material should consist of a free draining, granular material with a low plasticity index. If the material on site is to be used, standard laboratory tests should determine the suitability of the material and the design prepared accordingly. Adequate drainage works should be incorporated to safely deal with water likely to come in contact with the backfill material from any source. Furthermore, the placing of drainage works or water-bearing services in the backfill/reinforced soil zone directly behind the wall should be avoided.

To achieve proper compaction, the soil must be compacted in layers at its optimum moisture content to not less than 93% MOD AASHTO. The layer thickness should be compatible with the type of compaction equipment employed. Field density and moisture determination tests must be executed after the placement of each compaction layer and regular tests are to be undertaken to confirm the suitability of the material and the maximum dry density against which the field density tests are to be adjudicated.

Regular construction monitoring is of utmost importance and should be conducted on a regular basis. A site inspection checklist or construction monitoring guide should be drafted to ensure that the wall is constructed according to the design. A performance monitoring programme should be established to ensure the safety and economy of the structure. In addition, there is a need to specify requirements for contractor experience and the grey area between the responsibility of the supplier and contractor should be clarified.

A final inspection of the wall as-built should be done to ensure that the design engineer is satisfied with the retaining wall before the completion certificate is signed.

The contractor must be issued all plans, drawings and material specifications, as well as the construction sequence. If the reinforcement length is obstructed by surrounding structures, or the need for special construction procedures arise, it should be indicated on the construction drawings. Furthermore, the designer must ensure that all information is included on the construction drawings, and a thorough check of the drawings should be implemented before the drawings are issued for construction.

If any alterations are to be made to an existing CRB wall or alterations are made during construction, the wall should be verified to ensure that these alterations can be accommodated and that the stability of the wall will not be compromised through the execution of these alterations. If the original design does not accommodate the alterations, irrespective of the extent to which the wall should be altered, the CRB wall should be re-designed to ensure the stability of the wall against all failure modes.

SABS approved facing units should always be used, unless the facing units are tested and satisfy all the requirements of an accredited facing unit. Shear keys should always be incorporated. Furthermore, all CRB wall designs should consider the need for reinforcement or stabilisation of the backfill when the design height is above 2m and/or inclination more than 70^0 to the horizontal.

In addition to the main objectives of this study, the methods used to design gravity and reinforced soil CRB walls were reviewed and many of them were found to be flawed. The major oversights in the designs were identified. It was found that many designs did not consider all critical failure modes.

It is clear that some designers do not understand how to correctly design gravity and/or reinforced soil CRB walls and do not appreciate the sensitivity of these wall systems. Furthermore, some engineers showed a lack of understanding of fundamental soil mechanics in their designs. Specific design-related issues encountered in the current study are discussed below:

The designer should correctly calculate the length, spacing, anchorage and all other components relating to the reinforcement, as well as implement the correct type of reinforcement, which further complements the backfill material used. The founding soil should be evaluated for its suitability to support the structure and to ensure that its bearing capacity is sufficient. If the founding soil is inadequate, soil improvement techniques should be implemented.

In the case of tiered walls and walls on slopes, the overall stability of the entire system should be checked. This normally involves a slope stability analysis to ensure an adequate factor of safety against shear failure of the ground behind and below the wall rather than failure of the wall itself.

An environmental study and geotechnical site investigation is fundamental to assess the nature of the geology of the area, environmental influences which might threaten the stability of the system, geotechnical design parameters, the availability of suitable backfill sources and the presence of groundwater. The environmental study further assists to establish the feasibility of a project and assist in the selection of the type of CRB wall which would be most appropriate to construct on the specific site, or whether alternative retaining measures should be implemented.

Due to the adverse role of water in the failure of the CRB walls in previous and current studies, a comprehensive drainage system is recommended for most gravity and/or reinforced soil CRB wall systems. This recommended drainage system should ensure that water ingress to the entire reinforced soil/ backfill zone is minimised and that any adverse water pressures are addressed by means of a suitable drainage system.

10.3. Comparison with Previous Studies

The outcomes of the current case studies were compared to previous case studies of 171 failed MSE walls which form part of the GSI database. The statistical data in the current study was classified in a similar manner as to be added to this database. In addition, the reasons for failure and corresponding recommendations were compared to those raised by Koerner in his study of the 171 failed MSE walls.

The comparison of certain wall properties including wall ownership, location, maximum wall height, service life, facing and reinforcement type and soil properties, as well as the person responsible for the failure and basic failure mechanisms, highlighted similarities between the studies. Distinctive differences observed include the wall type, location, reinforcement type, backfill material and basic failure mechanisms.

Koerner studies the failures of mechanically stabilized earth walls, which are referred to as reinforced soil CRB walls in this report. The current study also includes the failure of gravity CRB walls. Furthermore, as the study by Koerner focuses on failed retaining walls on a global scale, the current study looks at failed walls in South Africa only.

The previous study further varies from the current study as Koerner classifies the backfill soil according to the grading of the soil, whereas the current study focuses on moisture sensitive soils and classifies the soils according to the formation to which they belong. Similarly, the classification of the basic failure mechanisms differ as Koerner focuses on the primary basic failure mechanism, while the current study found that, in many instances, there were more than one cause of failure of the CRB walls.

The major design and construction-related issues identified in both studies were notably similar. As Koerner’s study does not focus on gravity CRB walls, it is understandable that the current study would include the inadequate incorporation of soil stabilization in addition to those design and construction-related issues raised by Koerner.

Furthermore, the current study focused on additional construction-related issues, including the need for performance monitoring, and incomplete construction drawings. Oversights in design as well as fundamental design errors were highlighted in the current study. In addition, the use of inadequate facing units and the disruption of the CRB wall system due to variations in the as-built wall from the original design were highlighted in the current study.

In conclusion, design and construction-related problems regarding CRB walls exist and, as the use of these walls increase, failures become more common and the consequences of these failures will become more severe. Many of the designs were flawed, more due to inexperience and lack of communication between the two design entities than inadequate design manuals or codes. The current design codes and manuals covering gravity and reinforced soil CRB wall design are not inadequate, but can be improved to specifically focus on certain design aspects. Furthermore, focus should be placed on improving the quality of drawings issued for construction. CRB wall design should be considered a specialized field, not only in design, but in construction as well. Significant construction issues do exist and therefore a proposal is made to consider extending the scope of SABS 2001 to include standard specifications for the construction of CRB walls, as well as the incorporation of a performance monitoring programme.

Chapter 11

Recommendations for Future Studies

Although many studies have been conducted on CRB walls, the research can further be extended to improve the design and construction of CRB walls and eliminate the possibility of failure.

The following recommendations are highlighted for future research which would complement this study:

11.1. Alternative Soil Retaining Methods

Alternative soil retaining methods should be studied which are just as quick and easy to construct, and provide an economic means of retaining soil, but are not as sensitive as CRB wall structures and allow for higher construction and design tolerances.

11.2. Construction Monitoring

An extension on the available guidelines for construction monitoring should be studied, with particular guidance to avoid potential problems which might arise. Furthermore, guidelines for performance monitoring should be implemented to ensure the safety and economy of the structure.

11.3. Environmental Studies

Standards on environmental studies or geotechnical site investigations should be studied and drafted to publish guidelines for future CRB wall design and construction.

11.4. Standardised Design Methods

Studies should focus on the current design methods and how designs can be improved, taking all factors into account which could potentially cause instability of the retaining wall system. The design methods should be standardized in a registered SANS code with great focus drawn to drainage and the appropriateness of the backfill material used.

Future studies should conclude whether earth pressure forces should be treated as single entities, or whether the vertical component of the earth pressure forces can be regarded as giving rise to resisting moments, while the horizontal component of the same forces generates overturning moments as described in the CMA design manual for gravity CRB walls.

Furthermore, a list of design checks should be drafted to ensure that the design satisfies all failure modes and that all surcharges and factors which might influence the stability of the system are taken into account.

11.5. Practice Note for ECSA

Based on the information obtained from the research project a practice note can be compiled for ECSA in an attempt to improve the current shortcomings in the design and construction of gravity and reinforced soil CRB walls.

Chapter 12

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Annexure A

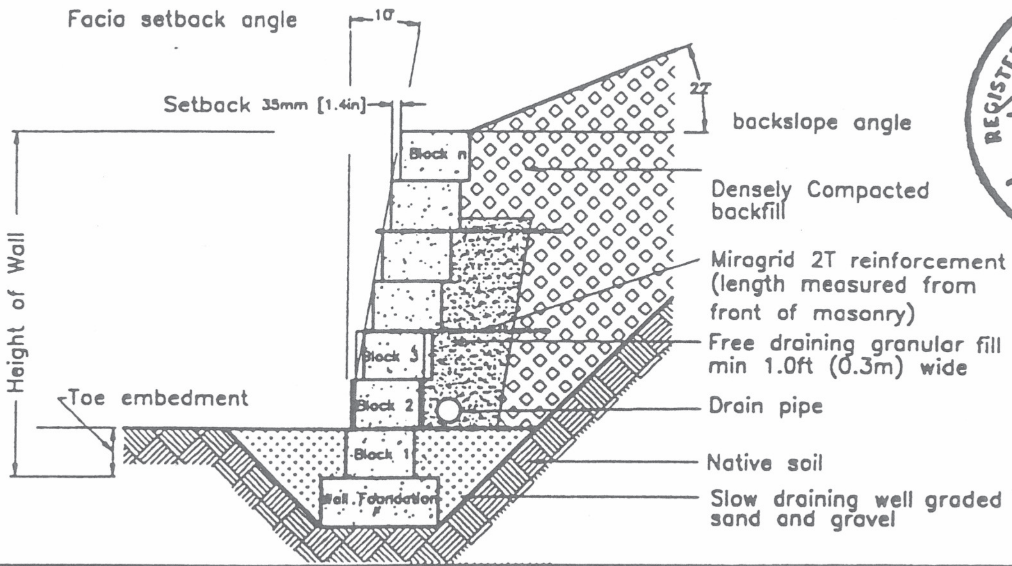
Typical TerraForce design chart for a reinforced soil CRB wall

CONDITION 8

The design tables are based on the Terraforce L.18/L.22 with a block height of 200 mm. Where the Terraforce L.11 or L.13 with a block height of 225 mm are used, the lengths of the geosynthetic layers must be increased proportionately, that is by 12.5%.

10° facia setback angle
22° backslope angle
nil surcharge
native sand/silt/clay backfill

Terraforce Design Chart 8:					
Wall Inclination from Vertical:	10 degrees		Setback of Each Block:	35 mm	1.4 in.
Backfill Soil Friction Angle:	26 degrees				
Backslope Angle:	22 degrees		Surcharge on Retained Soil:	0 kPa	0 psf
Backfill Soil Unit Weight	20 kN/m³	127 pcf			
Geogrid Reinforcement:	Miragrid 2T		Long Term Design Strength:	11.2 kN/m	770 lbs/ft
Wall Toe Embedment:	0.2 m	8 inches			



Height of wall (m)		4	3.8	3.6	3.4	3.2	3	2.8	2.6	2.4	2.2	2	1.8	1.6	1.4
Height of Wall (ft)		13.1	12.5	11.8	11.2	10.5	9.8	9.2	8.5	7.9	7.2	6.6	5.9	5.2	4.6
Grid Length (m)		4.8	4.6	4.3	4.1	3.8	3.6	3.3	3.1	2.9	2.6	2.4	2.1	1.9	1.9
Grid Length (ft)		15.7	15.1	14.1	13.5	12.5	11.8	10.8	10.2	9.5	8.5	7.9	6.9	6.2	6.2
Number of Layers of Geogrid		8	7	7	6	5	5	4	4	3	3	3	2	2	1
Number of Block Courses above Base															
Geogrid Layer Number	8	17													
	7	13	16	15											
	6	9	12	11	14										
	5	7	8	7	10	13	12								
	4	5	6	5	6	9	8	11	10						
	3	3	4	3	4	5	4	7	6	9	8	7			
	2	2	2	2	2	3	2	3	2	5	4	3	6	5	
	1 (Bottom Layer)	1	1	1	1	1	1	1	1	1	1	1	2	1	4

Notes:

1. Detailed notes regarding the use of this design chart are given in Appendix 'A' which should be thoroughly studied prior to construction.
2. This design applies to situations where the native soil consists of competent clay-silt-sand soil which has a safe bearing capacity of not less than 3000 p.s.f. (150 kPa) in regard to supporting foundations and can be densely compacted as a backfill material.
3. Geogrid reinforcement must be fully embedded between courses of Terraforce masonry units, and must be tensioned as the wall backfill is placed.
4. Wall foundation to comprise densely compacted well graded sand and crushed gravel material.

TERRAFORCE

Annexure B

Outcomes of the case studies of 18 failed gravity and reinforced soil CRB walls

Case study	Wall Classification					Wall Configuration	Reinforcement	Soil		Other				Failure Description		Basic Failure Mechanism			
	Wall Type	Max Height (m)	Service Life (years)	Inclination (degrees)	Top Slope (degrees)			Backfill	Compaction	Year of failure	Ownership	Location	Responsibility	Collapse	Deformation	External water (EW)	Internal water (IW)	External Instability (EI)	Internal Instability (II)
CS1	Gravity	5.5	3	69	0	Tiered	-	Berea Red	Poor	2007	Residential	KZN	Contractor	Behind & through	-	EW		EI	II
CS2	Gravity	3.6	3	70	26	Limiting bank	-	Berea Red	Poor	2007	Residential	KZN	Contractor	-	Localized	EW			II
CS3	Gravity	8	6	65	0	Uniform soil	-	Berea Red	Poor	2012	Residential	KZN	Designer	Upper section	-	EW	IW		
CS4	Gravity	10	<1	60	0	Tiered	-	Berea Red	Poor-Mod	2005	Residential	KZN	Designer	Behind & through	-	EW	IW	EI	II
CS5	Gravity	3, 4, 5	DC*	65	0	Tiered	-	Berea Red	Poor	2004	Residential	KZN	Designer	Full height	-	EW		EI	
CS6	Gravity	3.7	<1	90	0	Uniform soil	-	Berea Red	Poor	2011	Residential	KZN	Designer	Behind & beneath	-	EW		EI	II
CS7	Gravity	4.5	7	56	0	Uniform soil	-	Berea Red	Poor	2007	Residential	KZN	Designer	Behind & beneath	-	EW		EI	II
CS8	Gravity	7	<1	65	0	Uniform soil	-	Berea Red	Poor	2007	Residential	KZN	Designer	-	Complete			EI	II
CS9	Gravity	3.4	<1	70	0	Uniform soil	-	Residual Granite	Poor	2001	Service Station	Gauteng	Designer	Full height	-		IW	EI	
CS10	Gravity	4	4	62	3	Stable rock	-	Stable rock	Good	2008	Residential	Gauteng	Contractor & Manu.	Full height	-	EW		EI	
CS11	Reinf.	1.8	DC*	85	0	Uniform soil	Geocomposite	Residual Granite	Poor-Mod	2011	Residential	Gauteng	Designer	Behind & through	-	EW			II
CS12	Reinf.	3.3	<1	76	0	Tiered	Woven geotex.	Timeball Hill	Poor	2003	Residential	Gauteng	Designer	-	Complete		IW	EI	II
CS13	Reinf.	5.5	<1	80	0	Uniform soil	Non-woven geotex.	Residual Granite	Moderate	1997	Residential	Gauteng	Contractor	Full height	-			EI	II
CS14	Reinf.	5.8	1	75	0	Uniform soil	Geogrid	Residual Granite	Poor	2007	Business park	Gauteng	Designer	-	Complete	EW	IW	EI	II
CS15	Reinf.	15	Unknown	85	0	Tiered	Unknown	Residual Granite	Moderate	1994	Recreational park	Gauteng	Designer	Upper section	-	EW			
CS16	Reinf.	7.7	>10	80-90	0	Uniform soil	Woven geotex.	Mixture	Poor	2003	Hospital	Gauteng	Designer	-	Complete		IW		II
CS17	Reinf.	4.5	3	87	0	Uniform soil	Woven geotex.	Unknown	Poor	2000	Business park	EC[Æ]	Designer	-	Complete	EW	IW	EI	II
CS18	Reinf.	9.6	DC*	75	27	Limiting bank	Woven geotex.	Residual Granite	Poor	2014	Shopping center	Gauteng	Designer	Behind & beneath	-	EW	IW	EI	II

* During Construction

Æ Eastern Cape

CASE STUDY 1

Wall Properties:			
TYPE:	*Gravity*	**OWNERSHIP:**	*Residential*
LOCATION:	*Kwa-Zulu Natal*	**YEAR OF FAILURE:**	*2007*
WALL CONFIGURATION:	*Tiered*	**FAILURE DESCRIPTION:**	*Collapsed*

Design Parameters:			
WALL INCLINATION:	*69º*	**TOP SLOPE:**	*0º-2º*
BACKFILL:	*Berea Red*	**APPROX. HEIGHT:**	*5.5m*

Overview:

A tiered gravity CRB wall was constructed on a residential property in a suburban area to retain a slope on the property boundary to support the upper house. The site sloped very steeply downward and had been leveled by cutting on the one side and filling on the other.

Description of the failure:

Initially a slip plane formed behind the upper wall resulting in a partial state of failure in which the bottom portion of the wall bulged. This upper wall imposed unacceptable loads onto the lower wall which ultimately resulted in failure, followed by the upper wall and the entire embankment.

The wall failed within 3 years after completion of construction as a result of instability and external water. The failure stretched a total 10m along the length of the wall and evidence of soil subsidence behind the wall for a further 4m was observed.

Details of the problem:

No concrete foundation was present. The Berea Red soil found on site was reused as backfill and was incredibly loose. Furthermore, significant ant activity was encountered between the upper and lower walls, and a sprinkler system was placed in the backfill behind the retaining wall system.

The design did specify a cut-off drain above each wall, but none were built. Storm water landing on surrounding surfaces flowed towards the retaining wall. The storm water landing on the catchment areas above the wall soaked into the soil behind the wall due to the poor compaction of the soil, with excess water cascading over the top.

The contractor did not construct the wall according to the design and construction drawings which could have been prevented if adequate construction monitoring was performed.

