A Contribution to the Advancement of Geotechnical Engineering in South Africa

by
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Thesis presented in fulfilment of the requirements for the degree of Doctor of Engineering in the Faculty of Engineering at Stellenbosch University

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Declaration

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The gratification of curiosity rather frees from uneasiness than confers pleasure, we are more pained by ignorance than gratified by instruction.
(Johnson, 1751)

Curiosity is a gift, a capacity of pleasure in knowing.
(Ruskin, 1819)

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SUMMARY

Geotechnical engineering is a relatively young field of engineering and one in which there are still many unanswered questions and gaps in our knowledge. Added to this, the geotechnical materials on each new site on which geotechnical work is undertaken are the unique product of many influences including geology, geomorphology, climate, topography, vegetation and man. There is thus plenty of scope for innovation.

This dissertation describes the contributions made to Geotechnical Engineering in South Africa by the Candidate over a period of close on 40 years. It describes the three-step process followed in the majority of these contributions. Step one is the identification of a problem that requires investigation, the application of new techniques or simply the consolidation of existing knowledge. Step 2 is the investigation of the problem and the development of a solution. Step 3 is sharing the outcome of this work with the profession by means of publications, by presentations at seminars and conferences or by incorporation into standards / codes of practice.

Part 1 of the dissertation describes the exciting environment in which geotechnical engineers operate. This environment is characterised by openness and cooperation between practitioners of geotechnical engineering, be they geotechnical engineers, engineering geologists, contractors, suppliers or academics. This part also explores the parallels in the roles played by academics and practitioners and how each can contribute to the advancement and dissemination of knowledge. Part 2 describes contributions made in various fields including problem soils (dolomites, expansive clays, uncompacted fills, etc.), lateral support, pile design and construction, health and safety, and cooperation with international organisations. Part 3 describes the Candidate's involvement in the introduction of limit states geotechnical design into South African practice culminating in the drafting of SANS 10160-5 on *Basis of Geotechnical Design and Actions*. It also describes the Candidate's work with the ISSMGE Technical Committee TC23 dealing with limit states design. Part 4 deals with the Candidate's contribution to other codes and standards and his role on various committees of the Engineering Council of South Africa and the South African Bureau of Standards.

The final part of the dissertation provides an overview of the process followed in making such contributions, highlighting the role played by curiosity and a desire to share the knowledge gained with others in the profession. It continues by identifying work that still needs to be done in many of the areas where contributions have been made and concludes with a statement of what the candidate would still like to achieve during the remainder of his career.

The Candidate gratefully acknowledges the generous opportunities afforded to him by his colleagues at work and the invaluable guidance and mentorship received from fellow professionals in academia and practice.

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OPSOMMING

Geotegniese ingenieurswese is ‘n relatiewe jong wetenskap en een met vele kennisgapings en waarin daar nog talle vrae onbeantwoord bly. Daarby is geotegniese materiale uniek tot elke terrein waarop werk aangepak word en die produk van ‘n kombinasie van prosesse; insluitend geologie, geomorfologie, klimaats toestande, topografie, plantegroei en menslike aktiwiteite. Daar is dus nog ruim geleentheid vir innoverende bydraes.

Hierdie verhandeling beskryf die Kandidaat se bydraes tot Geotegniese Ingenieurswese in Suid-Afrika oor die afgelope 40 jaar. Dit beskryf ‘n drie-voudige benadering wat in die meeste van die bydraes gevolg is. Die eerste stap is om die probleem te definieer en te omskryf in terme van die ondersoek wat geloods moet word, asook die noodsaaklikheid vir die ontwikkeling van nuwe tegnologie teenoor die konsolidasie van bestaande inligting. Tydens die tweede stap word die probleem ondersoek en ‘n oplossing ontwikkel. Die derde stap is om die resultate te deel met die geotegniese bedryf by wyse van publikasies, voorleggings by konferensies en seminare, en insluiting in praktykkodes en standaarde.

Deel 1 beskryf die opwindende werksomstandighede waarbinne geotegniese ingenieurs hul bevind. Dit word geken aan die ope samewerking tussen belanghebbende partye; onder andere ingenieurs, ingenieursgeoloë, kontrakteurs, verskaffers en akademici. Deel 1 beklemtoon ook die parallelle rolle wat vertolk word deur akademici en praktiserende ingenieurs en hoe beide partye bydraes maak tot die ontwikkeling en verspreiding van tegnologie. Deel 2 beskryf die Kandidaat se bydraes tot verskeie navorsingsvelde; waaronder probleem-grondtoestande (dolomiet, swellende kleie, ongekonsolideerde opvullings ens.), laterale ondersteuning, ontwerp en konstruksie van heipale, beroepsveiligheid, en samewerking met internasionale organisasies. Deel 3 beskryf die Kandidaat se betrokkenheid by die bekendstelling van limietstaat geotegniese ontwerp in die Suid-Afrikaanse bedryf wat uitgeloop het op die samestelling van SANS 10160-5 *Basis of Geotechnical Design and Actions*. Dit beskryf ook die Kandidaat se samewerking met die ISSMGE *Technical Committee* TC23 wat te make het met limietstaat ontwerp. Deel 4 beskryf die Kandidaat se bydraes tot ander kodes en standaarde en die rolle wat hy vertolk het op verskeie komitees van die Suid-Afrikaanse Raad vir Ingenieurswese asook van die Suid-Afrikaanse Buro van Standaarde.

Die laaste deel van die verhandeling bied ‘n oorsig oor die proses wat gevolg is in bostaande bydraes met die klem op die rol van weetgierigheid en die begeerte om sulke kennis te deel met ander belanghebbendes. Om af te sluit, identifiseer die Kandidaat oorblywende tekortkominge in baie van die vraagstukke waar hy bydraes gelewer het en gee ‘n opsomming van wat hy graag nog sal wil bereik tydens die verdere verloop van sy loopbaan.

Die Kandidaat gee met dank erkenning aan sy kollegas vir die ruim geleenthede wat hom gebied is en die waardevolle leiding en mentorskap wat hy ontvang het van mede praktiserende ingenieurs en akademici.

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FOREWORD

This dissertation is submitted in fulfilment of the requirements of a Doctorate in Engineering degree. The requirements for this degree include that the Candidate should have carried out advanced original research and/or creative work in the field of Engineering Sciences and should submit both original and previously published works which indicate a significant and outstanding contribution to the enrichment of knowledge of the Engineering Sciences.

During the writing of this dissertation, I have faced two main challenges. Firstly, there is no template of a DEng thesis. Secondly, it goes against professional etiquette to be self-laudatory. However the purpose of this dissertation is to demonstrate my compliance with the requirements for the degree.

As such, this dissertation is a personal account of my contribution to the engineering profession in South Africa as I see it. At times, it may be more like a narrative than an academic work. This is because it is simply the story of my career.

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List of Acronyms

AASHTO	American Association of State Highway and Transport Officials
CBE	Council for the Built Environment
CCSA	Concrete Society of South Africa
CESA	Consulting Engineers South Africa
CGS	Council for Geoscience or Canadian Geotechnical Society
CSIR	Council for Scientific and Industrial Research
DSS	Draft South Africa Standard (issued for public comment by SABS)
ECSA	Engineering Council of South Africa
IEC	International Electrotechnical Commission
ICSMGE	International Conference on Soil Mechanics and Geotechnical Engineering
IStructE	Institution of Structural Engineers (London)
ISI	Institute for Scientific Information
ISO	International Organisation for Standardisation
ISSMFE	International Society of Soil Mechanics & Foundation Engineering (pre 1997)
ISSMGE	International Society of Soil Mechanics & Geotechnical Engineering (post 1997)
NBRI	National Building Research Institute (of the CSIR)
NHBRC	National Home Builder's Registration Council
SAAEG	South African Section of the Association of Engineering Geologists
SABS	South African Bureau of Standards
SADC	Southern African Development Community
SAISC	South African Institute of Steel Construction
SAICE	South African Institution of Civil Engineering
SAIEG	South African Institute for Engineering and Environmental Geologists
SANS	South Africa National Standard
SASFA	South African Light Steel Frame Building Association
TBT	Technical barriers to trade
WTO	World Trade Organisation

List of Abbreviations

conf.	conference
CPT	static cone penetration test (Dutch probe)
CPTu	static cone penetration tests with pore pressure measurements
ft	feet (0,3048 m)
int.	international
kPa	kilopascal (or kN/m^2)
kN	kilonewton
LSD	limit states design
m	metres
MN	meganewton
NDPs	nationally determined parameters
proc.	proceedings (of conference or seminar)
SC	Sub-committee (to SABS Technical Committee)
SLS	serviceability limit state
SPT	standard penetration test
TC	Technical Committee (SABS or ISSMGE)
ULS	ultimate limit state
WLD	Working load design

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Part 1: Background

This introductory part of the dissertation describes the development of geotechnical engineering and the environment in which South African geotechnical engineers operate. It attempts to convey some of the excitement and challenges of working in a developing field in which there are still many uncertainties and opportunities.

It explores the criteria used in the recognition of excellence in academia and practice.

This introduction also describes the Candidate's involvement in the process over the past 35 years, in particular by researching and sharing new developments and innovative ways of solving problems with the rest of the geotechnical profession in South Africa.

1. INTRODUCTION

1.1 The Allure of Geotechnical Engineering

There is a certain charm about geotechnical engineering that distinguishes it from other fields of civil engineering even though these may rely on the same fundamental principles. Maybe it is that one is dealing with natural materials whose origins stretch as far back as 4 000 million years. Maybe it is because the geotechnical materials on each site are the unique products of many influences including geological origin, age, tectonic environment, past and present climates, topography, vegetation and the influence of man. Or maybe it is because geotechnical engineering is a marrying of the natural and engineering sciences, of fieldwork and theory, of experimentation and analysis, and of experience and innovation. Whatever it is, one of the overriding attractions of this relatively young field of engineering is that we do not have all the answers. There are always challenges to be met, new techniques to be developed and new insights to be gained.

Mathematical solutions to geotechnical problems have been around for centuries. In 1776, the French physicist Charles-Augustin de Coulomb published an essay on *the application of the rules of maxima and minima to several problems of stability related to architecture*, including the calculation of earth pressure on retaining structures. Almost a century later, in 1857, Scottish engineer and physicist William Rankine explored the same topic when he wrote in the Transactions of the Royal Society on the *stability of loose earth*. In 1882, Christian Mohr, a German Civil Engineer proposed a graphical way or representing the relationship between shear and normal stresses known as the Mohr circle. He extended Coulomb's work to develop the Mohr-Coulomb failure criterion for soils. Moving away from the strength of soils for the time being, in 1885 the French mathematician and physicist Joseph Boussinesq, proposed equations for determining the stress distribution within an elastic solid which are still used today for predicting the settlement of soils.

It is, however, Karl Terzaghi (b1882, Prague – d1963) who is generally regarded as the person who founded modern soil mechanics with the publication of *Erdbaumechanik* in 1925. In all preceding work, soils had been treated as a single phase solid. Terzaghi was the first to consider saturated soil as a two phase material, soil grains and pore water, and partially saturated soil as a three phase material where the pore space is filled with water and air (Donaldson, 1985). His theory of effective stress[1] published in 1936 was probably one of the most important advances in the science and unlocked the door to new perspectives and computational methods in soil mechanics.

Thus, modern soil mechanics is less than a century old. The first International Conference on Soil Mechanics and Foundation Engineering was held in Cambridge Massachusetts in 1936, less than 80 years ago. Those of us born in the latter half of the 20th century may have missed out on the discoveries of those early years but nevertheless share the excitement of contributing to what is still a growing science.

Karl Terzaghi captured something of the challenge of working in a developing field of engineering with his words to his students at Harvard University: "*engineering is a noble sport … but occasionally blundering is part of the game. Let it be your ambition to be the first to discover and announce your blunders… Once you begin to feel tempted to deny your blunders in the face of reasonable evidence, you have ceased to be a good sport*." (Goodman, 1999). The "game" he referred to continues to be played today.

[1] the sharing of applied total stress (σ) on the soil between the effective stress on soil skeleton (σ') and the pore fluid pressure (u) such that $\sigma' = \sigma - u$

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1.2 Development of Geotechnical Engineering in South Africa

Long before the emergence of modern-day soil mechanics, road building pioneers in South Africa were forging links between the coastal areas and the hinterland, often in very challenging topographical and geological environments. One such pioneer was the Scottish-born Andrew Geddes Bain (1797 – 1864). In 1832, he was awarded a medal for the gratuitous supervision of the construction of the Van Ryneveld's Pass near Graaff-Reinet. As a military captain with no formal engineering training, he built the military road through the Ecca Pass in 1836. Bain went on to construct eight major passes in South Africa, including the pass near Wellington in the Western Cape that bears his name. In 1856, Bain produced the first comprehensive geological map of South Africa which was published by the Geological Society of London in 1856. This earned him the name of the "father of geology" in South Africa. His son, Thomas Bain, constructed a further twenty four passes (Storrar, 1984) including the 24km long Swartberg Pass between Oudtshoorn and Prince Albert in the Eastern Cape with its impressive hand-packed stone retaining walls.

As in other places in the world, soil mechanics continued to develop as much as an art as a science, driven by the need for railway lines, roads, dams and irrigation schemes. This was the situation in the 1930's when Jennings graduated as a civil engineer from the University of the Witwatersrand (Donaldson, 1985).

In the same way as Terzaghi is regarded as the father of modern soil mechanics, Jeremiah "Jere" Jennings (1912 – 1979) can certainly claim this title in his native South Africa. Jennings studied soil mechanics at the Massachusetts Institute of Technology under Terzaghi. As a young engineer, he attended that first International Conference on Soil Mechanics and Foundation Engineering in 1936. He was strongly influenced by Profs. Terzaghi, Taylor and Casagrande (Williams, 2006).

On his return to South Africa, Jennings worked for the South Africa Railways and Harbours. In August 1947 he took up the post of Director of the National Building Research Institute (NBRI) of the Council for Scientific and Industrial Research (CSIR), which was formed by an Act of Parliament in 1945 (Korf, 2006 and Donaldson 1985). He attracted several promising young engineers to join the staff, including Basil Kantey, Keeve Steyn, Lou Collins, George Donaldson, Ken Knight and Tony Brink. This was during the period when the mining sector was being revived under the interventionist policies of the National Party which came to power in 1948. Large scale dewatering of the dolomites at the Venterspost Gold Mine started in 1949 (Wagener, 1982) and mining operations commenced on the Free State Goldfields.

These mining developments brought new geotechnical challenges to the fore. Dewatering of the dolomites led to the formation of sinkholes and dolines on an unprecedented scale and the presence of expansive clay soils in the Odendalsrus and Welkom areas of the Orange Free State caused considerable economic loss due to cracking of houses built to accommodate the miners. Jennings and his team were at the forefront of researching these problems. At about the same time, the problem of collapsible soils was being tackled in an attempt to explain the sudden settlement of sandy soils in the Witbank area due to the ingress of water. Many papers were published by Jennings, Williams, Brink, Knight and others on these problem soil conditions.

After completing his stint at the CSIR, Jennings became professor of soil mechanics at the University of the Witwatersrand. Knight took up a similar position at the University of Natal (now University of Kwa-Zulu Natal). Brink served as a lecturer at the universities of the Witwatersrand, Stellenbosch, Pretoria, Cape Town and the Rand Afrikaans University (now University of Johannesburg).

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Dr A.B.A (Tony) Brink (1927 – 2003) was probably the next most influential person in the development of geotechnical engineering in South Africa. A geologist by training (BSc Geology, Pretoria, 1948), Brink shared the conviction of Terzaghi and Jennings that an appreciation of geology is fundamental to the practice of geotechnical engineering (Haaroff and Korf, 2008). Incidentally, he also shared a passion for amateur dramatics with Andrew Geddes Bain. Among his many achievements, he is credited with "discovering" the Pebble Marker, a layer of gravel that often occurs at the base of the transported horizon of the soil profile marking the boundary between transported and residual soils. He played a pivotal role in developing the "MCCSSO" nomenclature for description of soils (moisture, colour, consistency, structure, soil type and origin) that still forms the basis of modern-day description of soil profiles in South Africa. The "Jennings, Brink and Williams" paper (Jennings et al, 1973) is probably the most influential geotechnical paper published in the country to this day.

The other Brink publication that had a major effect on the South African geotechnical engineering and engineering geological fraternity was his series of four books on the Engineering Geology of Southern Africa. The "Brink books", as they have become known, fill the gap between site-specific geotechnical reports and general reference works such as geological maps and the Stratigraphy of South Africa (Geological Survey, 1980). They are an invaluable guide in the planning of geotechnical investigations and interpretation of the results, providing a broad overview of the engineering geology of the region and the type of problems likely to be associated with individual strata. For the young geotechnical engineer, having these books on one's bookshelf is rather like having free access to a vastly experienced group of engineers and geologists whose doors are always open to provide information and guidance (Day, 2006).

The next significant developments in geotechnical engineering were in the field of geotechnical contracting. These included the construction of deep basements in South Africa's major city centres, advances in ground anchoring technology and the availability of new methods and equipment for pile installation and ground improvement. These developments were spurred on by major projects including inner-city development, by the demands of industry such as the Sasol 2 and 3 projects and, lately, by the construction of the Gautrain rapid rail link between Johannesburg and Pretoria.

Lately, the advances have again been on the design side. Significant progress has been made in the development of computer programmes and design aids, often making use of sophisticated numerical analysis techniques that were previously only available in the research environment. Another significant step forwards has been the alignment of South African geotechnical design codes with international standards. This has included the finalisation of a number of standards, such as the standards for development on dolomite land, using a performance based regulatory system (Day, 2011), and the development of a *basis of design* code for geotechnical engineering (Day, 2007a). The development of codes of practice will be discussed further in Parts 3 and 4 of this dissertation.

1.3 South African Geotechnical Engineering Today

Since the early days of Jennings, Brink and others, geotechnical engineering has become firmly established as a recognised branch of the Civil Engineering profession. Jennings and Brink have both passed on, but they have left an indelible mark on the industry. The geotechnical industry of today is still characterised by a practical and innovative approach to solving problems and has, in true Jennings and Brink tradition, preserved a spirit of cooperation between geotechnical engineers and engineering geologists.

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1.3.1 *Professional and Learned Societies*

The "home" of South African geotechnical engineers is the Geotechnical Division of the South African Institution of Civil Engineering (SAICE). The Division was founded at the Second International Conference on Soil Mechanics and Foundation Engineering in Rotterdam in 1948 (Davis, 2006) as one of the original national member societies of the International Society of Soil Mechanics and Foundation Engineering (ISSMFE). The Division has a membership that ranges between 250 and 350 members from year to year, having been over 500 members in the last 20 years. It has a young and dynamic committee that seems to maintain a healthy balance of academics, consultants, contractors and suppliers. They organise, on average, eight events per year which normally include at least one major conference and a number of seminars, with specific events for young geotechnical engineers. The Division also hosts the annual Jennings Memorial Lecture which is delivered by a leading geotechnical engineer from abroad. Its annual awards include the South Africa Geotechnical Medal for outstanding contribution to the profession and the Jennings Award for the best geotechnical publication during the preceding year[2]. In all its activities and awards, the Division strives to serve the needs of all its members, whether academics, consultants, contractors, suppliers or clients. The health of the Division can be measured by its annual turnover and the size of its budget surplus which are among the highest of all the divisions of SAICE.

To celebrate the centenary of the South African Institution of Civil Engineering, the Division published a centenary edition entitled *"Commemorative Journal of the Geotechnical Division of the South African Institution of Civil Engineering"*. It contained a collection of 28 geotechnical papers from the Transactions of the Institution over the 100 year period (Geotechnical Division, 2006). The papers were selected either for their interest value, their reflection of the state of practice at the time, or on account of the influence they have had on the practice of geotechnical engineering.

The Geotechnical Division has a very good working relationship with the S.A. Institute of Engineering and Environmental Geologists. The two organisations frequently host joint events and cooperate in code writing and setting standards in the profession.

The Division remains an active member of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) as it is now known[3]. Unlike many other African member societies which register only a fraction of their membership with the ISSMGE, all members of the Division are automatically individual members of the ISSMGE. As a result South African membership of the International Society exceeds the combined membership from the rest of Africa. Every four years, the ISSMGE hosts an international conference and regional conferences in each of its six regions. The international conferences, which attract up to 3 000 delegates, are a fitting platform for leading geotechnicians the world over to present the latest research findings or case studies. The regional conferences are smaller and tend to have a more regional and practical flavour. The International Society hosts about 40 technical committees dealing with a wide range of issues including education, ethics, research, design, codes of practice and construction. South Africa is represented on about a third of these committees and has provided at least two chairmen in recent years (Blight and Day). In addition, five South Africans (Jennings, Kantey, Wilson, Donaldson and Day) have served as regional vice presidents of the Society.

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2 Received by the Candidate in 2005 and 1990 respectively.

3 In 1987, the name of the society changed from the International Society of Soil Mechanics and Foundation Engineering (ISSMFE) to the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE).

Geotechnical consultants are often affiliated to CESA (Consulting Engineers South Africa). There are currently more than 300 consulting practices (including branch offices of larger companies) on the Association's list of geotechnical consultants.

The Engineering Council of South Africa (ECSA) is the registering authority for all engineers, technologists and technicians in South Africa. At present, registration in any of the aforementioned categories does not differentiate between disciplines (electrical, mechanical, civil, etc.) let alone between specialities in these disciplines. However, the pending publication of the Regulations to the Engineering Professions Act dealing with identification of engineering work and the Codes of Practice on Structural and Geotechnical Engineering (ECSA, 2010) will go a long way to ensuring that engineering work is performed by suitably qualified persons. The Candidate was the principal author of the draft Geotechnical Engineering Code of Practice.

1.3.2 The Consulting Environment

In his introduction to the Geotechnical Division's Problems Soils[4] Conference in September 1985, Donaldson pointed out that, shortly after the Second World War, the National Building Research Institute of the CSIR employed five geotechnical engineers. This represented at least half of the trained manpower in this field in the country at the time (Donaldson, 1985). The NBRI's policy was to investigate a problem, find a practical solution, introduce the solution into practice, assist with its commercial application and then to withdraw from the scene. In this way, they provided support to the growing geotechnical engineering profession in South Africa. Many of the early employees of the NBRI moved into private practice. These included names like Kantey, Edwards, Van Niekerk, Collins, Brink and others who left to start consulting practices, many of which bore their names.

In the 1950's and 1960's, a number of the larger consulting companies opened geotechnical departments and a few specialist geotechnical consultancies came into being. Today, as indicated earlier, there are over 300 firms of consultants or branches of firms in the various centres that have geotechnical expertise. These vary from specialist geotechnical consultancies with between 2 and 20 geotechnical engineers on their staff to large multi-national practices. Most of the bigger companies have diversified from geotechnical engineering into related fields of mining, waste management, ground water and environmental studies. In addition, many larger consulting practices maintain a core of geotechnical engineers to service their in-house requirements.

One of the hallmarks of the industry has been a spirit of cooperation and sharing of knowledge between geotechnical engineers. This is probably due to two factors. Firstly, it is the legacy left to us by the great pioneers of geotechnical engineering in South Africa whose desire to share information and advance knowledge was paramount. Secondly, it is probably due to the scarcity of geotechnical engineering skills in the country fostering a spirit of cooperation rather than competition between the various geotechnical practices. This, together with the good collaboration between academics, consultants and contractors, has contributed in no small measure to the success of the Geotechnical Division and cooperation between its members.

[4] Problem soils include expansive clays, collapsible sands, dolomite residuum, dispersive soils and soft clays. All these soils can give rise to problems with the performance of foundations or of geotechnical structures such as slopes, dams, etc.

1.3.3 Geotechnical Contractors

In much the same way as geotechnical consultants form part of larger companies or practice independently, geotechnical contractors ply their trade either as specialist contractors or as the geotechnical department of one of the larger contracting firms. The specialist geotechnical contractors vary in size from companies providing a limited scope of services to multi billion rand companies listed on the Johannesburg Stock Exchange. One of the oldest piling companies in the country, McLaren & Eger, was founded in 1928 (Davis, 2006) followed by Frankipile in 1946. Both have now been assimilated into other companies.

The most successful of the geotechnical contractors have developed techniques which are particularly suited to geotechnical conditions in South Africa, a country where vast areas of land are underlain by a thick profile of transported and residual soils above the water table. These partially saturated soils, which are generally more forgiving than their saturated counterparts, have favoured the use of large diameter augered piles, the construction of deep basement excavations, the development of soft ground anchoring technology and the introduction of soil nails for lateral support. The use of innovative techniques has also been facilitated by the loosely regulated nature of the geotechnical industry in the sense of it not being bound by prescriptive standards, codes of practice or legislation. The extent to which such innovation will be stifled by the increasing emphasis on workplace and construction safety remains to be seen. Safety legislation has already had an effect on the way in which geotechnical investigations are carried out (Day, 1996 and 2006) with the emphasis shifting from in situ profiling of excavations to rotary core drilling.

As Donaldson remarked in 1985, the past 35 years have shown that with excellent cooperation between universities, consulting engineers, state bodies, contractors and research establishments, the profession has devoted detailed attention to special domestic problem areas in which South Africans have become world leaders.

1.3.4 Academic Institutions

For many years now, degrees in civil engineering have been offered by the six main universities in the country; Cape Town, Stellenbosch, KwaZulu-Natal, Johannesburg, Witwatersrand and Pretoria. All of these offer geotechnical engineering as a part of the civil engineering curriculum. The strength of the geotechnical department at each of these institutions varies with the staff employed at the time. In their day, many of the major universities enjoyed recognition for their geotechnical contributions. However, since the retirement of the “old guard”, it is only those universities that have succeeded in attracting the right calibre of senior academic staff that are still at the forefront.

Probably the most significant change in the academic environment is the emergence of technical universities and granting degrees in Engineering Technology. According to ECSA’s web site, there are 10 academic institutions in South Africa offering degrees in engineering technology, two of them specifically *“Civil: Geotechnical”* degrees.

There are also ten accredited institutions offering National Diplomas in Civil Engineering.

In the Candidate’s opinion, some of the leading academic institutions have done themselves a disfavour by admitting students from related disciplines (such as mining) from the previous “rural” universities to their post graduate programmes in engineering. This has resulted in these candidates obtaining a degree in engineering which entitles them to registration as professional engineers when they do not have the basic engineering competencies required for such a degree or registration.

1.4 Recognition of Expertise in Academia and in Practice

In the academic environment, there is a well-established system for the recognition of excellence and higher learning. The Bachelor's equips the graduate with broad training in engineering and is a requirement for professional registration. The Master's degree places the emphasis on advanced application of engineering sciences in design or on engineering research. The PhD (Doctor of Philosophy) degree requires a candidate to generate new engineering knowledge through original research. These higher degrees open the door to advancement in an academic career or equip the individual with the skills required to become an innovator or leader in industry.

In engineering practice, there is also a clear career path which moves through the stages of engineering education, in-service training as a Candidate Engineer followed by professional registration. Registration can be achieved as soon as three years after graduation. Beyond registration, however, there is no clear recognition of achievement, apart perhaps from awards that are made from time to time by professional bodies or the granting of a fellowship, or even an honorary fellowship, by such institutions.

In recent years, there has been considerable debate about the recognition of competence and many of our national standards refer to a "competent person". Some sub-disciplines in the civil engineering profession, notably the structural engineers, do not see professional registration as a measure of competence. They have been agitating for a register of competent persons which would be based either on a peer review system or on a professional examination by a recognised body such as the Institution of Structural Engineers. The International Society of Soil Mechanics and Geotechnical Engineering has also considered the compilation of a list of recognised competent professionals and has elected not to do so. Similarly, the geotechnical engineering fraternity in South Africa as represented by the Geotechnical Division has elected not to go down this road. Nevertheless, there is still a need for recognising those who qualify as experts in the profession.

Recently, some progress has been made in this regard with the drafting of an ECSA code of practice for geotechnical engineering, a process in which the Candidate was closely involved. The proposal was to establish four levels of competency for Geotechnical Practitioners (see Figure 54 in Section 14). Level 1 is a candidate engineer prior to professional registration with ECSA. Level 2 is a registered geo-professional with up to 5 years' experience. Level 3 is an experienced geo-professional. Any registered professional engineer or engineering technologist with more than 5 years' experience can achieve this status. Level 4 is referred to as an expert geo-professional with a minimum of 10 years' experience. However, there is a recognition that not all professionals will achieve this status as experience alone is not sufficient. Thus, two additional requirements were introduced namely that a Level 4 professional should:

i. enjoy recognition by the profession as a specialist geo-practitioner, possessing a level of specialist knowledge and experience above that expected of the profession, and
ii. be making a contribution to the state-of-practice of geotechnical engineering by the application of advanced techniques or by means of research, publications or involvement in engineering education.

Although the ECSA codes of conduct have become bogged down in the system, the Candidate was instrumental in having these requirements introduced into SANS 1936 for development on dolomite land. In the case of SANS 1936, there is a requirement for expert input into development on D4 dolomite land (where normal precautionary measures alone are inadequate) and for the review of such solutions by a similarly qualified, independent expert.

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As indicated in ii. above, an expert within the profession is expected to make a contribution to the application of advanced techniques by means of research, publications or involvement in engineering education. In each of these three aspects (research, publications, education), the activities of the practitioner may differ from those of the academic. The research activities[5] of the practitioner may not follow the same rigorous three-fold process of postulation, investigation and verification as applied in academic research. The practitioner's research may take the form of the identification of new materials, methods or procedures, trial implementation and assessment of the outcome. Similarly, the practitioner's publications may not be in peer-reviewed journals as is the preference in academia but could be in more popular journals, conferences and symposia likely to encourage the uptake of any new ideas by the rest of the industry. Their involvement in education may not be in teaching the rudiments of engineering science but rather in giving those whom they teach an insight into the practical application of the theory and sharing with them the excitement of meeting the challenges of the industry. Such teaching is probably better suited to the continuing professional development or post-graduate environment than to the undergraduate classroom.

Finally, on this subject, the ultimate form of publication for the practitioner (and for some academics too) is the publication of codes of practice or national standards. These require an in-depth knowledge of the subject and experience in its practical application. Not only are these documents scrutinised by the profession and public alike prior to publication, they are subject to continued peer review throughout their life. Furthermore, in the South African context where standard writing is a voluntary activity, involvement in this process is an indication of the individual's willingness (and that of his or her employer) to put something back into the industry.

1.5 Creating Opportunities

The geotechnical industry, both world-wide and particularly in South Africa, presents significant opportunities for enterprising individuals to contribute to the advancement of the profession.

Having been nurtured in the fertile environment described above and having had the advantage of generous mentorship by senior members of the profession, the Candidate has found himself in a position to make various contributions to the profession during his 35 years of practice as a geotechnical engineer. In this, he has been favoured by working for a company and with colleagues who share the conviction that knowledge should be shared with the rest of the profession and that anything that is given away will be returned with interest.

This has led to a recurring theme within the Candidate's professional life. It starts with the identification of a problem which requires investigation, the introduction of new techniques or simply a drawing together of available information. This is followed by investigation of the problem and development of a solution. If there is a significant contribution to be made, the final step is to share the fruits of this process with the profession by means of published papers, presentations at seminars and conferences or the development of standards or codes of practice.

The remainder of this dissertation describes some of the fields in which the Candidate has been involved and has tried to make a meaningful contribution. It seeks to demonstrate that original research and creative work is not limited to those who pursue an academic career, but can also be undertaken by professionals in engineering practice. This is in

[5] Research being defined in this case as a diligent and systematic inquiry or investigation into a subject in order to discover or revise facts, theories, applications (on-line dictionary.reference.com).

spite of the fact that the conditions for contributing to the general body of knowledge are not optimal.

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Part 2: Miscellaneous Contributions

This Part of the dissertation presents a number of areas where the Candidate has identified a need for more information on a particular topic, has carried out the necessary research / investigations and has then shared his findings with the profession.

It concludes with a brief look at the Candidate's involvement with geotechnical engineering on an international level.

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2. DEVELOPMENT ON DOLOMITES

2.1 Background

2.1.1 The Quest for Gold

Many of the richest gold deposits on the Far West Rand are overlain by dolomite, a carbonate rock which is prone to dissolution. Leaching of the rock leads to the formation of solution-widened joints (or grykes) and interconnected cavities. In 1910, an attempt was made to sink a shaft through the dolomites at Venterspost Gold Mine on the Far West Rand. So great was the flow of water and mud into the shaft through the interconnected network of joints and cavities that the project had to be abandoned (Wagener, 1985). It was only in 1937 that the first shaft was successfully sunk in this area using a combination of dewatering and cementation to stem the flow of water.

The dolomite on the Far West Rand is divided into a series of compartments bounded by watertight dykes of intrusive rock (syenite or diabase). Over the years, a number of these compartments have been dewatered to enable shafts to be sunk to the underlying gold bearing quartzites of the Witwatersrand Supergroup. This dewatering has had significant consequences. In 1962, the crushing plant at the West Driefontein Mine disappeared without warning into a 55m diameter sinkhole resulting in the death of 29 employees. In 1963, a doline developed at Lupin Place in Carletonville where 22 houses were affected by a large scale settlement of up to 5m (*ibid*). These sinkholes and dolines were the direct result of dewatering of the dolomitic formation to facilitate mining operations.

Expansion of the gold mines into the Stilfontein and Orkney areas also encountered dolomite. However, little or no dewatering has taken place in these areas.

2.1.2 Urban Development

Van Schalkwyk (1981) estimates that 14 percent of the densely populated and highly industrialised PWV area is underlain by dolomites. This includes areas like the southern parts of Pretoria, Tembisa, Carletonville, Orkney, Stilfontein, Katlehong, the south-western suburbs of Soweto, Lenasia, the Klip River Valley, parts of Springs and Delmas.

The demand for residential land in close proximity to the major centres has led to increased development of these areas, sometimes with serious consequences (Wagener, 1982). On 3rd August 1964, a 60m diameter sinkhole in Blyvooruitzicht swallowed four houses and a family of five. In October 1970, a sinkhole at the Venterspost tennis club engulfed part of the clubhouse and a spectator. In more recent years, numerous sinkholes have occurred in the Centurion area of Pretoria, most as a direct result of the urban development.

In 1937, sinkholes appeared below the Pretoria-Germiston railway line and the south abutment of the Fountains railway viaduct subsided in 1938 (Jennings, 1965). Sinkholes continued to form along this section of the railway line until well into the 1980s. On many occasions, sinkholes have led to the closure or temporary deviation of roads including the Ben Schoeman (N14) highway south of Pretoria.

Photo 1: Small sinkhole in residential complex in Centurion, Pretoria (2002)

2.1.3 Types and Causes of Subsidence

By the early 1960's, Jennings and his co-workers had already identified and distinguished between two forms of subsidence on the dolomite namely sinkholes and compaction subsidences (referred to in South Africa as dolines) and had described the mechanism by which these occur.

As indicated earlier, dolomites are carbonate rocks which are subject to dissolution by acidic ground water. The acid responsible for dissolving the carbonates in the dolomite rock is principally carbonic acid, which may be present in very small concentrations in the ground water (Jennings, 1965). As rain falls through the atmosphere, it absorbs some carbon dioxide. More carbon dioxide is dissolved as the water percolates through the root zone of the soil. When carbon dioxide dissolves in water it exists in chemical equilibrium producing carbonic acid (Greenwood and Earnshaw, 1997):

$$CO_2 + H_2O \rightleftharpoons H_2CO_3$$

At atmospheric pressure, most of the carbon is in the form of CO_2 resulting in a slightly acidic water (pH ~ 5,7).

The dissolution of the rock leads to the formation of solution widened joints producing a typically pinnacled rockhead topography (see Photo 2). Figure 1 (Wagener and Day, 1984) shows a typical dolomitic profile in which the pinnacled rockhead is overlain by dolomite residuum including a wad and chert gravels. Cavities below the water table, which represents the base level for subterranean erosion, are generally considered to be stable. However, lowering of the water table or the ingress of the surface water can lead to collapse of the soil arch over the cavity. If this collapse extends to the ground surface, a sinkhole is formed. In general, the deeper the soil profile above the rockhead (termed the "potential development space" by Buttrick et al, 2001), the larger the resulting sinkholes. Obviously, this mechanism relies on the presence of subterranean voids that are big enough to receive the material eroded from above and the mobilisation potential of the overburden.

Sinkholes are not necessarily the product of development or groundwater lowering. Sinkholes occurred long before the influence of man. The first published mention of sinkholes in South Africa was by Penning who records that, during the South African War, Colonel Deneys Reitz hid his whole commando from the British in a large sinkhole in the hills behind what is now the Doornfontein Mine (Jennings, 1965).

By contrast, compaction subsidences or dolines are the direct result of ground water lowering. One of the products of the decomposition of dolomite is wad, a light weight and compressible, manganese-rich residuum derived from the leaching of dolomite. While the wad remains saturated below the water table, the overburden pressure is shared between the stresses within the soil skeleton and the water pressure within the pores of the soil. If the water table is lowered, the pore water pressure decreases and the load is transferred to the soil skeleton causing it to compress. Where the thickness of wad is significant and the ground water is lowered considerably, many metres of settlements can occur as in the Lupin Place case referred to earlier.

Photo 2: Pinnacled dolomite rockhead (Dolomite Mine, Lyttleton, Pretoria, 1979)

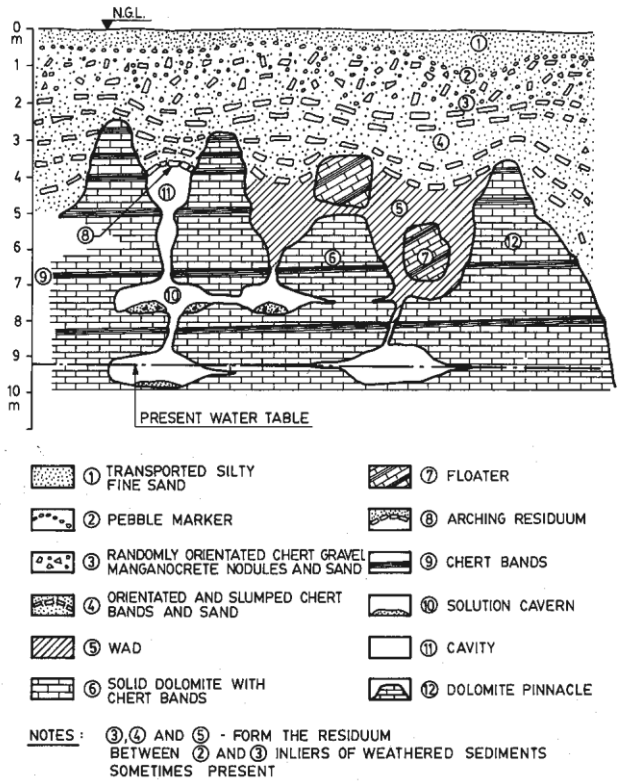

Figure 1: Typical profile on shallow dolomite (Wagener & Day, 1984)

2.2 Investigation Techniques on Dolomite.

2.2.1 Background

During the 1960's and extending well into the 1970's, much of the research on dolomite concentrated on areas of deep rockhead, particularly those affected by dewatering. However, considerable development was also taking place in areas of shallow rockhead (0 - 15m, or class A and B dolomite sites as defined by Wagener, 1982). Two examples of such development were the expansion of the gold mining activities in the Orkney area of the (then) South Western Transvaal and residential development in the suburb of Rooihuiskraal south of Pretoria. These areas of shallow rockhead provided an ideal opportunity to evaluate various methods of investigation and to compare the results with the soil conditions exposed in excavations on the site.

In November 1981, the Department of Geology at Pretoria University organised a seminar on the Engineering Geology of Dolomitic Areas. The session on investigation techniques contained five seemingly unrelated papers dealing with a historical overview, geophysical methods (principally gravity surveys), the use of telescopic benchmarks, seismic surveys and remote sensing. There was, however, no consensus regarding the efficacy of these investigation techniques.

2.2.2 Day and Wagener, 1981

The relatively shallow rockhead in the Orkney area, where the subsoil conditions inferred from the investigation could be verified in deep excavations formed during construction, provided an ideal opportunity for determining the efficacy of the various investigation

techniques. After spending many hundreds of man-hours profiling test pits, supervising the drilling of percussion boreholes and observing the conditions exposed on site during construction, Day and Wagener (1981) published a paper in which various investigation techniques on dolomites were compared and discussed. The paper appeared in the somewhat informal newsletter of the Geotechnical Division known at that time as Ground Profile. It set out the principle objectives of an investigation on dolomites, namely to establish the properties and thickness of the overburden, the condition and depth of the bedrock and the level and permanence of the ground water table. Armed with this information, it was usually possible to assess the magnitude of normal settlement and the risk of sinkhole or doline formation.

The paper concentrated on the investigation techniques which had proved most successful in the area and tentatively sub-divided these into “quantitative” and “qualitative” techniques. Quantitative methods were those which gave a numerical value representing some characteristic of the site such as depth to rock, consistency of the overburden, size of cavities, etc. Examples include percussion drilling, test pitting, gravity surveys, penetration testing, etc. Qualitative methods, on the other hand, included photo interpretation and thermal scanning which provide information from which ground conditions may be inferred.

Method	Measurement or observation	Measured or inferred material property
Quantitative Methods (methods that include physical measurements)		
Percussion drilling	Penetration rate Hammer tempo Air loss and sample return Chip samples Water strikes	Consistency, depth to rock Continuity and consistency Presence of voids / porous conditions Type of material Water table depth and yield
Backactor trenching	Visual assessment / profiling Depth of refusal Ground water	Nature and consistency of overburden Rock depth and topography Water table depth
Gravity survey	Gravitational attraction	Depth to rockhead
Qualitative Methods (no physical measurements)		
Photo interpretation	Topography, vegetation and surface texture from stereo aerial photos	Landforms, soil zone boundaries, outcrops, lineations (faults / fracture zones / dykes)
Thermal line scanning	Ground surface temperature	Indicative of soil type, moisture variations and shallow rock

Table 1: Information from investigation methods on dolomite (inferred from Day, 1981)

A case history was presented on a recently completed investigation for a 120ha residential area near Orkney where various investigation techniques were applied. One of the conclusions was that, although there is a definite correlation between the depth to bedrock measured in the percussion boreholes and that inferred from the gravity survey, there was still considerable scatter. This is hardly surprising given that the gravity survey provides an averaged measure of the depth to rock whereas percussion drilling provides point information on the depth to a highly undulating rockhead. A reasonable correlation was also noted between the results of the thermal line scanning (aerial infrared imagery) and gravity low trends (zones of deeper rockhead).

In a further case study, two of the seemingly established criteria of the day were put to the test. These were (i) that areas of steep gravity gradient have a high risk of sinkhole development and (ii) that 15m of overburden above the rockhead should be sufficient to prevent sinkhole formation. In this case study, the position of a number of sinkholes in a mine reduction works that had formed over the preceding 10 years were superimposed on the residual gravity map. It was found that sinkholes occurred in gravity highs or lows and on steep or flat gradients, and that they also occurred in areas where the depth of overburden exceeded 15m. In this case, it appeared that the position of the sinkholes correlated better with regional trend lines identified by photo interpretation than with any features of the gravity survey.

In the discussion, it was pointed out that there were as yet no firmly established criteria for the interpretation of the results of the geotechnical investigation on dolomites. Unlike classical soil mechanics, where the geotechnical engineer knows which tests to perform and how to interpret the results, there are so many uncertainties and factors that influence the stability of a dolomitic site that one is justified in carrying out a wide variety of tests even when the interpretation of these tests is uncertain. This was seen as the only way of eventually establishing valid criteria for stability assessments on dolomitic sites. The paper also cautioned against blindly accepting a seemingly established criterion for data interpretation and extrapolating these criteria from one area to another where conditions may be totally different.

2.2.3 Wagener, 1982

In his doctoral thesis, Wagener (1982) elaborated further on geotechnical investigation methods. In addition to covering the techniques referred to in the earlier paper, he provided further information on the use of rotary core drilling, large diameter auger holes and in-situ testing. He used the same two case histories to illustrate the correlation between the various methods of investigation.

In conclusion of his chapter on site investigation on dolomites, Wagener proposed a three-fold classification of sites based on the depth to the dolomite rockhead, namely Class A (0-3m), Class B (3-15m) and Class C (>15m). He then went on to provide some practical recommendations for investigation techniques appropriate to these site classes. He too cautioned against laying down fixed rules for the evaluation of site conditions and the extrapolation of these rules from one area to another urging investigators to gather as much geological and hydrological information on the site as possible and to interpret this with an open mind.

2.2.4 First Multi-disciplinary Conference on Sinkholes

In 1984, the first Multi-disciplinary Conference on Sinkholes was held in Orlando, Florida, (USA). At this conference, which brought together people from various disciplines including geologists, engineers, geophysicists and geohydrologists, South Africa's

prominence was enhanced by the invitation to Dr Tony Brink to present the keynote lecture on the formation and weathering of dolomites. His presentation was based both on his vast field observations and in his conviction that "the present is the key to the past" as far as geology is concerned.

At this conference, Day and Wagener (1984) presented a more formal version of their 1981 paper. This presentation stressed the two-fold purpose of investigation on dolomites, namely, (i) to evaluate the risk of subsidence and give recommendations for managing this risk and (ii) to provide the parameters required for economical design of foundations and infrastructure. The latter is common to most site investigations whilst the former is unique to sites underlain by dolomites.

The paper classified investigation methods into three categories, namely remote sensing (aerial photo interpretation, thermal line scanning and satellite imagery), geophysical methods (gravity survey, seismic refraction, resistivity, etc.) and direct methods (rotary core and percussion drilling, penetration testing, test pits, etc.). Following on from the work of Wagener (1982), the paper outlined the typical approach which would be adopted for investigations on dolomite in South Africa. It was stressed that the engineer or geologist responsible for making founding recommendations should be intimately involved in the fieldwork and that open-minded consideration should be given to all available information rather than concentrating on one set of results in isolation.

Two case histories were presented. The first was the township development case history from the 1981 paper which was enhanced by the inclusion of the positions of five sinkholes which occurred during the intervening period. Neither of these were within areas released for development. The second case history was the investigation of a bridge over the Vaal River which demonstrated the application of the various techniques of investigation available at the time.

2.3 Properties of Wad

2.3.1 Background

In addition to calcium and magnesium carbonate, dolomite also contains minor quantities of iron and manganese. In particular, the Oaktree Formation at the base of the Malmani[6] dolomites has a higher percentage of manganese than most of the overlying formations. This formation can contain in excess of 1% manganese whereas the underlying Black Reef Formation (predominantly quartzites) may contain up to 3% manganese (Brink, 1979).

Leaching of the calcium and magnesium carbonates by the passage of slightly acidic ground water leaves behind insoluble residues in the form of chert (predominantly silica dioxide), wad (or manganese earth) and iron oxide. The chert accumulates as angular slab-like gravels and slumped chert bands, generally with an infilling of transported sands from above. The iron oxide (in the form of hematite) is easily erodible and is often found in fissures and caverns into which it has been transported by moving ground water.

The manganese liberated by the weathering of dolomite remains behind in what Brink describes as "*an insoluble and highly compressible material ..... known as wad or manganiferous earth*" (Brink, 1979). The material is dark purple or black in colour and, in many instances, its dry density is so low that it will float on water. Large quantities of wad are present in the Oaktree Formation and the underlying Black Reef Formation. Wad also tends to accumulate at the contact between dolomite and other rock types such as syenite

[6] The dolomites that occur in Gauteng belong to the Malmani Sub-group, Chuniespoort Group, Transvaal Sequence.

dykes or sills. Brink continues to describe wad as the "*most highly compressible residual soil known to occur on the Highveld*".

2.3.2 *Properties of Wad (Day, 1981)*

In the early 1980's, residential development to the south west of Pretoria reached the suburb of Rooihuiskraal which is situated on the Oaktree Formation, immediately north of the contact between the dolomites and the underlying Black Reef quartzite. The area is also extensively intruded by syenite. Not surprisingly, significant quantities of wad were encountered during the installation of services. This raised the concern of the developers. Rather than abandoning the development, they decided to commission a thorough investigation into the properties of the wad, its founding characteristics in particular. The information gained during this investigation was published (Day, 1981a) at a seminar organised by the Geology Department of the University of Pretoria held in November 1981.

During the investigation, a practical approach was adopted. This involved mapping all the service trenches to determine the distribution of the material and to observe its properties in the field. Thereafter in-situ testing was done to determine the bearing capacity and compressibility of the material by means of plate load tests. These tests were conducted both at natural moisture content and after saturating the wad to ascertain whether its properties would change should it become wet.

During the mapping of the trenches, two forms of wad were observed. The first, referred to as an "intact wad", was a relatively homogeneous, dark coloured (purple to black) material with randomly orientated, iron-stained joints. The second was a material which had been entirely reworked and appeared either in the form of fine blocks of desiccated wad or a dark coloured powdery soil. Tests were carried out on both the intact and reworked forms of the material.

Photo 3: Fragments of intact wad from Rooihuiskraal, Pretoria (1980)

The tests confirmed the low density of the wad. The dry density was found to vary from 1 327kg/m^3 to as low as 225kg/m^3 with an average density of 670kg/m^3. The highest

densities were observed where the wad was mixed with an appreciable quantity of sandy (probably transported) material. When broken down, the grading of the material was generally a clayey silt or a silty clay. The high liquid limit (61% - 125%) and modest PI (14% - 27%) are characteristic of silty materials.

As predicted by Brink, the reworked or powdery wad proved to be highly compressible with an elastic modulus (as recorded by the plate load test) varying from 0,8MPa to 12MPa with an average of 5MPa. Furthermore, the material displayed a reduction in stiffness when saturated under load. Apart from the most compressible of the material, these results are not significantly different to that of a typical wind-blown sand on the Highveld which also exhibits a collapsible grain structure.

By contrast, the intact wad showed significantly better properties, both in the field and in the laboratory. In the field, the elastic modulus varied from 8MPa to 46MPa with an average of 24MPa, much the range one would expect from a firm silt. The angles of shearing resistance from triaxial and shear box tests were typically between 17° and 24° with cohesion intercepts of between 30kPa and 50kPa on average. Apart from a more cohesive appearance than the reworked wad, a characteristic of the intact wad was that it generally had a high moisture content. Its appearance in the field can best be described as being similar to the flesh of a ripe watermelon which is firm to the touch but, if struck by a blow (e.g. from a geological pick) or subject to high pressures, would expel water from its fabric and reduce significantly in both strength and stiffness.

Property		Min	Ave	Max	No. of Tests
INDEX PROPERTIES ALL SAMPLES	Dry density	225	666	1327	15
	LL (%)	61	83	125	15
	PI (%)	14	21	27	15
	Linear shrinkage (%)	5	9	12	15
	% Clay	17	29	56	15
	% Silt	32	49	67	15
	% Sand	3	22	39	15
COMPRESSIBILITY INTACT WAD	1/m_v (Oedometer) (MPa)	0,7	21	57	15
	E (Plate Load) (MPa)	No Collapse			
		7,5	24	46	5
REWORKED WAD	E (Plate Load) (MPa)	0,8	5	12	3
		Severe Collapse			
SHEAR STRENGTH INTACT WAD	ϕ' Triaxial (deg)	17	22	24	6
	c' Triaxial (kPa)	23	55	75	6
	ϕ Triaxial (deg)	17	18	19	6
	c Triaxial (kPa)	30	59	81	6
	ϕ Shear Box (deg)	27	29	34	5
	c Shear Box (kPa)	0	33	57	5

Table 2: Properties of Wad (Day, 1981a)

In addition to the physical tests undertaken on the wad both in the field and in the laboratory, samples of dolomite and of wad were inspected through an electron microscope at the CSIR. The fresh dolomite, even when magnified 3 000 times, showed a very angular structure with distinct stepped, planar cleavage. The wad, on the other hand, had a more amorphous appearance, described by Wagener as being similar to that

of the breakfast cereal “Rice Crispies”. Even at 3 000x magnification, the individual grains do not appear to be solid particles but rather exhibit a “fluffy” or “cloud-like” appearance.

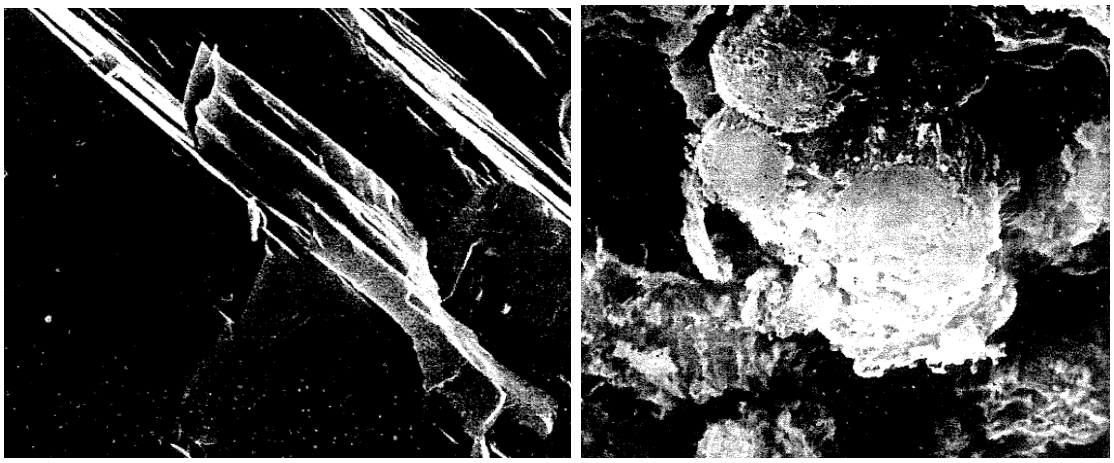

Photo 4: 3 000x images of (a) dolomite and (b) wad (Day, 1981a)

On the strength of the investigation, it was decided that all stands that were underlain by intact wad and possessed sufficient soil cover to ensure that the wad was not exposed during normal construction activities would be released for sale to the public with no restrictions. The stands were the wad came close to surface or where a reworked wad was present were held back and houses on these stands were built by the developer. These houses were founded on light, reinforced concrete raft foundations (slab-on-grade) placed on a 0,8m – 1,0m thick mattress of compacted granular fill material imported from elsewhere on the site. The intention of this design was to distribute the loads exerted by the walls of the houses to the underlying in-situ soils as widely as possible thereby ensuring that the applied pressure would be insufficient to cause significant settlement. It is now over 30 years since development of the township was completed and no reports of distress have been received from this area.

The paper cautioned that even intact wad would lose strength if subjected to impact loading. Thus, densification of the wad by vibratory or impact compaction would be likely to worsen the situation by destroying the structure of the material. Similarly, the wad would be unlikely to form an acceptable founding stratum for piles with hammer compacted bases. This latter contention was subsequently proved to be correct where percussion bored piles installed into the wad at Scaw Metals in Germiston had to be driven to considerable depth before their full capacity could be realised.

2.4 Engineering Construction on Dolomites

2.4.1 Background

The two essential requirements to be met by the design of any foundation are ensuring that the foundation has the capacity to support the proposed structure and that the settlement of the foundation will be within acceptable limits. With regard to the latter requirement, there are three types of settlement to be considered on dolomitic terrain (Wagener 1983): (a) normal settlement due to immediate elastic settlement and consolidation settlement, (b) sudden subsidence settlement in the form of sinkholes and (c) gradual subsidence due to the formation of dolines. Appropriate foundation design can go a long way to addressing the first two of these issues.

Apart from the risk of subsidence, there are three particular challenges to be overcome in the design of foundations on dolomitic ground. These are:

Reduction in consistency of overburden with depth: On dolomitic sites, the upper layers of the profile typically comprise partially saturated, transported materials which may be lightly cemented. Voids between any larger particles, such as transported chert gravels, are well filled with a sandy matrix. However, with depth, particularly on the chert-rich dolomite formations, one encounters slabs of chert and slumped chert bands often in a sparse sandy matrix. Pockets of wad or cavities may be present, particularly towards the base of the residual profile and between pinnacles. The result is a reduction in consistency of the residuum with depth with a corresponding decrease in stiffness and bearing capacity.

Presence of obstructions in residuum: As shown in Figure 1, the dolomite residuum frequently contains slabs of chert or dolomite floaters. Even in their weathered state, both are very hard materials and can pose significant obstacles to the installation of deep foundations such as piles, small diameter shafts, caissons or diaphragms walls.