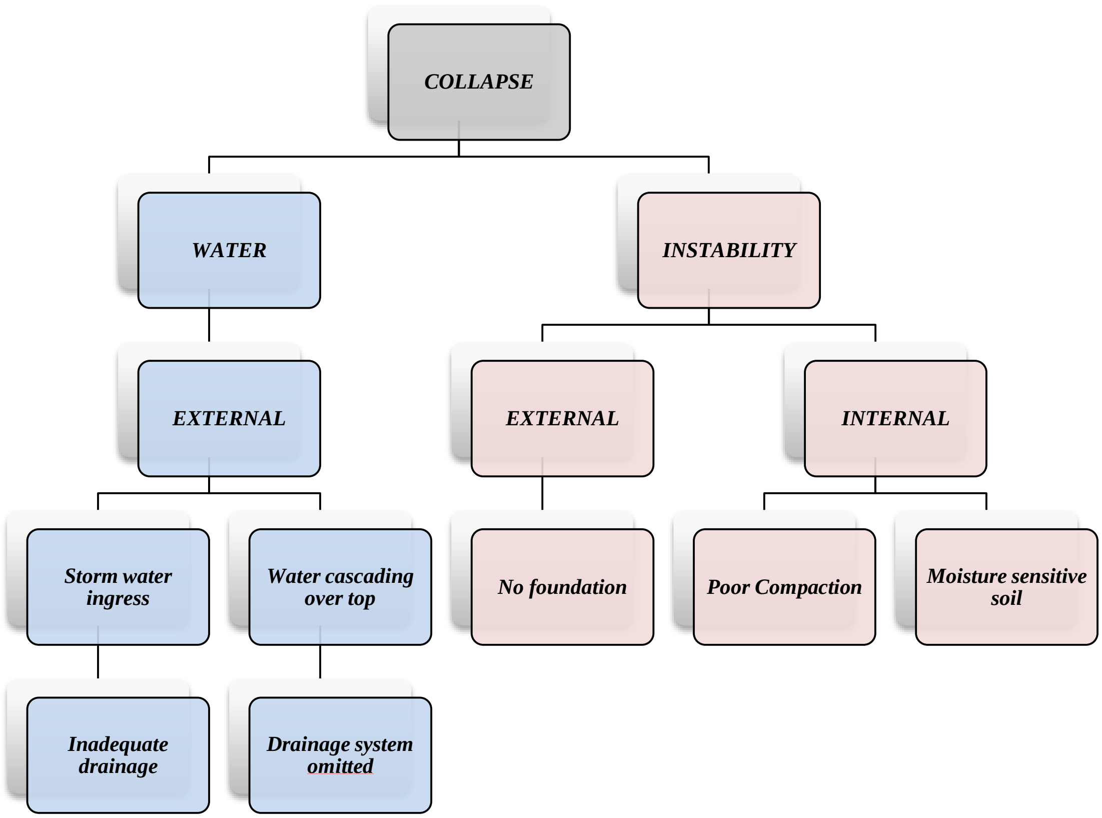
Design issues encountered by others:

Calculations showed that the wall as built did not comply with the requirements of a competent engineer designed wall. However, the original design was adequate, therefore the failure occurred due to a construction fault. The design engineer should have ensured that all critical design components, such as the concrete foundation and drainage system, were incorporated through adequate construction monitoring.

As poor soil compaction and excessive groundwater conditions existed, adequate drainage should have been incorporated to prevent the accumulation of water behind the wall. Especially as Berea Red soil is infamous for its large variation in soil properties and tendency to become a collapsible soil.

<u>Failure flow chart:</u>

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 2

Wall Properties:			
TYPE:	*Gravity*	**OWNERSHIP:**	*Residential*
LOCATION:	*Kwa-Zulu Natal*	**YEAR OF FAILURE:**	*2007*
WALL CONFIGURATION:	*Limiting Bank*	**FAILURE DESCRIPTION:**	*Deformation*

Design Parameters:			
WALL INCLINATION:	*70^o*	**TOP SLOPE:**	*26^o*
BACKFILL:	*Berea Red*	**APPROX. HEIGHT:**	*3.6m*

Overview:

A gravity retaining wall on a residential property in a suburban area was constructed at the rear of a house to retain an embankment sloping upward at approximately 26^o to the road above.

Description of the failure:

The bottom portion of the retaining wall bulged and numerous facing units cracked, all in the bottom four courses of the wall where the loads were the greatest. The wall failed within 3 years after completion of construction.

Details of the problem:

The Berea Red soil found on site was reused as backfill and was incredibly loose. The design did specify a cut-off drain above the wall, but none was built. Storm water landing on surrounding surfaces would flow towards the retaining wall. The storm water landing on the catchment areas above the wall would soak into the soil behind the wall due to the poor compaction of the soil, with excess water cascading over the top.

The contractor did not construct the wall according to the design and construction drawings which could have been prevented if adequate construction monitoring was performed.

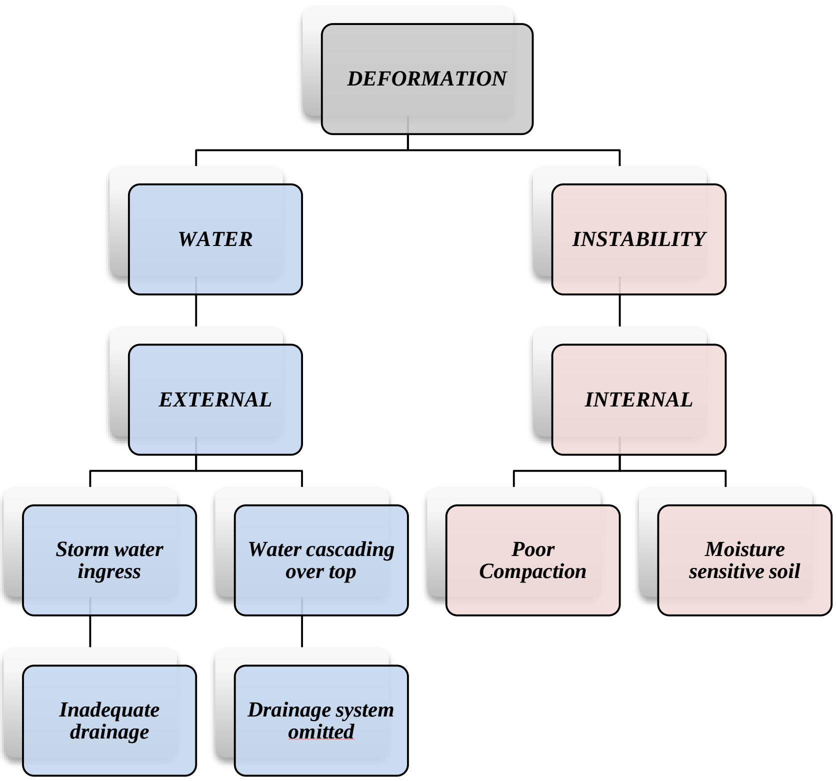
Design issues encountered by others:

Calculations showed that the wall as-built did not comply with the requirements of a competent engineer designed wall. However, the original design was adequate, therefore the failure occurred due to a construction fault. The design engineer should have ensured that all critical design components, such as the concrete foundation and drainage system, were incorporated through adequate construction monitoring.

As poor soil compaction and excessive groundwater conditions existed, drainage should have been handled correctly to prevent the accumulation of water behind the wall. Especially as Berea Red soil is infamous for its large variation in soil properties and tendency to become a collapsible soil.

<u>Failure flow chart:</u>

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 3

Wall Properties:			
TYPE:	*Gravity*	**OWNERSHIP:**	*Residential*
LOCATION:	*Kwa-Zulu Natal*	**YEAR OF FAILURE:**	*2012*
WALL CONFIGURATION:	*Uniform Soil*	**FAILURE DESCRIPTION:**	*Collapse*

Design Parameters:			
WALL INCLINATION:	*65°*	**TOP SLOPE:**	*0°-2°*
BACKFILL:	*Berea Red*	**APPROX. HEIGHT:**	*8m*

Overview:

A soilcrete enhanced gravity retaining wall on a residential property in a suburban area was constructed to retain a slope on the property boundary to support the upper two residential apartment blocks. At the rear of the two properties, a row of interconnected garages with a gap in the middle was constructed in front of the CRB wall.

Description of the failure:

A section of the retaining wall collapsed when a high intensity rainfall of 185.8mm concentrated storm water above the central section of the CRB wall. It appears that water must have accumulated behind a partially retaining brick boundary wall. Some of the water seeped into the backfill behind the CRB wall and some flowed through the drainage holes into the ground immediately behind the top of the CRB wall This would potentially have caused scour behind the wall as well as resulted in the overtopping of the wall. Furthermore, a storm water pipe discharged storm water directly into the subsoil immediately behind the wall.

The upper section of the wall failed six years after completion of construction.

Details of the problem:

Important information was omitted from the construction drawings issued to the contractor and hence were not incorporated in the final as-built wall system. The drawings did not include the following crucial information:

- *No specifications for the composition or compaction of the soilcrete to be constructed behind the blocks;*
- *No horizontal spacing for the drains extending from behind and through the soilcrete to the front*

of the wall;

- *No dimensions for the concrete storm water v-drain at the top of the wall;*
- *No information on the bidim wrapped sand drains; and*
- *No specifications for the compaction of the soil in and behind the facing units.*

Furthermore, the drawings specified a 5° back tilt for the facing units, with a wall slope of 65°, but during construction the facing units were laid horizontally with a wall slope of approximately 70°.

The contractor did not construct the wall according to the design and construction drawings. This could have been prevented if adequate construction monitoring was performed. However, the wall was not designed to have storm water discharged into the subsoil behind it or to accommodate major scour or water ingress into the backfill; therefore, the wall would still have collapsed even if it was constructed according to the design.

The engineer certified the wall while knowing that the wall had not been constructed in accordance with the design and specified (or unspecified) construction details.

Design issues encountered by others:

The design would have been acceptable provided that the following measures were taken:

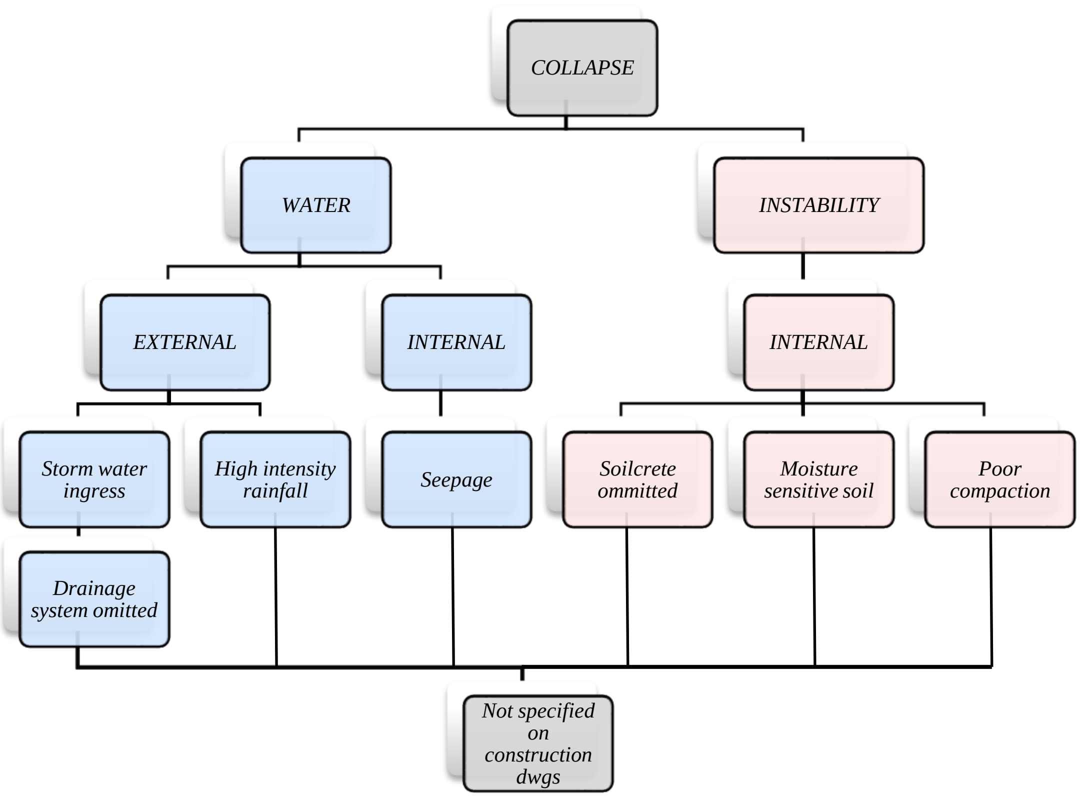
- *Storm water behind the top of the wall was controlled;*
- *The 8m high cut bank was assessed during excavation as being stable enough in the short term to enable safe construction;*
- *Wall subcontractor was fully informed as to the requirements for the inclusion of the soilcrete.*

It was noted that the engineer could not reasonably have foreseen that such a high flow of water would have been concentrated at a localized point at the top of the retaining wall. As the remainder of the retaining wall did not collapse, it would indicate that, had it not been for the high concentration of storm water behind the wall, the wall would not have collapsed.

The design engineer should have ensured that all critical design components were incorporated through adequate construction monitoring. Furthermore, the design engineer should have ensured that the wall was constructed in accordance with the design, and specified all construction details on the construction drawings issued to the contractor.

<u>Failure flow chart:</u>

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall. Certain failure mechanisms are included in the flow chart, but are not necessarily basic failure mechanisms of the wall. That is; it contributed to the failure, but was not a reason for the failure. For example; consider a moisture sensitive soil. Due to the saturation of the soil the wall failed, but the failure occurred as the drainage system, which allows for the use of this soil, was omitted and not due to the inadequacy of the soil itself.



If the drainage system was not adequately designed, internal instability would be included as one of the basic failure mechanisms as poor soil conditions existed. However, if the drainage was omitted from the as-built wall system and an adequate drainage system was included in the design, then internal instability was not necessarily a reason for the occurrence of the failure.

CASE STUDY 4

Wall Properties:			
TYPE:	*Gravity*	**OWNERSHIP:**	*Residential*
LOCATION:	*Kwa-Zulu Natal*	**YEAR OF FAILURE:**	*2005*
WALL CONFIGURATION:	*Tiered*	**FAILURE DESCRIPTION:**	*Collapse*

Design Parameters:			
WALL INCLINATION:	*60°*	**TOP SLOPE:**	*0°-2°*
BACKFILL:	*Berea Red*	**APPROX. HEIGHT:**	*10m*

Overview:

A CRB wall was constructed in a suburban area on a residential property. Five months after completion of construction, two lower walls were constructed. In conjunction with the construction of the lower CRB wall, a brick boundary wall was constructed to separate the upper and lower properties.

Description of the failure:

The existing CRB wall performed satisfactorily until construction of the two lower walls commenced. The foundation trench for the boundary wall was excavated very close to the front of the original CRB wall and a bank was cut into the natural ground in front of the trench to accommodate the new CRB wall. Heavy rain fell over 3 days while the trench was open and the cut bank was unprotected from the elements. After 6 days of construction, significant defects started showing in the upper, existing CRB wall.

The existing CRB wall was strengthened and the lower boundary and CRB walls were completed. Three months after construction heavy rains fell again resulting in the accumulation of muddy water behind the brick boundary wall. Two months later all 3 walls collapsed.

The three-wall-system collapsed as a result of a slip plane which formed behind and beneath the boundary wall and upper CRB wall, but behind and above the lower level of the bottom CRB wall. The slip caused the upper retaining wall to slide down the slope which in turn caused the collapse of the two lower walls.

Details of the problem:

The lower blocks of the bottom CRB wall rolled on themselves and the center at the top of the wall moved forward to beyond vertical, which is typical of a failure due to a high bearing pressure at the rear of the wall.

The construction of the lower walls threatened the stability of the upper CRB wall. As the foundation of the existing CRB wall was below the natural angle of repose from the heel of the boundary wall, the boundary wall should have been designed to be able to resist the surcharge from the upper wall.

During a high intensity storm, water and soil build up behind the boundary wall exerted additional loading on the boundary wall itself and saturated the water in front of the upper CRB wall. Further softening of the soil and subsidence of the facing units mobilized the upper CRB wall, pushing the boundary wall and the upper portion of the lower CRB wall, culminating in a final collapse with the mode of failure being that of a slip.

Furthermore, there was no indication of drainage for both CRB walls, except for a layer of geofabrics behind the facing units.

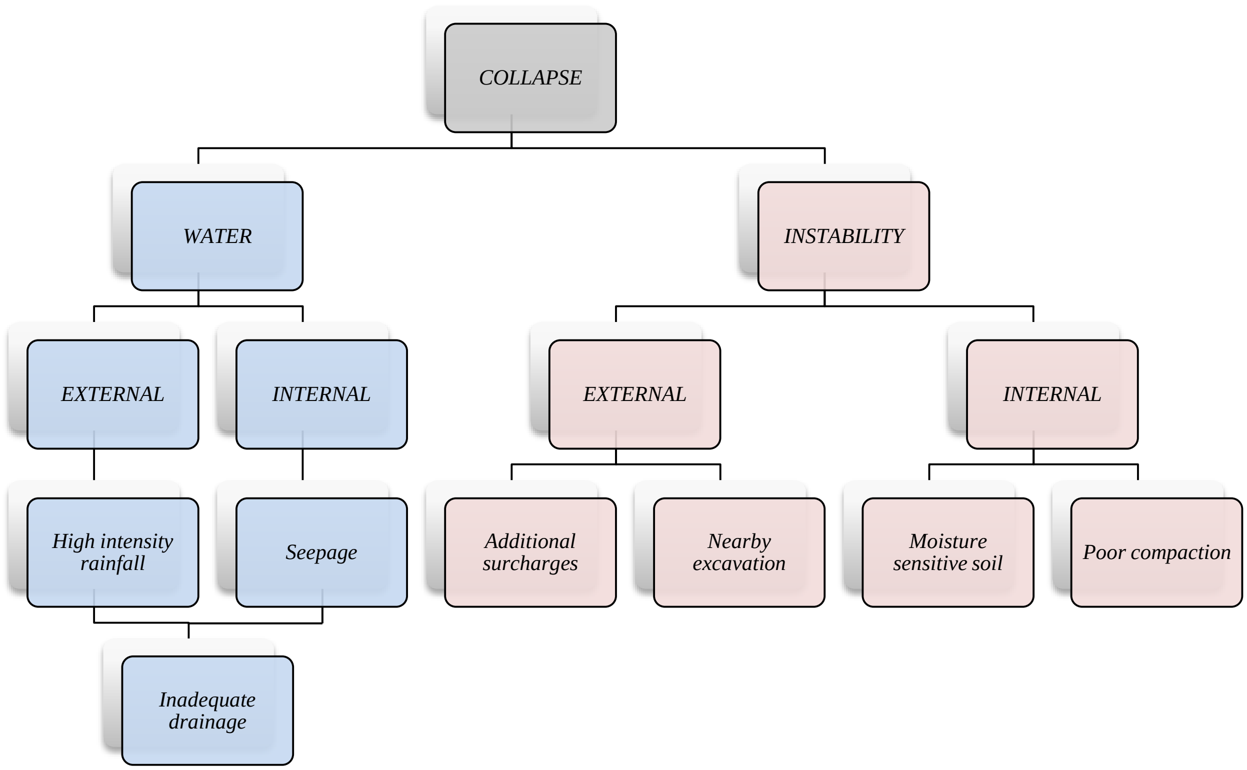
Design issues encountered by others:

No structural calculations were prepared for the lower CRB wall, no construction drawings were issued for the lower retaining wall and foundation of the brick boundary wall and no proper assessment was made as to whether or not the presence of the to-be-constructed lower walls would detrimentally affect the structural integrity of the existing CRB wall. No assessment was made as to whether or not the existing CRB wall would impose any surcharge loading upon the to-be-built lower walls.

The construction of a 60^o sloping embankment will result in a concentrated runoff, therefore, an adequate drainage system should have been incorporated in the design. Moreover, the upper CRB wall should have incorporated cut off drains at the top and bottom of the wall, especially in view of the soft, loose soil and high soil pressure at the base of the wall. Furthermore, the designer failed to undertake adequate construction monitoring.

<u>Failure flow chart:</u>

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 5

Wall Properties:			
TYPE:	*Gravity*	**OWNERSHIP:**	*Residential*
LOCATION:	*Kwa-Zulu Natal*	**YEAR OF FAILURE:**	*2004*
WALL CONFIGURATION:	*Tiered*	**FAILURE DESCRIPTION:**	*Collapse*

Design Parameters:			
WALL INCLINATION:	*65°*	**TOP SLOPE:**	*0°*
BACKFILL:	*Berea Red*	**APPROX. HEIGHT:**	*3m, 4m, 5m*

Overview:

A series of tiered gravity retaining walls located in a new residential development of over 40 units were to be constructed on an extremely difficult site, excessively sloping in two directions. A complicated arrangement of walls and terraces were constructed in order to accommodate the development. Substantial earthworks were included in the design of the complex and the units were occupied as they were completed.

The two adjacent units nearest to the entrance of the residential development failed. Upslope of the failed walls were several substantial terraced retaining walls and two other units.

Description of the failure:

Since the start of construction, the retaining walls were constantly damaged as a result of storm water, even from light rains. Due to the topography of the site, a natural surface drain ran down the extreme passageway between the two retaining walls.

While under construction, a major flash flood caused a torrent of uncontrolled storm water to pour off unfinished roofs and hardened areas with none of the designed storm water systems in place. This culminated in a considerable waterfall raging down between the two CRB walls which ultimately led to the complete collapse of the walls.

Details of the problem:

Due to time constraints, the project commenced on an on-site-daily-adjustment basis instead of from detailed drawings. This lead to numerous variations and on site platform level adjustments on an ongoing basis with resultant changes in shape, position, length and height of the CRB walls.

Walls varied in height up to 6m, while in certain locations, up to four wall terraces of a combined height of 12m had been constructed. The walls were designed for 3m, 4m and 5m maximum heights. Various wall batters had been constructed varying from 58^o to 68^o, while the walls were only designed for an inclination of 65^o to the horizontal.

The contractor did not construct the wall according to the design and the design was not a true representation of the conditions on site. In addition, critical system components such as the drainage systems were not yet constructed at the time of failure.

<u>Design issues encountered by others:</u>

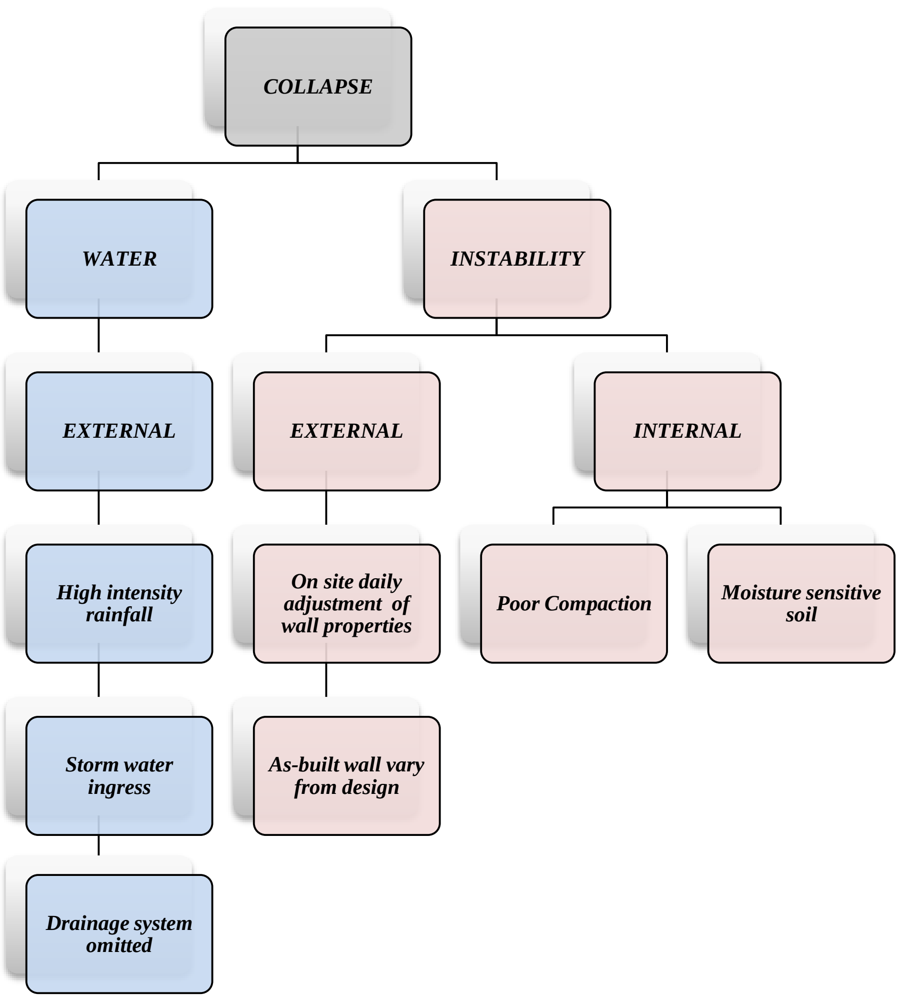
The engineer built up a design criterion for the walls from three typical CRB wall designs of 3m, 4m and 5m highs. The designs allowed for a wall inclination of 65^o to the horizontal. The design calculations and specifications were obtained from graphical outputs of a design software program. The actual loading conditions and site geometry were much more complex than presented in the design and drawings. Stability checks were required for the actual site geometries to assess the effects of the terracing of the walls.

The FOS against overturning and sliding was less than 1.5 and became progressively worse for an increased wall height. No surcharge loadings in the form of a UDLs or line loads were considered and the resultant force fell outside the middle third of the base.

The design reviews indicated that the design of the trial walls were more than adequate if they were constructed as individual walls, but the walls were under designed for terraced walls. Where the terraced walls met, the maximum design heights were exceeded. Overall stability does not appear to have been taken into account in the design of the walls.

Failure flow chart:

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall. Certain failure mechanisms are included in the flow chart, but are not necessarily basic failure mechanisms of the wall. That is; it contributed to the failure, but was not a reason for the failure. For example; consider a moisture sensitive soil. Due to the saturation of the soil the wall failed, but the failure occurred as the drainage system, which allows for the use of this soil, was omitted and not due to the inadequacy of the soil itself.



If the drainage system was not adequately designed, internal instability would be included as one of the basic failure mechanisms as poor soil conditions existed. However, if the drainage was omitted from the as-built wall system and an adequate drainage system was included in the design, then internal instability was not necessarily a reason for the occurrence of the failure.

CASE STUDY 6

Wall Properties:			
TYPE:	*Gravity*	**OWNERSHIP:**	*Residential*
LOCATION:	*Kwa-Zulu Natal*	**YEAR OF FAILURE:**	*2011*
WALL CONFIGURATION:	*Uniform Soil*	**FAILURE DESCRIPTION:**	*Collapse*

Design Parameters:			
WALL INCLINATION:	*90°*	**TOP SLOPE:**	*0°-2°*
BACKFILL:	*Berea Red*	**APPROX. HEIGHT:**	*3.7m*

Overview:

A gravity retaining wall supporting a residential property in a suburban area was raised when a pool was installed. Piles were incorporated to support the pool. Prior to the installation of the pool, the ground sloped up from behind the retaining wall towards the paving adjacent to the house.

Description of the failure:

A section of the retaining wall beyond the pool collapsed. The wall failed during a period of heavy rain at its highest point, approximately 4m in length. The collapse had resulted in debris and material behind sloughing down the steep bank. The area between the pool and wall was paved where surface water would flow off and over the CRB wall. Furthermore, in the event of the pool overflowing, the water would be directed towards and over the top of the wall.

Details of the problem:

The original retaining wall had been founded on sloping ground. The installation of the pool disrupted the CRB wall system. This CRB wall was raised up to 1000mm at certain sections to accommodate the pool behind the wall. The additional height exposed the wall system to excessive surcharge loads. Investigations into the failure of the wall concluded that the height of the retaining wall was too excessive for that particular system. In addition to the increased height of the wall, the problem stemmed from storm water damage and penetration of excessive water into the fill area.

The roof had no gutters at the time of the failure, therefore the storm water discharged directly onto the paving towards the wall. No drainage was designed or installed and hence, there was no drainage at the top of the wall to control the storm water. An adequate drainage system was crucial as the Berea Red soil has a tendency to dissolve and be dispersive at the same time, and the soil changes completely in behavior when it is saturated to when it's dry. Additional hydrostatic pressure exacerbated the problem *as the wall failed during a period of heavy rain. Furthermore, the wall had to accommodate an additional uphill phreatic profile as a wetland existed at the toe of the wall.*

The design review calculations indicate that the design and construction was inadequate and that failure was inevitable. Furthermore, the facing units were not in line with standard SABS approved specifications.

Design issues encountered by others:

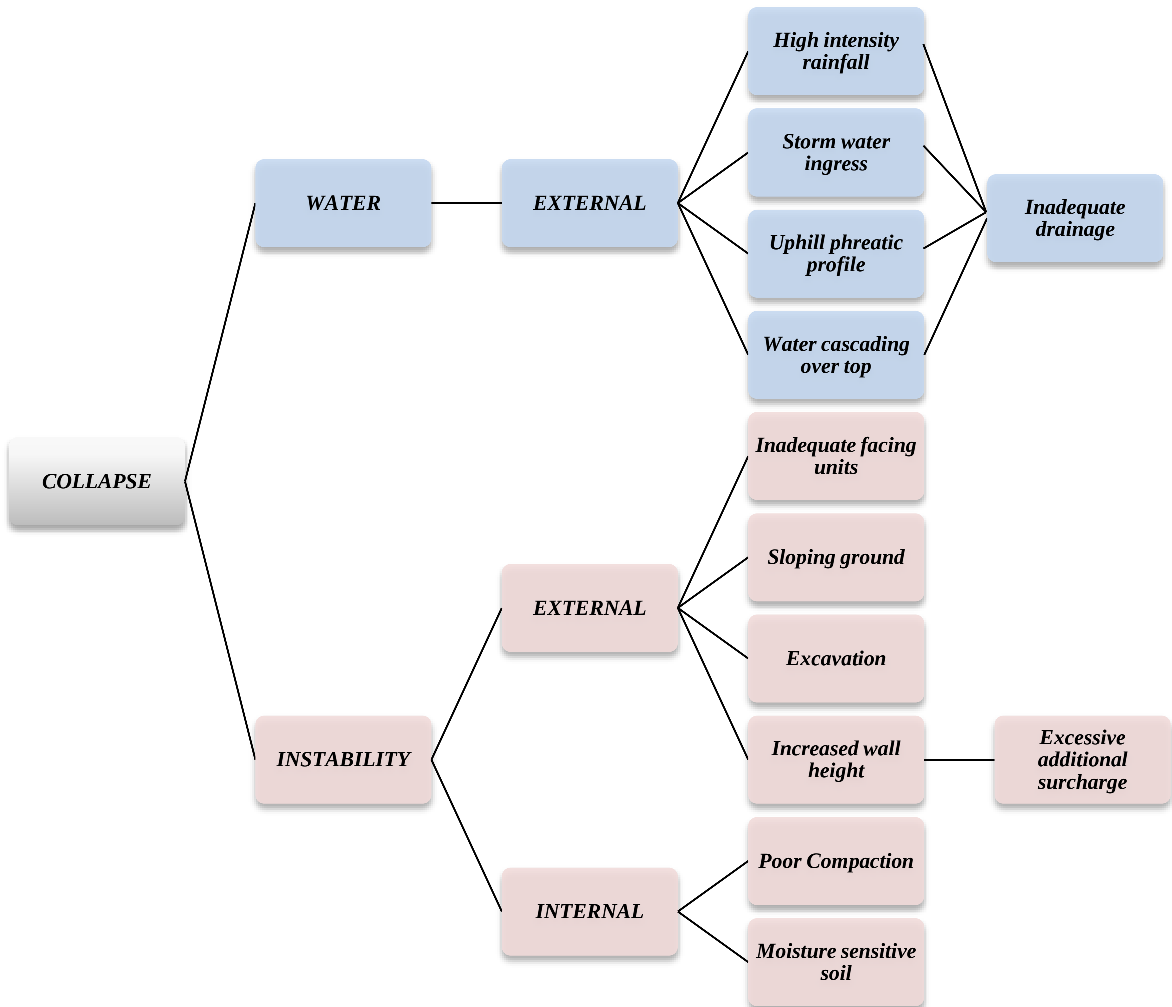
No drawings or design of the wall was available. The failure was brought about by a combination of problems which did not enjoy the attention to detail that was required at the design and construction stages of the initial CRB wall.

No engineering was carried out behind the design of retaining wall. It was a text book failure, as the failure occurred approximately at a third of the height of the wall, where the pressure on the wall was the greatest.

Non-standard facing units were used and the wall was built significantly past its design height. There had been no engineering carried out behind the retaining wall in the form of either cement stabilization or mechanical stabilization. No drainage was incorporated into the CRB wall system which was critical due to the collapsible potential of the Berea Red soil in the area.

<u>Failure flow chart:</u>

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 7

Wall Properties:			
TYPE:	*Gravity*	**OWNERSHIP:**	*Residential*
LOCATION:	*Kwa-Zulu Natal*	**YEAR OF FAILURE:**	*2007*
WALL CONFIGURATION:	*Uniform Soil*	**FAILURE DESCRIPTION:**	*Collapse*

Design Parameters:			
WALL INCLINATION:	*56°*	**TOP SLOPE:**	*0°-2°*
BACKFILL:	*Berea Red*	**APPROX. HEIGHT:**	*4.5m*

Overview:

A dwarf gravity retaining wall was initially constructed on a residential property in a suburban area, parallel to the top edge of an embankment, to prevent further erosion of the embankment. The filled embankment was constructed during the period mid 1993 to October 1995. The dwarf wall was constructed in late 2000's and the wall was raised in mid 2005. At the time the wall was raised, the embankment behind the wall was filled to extend the existing grass terrace behind the wall.

Description of the failure:

Investigations into the failure of the wall noticed that the wall was founded in the filled embankment and that the actual extent of the embankment was unknown. After heavy rain in 2007, a global failure occurred in which the upper portion of the embankment sank together with the facing units of the CRB wall. The slip failure formed behind and underneath the CRB wall, causing the wall to bulge and move horizontally and down vertically.

Details of the problem:

The slip was instigated by a shear failure in the lower section of the wall. This global failure occurred as a result of the following:

- *Excessive wall height;*
- *Lack of proper benching as the substantial water ingress weakened the interface connection between the original embankment and imported fill material behind the wall;*
- *The penetration of excessive water directly behind the wall, into the fill material, placed additional hydrostatic pressure on the wall and lowered the shear strength of the soil;*
- *There were zones of weaker material in the fill embankment;*
- *No drainage system was installed or designed;*

- *Foundation sliding due to the slip plane which formed behind and beneath the wall; and*
- *The lack of adequate shear keys.*

The partial collapse was primarily caused by inadequate design, inadequate pertinent specifications and a lack of appropriate drainage provisions.

Design issues encountered by others:

No calculations existed to determine the extent of shear keys required. According to the design chart which was used shear keys were required throughout the wall, but the keys were omitted in the final design and construction drawings.