Undulating rockhead: On dolomites, the rockhead is typically marked by an abrupt transition from relatively incompetent overburden to very hard or extremely hard rock which requires blasting for excavation. This, coupled with the highly undulating, typically pinnacled rockhead topography makes it difficult to achieve uniform founding conditions on rock over the full extent of the proposed foundation, particularly for deep foundations. A further complicating factor is the need to ensure that foundations have indeed penetrated to the rockhead rather than to large boulders or floaters within the residuum.

2.4.2 Dolomite Seminar, Pretoria 1981

At the dolomite seminar in 1981, a number of case histories were presented for the founding of various types of structures. Railway lines received particular attention. One proposal was to create a concrete mat reinforced with disused mine cables or to strap the railway tracks to render them capable of spanning in catenary across moderate size sinkholes.

At the symposium, Wagener (1981) produced a comprehensive summary of available founding methods and their suitability for use on the three site categories which he had proposed earlier (see Section 2.2.3). The founding methods included conventional foundations, mattresses of compacted granular fill material, founding on pinnacles, deep foundations (piles, shafts and caissons) and ground improvement (dynamic consolidation and Reinforced Earth®).

In the years leading up to the seminar, the use of mattresses constructed using well compacted, high quality fill material proved to be an efficient and economical way of founding industrial and residential structures, particularly in the non-dewatered areas around Orkney. Apart from the economic aspect, Day (1981b) cited three main reasons why this method of construction is used:

- to control both total and differential settlement;
- to limit the bearing pressures on the underlying compressible material; and
- to reduce the risk of sinkhole formation by limiting water ingress.

A mattress is generally constructed by excavating the natural material to a prescribed depth followed by compaction of the base of the excavation. The required thickness of selected fill (usually mine waste rock or chert gravel in the Orkney area) is then placed in layers and compacted to the specified density. The structures are then founded on top of this mattress usually on a raft foundation or on shallow spread footings.

In the paper on dumprock and chert gravel mattresses, Day (1981b) provided numerical examples of the extent to which a well-constructed mattress is capable of controlling total

and differential settlement and reducing the bearing pressure on the dolomite residuum below the mattress. These examples were based on the results of linear elastic finite element analyses using the geometry and material properties shown in left hand diagram in Figure 2. The validity of the analysis was limited in that the finite element package available at the time could only analyse axisymmetric or two-dimensional (plain strain) problems whereas accurate modelling of the dolomite rockhead requires three dimensional analysis. As such, the results are indicative only. In the case of differential settlement, he illustrated the point with an example of a 4m wide strip footing founded on a pinnacle dolomite site with an average depth to the top of the pinnacles of two metres as shown in Figure 2. The results showed that even a 1m thick mattress was capable of reducing both the total and differential settlements by 50%. The optimum thickness of the mattress was found to be down to the top of the pinnacles with little added benefit being obtained from extending the mattress into the grykes between pinnacles. This reduction in settlement is partly due to the superior stiffness of the fill compared to that of the residuum and partly due to the ability of the mattress to arch between the pinnacles as was demonstrated in Figure 3 (Wagener and Day, 1984) which shows the major principle stress vectors derived from the finite element analysis.

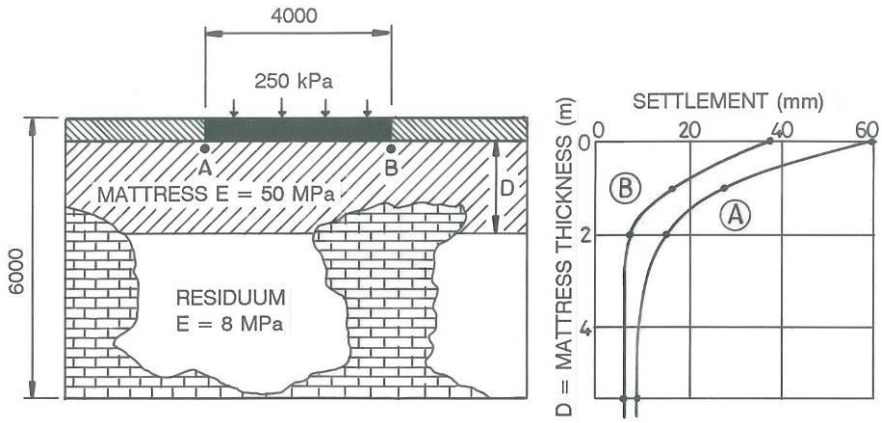

Figure 2: Effect of mattress on settlement of a foundation (Day, 1981b)

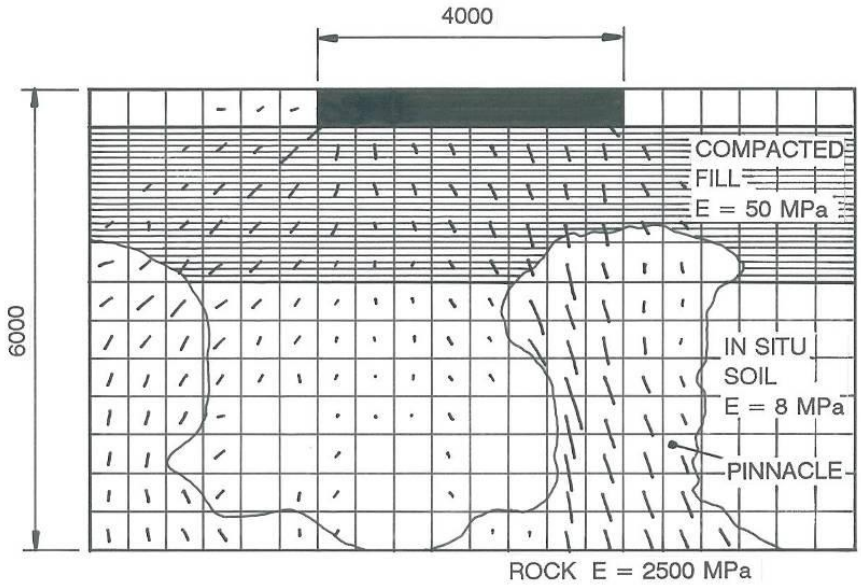

Figure 3: Arching effect of mattress (Wagener and Day, 1984).

On a deep layer of compressible soil, the same example (4m wide strip footing loaded to 250kPa) was used to illustrate the effect of varying the thickness of the mattress. The use of a mattress thickness equal to the width of the strip footing reduced the expected settlement of the foundation by half as shown in Figure 4.

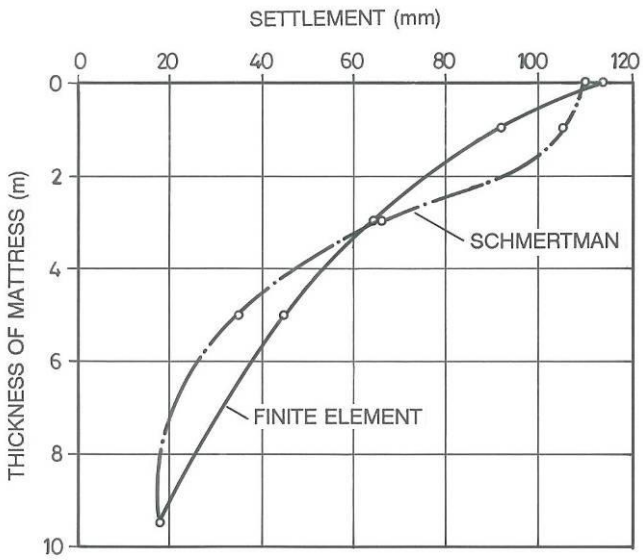

Figure 4: 4m wide strip footing on a mattress underlain thick residuum (Day, 1981b)

With regard to bearing pressure, Figure 5 shows that the effect of a mattress below either a spot footing or a strip footing is to increase the effective angle at which the foundations load was distributed to the underlying soils. With no mattress in place, a 30° load spread as recommended by Simons and Menzies (1975) gave a reasonable approximation of the peak bearing pressure at some depth below the footing as computed using a finite element analysis. With the mattress in place, the angle of load spread affectively increased to approximately 45°. These findings were also presented in the form of a non-dimensional table which related the ratio of footing width to mattress thickness to the reduction in bearing pressure at the underside of the mattress for strip and square footings as shown in Table 3. These stress distributions shown in Figure 5 were obtained from finite element analyses. As the underlying materials (mattress and residuum) were assumed to extend well beyond the width of the foundation, both problems could be adequately modelled in two dimensions. The strip footing was analysed as a plain strain problem and the spot footing as an axisymmetric problem.

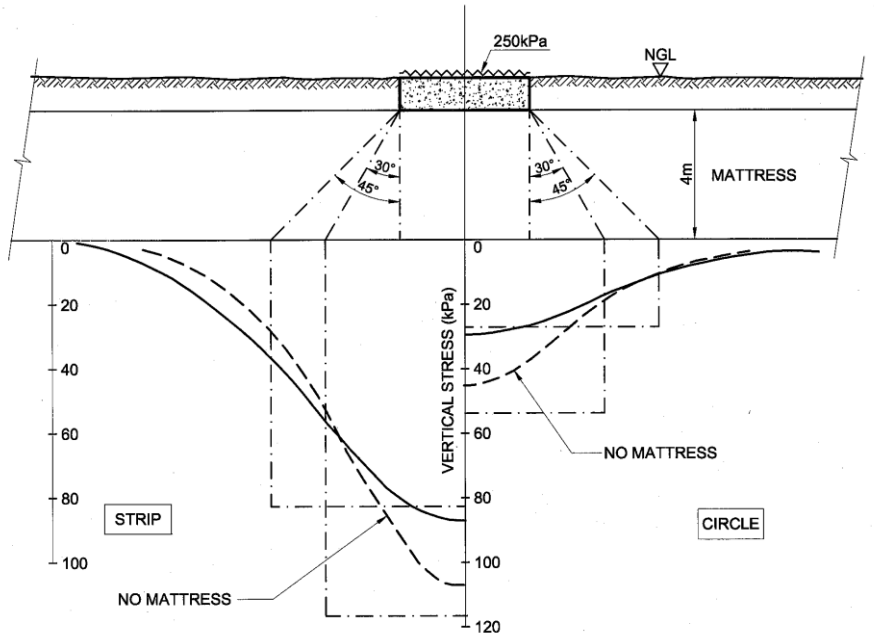

Figure 5: Bearing pressure at underside of mattress (Day, 1981b)

Table 3: Bearing pressures at underside of mattress (Day, 1981b)

Width of footing / Mattress thickness (B/D)	% of applied bearing pressure at underside of mattress	
	Strip Footing	Square Footing
8	80	64
4	67	44
2	50	25
1	33	11
0,5	20	4
0,25	11	1
0,125	6	0,3

Day (1981b) expressed the opinion that reinforcing the mattress to allow it to span over larger and larger sinkholes needs careful consideration as, the greater the span, the greater the damage will be when failure eventually occurs. It was considered preferable for the signs of subsidence to be detected at surface as soon as possible even if in the form of distress of surface structures. In this way, one is at least alerted to the existence of the potential danger and can take the necessary remedial action.

That is not to say that construction of a mattress has no benefit in reducing the risk of subsidence. The excavation of a large hole in the ground is, in itself, an excellent method of investigation. Inspection of the chert bands or detection of sandy infilling extending to a greater depth than in surrounding areas can provide evidence of geological features requiring further investigation. A further benefit of a mattress constructed using chert gravel is that the permeability of the compacted mattress is generally lower than that of the surrounding ground. This has the effect of limiting water ingress immediately below the structure and the immediate surrounds thereby reducing the risk of sinkhole formation. This benefit is partially lost when dumprock is used to form the mattress. For this reason, it is recommended that a dumprock mattress should always be capped with a minimum of 1m to 2m of compacted chert gravel to serve as a water barrier. Furthermore, the top of

the mattress should always be elevated slightly above ground level and the side slopes be graded to fall away from the structure.

2.4.3 Problems with piled foundations

The two main problems with piles foundations on dolomite have already been mentioned in Section 2.4.1. These are the presence of obstructions in the residuum and the highly undulating, pinnacle rockhead topography.

On a recent piling project in the Northern Cape, the unconfined compressive strength of the obstructions in the residuum (chert slabs and dolomite boulders) was as high as 300MPa classifying the material as an extremely hard rock (UCS > 200MPa). The underlying dolomite rock had showed UCS values as high as 460MPa. As one can imagine, there are few piling rigs capable of penetrating such obstructions. As a result, the use of piled foundations on dolomites is limited to cases where other solutions are impractical (e.g. bridge piers where deep foundations are required on account of the possibility of scour) or where the piles can be installed using percussion drilling equipment. The largest diameter down-the-hole percussion drilling equipment that is commercially available in South Africa at present is 457mm.

If larger diameter piles are required, oscillator piles are often used (Wagener and Day, 1984). This pile type uses heavy chisels to break the rock and then removes the broken rock fragments with a grab (see Photo 5).

Photo 5: Oscillator piling rig installing a raking pile – chisels in foreground

Prior to commencement of construction of the Gautrain rapid rail link between Johannesburg and Pretoria, the candidate served on a team of experts tasked with evaluating the risk of sinkholes below the sections of the track underlain by dolomite and proposing appropriate founding solutions. A question arose regarding the probability of a pile installed in the dolomite encountering the side of a pinnacle during installation. As has been the case so many times in the past, the candidate turned to Brink's *Engineering*

Geology of Southern Africa (Brink, 1979) for information and selected an example of a pinnacle layout from one of the many case histories given in Brink's books. The layout chosen was for a site in Zeerust where the tops of the pinnacles had been accurately mapped.

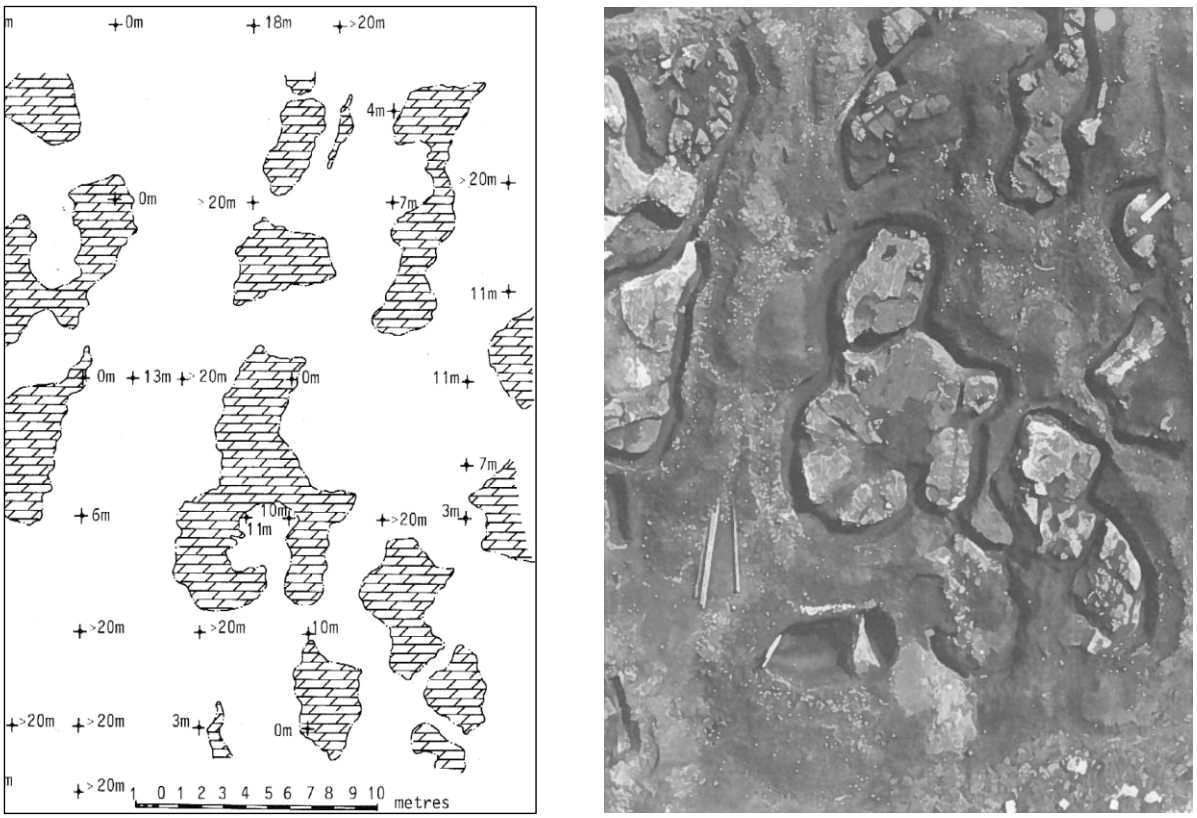

Figure 6: Mapping of tops of pinnacle on a site in Zeerust (Brink, 1979)

Using this mapping, the candidate came to the conclusion that this probability increased with increasing pile size. For the particular configuration of the pinnacles on the Zeerust site, the likelihood of a pile encountering the top of a pinnacle over its full bearing area, the side of a pinnacle or being installed in a gryke is shown in Figure 7.

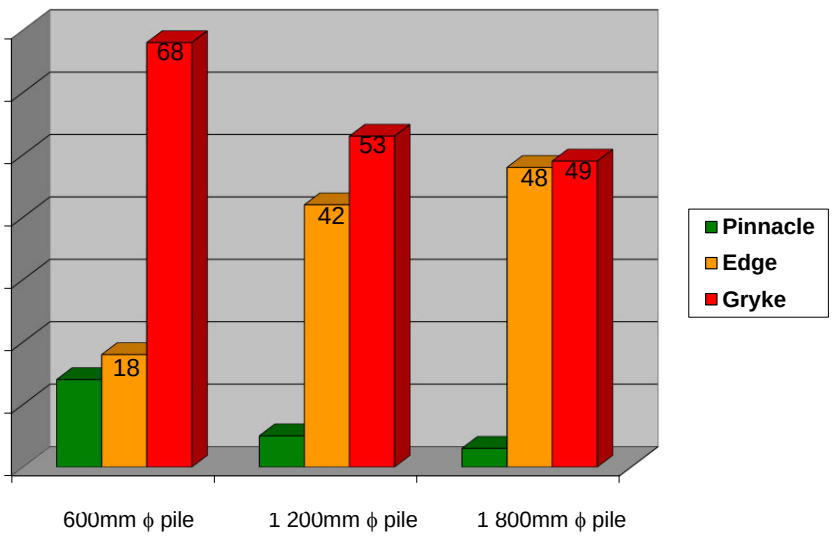

Figure 7: Probability of a pile encountering the edge of a pinnacle on Zeerust site

These figures were derived very simply by drawing the outline of the tops of the pinnacles in Autocad and then drawing two further outlines offset by half a pile diameter inside and outside the outline of the pinnacles. The area between the two offset lines divided by the total area represents the probability that a randomly placed pile on this site will intersect the side of a pinnacle. This illustrative approach is based on the assumption that the sides of the pinnacles are vertical. In reality, the grykes between pinnacles are likely to become narrower with depth. If the width of the grykes tapers gradually to zero, the probability that the pile will encounter the side of a pinnacle or the steeply sloping sidewall of a gryke is equal to the sum of the probabilities of the pile encountering the side of a pinnacle or being installed in a gryke.

2.4.4 Further Publications

The presentations by Wagener and Day at the 1981 dolomite seminar together with the information from Wagener's PhD thesis were combined into a paper entitled Construction on Dolomite in South Africa which was presented at the 1984 multi-disciplinary conference on sinkholes in Orlando Florida (Wagener and Day, 1984). An amended version of this paper was subsequently published in the Journal of Environmental Geology and Water Science in 1986 (Wagener and Day, 1986).

2.5 Subsequent Developments

2.5.1 The scenario supposition method (Buttrick et al, 2001)

Since these early days, considerable progress has been made in formulating a rational approach to risk assessment on dolomites which takes account of:

- the original elevation of the groundwater table and the potential for groundwater lowering;
- the presence of receptacles at depth within the profile to receive material eroded from above;
- mobilising agents, particularly ingress of water from leaking services;
- the potential sinkhole development space which is related to the thickness of the overburden above the rockhead;
- the nature of the blanketing layer;
- mobilization potential of the blanketing layer; and
- bedrock morphology.

The method, known as the "scenario supposition" method, was published by Buttrick et al (2001). It has led to the definition of eight inherent risk classes for dolomitic land ranging from a low inherent risk of sinkhole or doline formation to areas with a high inherent risk of very large subsidences.

2.5.2 Establishment of guidelines and manuals

The Council for Geoscience has issued formal guidelines for the development of dolomite land (CGS, 2007). These guidelines include the roles and responsibilities of the various parties; minimum requirements for geotechnical investigations on dolomite and for reporting on the results thereof; a method of classification of dolomite land in terms of the likelihood and magnitude of subsidence, appropriate types of development for each class

of dolomite land; and the approach the Council will adopt in reviewing development proposals on dolomite land. The classification of dolomite land in terms of inherent risk is based largely on the scenario supposition method of Buttrick et al.

The National Homebuilder's Registration Council has included dolomites in its site classification system together with specific precautions to be taken when developing these areas. In their Home Building Manual (NHBRC, 1999) they define dolomites in terms of a four-fold classification system, D1 - D4 where D1 is the lowest risk category where no precautions are required and D4 the highest where development of housing units is effectively prohibited.

2.5.3 SANS 1936: Development on Dolomite Land

The increasing demand for land in the Gauteng area in particular has led to the development of more areas underlain by dolomite. Due to the risks involved, such development needs to take place in a controlled manner. This has led to the compilation of a set of national standards (SANS 1936, Parts 1 – 4) dealing with development of dolomite land. These standards are referenced in the National Building Regulations and are therefore regarded as compulsory specifications, falling under the control of the National Regulator for Compulsory Specifications.

The objective of SANS 1936 is to set requirements to ensure that people live and work in an environment that is seen by society to be acceptably safe, where loss of assets is within tolerable limits, and where cost-effective and sustainable land usage is achieved (Foreword to SANS 1936-1).

The development of these standards and the Candidate's role in this process is described in Section 14 in Part 4 of this Dissertation.

2.6 References

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3. EXPANSIVE SOILS

3.1 Background

The establishment of the National Building Research Institute in the late 1940's coincided with the development of the Free State Goldfields centred on the town of Welkom in the (Orange) Free State. This area of South Africa is underlain by rocks of the Ecca and Beaufort Groups which form part of the Karoo Sequence. These 250 million year old sedimentary strata include siltstones and mudstones that decompose to form deep profiles of expansive soil. In keeping with the NBRI's policy of researching practical problems of national importance, the attention of Jennings and his team was directed towards the significant damage which these problem soils were inflicting on light structures, including housing developments on the mines. At the same time, development was taking place on clayey alluvial soils adjacent to the Vaal River in Vereeniging and on the norites of the Bushveld Complex stretching from the north-west of Pretoria to the Rustenburg area. Problems were also being experienced with expansive soil conditions on the weathered lavas which underlie portions of the city of Pretoria. In 1950, the NBRI issued Bulletins 4 and 5 written by A.B.A. Brink dealing with the engineering geology of the Vereeniging area and the stratigraphic profile of a test pit at St. Helena Gold Mine (Brink, 1950a & b). In 1955, this was followed by a paper in the *Transactions of the SA Institution of Civil Engineers* by Brink (who had now joined the firm of consulting engineers Kantey & Von Geusau) on the genesis and distribution of expansive soils in South Africa (Brink, 1955).

The same edition of the *Transactions* carried a paper by Jennings on the phenomenon of heaving foundations (Jennings, 1955). This paper pointed out the progressive and seasonal nature of heave movements resulting from the gradual build-up of moisture below the central portions of the structure coupled with seasonal moisture variations around the perimeter. This three page paper generated no less than 22 pages of discussion.

From this point on, the emphasis shifted from identifying the problem and its causes to the prediction of heave movements. Two papers were published by Jennings and Knight (Jennings and Knight, 1956 & 1957) on the prediction of heave using oedometer (consolidometer) test equipment.

For many years thereafter, the oedometer remained the soil test commonly used for the prediction of heave. However, this test method required undisturbed samples and meticulous preparation of test specimens in order to produce usable results. In 1964, another researcher at the NBRI, D.H. Van der Merwe produced what is probably one of the most widely consulted papers on the prediction of heave for South African soils. In this paper he correlated the potential heave of a soil to the plasticity index of the material and the percentage clay fraction (Van der Merwe, 1964). This simple and cost effective method of predicting heave of soil profiles using the most basic of soil mechanics tests is still widely used today.

In the interim, Jennings had left the NBRI to take up his post as professor of civil engineering at the University of the Witwatersrand. In conjunction with Professor Kerrich from the Department of Mathematical Statistics at the University, he produced a paper on the heaving of buildings and associated economic consequences with particular reference to the Orange Free State Goldfields (Jennings & Kerrich, 1962). Not only did this paper summarise the research work carried out to date but it also produced the first practical recommendations for the founding of single storey structures on expansive soils. The founding solutions contained in this paper form the core of the recommendations given in the National Home Builder's Manual (NHBRC, 1999). Only raft foundations have been added to the list.

From the mid-1960's, the focus of the NBRI shifted to other areas of research. However, new technologies including the ability to measure suctions (or negative pore water pressures) within the soil profile and the increasing use of stiffened raft foundations led to renewed interest in the subject. In the late 1970,s, J.T. Pidgeon joined the staff of the NBRI and produced a number of papers mainly on the design of stiffened raft foundations. In the early 1980's, I.J.A. Brackley (also at the NBRI) concentrated on the prediction of heave movements using suction measurements and correlated the swelling of clays with density, moisture content and loading (Brackley, 1983).

Since this time, very little fundamental research has been carried out on expansive clays, mainly as a result of a shift in focus of the Council for Scientific and Industrial Research to more commercially orientated goals.

3.2 Problem Soils: State of the Art

In September 1985, the Geotechnical Division of SAICE hosted a two day seminar on problem soils in South Africa. The five keynote papers at this seminar (SAICE, 1985) dealt with dispersive soils (Elges), soft clays (Jones & Davies), expansive soils (Williams, Pidgeon and Day), collapsible soils (Schwartz) and dolomites (Wagener). The aim of the symposium was to present the state of knowledge and practice in each of these fields.

In keeping with this aim, the authors of the expansive soils paper included two researchers (Williams and Pidgeon) and the Candidate who was actively engaged in development on expansive clay areas in the Free State Goldfields at the time. The paper started by defining the extent of the problem. Single storey buildings, particularly houses, are most susceptible to damage. Kitcher (1980) had estimated the cost of repairs to houses to be built on expansive clay during the 20 year period from 1980 to 2000 at one billion rand.

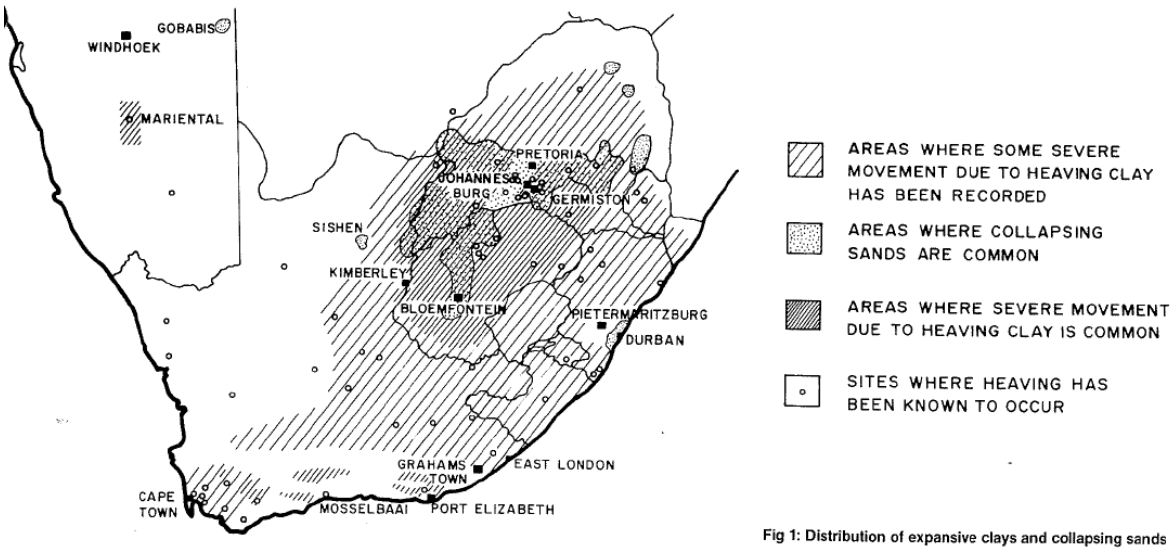

Figure 8: Distribution of expansive clays and collapsing soils (after Williams, Pidgeon and Day)

Figure 8 illustrates the significant areas of South Africa underlain by expansive transported or residual soils. Weathering of the primary rock forming minerals which,

depending on the nature of the parent rock and the environment under which weathering takes place, can either form non-swelling 1:1 lattice type clays or the expansive 2:1 lattice clay minerals. In these latter clays, orientated water molecules are present between successive sheets in the crystal structure. Changes in the amount of this water causes swelling or shrinking of the layered structure of the clay minerals and hence of the soil mass as a whole.

The weathering of Karoo dolerites and of the norites of the Bushveld Complex gives rise to a highly expansive layer of black "turf". Other igneous rocks which produce expansive soils include diabase and andesite. On the sedimentary strata, it is the finer grained rocks (mudstones, shales and siltstones) that decompose to produce expansive soil profiles, often of considerable depth. The Dwyka tillites can also be a problem depending on the origin of the source material. Transported soils derived from these rocks (whether in the form of alluvium, lacustrine deposits, gullywash or hillwash) may also possess expansive characteristics.

The present day climate and vegetation play a significant role in determining the potential heave magnitude. Where the present climate is arid or semi-arid, the most likely change in moisture content is due to wetting of the soil causing heaving to occur. In the more humid areas, drying out will cause the clays to shrink. Vegetation also plays a major role. Some forms of vegetation with deep and widespread root systems are capable of generating suctions of up to 1,5 MPa by the removal of moisture from the soil via evapotranspiration. Removal of this vegetation can lead to significant heave movements as was the case at Lethabo Power Station.

3.2.1 Investigation of testing

This section of the paper, written mainly by the Candidate, describes a typical investigation programme, including:

- A desk study of available information including aerial photographs, geological maps and any existing reports, preferably followed by a site visit during which field mapping is undertaken and the performance of existing structures is observed.
- A fieldwork programme including systematic description of the soil profile on site and taking of samples for laboratory testing. This may include tests on the in situ soil including soil suction measurements or large scale swelling trials where water is deliberately introduced into the soil profile.
- Testing of samples in the laboratory including grading and indicator tests, single or double oedometer tests and the determination of free swell, swell pressure, dry density, moisture content and specific gravity. Occasionally, the mineralogical composition of the material may be determined using x-ray diffraction analysis.
- The interpretation of the results of the field and laboratory tests, leading to the prediction of total and differential heave movements.

There are a number of simple observations that can indicate the presence of expansive soils. For example, expansive soils are often black, dark grey, red or mottled yellow-grey but seldom light grey, brown or white. The soil mass often has a shattered structure and slickensides (polished joint surfaces) are often visible in the soil profile. The performance of existing structures, particularly yard walls is also a useful indicator.

3.2.2 Prediction of total and differential heave

The paper provided an overview of the available methods of predicting heave movements and discussed their advantages and limitations. The first of these was the double

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oedometer test originated by Jennings & Knight in 1956 which compares the consolidation of two samples of clay, one at natural moisture content and the other saturated. It was noted that this test involves several assumptions and tends to over-predict the total heave magnitude.

The second method of predicting total heave is based on unit heave curves from which the total heave is obtained by integrating the contributions of each of the layers of soil below founding level. The Van der Merwe Method is a particular example of this procedure. In this method, the potential expansiveness of the soil is estimated from the plasticity index of the material and the percentage clay fraction as illustrated in Figure 10. The four categories of potential expansiveness shown in this figure are defined in terms of the potential heave movement per 300mm thickness of expansive soil at the ground surface as follows:

Low	0mm / 300mm
Medium	6mm / 300mm
High	12mm / 300mm
Very high	25mm / 300mm.

The potential heave for each 300mm layer in the profile is multiplied by a factor[7] which varies from 1,0 at the ground surface to near 0 at a depth of 30 ft (approximately 10 m) to predict the heave of that layer. The predicted heave of each 300mm layer is then summed over the depth of the expansive soil profile to obtain an estimate on the expected total heave magnitude.

Two further methods of heave prediction published by Brackley were presented. The first was an empirical relationship between percentage swell and the void ratio and moisture content of the in situ soil, the external load applied to the soil and the plasticity index of the material. The second method established a relationship between the percentage swell, the plasticity index of the material, the overburden pressure and the measured soil suction.

In the absence of further information, differential heave was taken as one half of the predicted total heave. A more rational approach was proposed involving the determination of the long term equilibrium suction profile below the foundation and an assessment of the worst likely suction at the edges caused by evaporation or evapo-transpiration which can be compared with the suction corresponding to the shrinkage limit of the undisturbed soil. The differential heave is then determined from the worst possible combination of suction changes likely to occur during the life of the structure, translating these into volume changes of the subsoil using one of the previously mentioned heave prediction methods.

3.2.3 Construction Methods

The paper extended the earlier work by Jennings, Kerrich and Evans by including recent work by Pidgeon on raft foundations. The applicability of these methods and the estimated additional cost over conventional construction was estimated in Table 4.

[7] This factor (F) was determined empirically by matching the observed variation in heave with depth in the profiles at Vereeniging and Odendaalsrust. It can be calculated from the equation $D = k \log F$ where D is negative depth below ground surface in feet and k is a constant. For a value of k = 20, it was found that the calculated heave matched the mean maximum measured heave on the sites investigated.

Table 4: Type of construction for various heave magnitudes

Type of construction (modified from Jennings and Kerrich)	Estimated total heave (mm)	Corresponding maximum deflection ratio	Estimated additional cost
Normal - continuous brick walls on strip footings	0 - 6	1:4 000	0%
Modified normal - high fanlights reinforced footings and lintels	6 - 12	1:2 000	1 - 3%
Split construction with reinforced brickwork	12 - 50	1:480	5 - 10%
Piles to limited depth with split construction and reinforced brickwork	50 - 100	-	20%
Underreamed piles with suspended floors	100+	-	30%+
Stiffened raft foundations	-		7 - 15%

Other methods described in the paper included pre-wetting, during which the soil is "pre-heaved" by increasing its moisture content and partial or total removal of the expansive soil layer with replacement by inert granular fill.

It is interesting to note that, close on 25 years after this paper was published, the methods given still form the backbone of the recommendations given in the NHBRC Home Builder's Manual.

3.2.4 *Design of stiffened rafts*

The purpose of a stiffened raft is to reduce the differential movement of the supporting soil to a level that can be tolerated by the superstructure. Since 1951, seventeen different design methods for stiffened rafts had been published. Most of these use either the "plate-on-mound" approach or the "swell-under-load" approach. The plate-on-mound approach assumes that the foundation is placed on an already formed mound which is taken to be that which would occur under an impermeable membrane covering the foundation area with no account being taken of the stress path followed by the soil. The more rational swell-under-load approach, which is more complex to apply, attempts to predict the foundation distortions resulting from the actual sequence of heave and/or shrinkage of the soil mass caused by changes in the moisture and stress conditions arising from construction of the building. The relative simplicity of the former approach and its tendency to err on the conservative side makes the plate-on-mound approach more convenient for everyday use.

A number of design methods reviewed by Pidgeon produced widely varying results. These results were compared with the behaviour of experimental stiffened rafts constructed at Onderstepoort and monitored over four and a half years during which severe drought conditions were experienced. Even the least conservative of these methods required a raft stiffness greater than that observed to perform satisfactorily during these trials. However, this was not the case in the Orange Free State Goldfields

where the adoption of the least conservative of the methods would have led to serious under design.

The paper made reference to a finite element programme known as FOCALS developed by Pidgeon at the National Building Research Institute. This method was capable of analysing rafts of any plan shape but was too general for routine use both from the point of view of time and expense. For routine design, the method proposed by Lytton (Lytton, 1972) was recommended. The paper then proceeded to provide recommendations on input parameters, mound geometry, soil properties, concrete design and allowable deformations for use in the analysis of raft foundations.

3.2.5 Protection of the home owner

The penultimate section of the paper dealt with the protection that the home owner could expect from his or her homeowner's insurance policy or by way of recourse against the designer of the structure. It was noted that damage due to expansive soils would be covered by a policy extension dealing with "subsidence and landslip" which was available on payment of an additional premium to the insurer.

At that time, there were no statutory requirements as to who should draw up building plans for houses. It was estimated that less than 1% of houses were professionally designed and that most mortgage lenders did not insist on any formal soil investigation. The decision whether or not to employ a professional architect or engineer for the investigation, design and supervision of construction rested largely with the home owner.

The paper noted that one of the difficulties faced by the home owner when resorting to litigation was defining "damage". Reference was made to a fivefold category of severity of damage recently developed in the United Kingdom and the desirability of introducing a similar classification in South Africa.

The paper expressed the need for an increased level of consumer protection such as that afforded by the defective buildings act in South Australia or the registration organisation of warranted houses in Japan. This has largely come to pass with the formation of the National Home Builders Registration Council or NHBRC.

3.3 CSIR Raft Design Method

To facilitate the design of raft foundations by practicing engineers, the CSIR developed an "expert system" for the rational design of raft foundations. This computer based system was marketed by the CSIR to registered users.

A conference to launch the CSIR design programme was held in Pretoria in November 1988. At this conference, both the background to the method and a series of supporting papers were presented. These supporting papers dealt with issues such as the prediction of total and differential heave, the evaluation of ground stiffness (a fundamental and often incorrectly determined parameter for raft design) and alternative simplified methods of raft design.

The "Lytton" method of raft design was presented in a paper by the Candidate (Day, 1988). The purpose of the paper was to provide a simple method which not only demonstrated the essential principles of raft design but also provided a means by which the output of the CSIR programme could be checked using hand calculations. The introduction to the paper defined the purpose of a raft foundation and provided a description of the raft foundations typically in use on the Free State goldfields during the 1980s. This was followed by a description of the various design approaches for raft

foundations based largely on the 1985 state of the art paper. It then proceeded to describe the Lytton method in particular, which uses the "plate-on-mound" approach.

The main problems with the use of the Lytton method were (and still are) the determination of the input parameters required and interpretation of the equations which were not always clear.

The paper provided practical guidance for the determination of differential heave, the modulus of subgrade reaction, the mound geometry and the allowable deformation of the supported structure. It then described the method of determining the moments and shear forces in the raft which, in the Lytton method, are based on idealizations of the loading and support conditions coupled with correction factors to take account of biaxial bending of the raft.

The paper also presented a number of practical hints on the application of the method derived from the author's experience of the design and performance monitoring of several hundred raft foundations constructed in the towns of Welkom and Virginia on the Free State goldfields.

A slightly updated version of the paper was presented at the course on *Design of Foundations to suit Various Soil Conditions* presented by the Structural Division of SAICE in May 1991. The paper was also being used by the candidate as the basis of teaching raft design to undergraduate students at Technikon Witwatersrand (now University of Johannesburg).

Despite the fact that the paper contains little more than the application of a design method developed in the United States to South African conditions, the Candidate has received more requests for copies of this paper than any other paper he has written. This clearly demonstrates the need for practical interpretation of new developments for South African conditions.

3.4 Subsequent Developments

3.4.1 Innovations in Raft Type

The traditional stiffened rafts consist of a grillage of reinforced ground beams, 300mm – 400mm wide at 2,5m to 4m spacing capped with a 125mm – 150mm mesh reinforced floor slab as shown in Figure 9a. Beam depths typically ranged from 0,5m to 1,0m. The beam was wide enough to facilitate excavation by hand or using a narrow bucket on a light excavator and the fixing of reinforcement in the trench.

In the late 1980s and early 1990s, two innovative raft types were developed. The first was known as the "Boucell" raft conceived by the NBRI. This consisted of a top and bottom slab separated by and tied into a grid of reinforced masonry walls as shown in Figure 9c. The second was the waffle raft which developed by Pidgeon. The waffle raft makes use of narrow, closely spaced stiffening ribs (Figure 9b) which are constructed in trenches formed using a mechanical trenching machine commonly known as a "Ditch Witch". Of these two methods, the waffle raft has fared best in the marketplace. This is due largely to its economical design, ease and speed of installation and successful marketing as a package solution. It was patented by Pidgeon a few years after he left the NBRI with the intention that only licence holders would be permitted to construct raft foundations using this method. This has resulted in a number of legal challenges mainly based on the patentability of such a concept. It is understood that the patent has since lapsed.

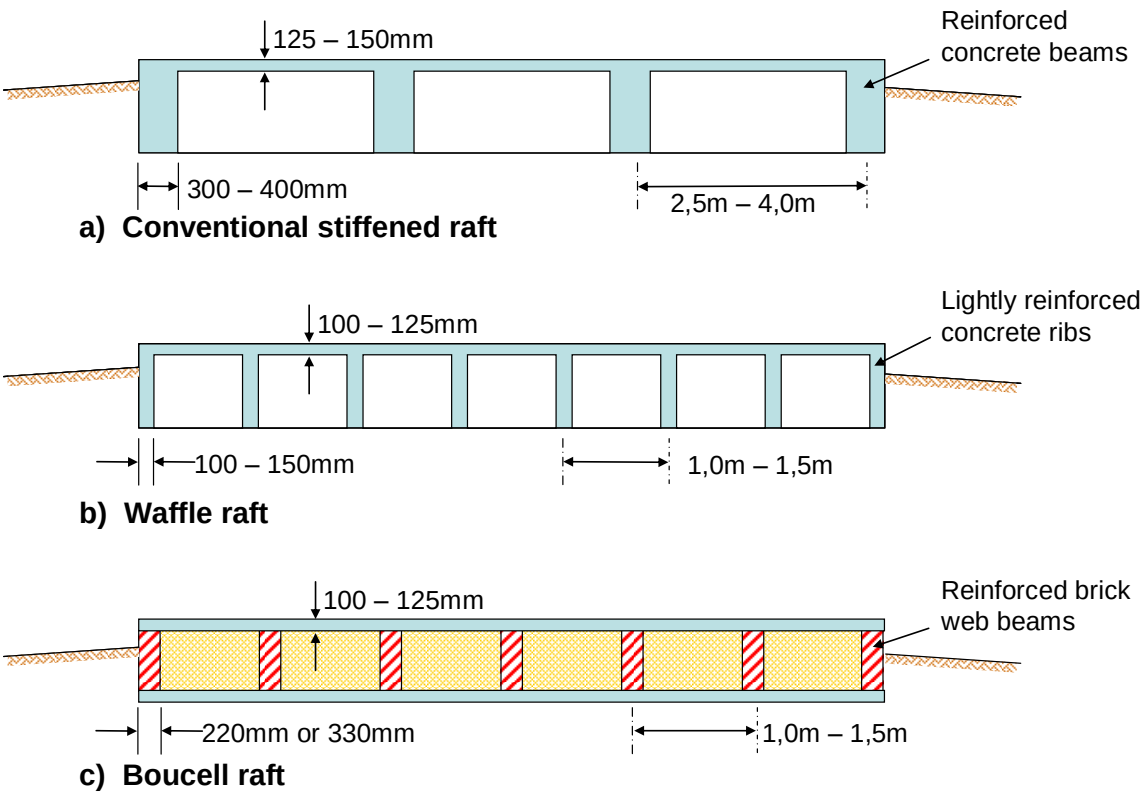

Figure 9: Types of stiffened raft foundations

3.4.2 *Protection of the Home Owner*

Looking back over the years, the problems associated with protection of the home owner given in Section 3.2.5 above have largely been resolved by the formation of the National Home Builder's Registration Council which provides a warranty scheme for residential structures enrolled with the Council. In terms of the Housing Consumer Protection Measures Act, 1998 (Act No. 95 of 1998), it is compulsory to register all commercially built housing units with the NHBRC.

The Home Builder's Manual produced by the NHBRC provides a fourfold classification of most of the problem soils encountered in South Africa including expansive soils. Raft foundations are recommended as standard solutions for highly collapsible and expansive soil profiles. The Home Builder's Manual, which is largely based on the 1995 Code of Practice for single storey masonry structures produced by the Joint Structural Division of SAICE (SAICE, 1995), also provides means by which the severity of damage to the structure can be categorized. Thus, the major concerns regarding protection of the home owner expressed in the 1985 paper have largely been addressed.

3.4.3 *Raft foundation failures*

Over the years, there have been a number of failures of raft foundations. These are generally "serviceability" failures in which the stiffness of the raft is inadequate to prevent cracking of the floors or superstructure rather than "ultimate limit state" failures due to inadequate strength.

In the Candidate's experience, the majority of these failures are not the result of deficiencies in the method of analysis. They are generally due to an incorrect assessment

of the input parameters for the foundation design. One of the most troublesome parameters is the prediction of the soil stiffness. Under-estimation of the stiffness of the soil results in the raft bedding down further into the mound of "expanded" soil which forms below the centre of the structure. This bedding down results in a decrease in the edge distance leading to an under-estimation of the hogging moments in the raft.

Under-prediction of the differential heave is another common problem. This is often caused by uncertainties associated with the thickness of the expansive horizon and the depth of the water table due to inadequate depth of investigation of the soil profile. In an attempt to address these and other issues relating to site investigation, the Geotechnical Division of SAICE has recently produced a Code of Practice for site investigation which provides guidance on the depth of investigation required and methods of determining relevant soil properties (SAICE Geotechnical Division, 2010) – see 15.3.

Another common problem that the Candidate has come across, particularly when reviewing raft designs on behalf of the professional indemnity insurers, is that the basic concept of the raft design has not been appreciated. The primary purpose of the raft is to create a stiff "plate" capable of limiting the deformations of the soil to a level that can be accommodated by the superstructure. This requires the raft to have high flexural and torsional stiffness. All too often, the required stiffness is provided below the walls of the structure only and not below the floors. Another common error is the use of cranked or discontinuous stiffening beams, resulting in a significant reduction in the stiffness of the raft.

3.4.4 Poor Standard of Laboratory Testing

A worrying factor that has contributed to at least two significant claims for damages against designers of foundations on expansive soils in recent years is the poor standard of laboratory testing in South Africa.

In a paper entitled "*Are we getting what we pay for from geotechnical laboratories?*", Jacobsz and Day (2008) presented the results of comparative testing of basic soil parameters by four commercial laboratories in South Africa and drew attention to the significant difference in the results. Their conclusion was that geotechnical laboratories in South Africa are not delivering the quality of testing that geotechnical engineers require to make critical design decisions. The opinion was expressed that the commercial testing laboratories have allowed themselves to stagnate. Despite a significant increase in the volume of testing, some laboratories have not increased their prices in over two years. As a result, there had been little or no investment in staff training and new equipment. The message of the paper was that the geotechnical fraternity is prepared to pay the price required for laboratory tests that are executed in accordance with acceptable standards using modern equipment operated by well trained staff.

Four years later, it would appear that the industry has not responded to this message. The laboratories are still competing on the basis of price instead of quality. This is demonstrated by the following set of results where a number of tests were undertaken on materials from the same site where there is a dispute regarding the classification of the site with regard to heave movements. Four of the samples (labelled A-D in Figure 10) were "parallel tests" by Laboratories 1 and 2 on the same samples whereas Laboratory 3 tested a different batch of samples from the same site. To compound matters, after the first batch of test results were received, Laboratories 1 and 2 were advised of the discrepancies and asked to repeat the tests. The outcome was an even bigger discrepancy. Both Laboratories 1 and 2 are SANAS accredited laboratories.

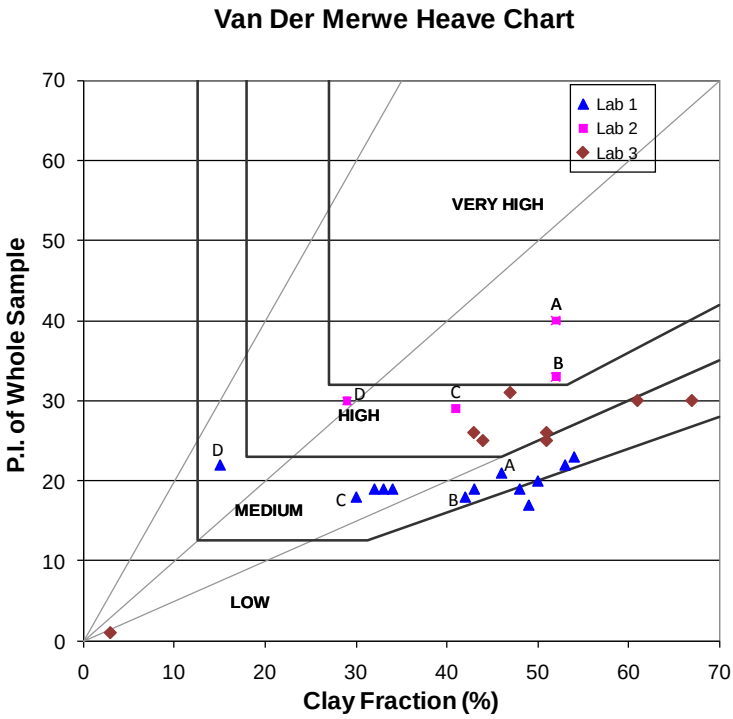

Figure 10: Results from three laboratories on soils from same site

The situation regarding strength testing of soils is even worse. Due to the large number of tests required for a recent multinational project in Southern Africa, samples were sent to a number of academic and commercial laboratories for triaxial testing. The Candidate visited a number of the laboratories to audit the test methods being used. Significant problems were found with sample preparation and methods of testing. These problems were reflected in the results. As a consequence, the results from all but one laboratory had to be discarded and all further tests carried out overseas.

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4. LATERAL SUPPORT IN SURFACE EXCAVATIONS

The contents of Sections 4.1 and 4.2 are mainly extracted from Day (2004).

4.1 Background

4.1.1 1960s and 1970s

The majority of buildings erected in central Johannesburg up to the late 1950's relied on natural light and ventilation and were of limited height. Most of these buildings occupied the entire site resulting in the creation of narrow streets and pavements with little or no space at ground level for the public. Internal courtyards provided light and ventilation for the inner areas of the building (Rhodes-Harrison, 1967). Basements constructed in Johannesburg's CBD at that time generally consisted of one or two levels with maximum excavation depths seldom exceeding 30 ft (about 9m). In many areas of the CBD, this resulted in the removal of the less competent surface soils permitting the use of conventional strip footings on the stable underlying strata while still remaining above the level of the water table.

During the early 1960's, there was a change in building design philosophy. Whole city blocks were developed and the height restrictions were relaxed by the municipality accompanied by an award of "bonus bulk" for the creation of open plazas at street level. Architects and developers followed the idea of concentrating bulk in tower blocks set well back from the street boundaries resulting in a substantial change to the appearance of the city scene, opening it up to more space, light and air (*ibid*). In addition, the Johannesburg Town Planning Scheme of 1946 classified sub-surface floors as "free bulk". As a result, the maximum utilisation of subterranean space provided the obvious answer to the increased demand for parking and the accommodation of air conditioning plant rooms, fire water tanks, electrical supply systems etc. (Hall, 1967).

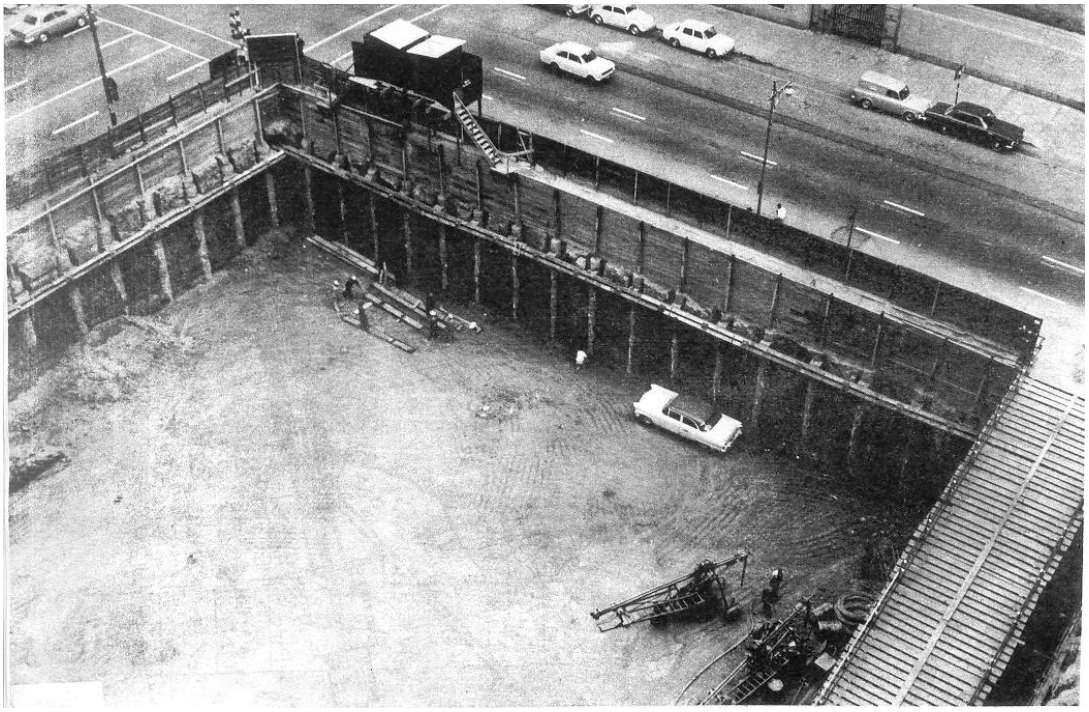
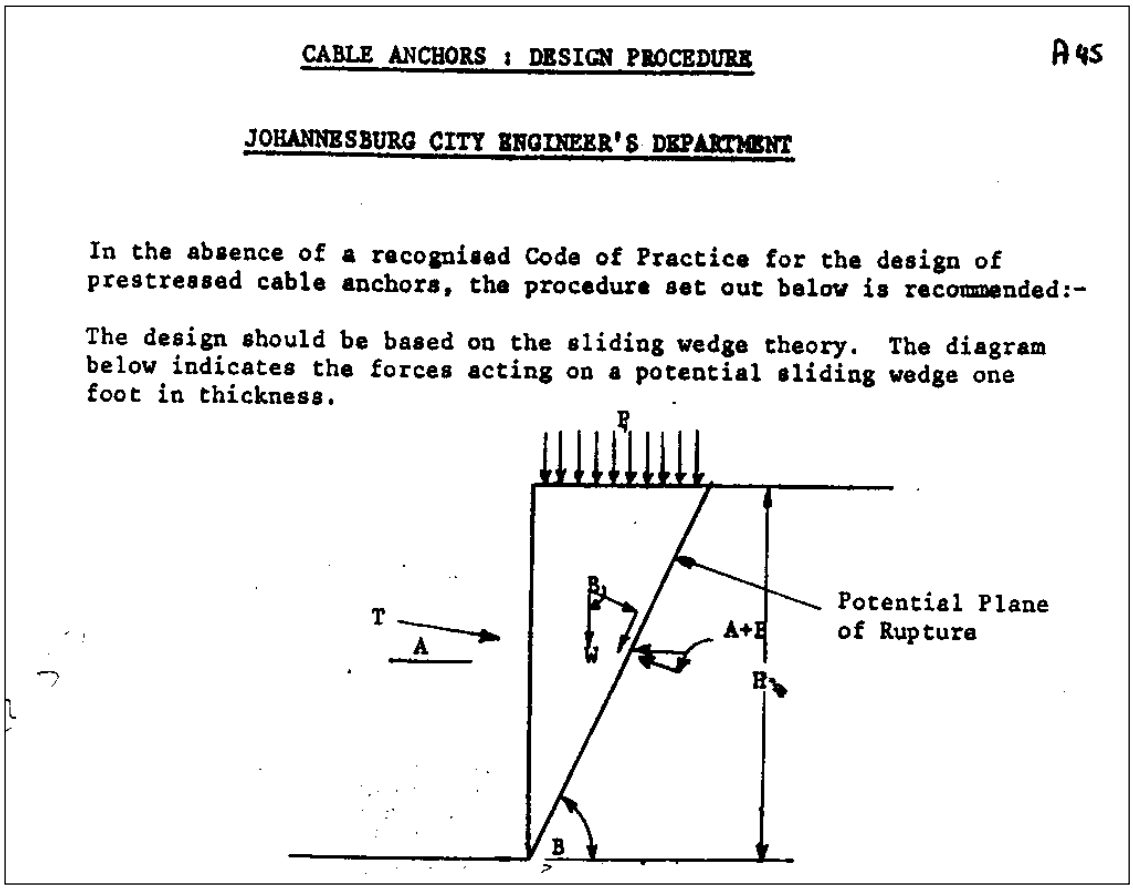


Photo 6: Anchored basement under construction, Johannesburg, 1967

In parallel with the changes in the design of CBD buildings and to the building regulations that governed such developments, changes were also taking place in the field of construction techniques. By 1953, the use of post tensioning techniques for the strengthening and raising of dams was fairly well known. In that year, the first known application of this technique to support the vertical faces of an excavation took place in a sub-vertical circular shaft at a depth of 2100m in the ERPM Mine on the East Rand (Parry-Davies, 1967). The next major development took place in 1958/59 on the S A Mutual site located between Main Street and Chapel Street in Port Elizabeth. The 15m elevation difference across the site resulted in the need to form a vertical excavation face of this height. This was achieved using temporary cable anchors with a capacity of 40 tons at depth intervals of 3,6m down the face. From then on, the acceptance of cable anchors grew and this method of support was used on two of the deepest basements in central Johannesburg, namely the South African Associated Newspapers basement and the Carlton Centre basement. The merits of creating a free (un-grouted) length on the anchor was soon realised although early attempts at pressure grouting of the fixed length met with mixed success.