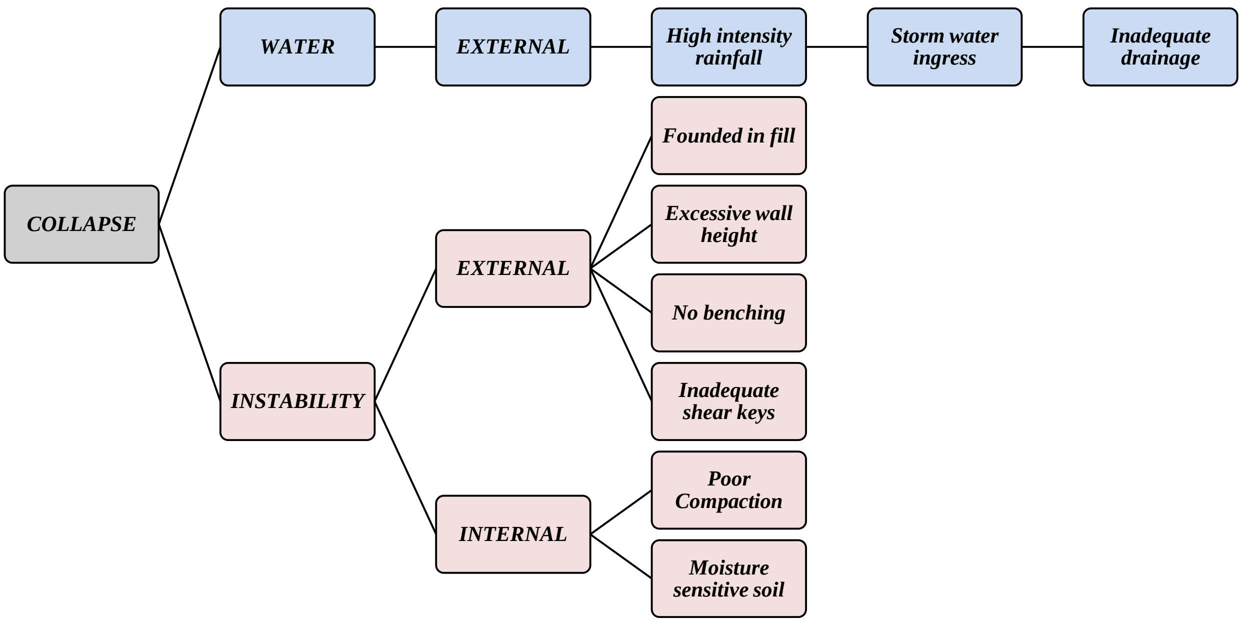
The block-on-block sliding capacity was calculated incorrectly, hence the factor of safety was below 1.5. Furthermore, a typical foundation was utilized and no calculations exited to determine the structural adequacy of the foundation.

The engineer did not accurately determine the design height of the wall from the top of the foundation to the top of the to-be-retained soil. A design should determine the actual row of blocks to be laid in order to accurately determine the to-be-constructed wall height and height of soil to be retained.

Furthermore, as poor soil compaction conditions existed and Berea Red soil was used as backfill material, adequate drainage should have been incorporated to prevent the accumulation of water behind the wall. Especially as Berea Red soil is infamous for its large variation in soil properties and tendency to become a collapsible soil.

Failure flow chart:

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 8

Wall Properties:			
TYPE:	*Gravity*	**OWNERSHIP:**	*Residential*
LOCATION:	*Kwa-Zulu Natal*	**YEAR OF FAILURE:**	*2007*
WALL CONFIGURATION:	*Uniform Soil*	**FAILURE DESCRIPTION:**	*Deformation*

Design Parameters:			
WALL INCLINATION:	*65°*	**TOP SLOPE:**	*0°-2°*
BACKFILL:	*Berea Red*	**APPROX. HEIGHT:**	*7m*

Overview:

A gravity retaining wall on a residential property in a suburban area was constructed to retain sloped fill on the property boundary.

Description of the failure:

A month after the completion of construction, the wall exhibited signs of visual distress. The facing units started cracking due to wall movement, which further exposed the foundation.

Details of the problem:

The wall failed due to inadequate design and poor workmanship during construction.

Design issues encountered by others:

The original design incorporated the Reynolds Reinforced Concrete designer's handbook using the Rankine formula. After the failure, the reviewed design incorporated the Concrete Manufacturers Association's recommended method, the Muller-Breslau method.

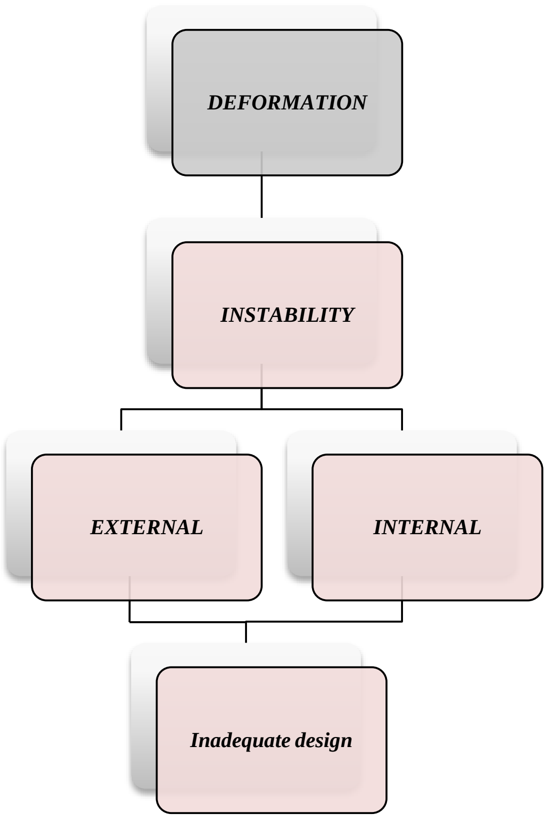
It was found that the original design calculated safety factors exceeding 1.5, compared to the reviewed design where the factors of safety did not exceed 1.5. It was established that the original design incorrectly calculated the active pressure coefficient, which resulted in a lower active pressure, approximately a quarter of what it should have been.

Furthermore, the vertical component of earth pressure was ignored, which for the parameters used the original design, was a destabilizing force. The wall friction was not incorporated in the original design. If the wall sloped towards the soil and there was no wall friction, an upward component of the earth pressure would have existed on the back of the wall. Hence, the assumption of zero friction is unrealistic.

In conclusion, the original design was erroneous and the arithmetic contained significant errors, hence the design was inadequate and flawed.

Failure flow chart:

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 9

Wall Properties:			
TYPE:	*Gravity*	**OWNERSHIP:**	*Service Station*
LOCATION:	*Gauteng*	**YEAR OF FAILURE:**	*2001*
WALL CONFIGURATION:	*Uniform Soil*	**FAILURE DESCRIPTION:**	*Collapse*

Design Parameters:			
WALL INCLINATION:	*70^o*	**TOP SLOPE:**	*0^o-2^o*
BACKFILL:	*Residual Granite*	**APPROX. HEIGHT:**	*3.4m*

Overview:

A gravity retaining wall was constructed at a service station to retain an embankment. The design and erection of the wall was intended to be a standard cut to fill on well compacted soil, using existing material on site as backfill material. Provisions were to be made for flow of storm water above the wall.

Description of the failure:

The entire wall collapsed due to external and internal water which completely waterlogged the soil behind the wall.

Details of the problem:

The wall as-built was completely unstable. The maximum inclined height of the wall as measured on site was 3.85m, which corresponds to a vertical height of 3.75m. This height exceeded the 3.4m height assumed in the design. Furthermore, the 10^0 top slope of the backfill behind the wall was not accounted for in the design. The inclination of the wall to the horizontal was designed as 70^o, while the inclination of the wall measured on site was between 77^o and 79^o. Moreover, a zero surcharge was assumed in the design, while buildings were in close proximity to the wall. The soil behind the wall was completely waterlogged for the full height of the wall. The waterlogged soil extended to quite a distance (approximately 3m) behind the wall.

The failure occurred as a result of the following:

- *Rupturing of the underground main pipes;*
- *A submerged sprinkler system which extended almost the full length of the wall;*
- *A number of leaking fittings behind the wall; as well as*
- *Leaking valves which were not completely sealed, directly behind the wall.*

No fabric reinforcement was provided, therefore, the wall had to rely solely on the shear strength of the backfill and the restraining moment generated by the weight of the facing units against the inclined face for its stability. Investigations onto the failure of the wall concluded that the design of the wall was inadequate.

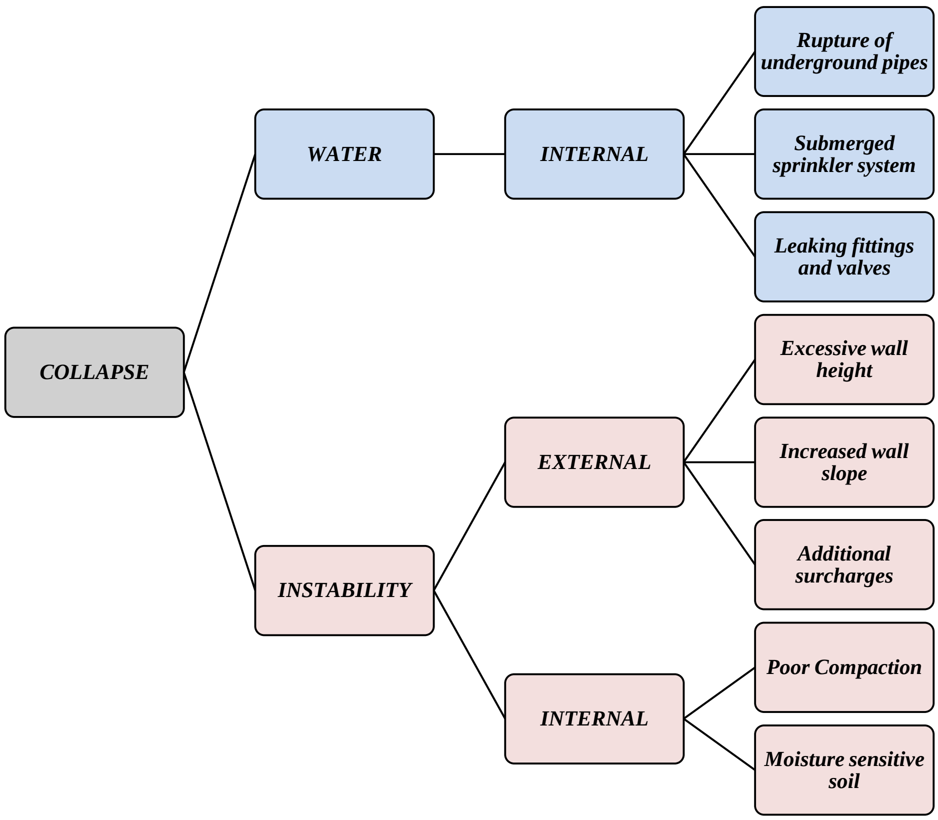
Design issues encountered by others:

The design did not represent the true conditions on site. Even with a double skin for the full height of the wall, it would still have been unstable. Fabric reinforcement should have been incorporated in the design. The wall was not designed for the hydrostatic pressures or surcharges it was exposed to and the design parameters were incorrect. The wall was built steeper than what it was designed for and a top slope was not accounted for in the design.

Furthermore, an adequate drainage system should have prevented the accumulation of water behind the wall, especially as residual granite is infamous for its large variation in soil properties and its collapsible grain structure.

Failure flow chart:

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall. Certain failure mechanisms are included in the flow chart, but are not necessarily basic failure mechanisms of the wall. That is; it contributed to the failure, but was not a reason for the failure. For example; consider a moisture sensitive soil. Due to the saturation of the soil the wall failed, but the failure occurred as the drainage system, which allows for the use of this soil, was omitted and not due to the inadequacy of the soil itself.



If the drainage system was not adequately designed, internal instability would be included as one of the basic failure mechanisms as poor soil conditions existed. However, if the drainage was omitted from the as-built wall system and an adequate drainage system was included in the design, then internal instability was not necessarily a reason for the occurrence of the failure.

CASE STUDY 10

Wall Properties:			
TYPE:	*Gravity*	**OWNERSHIP:**	*Residential*
LOCATION:	*Gauteng*	**YEAR OF FAILURE:**	*2008*
WALL CONFIGURATION:	*Stable Rock*	**FAILURE DESCRIPTION:**	*Collapse*

Design Parameters:			
WALL INCLINATION:	*62°*	**TOP SLOPE:**	*3°*
BACKFILL:	*Stable rock*	**APPROX. HEIGHT:**	*4m*

Overview:

A CRB wall was constructed on a moderately sloping residential property in a suburban area to protect a cut face from erosion/weathering. The cut face was 90m long and 2m to 4m in height. The cut was formed to create a level platform for development. The wall incorporated a horizontal subsurface drain behind the lowest facing units connected to vertical drains. Furthermore, a half round drain was incorporated along the top of the wall to intercept storm water runoff behind the wall. The contractor used non-standard facing units in the construction of the CRB wall. Subsequently after completion of construction, a new house was erected behind the wall.

Description of the failure:

The failure occurred after an intense rainstorm, four to five years after completion of construction. The portion that failed was 21m long where the wall was approximately 4m high. Some cracks formed in the facing units extending about 2m to 3m beyond the collapsed section of the wall.

The wall slowly, but steadily pushed out at the bottom causing the blocks at a higher level to follow. The founding blocks at the base of the wall had remained securely in place, while the blocks above had slid off at the first joint above. Many of the blocks cracked in half. The lowermost courses of the blocks had been filled with stabilized soil or weak mix concrete and showed no signs of distress. After the failure, the exposed face of the original bank showed no signs of distortion from the angle at which it was cut.

Details of the problem:

The agricultural half round drain which was part of the design had been omitted over the failed section of the wall, and pressure build up behind the wall resulted in the failure. Furthermore, the facing units were inadequate. No shear connectors existed between the lower courses and the inclination of the wall *to the horizontal was 58^o, which was much lower than the 62^o inclination angle for which the wall was design. As the facing units did not interlock, block-on-block sliding occurred.*

The contractor did not construct the wall according to the design. This could have been prevented if adequate construction monitoring was performed. Evidently failure would have occurred due to the inadequate strength of the facing units. The units were 410mm x 410mm x 205mm octagonal units with a 40mm wall thickness. After the failure, the blocks were tested in accordance with the CMA's code of practice for gravity walls.

It was found that the coefficient of friction corresponded to an angle of friction of 38^o which is acceptable. The two types of crushing strength tests, namely the back line load and front line load test, failed at 4.12kN and 1.71kN respectively. These values are considerably below the strength for commercial blocks which typically range between 40kN and 80kN per block and therefore, the facing units incorporated in the wall system were inadequate.

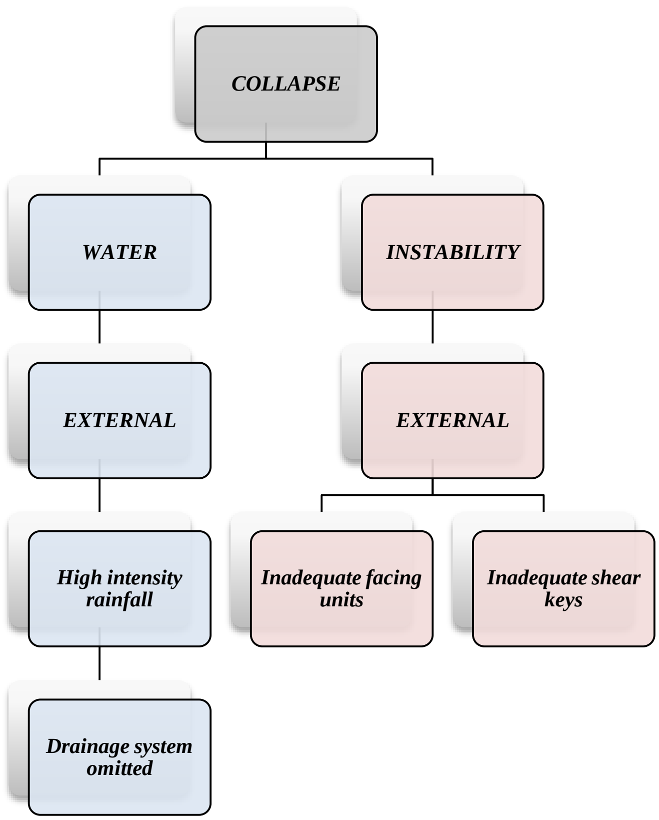
Design issues encountered by others:

Since the soil mass was capable of supporting itself, no earth pressures would have been exerted against the retaining wall. The CRB wall acted as a slender compression member; therefore, the height had a considerate effect on the stability of the wall. If SABS approved facing units were used, the failure could have been prevented.

Furthermore, an adequate drainage system should have been incorporated to prevent the buildup of water behind the wall, which exposed the wall to additional hydrostatic pressures for which it was not designed.

Failure flow chart:

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 11

Wall Properties:			
TYPE:	*Soil Reinforced*	**OWNERSHIP:**	*Residential*
LOCATION:	*Gauteng*	**YEAR OF FAILURE:**	*2011*
WALL CONFIGURATION:	*Uniform Soil*	**FAILURE DESCRIPTION:**	*Collapse*

Design Parameters:			
WALL INCLINATION:	*85º*	**TOP SLOPE:**	*0º-2º*
BACKFILL:	*Residual Granite*	**APPROX. HEIGHT:**	*1.8m*

Overview:

A gravity retaining wall was constructed on a residential property in a suburban area along the property boundary situated on the banks of a stream. The wall was constructed to retain a filled embankment. It was located at a point where the surface sloped more steeply towards the stream.

Description of the failure:

A section of the wall collapsed before completion of construction. An intense rainstorm raised the water level of the stream and flooded a portion of the wall.

Details of the problem:

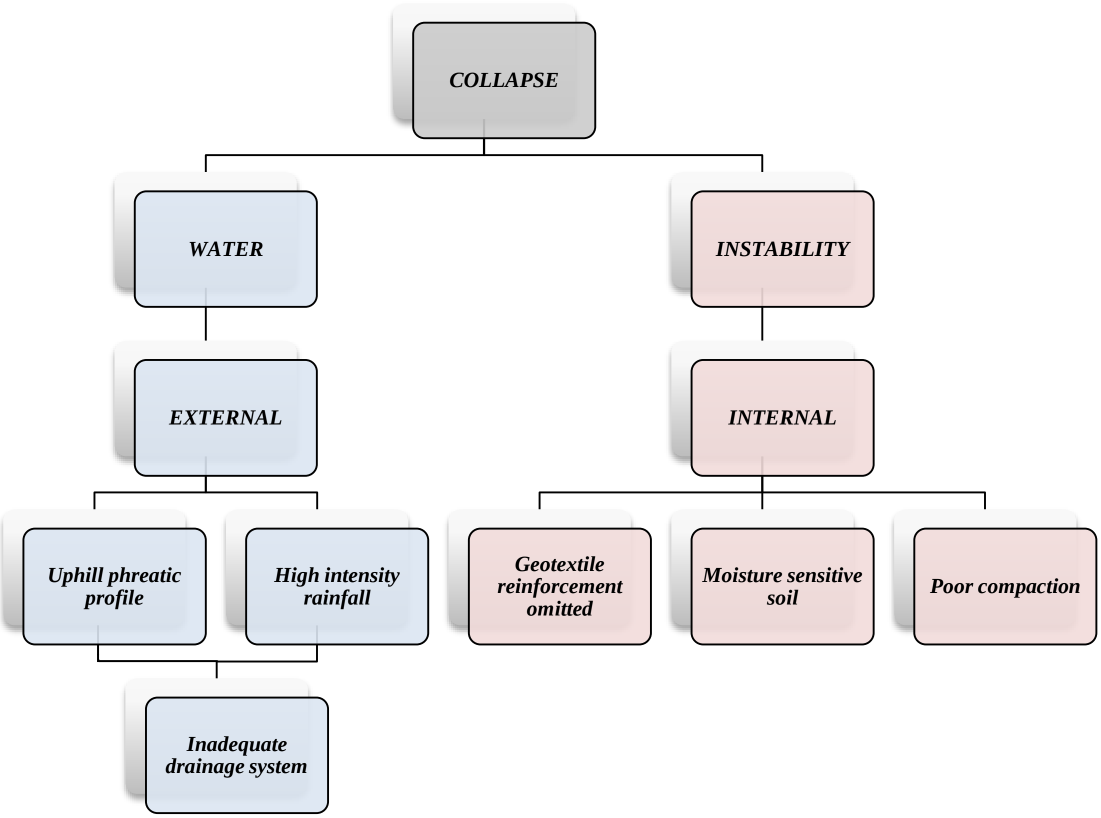
The stream flows all year round and drains a considerable catchment area, covered almost entirely by suburban housing and infrastructure. The CRB wall encroached on the flood line of this stream and constituted a restriction to the flow without the appropriate drainage measures in place. Furthermore, the contractor omitted the geotextile reinforcement on the side of the wall which failed. The fabric reinforcement that was omitted led to the collapse.

Design issues encountered by others:

Even though the failure occurred as a result of the wall not being constructed in accordance with the design, failure was inevitable. The design engineer was not familiar with the requirements for river/stream management before designing the wall. The drainage system was inadequate; the design did not consider certain critical failure modes and the calculations contained fundamental errors, indicating a lack of understanding of soil mechanics.

Failure flow chart:

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 12

Wall Properties:			
TYPE:	*Soil Reinforced*	**OWNERSHIP:**	*Residential*
LOCATION:	*Gauteng*	**YEAR OF FAILURE:**	*2003*
WALL CONFIGURATION:	*Tiered*	**FAILURE DESCRIPTION:**	*Deformation*

Design Parameters:			
WALL INCLINATION:	*76º*	**TOP SLOPE:**	*0º-2º*
BACKFILL:	*Timeball Hill*	**APPROX. HEIGHT:**	*3.3m*

Overview:

A soil reinforced retaining wall system was constructed on a property in a residential to retain a slope on the property boundary and support the house above.

The retaining wall system consisted of two sections namely a lower section at the east and north-east boundaries of the property forming a lower terrace, and a higher section on the north side of the residence supporting the upper terrace. Post construction of the boundary wall, a brick wall was constructed on top of the CRB wall without the knowledge of the Engineer.

Description of the failure:

Initially both walls were constructed without a foundation. Cracks formed in the facing units and partial collapse of the upper retaining wall occurred almost immediately after completion of construction. The walls were demolished and re-built, this time with a concrete foundation.

Cracks developed again in the upper and lower walls and the concrete columns of the residence shifted. The cracks were V-shape and vertical cracks were also observed in the facing units. Drainage pipes were then incorporated to divert the storm water runoff from the house away from the terrace supported by the wall.

Details of the problem:

The initial failure occurred as the walls were constructed on uncompacted soil with no concrete foundation. After the incorporation of the concrete foundation, some settlement of the foundation to the wall took place.

Due to the poor compaction, low density and low quality of the backfill material, stress concentrations caused cracks to form in the facing units and the top of the wall settled. This failure was likely caused by watering of the garden which softened the in-situ materials. The backfill softened with time due to the flaking of the shale under the action of water ingress.

Furthermore, the blocks were defective and the wall was constructed greater than its design height. The contractor did not construct the wall according to the design and construction drawings which could have been prevented if adequate construction monitoring was performed. Evidently failure would have occurred due to inadequate design and construction drawings, which lacked crucial information.

Design issues encountered by others:

Calculations showed that the wall was stable on a global scale, but localized stability was questionable. The design did not check the wall against all failure modes. The brick wall on top of the CRB wall was not considered in the design as the owner constructed it without the knowledge of the engineer.

As poor soil conditions existed, adequate drainage should have been incorporated to prevent the accumulation of water behind the wall.

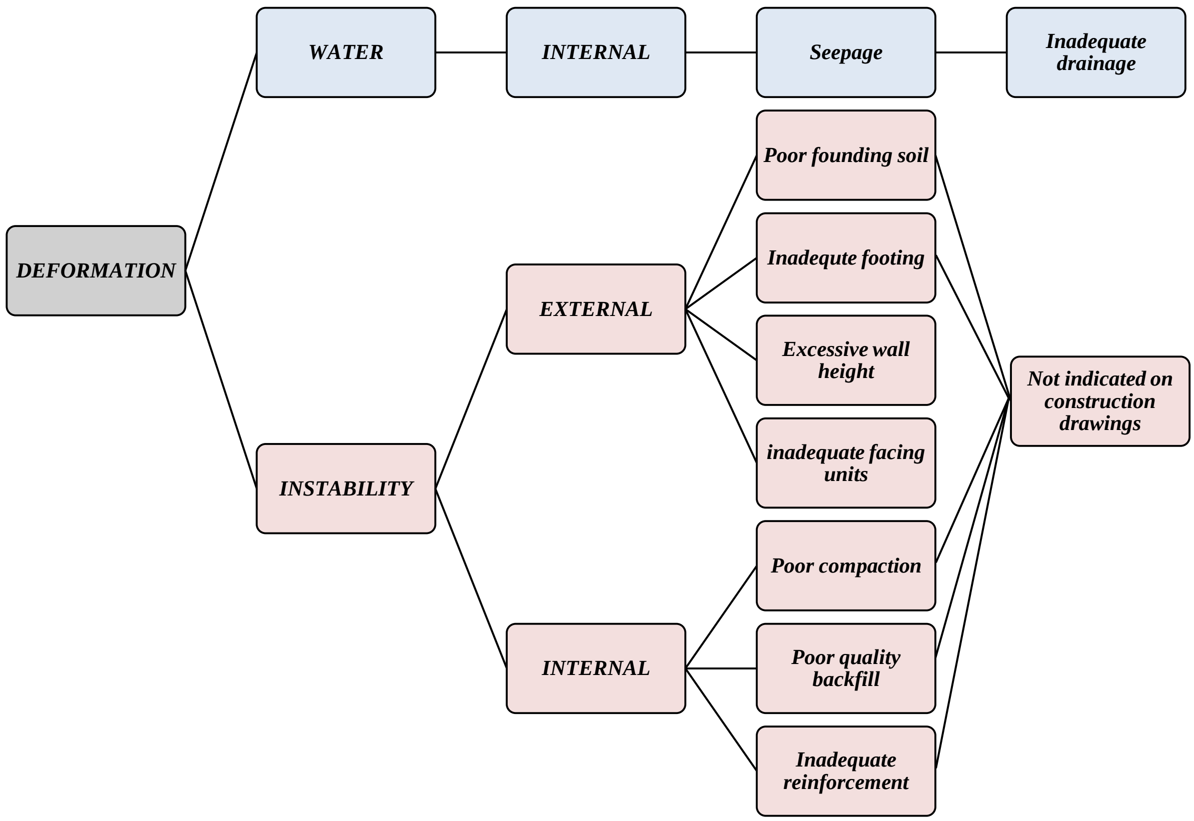
Furthermore, the following was not specified on the construction drawings:

- *Material properties of the backfill and compaction standards;*
- *Type and placement of the reinforcing;*
- *A concrete foundation; and*
- *The tensile capacity of the reinforcing.*

The design engineer should have ensured that all critical design components were incorporated through adequate construction monitoring.

Failure flow chart:

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 13

Wall Properties:			
TYPE:	*Soil Reinforced*	**OWNERSHIP:**	*Residential*
LOCATION:	*Gauteng*	**YEAR OF FAILURE:**	*1997*
WALL CONFIGURATION:	*Uniform Soil*	**FAILURE DESCRIPTION:**	*Collapse*

Design Parameters:			
WALL INCLINATION:	*80^o*	**TOP SLOPE:**	*0^o-2^o*
BACKFILL:	*Residual Granite*	**APPROX. HEIGHT:**	*5.5m*

Overview:

A gravity retaining wall was constructed on a property in a residential development to retain a slope on the property boundary and support the units above.

Description of the failure:

A portion of the wall facing units collapsed soon after completion of construction. The line of failure was observed 0.5m to 1m back from the crest of the wall. No settlement of the garden behind the line of failure was observed.

Details of the problem:

The wall system failed as a result of shortcomings in its construction. These shortcomings relate to the following:

- *Anchorage*

  A manhole was situated in the reinforced soil zone. Concertainer baskets were incorporated in the design to anchor the reinforcement restricted by the manhole. At the edge of the failure zone no continuity existed between the fabric placed between the blocks and the fabric sandwiched between the concertainer baskets. The two pieces overlapped with no bond between them and, therefore, no anchorage of the blocks was provided. The inclination of the wall and insufficient anchorage of blocks caused the blocks to fall away from the concertainer baskets and ultimately led to the failure of the wall.

  Furthermore, the top layer of the filter fabric stepped at the edge of the manhole and the second layer of fabric stopped 800mm short of the manhole. No continuing reinforcement was installed past the manhole as the concertainer baskets required to anchor the reinforcement in front of the manhole *were omitted during construction.*

- *Wall Inclination*

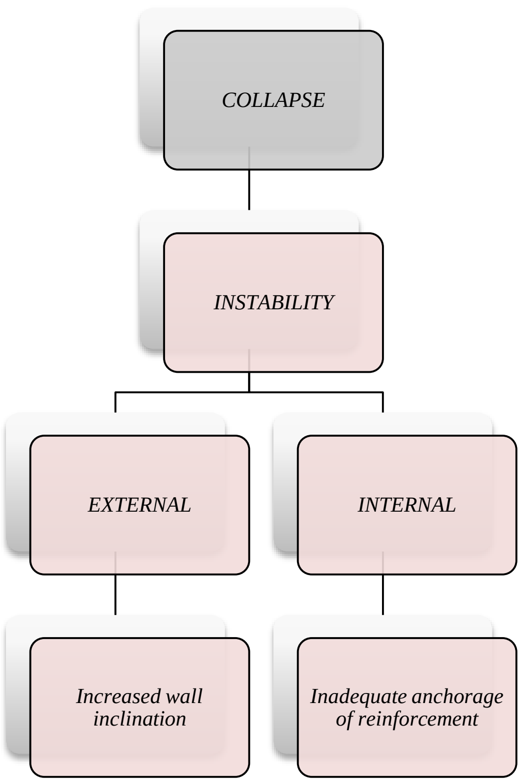
The wall was constructed at 86º though designed for an angle of 80º.

Design issues encountered by others:

The design was adequate. If adequate construction monitoring was performed, shortcomings in the construction could have been prevented.

Failure flow chart:

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 14

Wall Properties:			
TYPE:	*Soil Reinforced*	**OWNERSHIP:**	*Business Park*
LOCATION:	*Gauteng*	**YEAR OF FAILURE:**	*2007*
WALL CONFIGURATION:	*Uniform Soil*	**FAILURE DESCRIPTION:**	*Deformation*

Design Parameters:			
WALL INCLINATION:	*75°*	**TOP SLOPE:**	*0°-2°*
BACKFILL:	*Residual Granite*	**APPROX. HEIGHT:**	*5.8m*

Overview:

A soil reinforced retaining wall was constructed to retain the paved parking lot of an office park. A highway was situated in front of the wall and stretched along the entire length of the wall.

Description of the failure:

The soil-reinforced CRB wall collapsed due to the excessive saturation of the backfill in which the internal friction of the soil had reduced to the point where the design parameters no longer applied. The reduced angle of soil friction placed additional loads onto the retaining wall and reinforced fill which shifted and subsided.

The settlement of the underlying greenstone as well as the consolidation of the wet backfill caused the "block of reinforced soil" to move downwards, rotate forwards and create a crack at the contact surface between the in-situ embankment and the block of reinforced soil. The subsidence of the soil behind the CRB wall resulted in ponding of the rainwater which entered the cracks in the paving and further saturated the soil. This caused additional settlement of the wall and foundation. The water pressure build up behind the wall caused the wall to bulge, which resulted in cracking of the facing units and failure of certain sections of the wall.

Details of the problem:

The failure occurred as a result of the following:

- *Wall height:*

  The wall was constructed 7.1m high whereas the design allowed for a maximum height of 5.8m.

- *Nature of the backfill material:*

The wall was designed using a friction angle of 32° and the backfill material was required to comply *with these specifications. No tests were done to verify the properties of the soil and the investigations into the failure queried whether the wall had been constructed with unsuitable fill material for this particular application.*

The unsuitable material adversely affected the stability of the wall and effectiveness of the drainage measures.

- *Length of the reinforcement:*

The design indicated that the fabric should have extended 3m into the soil from the back of the facing units at the base of the wall, increasing to 4m near the top of the wall. It appeared as if the distance between the building and the cut face restricted the length of the reinforcement to less than specified.

Furthermore, the maximum design height of the wall was 6.5m from the bottom of the foundation to the crest of the wall, while the first remedial proposal indicated that the maximum height of the wall was 7.3m. For a wall of this height, the critical failure plane would tend to intercept the ground surface about 4m back from the crest of the wall, therefore, the reinforcing was slightly short.

An inadequate length of reinforcement will affect the movement of the retaining wall and can lead to overall instability.

- *Water ingress and poor drainage:*

The water originated from numerous sources and flowed directly to the area behind the wall. Saturation of the backfill material increased the loads on the retaining wall and decreased the shear strength of the fill.

The storm water originated from:

- *A donga behind boundary wall;*
- *The manhole which was left open;*
- *The flat fall of the paving;*
- *Cracks that formed in the paving behind the wall and were not repaired;*
- *Storm water which fell on the parking area discharged to an area behind the boundary wall;*
- *A trench behind boundary wall which neatly intercepted the overland storm water and allowed the water to stand, penetrating the fill beneath the car park and migrating to the fill behind the retaining wall; and*

- *The functioning of the drainage system was impaired by the poor quality of the backfill material.*

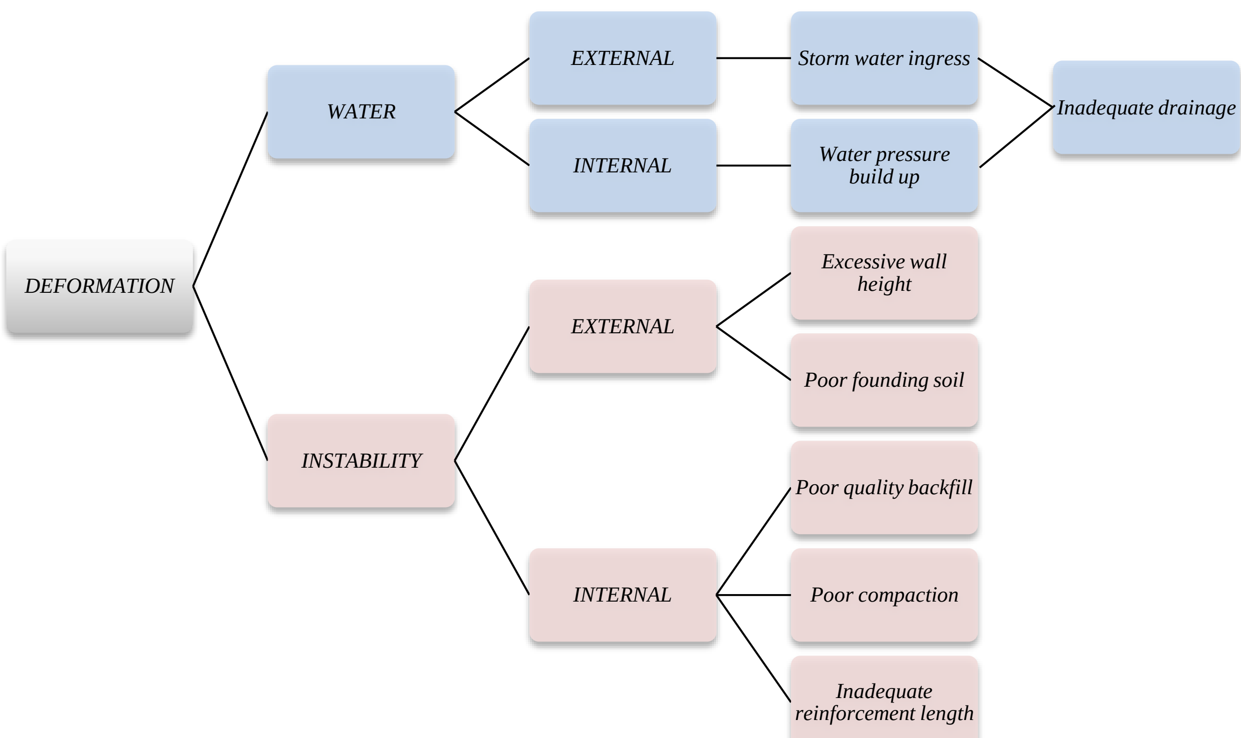
<u>Design issues encountered by others:</u>

The design of the wall did not incorporate the following criteria:

- *The residual greenstone on which the wall was founded is a highly compressible material and when saturated, it has a very low bearing capacity;*
- *The suitability of the material used as backfill behind the wall should have been at least G7 quality material;*
- *Proper drainage of the reinforced zone, both at interface with the in-situ embankment and directly behind the wall;*
- *Geology, adjacent in-situ material as well as the topography of site; and*
- *Adequate construction monitoring.*

<u>Failure flow chart:</u>

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 15

Wall Properties:			
TYPE:	*Soil Reinforced*	**OWNERSHIP:**	*Recreational Park*
LOCATION:	*Gauteng*	**YEAR OF FAILURE:**	*1994*
WALL CONFIGURATION:	*Tiered*	**FAILURE DESCRIPTION:**	*Collapse*

Design Parameters:			
WALL INCLINATION:	*85°*	**TOP SLOPE:**	*0°-2°*
BACKFILL:	*Residual Granite*	**APPROX. HEIGHT:**	*15m*

Overview:

A tiered soil reinforced retaining wall was constructed to retain the tail end of the starting grid of a race course track.