During 1966, a group of individual engineers held discussions with the City Engineer of Johannesburg with a view to clarifying problems relating to the safety of excavations for deep basements. Arising from these discussions, a symposium on deep basements was held in August 1967. The symposium was addressed by city authorities, architects, legal advisors and civil engineers. In addition to discussing methods of design and site investigation, a number of case histories were presented.

By 1967, there were fourteen basements in the Johannesburg city centre deeper than 10m. Notable amongst these were the South African Associated Newspapers building (30m), the Carlton Centre (29m), the Standard Bank Centre (20m) and the Trust Bank building (27m), all of which were under construction at that time. In the *Symposium on Deep Basements* held in August 1967, Mr. E.J. Hall, the Deputy City Engineer, indicated that "*no less than 36 100 persons at an average of 1.4 persons per car (use) private cars for the work journey while 31 300 use public road transport* " (Hall, 1967). While the desirability of providing off-street parking and the need to satisfy the aspirations of developers were recognised, these needed to be balanced against the factors associated with the construction of deep basements perceived by the City Fathers to be detrimental. Such factors included the disruptive influence on the life of the city citizens, temporary loss of amenities, the concentration of heavy vehicles and encroachment of support systems (particularly cable anchors) below adjacent public and private property. There were also considerations of public safety in the event of failure of deep basements during construction. This led the City Engineer to lay down requirements among which was that the structural systems or support to earth walls during excavations should be contained within the site boundaries. Encroachments under street surface, even of a temporary nature, "*were to be considered reluctantly and on the basis that the developer should produce a strong argument to support his case*" (*ibid*). All calculations had to be on a basis approved by the City Engineer and such calculations had to be carried out by competent and experienced engineers.



CABLE ANCHORS : DESIGN PROCEDURE

A 45

JOHANNESBURG CITY ENGINEER'S DEPARTMENT

In the absence of a recognised Code of Practice for the design of prestressed cable anchors, the procedure set out below is recommended:-

The design should be based on the sliding wedge theory. The diagram below indicates the forces acting on a potential sliding wedge one foot in thickness.

Figure 11: Extract from Johannesburg City Engineer's guidelines for Cable Anchors, 1962/3

At the conclusion of the symposium, it was suggested that a committee be established to draft a code of practice for lateral support which would be applicable not just to Johannesburg but to the whole of South Africa (Kriel, 1972). This culminated in the production of what is believed to have been the first code of practice on lateral support in surface excavations worldwide, published by the South African Institution of Civil Engineers in 1972. In his foreword to the code (*ibid*), Mr. J.P. Kriel, then President of the Institution, noted that it is not intended that the code would be the final word on lateral support but rather that it would be used, amended and updated by all engineers engaged in lateral support works. These were prophetic words as is set out in the section below.

4.2 1989 Code of Practice

4.2.1 Background

During the 1970's and 1980's, the construction of deep basements in the CBDs of most of South Africa's major cities continued. Grouting techniques improved with the introduction of the tube-à-manchettes as an integral part of the cable anchor assembly leading to the successful construction of high capacity ground anchors (up to 600 kN) even in soft and cohesive soils. Individual companies within the South African geotechnical market developed specialised techniques for the installation of ground anchors and, slowly but surely, specifications began to evolve for the installation, testing and monitoring of ground anchors. The use of the observational method as expounded by Peck in his Rankine

Lecture 1969 (Peck, 1969) became routine practice. DD81 (the forerunner to BS8081:1989) dealing with ground anchors was issued in the United Kingdom.

South Africa remained at the forefront of the development of prestressed ground anchors. In October 1979, a symposium on prestressed ground anchors was held under the auspices of the Concrete Society of Southern Africa (CSSA, 1979). This symposium dealt with many of the recent advances including drilling, installation and grouting techniques, stressing and testing of anchors, standards and specifications and case histories.

By 1984, it was realised that the South African Code needed to be revised to reflect the accelerating pace of development in the field of lateral support. As a result, the Geotechnical Division of the SAICE convened a committee under the chairmanship of Ross Parry-Davies to update the 1972 code. As was the case with the 1972 code, the drafting committee comprised academics, consultants, contractors and representatives of the legal and insurance professions. The Candidate represented the view of lateral support designers on this committee and acted as the committee's host for the entire four year drafting period. In 1989, the Geotechnical Division published the Code of Practice on Lateral Support in Surface Excavations (SAICE 1989) which stands to this day.

4.2.2 Contents of the Code

After dispensing with the formalities of Introduction, Scope and Definitions, Chapter 1 of the Code provides guidance mainly for Owners and Developers on the preliminary actions to be taken in the planning of a basement excavation. The main purpose is to alert the owner to the many activities required including structural and architectural requirements, investigation of soil conditions, obtaining necessary permissions from authorities, appointment of professionals and lateral support insurance.

Chapter 2, entitled "*Site Investigation*", was largely written by the Candidate. It provides an outline of the scope and technical requirements for site investigations on lateral support projects. While it contains little new for the experienced geotechnical engineer, this chapter provides guidance to owners and project managers for the specification of an adequate site investigation.

Chapter 3 deals with the selection of a lateral support system. It briefly describes the various options which designers have for excavation support and the factors that affect the selection. In each case, the application, advantages and disadvantages of the particular system are presented.

Chapter 4 dealing with earth pressures is the most theoretical chapter in the Code. It provides guidance for the calculation of active and passive earth pressures and the soil movements required to develop these limiting states. Guidance is also given on the selection of appropriate strength parameters and test methods for various soil types and design situations together with an assessment of the likely variation of the various parameters and the influence of this variation on the selection of design values. The chapter contains limitations on the value of effective cohesion (c') to be used in design and minimum earth pressures for which lateral support systems should be designed. The Candidate was also heavily involved with the writing of this section of the code, particularly the sections of selection of strength parameters and appropriate test methods.

Chapter 5 provides guidance on the design of lateral support systems, concentrating mainly on the local and overall stability of anchored and soil nailed walls. It provides guidance on the design of the structural elements of the system including anchors, struts and rakers, soldier piles, walers and lagging. It lays down deformation limits for urban and non-urban areas and for various types of surrounding development.

Chapter 6 deals with anchor design and construction. It defines the various components of a typical ground anchor, the methods of construction and provides guidance on the materials used in the construction of both temporary and permanent anchors. It provides comprehensive guidance on the stressing and testing of both test anchors and working anchors.

Chapter 7, the shortest chapter in the Code, deals with groundwater control. It examines the effect of water seepage and water pressures on the lateral support system, methods of dewatering and the influence that dewatering can have on adjacent structures.

The final chapter of the Code deals with control of the works, monitoring and records. The three main items to be monitored are ground movements and their effect on adjacent development, the water table and forces in anchors, struts and other components of the support system.

Appendices D and E in the Code deal with legal and insurance aspects of lateral support. It is pointed out that the owner of land is responsible for any damage caused by the removal of lateral support to his neighbour's property, even in the absence of negligence. The law pertaining to removal of groundwater is also dealt with. The need for lateral support insurance is outlined in the light of the fact that most other insurance policies specifically exclude the effects of removal of lateral support.

Appendix F provides case histories of observed movements of lateral support systems in tabular form. Both local and international case histories are covered.

Other appendices in the Code deal with laboratory and field tests, field descriptions of soil and rock, extracts from relevant Acts and Regulations, draft specification for blasting, suggested methods of measurement and definition of anchor terms.

4.2.3 Continued Relevance and Need for Revision

In the Candidate's opinion, the Code remains as relevant today as it was when first published in 1989. This is based on two observations. Firstly, the Code forms the basis of most lateral support contracts in South Africa today. Secondly, there have been few lateral support failures in South Africa that could not have been prevented by adherence to the requirements of the Code.

In all probability, the continued relevance of the Code stems from its practical approach, its concentration on underlying principles and its general acceptance as a standard of good practice within the industry. While there have been major advances in analytical methods, particularly in the field of numerical analysis and the modelling of complex wall systems, these are seen as complementing the Code rather than detracting from it. Although the code is a working load design code relying on global factors of safety, there is no reason why its provisions should not be used in conjunction with a limit states design approach.

The 1972 edition of the Code was written in response to the burgeoning demand for deeper and deeper basements in city CBDs. It was written by pioneers within the industry, engineers who had grappled with the problems of lateral support and understood the risks involved. The 1989 revision to the Code incorporated many of the advances, particularly in anchoring technology, that took place in the intervening period. It too was authored by experienced contractors and consultants.

One of the biggest changes that has taken place since the issue of the 1989 Code is a change in the contracting environment. Firstly, there has been a systematic erosion of the relationship of trust which existed between owners and consulting engineers and of the ability of the engineer to recommend a contractor most suited for the execution of the work. The emphasis has shifted from the competence of the engineer and the ability of

the contractor to execute the work, and the focus is now mainly on cost and programme. The second major change is the rise to prominence of the Project Manager and his role in the execution of the project. (Note that the 1989 Code refers to the owner, the architect, the engineer and the contractor but there is no mention of project managers). Project Managers tend to regard lateral support and the design thereof as commodities that can be purchased on terms dictated by themselves.

In the Candidate's opinion, the biggest challenges facing the lateral support industry at present stem from four principle factors:

Inadequate geotechnical investigation and geotechnical data on which to base the design or pricing of the lateral support project.

The award of lateral support contracts on the basis of price alone rather than a competent assessment of the value of the service offered.

The setting of unrealistic time frames for the execution of lateral support work often due to poor planning during the early stages of the project.

A failure to recognise the need for modifying the lateral support system in accordance with the observed performance and encountered conditions during the execution of the works as laid down by the observational method.

The section of the Code that is in most need of revision is the section that deals with safety legislation. The requirements of the Occupational Health and Safety Act (Act 85 of 1993) in general and of the Construction Regulations (2003) in particular have rendered certain sections of the Code obsolete and in need of revision. In general, the remaining legal aspects of the Code remain unaltered.

4.3 Candidate's Contributions

4.3.1 Joubert Park Post Office – Example of Client/Engineer/Contractor Cooperation

The investigation of the Joubert Park Post Office basement started in 1978 with the Candidate becoming involved from February 1979 onwards. This was to be the deepest soft soil basement in central Johannesburg making use of prestressed, high pressure re-injectable ground anchors. It was an 18m deep basement in the residual andesites of the Johannesburg graben. In this part of town, the andesitic lavas have weathered to silty soils to depths of 50m or more. The basement covered half a city block (very small for such a deep basement) with a 10 storey, concrete-framed, brick-infill building founded on friction piles at depth of 10m occupying the other half of the block.

The client appointed an Architect and an Engineer to advise on the execution of the work. The Engineer conducted a detailed geotechnical investigation including a detailed programme of field and laboratory testing. The project remains, to this day, the only project on which the Candidate has had sufficient soil test results to undertake a meaningful statistical assessment of the data.

Prior to the commencement of the design, the Candidate held discussions with various contractors regarding the methods of construction and the capabilities of their plant. As a result of these discussions, a number of changes were made to the design, including the underpinning of the adjacent building.

The design of the anchors and soldier piles was based on the City Engineer's design guidelines mentioned above and also on a method by Littlejohn et al (1971) which provided a means of assessing the bending moments and shears in the soldier piles at each stage of construction. This method treated the soldier pile as a continuous beam

loaded by the soil on one side with supports at the anchor positions. This resulted in a requirement for very heavy soldier pile sections, a problem that would be addressed in later designs.

The Candidate predicted the deflections of the excavation sidewalls using a linear-elastic finite element programme with the results of plate load tests and SPT tests as input for the stiffness of the soil. The deflected shape of the excavation was correctly predicted but the magnitude of the movement was over-predicted by a factor of about 3. The Candidate would later discover that this was probably due to use of the modulus of the soil on initial loading rather than the modulus on unloading.

Despite problems with a higher than expected water table at the time of construction, the project was satisfactorily completed. An account of the project from investigation to completion was presented at the ASCE Speciality Conference on Earth Retaining Structures in Cornell in 1990 (Day, 1990). The data from this project has also been used as the basis of final year design projects at the University of the Witwatersrand where the Candidate has served as an external examiner.

The project demonstrates the benefits of close cooperation and trust between the client, engineer and the contractor.

4.3.2 Bank City, Jeppe Street Face – Example of the Observational Method

A few years after completion of the Joubert Park Post Office basement, another major basement excavation project was planned for the city centre of Johannesburg. This was the Bank City excavation that covered a full two city blocks (about 130m x 65m in plan – see Photo 7. The excavation varied in depth between 9m and 15m and straddled the faulted contact between the Ventersdorp lavas of the Johannesburg Graben and the weathered shales and quartzites of the Witwatersrand Group to the south. The northern Jeppe street face of the excavation was situated entirely in lavas and orientated east-west, roughly parallel with the strike of the faulted contact about 40m to the south. The average height of this face was around 10m.

Photo 7: Basement excavation constructed under the new code: Johannesburg, 1989.

This tender was awarded on a design and construct basis with the Candidate acting as the contractor's designer.

The geotechnical investigation (conducted by others) recorded shear strengths in the lavas well in excess of those used for the design of the Joubert Park Post Office excavation, often with considerable cohesion. For design purposes, a conservatively assessed friction angle of 30° was used with a zero cohesion. The design was carried out using the wedge failure method for anchor force determination and a slip circle analysis to check the length of the anchors as advocated in the lateral support code which was then nearing finalisation. It was, however, complicated by the presence of two brick-lined sewer tunnels below Jeppe Street and a number of water pipes. These dictated the angle at which the anchors could be installed and resulted in a concentration of fixed anchorages in a limited zone within the soil profile (see Figure 12). Although no formal movement predictions were undertaken, movements were expected to be of the order of 0,1% of the excavation height, about 10mm (Day, 1990a).

The movement of the crest of the excavation was monitored by a line and level survey along the edge of the excavation on a weekly basis during excavation. It soon became apparent that the Jeppe Street face was moving more than the adjacent faces in similar material using the same support design. Additional monitoring points were installed in Jeppe Street, on the far pavement and on the excavation face. Upward-inclined drainage holes were drilled to just below the position of the sewers and equipped with stopcocks so that the presence of any leakage from the sewers could be detected by simply opening the valve and checking for any accumulation of seepage. Fortunately, there was none.

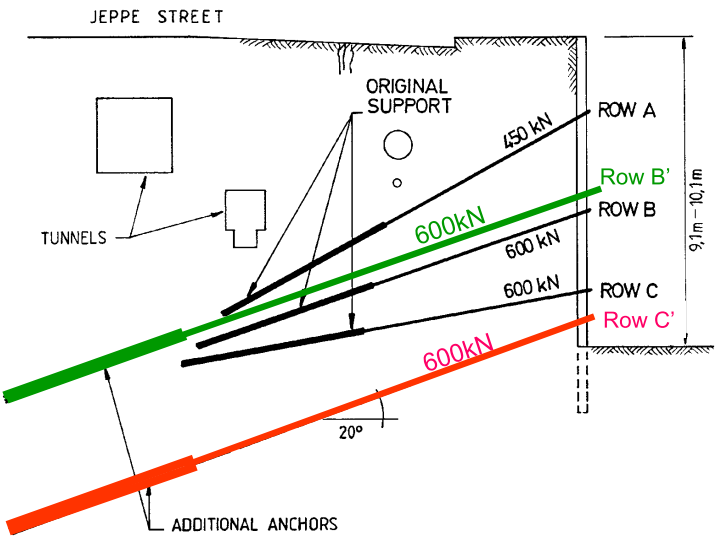


Figure 12: Lateral support to Jeppe Street face

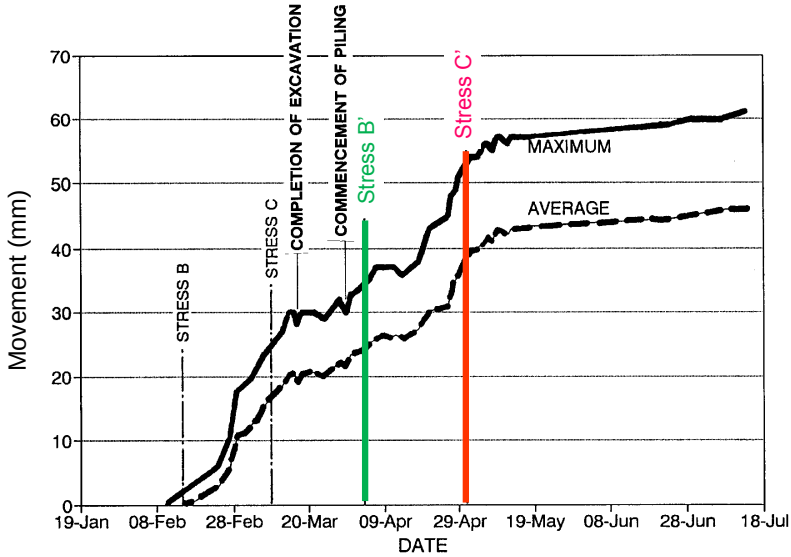


Figure 13: Recorded movements of Jeppe Street face

During installation of the final row of anchors, small blocks of soil began to fall from the face bounded on their lower surface by highly continuous, slickensided (polished) joint planes dipping into the excavation at inclinations of between 55° and 70°, close to the angle of the active failure wedge. An example of this is shown in Photo 8. These slickensided joints were highly continuous and could be traced for 20m or more along the face, always dipping at about the angle of the active failure wedge.

Photo 8: Small wedge of soil sliding on inclined, slickensided joint plane

As a contingency measure, the stability of the face was back-analysed assuming that the face as it stood was at limiting stability. This was to determine the minimum friction angle on the slickensided joints which was calculated as 17°, considerably lower than the shear strength of the retained soil assumed in the design. The additional anchor force required to restore the factor of safety to an acceptable value of around 1,5 was also calculated. It was decided that there was sufficient spoil available from exaction elsewhere on the site to back fill the face should the movements continue and no additional anchors were installed.

On completion of construction, the average recorded movement of the Jeppe Street face was 20mm (maximum 30mm), still within the 35mm permitted for excavations in urban areas. After a two week hiatus during which the installation of piles and construction of pile caps commenced adjacent to the face, the face again started moving downwards and into the excavation. The renewed movement triggered the remedial measures that had been previously identified, namely backfilling of the face and the installation of additional anchors. Two additional rows of anchors were installed increasing the anchor force on the face from 725kN/m to 1 240kN/m, an increase of over 70%. This was sufficient to stabilise the face. The average movement of this face was 46mm and the maximum movement slightly over 60mm as shown in Figure 13. The maximum movement recorded on the adjacent west face, which had the same support as in the original design, was only 7mm.

The client accepted the unforeseen nature of the ground conditions and reached a settlement with the lateral support contractor which included an agreement to negotiate the next two phases of the project with him.

This project delivered three valuable lessons:

1. In partially saturated or free draining soils, any movement of the face that persists more than a few days after completion of the excavation and installation of the support should be taken very seriously. This has been demonstrated on numerous subsequent excavation projects where on-going movements have required similar remedial action.
2. The movement of the Jeppe Street face occurred at the time the SAICE Lateral Support Code was being finalised. Both the designer (the Candidate) and the lateral support contractor were serving on the drafting committee. Their experience led them to ensure that the cautions regarding the use of cohesion in jointed materials contained in the CED's 1962/3 guidelines and the 1972 version of the code were included in the 1989 version of the code. The deformation of this excavation was included in the case histories of recorded movements of anchored walls given in Appendix F of the code.
3. The observational method is a very powerful tool when correctly applied. It was largely due to the benefits of this approach learnt from this project that led to the Candidate advocating the inclusion of this method in the lateral support code and in SANS 10160-5: Basis of Geotechnical Design and Actions.

The continuity of the joints, the uniformity of the observed dip, the proximity of the face to the southern edge of the graben and the strike parallel to the faulted contact between the lavas and the surrounding rock types led to the conclusion that these slickensided joints were associated in some way with the formation of the graben or subsequent shearing movements within the residual soils. As a result, the design of the next two phases of the Bank City project was based on a lower friction angle for the Jeppe Street face derived from back analysis of the problems experienced on the first phase. The parameters used for the remaining faces were left unchanged. All faces of these subsequent phases performed satisfactorily.

4.3.3 Research into Soldier Pile Stiffness

The southern half of the Bank City excavation was situated in the shales and quartzites which were significantly more competent than the andesites described above, so much so that it was not possible to auger large diameter holes for the installation of soldier piles. As a result, the soldier piles were reduced to back-to-back channel sections grouted into small diameter holes drilled using down-the-hole rotary percussion drilling methods.

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These small section soldier piles were incapable of resisting the bending moments from a continuous beam analysis using elastic design methods. It was, however, realised that three plastic hinges would have to form in the soldier pile before a mechanism would be created. This realisation justified the use of these small section soldiers.

In the end, it did not really matter what size section was used as the shale and quartzite rock was sufficiently competent for the soldier and the surrounding grout to act simply as a bearing pad behind the anchor faceplate. Nevertheless, this started the Candidate thinking about the possibility of using plastic design methods for routine design of soldier piles.

A request to the Candidate for suggestions for a final year design project topic from the University of Cape Town provide the opportunity for further research into this possibility. In 1991, C.T. Howie, a student at the university, produced a final year design project report (Howie, 1991) which presented a method of analysis for beams on an in-elastic foundation using a non-linear stress-strain relationship for the soil.

The next year, he enrolled for an Master of Science degree in engineering and extended his research to the development of a computer programme capable of analysing non-linear response of a layered soil coupled with plastic yield of the beam section. His research included model tests in the laboratory followed by full scale tests on a working soldier pile specially installed for this purpose on the second phase of the Bank City project. The Candidate arranged for two soldier piles with a smaller section to be installed in the andesites at a location where a 14m high face was supported by five rows of anchors. He also arranged for the middle anchor to be installed with a higher capacity than the surrounding anchors for the purposes of testing the soldier pile to failure. Howie then tested the soldier piles, succeeding in creating a plastic hinge in the pile immediately behind the anchor head in one test[8]. The tests were undertaken after completion of the excavation during which all anchors were tensioned to normal working load of 450kN. The load on the central anchor was then increased in increments by a further 300kN. Throughout the test, the deflections of the soldier pile were monitored over the full height of the face by measurement from a piano wire which was accessed by a scaffold erected for this purpose.

The results of this research were published in a paper by Howie, Scheele and Day (Howie et al, 1994) at the ISSMFE International Conference in New Delhi, January 1994. Figure 14 shows the comparison between the measured deflections of the pile and those predicted by Howie. The findings of this research are dealt with further in the following section.

[8] In the second test, the jack was not positioned centrally on the pile causing the pile to buckle sideways before failing in bending.

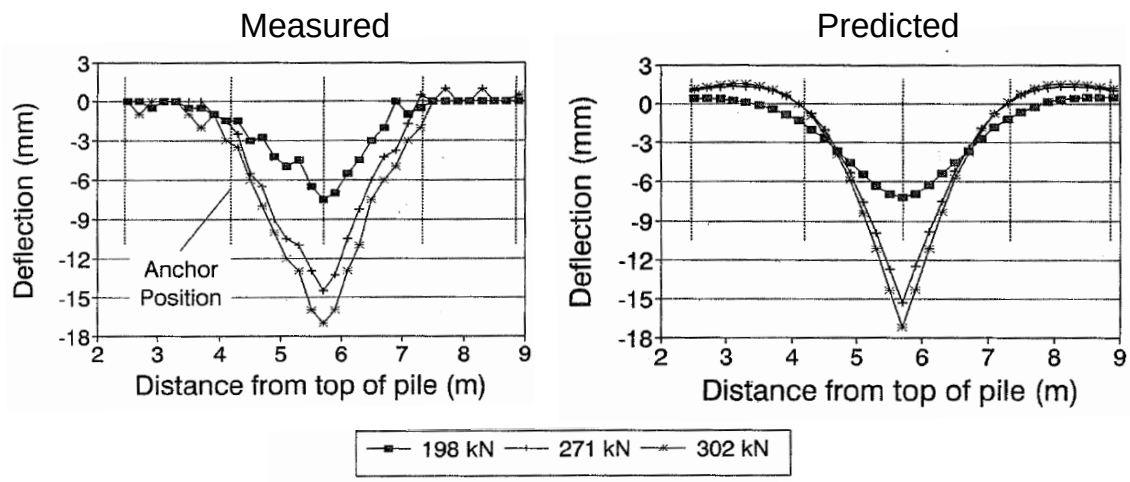


Figure 14: Measured and predicted soldier pile deflection for non-linear pile behaviour

4.3.4 *Factors Influencing the Movement of Retaining Structures*

At the same International Conference in New Delhi in 1994, the Candidate was invited to present a plenary session lecture on the effects of excavation sequence on the movement of retaining structures. After discussions, the conference organisers agreed to amend the topic to "factors affecting the movement of retaining structures". This allowed for the consideration of a wider range of factors than simply excavation sequence.

In his paper, the Candidate summarised the most important factors as follows (Day, 1994):

Category	Factors
Support system	Stiffness of support system Permitted relaxation of supported ground Vertical restraint of the wall Position of load transfer to the soil
Ground properties	Stiffness of supported material Soil/rock structure In situ stress conditions
Construction methods	Over-excavation or inadequate support Localised collapse of face Drilling and grouting methods Dewatering

Each of these factors was illustrated by reference to examples from the Candidate's experience with lateral support design, most of which had been carried out on behalf of geotechnical contractors. It was ironic that, during the research carried out for the production of this paper, the Candidate discovered one of the compelling reasons for not using plastic design of soldier piles as advocated in the work described above.

By far the most interesting factor that influences the movement of retaining structures is the stiffness of the support system itself. The paper looked at three aspects of support stiffness, namely lateral stiffness (the obvious one), vertical stiffness and flexural stiffness. It used the published movement records from a number of excavations to illustrate the role which each of these stiffnesses play. One of the examples used was the

underground parking garage for the House of Commons in London. This structure was constructed from the top down. Prior to any excavation, a permanent diaphragm wall was installed to serve as the exterior wall of the structure and piles were installed to form the internal columns. The ground floor slab was then cast to strut the top of the diaphragm wall. After curing of the slab, the soil below was mined out to the next floor level and the next slab cast, and so on to the bottom of the excavation.

Figure 15, shows the deformed shape of the diaphragm wall at each excavation stage as reported by Burland and Hancock (1977).

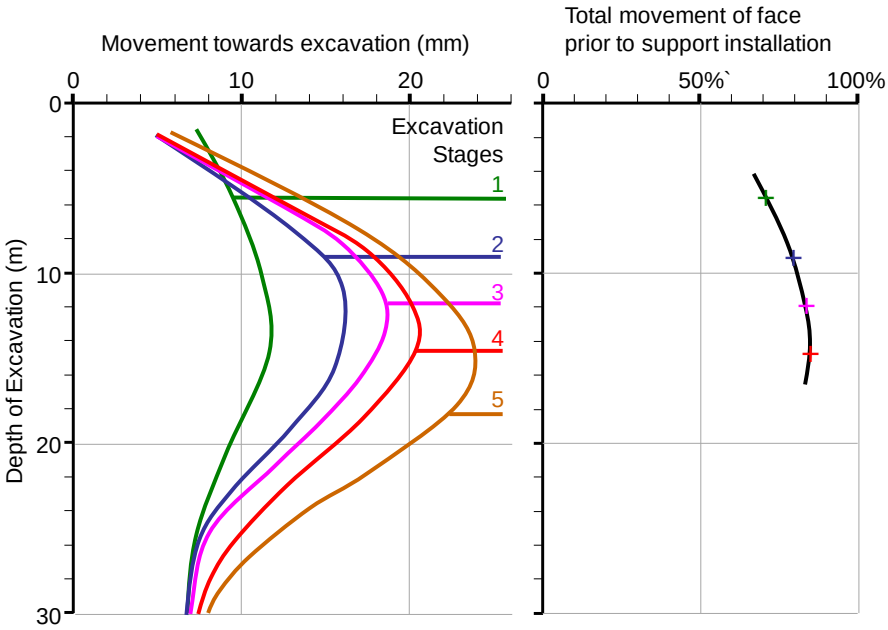


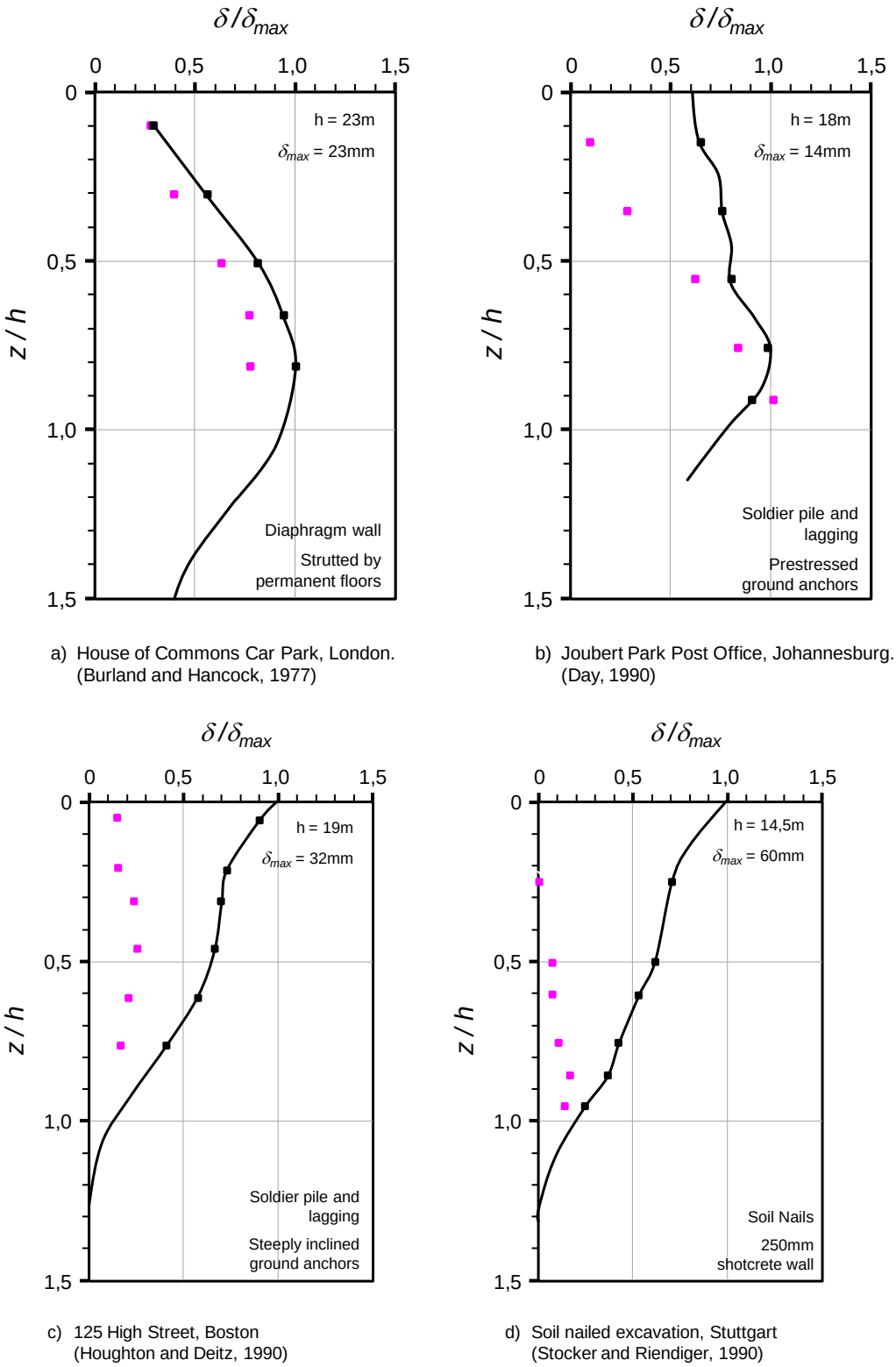
Figure 15: Excavation movements for House of Commons car park (after Day 1994)

From this figure, it can be seen that a large proportion of the movement of the diaphragm wall at any level occurred prior to installation of support at that particular level. Thus the imbalance of the earth pressures on the outside and the inside of the wall below excavation level was sufficient to cause flexure of the wall well below the current depth of excavation. The movements after casting of the floor slabs were relatively modest, constituting only 15% to 30% of the total movement. This illustrates the effects of the limited flexural stiffness of the diaphragm wall and the high axial (lateral) stiffness of the floors.

In all, five such case histories were analysed, including the Joubert Park Post Office excavation. These are summarised in Figure 16, again taken directly from the paper. In each of the diagrams in this figure, the final deflected shape of the excavation face is given, the dots on the lines indicating the movement at the support points (anchors, struts, etc.). The dots to the left hand side of the line indicate the movement of the face that occurred prior to installation of the support at that particular level. The horizontal distance between the dots is therefore the deflection of the face after the installation of the support.

Diagram (a) in this figure shows the movements recorded on the House of Commons excavation described above. Diagram (b) shows the results for the Joubert Park Post Office excavation where the low lateral stiffness of the upper rows of anchors is clearly evident. Diagram (c) (125 High Street, Boston) and Diagram (d) (soil nailed excavation in Stuttgart) are classic examples of low lateral stiffness provided by steeply inclined ground anchors and soil nails respectively.

The paper also dealt with factors such as inclination of anchors, hand-over-hand methods, localised face collapse, over-excavation, delayed support installation and poor drilling and grouting techniques.



a) House of Commons Car Park, London. (Burland and Hancock, 1977)

b) Joubert Park Post Office, Johannesburg. (Day, 1990)

c) 125 High Street, Boston (Houghton and Deitz, 1990)

d) Soil nailed excavation, Stuttgart (Stocker and Riendiger, 1990)

Figure 16: Movement of lateral support after support installation (Day, 1994)

This paper forms the basis of a lecture on movement of excavations presented bi-annually by the Candidate to post graduate students at the universities of the Witwatersrand and Pretoria. The content is augmented by information on typical ground movements, acceptable movement magnitudes and methods of predicting and measuring movements. A number of additional case histories are presented including some more recent problem excavations around the Cape Peninsula where the Candidate has acted as an expert in insurance investigations or legal proceedings.

One of the lessons from the investigation that went into this paper is the importance of the flexural stiffness of any vertical wall element installed before commencement of excavation (e.g. soldier pile or diaphragm wall). This flexural stiffness controls the deformation of the excavation face *prior* to installation of the lateral support (anchors, struts, etc.), including movements that occur below the level of the bottom of the excavation. Therefore, very flexible wall elements should only be used in stiff soils where movements are likely to be limited in any event. Some of our residual soils, e.g. the residual soils derived from the Ventersdorp lavas and the shales and quartzites of the Witwatersrand Supergroup in Central Johannesburg, fall into this category. For this reason, the use of soldier piles designed to permit plastic hinging as investigated by Howie (see 4.3.3 above) should be limited to such soils.

In the Candidate's experience, the residual granites in the Sandton area of Johannesburg do not behave in the same way as the stiff residual soils in the Johannesburg CBD. The residual granites appear to be very unforgiving of any deficiencies in the design of the lateral support system and surprisingly large movements can occur in these materials. A possible reason for this is that, although these soils have a higher frictional strength than the finer grained residual soils in the CBD, they have lower cohesion. As the cohesion is generally ignored or at least significantly down-rated by most prudent designers, the designs in the finer-grained residual soils tend to be more conservative with a corresponding reduction in movement. Although many tests on residual granites indicate friction angles in excess of 35°, the Candidate's experience is that the use of friction angles higher than 32° in the residual granites is inviting problems with movement of the excavation.

4.3.5 Other Contributions

<u>National Report, braced excavations in soft ground (1994)</u>

As a satellite event to the ICSMFE in New Delhi, the ISSMGE Technical Committee for underground construction (TC28) held an International Symposium on Underground Construction in Soft Ground. The Candidate was South Africa's representative on this technical committee. Each member society was requested by the symposium organisers to prepare a 3 page national report. South Africa's report (Day and Schwartz, 1994) was written by the Candidate and Ken Schwartz, a consulting engineer also engaged in design and construct projects with South African geotechnical contractors.

The report dealt with aspects of the local lateral support industry including the occurrence and nature of the soft soils, the size of the market, design methods used, monitoring and instrumentation, codes of practice and contracting procedures.

<u>XI ARCSMFE, Cairo (1995)</u>

At the African Regional Conference on Soil Mechanics and Foundation Engineering held in Cairo in 1995, the Candidate, a fellow geotechnical consultant and a geotechnical contractor presented a paper on the design, construction and performance of deep

basement excavations in South Africa and Zimbabwe (Day, Wardle and Krone, 1995). This paper went further than simply presenting case histories of successful projects but concentrated on how problems were dealt with during construction and the challenges faced by the industry.

The excavations ranged in depth from 4m to more than 20m. One of the excavations, for the mills at the Columbus Stainless Steel plant, was so complex that a model had to be built to illustrate the layout of the faces and how one face of the excavation would interact with those in close proximity. Each case history gave the excavation geometry, soil profile, design methods and parameters, support installation and excavation movements. Where problems were experienced, the paper described the ways in which these were addressed.

In the conclusions, the authors listed the two major challenges being faced by the lateral support market as quality of the site investigation data and the unrealistic pressures placed on contractors by project managers.

General Report on deformations of braced excavations (1999)

In 1999, the Japanese Geotechnical Society hosted an international symposium on geotechnical aspects of underground construction in soft ground known as IS Tokyo 99. This symposium was co-hosted by the ISSMGE's TC28, of which the Candidate was a member. The organisers invited the Candidate to present the general report on deformation and displacement of braced excavations (Day, 1999). The report examined 21 papers out of the 42 papers presented at the conference, all of which dealt with deformation of excavations.

The report looked at three particular aspects of the case histories presented in these 21 papers, namely methods of analysis and construction, movement predictions and the observational method, and control of deformations. The analysis methods generally fell into two categories, finite elements and subgrade reaction methods, most of them allowing for non-linear analysis. One case history made use of a frame analysis for the prediction of movements. About half of the case histories used diaphragm walls as part of the construction. The remainder were secant piles, sheet piles or deep soil mixing. Many of the authors referred to the use of the observational method for the control of movements during construction. The interventions introduced as a result of the observations made included both a reduction and increase in the support initially provided. With one exception, the reported deformations ranged from 0,03% to 0,73% of the depth of the excavation with an average of 0,35%. This is much the same range as we experience in South Africa. The lowest deformations were for two circular structures where the perimeter walls acted in hoop compression.

In the conclusion to the paper, the Candidate again noted the relationship between the stiffness of the support system and the movement of the excavation faces. However, a new element emerged, namely the stiffness of the soil in the passive zone in front of the wall. In particular, many authors reported on the adverse effects of any disturbance of this soil due to construction activities such as piling, installation of drains or jet grouting. Some authors also noted the need to take account of the reduction in stiffness of the basement walls due to cracking of the concrete.

Stellenbosch University http://scholar.sun.ac.za

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5. PILE DESIGN AND CONSTRUCTION PRACTICE

5.1 Background

As indicated in Part 1 of this dissertation, the first piling company to be established in this country was Maclaren & Eger in 1928. Almost twenty years later, the Franki Piling Company of South Africa was formed as a subsidiary of the Belgian company Frankipfahl which was established in about 1910 and was holder of the patent for the now well-known Frankipile (a driven displacement cast in situ pile). Today, there are about ten reasonably sized piling contractors in South Africa and many more small piling and underpinning contractors. Neither Maclaren & Eger nor Frankipile still exist in their original form having been taken over by or merged with other companies. The piling industry is probably one of the more "incestuous" in the construction arena with personnel from one company frequently popping up in a different coloured pair of overalls with a different company logo embroidered on the pocket.

The pile types that formed the backbone of the industry include augered (or bored) piles, precast piles, steel tube and H piles, oscillator piles, Frankipiles, percussion bored piles and underslurry piles. Of these, some were best suited to the deep alluvial sediments along the coast while others had application in the partially saturated residual soils of the interior. New techniques that have been introduced into the market include continuous flight auger (CFA) piles, down-the-hole percussion drilled piles, micropiles and full-displacement screw piles.

A number of local piling conferences have been held over the years. The most memorable of these was the "Piling Panorama" symposium hosted by the Concrete Society of Southern Africa in 1980. This was followed in 1988 by two courses on *Pile Design and Construction Practice* and *Design of Laterally Loaded Piles* organised by the Geotechnical Division with the Candidate as chair of the organising committee. Professor Lymon Reese of the University of Texas, Austin was the guest presenter of this course. Some time later, in March 2007, the Geotechnical Division again hosted a conference on Pile Design and Construction Practice with Dr Hillary Skinner of the UK and Dr Fiona Chow of New Zealand as guest speakers.

There are no codes of practice relating to piling in South Africa. SABS 088:1972 Piled Foundations was withdrawn some years ago on account of it being out-dated. The most frequently used reference in the industry is the "Frankipile book". The *Frankipile Guide to Piling and Foundation Systems* was first published in 1976 and is now in its 4th edition under the name of *A guide to Practical Geotechnical Engineering in South Africa*. The third edition of this book received the SAFCEC Presidential Award in 1995.

What follows is not an account of the development of the industry but a record of the Candidate's involvement, typically in the role as contractor's designer on design-and-construct projects.

5.2 Underslurry Piling Research at the University of Natal

5.2.1 Use of Underslurry Piles

The east coast of South Africa has a submerged coastline. During the Weichsellian regression approximately two million years ago, sea levels dropped 100m or more (Brink, 1985). This caused the rivers along the coast, the KwaZulu-Natal coast in particular, to incise their lower courses and exposed large areas of the continental shelf. Freshly exposed shelly sand on the seabed was redistributed by wind to form long-shore dunes. During the subsequent Flandrian transgression, these incised river valleys were

submerged. The long-shore dunes acted as barriers resulting in the formation of numerous lagoons and estuaries along the coast. The submerged valleys began filling with soft alluvial and estuarine sediments. In the Durban area, the Mgeni, Mbilo and Mlazi rivers formed an extensive inter-linked lagoonal system resulting in a complex and highly variable succession of sediments known as the Harbour Beds. These compressible sediments extend below much of central Durban as we know it today.

The presence of sand layers in the sediments combined with a high water table make it impossible to install conventional bored (open hole) piles in the harbour beds. As a result, the most popular pile types used in the area and for many of the bridges along the KwaZulu-Natal coast were driven precast piles, oscillator piles and underslurry piles.

An underslurry pile is constructed by boring the pile hole with a flight or bucket auger while keeping the hole filled with a bentonite slurry to a level a few metres above the level of the water table. The higher head of bentonite slurry in the hole and the marginally higher density of the slurry compared with the surrounding ground water prevents the ground water from entering the hole. As the bentonite slurry seeps into the surrounding soil, it deposits a filter cake of slurry mixed with the in situ sand on the sidewalls of the hole. The pressures on the sidewall caused by the excess head of bentonite in the hole acting on this low permeability filter cake preserve the stability of the bore. During drilling, the bentonite slurry is circulated through a de-sanding plant that removes the sand and other drilling spoils from the bentonite.

Once the pile hole has been formed to the required founding stratum, the base of the hole is cleaned by air-lifting any remaining spoil from the pile socket while continuing to de-sand the bentonite to achieve a sand content of less than 3%. The steel reinforcing cage is then inserted into the hole and the pile shaft is concreted from the bottom up using standard tremie concreting techniques, thereby displacing the bentonite slurry (Frankipile, 2008).

The rising concrete removes some of the bentonite filter cake from the sidewalls of the hole. Nevertheless, a weak layer of sand-bentonite filter cake remains in place between the shaft of the pile and the surrounding ground. This layer will have an effect on the skin friction between the pile shaft and the soil and, as a consequence, on the ultimate load capacity of the pile.

5.2.2 Research at the University of Natal

In the early 1970's, pile load tests showed that underslurry piles were capable of carrying far higher loads than previously thought possible (Wates, 1974 and Day et al, 1981). Research at the university had confirmed that the part of the filter cake was not removed during the concreting operation and that the thickness and composition of this layer was dependent on the soil type and the time for which the hole remained filled with bentonite (Scott, 1978). Although the angle of shearing resistance of pure bentonite is as low as 8°, it was shown that the shearing resistance of pure bentonite increases substantially in the immediate proximity of hydrating concrete. Tests carried out on samples of filter cake collected from the sides of piles showed the strength of the filter cake to be largely frictional with angle of shearing resistance of 21° to 38° being recorded. This increase in shear strength was attributed in part to contamination of the filter cake with in situ material and with fine aggregate from the concrete.

As a result of the frictional strength of the filter cake, the skin friction that can be generated on the pile depends on the radial pressure on this layer (Schreiner, 1978 and Day 1980). Schreiner conducted a series of model tests to determine the degree of consolidation of the filter cake that is achieved due to the pressure exerted on this layer by the fluid concrete. In the absence of any movement of the soil, full consolidation of the

filter cake will result in the radial pressure on the filter cake being equal to the pressure exerted by the concrete prior to undergoing initial set. If the filter cake is not fully consolidated by the fluid concrete, dissipation of the remaining excess pore water from the bentonite will cause the thickness of the filter cake to reduce giving rise to an inward movement of the soil around the pile. The magnitude of the resulting decrease in radial stress on the pile shaft will depend on the extent to which the radial stresses in the soil reduces as a result of this movement. In a clayey soil, the movement of the ground around the pile is time dependent and will be governed by the theory of consolidation.

The research described in the remainder of this section 5.2 was carried out by the Candidate in partial fulfilment of the requirements for an MSc Eng degree at the University of Natal in the mid 1970's.

5.2.3 Three Dimensional Consolidation Theory

It was expected that the consolidation of the soil around a pile hole would be affected by the three dimensional nature of the problem.

A poro-elastic theory of three dimensional consolidation was first developed by Biot in 1935 and subsequently refined to cater for such refinements as compressible pore fluid, fluid viscosity and partially saturated soils (Biot, 1941). Using this theory, Mandel (1953) showed that if a rectangular prism drained from the vertical faces was subjected to a sudden increase in the pressure applied to the horizontal faces, there would be a temporary rise in the pore water pressure near the centre of the cube. This became known as the Mandel-Cryer effect. This effect was demonstrated by Gibson et al (1963) by subjecting a sphere of clay, drained on the surface, to a sudden increment in the applied external pressure and measuring the pore pressure response at the centre of the sphere. The pore water pressure at the centre of the sphere was found to rise in excess of the applied stress increment before decaying as drainage took place.

This rise in pore water pressure can be explained by considering the temporal and special variation in Poisson's ratio as consolidation progresses (Day, 1977). Immediately after the load is applied, the clay is undrained and the Poisson's ratio is 0,5 throughout (i.e. no volume change). As drainage takes place from the surface of the sphere, the Poisson's ratio drops to its drained value and the material reduces in volume. However, at the centre of the sphere, the Poisson's ratio remains at 0,5 and no volume change takes place. The combined effect of the reduction in stiffness of the material around the outside of the sphere and the relatively incompressible material in the centre of the sphere causes a redistribution of total stresses with the less compressible central regions of the sphere "attracting" stress. This effect is illustrated in Figure 17.

5.2.4 Numerical Solution of the Biot Equation

Before further progress could be made on the application of three dimensional theory to the consolidation of soil around a pile hole, a numerical solution needed to be developed for the Biot equation. The Candidate set about doing this as part of his research towards an MSc Eng degree.

The approach adopted (Day, 1977) was to determine the pore pressure distribution within the sphere using finite difference methods thereby enabling the degree of consolidation at any radius to be computed. Using the relationships between degree of consolidation and Poisson's ratio developed by Mandel together with the equivalence of the shear modulus of the soil in total and effective stress, it was possible to determine the Poisson's ratio and elastic modulus of the soil at any radius within the sphere. These parameters were fed into a linear-elastic finite element analysis to determine the stress distribution within the

sphere. In a saturated soil, the change in total stress generates an equivalent change in pore water pressure. These pore water pressure changes were then fed back into the finite difference analysis and the process repeated for successive time steps.

The results of this numerical analyses for a Poisson's ratio of ν' = 0,0 and 0,33 are shown superimposed on the exact solution by Gibson et al in Figure 17. The agreement is acceptable.

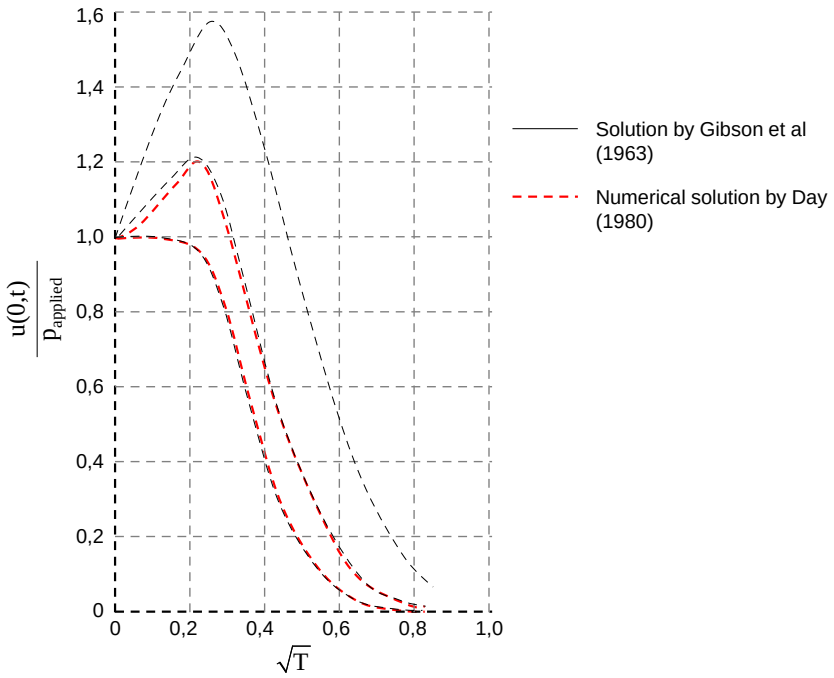

Figure 17: Pore pressure at centre of consolidation sphere

As an aside, the solution to this "coupled consolidation" problem is now standard in finite element and finite difference analysis programmes that are available off the shelf and run on desk-top computers. At the time this research was done, the finite element programme used was written in FORTRAN and ran on the university's Burroughs B5700 main frame computer. The finite difference programme was written by the Candidate to run on a one of Hewlett Packard's first programmable desk-top computers with a single line display and a built in thermal printer. Given the slow speed of the mainframe and the demand for computing time from the rest of the university, most of this work needed to be done at night. Each iteration in the process could take up to an hour to complete.

A further aside is that this solution process was picked up by researchers at the Bernard Price Institute at the University of the Witwatersrand who used it in an attempt to correlate changes in the polarity of the earth's magnetic field with periods of intense volcanic activity. Changes in pressure on the earth's core due to venting of deep mantle fluids result in the precipitation of either slag or iron on the earth's core, a process that can be correlated with changes in the polarity of the magnetic field. These changes in polarity are recorded in the ocean floors either side of mid-oceanic ridges. Previous attempts to correlate periods of volcanic activity with these changes in the polarity appeared to be out of phase as it was assumed that venting of deep mantle fluids from the surface would reduce the pressure on the earth's core. The Mandel-Cryer effect provided them with fresh impetus for their work. This resulted in two papers in which the Candidate was a co-author (Nicholaysen, Day and Hoch, 1982 and 1984).

5.2.5 *Application to a Pile Hole*

The solution developed was sufficiently general to be applied to any problem that could be analysed in two dimensions or as an axisymmetric solid, including the consolidation of soils around a pile hole.

However, as it turned out, the solution simply served to demonstrate that the consolidation of soil around a pile hole does not exhibit any three dimensional effects. This is because the Biot equation assumes the change in pore water pressure generated by a change in total stress on the soil is equal to the change in the average of the three principal stresses. In the case of a circular hole in an elastic solid, the change in the radial stress is equal and opposite to the change in the tangential stress. The vertical stress remains constant. Thus no excess pore water pressures are generated by changes in the pressure inside the hole and the resulting movement of the ground is instantaneous. The theory was, however applied to barrettes (oblong piles) and to circular piles by modifying the pore pressure response in line with Skempton's A- and B-parameters (Skempton, 1954). The nett result was not significantly different.

5.2.6 *Outcome*

Figure 18 sums up the findings of this research (Day et al, 1981) as presented to the ISSMFE International Conference in Stockholm in 1981. In order to satisfy the requirements of equilibrium and compatibility, the radial stress in the soil and the normal stress on the filter cake must be equal, and the inward displacement of the soil and the change in thickness of the filter cake after setting of the concrete must also be equal. This occurs at the intersection of the load deflection curves for the soil and the filter cake given in Figure 18.

The dotted lines in this figure represent the stiffness of the filter cake taking account of its thickness and composition. The intercept on the vertical axis represents the extent to which the filter cake has consolidated prior to setting of the concrete. The solid lines are the pressure – deflection curves for the soil around the pile and are shown for two soil stiffnesses and three pile diameters.

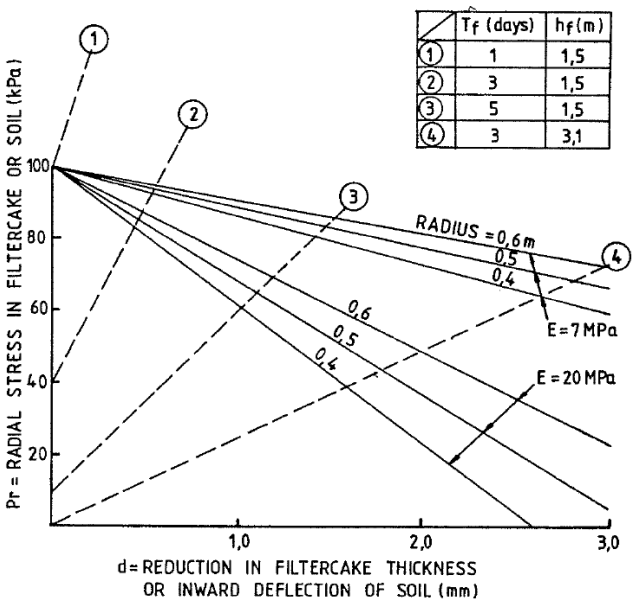

Figure 18: Load deflection curves for filter cake and soil

The numeric values for the load - deformation of the filter cake given in the figure (dotted lines) are indicative only.

The following conclusions were drawn from the combined research of Wates, Scott, Schreiner and Day (Day et al, 1981):

i. High filtration times (T_f) and high filtration heads (h_f) produce thicker and more compressible filter cakes.

ii. If the concrete is cast within 24 hours of completion of boring, the final stress on the filter cake is equal to the stress exerted by the wet concrete as the filter cake is fully consolidated by the time the concrete sets.

iii. Where the filter cake has not completely consolidated when the concrete sets, the final stress on the filter cake is dependent on the stiffness of the ground – the stiffer the ground, the lower the final stress.

iv. The final stresses are higher for larger diameter piles.

v. In the absence of creep, the residual stress on the filter cake is dependent on the pressure exerted by the wet concrete and not on the in-situ stress in the soil. Creep will increase the radial stress on the filter cake over time.