Description of the failure:

The upper portion of the wall failed. The problem started with the back rotation of the upper facing units accompanied by general bulging of the upper section of the wall. The remainder of the wall showed evidence of a general outward rotation with the blocks dipping slightly out of the face.

Details of the problem:

Three berms of approximately 800mm wide existed over most of the wall. On the crest of the CRB wall a reinforced concrete arrester wall of approximately 1.5m high with a 4m long toe extended towards the track. There was a down stand of approximately 600mm at the end of this toe below the underside of the foundation.

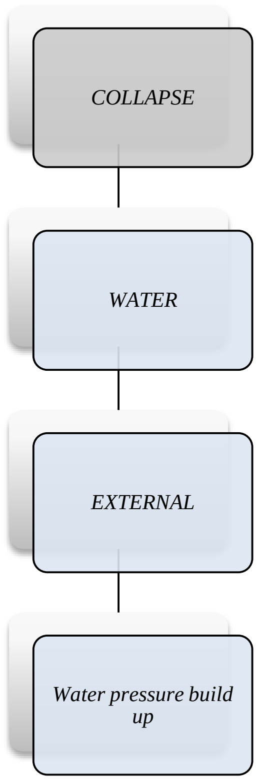
Recent rains saturated the pea gravel which covered the arrester bend. The saturation of the pea gravel caused the arrester wall to tilt outwards, resulting in bending of the vertical channel sections which supported the catch fence.

Design issues encountered by others:

The design was adequate, but a better drainage system should have been incorporated to prevent ponding of water in the pea gravel behind the wall.

Failure flow chart:

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 16

Wall Properties:			
TYPE:	*Soil Reinforced*	**OWNERSHIP:**	*Hospital*
LOCATION:	*Gauteng*	**YEAR OF FAILURE:**	*2003*
WALL CONFIGURATION:	*Uniform Soil*	**FAILURE DESCRIPTION:**	*Deformation*

Design Parameters:			
WALL INCLINATION:	*80º-90º*	**TOP SLOPE:**	*0º-2º*
BACKFILL:	*Mixture*	**APPROX. HEIGHT:**	*7.7m*

Overview:

A soil reinforced CRB wall was constructed to abut the parking lot of a hospital which was situated on an embankment. A highway was situated in front of the wall and stretched along the entire length of the wall. A pedestrian sidewalk was situated between the highway and the front of the wall.

A palisade fence was constructed immediately on top of the wall for most of its length. The parking lot above was paved with interlocking concrete blocks and subdivided into smaller parking areas by narrow traffic islands at right angles to the wall, bordered by barrier kerbs.

Description of the failure:

For a number of years the soil reinforced CRB wall showed signs of distress including cracking, settlement of the retained backfill and outward rotation of the wall. Most of the movement occurred as much as 9m behind the wall. The distortion was not new at the time of inspection which indicates that the movement took place over a long period of time. The significant settlement of the fill behind the wall severely impaired the drainage behind the wall.

Cracks occurred in the interlocking concrete block paving behind the wall over a distance of approximately 60m. The furthest crack was approximately 12m from the face of the wall. The position of the cracks had something to do with the length of the reinforced fabric or the position of the cut/fill line below the car park. Where the cracks crossed the traffic islands, the joints between the barrier kerbs opened up. Furthermore, the palisade fence above the wall leaned backwards and the face of the wall from the street level was no longer a "smooth surface".

Vertical cracks were observed in many of the facing units. In certain places the vertical cracks aligned *in several courses above one another forming a vertical discontinuity of the wall. Parts of the wall were stained red due to muddy water flowing over the wall and washing the soil out from the gaps in the facing.*

Details of the problem:

The length and spacing of the geosynthetic raised concern and the quality of the backfill material was relatively poor. The washout of backfill material through the facing occurred due to the poor quality backfill and it further indicated a water problem.

Design issues encountered by others:

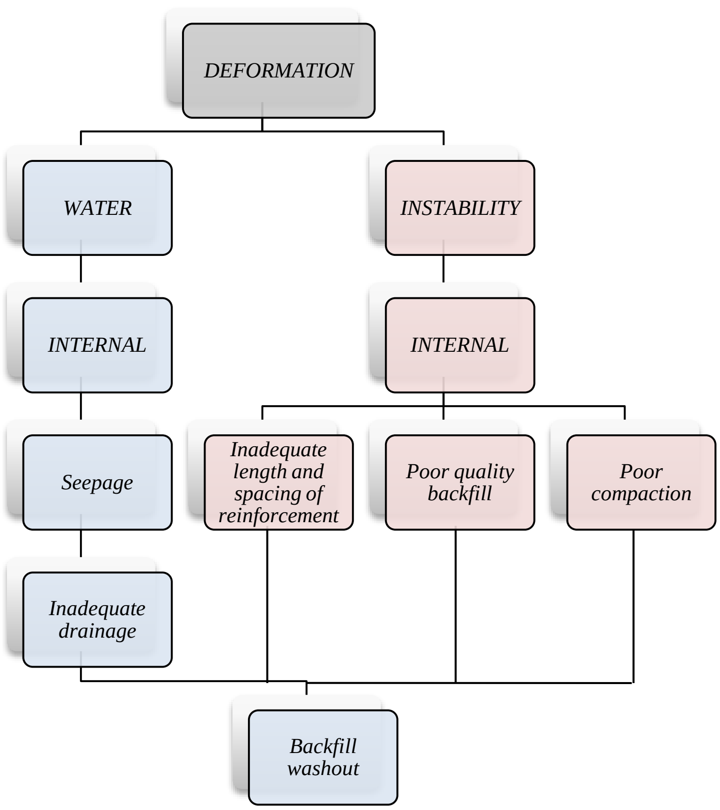
Unfortunately the design was not available for review, but investigations into the failure of the wall queried whether the design was inadequate due to the spacing and length of reinforcement.

As poor soil compaction and groundwater conditions existed, adequate drainage should have been incorporated to prevent the accumulation of water behind the wall, especially as the backfill material was of a poor quality.

The measures that were no properly incorporated to prevent the migration of soil particles through the gaps in the facing units largely contributed to the failure.

<u>Failure flow chart:</u>

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 17

Wall Properties:			
TYPE:	*Soil Reinforced*	**OWNERSHIP:**	*Business Park*
LOCATION:	*Eastern Cape*	**YEAR OF FAILURE:**	*2000*
WALL CONFIGURATION:	*Uniform Soil*	**FAILURE DESCRIPTION:**	*Deformation*

Design Parameters:			
WALL INCLINATION:	*87^{o}*	**TOP SLOPE:**	*0^{o}-2^{o}*
BACKFILL:	*Unknown*	**APPROX. HEIGHT:**	*4.5m*

Overview:

A soil reinforced CRB wall was constructed on the boundary of a vehicle dealership site to abut the expressway as part of the earthworks platforming of the site. The owner of the property was informed that the retaining wall was failing.

The wall was founded adjacent to one of the Council's sewer trenches. A 230mm face brick boundary wall of approximately 2m high was constructed on top of the CRB wall for most of its length.

Description of the failure:

Within three years after completion of construction, the retaining wall deformed to such an extent that the stability of the wall was compromised. The embankment exhibited an unacceptable degree of settlement and lateral movement causing the rotation and spalling of individual facing units in numerous places.

An earth berm of approximately 10m wide and 1.5m high pushed up against the existing CRB wall. The face brick boundary wall above appeared to have consolidated along a portion of the wall. Furthermore, the paving in the parking area behind the retaining wall pulled apart in various places, allowing ingress of water into the subgrade layers.

When the backfill settled to such an extent that the surrounding structures were damaged, an inspection was carried out to identify the cause of the failure. At the time of inspection, the retaining wall appeared to be at or close to the vertical along most of its length, and in some instances past the vertical.

<u>Details of the problem:</u>

The retaining wall failed due to the following:

- *The upper layers of reinforcement were marginally short;*
- *The reinforcement was over stressed which contributed to the ongoing creep movement of the wall;*
- *The base was too small;*
- *The wall was constructed at too steep of an angle for the type of design method utilized;*
- *Inadequate drainage;*
- *The backfill material was not free draining as assumed in the design;*
- *The backfill material did not bench into the existing competent material behind the backfill;*
- *Backfill washout occurred;*
- *Water entered the cracks in the paving and saturated the soil; furthermore,*
- *The wall did not constitute a "useable space fit for construction and free from the risk of subsidence".*

Portions of the fill were saturated at times. Literature explains that this is indicative of shortcomings in the design and construction of the fill. It particularly refers to deficiencies in the subsoil drainage and/or utilization of sub-standard material.

<u>Design issues encountered by others:</u>

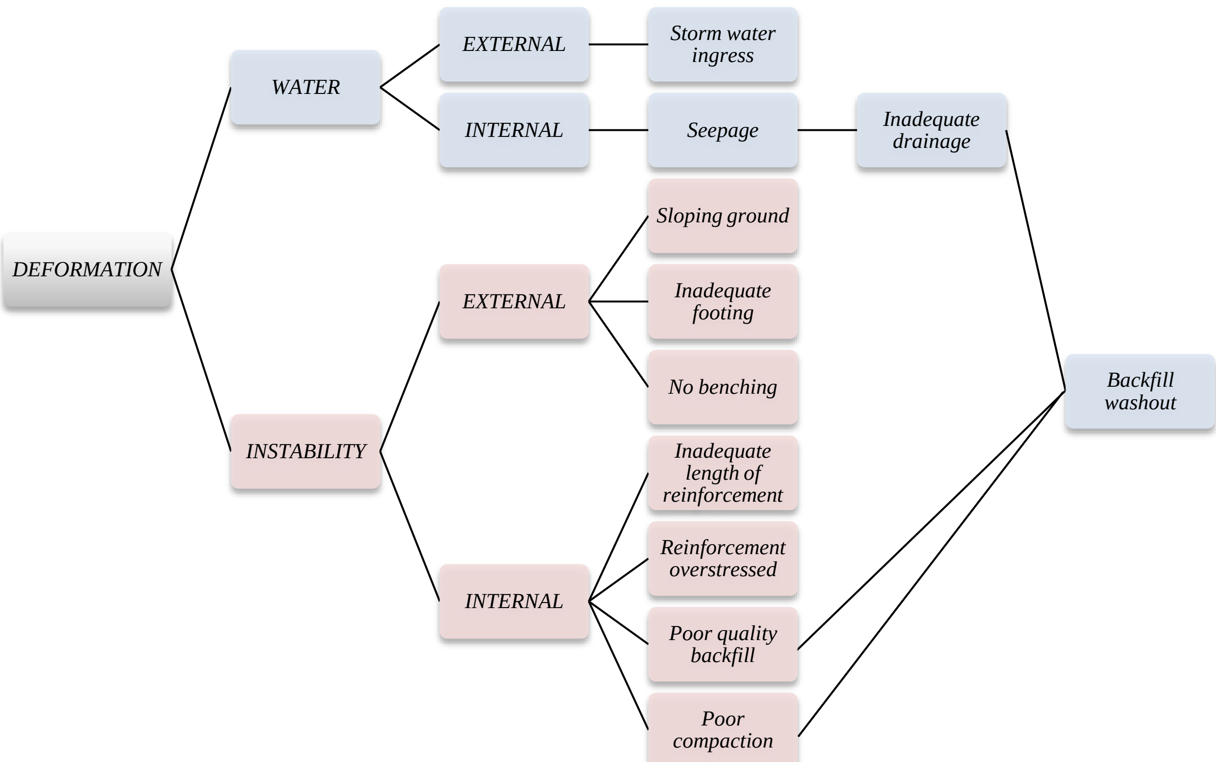
The failure of the wall was caused by inherent deficiencies in the fill/wall combination. According to the CMA design manuals:

- *The design should have provided for a drainage immediately behind the facing units;*
- *The bottom facing units should have been founded on a relatively large concrete footing, while the design depicted a relatively small mortar base;*
- *The base block should not have been installed at an angle larger than 15°, while the design required an instillation of 17°;*
- *The design should have incorporated a drain at the base/toe of the fill immediately behind the facing/wall; and*
- *The existing steep slopes should have been cut back in a stepped fashion whereas the design only required that the existing slope be roughened.*

Moreover the fact that the design of the embankment did not incorporate any measures to prevent the migration of soil particles through the gaps in the facing units largely contributed to the failure.

Failure flow chart:

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



CASE STUDY 18

Wall Properties:			
TYPE:	*Soil Reinforced*	**OWNERSHIP:**	*Shopping Center*
LOCATION:	*Gauteng*	**YEAR OF FAILURE:**	*2014*
WALL CONFIGURATION:	*Limiting bank*	**FAILURE DESCRIPTION:**	*Collapse*

Design Parameters:			
WALL INCLINATION:	*75°*	**TOP SLOPE:**	*27°*
BACKFILL:	*Residual Granite*	**APPROX. HEIGHT:**	*9.6m*

Overview:

A soil reinforced CRB wall was constructed to retain an entrance road to a shopping mall. A highway was situated in front of the wall. Furthermore, the wall served as an abutment, extending underneath the bridge leading to the mall's entrance road.

Description of the failure:

The failure occurred during construction. The wall failed at the length of the storm water pipe interface, in the zone where the tension crack in the overburden was found, for a length of approximately 30m. A slip occurred within the natural in-situ embankment. The rainfall the night before was 52mm. Water flow was not controlled due to work in progress; there had been washout in areas on either side of the bridge. The entire constructed wall was engulfed by a failure surface that passed through the residual granite behind the wall; hence an overall failure occurred.

Details of the problem:

The geotechnical soil profile of the site varied substantially and the lower 90% of the soil was quite loose and formed a weak spot. Furthermore, the backfill material did not bench into the existing competent material.

Nevertheless, the original failure was triggered by water ingress from a partially completed storm water drainage system. Water was pumped out of the storm water pipes as they had been blocked by siltation and debris. The storm water pipes may have been blocked for long periods of time and saturation of the material below the pipes resulted in some initial minor settlement and most likely slight opening of some pipe joints.

Trench settlement below the pipes developed the tension cracks in the paving, which further exposed the *material to water ingress and increased levels of saturation. Due to the high rainfall overnight, the open pipe joints allowed sufficient water ingress into the soil, developing hydrostatic pressure in the deeper tension cracks, thus localized failure occurred.*

Moreover, the contractor did not construct the wall according to the design and construction drawings which could have been prevented if adequate construction monitoring was performed.

Design issues encountered by others:

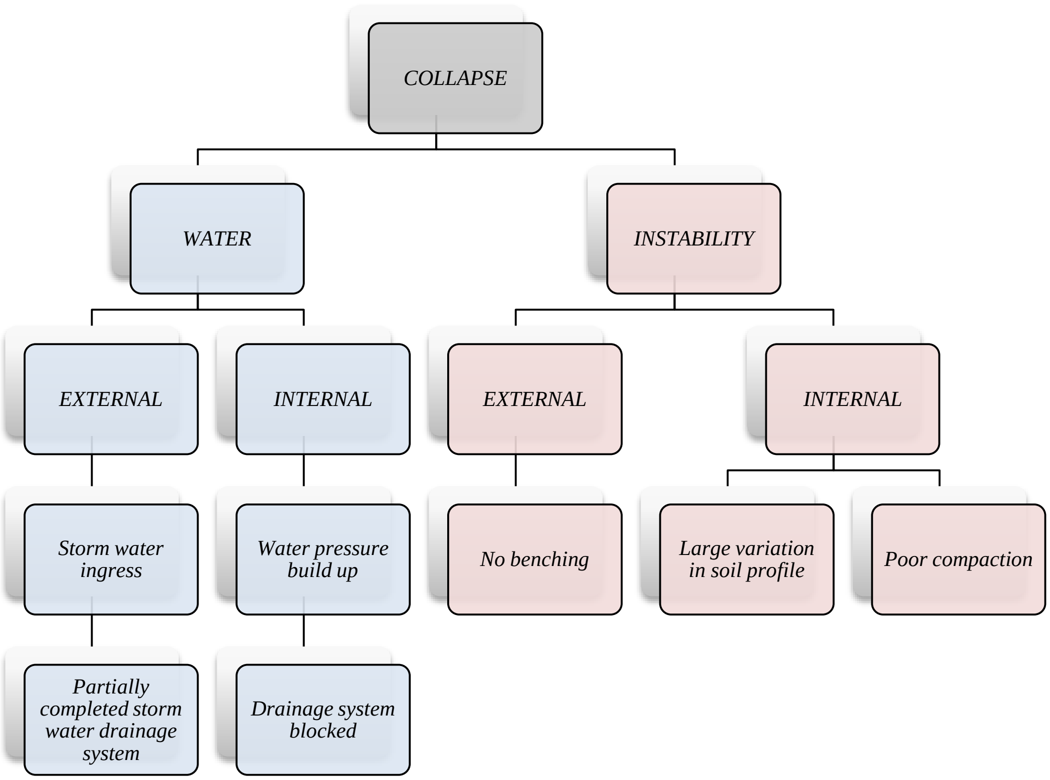
The reinforcement was inadequate. Furthermore, the following information should have been incorporated in the design and indicated on the construction drawings:

- *The horizontal geofabric should have extended 500mm into the virgin soil of the original embankment.*
- *As the natural cut face was at a slope, benching was compulsory. Unfortunately the slopes were very steep and benching was not possible. Best practice would have been to turn the end of the geotextile up vertically against the cut face for a minimum of 300mm.*

An adequate drainage should have been incorporated to prevent the accumulation of water behind the wall. Especially as the Residual Granite backfill material used is infamous for its large variation in soil properties and its collapsible grain structure.

Failure flow chart:

Note: This is a diagrammatic representation to describe all components which contributed to the failure of the wall.