Day et al (1981) describe a full scale load test on a pile below a bridge over the Umlaas Canal. The back-figured value for the angle of shearing resistance from this pile is 34° which falls within the 21° to 38° range recorded by Scott.

5.3 Reinforcement of Cast in situ Piles

At the time of the 1998 conference on Pile Design and Construction Practice, there was much debate in the industry regarding the design of reinforcement for cast in situ piles. In the words of Ian Braatveldt in the first edition of the “Frankipile Book”:

> *: ... the allowable concrete and reinforcement stresses in piles and the minimum amount of reinforcement required .... are not specified (by the codes) but are covered by recommendations, and these recommendations are inconsistent.”*

The two piling codes available at that time were *SABS 088-1972: Piled foundations* and the British *CP 2004-1972: Foundations*. On the subject of the design of reinforcement, these two codes referred the reader to the Standard Building Regulations and CP114 respectively. The latter was no longer in use at the time due to the introduction of limit states design codes for concrete design. The general provisions of these references did not necessarily apply to piles where possible aggressiveness of the soil requires consideration of crack widths rather than limiting stresses in the concrete or reinforcement. In view of this situation, many designers chose to apply the recommendations of BS 5337-1976 for water retaining structures.

In order to address this situation, the Candidate presented a paper at the 1998 conference (Day, 1998) giving a summary of the recommendations of the various codes and guidelines. He then proposed clear recommendations for the design of pile shafts using either working load design or limit states design methods.

With regard to working stress design, it was recommended that steel reinforcement stresses under long term loading be limited to the values specified in BS 5337 be adopted to control crack widths. For transient loads, the limiting steel stresses in CP 114 could be used. Concrete stresses were limited to those given in the “Frankipile Book” which

recommended concrete stresses depending on the diameter of the hole and whether or not the hole was cased. It was recommended that the design of shear reinforcement in piles be based on shear friction theory or on the analysis of an equivalent rectangular section.

At the time, the only design charts available for circular sections with appropriate steel and concrete stresses were those given in the Frankipile Book. These charts were for specific pile diameters. To overcome this problem, the Candidate produced non-dimensional design charts for circular sections in a similar format to those used in Part 3 of CP 110 based on cracked section analysis.

For limit states design, it was recommended that the design charts for circular sections given in CP 110 Part 3 be used but that the characteristic strength of the concrete should be reduced by 10MPa to allow for the method of placement, possible contamination and lack of adequate compaction. This reduction could be reduced to 5MPa for cased piles or clean rock sockets. The paper recommended an explicit check of crack widths using the method given in BS 8110 Part 2 with crack widths limited to 0,3mm for transient loads and between 0,1mm and 0,2mm under permanent loads depending on the conditions of exposure.

The paper also gave clear recommendations for the detailing of pile reinforcement in accordance with the codes and experience gained on site with heavy reinforcing cages.

An example was given of the design of a four pile group using both working load and limit states design demonstrating the use of the recommendations and also the near-equivalence of the two design methods.

5.4 Free-fall Placement of Concrete in Bored Piles

5.4.1 Background

In South Africa, it is common practice to cast concrete into bored pile holes directly from the chute at the back of the truck-mixer, allowing the concrete to fall freely to the bottom of the hole. This practice is frequently queried by structural engineers, many of whom insist on pouring concrete using a tremie even when the hole is dry. Often, this inappropriate use of the tremie causes more harm than good to the quality of the concrete due to the unnecessary complications introduced into what should be a straightforward piling activity.

In order to investigate the effect of free fall placement of concrete in pile holes, a series of tests were carried out by the author in 1991 under the auspices of the Research and Development Advisory Committee of the South African Roads Board. The results of these tests were first presented at a series of Joint Structural Division Courses on the Design of Foundations to suit Various Soil Conditions. They were partially written up for the 2007 SAICE Geotechnical Division Pile Design and Construction Conference (Day, 2007) and formally presented by Prof. George Fanourakis at the Pan-Am CGS Geotechnical Conference in Toronto in October 2011 (Fanourakis, Day and Grieve, 2011). A revised local version of the paper has recently been published in the SAICE Journal (Fanourakis et al, 2012).

5.4.2 Code Requirements

Clause 5.5.5.5 of SABS 1200G-1982 (Structural Concrete) requires that concrete not be allowed to fall freely through a height of more than 3m unless otherwise approved. It also requires that concrete be compacted by mechanical vibration or other approved methods (Clause 5.5.6.3). These are normal requirements for structural concrete where the slump is typically less than 75mm.

SABS 1200F-1983 (Piling) specifies a concrete slump of between 75mm and 175mm for various conditions depending on the method of placement, spacing of reinforcement and diameter of the pile hole (Clause 5.5.2.1(b)). In sub-sections (c), (d), (g) and (h) of the same clause, the code recommends that internal vibrators should not be used, that concrete should be placed in the dry or by means of a tremie, that concrete be placed in such a way that segregation does not occur and advocates the use of a chute extending far enough into the hole to ensure that the concrete drops vertically when leaving the chute. In the case of raking piles, the chute is required to extend to the leading edge of the newly placed concrete. Read together, these clauses from SABS 1200F imply that a free fall placement of concrete is permitted in vertical pile holes provided that the hole is dry and the concrete is permitted to fall unobstructed down the centre of the pile.

5.4.3 *Objectives and Methodology*

The aims of the research were to investigate

i. whether the placement of concrete by free-fall method results in segregation or loss of strength;

ii. to what extent the presence of water in the pile hole affects the integrity of the concrete; and

iii. what happens to any spoil remaining at the bottom of the hole during concreting.

The investigation was carried out by casting a number of trial “piles” using free-fall placement of the concrete with various amounts of water and spoil at the base of the pile. The “piles” consisted of 200 litre steel drums placed at the bottom of a 6m deep, 1,5m diameter auger hole with a 500mm diameter light-weight steel casing inserted about 100mm into the top of the drum to simulate the sidewalls of the pile shaft. A 50mm concrete blinding layer was cast at the bottom of each drum to provide a solid base. After placement of a measured depth of spoil and/or water in the base of the drum, concrete was discharged into the drums down the centre of the casing into each drum in turn as illustrated in Figure 19. On completion of the pour, the drums were lifted from the hole and left to cure on surface.

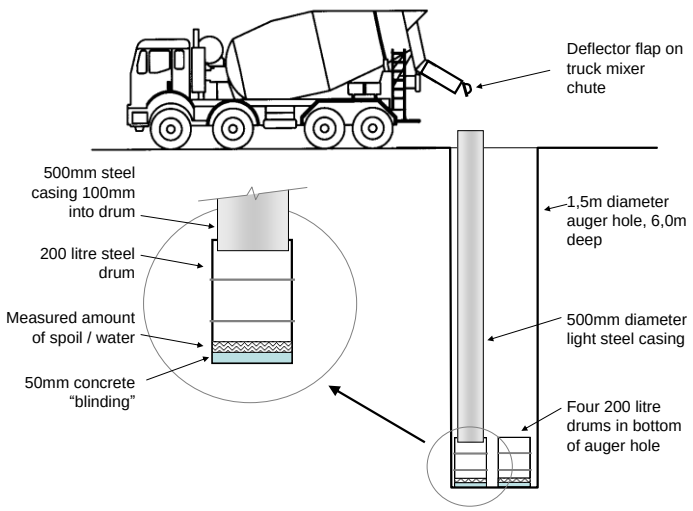

Figure 19: Method of casting test “piles”

In addition to casting concrete in this manner, two drums were filled with concrete on surface and vibrated with a poker vibrator to serve as control samples. All the concrete was supplied from the same batch plant according to the same specification. Excess concrete left in the truck mixer after pouring of working piles was utilised for these experiments.

After curing for two weeks, 100mm diameter core samples were taken from the bottom of each drum and were submitted for laboratory testing. The bottom of each drum in which spoil was placed was then cut away to observe the extent to which the spoil had been displaced by the falling concrete. In the laboratory, unconfined compressive strength tests and density determinations were undertaken. The percentage excess voids was assessed visually. Aggregate : cement ratios were determined for the nine samples of concrete cast through various depths of water using the soluble silica test method.

5.4.4 Results

Table 5 summarises the conditions under which the various "piles" were concreted and tabulates the results of the laboratory tests (Day, 2007).

Table 5: Summary of results from tests on concrete core

Test Ref.	Sample Ref.	Concrete Batch	UCS (MPa)	UCS (% of control)	Actual Density (kg/m³)	Excess Voids (%)	Aggregate / Cement Ratio	Test Conditions
C1	7B*	C1	51,0	100	2 450	0,0	9,5	Control test, vibrated
C2	7T*	C2	39,0	100	2 583	0,0		Control test, vibrated
C3	18B	C3	43,0	100	2 620	0,0		Control test, vibrated
C4	18T	C4	40,5	100	2 634	0,0		Control test, vibrated
W1	3B	C1	48,5	95	2 540	1,0	9,2	Free fall, dry
W2	13B	C4	48,5	120	2 611	1,5	11,9	Free fall, dry
W3	1B	C1	37,5	74	2 393	0,5	9,3	Free fall, 50mm water
W4	6B	C2	38,0	97	2 490	0,5	9,0	Free fall, 50mm water
W5	2B	C1	25,0	49	2 517	0,5	7,6	Free fall, 100mm water
W6	8B	C2	23,5	60	2 508	1,0	12,2	Free fall, 100mm water
W7	4B	C1	9,0	18	2 454	3,0	13,2	Free fall, 200mm water
W8	9B	C2	8,5	22	2 428	4,0	14,4	Free fall, 200mm water
W9	5B	C2	7,0	18	2 434	10,0	16,9	Free fall, 400mm water
W10	12B	C3	10,0	23	2 407	15,0		Free fall, 400mm water
S1	10B	C3	50,0	116	2 580	0,5		Free fall, 50mm silt, dry
S1	10T	C3	42,0	98	2 546	1,5		Free fall, 50mm silt, dry
S2	11B	C3	21,5	50	2 540	1,0		Free fall, 50mm silt, 100mm water
S2	11T	C3	22,0	56	2 522	1,0		Free fall, 50mm silt, 100mm water
S3	17B	C4	48,5	113	2 546	1,5		Free fall, 50mm silt, 50mm water
S4	14B	C4	50,5	125	2 564	2,0		Free fall, 50mm c.dust**, dry
S5	15B	C4	46,0	114	2 506	1,5		Free fall, 50mm c.dust, 50mm water
S6	16B	C4	31,0	77	2 569	1,0		Free fall, 50mm c.dust, 100mm water
R1	19B	C4	25,5	63	2 518	1,5		Free fall, with rebar, 100mm water
R2	20B	C4	20,0	49	2 637	1,5		Free fall, slow pour, 100mm water

Notes: * T indicates top of drum, i.e. about 700mm above bottom of pile
B indicates bottom of drum, i.e. at bottom of "pile"

** c.dust indicates crusher dust (sandy fines from crushed aggregate).

The effect of the depth of water in the pile hole prior to commencement of concreting on the strength of the concrete is shown in Figure 20. Figure 21, Figure 22 and Figure 23 show the effect of water depth on the actual density of the concrete, the percentage excess voids and the aggregate : cement ratio respectively.

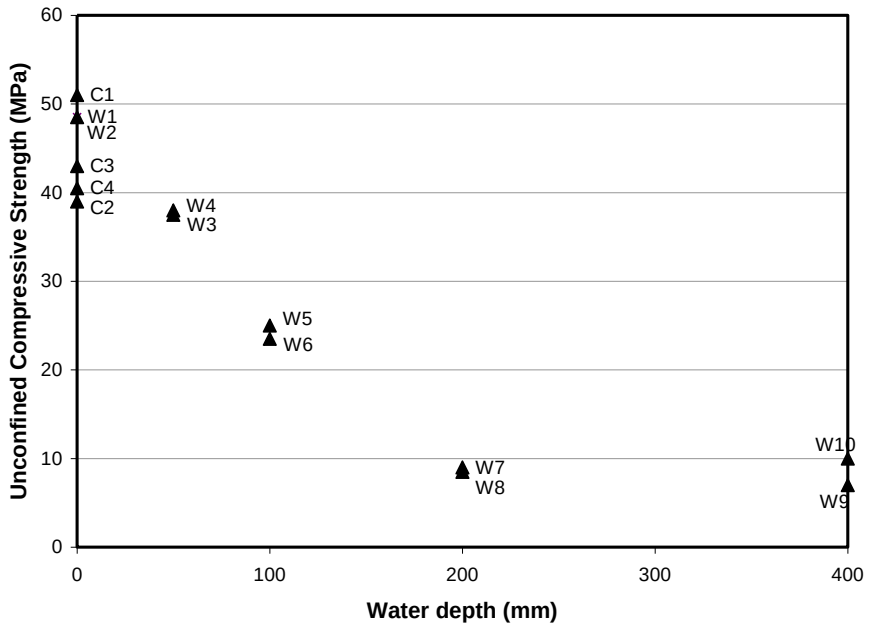

Figure 20: Effect of water depth on unconfined compressive strength (100mm core)

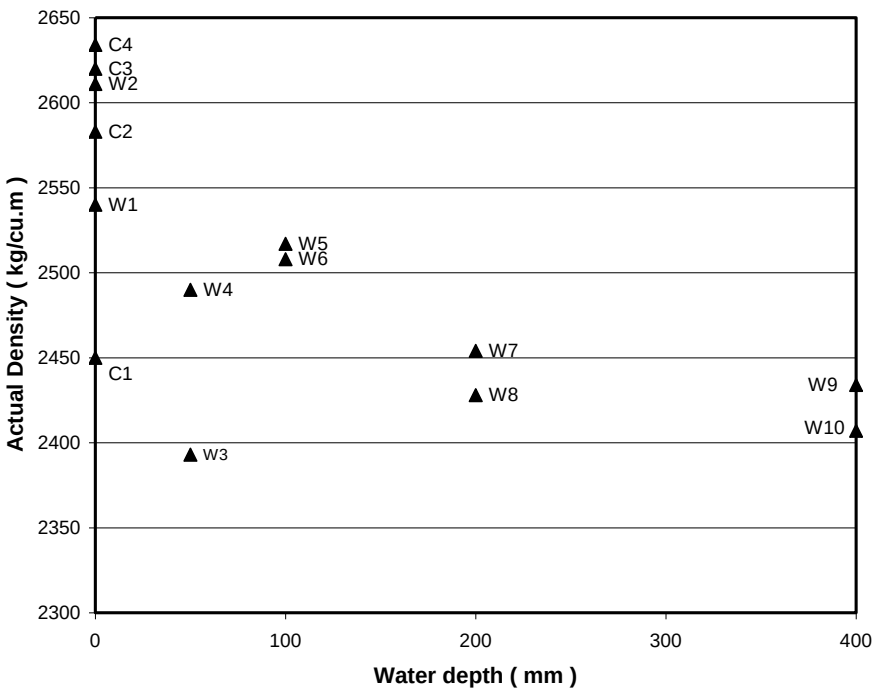

Figure 21: Effect of water depth on actual density

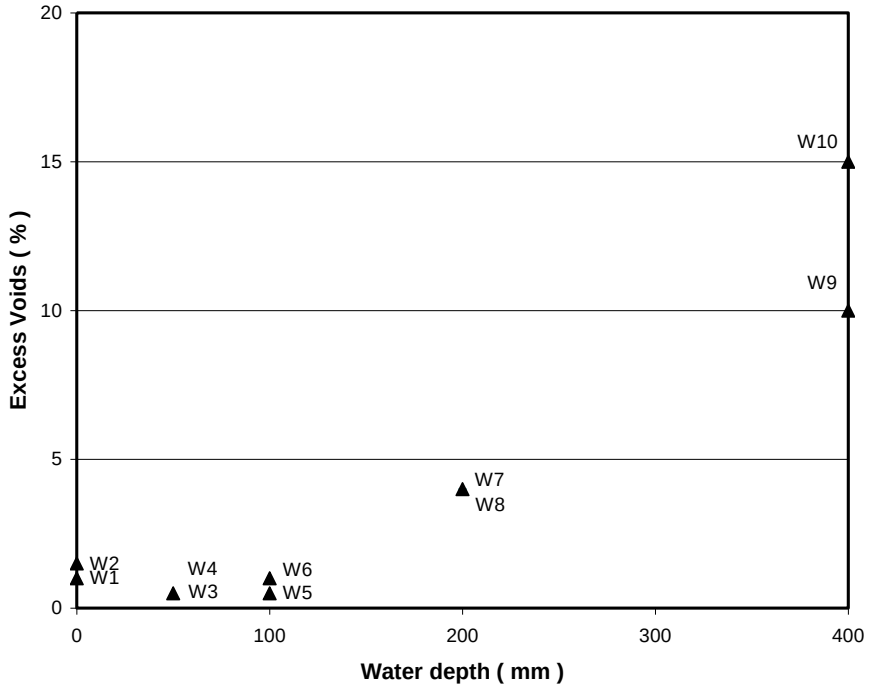

Figure 22: Effect of water depth on percentage excess voids

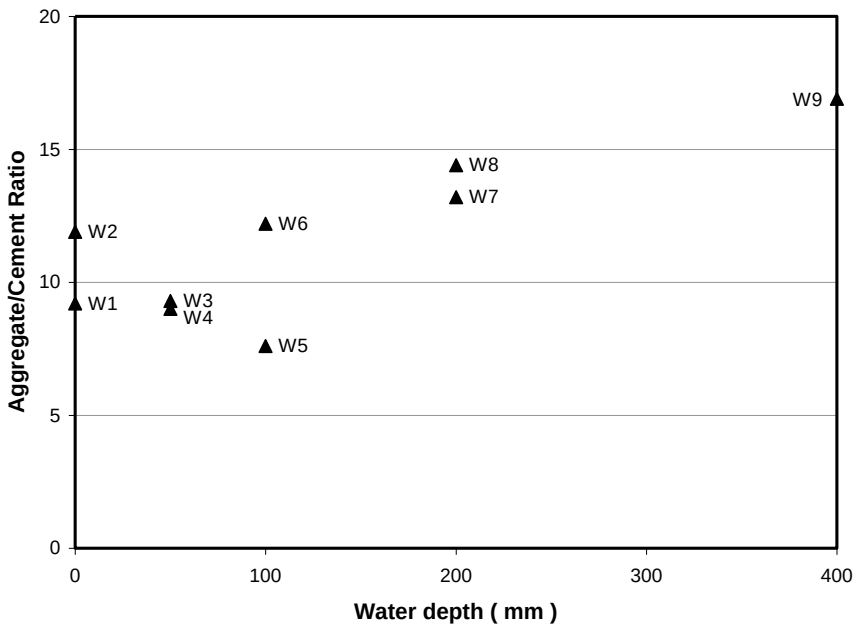

Figure 23: Effect of water depth on aggregate : cement ratio

5.4.5 *Discussion*

Segregation due to free fall placement

Photo 9 shows cores drilled through the bottom of the “piles” cast through various depths of water. The bottom of the core is facing away from the reader. The contact between the 50mm blinding concrete cast in the drums and the “pile” concrete is visible in some of the cores.

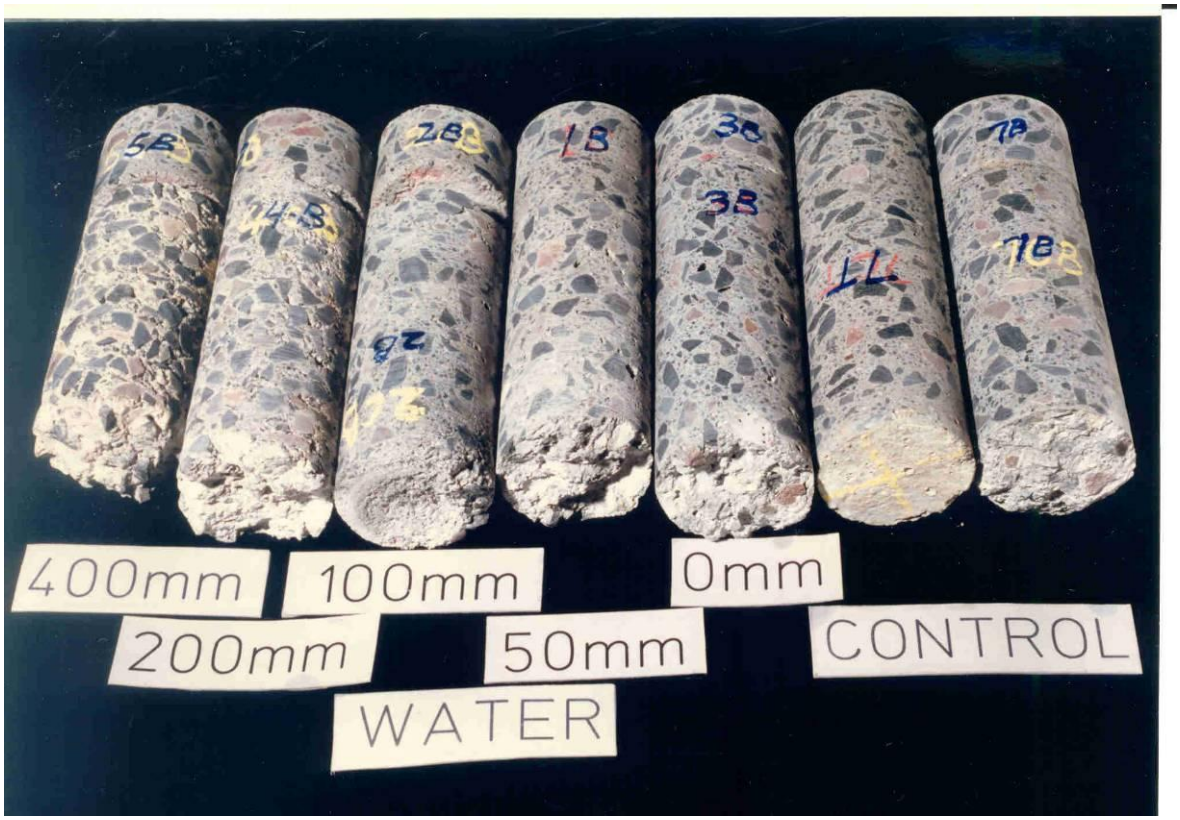

Photo 9: Concrete cores from concrete cast into water in pile hole.

In all these cores, there was an even distribution of aggregate, i.e segregation did not occur. The only case where segregation was evident was where the concrete was poured slowly into 100mm of water as shown in Photo 10.

In this photo, the bottom of the core is to the left of the picture. The disk of blinding concrete has separated from the pile concrete. The pile concrete shows classical signs of segregation, with unbonded aggregate at the toe of the pile and decreasing aggregate content with the accumulation of fines and latence towards the top of the pour. This is thought to be due to the absence of any “remixing” of the concrete at the base of the pile hole when the concrete is poured slowly.

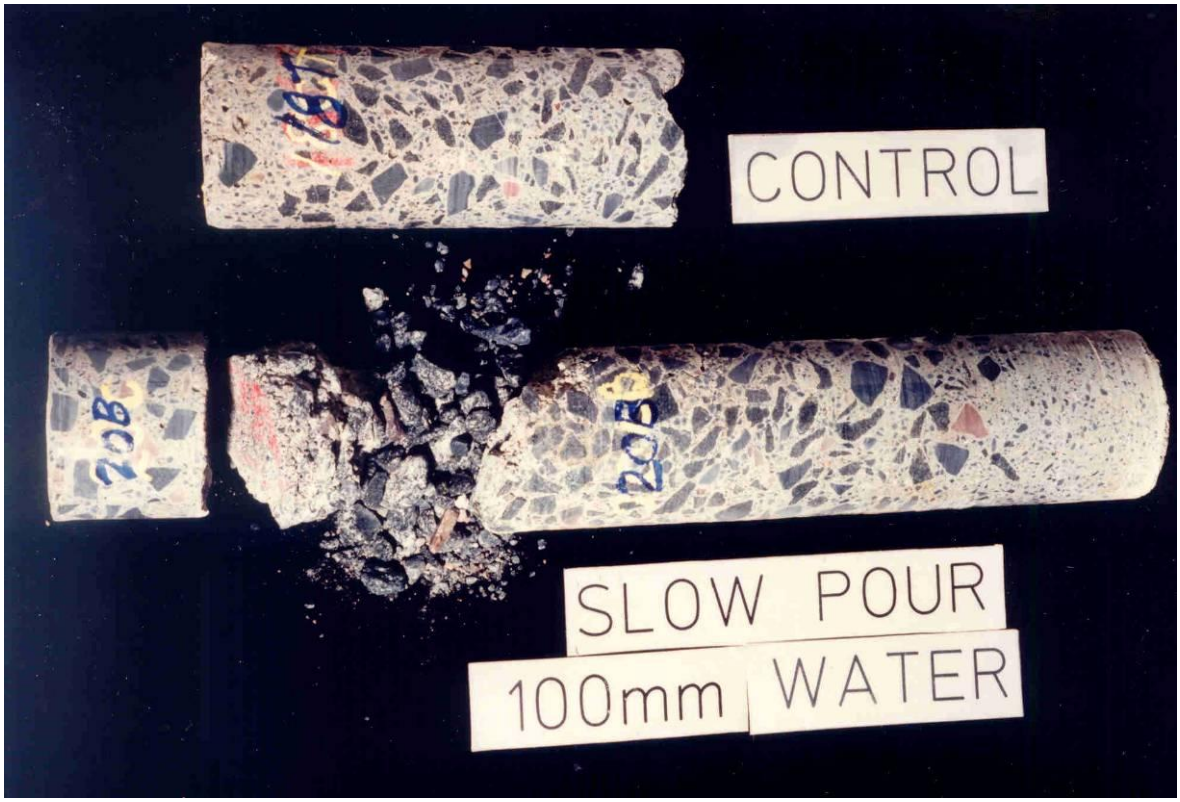

Photo 10: Segregation of concrete poured slowly into 100mm of water.

Effect of water in pile hole

Figure 20 clearly demonstrates the adverse effect which casting of concrete into water has on concrete strength. As little as 100mm of water in the bottom of the pile hole resulted in a 50% decrease in the strength of the concrete. For water depths in excess of 200mm, the concrete strength was reduced by 80% or more.

Figure 21 indicates that the actual density of the concrete is, on the whole, adversely affected by the depth of water in the pile hole. The correlation is spoilt somewhat by the results of tests on samples C1 and W3, both of which indicated lower than expected actual densities.

Figure 22 indicates a clear correlation between the depth of water and the percentage excess voids. The increase in voids with increasing water depth is also clearly visible in Photo 9.

In Figure 23, there is also a correlation between the depth of water in the pile hole and the aggregate : cement ratio. For water depths of less than 100mm, the average aggregate : cement ratio is of the order of 10. However, this increases to as much as 17 where concrete is placed through 400mm of water.

Displacement of spoil

By cutting away the bottom of the drum, the distribution of spoil in the bottom of the “pile holes” for tests S1 to S6 was observed and the percentage of contact between the pile concrete and the base of the pile was estimated. In the case of both the silty spoil material and the crusher dust, casting of concrete onto 50mm of dry spoil resulted in total

separation between the pile concrete and the base of the pile hole. However, the contact area increased to between 40% and 60% in the tests where 50m or 100mm of water was added to the base of the pile hole together with the spoil.

Photo 11 shows the contact between the blinding concrete at the base of the drum (representing the in situ founding material) and the pile concrete for piles cast onto 50mm layer of crusher dust at the bottom of the pile hole. The dry crusher dust (0mm water – left hand core sample) was trapped between the pile concrete and the bottom of the pile hole resulting in a total loss of contact of the pile with the founding material. With 50mm of water in the pile hole, the crusher dust over the middle of the hole was displaced by the falling concrete and assimilated into the pile concrete as a result of the remixing of the concrete at the bottom of the hole. With 100mm of water in the hole, the contact over the central portion of the pile was tight. However, the strength of the pile concrete had reduced to 75% of that of the control sample (40,5MPa to 31MPa) and the bearing area was reduced by about 40% due to trapping of crusher dust around the perimeter of the pile base.

Photo 11: Effect of 50mm of crusher dust on contact at base of pile.

5.4.6 Conclusions

On the basis of the above experimental data, the following conclusions are reached:

- Free fall placement of concrete into dry, vertical pile holes had no effect on the unconfined compressive strength of the concrete compared to that of the four control samples. Casting of concrete through 50mm of water at the bottom of the pile hole reduced the unconfined compressive strength by approximately 10%.
- Casting of concrete through 100mm or more of water in the bottom of the pile hole significantly reduced the compressive strength of the concrete.
- No segregation of the concrete (in the sense of an accumulation of aggregate at the base of the pour) was observed when the concrete was discharged from the truck mixer at a rapid rate even when the concrete was permitted to impinge on the reinforcing "cage". Clear signs of segregation were evident when the concrete was poured slowly into 100mm of water. It appears that the rapid discharge of concrete results in "remixing" of the concrete in the bottom of the pile hole.
- In addition to having an adverse effect on the strength of the concrete, casting of concrete into more than 100mm of water is detrimental to the actual density of the concrete, the percentage excess voids and the aggregate : cement ratio.
- As little as 50mm of dry spoil at the bottom of the pile hole can negate all direct contact between the pile concrete on the underlying founding stratum. Wet spoil is more readily displaced by the concrete but still results in significant reductions in base bearing area, mainly around the perimeter of the pile base.

On the strength of this research, it is concluded that the current practice of free fall placement of concrete in clean, dry, vertical pile holes has no detrimental effect on the quality of the concrete. It is, however, recommended that such techniques should not be used when the depth of water at the bottom of the pile hole exceeds 75mm.

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6. OCCUPATIONAL HEALTH AND SAFETY IN GEOTECHNICAL ENGINEERING

6.1 Background

6.1.1 Industry Practice

In many areas of South Africa, test pits or large diameter auger holes can be excavated safely to considerable depths in the stable, partially saturated soils above the water table. This provides an ideal opportunity for geotechnical engineers or engineering geologists to enter these holes for the purposes of inspecting the soil profile in situ, obtaining samples for laboratory testing and carrying out in situ tests. In his 2012 Jennings Memorial Lecture, Prof. John Burland (ex-South African now Professor at Imperial College) informed his audience that he has even used this technique in London to obtain vital information on the presence of sand lenses in the London Clays in the vicinity of the Houses of Parliament without which vital safeguards in the design of certain deep excavations would not have been implemented. This process is unrivalled in its ability to provide detailed information on the soil profile and high quality samples.

Test pits can readily be dug to a depth of 3m using a tractor-mounted loader / backhoe (TLB) or to depths of 6m or more with a large hydraulic excavator. The profiler enters the test pit using a ladder or other suitable means of access. Profiling and sampling seldom takes longer than 20 minutes after which the hole is backfilled with the excavated spoil. Depending on the depth of excavation, up to 20 test pits may be profiled per day using these methods.

Where a large diameter auger rig is used, 750mm diameter holes are drilled using a flight auger. Even the smaller auger rigs can drill to depths of 15m and many of the larger rigs can drill to depths of 36m or more. The profiler, wearing suitable protective equipment, enters the hole by means of a boatswain's chair suspended from a hand winch. He or she is then lowered down the hole to inspect and sample the soil profile in situ. Profiling is normally terminated above the point where there are signs of sidewall instability or significant water ingress.

In the piling industry, large diameter augered piles (or bored piles, as they are known in other parts of the world) rely on a combination of skin friction and end bearing for their load carrying capacity. On completion of drilling, about 250mm of spoil generally remains at the bottom of the hole. Even after cleaning the hole with a cleaning bucket, up to 50mm of spoil may remain around parts of the hole. As indicated in Section 5.4 above, this is capable of preventing intimate contact between the concrete and the full area of the base of the hole. It is therefore common practice to send a worker down the hole to clean the remaining spoil from the base of the pile hole by hand. This operation can take anywhere between 5 minutes and an hour depending on the conditions and the amount of spoil to be removed.

It is obvious that all these activities pose health and safety issues and it is not surprising that these practices are regulated by legislation or codes of practice.

6.1.2 Applicable Legislation

In the early days of geotechnical engineering in South Africa, work of this nature was carried out under the Factories, Machinery and Building Work Act (Act 22 of 1941) and the Mines and Works Act (Act 27 of 1956).

In 1983, the Factories, Machinery and Building Work Act was replaced by the Machinery and Occupational Safety Act (Act No. 6 of 1983). This act was accompanied by a number of Regulations including the General Safety Regulations. GSR13(2) stipulated that no

employer may '*require or permit any person to, and no person shall, work under unsupported overhanging material or in an excavation that is more than 1,5m deep and which has not been adequately shored or braced if there is a danger of the overhanging material or the sidewalls of the excavation collapsing*'. By implication, this required an assessment to be made of the danger of collapse before entering an excavation. It did not, however, prohibit working in excavations deeper than 1,5m as believed by some over-zealous safety officers.

The Mines and Works Act was replaced in 1996 by the Mine Health and Safety Act (Act 29 of 1996). This act applies only to work undertaken on mining land and is not considered further in this dissertation.

In 1993, the Occupational Health and Safety Act (Act No. 85 of 1993) replaced the Machinery and Occupational Safety Act. Initially, this Act made use of the existing Regulations pertaining to the 1983 Act, including the General Safety Regulations mentioned above, the Driven Machinery Regulations and the General Administrative Regulations.

Over the years, it was realised that it would be beneficial to consolidate all regulations pertaining to the construction industry. This resulted in the publication of the Construction Regulations in July 2003. In terms of these regulations, the forming of an excavation, irrespective of depth, constitutes construction work and falls under the requirements of the Act and the Regulations. Since 2010, there have been moves afoot to amend (tighten) the Construction Regulations. The proposed revisions have been circulated for public comment but have not yet been finalised or promulgated. The Candidate has provided comment to both ECSA and CESA on matters pertaining specifically to geotechnical engineering work for inclusion in their submissions and the response. The response of the authorities is awaited.

6.2 Application and Interpretation – Candidate's Contribution

6.2.1 Approach

The Candidate's first exposure to the requirements of the various acts was during the investigation for the proposed new steel mill at Saldanha Bay on the Cape West Coast. The client, through his appointed agent, insisted on full compliance with the then new Occupational Health and Safety Act of 1993. The decision was made at that stage to become fully conversant with the requirements of the Act and then to ensure compliance with these requirements in the simplest possible way with least deviation from already established safety requirements (see Section 6.3). This policy has been carried forward and now forms the backbone of the health and safety requirements for the Candidate's company.

6.2.2 LGI Seminar, 1994

In March 1994, the *Laboratorium vir Gevorderde Ingenieurswese* (LGI) at the University of Pretoria held a conference entitled *Safety and Health in Industry, Construction and Mining: the New Legislation*. The Candidate was invited to make a presentation on the requirements of the new (1993) Act pertaining specifically to geotechnical work in excavations. The content of the resulting paper (Day, 1994) is summarised below.

The presentation commenced with a description of current practice in the geotechnical engineering industry with regard to both geotechnical investigations and piling, much along the lines given in 6.1.1 above. It then described the codes of practice used by the industry (see 6.3 below).

The crux of the new Act is contained in a single sentence in Section 8 (1) "*every employer shall provide and maintain, as far as is reasonably practicable, a working environment that is safe and without risk to the health of his employees*". The rest of the Act merely provides requirements and mechanisms for the discharge of this general obligation. In particular, Section 8.2 sets out the specific duties of the employer, among which is a requirement [§ 8(2)(a)] to take "*such steps as may be reasonably practicable to eliminate or mitigate any hazard or potential hazard to the safety and health of employees, before resorting to personal protective equipment*".

It is interesting to note that both these obligations are qualified by the words "reasonably practicable". A definition of reasonably practicable is given in the Act but it is more easily understood by reference to English case law. This case law describes the process of assessing reasonable practicability as one in which the quantum of risk is placed on one scale and the sacrifice involved in the averting of this risk (whether in terms of money, time or trouble) in the other. If it is shown that there is a gross disproportion between the two with the risk being insignificant in relation to the sacrifice, the obligations of any clauses that carry this qualification are deemed to have been met (paraphrased from Edwards versus National Coal Board [1949] as cited by Professor P Benjamin, 1994).

There are no official records of fatalities or injuries that have occurred in test pits or auger holes. With the help of colleagues in the industry, the Candidate gathered information on 18 incidents over the past 30 year involving members of the geotechnical community engaged in soil profiling or inspection of pile holes. This data is shown in Table 6. Note that this table does not include construction activities other than piling.

Table 6: Injuries and fatalities resulting from soil profiling or inspection of piles

Type of Hole	Injury	Fatality
Auger or pile hole	5	1
Test pit	6	2

Construction industry fatality and injury statistics are difficult to obtain in South Africa. Davies and Tomasin (1990) reported that the number of fatalities in the construction industry in Britain amounted to 10 fatalities per 100 000 employees per annum in the 1980's. The corresponding figure for serious injuries was 230 injuries per 100 000 employees. If one assumes that the geotechnical community (geotechnical engineers, engineering geologists and other technical staff) averaged about 500 in total and that the above statistics are representative of the situation over a 30 year period, the fatality rate in the industry was about twice the above fatality rate and the injury rate was about one third of the average for the British construction industry. These statistics, although approximate, do not justify prohibition of profiling and inspection of activities. This view is supported by the observation that, since the paper was written in 1994, there have not been any further fatalities in the industry.

The cost of substituting alternative methods of investigation for open hole profiling was estimated at the time to be about R30m per annum. The time required for such alternative means of investigation would also be considerably greater. Elimination of risk by means of casing of all auger holes is not possible as it would negate the purpose for which the hole was drilled, namely in situ inspection of the soil profile.

After reviewing the general duties of the employer and the employee, the Candidate concluded that compliance with the Act would require employers to actively and systematically assess and eliminate or reduce hazards in cooperation with their

employees. In view of the safety record in the industry and the benefits to society of the current practices, the prohibition of current practices, including the profiling of auger holes is not warranted. It would certainly not be in the national interest to prohibit working in excavations for the construction industry in general where the effects on essential activities such as the laying and maintenance of buried services would cripple the industry. Adherence to the precautions set out in the available codes of practice coupled with sound judgement would go a long way to complying with the requirements of the Act.

6.2.3 Geotechnical Engineers and the Occupational Health and Safety Act

The 1994 paper was a good start but it did little to bring the requirements of the Act to the attention of practicing geotechnical engineers. As a result, a further paper was prepared for publication in the SAICE Journal (Day, 1996).

This paper presented much the same introductory information as was given in the 1994 paper. However, as its target audience was now geotechnical practitioners, the duties of the employer and the employee were more fully spelt out in a clause-by-clause examination of the relevant requirements of the Act. These requirements are summarised in Table 7 and Table 8.

6.2.4 Geotechnical Engineers and the Construction Regulations

Following the issue of the construction regulations in 2003, a further paper was written for publication in the SAICE Journal (Day 2006) with the same intention as the 1996 paper.

The Construction Regulations comprise a set of regulations pertaining specifically to the construction industry. Included in the definition of construction work, to which the regulations apply, is the making of an excavation, moving of earth, piling or any similar type of work. It is therefore clear that geotechnical engineers engaged in site investigation activities involving test pits or large diameter auger holes are undertaking construction work and, as such, must comply with the Construction Regulations.

Although the issue of the Construction Regulations resulted in the repeal of a number of older regulations, the Construction Regulations do not alter the basic requirements of the Act in any way. In short, the employer remains primarily responsible for ensuring the safety of employees at work. One of the “old” regulations which was repealed by the Construction Regulations is the often misinterpreted General Safety Regulation 13(2). The current regulations contain no reference to a “safe” excavation depth (previously taken as 1,5m) and the requirements of the regulations apply to excavations of any depth.

In contrast to the Act which deals mainly with the duties of employers and employees, the Construction Regulations define the duties of various other parties including the Client, the Agent, the Principal Contractor, the Contractor, the Designer, the Construction Supervisor and the Competent Person. In the course of a typical geotechnical investigation, geotechnical engineers may employ sub-contractors for plant hire, rotary core drilling, geophysical testing etc. In addition, their personnel will carry out work on site including profiling, testing, etc. In this context, geotechnical engineers assume the roles of Principal Contractor and Contractor under the Regulations. Should the geotechnical engineer be appointed to design any aspect of the works, such as lateral support or foundations, he or she would also assume the duties of the Designer.

Table 7: General duties of employers towards their employees

Clause	Précis	Note	Practical implication
§ 8(2)(a)	Provide and maintain systems of work, plant and machinery that are safe and without risks to health.	*	Ensure that all equipment (winches, safety chairs, safety harnesses, gas detection equipment, ladders, etc.) used for profiling activities are provided and maintained in working order.
§ 8(2)(b)	Take steps to eliminate or mitigate hazard or potential hazard to the safety or health of employees, before resorting to the use of protective equipment.	*	Adopt sensible precautions such as clearing around holes to remove risk of falling objects, backfill holes as soon as possible in preference to barricading, etc.
§ 8(2)(d)	Establish what hazards to health and safety are attached to the work to be performed and further establish what precautionary measures should be taken.	*	One of the principal aims of the SAICE code of practice is to identify the risks involved in working in excavations and to recommend precautionary measures. The requirements of this clause have thus been undertaken by the profession. Obviously, special circumstances require further attention.
§ 8(2)(e)	Provide information, instruction, training and supervision as may be necessary for health and safety of employees.	*	All persons engaged in soil profiling activities should be provided with a copy of the code. Inexperienced employees should not be permitted to work without supervision until they have sufficient knowledge and experience to recognize potentially dangerous situations.
§ 8(2)(f)	Not permit an employee to do any work or operate any plant or machinery unless the required precautionary measures have been taken.	*	Employers should visit sites on which their employees are working from time to time to ensure that the requirements of the code and of the Act are being implemented.
§ 8(2)(g) and (h)	Take all necessary precautions to ensure that the requirements of the Act are complied with by every person in his employment where plant and machinery is used, and enforce such measures as may be necessary in the interests of health and safety.		Note that these clauses do not contain the words 'as far as is reasonably practicable' and are thus mandatory. Disciplinary steps may be necessary where the required measures are not implemented.
§ 8(2)(i) and (j)	Ensure that all work is performed and that plant and machinery is used under the general supervision of a person trained to understand the hazards involved and with the authority to implement the required precautionary measures, and cause employees to be informed regarding the scope of their authority.		Also a mandatory clause. It is essential that the responsible person on the site has the necessary experience and is aware of the scope of his authority. This latter point is particularly important where the authority of the chief executive officer has been delegated to an employee in terms of S 16(2).
§ 13(a)	Cause every employee to be conversant with the hazards of his work and the precautionary measures that should be taken.	*	Provide all employees engaged in fieldwork with a copy of the SAICE code as well as adequate field training under the guidance of an experienced profiler.

Note: * indicates where the clause is qualified by the use of words 'as far as is reasonably practicable'.

Table 8: General duties of employees at work

Clause	Précis	Practical implication
§ 14(a)	Take reasonable care for the health and safety of himself and of any other persons affected by his acts or omissions.	Note that the employee has an obligation to look after himself as well and is not free to take risks even where he is the only person affected.
§ 14(b)	Co-operate with his employer or any other person in the carrying out of any duty or requirement of the Act.	Self-evident.
§ 14(c)	Carry out any lawful order and obey any rules and procedures laid down in the interests of health and safety.	Note that it is an offence not to obey safety requirements.
§ 14(d)	Report any unsafe or unhealthy situation to the employer as soon as possible.	Self-evident.
§ 14(e)	Report to his employer any incident in which he is involved and which may affect his health or cause injury to himself as soon as practicable but not later than the end of the shift during which the incident occurred.	Self-evident.

The major duties of these parties relevant to geotechnical investigations are summarised as follows.

Client: Prepare a safety specification; ensure that sufficient allowance is made by contractors for health and safety measures; inform the contractor of any aspects of the work that affect health and safety and appoint the contractor in writing.

Contractor: Provide a health and safety plan; produce a site safety file to be kept on site; appoint a full-time competent person to supervise the work and appoint a full-time or part time safety officer.

Principal Contractor: Fulfil all the duties of the client (with the exception of producing a safety specification) and of the contractor.

Designer Advise the client of any aspects of the design that may affect the pricing of the work; advise the contractor of any dangers or hazards associated with the work; provide the contractor with a geotechnical report where appropriate, the loading the structure is designed to withstand and the method/sequence of construction; inspect the works for compliance with the design intent and inspect the structure on completion.

In addition, the client, principal contractor and designer have an obligation to stop any work that is not in accordance with the health and safety plan or the design.

The Regulations also provide specific requirements for various construction activities including working at heights, formwork and scaffolding, excavations, demolition work, tunnelling, etc. The paper (Day, 2006) provided a clause-by-clause account of the specific requirements pertaining to geotechnical investigations and the documentation that is required to undertake such work. It also provided practical guidance as to how these requirements could be fulfilled with least disruption to the normal execution of a geotechnical site investigation.

Two problem areas were noted with the Regulations, both of which have been brought to the attention the Department of Labour in the comments on the proposed amendments to the Regulations. The first pertains to the testing of the quality of the air in excavations. The current regulations treat all excavations, no matter how deep or wide, as confined spaces. This requirement is unnecessary in all but a limited number of special circumstances. The second pertains to the performance testing of the boatswain's chair

after each erection. On a typical geotechnical investigation site using auger trial holes, this chair may be moved numerous times a day making this requirement impractical.

6.2.5 *The Cost of Safety*

The cost of compliance with health and safety involves a whole lot more than just the once-off preparation of a series of documents. Each project has site-specific safety induction requirements, medical requirements, access requirements and so forth. In addition, companies themselves have health and safety overhead costs that are not related to projects such as the maintenance of the company's health and safety plan, compliance audits, personal protective and other safety equipment, modifications to vehicles, etc.

One of the spin-offs of the Construction Regulations is that clients and principal contractors are required to ensure that contractors have made sufficient provision in their tenders for the cost of health and safety compliance. To facilitate this process, the Candidate's company reflects the cost of health and safety compliance separately from the cost of other project work in each bid that it submits. This enables them to extract the cost of such compliance on a project-by-project basis.

Every year, one of the country's largest petro-chemical companies holds a vendor day. In 2011, the focus of this day was on safety. The company selected four of its vendors to address the group and advise how they respond to the company's safety requirements. Being a petro-chemical company, there are significant hazards associated with working in their plants. The vendors selected to make presentations were a leading contractor, an international turnkey contractor and a large project management company. The Candidate's consulting company was invited to present the view of a smaller organisation and the lot fell on the Candidate to make the presentation.

Instead of describing the systems that the company has in place, the Candidate took the opportunity of advising company management of the cost of compliance with their safety, health and environmental (SHE) requirements. Two graphs were presented, both reflecting the cost of safety requirements for geotechnical investigations as a percentage of the professional fees on the project.

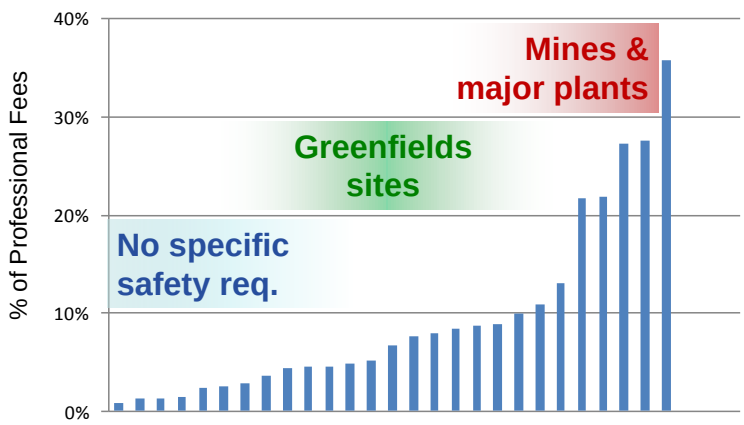

Figure 24: SHE costs of geotechnical investigations as a percentage of fees

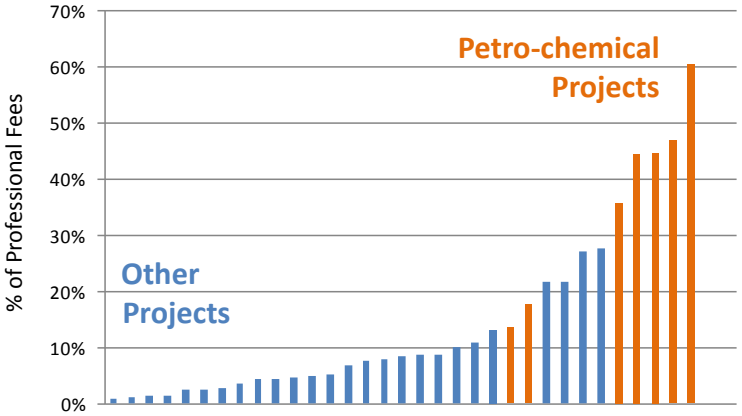

Figure 25: SHE costs of geotechnical investigations including the petro-chemical industry

The costs reflected above are in addition to a "background spend" of between 0,5% and 0,6% of total fee turnover of the company.

From this data, it is concluded that SHE compliance comes at a cost, and this can amount to a significant proportion of the total project cost. While there is no problem with this, it is essential that clients ensure that the cost of compliance is adequately reflected in any bid they consider during the tender adjudication process as the amounts involved are likely to be significant compared to the differences in bid prices.

6.2.6 The Outcome

The many hours spent on understanding the legislation and preparation of a comprehensive response to it has paid off in that the Candidate's company has been awarded a 5 Star NOSA rating.

6.3 Codes of Practice

The Geotechnical Division of SAICE has long recognised both the benefits and the risks associated with work in excavations, either for investigation purposes or for the construction of piled foundations. In 1960, they issued a code of practice on the safety of men working in small diameter shafts. This code was revised in 1980 (SAICE, 1980). Over the years, strict adherence to the requirements of the code led to a favourable interpretation of the legal requirements set out in the Acts referred to above.

Comprehensive as this code was, it applied only to circular shafts such as large diameter auger holes. Test pits were excluded, despite their popularity for the investigation of near surface soil conditions[9]. It was therefore decided by the Geotechnical Division and the South Africa Association of Engineering Geologists that the code should be amended to include test pits and to bring the code in line with the 1983 OSH Act. This amendment was completed in 1990. In September 1990, the revised code was sent to the Government Mining Engineer for comment. The GME confirmed two requirements, namely that the minimum diameter of a shaft into which a person may descend is 750mm (not 600mm as was becoming the norm with some piling companies) and that a second person must be in attendance on surface whenever a person is in an excavation. He further required more stringent requirements for testing for noxious gasses in unventilated holes, including keeping a record of such tests for all trial holes on mine property. This

[9] Due to their shape and length, test pits are inherently less stable than circular shafts.

latter requirement was probably in response to the fact that the only fatality recorded in an augered pile hole to date (see Table 6) was as a result of asphyxiation of the profiler in a hole drilled into mine tailings containing sulphides.

The 1990 version of the code made allowance for the use of a “safety chair” for the profiling of auger holes. Such a chair would be fitted with a built-in safety harness that would prevent the user from falling from the chair enabling the chair and user to be winched out of the hole in the case of an emergency. Although the logic of this decision may be sound, the concept has proved unpopular. This was due partly to the bulkiness of the prototype chairs and the restrictions that they posed to the free movement of the user.

For various reasons, the 1990 version of the code was never officially issued.

In 2007, following the issue of the Construction Regulations, the Geotechnical Division finally got their act together and issued the 2007 version of the code which is now freely available in electronic and hard-copy format (SAICE, 2007). Although the Candidate was not a member of the drafting committee, he wrote Sections 4 and 5 of the code dealing with Relevant Legislation and Personnel. He was also responsible for the final editing of the document.

6.4 References

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7. SOIL PROFILES NOT AMENABLE TO SMALL SCALE TESTING

7.1 Background

Significant research has been undertaken into the settlement of natural soil profiles, particularly in sands or saturated, normally consolidated, fine grained soils. Relationships have been developed between the results of in situ tests such as the Standard Penetration Test (SPT) and the Static Cone Penetration Test (CPT) and the parameters required for settlement prediction.

Problems arise, however, when dealing with either cemented soils or fill materials. The former occur extensively on the drier, western side of the country where the climate favours the formation of calcretes. The latter occurs in the coal mining areas of the country where opencast mines are backfilled with uncompacted spoils from the mining operations. Both the calcretised materials and the mining spoils are highly variable and not amenable to testing by means of conventional laboratory or field tests due to the large variation in particle sizes.

7.2 Calcretised Soils – Role of Small Strain Stiffness

7.2.1 Nature and Distribution

The distribution of calcretised soils in South Africa is shown in Figure 26 (after Brink, 1985).

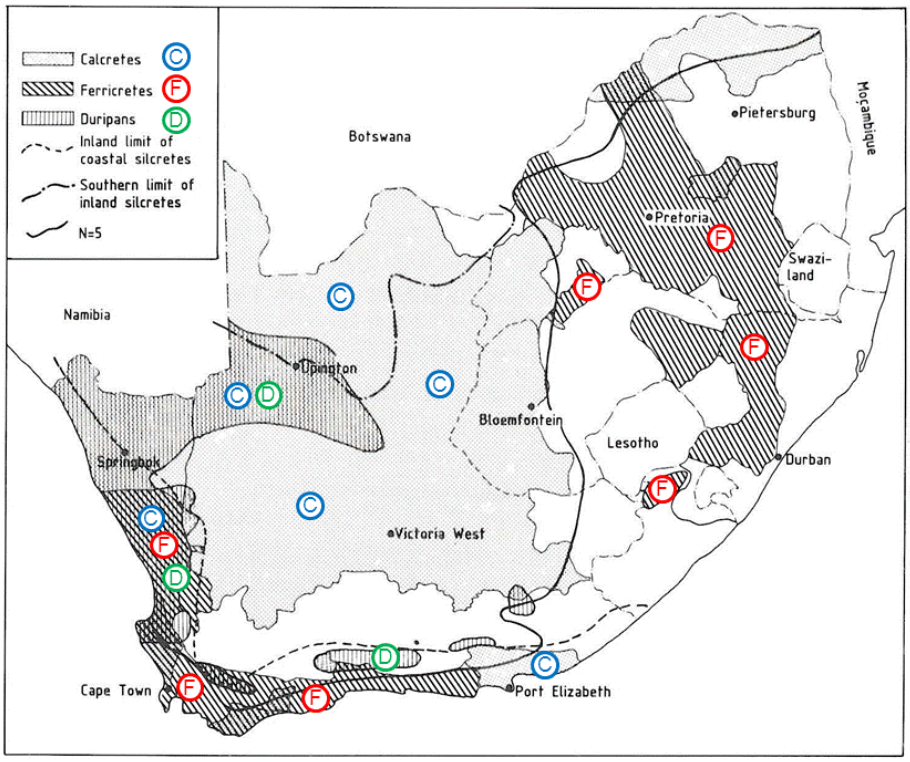

Figure 26: Distribution of common occurrences of pedocretes in South Africa

From this figure, it can be seen that much of the country west of the N=5 line (Weinert, 1980) is underlain by calcrete or combinations of calcrete and other pedocretes[10]. In particular, many areas where developments have taken place in the last 30 years are underlain by calcretes including the diamond mining areas in Botswana, the iron and manganese mining areas of the Northern Cape, the Cape West coast including the Saldanha development node and the industrial development zone at Coega.

In Botswana (below the Kalahari Sands) and the Northern Cape, the calcretes may attain thicknesses of twenty metres or more and are cemented to a hard rock. Photo 12 shows an exposure of the calcretes after blasting for the primary crusher excavation ramp at Kumba's Kolomela (Sishen South) Mine. This type of calcrete is not a problem due to its uniformly cemented nature and only minor occurrences of softer material.

Photo 12: Hard rock, well cemented hardpan calcrete in Northern Cape

In many areas, including in particular, the Coega IDZ (near Port Elizabeth) and the Saldanha area (Cape West Coast), the calcretes have formed hardpan lenses inter-bedded with softer materials as shown in Photo 13.

To illustrate how soft these intervening layers of material can be, Photo 14 shows a horizontal plate load test in progress in loose calcareous sands below a layer of hardpan calcrete at Saldanha. Note how the plate on the left has punched into the loose soils.

7.2.2 Conventional Test Methods

The most common methods for determining the compressibility of soils are laboratory tests such as the oedometer test or in situ tests including plate load tests, SPT and CPT tests.

10 Pedocretes are materials formed by cementation and/or replacement of pre-existing soils by various minerals (most commonly iron, calcium or silica) precipitated from the soil water or ground water. They can be indurated (forming hard layers or nodules) or non-indurated (soft or powdery forms). (Partly after Brink, 1985.)