Annexure C

Design examples of a gravity and a reinforced soil CRB wall according to the CMA design manuals

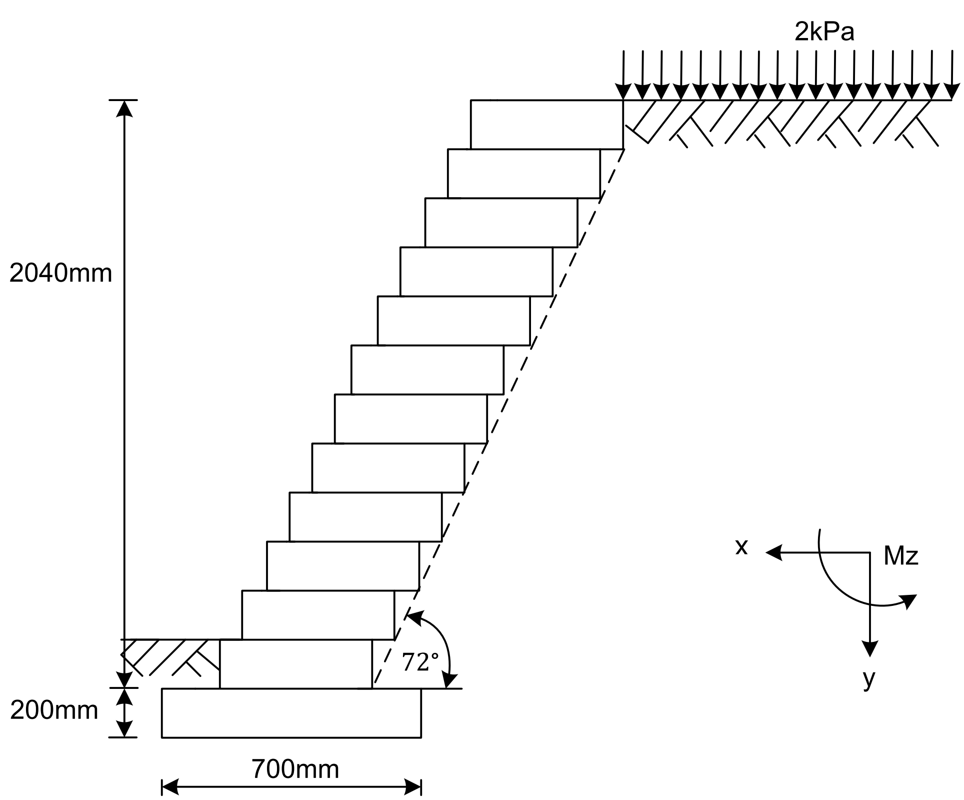
DESIGN EXAMPLE 1: GRAVITY CRB WALL

Note: This example is based on the author's interpretation of the requirements of the CMA design manual for gravity CRB walls.

A horizontal platform is to be constructed by retaining an embankment at a 72^0 slope to the horizontal and would require a retaining wall with a 2.04m vertical height from the top of the foundation to retain the upslope side of the platform.

The client requests a Loffelstein CRB wall. No slope exists in front of the wall. The soil is uniform, cohesionless and an angle of internal friction of 32^0 is applicable to the soil. Allow for a 2kPa surcharge behind the wall.

Assume soil density $\gamma = 20kN/m^3$ and the block friction and nib shear strength is 35^0 and 0^0 respectively. Assume that no water table influences the CRB wall system and that the allowable bearing pressure of the foundation soil is 100kPa. Initially design a Loffelstein L500 block wall.



WALL GEOMETRY SUMMARY			
Height of fill		2.04	m
Length of blocks		0.5	m
No of block courses		12	
Height of blocks		170	mm
Height of wall		2.04	m
Wall inclination	from horizontal	72	deg

STEP 1: Decide on the soil parameters

Given: $\emptyset = 32°$

$\gamma = 20kN/m^3$

From the given parameters, the following can be calculated:

Wall friction: $\delta = 0.9\ (\emptyset) = 0.9(32°) = 28.8°$

Foundation wall friction: $\frac{2}{3}\emptyset = \frac{2}{3}(32°) = 21.3°$

Soil/soil base friction: $\mu = \emptyset = 32°$

Soil/concrete base friction: $\mu = \emptyset = 32°$

The base friction and the friction angle of the soil are the same as the material on site is used as the backfill material and an in-situ cast foundation is incorporated.

MATERIAL PROPERTIES SUMMARY		
Friction angle of soil	32	deg
Density of soil	20	kN/m^3
Combined density of block and soil	1800	kg/m^3
Angle of wall friction	28.8	deg
Foundation wall friction	21.3	deg
Soil/soil base friction	32	deg
Soil/concrete base friction	32	deg

STEP 2: Select a trial wall inclination

No space constraints are present; a wall slope of 72^0 is requested.

STEP 3: Calculate the earth pressure

According to the CMA design manual for gravity CRB walls, the Muller-Breslau solution is used to calculate the active forces acting on the CRB wall system.

The active pressure coefficient is calculated using Equations (1.1) and (1.2).

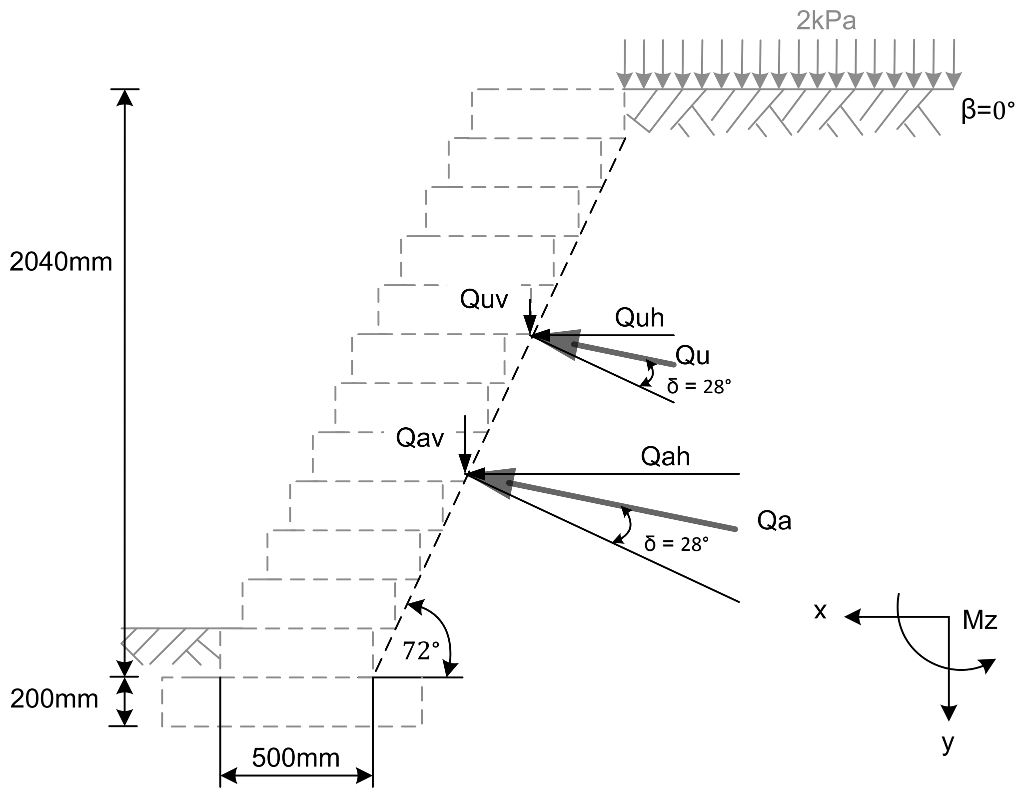
$$f_1 = \frac{\sin^2(180° - \alpha + \emptyset)\cos\delta}{\sin(180° - \alpha)\sin(180° - \alpha - \delta)\left[1 + \sqrt{\frac{\sin(\emptyset+\delta)\sin(\emptyset-\beta)}{\sin(180°-\alpha-\delta)\sin(180°-\alpha+\beta)}}\right]^2} \tag{1.1}$$

$$\therefore f_1 = 0.134$$

$$k_a = \frac{f_1}{\sin(180° - \alpha)\cos\delta} \tag{1.2}$$

$$\therefore k_a = 0.160$$

Where,



With k_a calculated, the active forces due to earth pressures and the force on the wall due to the UDL can be calculated with Equation (1.3) and (1.4) respectively. Take note that H is 2.04m as the active forces only act over the retained soil height.

$$\boldsymbol{Q_a = \frac{1}{2}\gamma H^2 k_a} \tag{1.3}$$

$$\therefore \boldsymbol{Q_a = 6.668kN/m}$$

$$\boldsymbol{Q_u = wHk_a} \tag{1.4}$$

$$\therefore Q_u = 0.654kN/m$$

STEP 4: Calculate the resultant force

Equations (1.5) to (1.8) are used to calculate the horizontal and vertical components of the forces acting on the wall per meter run. Equation (1.9) calculates the inclination of the resultant force to the horizontal. Hence the resultant force can be calculated using Equation (1.10).

$$\textit{Active force below horizontal} = \delta + \alpha - 90°$$

LOADING SUMMARY		
To top of foundation (Point A):		
Active force due to earth pressure	6.668	kN/m
Active force due to UDL	0.654	kN/m
Active force below horizontal	10.8	deg

$$Q_{av} = Q_a \sin(\delta + \alpha - 90°) \tag{1.5}$$

$$Q_{ah} = Q_a \cos(\delta + \alpha - 90°) \tag{1.6}$$

$$Q_{uv} = Q_u \sin(\delta + \alpha - 90°) \tag{1.7}$$

$$Q_{uh} = Q_u \cos(\delta + \alpha - 90°) \tag{1.8}$$

$Q_{av} =$	1.249kN/m
$Q_{ah} =$	6.55kN/m

Q_{uv} =	0.122kN/m
Q_{uh} =	0.642kN/m

$$\boldsymbol{w_e = 12 \times 0.17 \times 1800 \times \frac{9.81}{1000} = 18.011kN/m}$$

$$\boldsymbol{\psi = tan^{-1}\left(\frac{Q_{av} + Q_{uv} + w_e}{Q_{ah} + Q_{uh}}\right)} \tag{1.9}$$

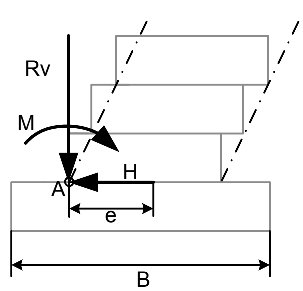
$$\boldsymbol{\therefore\ \psi = 70°}$$

$$\boldsymbol{R = \left(\frac{Q_{ah} + Q_{uh}}{cos\,\psi}\right)} \tag{1.10}$$

$$\boldsymbol{\therefore\ R = 20.674kN\,/m}$$

<u>**STEP 5**: Check the line of action of the resultant force</u>

By taking moments about the toe of the bottom row of blocks (M_A) and dividing the moment by the total vertical force, the eccentricity of the resultant force in the negative x-direction from the origin (A) can be computed.



$$\boldsymbol{e = \frac{M_A}{R_v} = \frac{6.366}{19.383} = 0.328m}$$

The resultant force is 328mm behind the toe of the bottom row of blocks, in the negative x-direction. Hence, the middle third condition is satisfied and the resultant force passes within the bottom row of blocks.

STEP 6: Check the mode of failure against overturning

By calculating the overturning and restoring moments about the toe, the overturning factor of safety can be computed according to Equation (1.11).

$$\boldsymbol{FOS_{overturning} = \frac{M_{resisting}}{M_{overturning}} \geq 1.5} \tag{1.11}$$

$$\boldsymbol{M_{resisting} = W_e L + Q_{av} L_{av} + Q_{uv} L_{uv}} \tag{1.12}$$

$$\boldsymbol{M_{overturning} = Q_{ah} L_{ah} + Q_{uh} L_{uh}} \tag{1.13}$$

$$\boldsymbol{FOS_{overturning} = \frac{11.475}{5.109} = 2.2 \geq 1.5 \quad \therefore OK}$$

STEP 7: Check the mode of failure against block-on-block sliding

The factor of safety against block on block sliding is calculated in Equation (1.16) at the most critical level, between the bottom two rows of blocks.

$$\boldsymbol{F_{resisting} = R \sin(\psi + \omega) \tan \rho + N_s} \tag{1.14}$$

$$\boldsymbol{F_{resisting} = 13.572kN/m}$$

$$\boldsymbol{F_{mobilizing} = R\cos(\psi + \omega)} \tag{1.15}$$

$$\boldsymbol{F_{mobilizing} = 7.192kN/m}$$

$$\boldsymbol{FOS_{block\ sliding} = \frac{F_{resisting}}{F_{mobilizing}} \geq 1.5} \tag{1.16}$$

$$\boldsymbol{FOS_{block\ sliding} = \frac{13.572}{7.192} = 1.9 \geq 1.5 \quad \therefore OK}$$

<u>**STEP 8**: Determine a suitable founding depth</u>

Calculate the active forces by extending the pressure distributions down to the base of the foundation. H is taken as 2.24m for the active pressures, passive pressure is ignored as all passive resistance would be lost if someone were to dig a trench in front of the wall for the instillation or repair of services. No excavation is allowed up to a depth below the foundation of the wall.

LOADING SUMMARY		
<u>To bottom of foundation (point B):</u>		
Active force due to earth pressure	8.039	kN/m
Active force due to UDL	0.718	kN/m
Active force below horizontal	10.8	deg
Wall weight (Vertical)	18.011	kN/m
Foundation weight (Vertical)	2.800	kN/m

$Q_{av} =$	1.506kN/m

Q_{ah} =	7.897kN/m
Q_{uv} =	0.134kN/m
Q_{uh} =	0.705kN/m

$$\boldsymbol{F_{resisting} = (Q_{av} + Q_{uv} + Q_p \sin(\delta_f) + W_e + W_f) \tan(\mu) + Q_p \cos(\delta_f)} \tag{1.17}$$

$$\boldsymbol{F_{resisting} = 14.03kN/m}$$

$$\boldsymbol{F_{mobilizing} = Q_{ah} + Q_{uh}} \tag{1.18}$$

$$\boldsymbol{F_{mobilizing} = 8.602kN/m}$$

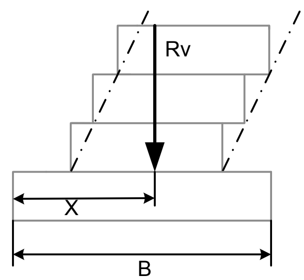
$$\boldsymbol{FOS_{foundation\ sliding} = \frac{F_{resisting}}{F_{mobilizing}} \geq 1.5} \tag{1.19}$$

$$\boldsymbol{FOS_{foundation\ sliding} = \frac{14.03}{8.602} = 1.6 \geq 1.5 \quad \therefore OK}$$

A satisfactory factor of safety was obtained, hence no more iteration is deemed necessary.

STEP 9: Check mode of failure against excessive settlement

The standard method applicable to eccentrically loaded foundations are utilized to calculate the foundation bearing pressures. The middle third rule is satisfied, therefore Equation (1.23) and (1.24) are applicable.



$$\boldsymbol{R_v = R\sin(\psi)} \tag{1.20}$$

$$\boldsymbol{R_v = 19.383kN/m}$$

$$\boldsymbol{E_f = \frac{B}{2} - X} \tag{1.21}$$

$$\boldsymbol{E_f = -0.07843m}$$

$$\boldsymbol{M = 6R_v\frac{E_f}{B^2}} \tag{1.22}$$

$$\boldsymbol{M = -18.615kNm/m}$$

$$\boldsymbol{Front\ bearing\ pressure} = \frac{\boldsymbol{R_v}}{\boldsymbol{B}} - \boldsymbol{M} \tag{1.23}$$

$$\boldsymbol{Front\ bearing\ pressure} = \boldsymbol{46kPa} < \boldsymbol{100kPa} \quad \therefore \boldsymbol{OK}$$

$$\boldsymbol{Back\ bearing\ pressure} = \frac{\boldsymbol{R_v}}{\boldsymbol{B}} + \boldsymbol{M} \tag{1.24}$$

$$\boldsymbol{Back\ bearing\ pressure} = \boldsymbol{9kPa} < \boldsymbol{100kPa} \quad \therefore \boldsymbol{OK}$$

STEP 10: Optimize the block mix

Optimize the block mix and repeat the design procedure.

STEP 11: Repeat if the design criteria limits are not satisfied

Ensure the design criteria limits are satisfied with the optimized block mix.

STEP 12: Check global stability

Overall slope stability analyses are beyond the scope of the CMA code of practise for gravity walls. No adverse conditions give rise to slope instability. The slope behind and in front of the wall is horizontal, therefore slope stability is not a critical failure mechanism, and hence global stability is OK. Furthermore, as the following conditions exist, the wall system is stable:

- No large mass of soil/rock surrounds the wall;
- Soft clays and bedrock with planar weaknesses are not present.