The difficulty with oedometer testing is obtaining undisturbed samples for testing and the small sample size. It is often not possible to retrieve undisturbed samples of un-cemented sandy soils.

Photo 13: Layer of hardpan calcrete overlying softer soils (Coega, Eastern Cape)

Photo 14: Plate load testing in calcareous sands – Saldanha Bay

Plate load tests may be carried out using either cross-hole tests with 200mm diameter plates as the test shown in Photo 14 or by larger diameter vertical plate load tests as shown in Photo 15.

Photo 15: Vertical plate load tests using a 1m diameter plate

7.2.3 Test Results from Saldanha Steel Site

The Saldanha Steel site in Saldanha Bay was underlain by calcretes of the Langebaan Limestone Formation to a depth of about 10m followed by mainly sandy sediments of the Varswater and Elandsfontein Formations to 30m. These cenozoic[11] sediments were underlain by residual granites that had decomposed to a clayey silt to depths of up to 50m (Wardle and Day, 2003).

SPT tests were used to provide an indication of the consistency of the profile. Even though these tests met with refusal on calcrete layers, they provided good data in the softer layers between the calcrete bands and in the underlying sandy sediments and residual granites. CPT testing, which involves pushing a cone into the soils from the surface, was not even considered as refusal would have occurred at shallow depth on the calcretes.

Oedometer tests were carried out in the laboratory on samples of lightly cemented, loose sands between the layers of hardpan calcrete. Despite the care taken in retrieving these samples, the oedometer tests showed these sands to be highly compressible with elastic moduli generally below 10MPa (Day et al, 2001; published by Wardle and Day, 2003).

Both cross-hole and vertical plate load tests were carried out. The results of a typical plate load test on a sand layer within the calcrete horizon are shown in Figure 27. From this figure, it can be seen that the load displacement plot is curved with a high initial modulus and a lower final modulus. The reduction in modulus is thought to be due to the breaking down on the lightly cemented bonds between the sand grains which appeared to occur at an applied pressure of 400kPa to 600kPa below a 200mm diameter plate. From a total of 20 tests, an average initial modulus of 85MPa was recorded with a final modulus averaging 27MPa.

[11] The Cenozoic Era is the most recent of the geological eras stretching from 65,5 million years ago to the present. The name comes from the Greek words meaning “new life”.

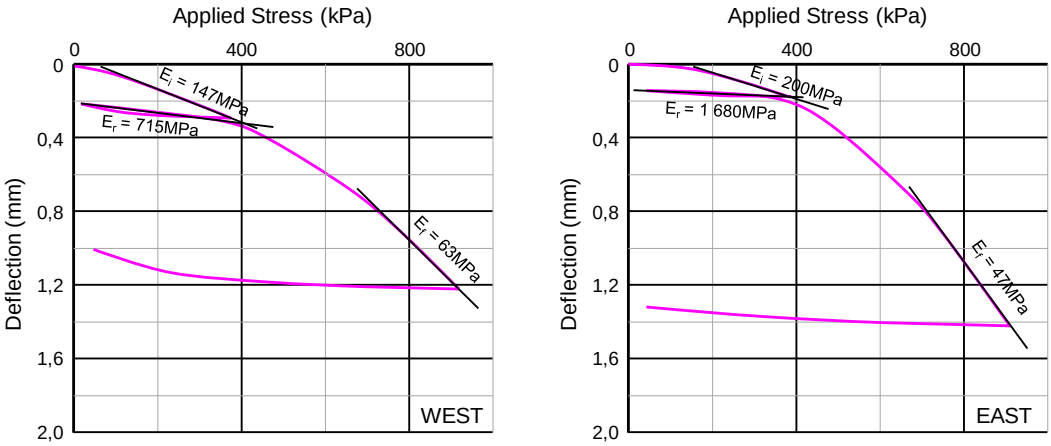

Figure 27: Typical cross-hole plate load test result.

7.2.4 Large Scale Load Test

All the above tests provided information only on the layers of softer material between the calcrete lenses. In addition, the tendency for the stress-strain curve from the plate load tests to be curved with higher moduli at low stresses led to the suspicion that the soils had a higher modulus in situ than could be determined by any large-strain tests on small volumes of soil. A proposal was therefore made to the client that a large scale load test should be carried out to determine the stiffness of the full depth of the profile.

Prior to embarking on this test, it was calculated that the average pressure below the mill building was a mere 85kPa. If the higher loads below the building columns and the mill bases could be distributed by means of a compacted fill mattress, the effect of the distributed load on the soils below the mattress could be simulated with as little as 5m of fill surcharge. In any event, large volumes of imported fill were required for the terracing of the site. Thus, the cost of importation of fill material for the surcharge test was recouped during construction.

The load test entailed the measurement of settlement at various depths in the soil profile under an 85kPa surcharge exerted by a 5m high, 50m diameter earth embankment as shown in Photo 16. Settlements were monitored by means of monitoring rods grouted into bottom of boreholes at depths of 2m, 5m, 10m, 15m, 20m and 25m below ground level. Two additional monitoring rods were drilled into the granite rock below 50m to serve as benchmarks.

Photo 16: 50m diameter by 5m load test embankment under construction.

The results of the monitoring are shown in Figure 28. These results provide two important observations. Firstly, the settlements are much lower than could have been predicted by the oedometer or plate load tests, confirming the high small-strain stiffness of the soils. Secondly, the settlement was completed within a few weeks after application of the load.

7.2.5 Settlement Predictions and Measurements

On the strength of the above observations, a decision was taken to found much of the plant on spread footings or raft foundations placed on a mattress of compacted fill. However, a method needed to be found to extrapolate these results to other areas of the site where no such tests had been undertaken. The SPT test results were chosen for this purpose.

Stroud (1989) presented a correlation between SPT N values and the drained elastic modulus of the soil that took account of the high stiffness of soils at low strain levels based on the ratio the net bearing pressure below the loaded area (q_{net}) and the ultimate bearing capacity (q_{ult}) as shown in Figure 29. For the appropriate ratio below the load test embankment (q_{net} / q_{ult} = 0,013), the multiplier (E' / N_{60}) for the over-consolidated sands (representing the calcareous sands over the upper 10m of the profile) is 9,0. For over-consolidated clays (representing the remainder of the profile which contains layers of muddy sands, peat and clayey residual soils) the multiplier is 5,4. Applying these multipliers allowed a direct comparison between the elastic moduli inferred from the load test data and the elastic moduli inferred from the SPT tests in the borehole below the centre of the load test embankment. This comparison is shown in Figure 30.

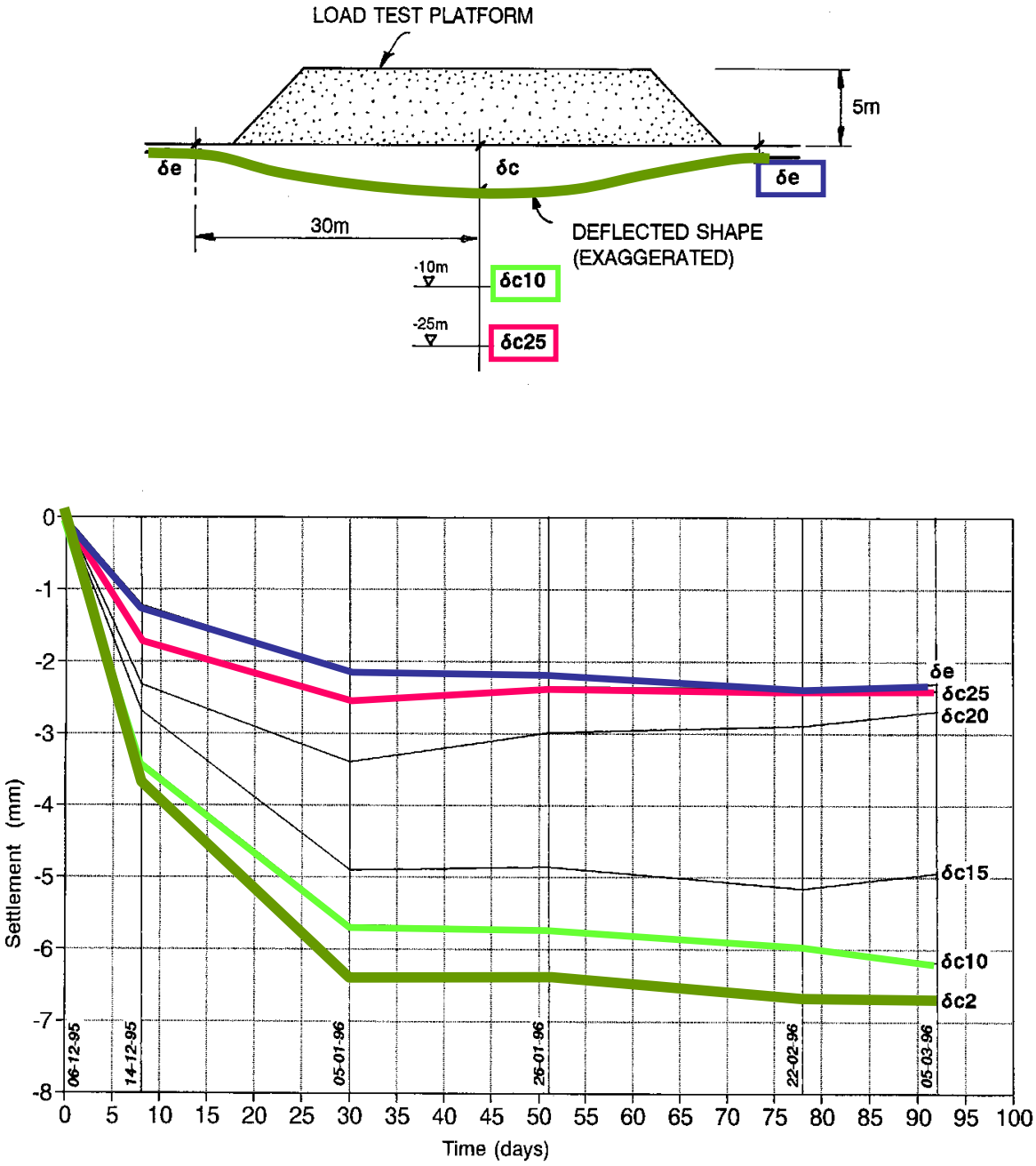

Figure 28: Settlement observations for large scale load test

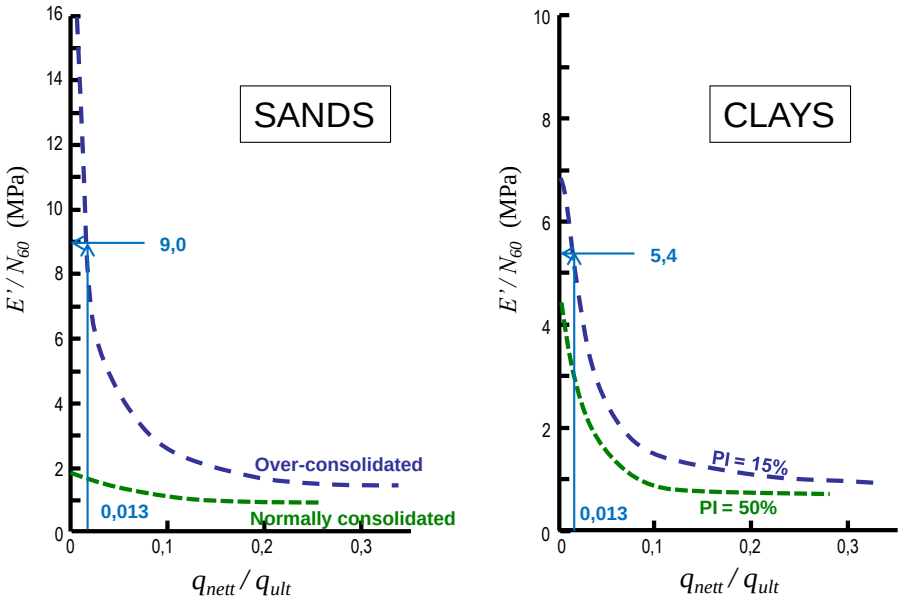

Figure 29: Drained elastic modulus for sands and clays (after Stroud, 1989)

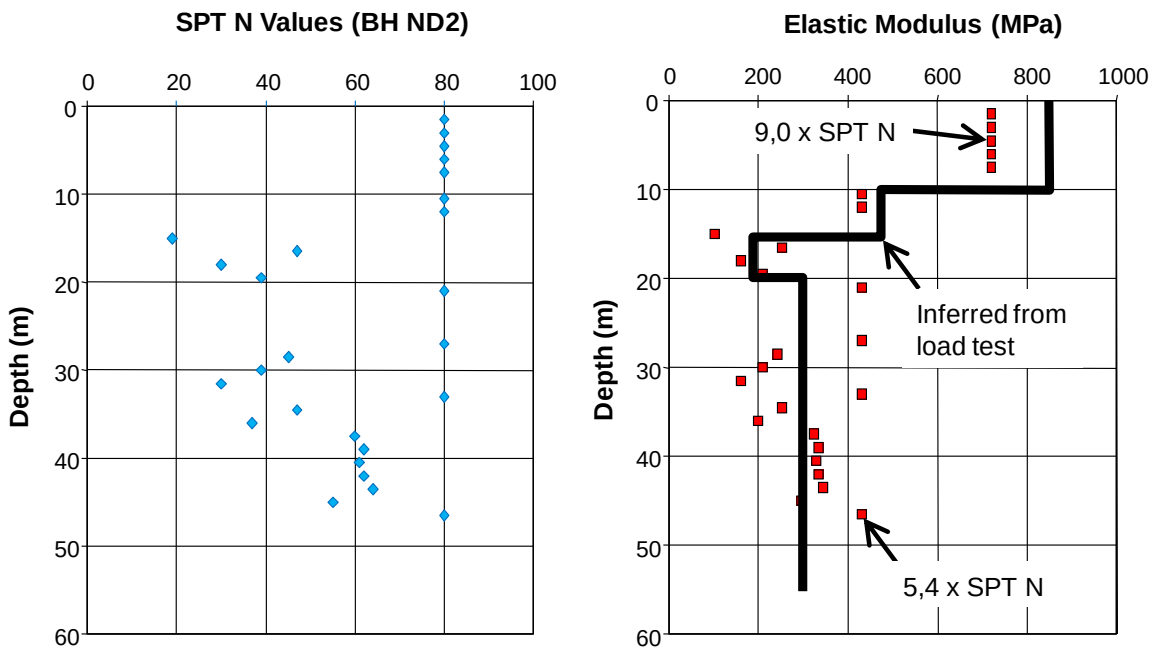

Figure 30: Elastic moduli from results of load test and SPT N values

Based on the good correlation between measured elastic moduli and the values predicted by the SPT tests, this method was adopted for routine design. In particular, it was used for a Class A prediction (before construction) of settlement below the 90m high Correx tower for which a limiting settlement of 30mm and a maximum tilt of 1:1500 was specified. Settlements were then monitored on the four corners of the raft foundation during

construction. The observed short term settlements varied from 13mm – 21mm (15mm – 20mm predicted) and the long term settlement varied from 21mm – 28mm (25mm – 30mm predicted). The maximum tilt was 1:2000 (Day et al, 2001).

7.2.6 Conclusions

The use of a simple large scale load test provided better and more reliable data than could have been obtained from conventional means of investigation. Small strain stiffness of up to 850MPa were measured for the cemented sands compared to average values of 85MPa and 10MPa predicted by plate load tests and oedometer tests respectively. The test provided the client with good value for money with the final cost of the experiment amounting to less than 5% or the total cost of the investigation.

This method has subsequently been employed to good effect in determining the settlement parameters for mine backfill as described below, albeit with more sophisticated monitoring equipment.

7.3 Settlement of Mine Backfill

7.3.1 Background

With the advent of ever bigger and bigger mining equipment and the multiple coal seams present in some of South Africa's coal mining areas, opencast coal mines are getting larger and deeper. For example, the open pit at Exxaro's Grootegeluk coal mine in Limpopo covers an area of about 8 km^2 and reaches a depth of 90m below original ground level. Most opencast pits are backfilled and rehabilitated as the mining progresses. Many of these pits require the relocation of roads, railway lines and streams.

Given the vast areas involved, it is preferable that the land be restored to beneficial use after completion of mining operations. The major factor that affects such restoration is the settlement of the backfill in the open pit. This settlement persists for many years after backfilling and could be re-initiated if the water table in the pit rises after closure of the mine. The understanding and prediction of such settlement is therefore important for determining the use to which backfilled mines can be put.

7.3.2 Components of Backfill Settlement

The total settlement of opencast backfill due to its own weight and under external loads may be subdivided into the following components (Day & Wardle, 1996 and Hills, 1994) as shown in Figure 31:

- Immediate settlement: Settlement under constant volume due to shear strains in the material as a result of the application of load. Typically elastic (recoverable).
- Consolidation settlement: Settlement, including change in volume, due to the dissipation of excess pore (air and water) pressures. Largely plastic (non-recoverable).
- Creep settlement: Long term settlement under conditions of constant stress and moisture content due mainly to the rearrangement of material fragments caused by crushing of highly stressed particle contacts.
- Collapse settlement: Additional settlement under constant total stress resulting from an increase in the water content of the backfill. This may either be due to surface water infiltration or re-establishment of the water table. Collapse settlement is irreversible and occurs on first wetting after application of load to the backfill.

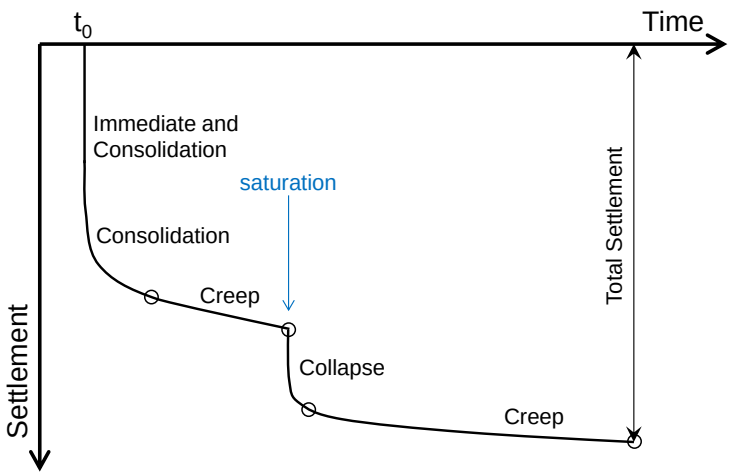

Figure 31: Components of total settlement for pit backfill

Immediate settlement, as its name suggests, takes place immediately on application of a load to the fill material. As such, it is not observed during the construction of fills where any settlement which occurs during the placement is "built out" during placement.

In the case of partially saturated, granular soils, the development of excess pore pressures in the material during placement is limited and the dissipation of these pressures is rapid. Thus, most of the consolidation of pit backfill under its own weight also occurs during placement of the material which often stretches over a period of many months, if not years.

The components of total settlement of the fill under its own weight of relevance to long term development are thus creep settlement and collapse settlement. If structures of any magnitude are built on the backfill, immediate and consolidation settlement under the load exerted by the structures may also be relevant.

7.3.3 Determination of Drained Elastic Modulus

It is not possible to determine the drained elastic modulus of pit backfill simply by monitoring the surface of the completed fill. This is because most of the compression of the fill due to immediate and consolidation settlement occurs during placement. It is therefore necessary to determine the elastic modulus by applying an additional load to the fill and monitoring the additional settlement due to the load application.

<u>Tests by Day, 1992</u>

Day (1992) describes a full scale load test used to determine the settlement of opencast mine backfill at the New Vaal Colliery. This colliery is situated on the southern bank of the Vaal River between Vereeniging and Sasolburg and supplies coal to the Lethabo Power Station. The dry ash dump for the power station was to be constructed on the backfilled pit. One of the key requirements was that settlement of the spoils below the dump should not result in the base of the ash settling to below the level of the water table and thereby creating the potential for pollution of the Vaal River. Raising the platform would require the importation of vast amounts of fill from other areas of the mine. Thus, a reasonably

accurate assessment of the settlement of the backfill under the load of the dump was required.

An area of the pit was chosen that had been backfilled in a similar manner to the bulk of the ash dump site and which coincided with the position of the 11m high starter embankment for the ash stacking system. Four 1,2m square slabs were cast on the surface of the backfill in advance of the construction of the starter embankment. Their position and elevation were determined by survey. The starter embankment was then constructed above these slabs by end-tipping and dozing. After levelling of the top of the starter embankment about 8 weeks after the slabs were covered, small diameter (NX) holes were drilled through the fill above each slab using wash boring methods capable of penetrating the fill but not the concrete. Once the top of the concrete was encountered, a level was taken on the top of the drill string which was then removed from the hole and re-assembled on surface to determine its length thereby allowing the level of the slab to be determined. Each hole was then advanced through the slab using rotary core drilling methods and a galvanised iron pipe was grouted into the slab by pumping a measured quantity of grout through the pipe. This pipe, which was isolated from the surrounding material by filling the hole with vermiculite, then served as a monitoring peg for further settlement readings. Unfortunately, these monitoring points were destroyed by a badly informed contractor after only the second reading.

The first readings eight weeks after placement of the ash recorded an average settlement of 989mm with a variation of only 40mm between the four test locations. This had increased to 997mm at the time the second set of readings was taken some 12 weeks later indicating that immediate and consolidation settlement of the backfill was largely complete before the first readings were taken. Based on the assumption that the elastic modulus of the fill was uniform with depth, the average value of the drained elastic modulus (E') of the pit backfill was 5,6MPa. If a linear increase in modulus with depth from 0,5 E' on the surface of the backfill to 1,5 E' at the level of the pit floor (33m down) was assumed, the average modulus E' increased marginally to 6,2MPa.

Tests by Others

In 1989, Wates and Wagner carried out a test at Kriel Pit 3 North. During this test, 3,5m of pit backfill was excavated and the resulting excavation was lined with clay. The backfill was then replaced and the settlement monitored during surcharging of the material with 7m of soil. Back-analysis gave a drained elastic modulus E' of 1,2MPa. This low value is believed to be due to the excavation and replacement of the material immediately prior to testing and is not thought to be representative.

Hills (1994) presents a table of typical constrained moduli ($1/m_v$)[12] for various materials including poorly compacted colliery spoils. In this table, the constrained modulus of well-graded sandstone rockfill was given as 4MPa – 15MPa (loose and dense) and that of poorly compacted colliery spoils as 2MPa.

12 The value of $1/m_v$ is related to E' by the following equation:

$$\frac{1}{m_v} = \frac{E'(1-\nu')}{(1+\nu')(1-2\nu')}$$

For ν' = 0,25 (typical value), E' = 0,83.$1/m_v$.

7.3.4 *Determination of Collapse Settlement Percentage*

Collapse potential is typically expressed as a percentage of the thickness of fill material subjected to saturation. In most cases, this thickness is the rise in the water table within the backfilled pit.

Flooding Experiment (Day, 1992)

On the same New Vaal Colliery site described above, the Candidate carried out a controlled flooding experiment in an area where a pollution control dam was to be constructed on the pit backfill Day (1992). Prior to flooding of a 30m x 30m test site, telescopic benchmarks were installed at depths of 5m, 10m, 15m, 20m and 25m. The area was then flooded for a period of nine months commencing in November 1989 and the resulting settlement was monitored. The results are given in Figure 32. Problems were experienced with silt accumulation sealing the floor of the impoundment and surprisingly little water was required to keep the impoundment flooded. This was remedied to some extent by digging trenches into the base of the impoundment to increase infiltration of water into the backfill through the sidewalls of the trenches which are unaffected by siltation.

From these results, it is clear that little or no collapse settlement occurred over the upper 5m of the profile. Between a depth of 5m and 10m, a reduction in the layer thickness of 205mm was recorded, equivalent to 4% of the layer thickness. At greater depth, the observed collapse was less than 1%. The difficulty with the interpretation of these results is that it is not known how far the water permeated into the backfill despite the prolonged flooding of the area.

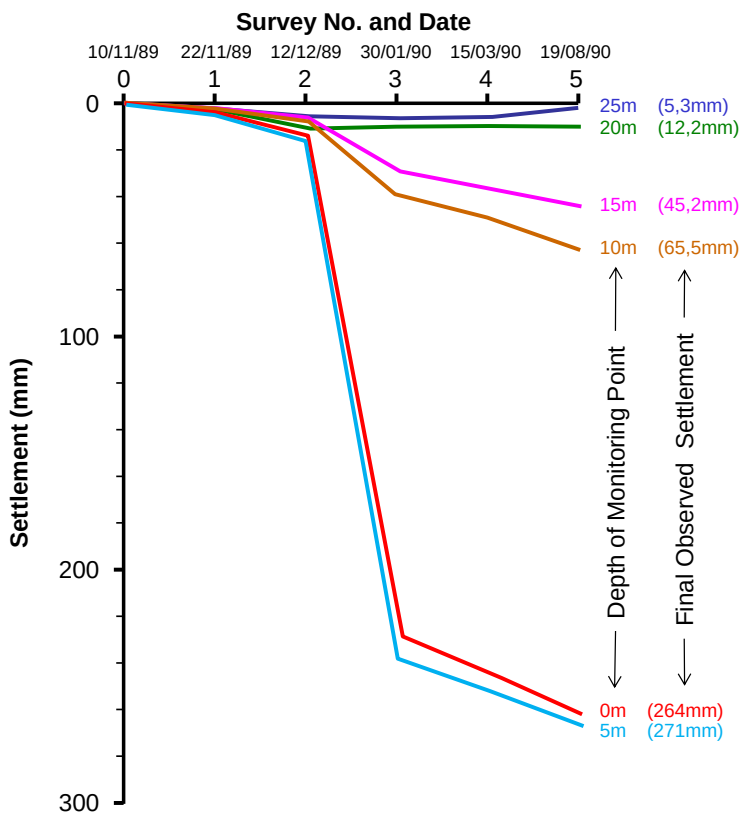

Figure 32: Settlement below controlled flooding experiment (After Day, 1992)

Small Scale Laboratory Tests (Day, 1992)

In order to establish an upper bound for the likely collapse settlement at New Vaal Colliery, a series of small scale tests was carried out in the laboratory using the double oedometer test. Samples of the fines from various types of spoil on the mine were placed loosely into two oedometer rings. The one was tested at natural moisture content and the other in a saturated condition. By comparing the "dry" and saturated curves, the collapse of the soil can be determined at various applied pressures. The results are given Figure 33.

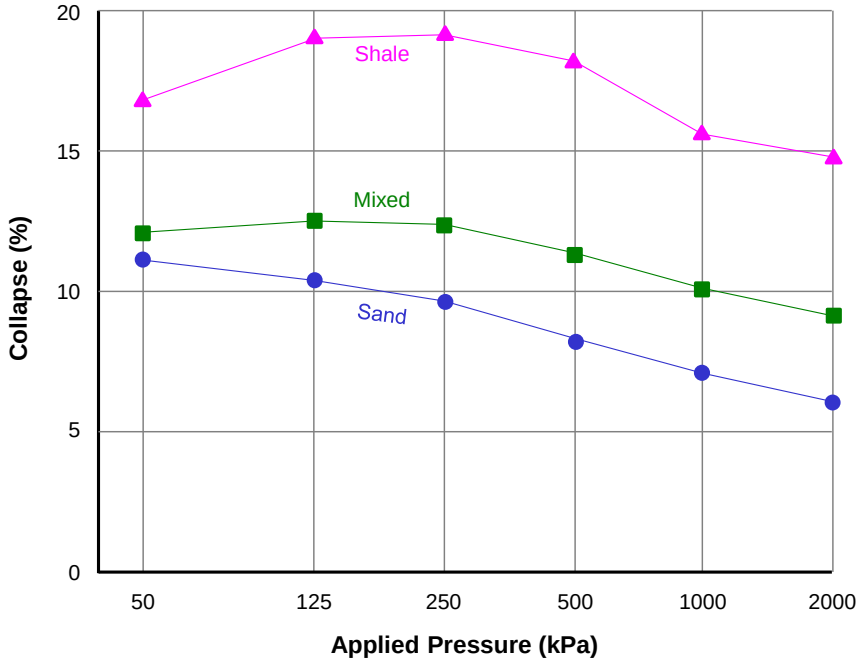

Figure 33: Collapse of pit backfill fines from double oedometer tests (after Day, 1992)

Apart for the magnitude of the collapse settlement (6% to 19% of the sample thickness), there are two other points worth noting. Firstly, the maximum collapse occurred for the material with the softest particles (shale). Secondly, the magnitude of collapse settlement is not as dependent on applied load as one may imagine. This is probably because the tendency for compression on wetting to increase with applied pressure is counteracted by the higher initial density of the material prior to wetting. Both these observations accord with the findings of Hills (1994).

Tests by Others

In the above experiment by Wates and Wagner at Kriel Pit 3 North, the excavation into which the fill was placed was flooded and the resulting settlement recorded. The observed collapse on saturation amounted to 14% of the 3,5m thick fill layer. This is also considered to be unrealistically high due to the excavation and replacement of the fill immediately prior to testing.

At Horsley (England), Charles et al (1984) reported on the settlements observed during the rise of the water table in a 70m deep opencast pit containing backfill which was 5 and 15 years old. A 40m rise in the water table caused settlements of 100mm – 500mm. From the settlement observations, the percentage compression was calculated for various

depths in the profile. The maximum compression of 2% was noted immediately below the highest position of the water table decreasing to less than 1% for the backfill near the bottom of the pit.

Hills (1994) attributed collapse settlement to three potential mechanisms:
- the weakening of mineral bonds in the rock by wetting;
- failure of the rock due to water entering microfissures;
- and, where mudstones or shales make up a proportion of the fill, breakdown of such rocks in water.

Hills also observed that the percentage settlement shows no apparent relationship with depth within the fill. This observation and the attribution of the collapse to failure of the rock contacts is consistent with the observations from the oedometer tests at Letabo described above.

Hills gave the following typical values for collapse expressed as a percentage of the thickness of the saturated layer.

Table 9: Mean values and standard deviation % collapse (after Hills, 1994, Table 8.2)

Placement Method		**Collapse (%)**	
		Mean	**Std Deviation**
Controlled[13]	Performance	0,25	0,04
	Method	0,40	0,08
	Thick Layer	0,90	0,28
Uncontrolled		1,20	0,41

7.3.5 *Determination of Creep Parameters*

No local determination of creep of pit backfill has been undertaken. Some long term measurements were taken at the airfield and a district road at Optimum Colliery but these have not been systematically reported or analysed. The problem of creep settlement of fills was identified by the Candidate as a potential research area during an invited contribution to the Academic-Practitioner Forum at the 16th ISSMGE International Conference in Japan in 2005 (Day, 2005).

Hills (1994), suggested the adoption of the linear relationship with log time proposed by Sowers *et al* (1965) for predicting creep of opencast backfill settlement using the following equation:

$$s = \alpha\left(log_{10}\, t_2 - log_0\, t_1\right)$$

He also provided the following typical values for the creep rate parameter α as shown in Table 10.

13 Controlled fill may be placed using a performance specification (specified % compaction), a method specification (specified method of placement aimed at achieving a compaction density comparable to that achieved with a performance specification) or a thick layer specification (material placed in 1m layers and compacted by the passage of construction equipment).

Table 10: Mean values and standard deviation of α values (after Hills, 1994)

Placement Method		Alpha α (%)	
		Mean	**Std Deviation**
Controlled	Performance	0,15	0,03
	Method	0,25	0,05
	Thick Layer	0,50	0,15
Uncontrolled		0,80	0,28

7.3.6 *Application to Practical Situations*

Bothashoek Rail Deviation, Optimum Colliery

Day and Wardle (1996) and Day (2001) reported on the construction of a public railway line across 28m of opencast backfill at Optimum Colliery. In this case, the backfill was placed in two 11m lifts, each of which was compacted using dynamic compaction. The final few metres of the embankment were constructed using conventional earthworks.

During construction, 1,2m diameter plate load tests were undertaken on the fill before and after compaction. When tested in a saturated condition, the uncompacted fill had a drained elastic modulus E' of 5,5MPa whereas the compacted material (between DC print positions) had a drained elastic modulus E' of 31MPa. The uncompacted material exhibited significant collapse when saturated at an applied load of 200kPa whereas the compacted material showed virtually no collapse.

Photo 17: Dynamic compaction underway on the Bothashoek Rail Deviation.

Settlement predictions based on these values, including the effects of downdrag of the surrounding material on the compacted prism, gave settlements of the order of 350mm for the railway line and 1,2m – 1,5m for the uncompacted material on either side.

Monitoring of the track was carried out for a period of three years after construction. No untoward performance was noted.

Duvha-Middelburg Rail Line

Day and Wardle (1996) and Day (2001) also reported on a new railway link between Duvha and Middelburg mines in Mpumalanga. Unlike the Bothashoek Rail Deviation, this new railway line was owned by and financed by the mine. A decision was taken by the mine to construct the railway line over unconsolidated spoils and to deal with any settlement that may occur. Settlements due to creep and collapse due to reestablishment of the water table were predicted to be between 200mm and 500mm at various points along the line.

Over much of the backfilled sections of the track, performance of the railway line was adequate and little additional maintenance was required. However, problems were experienced in two of the cuttings where stormwater ingress into the fill led to differential settlements of up to 600mm.

Photo 18: Differential settlement of Duvha-Middelburg rail link in cutting (Day, 2005)

As a result, the side drains had to be rebuilt. The formation itself was not rebuilt but the ballast in now up to 900mm thick in places. In retrospect, the decision to save on initial construction costs and put up with on-going maintenance has probably paid off in the long run. However, in future projects of this nature, the disruption during maintenance should be taken into account together with the difficulty that management personnel who were not party to the original decision have with accepting the situation.

Grootegeluk Backfill Project

A mechanised stacking system is being implemented at the Grootegeluk Mine near Lephalale which will dispose of both mining spoils (overburden and interburden) and plant discard, using a two tier stacking system. The infrastructure for this system is situated within the pit in an areas where 90m of backfill has already been placed. This

infrastructure, which includes conveyors, transfer bins and drive houses, is required to last the life of the mine.

Based on information gathered from the above projects and on the recommendations given by Hills (1994), predictions were carried out for the normal settlement (immediate and consolidation settlement) of the structures due to the load on their foundations and creep settlement of the fill under its own self-weight. The normal settlement of the largest structure on site, a 2 000 ton bin, was estimated to be 125mm – 150mm. The predicted creep settlement of the fill is given in Figure 34. The points A, B and C refer to locations on the backfill where the fill is 89m, 63m and 78m thick respectively. These calculations were based on a drained elastic modulus E' of 6MPa and a creep rate parameter α = 0,7.

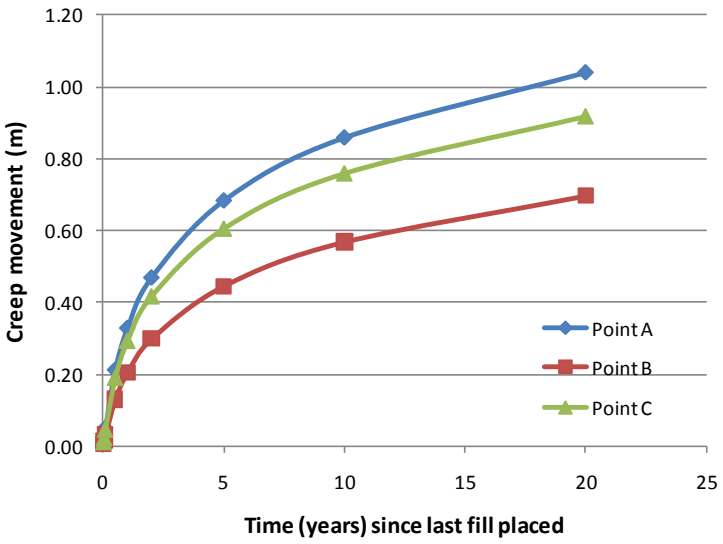

Figure 34: Creep settlement predictions at various locations at Grootegeluk Mine

In order to control the differential settlements and normal settlements of structures founded on the fill, the upper 20m of the fill was compacted in two lifts using dynamic compaction techniques. The design of the structures and the interconnecting conveyors was adapted to accommodate the expected movement.

7.4 Current Research

7.4.1 Background

In an effort to obtain more monitoring data on the settlement of pit backfill, the fill below the transfer bin at Grootegeluk has been instrumented and will be monitored for as long as possible before the instrumentation is covered with backfill. By this time, it is hoped that the structures on top of the fill will be completed and monitoring of these structures will already be in place thereby extending the monitoring of the fill well into the future. The aim is to obtain high quality monitoring data from which the drained elastic modulus and the creep rate parameter of the pit backfill can be determined.

7.4.2 Instrumentation

The area that has been instrumented is below the main pedestal where the backfill is 90m thick. During a break in placement when the fill was 40m below its final elevation, three

150m long x 50mm diameter UPVC monitoring tubes were installed in shallow trenches in the surface of the backfill. A pulley-box was provided at the far end of each tube and a return pipe, fitted with a stainless steel draw wire. Headwalls were constructed at the proximal end of each installation through which the monitoring tube and the tube for the draw wire emerge. The headwalls were located on level ground about 10m outside the area where further backfilling was to take place – see Figure 35. The level of the tubes was determined at 5m intervals along their entire length by survey immediately after installation. The trench in which the tubes were laid was then backfilled and the placement of the remainder of the backfill proceeded over a period of about six months.

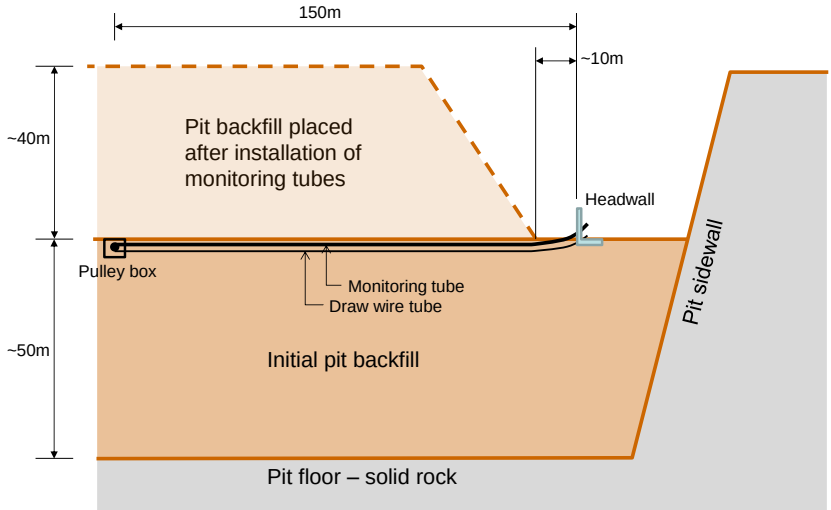

Figure 35: Schematic of settlement monitoring installation

Photo 19: Headwall and "hydro-profiler" instrumentation

At intervals after recommencement of backfilling, a "hydro-profiler" settlement gauge was pulled along the monitoring tube using the draw wire. The hydro-profiler consists of a monitoring probe that is connected via a small diameter plastic pipe to a reservoir of water/glycerine mix contained in the centre of the hose reel (see Photo 19). The probe contains a sensitive pressure transducer that reads the head difference between the level

of the fluid in the reservoir and the probe to the nearest millimetre and displays the reading on a read-out unit mounted in the hose reel. The probe is drawn into and extracted from the monitoring tube in 3m increments and a reading taken at each increment. The readings taken on the way in and those taken on the way out are averaged to obtain the settlement profile of the monitoring tube. This process is repeated at each of the three monitoring installations. The resolution of the gauge is 1mm and a total system accuracy of ±20mm is claimed by the manufacturers. Tests on site suggest that the system accuracy may be closer to ±50mm.

7.4.3 Preliminary Results

The results to date from one of the three probes are given in Figure 36 and Figure 37.

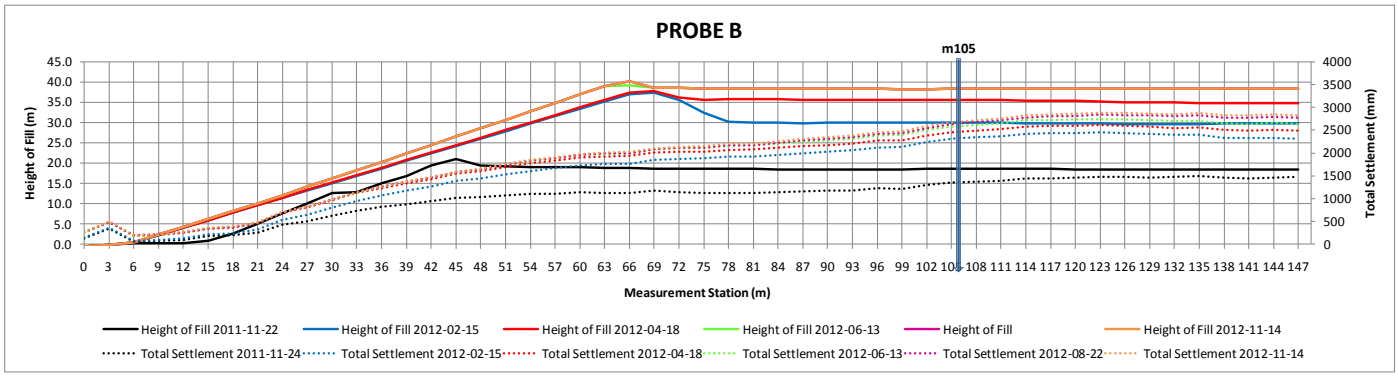

Figure 36: Settlement profile along length of probe 'B'

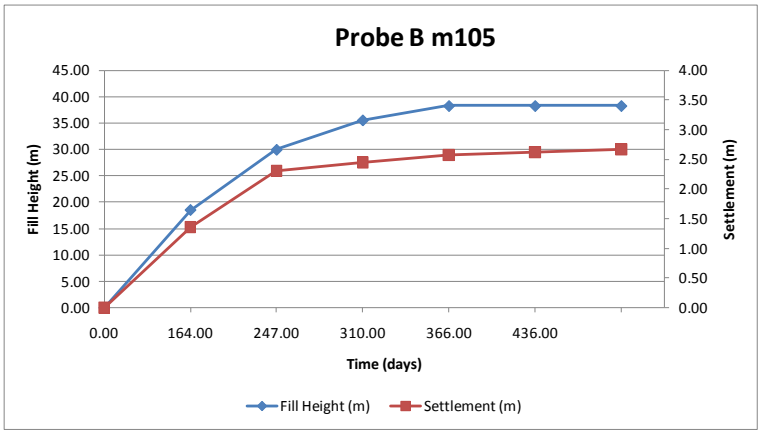

Figure 37: Fill height and settlement record at chainage 105m for probe 'B'

Based on the most recent readings from each probe, the drained elastic moduli E' of the bottom 50m of pit backfill at the three test locations are 10,1MPa, 10,6MPa and 13,6MPa respectively. This provides a second set of credible data from full scale monitoring of pit backfill in South Africa and should be compared to the value of 6MPa measured at New Vaal Colliery (see 7.3.3 above). One of the possible reasons why the observed modulus at Grootegeluk is higher than that at New Vaal is the method of placement used. At Grootegeluk, the fill was placed in 1,5m layers and then levelled with a dozer before the next layer was placed. The material was thus nominally compacted by the dozer and the dump trucks used to deliver the fill. At New Vaal Colliery, the material was placed partly by trucking and dozing and partly by dragline.

The height of the backfill above the monitoring tubes has been constant since 13 June 2012. Since then, two sets of readings have been taken, on 22 August and 14 November. Table 11 summarises the average of the recorded readings over the final 30m of Probes B and C, both of which are close to a point on the fill for which creep movements were predicted. The information from Table 11 is plotted in Figure 38 together with the creep predictions for the fill at the level of the monitoring probes (~860m amsl) and on the surface of the fill (~900m amsl).

Table 11: Summary of creep measurements

Observation	Probe B	Probe C
Average Settlement:		
13 Jun 2012	2,692m	2,492m
22 Aug 2012	2,802m	2,669m
14 Nov 2012	2,852m	2,715m
Average movement:		
13 Jun – 22 Aug	110mm	177mm
13 Jun – 14 Nov	160mm	223mm
Rate of movement:		
13 Jun – 22 Aug	1,57mm/day	2,52mm/day
22 Aug – 14 Nov	0,59mm/day	0,55mm/day

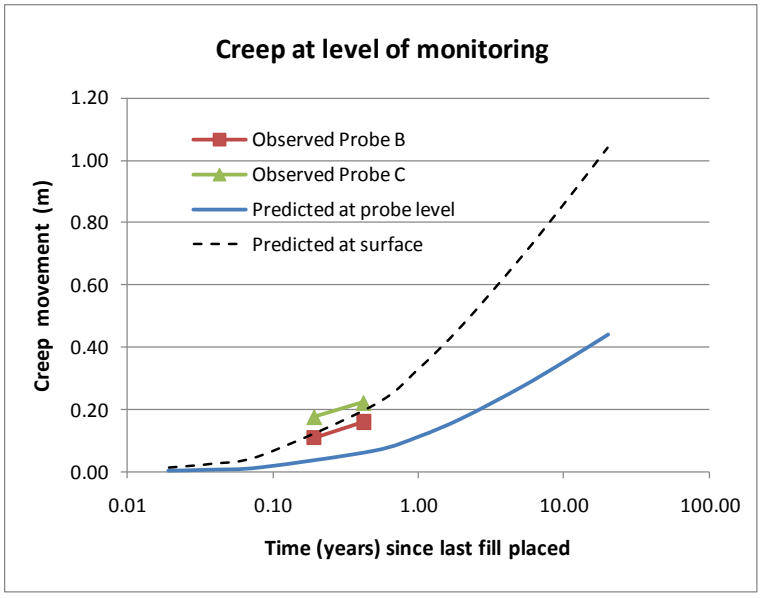

Figure 38: Plot of observed and predicted creep movement

The following observations are made from Table 11 and Figure 38:

- The creep predictions are non-linear on a log-time plot. This is because the fill was not all placed at the same time with the fill in the lower layers being significantly older than that near the top of the embankment. As time progresses (t > 1 year), this influence becomes less marked and the creep prediction becomes more linear.

- The magnitude of the observed creep movement over the past five months (13 Jun to 14 Nov) is higher than predicted at the level of the probes, being closer to that predicted for the surface of the fill.
- The rate of creep (slope of lines) observed over the most recent monitoring interval (22 Aug – 14 Nov) lies somewhere between the creep rate predicted for the surface of the fill and that predicted at the level of the probes. It has reduced significantly from that in the monitoring interval from 13 Jun – 22 Aug. The rate of creep over the last monitoring interval is between 0,55 and 0,60 mm/day compared to a predicted creep rate at the level of the monitoring probes of between 0,35 and 0,40 mm/day.

The fact that the observed creep movement exceeds that predicted at the level of the probes could be the result of an incorrect assumption regarding the value of the creep rate parameter or some normal[14] settlement extending into the monitoring period. Survey accuracy could also be a factor.

Based on the current observations, the observed creep movements are greater than the predicted movements but the creep rate is similar to that predicted. It is too early to predict the final creep settlement of the fill. Such predictions should become possible within the next 12 months when the predicted creep movement becomes more linear on the log-time plot and the possible influence of lingering normal settlement decreases.

7.5 Conclusions

From the above case histories, it is clear that significant benefit can be obtained by monitoring full scale load tests as part of the site investigation process. In addition, if correctly planned and executed, this monitoring need not be prohibitively costly.

At Saldanha Steel, the full scale load test added less than 5% to the cost of the investigation but allowed the designers to omit costly pile installation below large areas of the plant. At New Vaal Colliery and Grootegeluk Mine, the cost of the full scale load tests amounted to the cost of the monitoring alone as the embankments that were monitored formed part of the normal construction process. The data obtained from these projects is regarded as significantly more reliable than can be obtained even from sophisticated laboratory tests and calculation models.

By conducting such tests and monitoring of completed structures on a routine basis, we will be in a better position in future to estimate parameters that are vitally important to our prediction of how structures on fills or soil profiles not amenable to conventional analysis will perform.

Publishing the results of such studies is of critical importance to the industry and the Candidate wishes to express his gratitude to Exxaro for permission to present the results from the Grootegeluk monitoring in this dissertation.

7.6 References

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[14] Normal settlement is settlement due to load application. Creep is settlement under constant load.

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8. INTERNATIONAL ACTIVITIES

This chapter of the dissertation describes the Candidate's involvement in geotechnical engineering on an international level.

8.1 ISSMGE TC 23: Limit States Design in Geotechnical Engineering

The Candidate's involvement with and chairmanship of this technical committee are dealt with in Section 12.

8.2 Representing Africa on the ISSMGE Board

The membership of the International Society of Soil Mechanics and Geotechnical Engineering comprises member societies from the various countries around the world. Its membership is subdivided into six regions; Africa, Asia; Australasia; Europe, North America and South America. At the Council Meeting corresponding with the International Conference which is held on a four year cycle, each region elects a Vice President to represent the region on the Board of the ISSMGE for the next four years. In August 2001, at the Istanbul Council meeting, the Candidate was elected as the Vice President for Africa.

During his term on the board, the Candidate assisted in formulating changes to two of the most vexing questions facing the Society at the time, namely voting rights and membership fees.

The various regions consist of the following number of member societies, each having one vote.

Africa	9
Asia	23
Australasia	2
Europe	38
North America	3
South America	13

Although there are a number of issues that are put to the vote, the really hotly contested issues are the election of the President of the Society and the venues for the next mid-term Council Meeting and International Conference. The disparity in the number of member societies in the various regions puts some of the regions at a disadvantage despite the number of individual members they represent. For example, Australasia has never hosted an international conference nor has one of its members been elected as President. This is despite the significant stature of individuals like Professor Harry Poulos who is without peer in the industry. Such is the power of the larger regions.

After much debate on voting rights proportional to the individual membership of each member society, the issue regarding the venue of meetings and conferences was resolved by a simple change to the bye-laws of the Society to the effect that a region that has hosted one of these events is not eligible to bid for the hosting of the next two such events. The nett result of this was that an international conference or mid-term council meeting can only return to a particular region once every twelve years. This provided a solution that was simple to understand, accept and administer.

At that time, the fees paid by each member society consisted of a basic fee per member society and a per-capita fee for the number of individual members. The basic fee per society was weighted according to a Group Number based of the Gross Domestic Product per capita of the country concerned. The intention was to ease the burden on the poorer

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countries. Unfortunately, this formula had the opposite effect with countries like the UK and the USA paying among the lowest fee per capita and countries like Morocco and Tunisia paying among the highest fees per capita. This was due to the differences in individual membership numbers. The USA and UK had 2400 and 1400 members respectively among whom to spread the basis fee per society whereas both Morocco and Tunisia had less than 15 members. A new fee scale was needed that would not only remove these disparities but also discourage the practice among some of the smaller societies to register a minimum number of their members in order to reduce the fees paid.

To resolve the issue, the basic fee per society was scrapped and the fee paid per society was based purely on a per capita fee. To take account of the relative wealth of the country, a discount of up to 75% of the fee per capita was determined on a sliding scale for countries with a PPP (purchasing power parity) of less than 15 000[15]. However, this measure alone would not encourage membership. To do so, a further discount of up to 60% of the already discounted fee was offered on a sliding scale for societies with more than 250 individual members. In addition, member societies were obliged to pay for a minimum of 30 individual members per society.

The net result of this was that the richer, more populous member societies received a fee increase of about 15% while the poorer countries not affected by the minimum membership provision received discounts of up to 67% on what they were paying previously. Incidentally, South Africa with its 261 members at the time received only a minor discount for numbers and landed up paying 45% more. The success of this revised fee scale has been immediately evident in countries like Egypt and Tunisia where membership numbers have grown from less than 20 to 108 and 40 respectively.

These achievements may sound trivial but it is never easy to get agreement among so many diverse opinions and cultures from all over the world.

In addition to serving on the task teams for voting and fees, the Candidate also served on the Industrial Liaison task team aimed at ensuring that the Society, which is largely dominated by academics, remains relevant to geotechnical engineers in industry (consultants, contractors, suppliers, etc.). He also served on the task team that looked into the formation of an umbrella body for the geotechnical and geological sciences including the Association of Engineering Geologists, the International Society of Rock Mechanics and the ISSMGE. This resulted in the formation of a body known as *FIGS* (Federation of Geo-engineering Societies). Unfortunately, it was a short-lived organisation as the incoming presidents of two of the founding societies did not support the idea.

Apart from being one of the few non-academics to be elected to the ISSMGE Board, one of the Candidate's other "claims-to-fame" was arranging the first ever ISSMGE Board meeting that was not associated with another official ISSMGE event (such as a conference or other such event). It was also the first Board meeting to be held in sub-Saharan Africa (the only other one having been in Cairo) and the only Board meeting to be interrupted by elephants straying too close to the meeting venue in the middle of an unfenced game park. It was also one where many of the Board members brought their families along just to enjoy the event.

15 To put this figure in context, in 2004 the USA, Norway and Switzerland had PPPs in excess of 35 000. Iraq, Nigeria and Kenya had PPPs of less than 1000. Countries like the Czech and Slovac Republic, Singapore and Korea had PPPs around the 15 000 mark. South Africa's PPP at the time was 9 870.

8.3 Conferences and Lectures

8.3.1 African Regional Conferences

Regional Conferences of the ISSMGE are held every four years, midway between the International Conferences. There is considerable competition between the nine African member societies to host the conference. Despite South Africa having more individual members of the ISSMGE than all the other African countries combined, it must wait its turn to host such a conference, not only as a result of trying to get a reasonable geographical spread north and south of the equator, but also having to accommodate the needs of both French and English speaking nations. There have also been political situations that had a bearing on the situation. For example, South Africa was unable to attend the 7th African Regional Conference in Accra Ghana in 1980 and held its own satellite conference in Pretoria at which the South Africa papers to the Ghana conference were presented.

A list of the venues of the Africa Regional Conference to date is given in Table 12.

Table 12: Venues of ISSMGE African Regional Conferences

No.	Year	Venue
1st	1955	Pretoria, South Africa
2nd	1959	Lourenço Marques, Mozambique
3rd	1963	Salisbury, Rhodesia
4th	1967	Cape Town, South Africa
5th	1971	Luanda, Angola
6th	1975	Durban, South Africa
7th	1980	Accra Ghana
8th	1984	Harare, Zimbabwe
9th	1987	Lagos, Nigeria
10th	1991	Maseru, Lesotho
11th	1995	Cairo, Egypt
12th	1999	Durban, South Africa
13th	2003	Marrakesh, Morocco
14th	2007	Yaoundé, Cameroon
15th	2011	Maputo, Mozambique

The Candidate has been involved the organisation of five of these conferences. The Lesotho conference (1991) was organised by South Africa but held in Lesotho for political reasons. This was when the Candidate was secretary of the Geotechnical Division and was intimately involved in the organisation of the conference. For the Durban Conference in 1999, the Candidate was chairman of the organising committee. His election as vice-president for Africa can be attributed directly to the success of this conference. The Morocco Conference in 2003 was during his term as vice president and he was part of the organising committee. For the two remaining conferences in Cameroon and

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Mozambique, he served on the scientific committee and assisted with the refereeing of papers.

8.3.2 Jennings Lectures

The Jennings Memorial Lectures are hosted by the Geotechnical Division on a more-or-less annual basis. The aim is to invite respected international speakers to South Africa to deliver the main lecture in Johannesburg, often with repeat lectures in Durban and Cape Town.

As a result of his international involvement, the Candidate has been responsible for inviting and hosting five of the ten Jennings Lecturers to date including Prof Bengt Broms (Sweden), Prof Luiz de Mello (Brazil), Prof Malcolm Bolton (UK), Dr Jorgen Steenfelt (Denmark) and Prof Roger Frank (France) with Prof K.K. Phoon still to come in 2013.

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Part 3:
Limit States Design in Geotechnical Engineering

This Part of the dissertation describes the introduction of limit states design in geotechnical engineering in South Africa. It reviews the contribution made by the Candidate including his contribution to the writing of a new *basis of design and actions* code making specific provision for geotechnical design and actions.

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9. OVERVIEW AND TIMELINE

9.1 South African Geotechnical Design Codes in the 1990's

Traditionally, geotechnical engineering practice in South Africa has not relied heavily on codes or standards. Instead, South African geotechnical engineers have based their practice on authoritative technical publications from South Africa and abroad, respected text books and reference works. In some instances, notably geotechnical field and laboratory testing, reference was made to international standards mainly ASTM and British Standards.