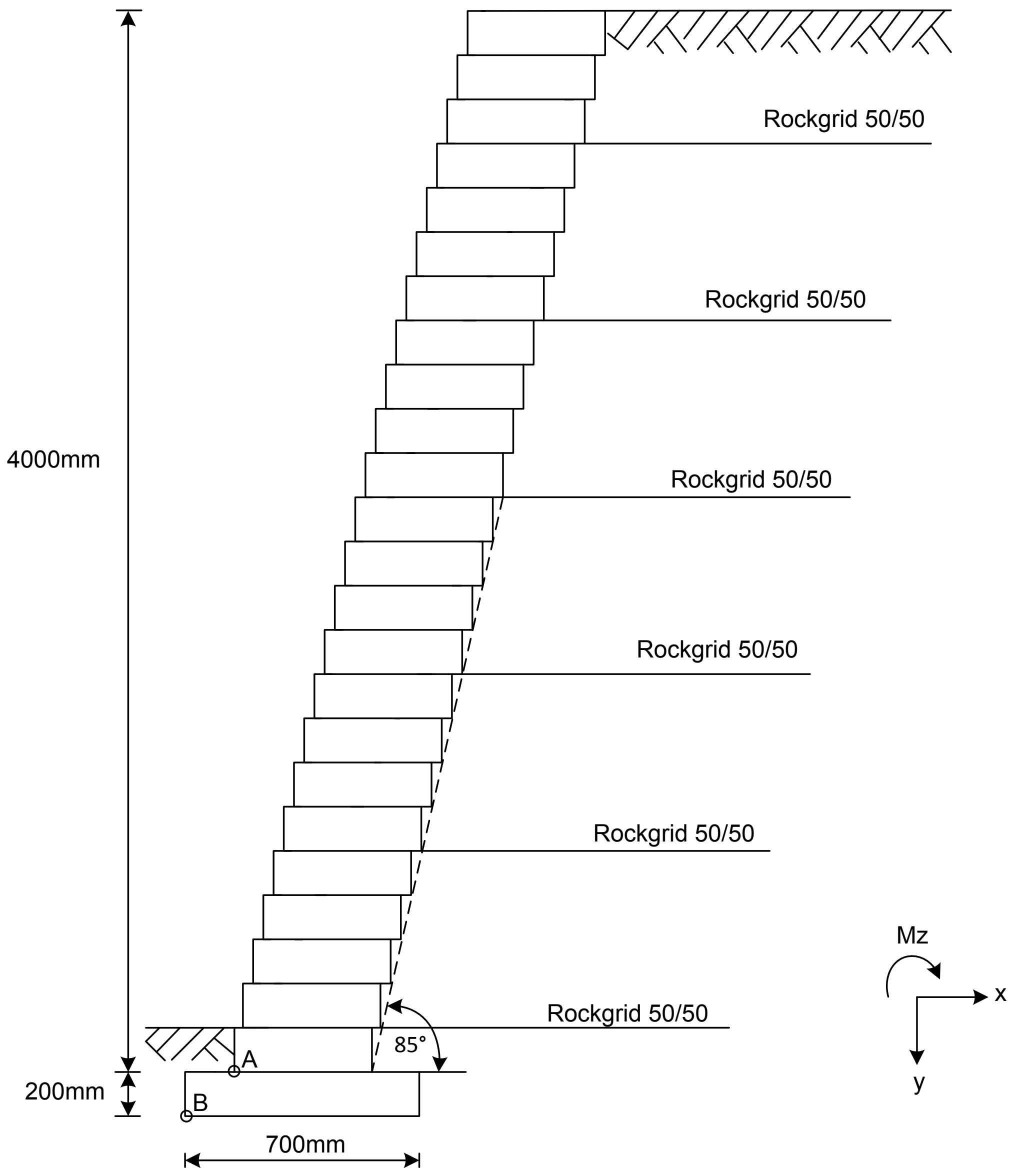
DESIGN EXAMPLE 2: REINFORCED SOIL CRB WALL

Note: This example is based on the author's interpretation of the requirements of the CMA design manual for reinforced CRB walls.

A horizontal platform is to be constructed by retaining an embankment at an 85^0 slope to the horizontal and would require a retaining wall with a 4m vertical height from the top of the foundation to retain the upslope side of the platform.

The client requests a soil reinforced Loffelstein L500 CRB wall. No slope exists in front of the wall. The soil is uniform, cohesionless and an angle of internal friction of 30^0 is applicable to the soil.

Assume soil density $\gamma = 20kN/m^3$ and the block friction and nib shear strength is 35^0 and 0^0 respectively. Assume that no water table influences the CRB wall system and that the allowable bearing pressure of the foundation soil is 200kPa.



WALL GEOMETRY SUMMARY			
Height of fill		4.08	m
Length of blocks		0.5	m
No of block courses		24	
Height of blocks		170	mm
Height of wall		4.08	m
Wall inclination	from horizontal	85	deg

STEP 1: Check the external stability

Given: $\emptyset = 30°$

$\gamma = 20kN/m^3$

From the given parameters, the following can be calculated:

Wall friction: $\delta = 0.9\ (\emptyset) = 0.9(30°) = 27°$

Foundation wall friction: $\frac{2}{3}\emptyset = \frac{2}{3}(30°) = 20°$

Soil/soil base friction: $\mu = \emptyset = 30°$

Soil/concrete base friction: $\mu = \emptyset = 30°$

MATERIAL PROPERTIES SUMMARY		
Friction angle of soil	30	deg
Density of soil	20	kN/m^3
Combined density of block and soil	1800	kg/m^3
Wall friction	27	deg
Foundation wall friction	20	deg
Soil/soil base friction	30	deg
Soil/concrete base friction	30	deg

The approximate check for overall stability is based on earth pressure acting horizontally. The earth pressure is simplified to the Rankine equation as the wall is near vertical:

$$\boldsymbol{k_a = \tan^2\left(45 - \frac{\emptyset}{2}\right) = 0.3333} \tag{2.1}$$

Calculate the active force acting on the wall and the factor of safety against sliding and overturning. These safety factors should be larger than 1.5 (according to the CMA) or 2 (according to Alan Block).

This active force acts over the retained height with H = 4.08m:

$$\boldsymbol{Q_a = \frac{1}{2}\gamma H^2 k_a} \tag{2.2}$$

$$\boldsymbol{\therefore Q_a = 55.488kN/m}$$

This active force acts over the entire height to the bottom of the foundation with H = 4.28m:

$$\therefore \boldsymbol{Q_a = \ 61.061 kN/m}$$

The total reinforcement length from the front of the wall is approximately 80% of its height therefore,

$$\boldsymbol{L = 0.8 \times H = 3.264 m}$$

LOADING SUMMARY		
To top of foundation (Point A):		
Active force due to earth pressure	55.488	kN/m
Weight of the soil (Vertical)	225.542	kN/m
Weight of the wall (Vertical)	36.022	kN/m
To bottom of foundation (Point B):		
Active force due to earth pressure	61.061	kN/m

The factor of safety against sliding is:

$$\boldsymbol{FOS_{sliding} = \frac{W_s \tan(\emptyset) + W_w \tan(\emptyset)}{\frac{1}{2} \times \gamma \times H^2 \times k_a} \geq 1.5\ or\ 2} \qquad (2.3)$$

$$\boldsymbol{FOS_{sliding} = \frac{225.542 \tan(30) + 36.022 \tan(30)}{55.488} = 2.7 \geq 1.5\ or\ 2}$$

The factor of safety against overturning is:

$$\boldsymbol{FOS_{overturning} = \frac{Weight\ \times\ lever\ arm\ about\ A\ or\ B}{Active\ soil\ force\ \times lever\ arm\ about\ A\ or\ B} \geq 1.5\ or\ 2} \qquad (2.4)$$

COMPLETE LOADING SUMMARY (Point A):							
			Forces/m		Lever Arm		Moment
			Fx	Fy	y	x	Mz
			(kN/m)	(kN/m)	(m)	(m)	(kNm/m)
OVERTURNING:							
Earth pressure	horizontal	(Qah)	-55.488		1.360		75.464
Earth pressure	vertical	(Qav)		0		3.383	0.000
TOTAL OVERTURNING			**-55.488**	**0**			**75.464**
RESISTING:							
Wall weight	vertical	(Ww)		-36.022		0.428	-15.435
Soil weight	vertical	(Ws)		-225.542		2.060	-464.725
TOTAL RESISTING			**0.0000**	**-261.564**			**-480.16**
TOTAL (at origin A):			**-55.488**	**-261.564**			**-404.696**

About A:

$$FOS_{overturning} = \frac{480.16}{75.464} = 6.3 \geq 2 \quad \therefore OK \tag{2.5}$$

COMPLETE LOADING SUMMARY (Point B):							
			Forces/m		Lever Arm		Moment
			Fx	Fy	y	x	Mz
			(kN/m)	(kN/m)	(m)	(m)	(kNm/m)
OVERTURNING:							
Earth pressure	horizontal	(Qah)	-61.061		1.427		87.114
Earth pressure	vertical	(Qav)		0		3.483	0
TOTAL OVERTURNING			**-61.061**	**0**			**87.114**
RESISTING:							
Wall weight	vertical	(Ww)		-36.022		0.528	-19.037
Soil weight	vertical	(Ws)		-236.198		2.160	-510.301
Foundation weight	vertical	(Wf)		-2.800		0.350	-0.980
TOTAL RESISTING			**0**	**-275.021**			**-529.338**
TOTAL (at origin B):			**-61.061**	**-275.021**			**-442.224**

About B:

$$FOS_{overturning} = \frac{529.338}{87.114} = 6.1 \geq 2 \quad \therefore OK \tag{2.6}$$

Bearing capacity and excessive settlement

By treating the reinforced CRB wall as a stable rigid block and taking the net moments of the loads about A, the foundation pressure can be calculated. The full weight including the weight of the soil and weight of the wall are taken into account in the calculations to determine the foundation pressure.

$$B' = \frac{2M_A}{W} \tag{2.7}$$

$$B' = \frac{2 \times (-404.696)}{-261.564} = 3.094m$$

The highest likely load on the foundation is calculated as:

$$P_s = \frac{W}{B'} \tag{2.8}$$

$$\therefore P_s = 84kPa$$

$$84\,kPa < 200kPa \;\therefore OK$$

<u>**STEP 2**: Check the internal stability</u>

Maximum tension

To determine the maximum tension in each reinforcement layer, the following equations are used and the values are determined in an EXCEL spreadsheet:

The horizontal stress at any given depth is calculated using the following equation:

$$\sigma_H = k_a \times \gamma \times Z \tag{2.9}$$

By multiplying this stress with the contributory vertical spacing for each reinforcement layer, the maximum tension in each reinforcement layer per unit width of wall can be calculated:

$$T_{MAX} = \sigma_H \times S_v \tag{2.10}$$

The maximum vertical spacing is given as 0.3 times the length of the reinforcement layer:

$$S_v < 0.3 \times L$$
$$S_v < 0.98$$

This spacing is limited by the block height; therefore the maximum spacing can be 0.85m.

Reinf.	Z (m)	Top layer (m)	Bottom layer (m)	σh top (kPa)	σh bottom (kPa)	Sv (m)	Nr blocks	Max Tension (kN/m)
1	0.51	0	0.85	0.000	5.667	0.85	5	2.408
2	1.19	0.85	1.53	5.667	10.200	0.68	4	5.395
3	1.87	1.53	2.21	10.200	14.733	0.68	4	8.477
4	2.55	2.21	2.89	14.733	19.267	0.68	4	11.560
5	3.23	2.89	3.57	19.267	23.800	0.68	4	14.643
6	3.91	3.57	4.08	23.800	27.200	0.51	3	13.005
Total								**55.488**

Pull-out

A preliminary check is conducted using the simplified Rankine method to determine the pull-out resistance of the reinforcement for each layer. More rigorous checks are required to verify the critical failure surface.

The wall inclination is between 70^0 and 90^0 to the horizontal, therefore the single wedge method will be used. As the face of the wall is near vertical, an initial critical failure surface is assumed to be:

$$45° + \frac{\emptyset}{2} = 60°$$

This failure surface passes through the toe of the wall.

The total length of reinforcement required for internal stability is determined from:

$$L = l_a + l_e + length\ of\ the\ blocks \tag{2.11}$$

Where l_e is the required embedment length of the reinforcement in the resistance zone and l_a is determined from the following equation for a critical failure surface of 60^0:

$$\boldsymbol{l_a = (H - Z)\tan\left(45° - \frac{\emptyset}{2}\right)} \tag{2.12}$$

Hence, the pull-out resistance on both sides of the failure surface is determined from Equations (2.14) and (2.15). The interface shear properties for the Rockgrid/soil interface are typically determined by tests. For the purpose of this design example, the friction angle is assumed to be 27°.

Therefore,

$$\boldsymbol{\mu = \tan(27°) = 0.510} \tag{2.13}$$

Based on l_e :

$$\boldsymbol{Pull - out\ resistance = \sigma_v \times \mu \times l_e} \tag{2.14}$$

Based on l_a:

$$\boldsymbol{Pull - out\ resistance = \sigma_v \times \mu \times l_a} \tag{2.15}$$

The combined pullout resistance of the reinforcement at all levels are compared to the load required to prevent failure to determine the factor of safety against pullout.

Reinf.	Z (m)	La (m)	σv (kPa)	μ	Le (m)	Pullout R (kN/m)	Pullout R (kN/m)	Max Tension (kN/m)	Min pullout (kN/m)	FOS	Check FOS>1.5
1	0.51	2.061	10.2	0.510	0.703	3.653	10.712	2.408	3.653	1.5	OK
2	1.19	1.669	23.8	0.510	1.095	13.284	20.234	5.395	13.284	2.5	OK
3	1.87	1.276	37.4	0.510	1.488	28.357	24.315	8.477	24.315	3.3	OK
4	2.55	0.883	51	0.510	1.881	48.870	22.954	11.560	22.954	4.2	OK
5	3.23	0.491	64.6	0.510	2.273	74.825	16.153	14.643	16.153	5.1	OK
6	3.91	0.098	78.2	0.510	2.666	106.221	3.911	13.005	3.911	8.2	OK

Tensile overstress

From the Kaytech technical data sheet attached to this design example, the tensile strength for the Rockgrid 50/50 in the machine and transverse direction is 50kN/m.

These ultimate tensile strengths are factored down using the following reduction factors to determine the allowable tensile strengths of the geosynthetic reinforcement used for design. These reduction factors are provided from the manufacturer of the geosynthetic.

- **Creep factor:**

Based on 120 year design life, $f_c = 1.65$.

- **Instillation damage:**

As the geotextile should be able to resist damage during instillation, $f_i = 1.05$.

- **Environmental factors:**
  - Chemical degradation: $f_{e1} = 1.0$
  - Sunlight degradation: $f_{e2} = 1.1$
  - Temperature degradation: $f_{e3} = 1.0$
  - Hydrolysis degradation: $f_{e4} = 1.0$
  - Biological degradation: $f_{e5} = 1.0$
  - Polymeric ageing: $f_{e6} = 1.0$

- **Soil material factors:**

To allow for uncertainties in the soil properties, $f_{ms} = 1.0$.

- **Class of structure:**

As the wall inclination to the horizontal is steeper than 70^0, $f_s = 1.1$.

Therefore, the allowable tensile strength in both the machine and transverse directions can be calculated with the following equation:

$$\boldsymbol{T_{all}} = \frac{\boldsymbol{T_{ult}}}{\boldsymbol{f_m \times f_c \times f_i \times f_e \times f_{ms} \times f_s}} \tag{2.16}$$

The allowable tensile strength in the machine and transverse direction is 23.851kN/m.

This allowable tensile strength is compared to the internal stability and pull-out resistance of the reinforcement to determine the factors of safety which should be larger than 1.5.

Reinf.	Z (m)	Max Tension (kN/m)	FOS	Check Tmax<Tall	Check FOS>1.5
1	0.51	2.408	9.9	Tmax<Tall OK	OK
2	1.19	5.395	4.4	Tmax<Tall OK	OK
3	1.87	8.477	2.8	Tmax<Tall OK	OK
4	2.55	11.560	2.1	Tmax<Tall OK	OK
5	3.23	14.643	1.6	Tmax<Tall OK	OK
6	3.91	13.005	1.8	Tmax<Tall OK	OK
Total		**55.488**			

Internal sliding

To evaluate internal sliding, the tangent of the inclination of the resultant force to the horizontal is compared to the tangent of the soil to fabric friction angle.The soil to fabric friction angle is 17^0 for a non-woven, needle punched high strength composite geotextile such as the Rockgrid 50/50 with a soil of $\emptyset = 30°$, provided by the geosynthetic reinforcement manufacturer.

$$Q_{av} = Q_a \sin(\delta + \alpha - 90°) \tag{2.17}$$

$$\therefore Q_{av} = 0kN/m$$

$$Q_{ah} = Q_a \cos(\delta + \alpha - 90°) \tag{2.18}$$

$$\therefore Q_{ah} = 55.488kN/m$$

$$\psi = \tan^{-1}\left(\frac{Q_{av} + Q_{uv} + w_e}{Q_{ah} + Q_{uh}}\right) \tag{2.19}$$

$$\therefore \psi = 78.023°$$

$$FOS = \frac{\tan(78.023°)}{\tan(27°)} = 9.3 \gg 1.5 \ \therefore OK$$

STEP 3: Determine the type of facing

Connection failure

This check must be done through laboratory tests to determine the pull-out of the reinforcement from the facing and is dependent on the type of reinforcement used and the type of facing and how the two components are connected to each other.

Shear failure and bulging

The factor of safety against block on block sliding is calculated in Equation (2.23) at the most critical level, between the bottom two rows of blocks.

$$\boldsymbol{R = \left(\frac{Q_{ah}}{cos\,\psi}\right)} \tag{2.20}$$

$$\boldsymbol{\therefore R = 267.386kN/m}$$

$$\boldsymbol{F_{resisting} = R\sin(\psi+\omega)\tan\rho + N_s} \tag{2.21}$$

$$\boldsymbol{\therefore F_{resisting} = 183.15kN/m}$$

$$\boldsymbol{F_{mobilizing} = R\cos(\psi+\omega)} \tag{2.22}$$

$$\boldsymbol{\therefore F_{mobilizing} = 55.488kN/m}$$

$$\boldsymbol{FOS_{block\ sliding} = \frac{F_{resisting}}{F_{mobilizing}} \geq 1.5} \tag{2.23}$$

$$\boldsymbol{FOS_{block\ sliding} = \frac{183.15}{55.488} = 3.3\ \geq 1.5\ \ \therefore OK}$$

Toppling

No unreinforced height exists at the top of the structure; therefore toppling is OK.

STEP 4: Check global stability

The slope behind and in front of the wall is horizontal, therefore slope stability is not a critical failure mechanism, and hence global stability is OK.

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TECHNICAL DATA SHEET

Product Name	ROCKGRID PC
Reference No:	DS REIN 0455-01/2013
Date of Issue	21 December 2012
Description	High strength composite geotextile offering high modulus characteristics for reinforcement applications, with the additional benefits of in-plane capacity and high installation survivability

			50/50	100/100	200/200	
Material		Polyester, staple fibre 150 g/m^2 needle punched, nonwoven / high strength polyester yarns				
Short Term Tensile Strength (T_u)	Machine	kN/m	50	100	200	ISO 10319
	Across	kN/m	50	100	200	
	Elongation	%	10	10	10	
Long Term Design Strength (LTDS*) 120 Years		kN/m	26	52	105	ISO 10319
Creep Limited Strength 120 Years		kN/m	30	60	120	ISO 13431
Water Flow Rate	Normal to Plane	ℓ/s/m^2	150			ISO 11058
	In Plane 20 kPa	ℓ/s/m/hr	20			ISO 12958
Roll Dimensions		m	5 x 100			ISO 12958

$$LTDS = \frac{T_u}{f_c.f_d.f_e.f_m}$$

f_c (creep) = 1.65 (120 years)

f_d (damage) = 1.05 (sand, silt, clay, yarn facing soil)

f_e (environment) = 1.10 (pH 4-9)

f_m (material) = 1.00

The above results represent laboratory averages
Kaytech reserves the right to make technical modifications to its products