A good example of this situation was the South African piling industry. Although SABS 088 - Piled Foundations had been issued in 1972, this standard had limited technical merit and was largely ignored by the industry. This code has subsequently been withdrawn. In the 1990s, the references most commonly used in the industry were Poulos and Davis (1980), Tomlinson (1987) and the second edition of the "Frankipile book" (Frankipile, 1986). The proceedings of local, regional and international conferences arranged by the ISSMFE, SAICE and others provided additional information.

Other sectors of the industry developed similar collections of reference works. For example, soil profiling and rock logging by geotechnical engineers and engineering geologists used the Jennings, Brink and Williams paper (Jennings et al, 1973) and the SAAEG guidelines (SAAEG, 1976) as *de facto* standards. Roads engineers and those involved in geotechnical investigations for roads used TRH14:1985 as a standard for road construction materials and TMH1:1986 as a standard for testing of materials. Bridge designers had their own code in TMH7:1981.

Two of the few formal codes in use at the time were the recently completed lateral support code (SAICE, 1989) and the code of practice for safety of men in trial holes (SAICE, 1980).

The "loading code" (SABS 0160-1989) made scant reference to geotechnical loading (Day, 2000). Loads on earth retaining structures and internal pressures in silos, tanks, etc. were specifically excluded from the code. In Table 2 of the code, no load factor was assigned to the earth pressure. The load factor assigned to "other types of imposed loads" was 1,6. Clause 5.8.3 on lateral and uplift forces simply states that *basement walls and similar members must be designed for the lateral forces applied by the adjacent soil, for any surcharge on the soil and for hydraulic forces*.

SABS 0161-1980 (design of foundations for buildings) was another example of a code with limited technical merit. Although this code has not been withdrawn, it has largely been superseded by the SAICE 1995 *Code of Practice for Foundations and Superstructures for Single Storey Residential Buildings of Masonry Construction* and by the NHBRC's *Home Building Manual*.

In 1993, when the Candidate was Chairman of the Geotechnical Division, the Division formed a sub-committee to look into the possibility of writing a new code of practice for piling and possibly even a geotechnical design code. The sub-committee met only once. Three things were agreed at this meeting. Firstly, any new code should preferably be written in terms of limit states design. Secondly, there was insufficient expertise in South Africa in the field of geotechnical limit states design to write such a code. Finally, it was agreed that the Division should keep an eye on the development of the Eurocodes, Eurocode 7 in particular.

9.2 Limit States Design Seminar - 1995

In May 1993, the Danish Geotechnical Society held a conference on Limit State Design in Geotechnical Engineering in Copenhagen. In keeping with the last of the three decisions recorded above, the Candidate attended the conference. This brought the South African geotechnical fraternity into contact with Dr Niels Krebs Ovesen of the Danish Geotechnical Institute and Dr Brian Simpson of Arup (Consulting Engineers) in London. Krebs Ovesen was convenor of the conference and Chairman of the ISSMGE's Technical Committee TC23 on Limit States Design and Simpson was the deputy chairman.

After returning to South Africa and holding discussions with the Geotechnical Division Committee, the Candidate invited Drs Krebs Ovesen and Simpson to visit South Africa and present a seminar on limit states design in South Africa. Both agreed and the seminar was held in Johannesburg from 16 – 18 November 1995.

For course notes, the seminar made use of the European pre-standards ENV1991-1:1994 (*Basis of Design and Actions on Structures*) and of ENV1997-1:1994 (*Geotechnical Design*) together with various technical publications by the presenters. Krebs Ovesen dealt mainly with the requirements of Eurocode 7 while Dr Simpson dealt with the selection of characteristic values and the design of retaining structures. In his contribution to the "Spirit of Kerbs Ovesen" session at the 14th ISSMGE European Conference in Madrid following Krebs Ovesen's death in 2005, the Candidate recalled how Krebs Ovesen had lectured from his personal copy of the ENV standard with his meticulously hand-written comments and alterations in the margins (Day, 2007). It was small things like these that gave the delegates to this seminar a sense of the dynamic nature of the emerging Eurocodes.

One of the main outcomes of the seminar was realisation that revisions would be required to the South African "loading code" (SABS 0160) if Eurocode 7 was ever to be adopted as a geotechnical design code in South Africa.

9.3 South Africa National Conference on Loading – 1998

9.3.1 Background to SANS 10160 (Day and Kemp, 1999)

In 1983 a Working Group was established by the Structural Division of SAICE to make recommendations on an appropriate policy for limit-states formulations in South African structural codes. This Working Group identified the following objectives:

> *"to select a limit-states model from those under consideration internationally and to define the structure for the preparation of future limit-states codes for structural loading and all individual structural materials in South Africa on a consistent basis".*

The Committee surveyed the limit states proposals being developed in Europe, Britain, United States, Canada and Australia and identified the following principles:

The partial load factors, reflecting uncertainty in the applied loads and other actions, should be specified in the code and should apply uniformly to all materials.

The Turkstra proposal for load combinations at the ultimate limit-state should be considered.

The loading code should identify a partial factor which would increase the margin of safety in the case of structures where crowds gather or which are of national importance, or reduce this margin for minor structures not generally accessible to the public.

The resulting load factors and combination factors, formed the basis of the limit-states provisions in the revised South African "loading code" SABS 0160:1989. Despite being

developed independently, the load factors and combination factors contained in this code were similar to those used in North America. They did, however, differ significantly from those contained in the draft Eurocode 1.

In SABS 0160, the load factors and combination factors were chosen to yield a load index of approximately 2,0 at the ultimate limit-state over a wide range of load combinations. This corresponds to a design load which has a 1% probability of being exceeded during a 50 year design life. Although these factors were chosen without consideration of the statistics of member resistances, analysis by Milford (1988) showed that the load factors in SABS 0160 would result in a relatively uniform probability of failure over a wide range of load combinations. A safety index of approximately 3,0 could be obtained by selecting material factors to achieve a design member resistance equivalent to a 1% fractile. (Day and Kemp, 1999.)

9.3.2 Background and Aims of the Conference

After the Seminar on Limit States Geotechnical Design in South Africa, the Geotechnical Division engaged with the Joint Structural Division of SAICE and the SA Institute of Steel Construction regarding the possibility of convening a forum for discussions on the revision of the South African loading code. The idea found support from both these bodies and, in particular, from Prof Alan Kemp who had been a leading figure in the drafting of the 1989 edition of the code. The three institutions decided to convene a South African National Conference on Loading and appointed the Candidate as chairman of the organising committee.

In the final announcement of the conference issued in mid-1998, the Candidate wrote as follows regarding the background to the conference:

> *In the early 1980's, the Structural Division of SAICE made recommendations on the formulations which are included in the current "loading code" SABS 0160-1989. The intention was that these recommendations would form the basis of future South African structural codes. Since this time, very few South African structural codes have been written. Instead, we have continued to use codes from overseas, adapting these to be compatible with the load factors given in our local loading code. The question arises whether we should continue with this practice. If we are going to use overseas codes, should we not adopt them as they stand – load factors and all? Is there any real advantage in having a common set of load factors for all disciplines in South Africa?*
>
> *Publication of the Eurocodes has rekindled interest in this issue. The Structural Division's links with the Institution of Structural Engineers will favour the adoption of the Eurocodes for concrete design. The Geotechnical Division has agreed in principle to the adoption of Eurocode 7 in preference to writing a South African geotechnical design code. It, however, appears unlikely that the steel designers will follow suit, preferring instead to use the Canadian Code.*
>
> *The South African civil engineering fraternity is now faced with a choice. Should we:*
> - *continue to adapt overseas codes to conform with SABS 0160,*
> - *adopt the Eurocodes for all disciplines of civil engineering in South Africa,*
> - *look around for an alternative set of harmonised codes, or*
> - *abandon the ideal of common load factors applicable to all disciplines?*

In a similar vein, the aim of the conference was given as follows:

> *The conference will have a dual purpose:*

a) To review the limit states design approaches and loading codes used internationally and to compare these with South African practice.
b) To provide guidance to practicing engineers on various common types of loading and to discuss the requirements of the available codes.

The aim of the conference is to canvas the opinion of practicing engineers, researchers and academics on the approach to be adopted in South Africa. The decisions taken will have an influence on materials codes in all disciplines.

9.3.3 The Conference

The conference was held on 9th and 10th September 1998 and attracted 105 delegates.

Day 1 was devoted to loading philosophy and international practice.

Professor Laurie Kennedy (professor of Civil Engineering, University of Alberta, Edmonton, Canada) reviewed the development of loading codes in Canada and the United States. He expressed the view that the differences that exist between the codes on the North American continent are likely to disappear. He was complimentary of the rational approach adopted by South Africa in the development of their code.

Dr John Menzies, (UK engineering consultant and chairman of the Eurocode 1 drafting committee) outlined the loading provisions contained in Eurocode 1. Dr Brian Simpson, (UK engineering consultant and vice-chairman of the Eurocode 7 drafting committee) dealt with the rather unique problems associated with the application of limit-states design to geotechnical engineering. Both authors were subjected to some rather probing questions about the lack of calibration of the Eurocodes and the high variability of the load index across the range of loading.

Dr Lam Pham (Chief Research Scientist, CSIRO, Victoria, Australia) dealt with current practice and expected developments in Australia and the Asian Pacific Economic Community. He indicated that Australia finds itself in much the same position as South Africa, having difficulty reconciling the differences between the European and North American approaches. (Day and Kemp, 1999.)

Local contributions were also received from Prof Alan Kemp (introduction and background), Dr Rodney Milford (philosophy behind SANS 0160) and Dr Graham Cross (interaction between disciplines).

Day 2 dealt with specific loading situations which included wind loading (Dr Adam Goliger), seismic loading (Dr Danie Wium), bridge loading in South Africa and North America (Peter Fitzgerald and Laurie Kennedy), silos and tanks (Prof Geoff Blight), earth pressures (Dr Brian Simpson), crane and machinery loads (Dr Geoff Krige), conveyor loading (Graham Spriggs), floor loading (Dr John Menzies), temperature loading (Dr Geoff Krige) and finally shrinkage and creep (Dr Graham Cross).

The conference wrapped up with a panel discussion chaired by Professor Kemp and the Candidate.

9.3.4 The Outcome

During the discussions, it was generally agreed that SABS 0160 is technically sound. In particular the principle of a uniform load index and the use of the Turkstra rule received strong support. A warning was sounded not to confuse the ultimate and serviceability limit-states by attempting to control serviceability by using conservative load factors.

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The following resolutions were taken (Day and Kemp, 1999):

- A working group of the Structural Division is to be formed to review the S.A. approach and to extend and revise SABS 0160 where necessary.
- South Africa will increase its co-operation with ISO by means of technical representation from the Institution in conjunction with SABS.
- The various Divisions will establish technical committees to look at material codes.
- Co-operation with SADC countries is required.

The Joint Structural Division requested Prof. Alan Kemp to convene a committee to consider possible changes to the South African loading code. This committee was to include representation from the SABS, SAISC and the Geotechnical Division. The first meeting was scheduled for March 1999.

Following their visit to South Africa, John Menzies and Brian Simpson compiled a joint report which was submitted to the European Committee for Standardisation. They concluded that the difficulties identified with the Eurocode at the South African conference could militate against international adoption of the Eurocodes and recommended increased collaboration between European and American code development agencies.

9.4 Development of SANS 10160:2011

The description of the development of the new code SANS 10160 below is intended to provide a broad overview of the process. Particular mention is made of the provisions for the inclusion of geotechnical actions and basis of design. Comments are also provided on the "evolution" of the section of the code dealing with wind actions as this section, more than any other, illustrates the extent to which the later development of the code was influenced by the Eurocodes.

9.4.1 Early Meetings

The first meeting of the "loading code committee" was held on 23 March 1999 with Prof Alan Kemp in the Chair. It was attended by 10 delegates representing a wide range of interests in the civil engineering field. Among these delegates was Prof Peter Dunaiski who would later take on the position of Chairman of SABS SC 59I and see the new loading code through to publication. The Candidate took on the duties of committee scribe and host, a role which he fulfilled for the ten years of the committee's existence. A list of 15 committee members was established.

Following a meeting between Prof Kemp and the SABS the previous week, it was agreed that the committee would serve both as the Joint Structural Division Loading Code Committee and as a sub-committee of a SABS Technical Committee 5120 "*Design and Construction of Buildings and Industrial Structures*", namely "*SC1: General Procedures and Loading*". The SABS status of the committee was to change many times during the years of its existence ending up finally as SC 59I of Technical Committee TC 59 *Construction Standards*.

Prof Kemp was of the opinion that the Loading Conference has focused too much on load factors and that this was to the detriment of other areas requiring consideration including wind loads, floor loads, earth pressures, earthquake loads, crane loads, etc. A number of topics that required consideration were identified (including geotechnical loading) and champions were appointed to prepare short reports on each of these subjects. It is interesting to note that the report prepared on geotechnical loading (Day, 1999) only made recommendations for dealing with earth pressures as loading on structures within the scope of the code. This somewhat limited vision of looking only at loads exerted by soils

on structures persisted for many years before it was decided to include comprehensive treatment of the basis of geotechnical design in the new code.

The second meeting of the committee in May 1999, Professor Kemp stated "*The objective of the Working Group is to produce an up-to-date, technically comprehensive and accurate loading code. The adoption of an internationally recognised code (and adaptation to South African conditions) can be considered provided this does not compromise accuracy and comprehensiveness.*" The scope of the code would be consistent with the requirements set by the SABS Technical Committee and would therefore not deal with bridge loading or loads exerted by stored materials. The code would define a single set of load factors that would apply to all materials codes. Even at this early stage, the intention to produce an entirely new code and not just a re-vamp of the old code was clear.

In June 1999, a special meeting was held which was attended by Prof Ted Galambos, Chairman of the Sub-Committee on Load Combinations for the ANSI/ASCE 7-95 Code. The main purpose of this meeting was to discuss the ANSI/ASCE code with Prof Galambos. Far ranging discussions were held on topics including serviceability requirements, classification of structures, basic load combinations, accidental loading, imposed loads including cranes and machinery loads and, finally, earthquake loading. There was also discussion on the Eurocodes with Prof Galambos expressing the view that it was unlikely that the Eurocodes would ever take precedence over the ASCE codes in the USA.

At the next meeting in August 1999, reports were again received from the various champions. Professor Kemp expressed concern about the slow progress. Little did he know at that stage that the code would take another ten years to finalise and that he would not live to see the final document. Discussions were held regarding the merits of drafting a uniquely South African code as opposed to adopting an existing code. The meeting ended with two critical questions being asked of the committee:
Do we agree that we can use the South African code in conjunction with the material codes of choice provided the resistance factors implicit in these materials codes fall within a specified range? There appeared to be agreement that this was indeed the case but that the matter needed to be referred back to the materials disciplines for their input.

Are the champions prepared to invest time in the process of drafting a new "best practice" South African code? This was left as a rhetorical question.

At the November 1999 meeting, the following guiding principles were adopted:
The revised loading code should be suitable for use in conjunction with any internationally acceptable materials code which may be selected by the materials code sub-committees. The only proviso is that the probabilistic model for material resistance in the chosen materials code should be evaluated and found to be compatible with the revised SA Loading Code.

The SA Loading Code should remain a "best practice" code and should be compatible with an established overseas code such as ASCE-7.

The revised SA Loading Code could allow for the alternative use of Eurocode 0 and Eurocode 1 in conjunction with the materials Eurocodes.

Recent research based on available load statistics has confirmed that SABS0160 consistently achieves a reasonable uniform safety index over the practical range of load ratios for various material types and failure mechanisms. This means that the current SABS0160 code is, in principle, compatible with a range of internationally accepted materials codes (those based on the principles of limit states design). The emphasis of the committee's work should therefore be placed on how compatibility can best be achieved with the materials codes selected by the relevant sub-committees.

It was reported at this meeting that the Joint Structural Division was considering the adoption of Eurocode 2 for concrete design. It was also reported that the University of Stellenbosch was engaged in research on crane loading. The results of this research were to be available within 18 to 24 months after which they could be incorporated into the code. The minutes again indicated the chairman's expectations of an early conclusion of the task with the words "*This may be too late*".

Three meetings were held during 2000 at which slow but steady progress was made in each of the focus areas. Most of these focus areas were later to become independent parts of the new code. Since establishment of the committee, there had been discussion on the merits of the various approaches to wind loading in codes throughout the world. The view was expressed that the draft Eurocode was overly complicated but that the United States (ASCE) code and the Australian (AS) code should be considered as models for the new South African Code. In June 2000, the scales began to tip in favour of the Australian Code. There was still optimism regarding early completion of the code with a tentative date being set at two years after the date of the first meeting.

There were three meetings of the committee in 2001. On the geotechnical front, it was reported that the Geotechnical Division saw more urgency drafting a code of practice on site investigation than drafting a geotechnical design code. The Division also put forward a proposal for the withdrawal of SANS 088 (Piled Foundations) and SANS 0161 (Foundations for Buildings) both of which were out-dated[16]. After a further expression of frustration by the chairman about the slow progress on the code it was proposed that the committee should move from "discussion and information gathering mode" to "drafting mode". As a result, the chairman agreed to work through the old code and allocate responsibility for the drafting of each section of the new code to members of the committee. These responsibilities were tabled at the first meeting of 2002. At this stage, there was still no talk of a geotechnical design section in the new code. The first draft of the section on wind loading based on the Australian code was tabled in October 2001.

In 2002, the further work being done by the Australians in the field of wind loading was noted. Concerns were expressed that the new Australian code would be more complicated to apply than the existing code. Discussions at meetings were focused on particular topics rather than holding general discussions on all aspects of the code.

In 2003, there were further discussions on the merits of the Australian approach to wind loading versus the more complicated approach that was now emerging from the Eurocodes. In the November meeting, the Candidate questioned the relevance the new code would have for geotechnical design. In the case of the Eurocodes, Eurocode 0 and Eurocode 1 laid the basis for geotechnical design in accordance with Eurocode 7. This was not the case with the emerging South African code. He requested that the committee should ensure sufficient compatibility with the Eurocodes to enable adoption of Eurocode 7 as a South African geotechnical design code. Despite again noting the complexity of the wind loading requirements in the draft Eurocode, further comparisons were undertaken between the draft proposals for the South African code and the draft Eurocodes.

9.4.2 2004 and a Move towards the Eurocodes

The April 2004 meeting of the committee was attended by Prof. Milan Holický, a visitor to the University of Stellenbosch and Head of the Department of Structural Reliability at the Klokner Institute, Czech Technical University in Prague. At the meeting Prof. Holický made a presentation on EN1990-2002. There were further discussions regarding the

[16] SABS 088 was subsequently withdrawn. However, SABS 0161 was referenced by other codes and has still not been withdrawn.

treatment of wind loads in the Eurocodes and it was noted that considerable progress had been made with re-drafting the section on wind loading in terms of the Eurocodes. The point was again raised that the code did not make provision for geotechnical design and dealt only with earth pressures on structures. It was agreed that this was the right way to proceed until such time as the Geotechnical Division had made up its mind on the drafting of a geotechnical design code.

At the first meeting of 2005, it became clear that the University of Stellenbosch had decided to take the lead. Working groups had been set up for each of the eleven sections of the code, all but one headed by representatives of the University. At this meeting it was decided that the term "characteristic value" which had been debated at previous meetings was now sufficiently defined in the Eurocodes for it to be adopted in the new code.

The June 2005 meeting of the committee was a turning point for the inclusion of geotechnical design in the new code when it was agreed at the meeting that a new section be added to the code for geotechnical design. It was also an important meeting with regard to harmonisation of the new code with the Eurocodes. Tim ter Haar reported on the CEN TC250 meeting that he had attended the previous week indicating the CEN was actively working on the adoption of the Eurocodes outside Europe and that CEN TC250 fully supports the use of sections of the Eurocodes in the new South African Code.

Prof. Holický again attended the September 2005 meeting of the committee and made a presentation on *Reliability Basis of Partial Factors, Load Combinations and Geotechnics*. By this time, drafts of most of the section of the new code apart from geotechnical loading had reached an advanced stage. There was a now stated commitment to using the Eurocodes as reference codes for the new South African code.

By mid-2006, first drafts of all sections of the code were tabled. The section on geotechnical loading was still at concept stage and the focus had shifted from geotechnical loading alone to include the basis of geotechnical design. The June 2006 meeting was the last to be chaired by Professor Kemp who was not in good health. Chairmanship of the committee was taken over by Dr Graham Grieve. By November 2006, all sections of the code, including the geotechnical section, were completed and were circulated to various members of the committee and to selected members of the profession for review. Modifications were proposed to the combination of actions to be used in the verification of the ultimate limit state. A new GEO limit state was introduced into the first draft of what was to become Table 3 of SANS 10160-1.

In 2007, the University of Stellenbosch's Institute of Structural Engineering requested permission to prepare a commentary on the new code. This was later to be published as a background report in 2008 with sections dealing with each part of the code. Preparations commenced for a series of seminars in the major centres around South Africa to present the draft code to the profession. At the May meeting of 2007, the format for publication of the code by the South African Bureau of Standards (SABS) was discussed. The proposal was that the code would consist of nine parts and would be published as a single document.

In 2008, the final format of the code was decided in conjunction with SABS. The code would be published in eight separate parts, with Part 1 (Basis of Structural Design and Actions) being referenced by each of the other parts. The remaining seven parts dealt with the various categories of loading. Part 5 was somewhat of an exception as, in the absence of a geotechnical design code in South Africa, this part presented the *basis of geotechnical design* in addition to dealing with *geotechnical actions*. The committee drafts of all parts of the standard were presented at a series of seminars held in Port Elizabeth (7th October), Cape Town (9th October), Durban (14th October) and Midrand (16th October).

During 2009, the standard passed through the Committee Draft stage. It then went on to the DSS (Draft South African Standard) stage during which it was open to comment by the public and the profession. The final code was published by SABS in May 2010. Minor revisions were made in 2011 with the current version of the code being SANS 10160:2011.

9.5 Towards a South African Geotechnical Design Code

On 22 May 2008, geotechnical designers from around the country met at SAICE's offices in Midrand to discuss the way ahead for writing a South African geotechnical design code. The discussions were prompted by the provisions made in the revised SANS 10160: *Basis of structural design and actions for buildings and industrial structures* for geotechnical design and the compatibility of this code with the Eurocodes.

Three possible courses of action were debated at the meeting. These were:

Adopting EN1997-1 (Geotechnical Design – General Rules) as a South African design code. This would entail writing what amounts to a South African National Annex to the code.

Writing a South African design code based on SANS 10160 and EN1997. Such a code would contain only those aspects of the Eurocode relevant to South African conditions.

The *laissez-faire* approach. This was effectively the situation at the time where, in the absence of a geotechnical design code, designers used whatever design method was best suited to the problem at hand.

Option 3 was seen as the easy way out but one which did not hold any benefits for the profession.

The meeting acknowledged that drafting a South African design code would be beneficial. The new code would be a practical design code, relevant to South African conditions, written by engineers for engineers. The main drawbacks were seen to be the amount of professional time required to write such a code and that it would be difficult to write a code of this nature before the profession had more experience in the use of limit states design in geotechnical engineering.

It was agreed that geotechnical designers should use EN1997-1 in conjunction with SANS 10160 over the next five years. Thereafter, a more informed decision can be taken whether to adopt or adapt EN1997-1 or another international design code. This agreement was ratified by the Geotechnical Division Committee during their meeting later on the same day.

The five year period expires in May 2013. It remains to be seen what course the Geotechnical Division will take with regard to a geotechnical design code.

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10. PROVISION IN SANS 10160-5 FOR GEOTECHNICAL DESIGN

Much of the contents of this section is based on Chapter 5-1 of the Background Report (Day and Retief, 2008). Part 5 of SANS 10160 was drafted by the Candidate under the guidance of Professors Retief and Dunaiski. It was reviewed by members of the profession. Dr Dithinde made a significant contribution to the determination of model uncertainty factors for piles.

10.1 Need for Changes to SABS 0160

As described in Section 9.1, the old "loading code" SABS 0160:1989 did not make any provision for geotechnical loading on structures other than stipulating in Clause 5.8.3 that basement walls and similar members must be designed for the lateral forces exerted by adjacent ground and for any surcharges on this ground. No guidance was given on the load factors to be applied to such loading. Furthermore, the code specifically excluded loads on earth retaining structures and internal pressures in silos and tanks (Day, 2000).

For the first five years of the development of the new code SANS 10160, the intention was simply to make provision in the code for geotechnical actions likely to be exerted on buildings together with the appropriate partial action factors to be used. A major change came about in June 2005 when it was decided to make specific provision for geotechnical actions in the code in a manner similar to the recently published EN1990:2002. In the European code, provision is made in clause 6.4.1 for the "GEO" ultimate limit state to deal with design situations involving failure or excessive deformation of the ground where the strengths of soil or rock are significant in providing resistance. In the verification of this limit state, partial action factors on permanent actions are set to 1,0 but the strength of the ground is factored in accordance with the requirements of EN1997-1:2004. In this sense, geotechnical design is unique.

One of the principles applied during the development of SABS 0160 was that the partial load factors specified in the code should apply to all materials. The load factors and combination factors were chosen to yield a load index of approximately 2,0 at the ultimate limit state over a wide range of load combinations. These factors were chosen without consideration of the statistics of member (material) resistance. When combined with appropriate material factors in the materials codes, a reliability index of approximately 3,0 could be obtained. (Day and Kemp, 1999.) This allowed the development of the loading code to proceed independently from the materials codes.

The introduction of a GEO limit state to SANS 10160 that requires factoring of material properties in a different manner to that required for the other limit states represents a departure from this principle. In effect, it presumes the existence of a compatible materials code for geotechnical design. As South Africa does not yet have a geotechnical design code, it became necessary to include the *basis of geotechnical design* in SANS 10160 and not simply a section on geotechnical actions similar to the sections of the code dealing with other types of actions. From the outset, it was realised that some of this information would migrate to a future South African geotechnical design code and be removed from SANS 10160.

10.2 Scope of SANS 10160-5

SANS 10160-1 states that the standard covers building structures and industrial structures utilising structural systems similar to those of building structures. It specifically excludes

structures subject to internal pressure from the contents (e.g. silos and reservoirs), chimneys, towers and masts and bridges.

SANS10160-5 amplifies the scope of the code as providing design guidance on the determination of geotechnical actions on buildings and industrial structures including:
- vertical earth loading,
- earth pressures,
- ground water and free water pressure,
- downdrag or uplift caused by ground movements, and
- actions caused by ground movements.

Geotechnical structures such as slopes, embankments and retaining structures are not covered by the code. The term “not covered” was chosen in preference to the word “excluded”. In fact, an attempt was made to achieve as much compatibility between SANS 10160 and EN1997-1 as possible thereby opening the way to using SANS 10160 in conjunction with EN1997 or a future South African geotechnical design code based on EN1997 for all geotechnical structures.

It may seem strange to include earth pressures in a code that does not cover retaining structures. The rationale was to provide sufficient information for the design of building components such as basement walls which are subject to earth loading but not to cover free-standing retaining walls that rely mainly on the resistance of the ground for their stability.

10.3 Classification of Geotechnical Actions

Geotechnical actions are classified as permanent or variable, fixed or free. The classification of actions as permanent or variable is based not only on the length of time for which these effects act but also the uncertainty attached to their prediction. In certain cases, the designer may elect to classify certain long term actions such as uplift on piles due to heave as a variable action due to the uncertainty associated with the prediction of the uplift force.

Vertical earth loading and earth pressure are regarded as permanent fixed actions. Temporary stockpiles of earth are, however, regarded as variable or quasi-permanent fixed or free actions. Ground water pressure is not regarded as a separate action but as a component of vertical earth loading or lateral earth pressure. Free water above the ground surface including the additional water pressure within the ground caused by such water is classified as a variable action. Water pressure arising from temporary flooding is classified either as a variable action or an accidental action depending on circumstances.

Actions caused by ground movement include those that give rise to additional loading on the structure (e.g. downdrag on piles) or those that impose deformations on the structure (e.g. differential settlement). The code classifies these as permanent actions in view of the length of time over which they act. The exception is actions that result of seasonal variations in moisture content of the soils which are classified as variable actions.

In most instances, the above classification of actions is intuitive, apart possibly from the classification of earth pressure as a permanent action. This is contrary to ASCE 7-95 which groups “load due to the weight of soil and lateral pressure of soil and water in soil” with live loads which attract a partial load factor of 1,6 (basic combination 2, Clause 2.3.2) (Day, 2000). This factor is applied to earth pressures calculated using un-factored material properties. Australian Standard AS/NZS 1170.1 classifies earth pressure separately from permanent and imposed actions (clause 4.5). From clause 4.2.3(f) in AS/NZS 1170.0, earth pressures calculated using un-factored material properties are

factored by 1,5. The classification of earth pressures adopted in SANS 10160 is the same as that used in EN1990 / EN1997.

10.4 Geotechnical and Geometric Data

10.4.1 Geotechnical parameters

General

Ground properties are different to the properties of other structural materials in that, rather than having a single measurable value with an assessable statistical variation, the value of a ground property is often dependent on a variety of factors such as stress level, mode of deformation, rate of loading, stress history and moisture content. The measured value may be affected by the method of sampling and measurement. Spatially, variations in ground properties may be localised or general, random or systematic. Although SANS 10160-5 lists the factors that should be taken into account in assessing the properties of the ground and emphasises the need for an appropriate level of site investigation, it is expected that the designer has sufficient knowledge of geotechnical engineering to take these factors into account in selecting parameter values. Further elaboration would be more appropriate to a geotechnical design code than a basis of design code.

Characteristic Values

The characteristic value of a geotechnical parameter is given in Clause 6.3.1.6 as "*the value so determined that the probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%*". In most practical design situations, the designer has insufficient test results to justify a statistical analysis of the data. In such instances, Clause 6.3.1.2 provides a more intuitive definition, namely *a cautious estimate of the value affecting the occurrence of the limit state under consideration*.

The reference in both the above definitions to "the occurrence of the limit state under consideration" requires explanation as it implies that different characteristic values are to be selected for the same geotechnical conditions, depending on the limit state under consideration. For example, the bearing capacity of an end bearing pile is governed by the properties of the ground in close proximity to the base of the pile. Where the soil is variable, the possibility of a low value being encountered in the localised area around the pile base is significant and the chosen value should thus be a cautious estimate of the minimum strength of the material. On the other hand, the behaviour of a friction pile is governed by the average properties of the ground along the full length of the pile shaft. As a result, the effect of high and low values is averaged out along the pile shaft and a cautious estimate of the mean value would be appropriate. In statistical terms, the former may be taken as the 5% fractile of the assumed distribution of the strength of the soil whereas the latter may be determined using the Student's t-distribution.

It is also noteworthy that reference is made to a "worse" value rather than a "lower" value. This is because, in certain design situations, it is the upper characteristic value that governs the performance of the structure. An example of this is the case of downdrag forces on piles where the maximum average strength of the soil around the pile shaft determines the magnitude of the downdrag force for which the pile should be designed.

The selection of characteristic values, particularly for soil strength parameters, is one of the most critical aspects of any design and probably one of the most subjective.

Design Values

The design value of a geotechnical parameter is determined by dividing the characteristic value by a partial material factor. Values of these partial factors are given in Annex B of

the standard. In instances where the upper characteristic value applies, provision is made in Annex B of the standard (B.2.2) for the characteristic value so obtained to be divided by the reciprocal of the partial factor to obtain the design value where this produces a more onerous effect than dividing by the partial factor itself.

The standard also allows for the direct determination of design parameters where this is considered appropriate.

10.4.2 Geometrical Data

Ground level and slope, water levels, depth of interfaces between strata and excavation levels are all examples of geometrical data used in geotechnical design. The standard defines the characteristic value of geometrical data to be a measured, nominal or estimated upper or lower value. Design values may be derived from characteristic values by adjusting them up or down by a specified tolerance (e.g. ±75 mm on the plan position of a vertical pile) or may be determined directly.

In cases where the level of groundwater has a significant effect on the reliability of the structure, the design value of the ground water level is often determined directly taking account of variations in ground permeability and physical controls on the level of the water surface. In some instances, it is appropriate to assume that the ground water level could rise to the ground surface.

10.5 Verification of Ultimate Limit States

10.5.1 Design Approaches

Part 1 of the standard requires that, when considering the ultimate limit state, it shall be verified that:

$$E_d \leq R_d \qquad 10.1$$

where E_d is the design value of the effect of actions, and
R_d is the design value of the corresponding resistance.

In the case of most structural materials, it is a relatively straightforward matter to verify whether this condition is met. One simply “factors up” the loads and “factors down” the strength and then checks that the design resistance exceeds the design action effect. This is because the resistance and the actions are independent of one another. However, in the case of frictional materials (including soils) the strength of the material is stress dependent, i.e. the resistance is influenced by the loads. The converse also applies where the value of a geotechnical action (e.g. earth pressure) varies according to the strength of the material, i.e. the load is influenced by the resistance.

To complicate matters further, the self weight of the soil may be a destabilising action in one part of the soil mass and a stabilising action in another. For example, the weight of soil near the top of a slope has the effect of driving the failure whereas that near the toe resists failure. Frank et al (2004) describe the problem along the following lines: “*In geotechnical design, the self weight of the ground is usually the dominant action; however, it is very often difficult to determine which part of the ground contributes to favourable action and which to unfavourable action”*.

In the early versions of Eurocode 7 (ENV 1997) these difficulties were resolved by using two design calculations, each with a different set of partial factors:

Calculation 1: in which the partial factors were applied to permanent and variable actions while the ground strength was not factored.

Calculation 2: in which partial factors were applied to ground strength while permanent actions (including self weight) were not factored.

This led to considerable debate in Europe and pressure to reduce the perceived number of calculations from two sets to one and to simplify the calculations by applying partial factors to resistances and the effects of actions rather than to the material properties and the actions themselves. In the end, three separate design approaches for ultimate limit state design were included in EN 1997-1:2004 (Frank et al 2004):

Design Approach 1: which is similar to the approach described above and involves two separate calculations. This is an "action or material factor" approach.

Design Approach 2: which requires a single calculation with partial factors applied to actions (or the effect of actions) and to resistances. This is an "action (or action effect) and resistance factor" approach. This approach, with the application of partial factors to the effect of actions and to resistances, is similar to traditional factor of safety design.

Design Approach 3: also requires a single calculation in which partial factors are applied to actions or the effect of actions and to ground strength parameters. Design Approach 3 is an "action (or action effect) and material factor" approach.

SANS 10160, has opted for Design Approach 1 for three main reasons. Firstly, the partial factors are applied at the source of the uncertainty, i.e. to the individual actions rather than the effect of the actions and to material properties rather than to resistances. Secondly, the results obtained using this method agree reasonably with current South African practice based on an overall factor of safety approach. Finally, it is not unduly conservative as is often the case with Design Approach 3.

When compared with Design Approach 1, Design Approaches 2 and 3 tend to produce less conservative and more conservative results respectively. These observations by the Candidate are supported by the design examples given in the proceedings of the international workshop held in Dublin in 2005 (Orr 2005).

A more in-depth analysis of the reasons for the selection of Design Approach 1 is given by Dithinde (2007) based on three criteria, namely, the ability of the selected approach to

a) accommodate a wide spectrum of geotechnical design situations;
b) give results that are close to those yielded by current design methods; and
c) produce safe and economical designs.

Dithinde assesses each of the three Eurocode Design Approaches in accordance with these criteria and comes to the conclusion that Design Approach 1 is the preferred approach for South Africa for much the same reasons as given above.

10.5.2 Limit States and Partial Factors

<u>STR and STR-P Limit States</u>

In the case of design for structural resistance, the use of two separate expressions for the combination of actions was found to produce a more uniform level of reliability than a single expression (Kemp et al 1987; Kemp et al 1998). Such a scheme is used in SABS 0160, as given by equations 10.2 and 10.3. It is also similar to that used in ASCE-7. In the case of Eurocode, three alternative combinations schemes are applied, consisting of a

single expression, given as Expression 6.10 of EN 1990; a dual expression given as Expressions 6.10 (a) & (b); and a similar dual expression with a special interpretation of Expression 6.10 (a) which is similar to equation 10.3 below.

The combination scheme for the STR limit state has been carried forward unaltered from SABS 0160 into SANS 10160-1, as given by equation 10.2. For the case where permanent actions make a dominant contribution, the SABS 0160 equation 10.3 was modified as given by equation 10.4 and designated as STR-P. The modified expression not only improves the uniformity and consistency of the resulting reliability, but also results in closer agreement between the SANS 10160-1 combination scheme and the dual Expressions 6.10 (a) & (b) of EN 1990. The revised action combination scheme used in SANS 10160-1 is discussed extensively in Chapter 1-2 of the Background Report (Retief and Dunaiski, 2008).

$1{,}2\,G_k$ "+" $1{,}6\,Q_k$	for structures with significant imposed loads (STR)	10.2
$1{,}5\,G_k$	for self-weight dominated structures	10.3
$1{,}35\,G_k$ "+" $1{,}0\,Q_k$	STR-P	10.4

where "+" indicates "combined with".

One of the reasons for the modified STR-P combination was to provide specifically for geotechnical design situations where permanent actions frequently dominate.

In both the STR and STR-P limit states, partial factors are applied to the actions and the ground parameters are not factored. For example, the earth pressure acting on a basement wall is calculated using the characteristic (i.e. un-factored) values of soil parameters and the resulting pressure is then treated as a permanent action in the analysis of the structure and attracts the appropriate partial action factor.

GEO Limit State

The second of the two calculations is based on Design Approach 1, Combination 2 as described in 2.4.7.3.4.2 of EN 1997-1. It required the introduction of a new limit state in SANS 10160 known as the GEO limit state. This limit state generally governs the design where failure occurs in the ground, e.g. bearing capacity or slope stability.

In this limit state, partial factors are applied to the soil strength parameters but permanent actions are not factored. Variable actions are factored with a reduced partial action factor value.

The exception to the above rule is in the design of piles and anchors where a resistance factor is used in preference to factoring the material properties in situations where the resistance of the pile is determined using empirical correlations. In the Eurocodes, different resistance factors are applied to different types of piles and to the contributions of shaft friction and end bearing. Analyses of model uncertainty carried out on South African data (Dithinde 2007) do not support the adoption of different partial resistance factors for the various pile types. In addition, the data is insufficient to permit differentiation between the contributions of shaft and base resistance in assigning resistance factors.

Partial Action, Material and Resistance Factors for STR, STR-P and GEO Limit States

The partial action factors used in SANS 10160 are specified in SANS 10160-1 in the normative section of the standard. The partial material and resistance factors for geotechnical design should ideally be specified in a geotechnical design code. In the absence of such a code, a set of partial factors compatible with the action factors in SANS 10160-1 has been given in Annex B of SANS 10160-5. As these factors may be amended

when a South African geotechnical design code Africa is compiled, this Annex is regarded as informative rather than normative.

Table 13 summarises the partial action and material factors for persistent and transient design situations in the ultimate limit state that are applicable to the STR, STR-P and GEO limit states. For completeness, the partial factors for the equilibrium (EQU) and accidental (ACC) limit states are also given. In each case, the values given in SANS 10160 are compared with the corresponding values from EN 1990 and EN 1997-1. The symbol ψ is used for the action combination factor.

Table 13: Summary of partial factors in the ultimate limit state

	Limit State *EN 1990 reference:*	**Partial factor γ**				
		EQU	**STR** *Eq 6.10b*	**STR-P** *Eq 6.10a*	**GEO**	**ACC** *Eq 6.11b*
Partial action factors	**Permanent Actions**		*Set A1*[8]	*Set A1*[8]	*Set A2*[8]	
	Unfavourable					
	SANS 10160	1,2	1,2	1,35	1,0	1,0
	EN 1990/1997-1	*1,1/1,0*[3]	*1,15*	*1,35*	*1,0*	*1,0*
	Favourable					
	SANS 10160	0,90	0,90	-	1,0	1,0
	EN 1990/1997-1	*0,90*	*1,0*	-	*1,0*	*1,0*
	Variable Actions					
	Leading action – Unfavourable					
	SANS 10160	1,3/1,6[1]	1,3/1,6[1]	1,0	1,3	1,0 [4]
	EN 1990/1997-1	*1,5*	*1,5*	*1,5 $\psi_{0,1}$*	*1,3*	*$\psi_{1,1}$ or $\psi_{2,1}$*
	Accompanying action – Unfavourable					
	SANS 10160	1,3 ψ_i	0/1,6 ψ_i [1]	0[2]	0/1,3 ψ_i [1]	0/1,0 ψ_i [1]
	EN 1990/1997-1	*1,5 $\psi_{0,i}$*	*1,5 $\psi_{0,i}$*	*1,5 $\psi_{0,i}$*	*1,3*	*$\psi_{2,i}$*
	All variable actions – Favourable					
	SANS 10160	0	0	0	0	0
	EN 1990/1997-1	*0*	*0*	*0*	*0*	*0*
Partial material & resistance factors	**Soil Parameters** SANS 10160					
	and *EN 1990/1997-1*	*Table A2*	*Set M1*[8]	*Set M1*[8]	*Set M2*[8]	
	Angle of shearing resistance [5] ϕ'	1,25	1,0	1,0	1,25	1,0[7]
	Effective cohesion c'	1,25	1,0	1,0	1,25	1,0[7]
	Undrained shear strength c_u	1,4	1,0	1,0	1,4	1,0[7]
	Unconfined strength q_u	1,4	1,0	1,0	1,4	1,0[7]
	Weight density γ	1,0	1,0	1,0	1,0	1,0[7]
	Resistances		Set R1[8]	Set R1[8]	Set R4[8]	
	Pile – compression SANS 10160	-	1,0	1,0	1,6[6]	1,0[7]
	EN 1990/1997-1	-	*1,0-1,15*	*1,0-1,15*	*1,3-1,5*[6]	*1,0*
	Pile – tension SANS 10160	1,4	1,25	1,25	1,7[6]	1,0[7]
	EN 1990/1997-1	*1,4*	*1,25*	*1,25*	*1,6*[6]	*1,0*
	Anchors SANS10160	1,4	1,1	1,1	1,1[6]	1,1[7]
	EN 1997-1	*1,4*	*1,1*	*1,1*	*1,1*[6]	*1,0*

Notes:
(1) Values apply to wind actions and to variable actions other than wind respectively.
(2) For the STR-P combination, only permanent actions and the leading variable action are combined. Accompanying variable actions not considered.
(3) Values apply to EQU and to UPL limit states respectively.
(4) Design value of accidental action regarded as a leading variable action.
(5) Factor applies to tan ϕ'.
(6) Resistance factor applied to pile and anchor capacities calculated using un-factored ground parameters. In the case of unfavourable actions on piles (e.g. due to downdrag or transverse loading on the piles) the resistance factor is applied to the actions calculated using factored ground parameters.

(7) Generally un-factored soil properties, which may be modified depending on the accidental situation. See B.2.5 of SANS 10160-5.
(8) Set A1, Set M1, etc. refer to the sets of partial factors given in Annex A of EN 1997-1.

Partial Factor for Uncertainty in the Resistance Model

Clauses 2.4.1(6) - (9), EN 1997-1 introduces a model factor to account for:
- the range of uncertainty in the results of the method of analysis
- any systematic errors known to be associated with the method.

The use of a model factor is specifically mentioned in Clause 7.6.2.3 (2) of EN 1997-1 which deals with the determination of the compressive resistance of piles from ground tests. EN 1997-1 is, however, silent on the value to be used for this important parameter, preferring instead to leave the assignment of suitable values to the National Annex. This is probably due to the wide range of design methods used by European member states and the differences in the results obtained (see De Cock *et al* 1999).

The UK National Annex (NA to BS EN 1997-1:2004, Clause A.3.3.2) recommends a value 1,4 which may be reduced to 1,2 if the resistance is verified by a maintained load test taken to the calculated, un-factored ultimate resistance of the pile. These values must, however, be read in conjunction with the values recommended in the UK National Annex for the partial resistance factors to be used in pile design which vary from 1,7 to 2,0 for piles in compression compared to a value 1,6 recommended in SANS 10160-5.

Dithinde (2007) gives the results of an analysis of 174 documented South African pile load tests for which sufficient data was available to calculate the resistance of the pile based on ground test results. Statistical analysis of this data has indicated that a model factor of 1,5 used in conjunction with the partial resistance factor of 1,6 for piles in compression will be sufficient to achieve the target reliability index of β = 3,0 as aimed for in SANS 10160. Accordingly, a value of 1,5 has been recommended in B.3.3 of SANS 10160-5 for the design of piled foundations by calculation using ground test results.

10.6 Verification of Serviceability Limit States

The serviceability limit state is verified in accordance with the criteria set out in Clause 8 of SANS 10160-1. Partial action factors and action combination factors are given for three serviceability limit states, namely irreversible, reversible and long term.

In selecting which of the combinations of actions to use in the analysis of the various serviceability limit states, SANS 10160-5 recommends that account should be taken not only of the reversibility of the effect but also the period of time required for the effect to take place (Clauses 7.2.2 and 7.2.3). The irreversible serviceability combination is one which will occur infrequently and probably for limited periods of time but its occurrence cannot be excluded. The reversible and long term combinations are more representative of the average conditions likely to occur over a period of time. As such, SANS 10160-5 requires that the irreversible action combination be used in the assessment serviceability conditions that develop rapidly, such as settlement of granular soils. The long term action combination is to be used in the assessment of conditions that develop over a longer period of time such as consolidation settlement or creep.

An alternative approach to the verification of the serviceability limit state included in Clause 7.2.6 of SANS 10160-5 is to ensure that a sufficiently low fraction of the strength of the ground is mobilised to keep deformations within the required limits. This approach, which has been adopted from EN 1997-1, applies only when a quantification of the deformation is not required and where comparable experience exists under similar circumstances.

Movement caused by heave or collapse of soils is not dealt with specifically. However, the standard does require that the effect of changes in ground properties that may occur during the life of the structure should be taken into account, including desiccation of the soil, saturation, ground water lowering, etc.

10.7 Determination of Geotechnical Actions

10.7.1 Vertical Earth Loading

The requirements of the standard for the determination of vertical earth loading amount to little more than common sense and sound engineering practice. Attention is drawn to the following:

- The bulk unit weight of the ground is used in the calculation thereby including the self weight of the groundwater with the self weight of the ground and not regarding the self weight of the water as a separate action. Allowance should be made in the assessment of the bulk unit weight for variations in moisture content.
- In the case of non-uniform loading, e.g. conical stockpiles, allowance may be made for redistribution of the loads due to arching.
- On non-uniform subgrades, cognisance should be taken of the tendency for stiffer areas to “attract” load, e.g. as is the case with positive projection culverts below fill embankments.
- Any surcharge on the ground surface or free water above the surface should be regarded as a variable action and considered separately from the vertical earth loading.

10.7.2 Earth Pressures

The standard draws attention to the effect of the magnitude, mode (rotation, translation, etc.) and direction of wall movement (into or away from the retained material) on the magnitude and distribution of the earth pressure. It also lists factors to be taken into account in the determination of earth pressure including wall roughness, inclination of the wall and the ground surface, compaction forces, swelling pressures, etc. The normative sections of the standard do not provide methods for the calculation of earth pressure as this information is better suited to a geotechnical design code than a basis of design code. Guidance is, however, given in informative Annex C for the determination of earth pressures exerted by granular backfills on vertical basement walls.

One somewhat contentious requirement included in SANS 10160 from EN 1997 is that, unless a reliable drainage system is provided or infiltration into the soil is effectively prevented, the water table in retained earth of low or medium permeability should be assumed to be at the surface unless indicated otherwise by comparable local experience. This is consistent with Clause 9.6(3) of EN1997-1:2004 and Clause 4.3 of AS 1170.1.

10.7.3 Actions due to ground movement

Actions due to ground movement are considered in two categories, namely those that exert a force on the structure and those that cause the structure to deform.

The first category includes uplift and downdrag forces on piles caused by heave or settlement of the surrounding ground respectively. In such cases, the upper characteristic value of the shear strength (above the mean value) should be used in the design. In addition, the standard draws attention to the fact that the shear stresses mobilised in the ground strata overlying the expansive or compressible horizon must also be considered in the calculation of uplift or downdrag forces.

The second category includes the effects of ground movements such as differential heave or settlement on structures and services. It is pointed out that predictions of differential movement which ignore the stiffness of the structure tend to overestimate the distortion of the structure. The standard cautions that the accuracy of settlement predictions tends to be poor.

10.8 Geotechnical Categories

Annex A of SANS 10160-5 introduces the concept of Geotechnical Categories based on the nature of the ground, the complexity of the structure, the intensity of loading and the associated risks. It goes on to stipulate the minimum requirements for investigation, design, construction control and monitoring for each category. The content of this annex is based on a similar section in EN 1997, modified for South African conditions, legislation and practices.

This information is included as an informative annex rather than being placed in the main body of the report as many sections of the Annex pertain more to geotechnical design than to basis of design. In all probability it will be moved to the geotechnical design code once such a code is written. Its inclusion in the standard was considered warranted by the committee in view of the widespread tendency in South Africa to limit the scope of geotechnical investigations and monitoring during construction to levels that are not compatible with the scope of the project and the risks involved. This Annex supplements the requirements of the Construction Regulations of the Occupational Health and Safety Act and provides a standard of good practice against which the requirements of the Act can be adjudicated.

Four Geotechnical Categories are applied, similar to the four level classification applied in SANS 10160-1 for buildings and the consequences of accidental actions. This four level classification scheme is somewhat different from the three level classification generally applied in the Eurocodes which often requires a sub-division of the second class. The following four Geotechnical Categories are presented in Annex A of SANS 10160-5:

Category 1 includes small and relatively simple structures constructed on non-problematic ground where there is negligible risk of instability or of significant ground movements (e.g. houses or other simple structures on stable soil profiles). In such cases, the investigation may be limited to a qualitative assessment based on a systematic description of the soil profile. Deemed-to-satisfy design procedures will suffice and only routine inspections at critical stages of construction are required.

Category 2 includes conventional structures and foundations for which design methods are well established, where there are no exceptional risks in terms of overall stability or difficult ground conditions (e.g. conventional buildings on spread footings, rafts or piled foundations). Here, the geotechnical investigation should include routine field and laboratory tests producing quantitative geotechnical data for design purposes. In these cases, design calculations are required including the assessment of bearing capacity, settlement, earth pressure, etc. Systematic checking by the designer is required during construction to confirm the validity of the design assumptions coupled with periodic inspections by the geotechnical engineer. Additional field and laboratory testing may be required during construction. Monitoring will generally be limited to ensuring that critical performance criteria are met (e.g. total settlement of foundations or movement of retaining structures).

Category 3 structures include conventional structures and foundations with no exceptional risks or loading conditions, but for which the nature of the ground or complexity of the design requires specialist geotechnical input (e.g. anchored retaining systems, deep excavations below the water table, problems requiring soil-structure

interaction analysis, etc). The geotechnical investigation requirements are similar to those for Category 2 supplemented by specialised field and laboratory tests as specified by the geotechnical engineer. Specialised geotechnical design and cooperation between the geotechnical and structural engineers are required as are regular and detailed monitoring by the geotechnical engineer with additional field and laboratory tests as appropriate. A rigorous construction quality control programme is essential. Detailed monitoring should include, as appropriate, piezometer levels, ground movements, anchor loads often coupled with the use of the observational method. On-going monitoring of the structure may be required after completion.

Category 4 includes structures or parts of structures that lie outside Categories 1 to 3, e.g. very large or complex structures, structures involving abnormal risks or in unusual, unstable or exceptionally difficult ground conditions. Such projects require the application of the requirements of Categories 1 – 3 supplemented by requirements in addition or alternative to those in the standard.

Annex A includes a brief description of the observational method as a means of adapting the design to suit conditions encountered on site. Its inclusion is intended to establish the "legitimacy" of a procedure that is regarded by some as an excuse by geotechnical engineers to change their minds during construction.

10.9 Guidance for Structural Designers

Annex C of SANS 10160-5 provides guidance for structural engineers on typical geotechnical aspects of the design of buildings and industrial structures. This includes charts for assessing the bearing capacity of soils, values of earth pressures (including compaction pressures) exerted by granular soils on vertical walls and guidance on the design of piles. This informative Annex is provided in the absence of a South African geotechnical design code.

10.9.1 Design of Spread Footings

Annex C.2.2 gives charts for determining the design (bearing) resistance of shallow foundations founded on two classical soil types, namely an undrained normally consolidated clay (ϕ = 0) and a non-cohesive granular soil (c' = 0).

In the case of the granular soils, separate charts are given for various positions of the water table, namely at the surface, at founding level and below the depth of influence of the foundation. The charts are in non-dimensional form to allow for the evaluation of the design resistance of square and strip footings of any size of footing (width B) and at depths of founding (Z) ranging from surface (Z/B = 0) to Z/B = 1,0. The assessment of the bearing pressure is based on the method given in EN 1997-1, Annex D.4.

For undrained conditions, only one chart is required as the value of the bearing capacity factor N_c is independent of foundation size and depth of founding. The depth of the water table plays no part in the calculation provided it is below founding level. This chart is based on the classical Skempton (1951) bearing capacity equation. It differs from the approach adopted by EN1997-1 which does not provide for variation in the value of the bearing capacity factor N_c with depth.

10.9.2 Design of Axially Loaded Piles and Pile Groups

Annex C.3 summarises the various approaches given in EN 1997-1 to the design of piles namely load testing (static or dynamic), analysis of pile driving records and calculations

based on ground test results. Details of the first two methods are, however, too lengthy to be dealt with in a basis of design code and correctly belong in a geotechnical design code. Although some pointers are given on the latter method, reference to EN 1997-1 will still be required to apply the method.

10.9.3 Earth Pressures

Unlike the design of piles or spread footings which are often left to the geotechnical engineer, structural engineers are frequently required to estimate the earth pressure exerted on a buried structure as an input to the structural design. In order to facilitate such calculations, an approach similar to that contained in TMH 7:1981 is adopted where two types of backfill are defined and typical parameters are assigned to these materials. These parameters are then used to determine the earth pressure acting on yielding and rigid structures (active and at-rest conditions respectively).

The approximate earth pressure distribution given ignores cohesion in the backfill and wall friction and its application should be limited to walls lower than 7,5m. The distribution given in Annex C.4 includes the effect of compaction and a water table within the retained material. Compaction pressures are calculated using the method recommended by Clayton *et al* (1993).

10.10 References

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11. ADDITIONAL BACKGROUND INFORMATION ON SANS 10160-5

Section 10 describes the provision made in SANS 10160 for geotechnical design. The section below provides some additional background information on the code.

11.1 Compatibility with the Eurocodes

One of the key objectives in re-writing the SABS 0160 was to achieve compatibility with international standards. The extent to which the provisions of SANS 10160-5 for geotechnical design are compatible with those given in EN 1990 and EN 1997-1 is explored below.

Table 13 in Section 10 summarises the action combinations, partial material factors and resistance factors for persistent and transient design situations in the ultimate limit state as included in SANS 10160 and compares them with those given in the Annexes to Eurocodes EN 1990 and EN 1997. Table 14 below gives typical load combinations for residential buildings including the appropriate numerical values of the action combination factors ψ.

Table 14: Typical load combinations of G_k, Q_k and Q_W (illustrative only)

Limit State		Action Combination	Comments
EQU	SANS 10160	$0{,}9G_k$ "+" $1{,}3Q_W$	G_k and Q_k both assumed to be favourable actions
	EN 1990/1997-1	*$0{,}9G_k$ "+" $1{,}5Q_W$*	
STR	SANS 10160	$1{,}2G_k$ "+" $1{,}6Q_k$ and $1{,}2G_k$ "+" $0{,}48Q_k$ "+" $1{,}3Q_W$	Q_k leading (ψ_{wind} = 0,0) Q_W leading
	EN 1990/1997-1	*$1{,}15G_k$ "+" $1{,}5Q_k$ "+" $0{,}90Q_W$ and* *$1{,}15G_k$ "+" $1{,}05Q_k$ "+" $1{,}5Q_W$*	*Equation 6.10b – Q_k leading* *Equation 6.10b – Q_W leading*
STR-P	SANS 10160	$1{,}35G_k$ "+" $1{,}0Q_k$ and $1{,}35G_k$ "+" $1{,}0Q_W$	Q_k leading Q_W leading (no accompanying variable action)
	EN 1990/1997-1	*$1{,}35G_k$ "+" $1{,}05Q_k$ "+" $0{,}90Q_W$*	*Equation 6.10a*
GEO	SANS 10160	$1{,}0G_k$ "+" $1{,}3Q_k$ and $1{,}0G_k$ "+" $0{,}39Q_k$ "+" $1{,}3Q_W$	Q_k leading (ψ_{wind} = 0,0) Q_W leading
	EN 1990/1997-1	*$1{,}0G_k$ "+" $1{,}3Q_k$ "+" $0{,}78Q_W$ and* *$1{,}0G_k$ "+" $0{,}91Q_k$ "+" $1{,}3Q_W$*	*Equation 6.10 – Q_k leading* *Equation 6.10 – Q_W leading*
ACC	SANS 10160	$1{,}0G_k$ "+" A_d "+" $0{,}3Q_k$	Q_k or Q_W both accompanying variable actions (ψ_{wind} = 0,0)
	EN 1990/1997-1	*$1{,}0G_k$ "+" A_d "+" $0{,}5/0{,}3Q_k$ and* *$1{,}0G_k$ "+" A_d "+" $0{,}3Q_k$ "+" $0{,}2/0Q_W$*	*Equation 6.11b – Q_k leading* *Equation 6.11b – Q_W leading*

Where G_k = characteristic value of a permanent action
Q_k = characteristic value of a variable action and
Q_W = characteristic value of wind action

As far as the new GEO limit state is concerned (generally associated with failure in the ground), the differences between SANS 10160 and EN 1990 / EN 1997-1 pertain mainly to the effect of wind. For most geotechnical structures apart for foundations of tall structures, wind loading plays a relatively minor role. If wind loading is omitted, the combinations of the permanent actions and variable actions are identical. All the partial material factors used in SANS 10160 have been taken directly from the values recommended in Annex A of EN 1997-1 and are therefore compatible. The resistance

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factors used are also similar. Thus, as far as the GEO limit state is concerned, SANS 10160 is compatible with the Eurocodes with respect to the application of Design Approach 1.

With regard to the STR and STR-P limit states, the differences in the partial action factors are either minor or arise from the treatment of the accompanying variable action. The factoring of accompanying variable actions is in accordance with SABS 0160:1989 which includes accompanying variable actions at their "arbitrary point in time" values in accordance with the principles of the Turkstra rule (Kemp 1987). This was a well-researched decision based on reliability analyses and is retained in the SANS 10160, as is discussed in Chapter 1-1 and 1-2 of the Background Report to SANS 10160 (Retief & Dunaiski, 2008). The partial action factors and the combination factors used in SANS 10160 are thus rational values regarded by the committee as preferred alternatives to the values given in the Eurocodes yet fully compatible with the Eurocode approach as embodied in Equations 6.10(a) and 6.10(b) of EN1990.

It should be noted that the partial factors given in EN 1990 and EN 1997-1 are all classified as National Determined Parameters (NDPs) and CEN Member States are required to select appropriate values by means of a National Annex. In addition, member states are free to select between any one of the Design Approaches 1, 2 or 3. It is interesting to note that alternative design approaches are selected for different geotechnical design situations by various Member States, reflecting an even wider interpretation of the NDP options than envisaged by CEN during the formulation of the Eurocodes. These selections can lead to wide variation in the results obtained. Against this background, the differences reflected in Table 14 are well within the range of values likely to be chosen by European member states.

11.2 Application of SANS 10160-5

11.2.1 Status

SANS 10160-5, and SANS 10160 in general, is a standard written by engineers for engineers. It is intended to assist designers with the application of reliability based limit states design methods that are in line with international practice. The methods given in the standard represent a logical and harmonised approach to limit states design but are not the only methods that may be used nor are they necessarily applicable to every design situation. The standard is an aid to the application of engineering judgement and not a substitute for it.

Along with the majority of South African National Standards, SANS 10160 is a statement of good practice. Despite the prescriptive language used in the normative sections, the standard itself is not mandatory and has no legal status. Specifically, SANS 10160 is not a document written by engineers to be used against them in a court of law. Compliance with the standard will, in general, be sufficient to demonstrate the acceptability of the design approach used. However, the converse does not necessarily apply.

As indicated by Retief et al (2008), SANS 10160 has used the Eurocodes as reference documents. There are, however, significant differences in the application of the Eurocodes among the CEN Member States and the application of SANS 10160 in South Africa.

The Eurocodes are sponsored by the European Commission with a view to eliminating technical barriers to trade and harmonisation of technical standards between the member states. As safety is regarded as a national issue under the control of each individual member state, the selection of parameters which relate to safety (or more correctly, to

reliability) of structures are not prescribed in the normative sections of the Eurocodes but fall to be defined in National Annexes prepared by the member states. The parameter values contained in the informative annexes to the various Eurocodes are for guidance only. By treaty, all European member states are obliged to afford the Eurocodes the status of national standards, compile National Annexes to define the selection of NDP options and to withdraw all conflicting codes and standards within a prescribed timeframe.

Although South Africa has elected to model its *basis of design* code on the Eurocodes, it is not obliged to adhere to any of the requirements imposed on European member states with regard to the implementation of the code. It is free to change, omit or add whatever it deems fit in the compilation of its own national standards. There are, however, many benefits for the country in remaining reasonably aligned with the basis of design embodied in the Eurocodes. In particular, this will facilitate technical exchange and trade with one of our biggest trading partners. This will also permit the use of Eurocodes where no equivalent South African Standard exists.

11.2.2 Application

SANS 10160-5 provides design guidance sufficient for projects that lie in Geotechnical Categories 1 and 2. Simplified design rules, including deemed-to-satisfy requirements may be used for Category 1 structures as stated in Annex A. Category 3 and 4 projects will generally require alternative or additional design rules to those contained in SANS 10160-5.

As indicated above, the use of the standard is not mandatory for any Geotechnical Category. Designers may choose to use other established design methods as they see fit. However, should they do so, they may be called on to defend the method adopted. They would no longer enjoy the protection that compliance with the standard affords.

11.2.3 Use of SANS 10160 in conjunction with Eurocodes

SANS 10160-1 and SANS 10160-5 are sufficiently compatible with EN1990 and EN 1997-1 to permit the use of EN 1997-1 as a geotechnical design code in conjunction with SANS 10160 in South Africa. The use of EN 1997-1 is seen as an interim step to either writing a South African geotechnical design code or formally adopting EN 1997-1 as a South African standard.

11.3 Selection of Characteristic Values

The information in Section 10.4.1 above on characteristic values was an explanation of what is included in the code. It is, however, far from the last word that can be said on this subject.

The selection of the characteristic value for a geotechnical parameter depends on the context in which that parameter is to be used. In Section 10.4.1, it was pointed out that the most appropriate value may be a conservative estimate of the mean value or of the minimum / maximum value. The appropriate value could be a lower or an upper characteristic value. However, it does not end there. The question needs to be asked: *for which parameter is the characteristic value required*? In the case of strength, for example, is it the peak, critical state, residual, etc. strength that is required? For stiffness, is it the small strain stiffness, the stiffness on first loading, the re-loading stiffness, etc.? As the Candidate put it in his presentation to the European Danube Conference in Bratislava (Day, 2010) "*geotechnical design is 80% thinking and 10% calculation, ... and then you*

think some more". Those who hold the view that geotechnical design codes like EN 1997-1 remove the judgement from geotechnical design should think again.

Dithinde (2007) makes the point that the selection of the characteristic value needs to take account of all the uncertainties in the parameter including special variability, measurement bias and statistical uncertainties among other factors. The application of a partial factor to this value ensures that the required safety level is achieved. He recounts the argument by Cardoso and Fernandes that it is not logical to apply partial factors to a poorly defined characteristic value.

Dithinde also quotes Krebs Ovesen who carried out an experiment where 25 Eurocode 7 committee members were asked to determine the characteristic value for the ultimate limit state for the following ten test results; 138, 140, 170, 171, 179, 182, 232, 258 and 272kPa. The characteristic values given ranged from 145 to 200kPa. What is not stated in this case is the use to which the characteristic value is to be put. As indicated above, this could have a considerable bearing on the value chosen.

During their visit to South Africa in 1995, Drs Simpson and Krebs Ovesen conducted a similar experiment, which Simpson now refers to as the "Johannesburg Experiment" (Simpson and Driscoll, 1998). The delegates to the Limit States Design Symposium were requested to estimate the characteristic value they would adopt for the friction angle of a large body of material with no systematic variations where the occurrence of the limit state would be governed by the mean value. The data set provided is shown in the top left hand diagram in Figure 39.

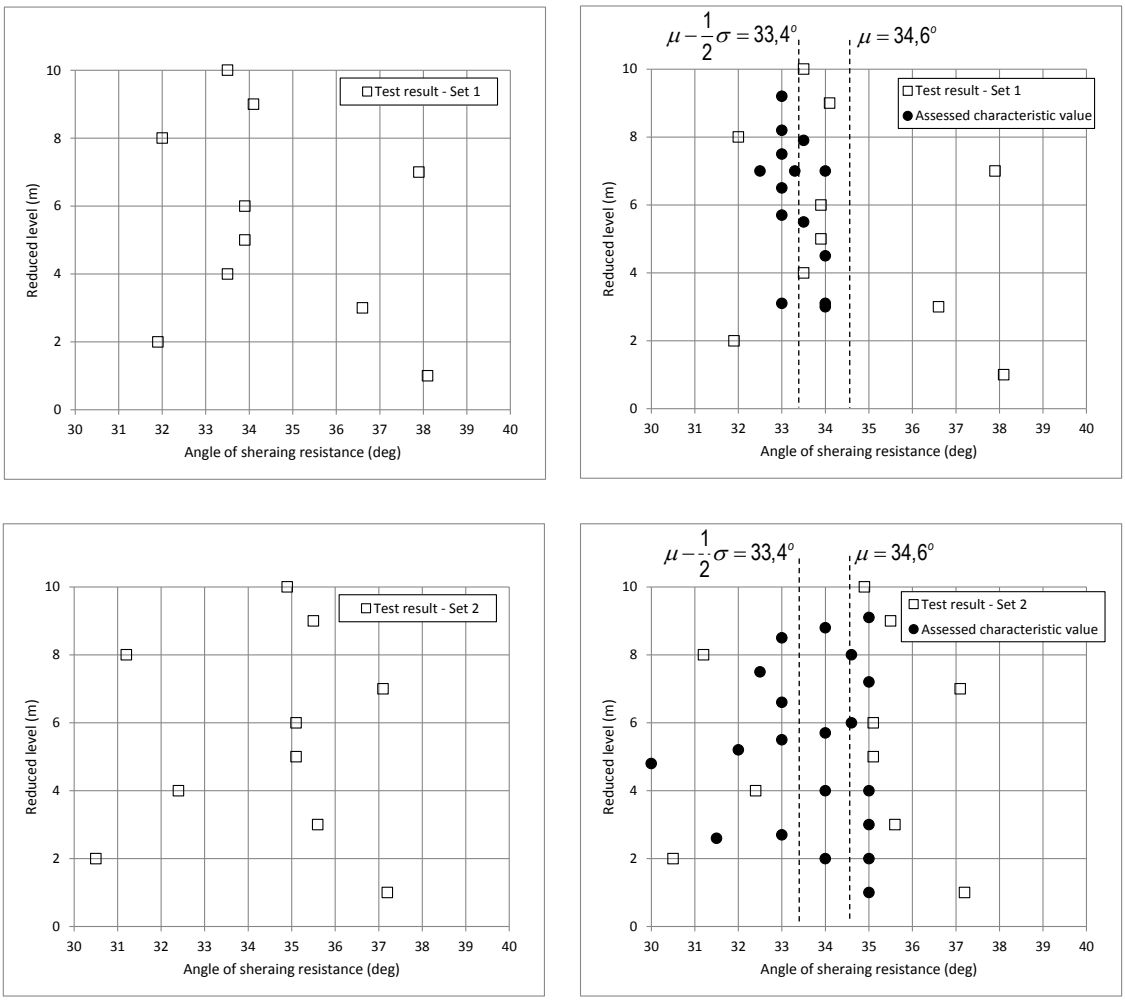


Figure 39: The Johannesburg experiment (after Simpson and Driscoll, 1998)

The assessments of the characteristic values provided by the delegates to the Johannesburg symposium are shown in the top right hand diagram together with the mean value and one half standard deviation below the mean. The assessments cluster around the half standard deviation below the mean. Comments from the audience were that the assessment they had made was essentially the same as they would previously have made in geotechnical design practice (Simpson and Driscoll, 1998).

The same audience was then shown the bottom left diagram in Figure 39 and asked the same question. The results are shown on the bottom right diagram. Clearly, the audience were more uncertain in this case despite the (unrevealed) fact that the data has the same mean and standard deviation. Simpson considers this experiment to support the idea that the mean less half a standard deviation is a useful guide but that reliance on statistics alone would be dangerous.

In 2001, the Candidate, who was chairman of TC23[17] at the time, together with Dr Trevor Orr of Ireland and Dr Kenji Matsui of Japan distributed a questionnaire to TC23 members around the world to assess the investigation techniques employed and the methods used to determine geotechnical parameters. The questions related to who is responsible for planning of geotechnical investigations, common problems with investigations, tests used for various parameters, who determines the parameters to be used in design and how design parameters are selected. One of the questions was "*do you think that the statistical approach is a useful tool?*". 64% of the respondents replied that a statistical approach is effective while 33% rejected it. Interestingly, all the respondents from Russia supported the statistical analysis of data. Most of the respondents who supported the use of statistics indicated that these should be used in combination with experience and judgement. Many respondents commented that few geotechnical engineers are familiar with the use of statistics.

A further point about the use of statistics is: *what data do you use in the statistical analysis*? Consider a 5m thick layer within a soil profile where the measured drained elastic modulus of the material is 15, 18, 32, 12 and 8MPa. If the average value of 17MPa is used in the calculation of settlement, the settlement would be calculated as 29mm/100kPa applied pressure. However, if the layer was to be split into five sub-layers with the measured modulus attributed to each sub-layer, the calculated settlement would be 36mm/100kPa. This is because it is the inverse of the elastic modulus is used to calculate settlement. It would therefore be preferable to apply the statistics to the inverse rather than to the actual value. The same applies to the friction angle where the strength of the soil determined by the tangent of the friction angle rather than the friction angle itself.

Section B4 in Simpson and Driscoll (1998) provides some of the most informed guidance on the selection of characteristic values and anyone seeking further information on this matter is encouraged to refer to this publication.

11.4 Design of Spread Footings (Annex C.2)

As indicated in Section 10.9 above, South Africa does not as yet have a limit states geotechnical design code. It is not expected that every structural engineer wishing to carry out a routine calculation of (say) the bearing capacity below a spread footing or the earth pressure on a basement wall should go out and buy a copy of Eurocode 7. For this reason, an informative annex was added to the code to provide guidance on these two issues in particular. This Section 11.4 explores the background behind the Annex. It is based on the calculations carried out by the Candidate during drafting of Part 5 of the

[17] ISSMGE Technical Committee TC23 on Limit States Design in Geotechnical Engineering.

code and subsequent comparisons of the results from limit states and working stresss designs.

11.4.1 *Bearing Capacity of Undrained Clay (Figure C.1)*

Figure C.1 in Annex C of SANS 10160-5 is nothing more than a plot of the Skempton (1951) bearing capacity factor N_c. As such, it does not differ from current practice for determining bearing capacity of spread footings on undrained clay profiles.

The approach differs slightly from that given in Annex D of EN 1997-1. In Section D.3 of that code, the value of N_c is taken as $\pi + 2$ (or 5,14) for a strip footing irrespective of the depth of founding. The shape factor for a square footing is given as 1,2 giving resulting in an N_c value of 6,17. These two values (5,14 and 6,17) are identical to the values used in Figure C.1 for Z/B = 0.

Given that the equations for undrained bearing capacities of clays used in SANS 10160-5 are identical to those traditionally used for working load designs, the only difference is in the design values inserted into the equations for the working load or limit states design methods.

Consider a square footing founded on the surface (i.e. $q_o = \gamma.Z = 0$) of a layer of firm clay with a characteristic undrained shear strength $c_{u,k}$ = 100kPa. For the GEO limit state, the partial material factor applied to undrained shear strength is γ_{cu} = 1,4 giving a design value of $c_{u,d}$ = 71kPa. From Figure C.1 in SANS 10160-5, N_c = 6,2 and the design resistance R_d of the footing is 440kPa x the area of the footing, irrespective of the proportions of the permanent and variable components of the applied load. If the applied load comprises entirely permanent actions (partial action factor γ_G = 1,0), the characteristic load is equal to the design load. If this same footing were to be analysed using working stress design methods, the factor of safety would be 1,4 for a footing subjected to a load equivalent to the characteristic load referred to above.

As the portion of live load increases, the sum of the characteristic values of the permanent and applied actions reduces for a constant value of R_d due to the partial action factor γ_Q = 1,3. If all the load on the footing was due to imposed actions (i.e. $G_k / (G_k + Q_k)$ = 0,0), the factor of safety computed using working load design methods for a footing loaded with the sum of the characteristic values of the actions, the factor of safety would be 1,82 (i.e. 1,4 x 1,3). For intermediate values of $G_k / (G_k + Q_k)$, the factor of safety varies linearly between 1,4 and 1,82.

These values of the factor of safety are well below those used with working stress design methods where the factor of safety used for a cohesive material would typically be around the 3,0 mark. In the Candidate's opinion, the value of γ_{cu} of 1,4 taken from Annex A of EN1997-1 is too low.

11.4.2 *Bearing Capacity on Granular Soils (Figure C.2)*

The bearing capacity of granular soils given in Figure C.2 in Annex C of SANS 10160-5 is based on the method given in Clause D.4 of EN 1997-1.

<u>Derivation of Figure C.2</u>

To illustrate how this figure was derived, consider the footing shown in Figure 40. It is a square footing with the water table at founding level, i.e. the dotted lines in Figure C.2(b) of SANS 10160-1 apply. Figure 41 gives a specimen ultimate limit state calculation for this base using the GEO limit state (factored soil strength; ϕ'_d =30°). The result R_d =

4 548kN, which is equivalent to $R_d / (L\ B^2)$ = 568kN, is superimposed on Figure C.2 b) with a square symbol in Figure 42.

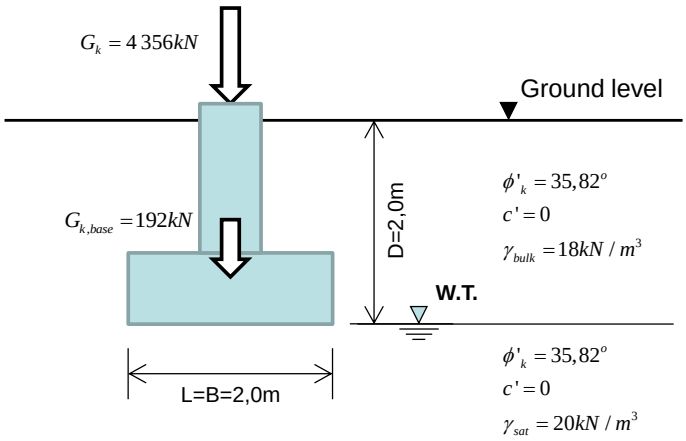


Figure 40: Footing assumed for specimen bearing capacity calculation

<u>Comparison with Working Stress Design</u>

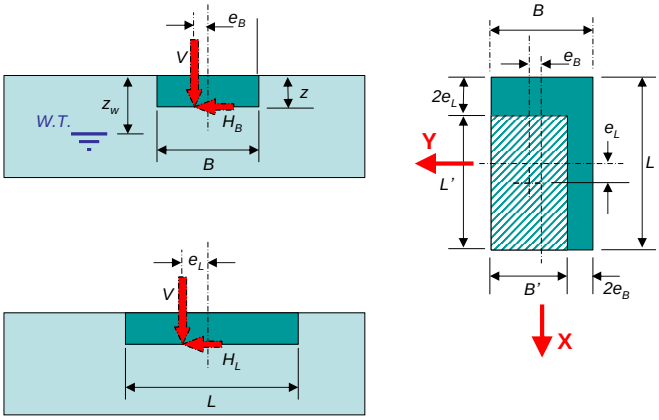
If it is assumed that one would choose soil properties for working states design that are the same as the characteristic values used in limit states design, it is possible to obtain the factor of safety from the working load design calculation for the same total applied load (G_k + Q_k) as used in the ULS design. As the check is for bearing capacity, i.e. failure in the ground, the LSD calculation should be carried out using the GEO limit state (unfactored permanent actions, factored soil strength). The WSD calculation equivalent to the LSD calculation in Figure 41 is shown in Figure 43. The resulting factor of safety is 2,61, within the range of 2,5 to 3,0 typically used in WLD bearing capacity calculations.

The numbers superimposed on Figure 42 show the factors of safety that are obtained in the case where the footing carries only permanent actions, i.e. $G_k / (G_k + Q_k)$ = 1,0. In this case, because γ_G = 1,0 for the GEO limit state, the total load on the footing (G_k + Q_k) is the same for both the WLD and LSD calculations.

The calculations can be repeated for various proportions of imposed load. As the imposed load is factored by γ_Q = 1,3, the total load on the footing assumed for the WSD check (G_k + Q_k) will be less than the design resistance of the footing which, at the limite state, is equal to the design action $G_k + 1{,}3.Q_k$. The factor of safety from the WSD calculation for the same total load will therefore increase. This is equivalent to saying that the LSD calculation becomes more conservative as a result of the application of a partial action factor to part of the loading. The factors of safety obtained when the imposed load represents 50% of the total load, i.e. $G_k / (G_k + Q_k)$ = 0,5, are shown in Figure 44.

Bearing capacity of shallow footing under inclined load

using Appendix D.4 of EN1997



Design Apporach DA1(C2)

FOOTING

Width	B	2.00	m	shorter physical dimension
Length	L	2.00	m	longer physical dimension
Founding Depth	z	2.00	m	below ground level

LOADING Input characteristic loads and appropriate load factor

Load number		1	2	3	4	5	6	Design
Load description		Perm	Footing	Imposed	Wind			
Partial action factors	γ_E	1.00	1.00					
Load combination factor	ψ	1.00	1.00					
Loads	F_x							0 kN
	F_y							0 kN
	F_z	4356	192					4548 kN
Moments	M_x							0 kNm
	M_y							0 kNm

SOIL PROPERTIES

		γ_M	Characteristic value	Design value		
Friction angle	ϕ'	1.25	35.8	30.0 degrees		0.524 radians
Cohesion	c'	1.25	0.0	0.0 kPa		
Bulk unit weight	γ_{bulk}	1.00	18.0	18.0 kN/m^3		
Saturated unit weight	γ_{sat}	1.00	20.0	20.0 kN/m^3		
Depth of water table	z_w			2 m	below ground level	

RESULT

Criterion		$V_d < R_d$	$H_d < R_d$		$e_L < L/3$	$e_B < B/3$
	E_d	4548	0 kN	e	0.000	0.000 m
	R_d	4548	2626 kN	(L or B)/3	0.667	0.667 m
		ACCEPT	ACCEPT		ACCEPT	ACCEPT

CALCULATIONS

Eccentricity and effective base size

Eccentricity along base	e_L	0.000	m
Eccentricity across base	e_B	0.000	m
Effective length	L'	2.00	m
Effective width	B'	2.00	m

Unit weight adjusted for water table:

		Condition	h>B	0<h<B	-z<h<0	h<-z
		True?	FALSE	TRUE	FALSE	FALSE
Below founding level	10.19		0	10.19	0	0
Above founding level	18		0	18	0	0
Depth of WT below footing	0					

Bearing Capacity Factors

	N_c	30.140
	N_q	18.401
	N_γ	20.093

Shape Factors

	S_c	1.529
	S_q	1.500
	S_γ	0.700

Inclination Factors

Horizontal resultant	H	0 kN	
Angle to L' direction	θ	1.571 radians	90 degrees
	m_B	1.500	
	m_L	1.500	
	m	1.500	
	i_c	1.000	
	i_q	1.000	
	i_γ	1.000	

Bearing Capacity

	c	0
	q	994 kPa
	γ	143 kPa
	R_d	4548 kN

Sliding Resistance

	$\gamma_{R;h}$	1.00 1.0 for design approach 1
	R_d	2626 kN

Figure 41: Specimen bearing capacity calculation using EN1997-1

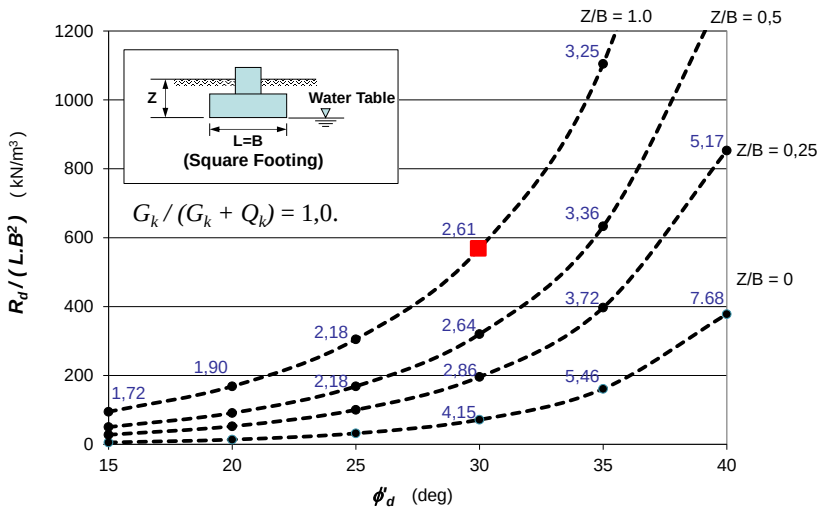
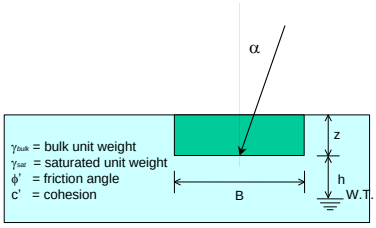


Figure 42: Factors of safety from WSD for GEO limit state no live load

Bearing capacity of shallow footing under inclined load

using Meyerhof's method and bearing capacity factors



Friction angle		35.81753	degrees	Nc	49.7
Cohesion	c'	0	kPa	Nq	36.9
Bulk unit weight	$gamma_{bulk}$	18	kN/m³	Ng	43.0
Saturated unit weight	$gamma_{sat}$	20			
Load inclination	alpha	0	degrees		
Width of footing	B	2.00	m		
Length of footing	L	2.00	m		
Depth of founding	z	2.00	m		
Depth of water table	h	0.00		NB: from founding level (+ve down, -ve up)	
Factor of safety	FOS	2.61			

Vertical bearing capacity q_f	**2905**	**kPa**
Vert. All. bearing stress q_a	**1137**	**kPa**

Allowable load **4548 kN**

k_p 3.821667

Shape factors	Lc	1.764	Unit weight adjusted for water table:	
	Lq	1.382	Below founding level	10 kN/m³
	Lg	1.382	Above founding level	18 kN/m³
Depth factors	Dc	1.391		
	Dq	1.195		
	Dg	1.195		
Inclination factors	Ic	1.000		
	Iq	1.000		
	Ig	1.000		

Figure 43: Equivalent working load design

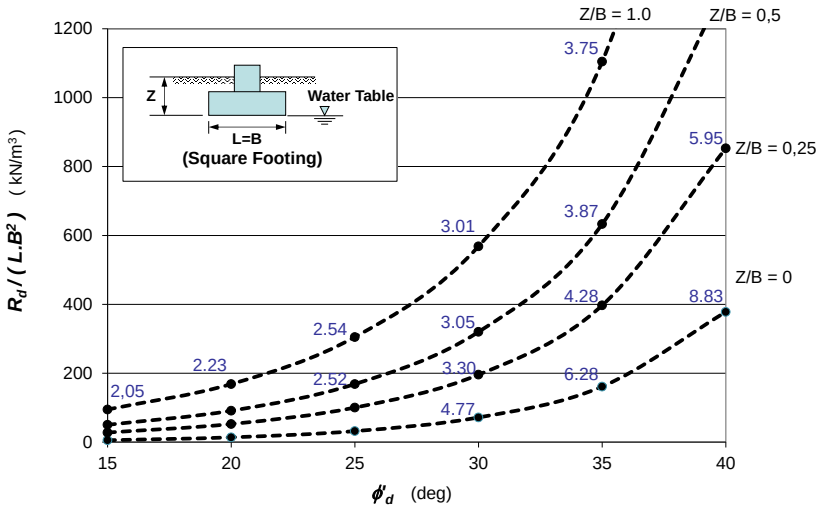


Figure 44: Factors of safety from WSD for GEO limit state with 50% live load

The equivalent WSD design in Figure 43 has been undertaken using the Meyerhof (1963) bearing capacity equation and bearing capacity factors. This was the approach used by the Candidate prior to the introduction of limit states design. The bearing capacity factors used in in EN1997-1 and those used in the working stress design are compared in Table 15.

Table 15: Comparison of bearing capacity factors

Limit States Design (EN1997-1 Annex D.4)	Working Load Design (Meyerhof, 1963)
Bearing Capacity Factors: $N_c = (N_q - 1) \cdot \cot \phi'$ $N_q = e^{\pi . \tan \phi'} \cdot \tan^2 (45^o + \phi'/2)$ $N_\gamma = 2(N_q - 1)$ for a rough base	Bearing Capacity Factors: $N_c = (N_q - 1) \cdot \cot \phi'$ $N_q = e^{\pi . \tan \phi'} \cdot \tan^2 (45^o + \phi'/2)$ $N_\gamma = 2(N_q - 1) \cdot \tan(1,4 \cdot \phi')$
Shape Factors: $s_c = (s_q \cdot N_q - 1)/(N_q - 1)$ $s_q = 1 + (B'/L') \cdot \sin \phi'$ $s_\gamma = 1 - 0,3(B'/L') \cdot \sin \phi'$	Shape Factors: $s_c = 1 + 0,2 N_\phi \cdot B/L$ $s_q = s_\gamma = 1,0$ for $\phi = 0^o$ $s_q = s_\gamma = 1 + 0,1 \cdot N_\phi \cdot B/L$ for $\phi > 10^o$ where $N_\phi = [\tan^2 (45^o + \phi'/2)]$

Limit States Design (EN1997-1 Annex D.4)	Working Load Design (Meyerhof, 1963)
Depth Factors: $d_c = d_q = d_\gamma = 1,0$	Depth Factors: $d_c = 1 + 0,2\sqrt{N_\phi \cdot D/B}$ $d_q = d_\gamma = 1,0$ for $\phi = 0^o$ $d_q = d_\gamma = 1 + 0,1\sqrt{N_\phi \cdot D/B}$ for $\phi > 10^o$
Load Inclination Factors: $i_c = i_q - (1 - i_q)/(N_c \cdot \tan\phi')$ $i_q = \left[1 - H/(V + A' \cdot c' \cdot \cot\phi'\right]^m$ $i_\gamma = \left[1 - H/(V + A' \cdot c' \cdot \cot\phi'\right]^{m+1}$ Where: $m = m_b = [2 + B'/L']/[1 + B'/L']$ for H//B' $m = m_l = [2 + L'/B']/[1 + L'/B']$ for H//L'	Load Inclination Factors: $i_c = i_q = (1 - \alpha/90^o)$ $i_\gamma = 1 - \alpha/\phi)^2$

The comparison of LSD and WSD can also be done in a different way by comparing the design resistance of the footing (LSD) with the allowable load obtained from a WSD calculation for a factor of safety of 2,5. This comparison is shown in Figure 45. From this figure, it can be seen that the two methods yield reasonably comparable results for values of the friction angles up to 35° (ϕ'_d =30°). Beyond this value, the very significant increases in the values in the bearing capacity factors N_q and N_γ for the higher friction angles used in the working load design calculations influence the calculation. It is important to note that it is not the differences in the factors used in the two design methods that has the overriding effect, it is the application of the partial material factor to $\tan \phi'$ as opposed to a constant factor of safety that causes the difference. In most practical situations, the value of ϕ' would be less than 40° (i.e. ϕ'_d for the GEO limit state <34°).

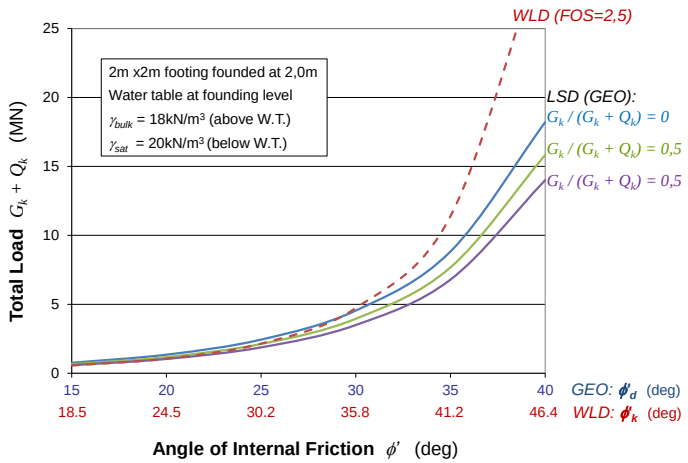


Figure 45: Comparison of results from LSD(GEO) and WLD (FOS=2,5)

<u>Application in Typical South African Conditions</u>

Since SANS 10160-5 was first introduced in 2010, the Candidate has received a number of enquiries regarding the application the bearing capacity provisions of Annex C.2 using the data given in a typical South African geotechnical report. The majority of such reports provide a description of the soil profile together with an estimated allowable bearing pressure (EABP, as it has become known) for various layers in the profile. Very few reports will provide strength parameters for the soil i.e. c_u for undrained clays, or ϕ' and c' for frictional soils. In most instances, the allowable bearing pressure is based on the consistency of the soil profile as observed during profiling rather than test results, and takes no account of the geometry of the footing.

This approach is flawed.

Allowable bearing pressure is the pressure that can safely be applied to the soil by the foundation without causing failure of the soil or excessive settlement. However, the allowable bearing capacity of a foundation and the settlement that it will undergo are not properties of the soil type and its consistency alone, they are also dependent on the geometry of the footing. From Figure 44 above, it can be seen that for a design friction angle (ϕ'_d) of 30°, the design resistance of the foundation expressed in terms of a bearing pressure increases from 72xB kPa to 570xB kPa as the depth of founding goes from surface to 1B below ground level (B = width of footing). Thus, not only is the bearing capacity dependent of founding depth, it also varies in proportion to the width of the footing. The Candidate frequently uses the example given in Figure 46 to illustrate this point. The same is true of settlement which is influenced by the size (particularly the width) of the footing and by the depth of founding. In short, the EABP given in a typical foundation investigation report is nothing more than a “gut feel” by the author of the report and has no theoretical basis unless it is specific to a given foundation geometry. There is, therefore, no way that the EABP value given can be used to derive the design resistance of a foundation for use in a limit states design calculation.

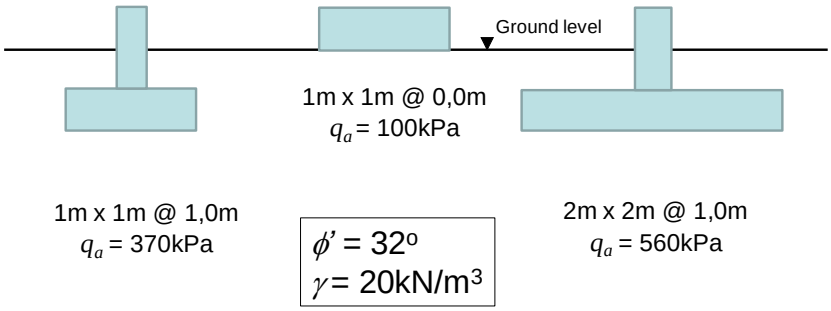


Figure 46: Influence of foundation geometry on allowable bearing pressure.

The best that a structural designer can do in this situation is to go back to any test results that may be included in the report, such as SPT or CPT test, and derive design parameters (c_u , or ϕ' and c' for strength and E' for compressibility) from these test results. These parameters can then be used in the calculation of the required design resistance of the foundation and an estimate of its settlement.

A further point to note is that the allowable bearing pressure is, as stated earlier, the pressure exerted by a foundation that will satisfy both settlement and bearing capacity requirements, i.e. it combines the serviceability and ultimate limit states into a single value. In limit states design, these calculations are treated separately.

11.5 Earth Pressure Distributions (Annex C.4)

Annex C.4 of SANS 10160-5 gives guidance to designers of buildings for the calculation of earth pressure. The values in Table C.1 are given without any explanation of their derivation. The theory behind the derivation of these values used by the Candidate during development of the code is set out below.

11.5.1 Background to Calculations

<u>Assumed Soil Types and Soil Properties</u>

Two soil types have been assumed for the calculation of earth pressures. These are based on what is described in TMH 7 (1981) as a “Type 1” and a “Type 2” backfill. These materials are described as follows:

a) Type 1: Coarse grained sands or gravels with a low fines content such that the compacted material has the properties of a free draining granular material.

b) Type 2: Fine grained silty sands with low plasticity fines.

The following material properties have been assigned to these materials based on the Candidate’s experience. The values chosen for the angle of shearing resistance are conservative.

Table 16: Soil parameters assumed for earth pressure calculations

Parameter	Symbol	Type 1 Backfill	Type 2 Backfill	Notes
Effective friction angle	ϕ'_k	35°	30°	Assumed
Effective cohesion	c'	0	0	Non-cohesive
Wall friction	δ	0	0	Ignored
Dry unit weight	γ_d	20kN/m^3	18kN/m^3	Assumed
Compaction moisture content	w	8%	11%	Assumed
Bulk unit weight	γ_{bulk}	21,6kN/m^3	20,0kN/m^3	Assumed
Void ratio	e	0,30	0,44	SG = 2,65
Saturated unit weight	γ_{sat}	22,2kN/m^3	21,1kN/m^3	$(1+w).\gamma_d$

Limitations

Because the pressure distribution is based on assumed soil types, the resulting pressure distribution will be dependent on the extent to which the backfill placed behind the wall accords with these assumptions. Wall friction has been ignored, which is conservative in all cases. In view of these "approximations", the applicability of the resulting earth pressure distributions has been limited to backfill depths of less than 7,5m (approximately the depth of a double basement.

Components of Earth Pressure

The earth pressure against a basement retaining wall will generally comprise the pressures due to

- soil and ground water[18];
- surcharge; and
- compaction.

Depending on the extent to which the wall is permitted to move, these pressures may be active pressures (movement of wall away from soil) or at-rest pressures (no wall movement). Passive pressures are unlikely to develop in the case of basement walls unless the backfill swells for some reason.

The compaction pressure distribution moves up the wall as the material is placed. It remains at its maximum value until the earth pressure due to the soil/groundwater and surcharge exceed the earth pressure locked in by compaction. In other words, it acts on its own or has been exceeded by other components of earth pressure.

These components of earth pressure are depicted in Figure 47.

[18] According to Clause 5.3.3, the earth pressure due to soil and ground water are regarded as a single action.

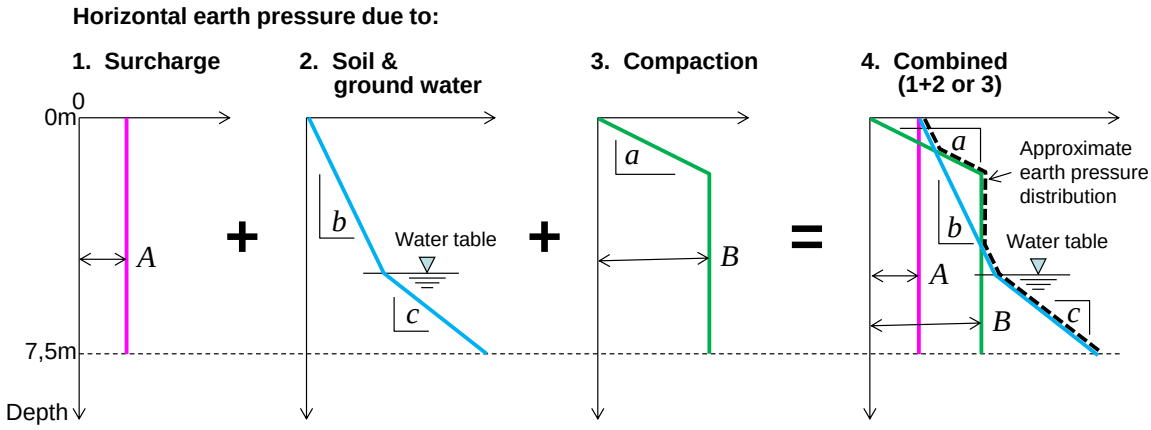


Figure 47: Pressure distribution diagrams for each component of earth pressure

Calculations

General

Due to the approximate nature of the earth pressure distribution given in Annex C.4 of SANS 10160-5, a partial factor for uncertainties in modelling the effects of actions ($\gamma_{S,d}$) of 1,1 has been included in all earth pressure calculations given below.

In the case of the GEO limit state, the applicable partial action factors (γ_Q = 1,3) and partial material factors ($\gamma_{\phi'}$ = 1,25) have been included in the calculation. The values produced are therefore design values of earth pressure. For the STR and STP-P limit state, the partial material factors are all 1,0. No partial action factors have been included. The values obtained are characteristic values to which partial action factors and action combination factors must still be applied.

The coefficients of earth pressure for the GEO and STR / STR-P limit states are given in Table 17. The active pressure applies to a yielding wall and the at-rest pressure to a rigid wall.

Value	Formula	Type 1 Backfill	Type 2 Backfill
GEO LIMIT STATE			
Design friction angle	$atan(tan\phi'_k/\gamma_{\phi'})$	29,26°	24,79°
Active earth pressure coefficient (K_a)	$(1-sin\phi'_d)/(1+ sin\phi'_d)$	0,343	0,406
At-rest earth pressure coefficient (K_0)	$(1 - sin\phi'_d)$	0,511	0,581
STR / STR-P LIMIT STATE			
Design friction angle	$\phi'_d = \phi'_k$ ($\gamma_{\phi'}$ = 1,0)	35,0°	30,0°
Active earth pressure coefficient (K_a)	$(1-sin\phi'_d)/(1+ sin\phi'_d)$	0,271	0,333
At-rest earth pressure coefficient (K_0)	$(1 - sin\phi'_d)$	0,426	0,500

Table 17: Calculation of earth pressure coefficients

Earth pressure due to surcharge (A)

At rest $A = q \,.\, K_0 \,.\, \gamma_Q \,.\, \gamma_{S,d}$ (kPa)
Active $A = q \,.\, K_a \,.\, \gamma_Q \,.\, \gamma_{S,d}$ (kPa)

where q is the uniformly distributed surcharge load on the ground surface.

Earth pressure due to soil and ground water (b and c)

At rest $b = (K_0 \,.\, \gamma_{bulk}) \,.\, \gamma_{S,d}$ (kPa/m)
Active $b = (K_a \,.\, \gamma_{bulk}) \,.\, \gamma_{S,d}$ (kPa/m)

At rest $c = [K_0 (\gamma_{sat} - \gamma_w) + \gamma_w] \,.\, \gamma_{S,d}$ (kPa/m)
Active $c = [K_a (\gamma_{sat} - \gamma_w) + \gamma_w] \,.\, \gamma_{S,d}$ (kPa/m)

Earth pressure due to compaction (a and B)

The earth pressures due to compaction are based on the work of Clayton et al (1993). According to this reference, the earth pressure due to compaction builds up linearly until, at some depth below the compaction surface, it reaches a maximum value. This maximum value is a function of compaction line load per unit length (p) taken as the sum of the static load and the centrifugal (vibratory) load per unit width of the roller drum.

The linear rate of increase (a) is given as follows:

At-rest $a = (\gamma_{bulk} / K_0) \,.\, \gamma_Q \,.\, \gamma_{S,d}$ (kPa/m)

Active $a = (\gamma_{bulk} / K_a) \,.\, \gamma_Q \,.\, \gamma_{S,d}$ (kPa/m)

The maximum value (B) is given by the following equation:

Active and at-rest $B = \sqrt{\dfrac{2 \cdot p \cdot \gamma_{bulk}}{\pi}} \cdot \gamma_Q \cdot \gamma_{S,d}$ (kPa)

where p is the sum of the static load and the centrifugal (vibratory) load per unit width of the roller drum in kN/m.

11.5.2 *Specimen Earth Pressure Calculation*

Figure 48 shows the cantilever retaining wall used to compare the results obtained using Annex C.4 with the results of a specimen calculation. The objective of the analysis is to determine the length of the heel (L). The solution is given in Figure 49. Note that in this calculation, surcharge behind a vertical line through the back of the heel (AB) has been included as an unfavourable vertical load. The surcharge vertically above the heel is regarded as an unfavourable or a favourable vertical action, whichever gives the more conservative result. As the surcharge is a variable action, this is equivalent to assuming the surcharge to act or not to act. Similarly, in the case of the EQU limit state, the soil above the heel, i.e. to the left of line AB, is also assumed to be an unfavourable or a favourable permanent action and is factored accordingly. Neither of these assumptions has any effect on the earth pressures against the wall which, for the purposes of this calculation, are assumed to act on the virtual back of the wall AB. They do, however, affect the bearing pressure below the toe of the wall, and the resistance to sliding and overturning.

This example is based on Example 5 used in the Workshop on the Evaluation of Eurocode 7 (Orr, 2005). The density of the soil has been altered to accord with that assumed for a Type 1 material in Table 16 above and the ground surface has been assumed to be horizontal in order to keep the problem simple and not to introduce factors that may obscure the trends.

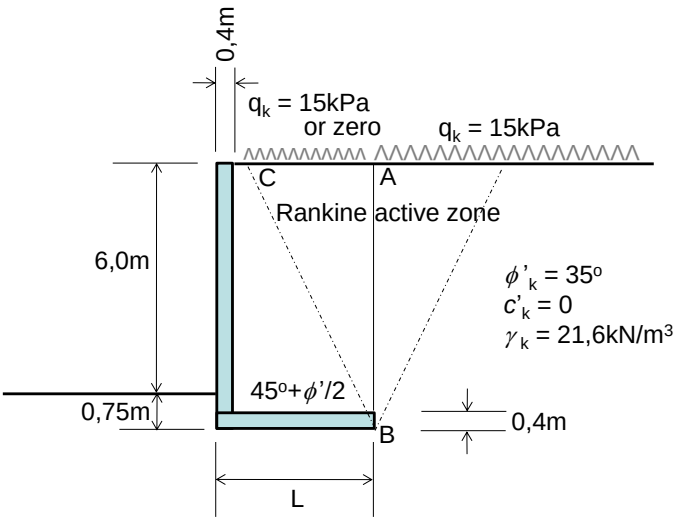


Figure 48: Specimen earth pressure problem

From the solution given in Figure 49, it can be seen that the bearing capacity below the foundation for the GEO limit state is the controlling factor. A heel length of 3,99m (say 4,0m) is required to ensure that the design effect of actions (vertical load of 5 985kN on the footing below a 10 m length of retaining wall) is less than the design (bearing) resistance (5 991kN). These two values are highlighted near the bottom of Figure 49. This is a common situation using the calculation of bearing pressure according to Annex D of EN 1997-1 where the method incorporates a check for sliding resistance and the check for overturning is implicit in the calculation of the effective width of the base. The EQU sliding and overturning checks will only be the controlling factors in situations where there is a high proportion of imposed load (surcharge in this instance) which attracts a higher partial action factor (1,6) than is the case in the GEO limit state (1,3) or the where the lower value of $\gamma_{G\text{-}fav}$ (0,9) has an effect. Such situations may occur when the retaining wall has a toe.

Note that no bearing capacity check has been undertaken for the EQU limit state. This is because Clause 7.1.1 b) of SANS 10160-1 indicates the EQU limit state considers the *loss of static equilibrium of the structure or any part of it or the ground considered as a rigid body or involving uplift due to water pressure (buoyancy) or other vertical actions, where the strengths of construction materials or ground are generally not governing*. Bearing capacity failure is dependent on the strength of the ground and should therefore not be considered using the EQU limit state.

Parameter	Description / Calculation		WSD		LSD: GEO		LSD: EQU		
			V favourable	V unfavourable	V favourable	V unfavourable	V favourable	V unfavourable	
Wall Geometry									
H	depth of backfill at wall (above u/s wall)	m	6.75	6.75	6.75	6.75	6.75	6.75	
L	Length of Heel (from front of wall)	m	3.99	3.99	3.99	3.99	3.99	3.99	
d	Thickness of base	m	0.40	0.40	0.40	0.40	0.40	0.40	
t	Thickness of stem	m	0.40	0.40	0.40	0.40	0.40	0.40	
Partial material factors									
$\gamma_{\phi'}$	Partial material factor tanϕ '		1.00	1.00	1.25	1.25	1.25	1.25	
$\gamma_{c'}$	Partial material factor c'		1.00	1.00	1.25	1.25	1.25	1.25	
γ_R	Partial resistance factor		1.00	1.00	1.00	1.00	1.00	1.00	
Partial load factors									
γ_G-unfav			1.00	1.00	1.00	1.00	1.20	1.20	
γ_G-fav			1.00	1.00	1.00	1.00	0.90	0.90	
γ_Q-unfav			1.00	1.00	1.30	1.30	1.60	1.60	
γ_Q-fav			0.00	0.00	0.00	0.00	0.00	0.00	
Characteristic Values									
ϕ'_k	Friction angle	deg	35	35	35	35	35	35	
c'_k	Cohesion	kPa	0	0	0	0	0	0	
γ_k	Density	kN/m^3	21.6	21.6	21.6	21.6	21.6	21.6	
γ_{conc}	Concrete density	kN/m^3	25	25	25	25	25	25	
q_k	Surcharge	kPa	15	15	15	15	15	15	
Design Values									
ϕ'_d	Friction angle	deg	35.00	35.00	29.26	29.26	29.26	29.26	
c'_d	Cohesion	kPa	0	0	0	0	0	0	
γ_d	Density	kN/m^3	21.6	21.6	21.6	21.6	21.6	21.6	
γ_{conc}	Concrete density	kN/m^3	25	25	25	25	25	25	
Earth Pressure (design Values)									
α	Angle of back of wall	deg	0.00	0.00	0.00	0.00	0.00	0.00	from vertical (+ve into soil)
δ	Angle of wall friction	deg	0.00	0.00	0.00	0.00	0.00	0.00	
θ	Inclination of earth pressure vector	deg	0.00	0.00	0.00	0.00	0.00	0.00	below horizontal
K_{ah}	Coeficient of horizontal earth pressure		0.271	0.271	0.343	0.343	0.343	0.343	Uses KaKp programme
E_h	Horizontal earth pressure - fill	kN/m	133.3	133.3	169.0	169.0	169.0	169.0	
E_v	Vertical earth presure - fill	kN/m	0.0	0.0	0.0	0.0	0.0	0.0	
Q_h	Horizontal earth pressure - surcharge	kN/m	27.4	27.4	45.2	45.2	55.6	55.6	Surcharge unfavourable
Q_v	Vertical earth presure - surcharge	kN/m	0	0	0	0	0	0	
Sliding Resistance									
	Width of soil above base (top)	m	3.59	3.59	3.59	3.59	3.59	3.59	
	Width of soil above base (bottom)	m	3.59	3.59	3.59	3.59	3.59	3.59	
$W1_d$	Weight of concrete stem	kN/m	63.5	63.5	63.5	63.5	57.15	76.2	
$W2_d$	Weight of base	kN/m	39.9	39.9	39.9	39.9	35.9	47.9	
$W3_d$	Weight of soil above base (triangle)	kN/m	0.0	0.0	0.0	0.0	0.0	0.0	
$W4_d$	Weight of soil above base (rectangle)	kN/m	492.4	492.4	492.4	492.4	443.2	590.9	
$W5_d$	Surcharge on soil in front of wedge	kN/m	0.0	53.9	0.0	70.0	0.0	86.2	
V_d	Vertical Loads	kN/m	595.8	649.7	595.8	665.8	536.2	801.1	
H_d	Horizontal Loads	kN/m	160.8	160.8	214.2	214.2	224.6	224.6	
R_d	Sliding resistance	kN/m	417.2	454.9	333.7	373.0	300.4	448.8	
FOS	Factor of safety (WLD)		2.59	2.83	1.56	1.74	1.34	2.00	
R_d-V_h > 0	LSD design Verification				TRUE	TRUE	TRUE	TRUE	
Overturning about toe									
x_{W1}	Lever arm W1	m	0.20	0.20	0.20	0.20	0.20	0.20	
x_{W2}	Lever arm W2	m	2.00	2.00	2.00	2.00	2.00	2.00	
x_{W3}	Lever arm W3	m	3.99	3.99	3.99	3.99	3.99	3.99	
x_{W4}	Lever arm W4	m	2.20	2.20	2.20	2.20	2.20	2.20	
x_{W5}	Lever arm W5	m	2.20	2.20	2.20	2.20	2.20	2.20	
x_{Ev}	Lever arm Ev	m	3.99	3.99	3.99	3.99	3.99	3.99	
x_{Qv}	Lever arm Qv	m	3.99	3.99	3.99	3.99	3.99	3.99	
y_{Eh}	Level arm Eh	m	2.25	2.25	2.25	2.25	2.25	2.25	
y_{Qh}	lever arm Qh	m	3.38	3.38	3.38	3.38	3.38	3.38	
M_d	Overturning moment	kNm/m	392.6	392.6	532.8	532.8	568.0	568.0	
R_d	Resisting Moment	kNm/m	1173.1	1291.3	1173.1	1326.8	1055.8	1596.9	
	Net moment	kNm/m	780.5	898.7	640.3	794.0	487.8	1028.9	
FOS	Factor of safety (WLD)		2.99	3.29	2.20	2.49	1.86	2.81	
R_d-M_d > 0	LSD design Verification				TRUE	TRUE	TRUE	TRUE	
Bearing Resistance									
L'	Length of wall considered	m	10	10	10	10			Assumed
V_d	Vertical Loads	kN	5958	6497	5958	6658			
H_d	Horizontal Loads	kN	1608	1608	2142	2142			
α	Load inclination	deg	15.1	13.9	19.8	17.8			
	Net moment	kNm	7805	8987	6403	7940			
B'	Effective width of base	m	2.620	2.767	2.149	2.385	NOT APPLICABLE		
z	Founding Depth	m	0.75	0.75	0.75	0.75			
$R_{d,v}$	Vertical Resistance	kN			5991	6997			
$R_{d,h}$	Horizontal Resistance	kN			3730	3730			
R_d-E_d > 0	LSD design Verification				TRUE	TRUE			
	Vertical bearing Pressure	kPa	227	235					
	Ultimate vertical bearing Pressure	kPa	825	847					
FOS	Factor of safety (WLD)		3.83	3.80					

Figure 49: Specimen earth pressure calculation

11.5.3 *Comparison with SANS 10160-5: Annex C.4*

The wall analysed in Figure 49 is a "yielding" wall (active pressure condition) and the soil assumed has the properties of a "Type 1" material as given in Table 16. As such, the earth pressure on the wall due to the soil and the surcharge can be calculated using the values from Table C.1 in SANS 10160-5. The results of this calculation for the GEO limit state are given in Table 18. The difference between the results is the partial factor for uncertainties in modelling the effects of actions ($\gamma_{S,d}$) of 1,1 included in Annex C.4.

Table 18: Specimen calculation – comparison with result from Annex C.4

Component of earth pressure	Value from Table C.1	Calculation of earth pressure	Result from Figure 49
Soil and ground water	$b=8{,}2kPa/m$	$E_d= 8{,}2 \,.\, 6{,}75^2 / 2$ = 186kN	169kN (x1,1 = 186kN)
Surcharge	$A=0{,}49q$	$Ed = 0{,}49 \,.\, 15 \,.\, 6{,}75$ = 50kN	45kN (x1,1 = 50kN)

11.5.4 *Comparison with Working Load Design*

The WLD calculations are included in Figure 49. From this figure, it is noted that factor of safety from the bearing pressure calculation is 3,8[19]. This is well above the factor of safety of 2,5 normally required for the bearing capacity of a granular founding material. If the calculation is repeated assuming the factor of safety of 2,5 in the working stress design to be the controlling factor, the required length of the heel reduces from 3,99m to 3,48m. Thus, for this particular situation, the requirements of SANS 10160 are more onerous than current practice using WSD.

To investigate this situation further, the above calculation was repeated for a range of ϕ'_k from 25° to 40°. In all cases, bearing pressure below the toe was found to be the controlling factor for both WSD and LSD calculations. For each value of ϕ'_k, the required length of the heel was calculated to satisfy the requirement that the factor of safety against bearing capacity failure should be at least 2,5 for working load designs or, in the case of limit state designs, $E_d<R_d$. The results are depicted in Figure 50. As in the case of bearing capacity (Figure 45), the limit states design again produces a more conservative result for values of ϕ'_k greater than about 30°. At lower friction angles, the working load design produces a more conservative result.

[19] It is interesting to note that this minimum factor of safety occurs for the "V unfavorable" situation (i.e. surcharge to back of wall) whereas the more critical situation in the LSD analysis was the "V favorable" situation. This illustrates the need to check all possible design situations both in WDS and LSD calculations.

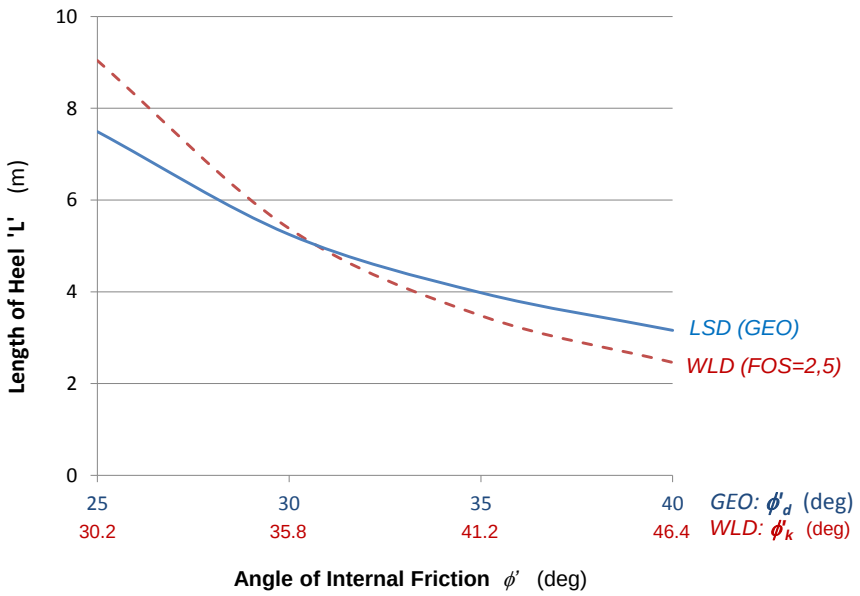


Figure 50: Specimen calculation – required length of heel

11.5.5 *Alternative Method of Earth Pressure Calculation*

The above calculations all considered the earth pressure acting on the virtual back of the wall (line AB in Figure 48). The reason why the angle of wall friction on this virtual wall is zero is because this is the plane of symmetry of the Rankine active wedge that forms in the backfill behind the wall. One could equally well have determined the earth pressure acting on plane BC using a wall friction equal to the angle of internal friction of the soil. The resulting loads on the wall, both vertical and horizontal, will be identical to those obtained from an analysis of the earth pressure on plane AB for the GEO limit state. This is because the vertical component of the earth pressure on Plane BC is exactly equal to the weight of the block of soil ABC (Figure 48) and the horizontal component is equal to the (horizontal) earth pressure on AB. This is illustrated graphically in Figure 51.

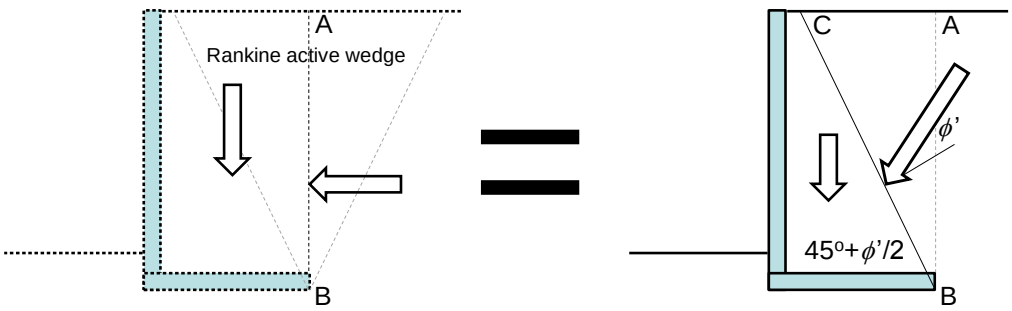


Figure 51: Equivalence of methods of earth pressure calculation (GEO limit state)

This is, however, not the case for the EQU and STR limit states where the weight of soil above the heel of the wall is a favourable permanent action which attracts a partial action factor of less than 1,0 (0,9 in both instances). For these limit states, the vertical action of the wall will be lower for the left hand sketch in Figure 51 where the weight of a larger block of soil above the heel is “factored down” than for the right hand sketch.

There is little that can be done about this situation apart from being aware that the two methods that are equivalent in working load design will give different results for certain limit states in the limit states design process.

11.6 Code Review

Codes of practice are living documents that require periodic review. At SABS, such reviews generally take place every three years. The items listed below need to be considered during the next review of SANS 10160-5.

11.6.1 Resistance Model Factor for Piles

Clause B.3.1 gives the design resistance of an axially loaded pile as

$$R_d = R_k / (\gamma_R \cdot \gamma_{R,d})$$

where γ_R is a partial resistance factor, and

$\gamma_{R,d}$ is a partial factor for uncertainty in the resistance model.

For the GEO limit state, Table B.2 gives the value of γ_R as 1,6 for piles in compression and 1,7 for piles in tension. Clause B.3.3 gives the value of $\gamma_{R,d}$ as 1,5 for situations where the characteristic resistance of a pile is derived from calculations based on shear strength parameters and 1,0 where the characteristic resistance is determined from profiles of in situ test results.

The value of 1,5 was determined from a preliminary analysis of the data compiled by Dithinde (2007). No justification was found at the time for differentiating γ_R and $\gamma_{R,d}$ on the basis of pile type or the nature of the ground (granular or cohesive).

More recent work by Dithinde and Retief (2012) has derived values for model factors for various pile types and ground conditions such that there is only a 5% chance that a lower value may be required. The values derived on this basis are given in Table 19.

Table 19: Resistance Model Uncertainty Factors for Piles

Pile class	**Model Factor** $\gamma_{R,d}$
Driven piles – non cohesive soil	2,1
Bored piles – non cohesive soils	1,7
Driven piles – cohesive soils	1,5
Bored piles – cohesive soils	1,5
All pile classes combined	1,6

Dithinde and Retief then determined the reliability index β implied by the value of $\gamma_{R,d}$ = 1,5 given in the current version of SANS 10160-5 for both single piles and pile groups. Redundancy due to group and system effects causes the reliability index of the pile group to be higher than that of a single pile with multipliers of 1,07 to 1,5 being reported in the literature. The factor used by Dithinde and Retief to derive the reliability index of a pile

group from that of a single pile was 1,3. The determination of reliability index took account of the load statistics for permanent actions and variable actions and the partial action factors of 1,0 and 1,3 applied to these two categories of actions in the GEO limit state. The reliability index was found to increase as the proportion of live load (variable action) on the pile increases. Table 20 gives the reliability index β implied by the value of $\gamma_{R,d}$ = 1,5 given in the current version of SANS 10160-5 for both single piles and pile groups for the case where the live (variable) load on the structure is 50% of the dead (permanent) load. This is considered to be the lower end of the range of live loading on most typical structures.

Table 20: Reliability Indices for Piles implied by SANS 10160-5

Pile class	Reliability Index, single pile	Reliability Index, pile group
Driven piles – non cohesive soil	2,24	2,91
Bored piles – non cohesive soils	2,71	3,52
Driven piles – cohesive soils	2,60	3,38
Bored piles – cohesive soils	3,21	4,17
All pile classes combined	2,70	3,51

Note that the target reliability index for SANS 10160 of 3,0 is for individual components of the structure and not the structure as a whole. Thus, the reliability index for single piles should meet this requirement. The value of $\gamma_{R,d}$ = 1,5 in the current version of the code may therefore be marginally low.

11.6.2 Partial Material Factor on Cohesion

The partial material factors on effective cohesion (c') and undrained shear strength (c_u) given in Table B.1 of SANS 10160-5 are 1,25 and 1,4 respectively. These values are in keeping with the default values given in Eurocode 7 and also with the values in the UK National Annex.

As indicated in Section 11.4.1, the design bearing resistance of footings on undrained clay calculated in accordance with Figure C.1 of Annex C in SANS 10160-5 corresponds to a factor of safety of between 1,4 and 1,82 depending on the proportion of live load on the structure. This assumes that the undrained shear strength of the clay used in the WLD calculation is the same as the characteristic value use in the LSD. These factors of safety are considerably lower than those that would normally be accepted in practice, typically 3,0 for footings on cohesive soils. They are also lower than the factors of safety obtained from similar calculations for footings on non-cohesive soils. In practice, a lower factor of safety of around 2,5 would be accepted for such soils.

A possible reason for this anomaly is that the undrained shear strength used in WSD is probably closer to the mean value rather than the characteristic value. Given that the coefficient of variation on undrained shear strength quoted in the literature is around 40% (Harr, 1987), this could result in a significant difference between the characteristic value used in the LSD and the value used in the WDS calculation. In the case of granular soils, where the coefficient of variation on the friction angle is typically 8% to 10%, the difference will be less pronounced.

As far as effective cohesion (c') is concerned, the poor standard of laboratory testing in South Africa often yields values of c' that are considerably higher than would have been obtained if the tests had been carried out correctly. High values of effective cohesion can have a significant effect on the calculated bearing capacity of small, shallow footings and on the stability of slopes.

There may be merit in considering an increase in the partial material factor on effective cohesion ($\gamma_{c'}$) from 1,25 to 1,4. In addition, consideration should be given to drawing more attention to the variability of undrained cohesion and the effect of soil structure (e.g. jointing or slickensiding) on this parameter possibly in the Annex B which is an informative section of the code. Increasing the partial material factor on undrained cohesion (γ_{cu}) is not considered to be appropriate as the statistical distribution of the undrained cohesion is something that should be taken into account in the selection of the characteristic value rather than by means of an increased partial material factor.

11.6.3 Removal of Basis of Geotechnical Design from SANS 10160-5

The reason for the inclusion of the *basis of geotechnical design* in SANS 10160-5 is that South Africa does not have a geotechnical design code at present. Once a geotechnical design code has been written or the decision taken to adopt Eurocode 7 for use in South Africa, the *basis of design* sections should be removed from SANS 10160.

11.6.4 Erratum

Table 21 lists the amendments required to SANS 10160-5:2011 that have been identified over the past year. Discrepancies of this nature become apparent as the profession works with the code. The Candidate keeps a log of any errors that he finds or that are brought to his attention.

Table 21: Log of amendments required to SANS 10160-5:2011

Date	Clause	Correction required
19 Oct 2011	C.4.2.2	NOTE The distributions given below are based on an effective angle of shearing resistance of 35° for a type 1 material and 30° for type 2 material (characteristic values). The *dry* weight density of the compacted material is assumed to be 20 kN/m^3 and 18 kN/m^3, respectively, for type 1 and type 2 materials *and the assumed moisture content above the water table is 8% and 11% respectively*. In both cases, wall friction and cohesion have been ignored.
02 Aug 2012	Table C.1	The value of c in the last column of table C1 for the STR-STR-P Limit state, Type 2 material, Rigid Wall should be 17,0 (not 7,3).
02 Aug 2012	Table C.1	Need to clarify that values of q and p used in the table are the characteristic values – even for the GEO limit state. The partial action factor for variable actions in the GEO limit state (1,3) is included in the calculation.

Date	Clause	Correction required
05 Aug 2012	Figure C.2	In Key above the figure caption on p33: 1. For GEO and EQU limit states: $\phi'_d = \text{atan}\left(\tan\phi'_k / 1{,}25\right)$ (present equation has a bracket in wrong place, atan not written as one function and there is an extra ϕ'_k at the start of the equation) 2. Last line of Key to be amended to reinstate the superscript in the cubic metre symbol.
15 Sep 2012	Figure C.1	The "*g x Z*" in the final term of the equation should be "$\gamma.Z$". Equation needs to be reformatted to read more easily.
16 Sep 2012	Figure C.2	Dotted lines need to be more clearly distinguishable from solid lines.
18 Sep 2012	Table C.1	The four values of a in column 5 for the GEO limit state need to be multiplied by γ_Q (1,3) as the compaction load is an imposed load. The value of B already includes this factor but it should also have been included in a. These values should read (from top to bottom) 60, 90, 49, 70 (previously 46, 69, 38, 54).
28 Sep 2012	B.3.1	In the paragraph immediately above Note 1, change $\gamma_{R,D}$ to $\gamma_{R,d}$

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12. ISSMGE TECHNICAL COMMITTEE TC23: LIMIT STATES DESIGN

12.1 The Formation of TC23

The initiative to develop harmonised standards for structural design in Europe commenced in 1974. In 1975, a formal decision was taken by the European Committee for Standardisation (CEN) to prepare the Eurocodes. In 1980, the ISSMFE offered its support to CEN with work on Eurocode 7. Professor Fukuoka, the then president of the ISSMFE, formed an ISSMFE sub-committee on Eurocode 7 with N. Krebs Ovesen of Denmark as its chairman. However, after a change in the presidency of the ISSMFE, official support for this sub-committee was withdrawn in 1982 as it was not a properly constituted technical committee and there was doubt among the leadership of the organisation whether the Society should be involved in the drafting of codes of practice. The sub-committee continued to function as an ad-hoc committee consisting of representatives of the nine member societies of the ISSMFE in the European Economic Community, still under the leadership of Krebs Ovesen. (Orr, 2007).

The first draft of the model code for Eurocode 7 was submitted to the Commission for the European Community (CEC) in 1987. In 1990, the responsibility for the drafting of the Eurocodes was transferred from CEC to CEN and Krebs Ovesen was appointed as convenor of drafting sub-committee SC7. Also in 1990, the ISSMFE officially established a technical committee TC23 on Limit States Design with Krebs Ovesen in the Chair. This TC differed from the sub-committee established in 1980 in that it was not intended to be linked to the drafting of the Eurocodes. Nevertheless, given the developments in Europe at the time, the Eurocodes tended to dominate proceedings.

The first draft of Eurocode 7 (ENV 1997-1) was approved by SC 7 and ratified by CEN in 1993, the year in which the Danish Geotechnical Society held the symposium on Limit States Design in Geotechnical Engineering referred to in Section 9.2. The South African Limit States Design in Geotechnical Engineering symposium described in Section 9.2 was held in Johannesburg in 1995. In 1996, a second international seminar *Eurocode 7 – Towards Implementation* was held in London at which the Candidate presented the outcome of the deliberations that had taken place in Johannesburg a year earlier (Day, 1996). In September 1997, the Candidate was invited to contribute to a discussion session at the 14th International Conference of the ISSMFE in Istanbul and presented a paper on the South African perspective on limit states geotechnical design (Day, 1997).

12.2 Start of the 1997 – 2001 Term

Technical committees of the ISSMGE are appointed for a four year term that is renewable at the behest of the ISSMGE President. In 1997, Professor Ishihara, the newly installed president of the ISSMGE invited the Candidate to take over the chairmanship of TC 23 from Krebs Ovesen, who had served eight years as chairman of the committee. The most likely reason for the change in leadership was a desire to shift the focus away from Europe and the Eurocodes to the investigation of limit states design implementation across the world.

The terms of reference of TC 23 for the period 1997 – 2001 were as follows:

- to review the progress made on the implementation of limit states design in geotechnical engineering in all of the ISSMGE member societies;
- to identify problems experienced by the member societies with the introduction/use of limit states design in geotechnical engineering and with the marriage of these codes with other national codes;

- to compare design approaches, partial factors and selection of design values advocated in various countries with a view to identifying differences and exploring opportunities for harmonisation;
- to encourage dialogue between the technical / drafting committees of the major standards organisations; and
- to explore the feasibility of an international symposium, possibly allied with GeoEng 2000, at which these issues could be debated.

The first meeting of the committee was held during the 12th European Regional Conference in Amsterdam in June 1999. At this meeting, TC 23 committed itself to a questionnaire to member societies addressing the first three items of the terms of reference, a workshop in Melbourne to coincide with GeoEng 2000[20] and a speciality session at the next International Conference to be held in Istanbul in 2001.

The meeting also received a report on the progress of the Eurocode and reports from six Member Societies outside of Europe which are briefly summarised below.

Eurocodes: The preliminary draft of EN1997-1 had been tabled. Parts 2 and 3 had been published in ENV form (pre-standards) on which voting would take place in 2001. The first official draft of EN1990 was expected by the end of the year. The main changes from the ENV to the draft EN version of EN1997-1 were outlined.

Japan: The major design codes in Japan are produced by various state authorities dealing with roads, railways, harbours, etc. The highway bridge design code was in the process of being converted from working load design to limit states design. Performance based design codes were gaining favour. Japan attempts to ensure harmonisation with ISO codes including ISO 2394 (reliability of structures) and ISO 3010 (seismic actions on structures).

USA: AASHTO was driving the move to LRFD (load and resistance factor design) in geotechnical engineering. The process was likely to take 5-10 years to complete.

Canada: Limit states design methods had been introduced into Canada 35 years ago by Meyerhof and others. The Canadian Foundation Engineering Manual was now in its third edition. Problems that had been experienced included double factoring, compatibility between geotechnical and structural designs, lack of code calibrations and poor education in the use of the codes.

Australia: Limit states design principles were used by structural engineers but had not been accepted by the geotechnical profession. Standards are non-statutory and are written by volunteers. Many standards are adopted from ISO, British, ASTM or DIN standards. The Australian loading code AS 1170 (structural design actions) would make it difficult to adopt Eurocode 7 without alteration.

S. Africa: S.A. finds itself in very much the same situation as Australia. There is a willingness among geotechnical engineers to adopt Eurocode 7 but this will require revision of the loading code. The approach adopted in the S.A. loading code had been to achieve a uniform load index and concern was expressed about the variability of the load index implied by the partial factors proposed for the Eurocodes.

[20] The GeoEng2000 conference was awarded to the Australian Geomechanics Society by the ISSMGE in view of their repeated failure to win a bid for holding the international conference of the ISSMGE in Australia. The Euro-centric nature of the Society was becoming an increasing concern at that stage.

Hong Kong: The Hong Kong Geotechnical Engineering Office produces a number of manuals and guides. The 1984 GEO manual for slopes is a working stress design document. The GEO manuals are being superseded by Geoguides. The Geoguide for retaining structures is a limit states design document. The partial factors are similar to those in the Eurocodes but with a single partial factor on soil strength. There will be a period of grace in which either the GEO manuals or the new limit states design Geoguides may be used.

A sub-committee was appointed to draw up a questionnaire on limit states geotechnical design practice for distribution to all member societies of the ISSMGE, the results of which were presented in Melbourne in 2000. This committee comprised Steenfelt (Denmark), Simpson (UK), Orr (Ireland), Stagys (Lithuania) and Day (South Africa).

12.3 LSD 2000 Workshop

The International Workshop Limit States Design in Geotechnical Engineering, LSD 2000, was held in Melbourne Australia on 18 November 2000, immediately prior to the start of the GeoEng 2000 special ISSMGE conference. It attracted 40 delegates, the highest attendance of all the pre-conference activities. The proceedings included 17 national reports and 13 papers, the majority of which were presented at the workshop. The proceedings were compiled by Krebs Ovesen and Day (2000) and were made available on CD and on the web.

12.3.1 National Reports

Some months prior to the workshop, the questionnaire prepared by the sub-committee set up in Amsterdam was sent to all member societies represented on TC23. In the end, the questionnaire took the form of a request for a national report which addressed a list of topics including codes of practice; design methods; load-, material- and resistance-factors; target safety indices; determination of material properties and specific problems experienced. Each of these topics was subdivided into a number of sub-topics. For example the “codes of practice” topic was sub-divided into six sub-topics which included codes used, status (legal standing) of codes, future codes, drafting of new codes, harmonisation of codes and international codes. Notes for the guidance of authors indicated what information was required on each sub-topic. As a result, the reports all had a fairly uniform format and were amenable to analysis and comparison.

Eighteen of the twenty six member societies that were approached produced national reports, including South Africa (Day, et al, 2000). The main findings of the reports were summarised in a presentation by the Candidate at the Workshop (Day, 2000). Some of these findings are summarised below.

There were many common threads running through the various national reports, some of them influenced by the fact that just over half the reports were from European countries:

- Partial factors should best be applied at the source of the uncertainty.
- The characteristic value is a cautious estimate of the mean value relevant to the limit state under consideration.
- Statistical methods are seldom used in the selection of the characteristic value.
- Selection of material properties is not a major problem.
- Safety of its citizens is the responsibility of each individual nation.
- Strength and deformation should be treated separately.
- Code writing is generally a voluntary, unpaid task.

On design methods, Canada, Czech Republic, Hong Kong, Denmark, France, Lithuania, Netherlands and Russia reported that they were using limit states geotechnical design, Belgium, Germany, Portugal, South Africa and Spain were using working stress design and Australia, Ireland and Japan were using both.

On future codes, all eleven European countries indicated that they would be using the Eurocodes. Canada, Hong Kong, Japan and Australia were to write their own codes and South Africa was still undecided.

Codes of practice are mandatory in Canada, Czech Republic, Denmark, Germany, Japan, Netherlands and Portugal. They are voluntary in Australia, Belgium, Hong Kong, Ireland, Lithuania, United Kingdom and South Africa. In France and Spain, some codes are mandatory and some not.

After a short summary of the highlights of each national report, the presentation drew the following conclusions from the report:

- the Eurocodes are having a major influence worldwide;
- insufficient attention is being paid to reliability studies and code calibration;
- reliability studies are hindered by lack of data; and
- the likelihood of ever having a universal geotechnical design code (e.g. ISO) is remote. (Day, 2000.)

12.3.2 Papers Presented

The thirteen papers presented covered a wide range of topics. These included derivation of parameters and dealing with uncertainties; design of piles, retaining walls and spread footings using LSD and other methods; LSD and numerical modelling; future Japanese codes using performance base design; and comparison of existing design approaches with Eurocode proposals.

12.4 Survey of Investigation Methods and Determination of Parameters

One of the spin-offs of the LSD 2000 conference was a survey of geotechnical investigation methods and the determination of geotechnical parameter values based on the investigation findings. The survey was formulated as a TC 23 activity by Trevor Orr of Trinity College, Dublin, and Kenji Matsui of Japan who was at Trinity College at the time, with assistance from the Candidate. Although it was initiated during the then current term of TC23, only the results of a pilot survey in Japan were available for the final meeting of TC23 in 2001. The final results of the international survey were only published at the International Workshop on Foundation Design Codes, IWS Kamakura 2002 (Orr, Matsui and Day, 2002).

The aim of the survey was to obtain information on common investigation methods used in various countries, the methods used to determine parameter values including characteristic values and how the quality of the geotechnical data is taken into account. The 40 respondents included clients, investigators, designers, contractors and others from 11 different countries, including three replies from South Africa. The largest number of responses was from Spain followed by Russia and Ireland. The results of the pilot survey in Japan was reported separately.

Some of the key findings of the survey are outlined below.

Responsibility for planning the investigation	Slightly more than half the respondents saw the planning of the investigation to be the responsibility of the investigator and slightly less saw it as the designer's function. Not surprisingly the majority of the investigators held the former opinion and the majority of the designers held the latter. The results

	were fairly similar from all countries.
Site investigation methods	For spread footings on sand, the overwhelming majority of respondents favoured the use of SPT tests. For spread footings on clays, the SPT test, CPTu test and laboratory triaxial testing were the favoured methods. For piled foundations, the same methods are employed with the in situ test methods (SPT and CPT) being favoured.
Responsibility for determining parameter values	The views on this topic differed markedly from country to country. 88% of the Germans said the investigator determines the parameters, 67% of the Irish said the designer and 89% of the Russians said the client. Again there was a noticeable bias with the majority of both investigators and designers believing this to be their responsibility.
Methods of determining parameters from test results	Here the options were between (in descending order of popularity) experience and judgement; mean ± SD/2, simple mean excluding outliers and lower bound excluding outliers. Again there was variation between countries. Respondents from Germany, Ireland and UK favoured experience and judgement while those from Russia opted for the mean or lower bound, both excluding outliers. Investigators tended more towards calculated values while designers tended towards values determined by judgement.
Usefulness of statistics in parameter determination	The majority of respondents (including all the Russian respondents) supported the use of statistics. 33% of the respondents held the opposite point of view. The general view was that the used of statistics should be tempered with judgement. Many commented that few geotechnical engineers have an adequate knowledge of statistics.
Relationship between investigators and designers	Adequate communication between investigators and designers was seen as essential but, sadly, this seems to be the exception rather than the rule.

12.5 End of the 1997 – 2001 Term

The final meeting of the 1997 – 2001 TC 23 committee was held in Istanbul in August 2001 during the 15th International Conference on Soil Mechanics and Geotechnical Engineering. With the term of the committee having come to an end, the meeting was somewhat retrospective. The outcome of the LSD 2000 workshop were presented and discussed as were the results of the pilot survey in Japan on site investigation methods and the determination of geotechnical parameters.

In retrospect, the committee had achieved four of the five aims set out in its terms of reference. The one area in which it had failed was in encouraging dialogue between the various standards organisations. More importantly, though, the committee had shifted the focus away from the Eurocodes and opened the debate on limit states design to the wider geotechnical community.

The outgoing committee expressed the view that the work of the committee should continue for at least one more term but that the scope of the committee should be broadened to include other design approaches. This view was communicated to the ISSMGE in the Administrative Report produced by the Candidate on the activities of TC23 and in a confidential report to the ISSMGE Board and Incoming President submitted in April 2001 (Day, 2001).

12.6 Evolution of TC 23 into TC205

Professor William van Impe, the incoming President of the ISSMGE, agreed to the extension of TC23's mandate for a further four years and the broadening of its terms of reference. Professor Yusuke Honjo of Japan took over the chairmanship of the committee

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as the Candidate moved on to become Vice President for Africa of the International Society and a member of the ISSMGE Board for the period 2001 – 2005. Under the chairmanship of Professor Honjo, focus of the committee was broadened with consideration of all forms of geotechnical design including performance based design standards and reliability based design methods.

In 2009, the structure of the ISSMGE technical committees was changed. The committees were subdivided into *Fundamentals* (TC 101 – 107), *Applications* (TC 201 – 216) and *Impact on Society* (TC 301 – 307). TC 23 was replaced with TC 205 *Safety and Serviceability in Geotechnical Design*. This committee is now chaired by Dr Brian Simpson of the UK. This continues the trend to widening the scope of the committee's activities from a narrow focus on limit states design. In this modern day, the meetings are more frequent and generally take the form of tele-conferences.

The Candidate remained a core member of TC 23 and is now South Africa's representative on TC 205.

A speciality session on design codes was held at the 17th ICSMGE held in Alexandria, Egypt, in October 2009. A number of members of TC 205, including the Candidate, took the opportunity of presenting a paper entitled *Eurocode 7 for geotechnical design – a model code for non-EU countries?* (Schuppener et al, 2009). The following year, TC 205 sponsored a session at the Danube European Conference on code application. The candidate presented a paper on the application of limit states design to the design of the foundations for the roof-support arch over the Moses Mabida stadium in Durban (Day and Jaros, 2010). This was one of the first applications of limit states design according to SANS 10160-5 to a major project. The Candidate acted as checking engineer for the contractor's design and liaison with the German engineers responsible for overall design of the roof structure. The benefits of a harmonised geotechnical design code were immediately evident.

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Part 4:

Other Standards Writing Activities

This Part of the dissertation describes the other standards writing activities in which the Candidate has been involved apart from SANS 10160. It focuses mainly on work done with the South African Bureau of Standards over the past three years.

13. SABS PROCEDURES AND COMMITTEES

13.1 The South Africa Bureau of Standards (SABS)

The South African Bureau of Standards is a statutory body established in terms of the Standards Act (Act 24 of 1945). It currently operates in terms of the latest edition of the Standards Act (Act 8 of 2008) as the national institution for the promotion and maintenance of standardisation and quality (SABS, 2012).

The objectives of the SABS, as stated in the said Standards Act, include the following (SANS 1-1):

- to develop, issue, promote, maintain, amend or withdraw South African national standards and related normative publications that serve the standardization needs of the South African community;
- to obtain membership of foreign or international bodies that have objectives similar to those of the SABS, and to interact with representatives of other national standards bodies; and
- to provide a procedure whereby other bodies with sectoral expertise can be recognized to produce standards that are issued by the SABS as South African national standards.

In addition, the bureau (SABS, 2012):

- participates in the development of regional (SADC) and international (ISO and IEC) standards;
- provides information on national standards of all countries as well as international standards;
- tests and certifies the conformance to standards of products and services;
- provides training on aspects of standardisation; and
- manages WTO/TBT enquiry point in South Africa.

Under the WTO/TBT, South Africa has an obligation to base its national standards on international standards where possible. The SABS Standards Division has the right to adopt ISO and IEC standards as national standards (SANS 1-1).

13.2 Standards Writing and Approval Procedures

In response to a request from SAICE for a contribution to the annual special edition of the Civil Engineering magazine focusing on networking, the Candidate produced a “laymans guide” to how national standards are written in South Africa (Day, 2011). Most of the contents of this section come from this article.

13.2.1 Standards Writing

All South African National Standards should written in accordance with SANS 1-1:2009.

In South Africa, national standards are not written by the SABS, but by the profession. The SABS is responsible for coordinating standards writing activities, providing logistical support, ensuring compliance with local and international norms and for the approval, publication and distribution of the standards.

The writing of national standards takes place under the auspices of a particular technical committee. Figure 52 shows the structure of one such technical committee which is particularly relevant to SAICE, namely SABS TC59 – Construction Standards,.

Technical committees are broadly constituted and include representation from industry, regulatory authorities and from society in general. For example, disabled persons are represented on TC59 through the relevant NGO.

Subcommittees tend to be more technically focused and would typically deal with a particular specialist field. In the case of TC59 there were 10 subcommittees. These included SC59A (cement, lime and concrete), SC59I (basis for the design of structures), SC59P (geotechnical standards), etc.

The actual standards writing normally occurs at working group level. The working groups are populated by participants in the particular activities that will be controlled by the standard. In the case of construction standards, the working group members should ideally include practitioners (designers and contractors), suppliers, academics, researchers and regulators as appropriate.

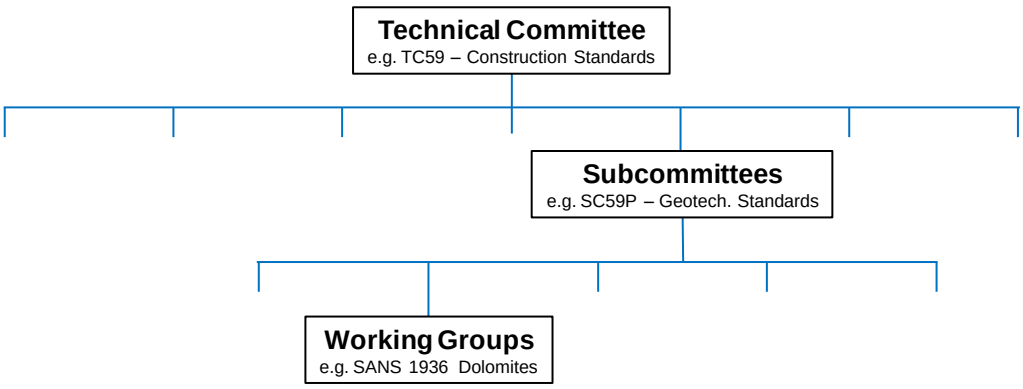

Figure 52: Typical Technical Committee structure (Day, 2011)

13.2.2 Proposals for New Standards

Any person or organisation can identify the need for a new standard or amendment of an existing standard and submit a proposal to the SABS. The SABS will formulate a "new work item" and circulate it to members of the appropriate technical committee for voting. If the proposal is accepted, the technical committee will allocate it to the relevant subcommittee who will nominate a working group to draft or amend the standard. If a suitable international or regional standard already exists, the technical committee may recommend adoption of such a standard in preference to writing a new standard.

13.2.3 Standards Approval Procedures

Once the working group has produced a draft standard, it is submitted to the subcommittee for review and onward submission to the SABS. The Bureau will edit the standard to ensure compatibility with other relevant standards and format the document. The standard will then be issued as a Committee Draft to be voted on by the Subcommittee or Technical Committee that appointed the working group. All P-members (i.e. participating members, as opposed to observers) of the relevant committee are obliged to vote and are invited to submit comments on the proposed draft.

If the vote on the draft standard is positive, the relevant committee considers the comments received and makes the necessary changes to the draft. Any significant changes are referred back to the working group for their input. The principle of

consensus[21] is applied throughout and an appeals procedure exists. Once the necessary changes have been made and consensus achieved, the document is sent back to the SABS.

The SABS then edits the document and publishes it as a Draft South African Standard (DSS) document for public comment. The comments received are used to assess any changes required and whether there is general consensus on the contents of the standard. After all the comments received have been dealt with, the standard is published as a South African National Standard.

13.3 TC59: Construction Standards

13.3.1 TC 59

Up to the end of 2011, all construction standards were dealt with under SABS Technical Committee TC59: Construction Standards. This committee found itself dealing with 500 – 600 standards. These included about 200 standards associated with the SABS 1200 series (*Standardised specifications for Civil Engineering Construction*) and the related SABS 10120 standards, twenty two SANS 10 400 standards (*Application of the National Building Regulations*) and about fifty of the new SANS 2001 and 3001 series of standards dealing with construction works and civil engineering test methods. SANS 10160 (*Basis of design and actions for buildings and industrial structures*) also fell under TC 59.

Although many of the standards under the control of TC59 were allocated to sub-committees, members of the Technical Committee itself found themselves being bombarded with voting requests on standards that were not in their field of expertise due to the breadth of TC59's scope. This was an unmanageable situation. Furthermore there were disputes within the committee regarding the legitimacy of certain actions (or lack thereof) by the SABS.

13.3.2 SC 59P: Geotechnical Standards

TC 59 has a number of specialised sub-committees under it, including sub-committee SC 59I which was responsible for SANS 10160.

One of the most recently established of these sub-committees was SC 59P: *Geotechnical Standards*. This subcommittee was formed in late 2009 and the Candidate was elected chairman of the subcommittee in his absence. The standards allocated to the sub-committee had all been drafted and been through the required approval processes. However, there was sustained objection to the publication of these standards due to a lack of participation by the geotechnical engineering and engineering geological professions in their compilation. These standards were:

SANS 633	Soil profiling and borehole logging
SANS 634	Geotechnical investigations for township development
SANS 10400-H	Application of National Building Regulations – foundations
SANS 1936-1	Development of dolomitic land – general principles and requirements
SANS 1936-2	Development of dolomitic land – geotechnical investigations and determinations
SANS 1936-3	Development of dolomitic land – design and construction of buildings, services and infrastructure

[21] According to SANS 1-1, consensus does not mean unanimity. It is simply means general agreement, characterized by the absence of sustained opposition to substantial issues.

SANS 1936-4	Development of dolomitic land – risk management
SANS 2001-BE3	Repair of sinkholes and dolines
SANS 2001-DP8	Construction works – pipe jacking

The main task of this sub-committee was to engage the geotechnical engineering and engineering geological professions in the review, approval and publication of these standards. Seven working groups were set up to deal with each of these standards and the work was completed by the end of 2011. All these standards have now been published.

13.3.3 Restructuring of TC 59

At a meeting of TC 59 in November 2009, a task team was set up to investigate the restructuring of TC 59. For various reasons, this task team did not meet. In June 2011, it was decided that TC 59 would be split into four separate technical committees, one of which would be TC 98: Standards for Structural Design. A working group was established to determine the allocation of standards to these new TCs. The Candidate was a member of both the task team and the working group.

The working group met in August 2011 and proposed the establishment of five separate technical committees dealing with Building and Civil Engineering Practice, Design of Structures, Construction and Procurement of Buildings and Civil Engineering Works, Building and Civil Engineering Materials, and Building and Civil Engineering Products.

This proposal was partially accepted and, as of 29 August 2012, the restructured TC59 is as shown in Figure 53.

13.3.4 Establishment of TC 98: Structural and Geotechnical Design Standards

At the beginning of November 2011, the first of the new technical committees was established as *TC 98: Structural and Geotechnical Design Standards*. The first meeting of this technical committee on 10 February 2012 was attended mainly by members of the old SC 59I. The Candidate was elected chair of the technical committee, again in his absence.

Due to the limited number of members present at the inaugural meeting of TC98 in February, a further meeting was held on 31 July that was more widely constituted. All the decisions of the February meeting were ratified including the appointment of the chair.

Six sub-committees have been established to deal with various aspects of the committee's work. These are (using the new ISO committee numbering system adopted by SABS):

SABS TC98/SC 001	*Basis of Structures Design & Actions* (incl. Earthquake Design)
SABS TC98/SC 002	*Design of Concrete Structures*
SABS TC98/SC 003	*Design of Metal Structures*
SABS TC98/SC 004	*Design of Timber Structures*
SABS TC98/SC 005	*Design of Masonry Structures*
SABS TC98/SC 006	*Geotechnical Design*

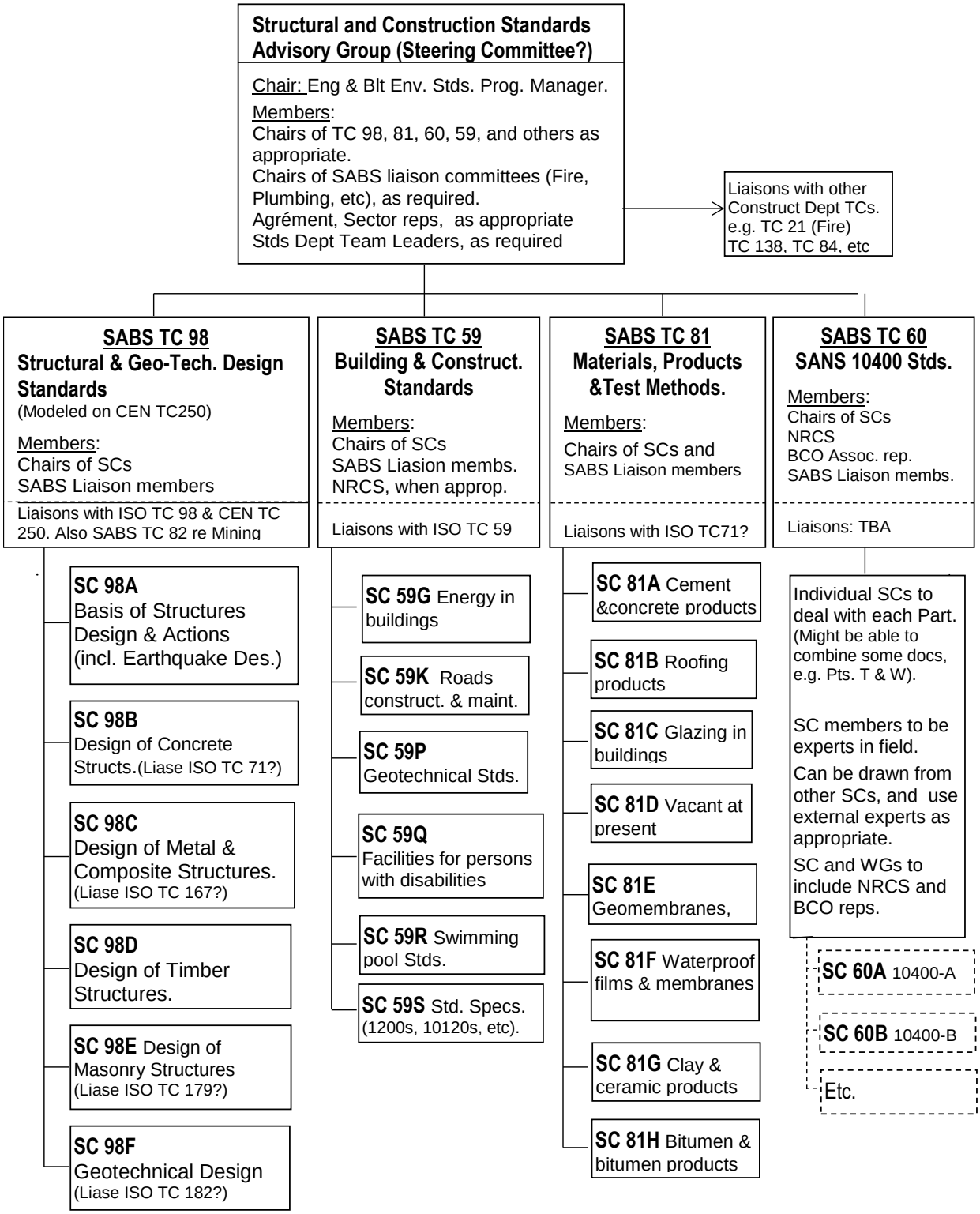

Figure 53: Restructuring of TC 59 (information from SABS)

Convenors have been appointed for these sub-committees and they are being populated at present. In the case of SC 002, the sub-committee is already at work adapting EN 1992 *Design of Concrete Structures* for use in South Africa.

The Candidate has been appointed convenor of SC 006. The first task that will be tackled by this sub-committee in 2013 is the drafting of a geotechnical design code for South Africa. This will require a decision by the Geotechnical Division whether to adopt Eurocode 7 or to write its own code.

13.4 References

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14. SANS 1936: DEVELOPMENT ON DOLOMITE LAND

14.1 Background

As indicated in Section 13.3.2 above, among the standards allocated to SC 59P were the four parts of SANS 1936. These parts deal with *general principles and requirements*, *geotechnical investigations and determinations*, *design and construction of buildings and infrastructure*, and *risk management*. SANS 1936 is referenced by the National Building Regulations and many parts of SANS 10400: *Application of the National Building Regulations* refer to SANS 1936. As such, its completion was crucial to the full implementation of the National Building Regulations.

The first drafts of the standards were produced by a relatively small group of individuals without the full participation of the geotechnical and engineering geological professions. Although drafts of all four parts were completed by mid-2008, the publication of the standards was delayed by both bureaucracy and controversy. Most of the controversy over the codes centred on three main issues (Day, 2011). These were (a) the definition of a person competent to undertake the classification and development of dolomite land, (b) the reliance on the scenario supposition method for the assessment of inherent hazard classes, and (c) the prohibition of development on certain categories of dolomite land. However, before dealing with the resolution of these controversies, the whole question of compiling codes to deal with poorly quantified risks needs to be explored.

14.2 Dealing with Poorly Quantified Risks in National Standards (Day 2011)

The quantification of risk is fundamental to assessing its acceptability (Day, 2011). Probability distribution functions for actions and resistances play a key role assessing the likelihood of failure of structures. In structural design codes, this information can be used to determine the reliability of structures allowing minimum acceptable levels of reliability to be specified. The same principles apply to codes of practice for geotechnical design, even though it may be difficult to account for large variations in material properties and for the many factors that could influence the performance of geomaterials.

The quantification of risk becomes more complicated as the number of factors that influence performance increases and as the likelihood that these will impact on the structure becomes difficult to predict. These problems are epitomised in the development of dolomite land. The main geological factors that influence the occurrence of sinkholes include the presence of cavities in the rock, rockhead topography, properties of the overburden (thickness, erodability, strength, etc.) and the past and present level of the water table. Many of these are difficult to determine even at borehole locations and their variation between boreholes can be significant. To complicate the issue even further, there is no certainty that even the most severe combination of adverse geological factors will result in subsidence in the absence of a trigger mechanism. By far the most common trigger mechanism is water ingress which is, in itself, a somewhat random occurrence.

Credit must be given to the authors of the original drafts of SANS 1936 for finding a solution to this dilemma by introducing a performance based regulatory system as described by Watermeyer et al (2008). The four levels of this system are summarised below (Day, 2011).

- Level 1 is a broad statement of the objective or goal of the regulatory system. The stated objective of SANS 1936 is to provide for the development of dolomite land in a manner that ensures that people live and work in a safe environment, damage to or

loss of assets is within limits acceptable to society, and the cost effective and sustainable use of land.

- Level 2 states the functional requirements in qualitative terms. In SANS 1936 this is that land underlain by dolomite shall present an acceptable risk of sinkhole and subsidence formation over time.
- Level 3 is the establishment of quantitative performance requirements to give effect to the functional requirement defined in Level 2. Based on the work of Buttrick et al (2001), SANS 1936 defines the tolerable hazard as one where the number of events (sinkholes or subsidences) that occur per 20 years is less than 0,1 events per hectare. The code then prescribes the permissible type and density of development and the mitigating measures to be put in place in order to achieve the tolerable hazard level.
- Level 4 specifies the method of compliance with the performance requirements. In SANS 1936 this is achieved by stipulating that development of dolomite land is to be undertaken under the control of a competent person and by laying down requirements for the investigation of dolomitic land, the design and inspection of precautionary measures and the development of dolomite risk management strategies.

The controversies surrounding SANS 1936 alluded to in Section 14.1 above arise mainly during levels 3 and 4 of the above process. (Day, 2011.)

14.3 Resolution of Controversies

14.3.1 The competent persons debate

The difficulties surrounding the definition of a competent person is part of a larger debate taking place in the engineering profession in South Africa. At the heart of the debate is the need felt by certain bodies within the profession and some of the regulators for a means of identifying truly competent persons within the profession. The Joint Structural Division of SAICE, with its links to the UK based Institution of Structural Engineers, favours the compilation of a list of peer-reviewed competent structural engineers. Passing the IStructE exams would automatically qualify a structural engineer for inclusion on that list. This proposal has been vigorously resisted by others within SAICE and by the Engineering Council of South Africa. ECSA's position is that they alone are responsible for the registration of engineering professionals in South Africa. They reject the notion of supplementary registration and, in particular, the use of a foreign organisation in the accreditation process. The geotechnical engineering fraternity do not feel the same need for a register of competent persons and have no international body similar to the IStructE that is involved in the accreditation of individual members.

ECSA, however, has challenges of its own with the definition of engineering work as is required by Section 26 of the Engineering Profession Act (Act 46, 2000). Its proposals for a generic definition have been rejected by the Council for the Built Environment. In the absence of such a definition, ECSA is unable to enforce the provisions of the Act that reserve engineering work for persons registered under the Act. ECSA has also recognised that not all registered persons are competent to undertake any type of engineering work. There is a need to ensure that registered persons possess the necessary level of education, training, experience and contextual knowledge to undertake tasks of varying levels of complexity in different disciplines (e.g. civil, mechanical, etc.) and sub-disciplines (e.g. geotechnical, structural, etc.) of engineering. To address these issues, ECSA decided in 2009 to draft codes of practice (as they are permitted to do in terms the Engineering Professions Act) starting with codes of practice for structural and geotechnical engineering.

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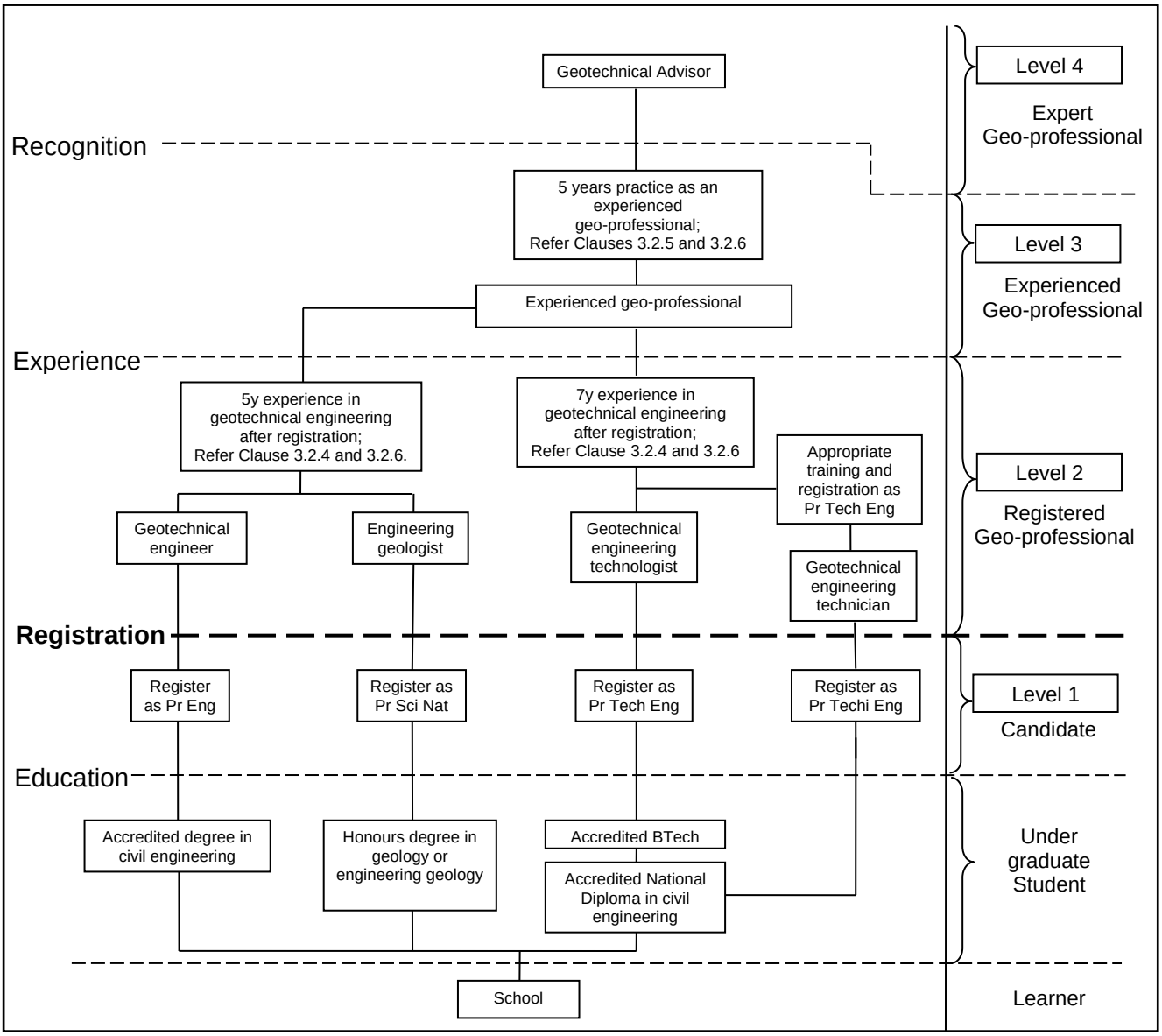
In geotechnical engineering, there is overlap between the work of geotechnical engineers and engineering geologists. There is also a need to cater for the various categories of professional registration namely professional engineer, professional technologist and professional technician. The geotechnical code working group chose to define levels of competence in the manner depicted in Figure 54, based on the approach adopted by the Site Investigation Steering Group (1993) in the United Kingdom. Then, based on the Candidate's experience with the drafting of SANS 10160-5 (see Part 3 of this dissertation), a four-fold classification of geotechnical engineering work in terms of its complexity was adopted, very similar to that which appears in Annex A of SANS 10160-5. It then became a simple matter to link the various levels of competence to the complexity of the work in order to define "who can do what".

This approach met with the approval of ECSA's Codes of Practice Steering Committee and was also adopted, with the necessary modifications, in the Structural Engineering code. Unfortunately, the codes have become bogged down in bureaucracy and have not yet been published. Nevertheless, the objective of defining not only competence but also levels of competence in a manner acceptable to the geotechnical fraternity and ESCA had been met.

The original definition of a competent person in SANS 1936-1 that sparked the controversy was based on prescribed minimum numbers of hours of experience in various aspects of geotechnical work in general and work on dolomites in particular. The definition, which among other things included 25 000 hours of experience in site investigations in partially saturated soils in South Africa, was dismissed outright by the geotechnical fraternity as it was doubtful that more than a handful of practitioners would meet these stringent criteria. In its place, a simple, generic definition was adopted which reads "*a person who is qualified by virtue of his experience, qualifications, training and in-depth contextual knowledge of development on dolomite land to*.." undertake certain defined tasks. Figure 54 was included in an Annex to the SANS 1936-1 entitled "Required competence levels for geo-professionals". This opened the way to reserving certain tasks associated with the development of dolomite land with a high inherent risk of subsidence for individuals with the required level of competence.



Notes:

1. Adapted from "Site investigation in construction series – Document 2. Planning, procurement and quality management." Recommendation from the Site Investigation Steering Group. Published by Thomas Telford, 1993.
2. The uppermost block in each level is the requirement for progression to the next level.
3. The number of years of experience is a guideline and assumes that the person is practises predominantly in geotechnical engineering during this period.

Figure 54: Competence levels for geotechnical practitioners

14.3.2 Reliance on the scenario supposition method

The original draft of SANS 1936 prescribed methods whereby the inherent hazard present on a particular tract of dolomite land should be assessed based on the Buttrick's scenario supposition method. This was not acceptable to many in the profession who saw the scenario supposition method and the interpretations that flow therefrom simply as *an* acceptable method of assessment of dolomitic terrain but not the only method. Concern was also expressed that entrenchment of the method into the code would stifle further research into alternative methods of assessment.

The viability of alternative methods of risk assessment was clearly demonstrated during planning and design of the Gautrain high speed rail link which crosses some of the most treacherous dolomite in the country. This work included gathering of data on subsidences that have occurred in the area and a statistical analysis of that data.

In the revised draft of the SANS 1936-2, the scenario supposition method has been moved to an informative annex and is regarded as a "deemed-to-satisfy" method for the assessment of dolomite land. The normative section of the code permits the use of the scenario supposition method or rational assessment by a competent person. This is in keeping with the philosophy adopted in the National Building Regulations where SANS 10400 presents deemed-to-satisfy provisions that may be used as an alternative to rational design.

The investigations described in SANS 1936-2 and the determinations in SANS 1936-1 lead to the classification of dolomite land in terms the precautionary measures required to achieve and maintain a tolerable hazard. These precautionary measures are determined by what is referred to as the Dolomite Area Designation. The main determinants of the Dolomite Area Designation are the likelihood of a sinkhole (or other subsidence) occurring, the predicted size of the sinkhole and the nature of the proposed development. The revised code permits rational assessment of both the likelihood of occurrence and the size of the sinkhole.

14.3.3 Prohibition of development of certain categories of dolomite land

The Dolomite Area Designation referred to above is a fourfold classification (D1 – D4) where no precautionary measures are required on D1 land and maximum precautions are required on D4 land. In the original draft of SANS 1936, D4 land was described as land on which "*no precautionary measures can reduce the development risk to acceptable limits so as to support development, or the required precautionary measures are impractical to implement*". This is similar wording to that used in the NHBRC Home Building Manual which is applicable to residential development. Its inclusion in SANS 1936 makes it applicable to all forms of development, not just housing. Even though the standard had not yet been finalised, this principle was adopted by many authorities to prohibit development on certain categories of dolomitic land, in much the same manner as the NHBRC had done for housing on D4 land. This led to a situation where land situated immediately adjacent to existing developments which have been in place for decades was now deemed to be unsuitable for development.

This prohibition was seen by many to be at odds with the capabilities of modern day engineering. Why should a profession that is capable of formulating solutions to a myriad of problems, from bridging the sea to the providing foundations for skyscrapers over nearly a kilometre high, be denied the opportunity of providing safe founding solutions on any type of dolomitic land? The ability of the profession to do just this has once again been demonstrated by the construction of the Gautrain. In this instance, appropriate founding solutions were devised including dynamic compaction, compaction grouting, large diameter shafts to depths in excess of 50m for the founding of some of the viaducts and providing structural solutions for the bridging of sinkholes which may occur below the railway formation. Against this background, the working group was requested to find an appropriate way to control development on such land rather than prohibiting it.

The solution adopted by the working group was, once again, aligned with the approach adopted by SANS 10160-5 where the provisions of the standard apply directly to the design of structures in geotechnical categories 1 and 2 and serve as a guide in the design of Geotechnical Category 3 or 4 structures. Category 3 and 4 structures are likely to require additional or alternative rules and provisions to those given in the standard. In the

revised version of SANS 1936-1, the description of D4 dolomite land was changed to read "*The precautionary measures required in terms of SANS 1936-3 are unlikely to result in a tolerable hazard. Site-specific precautionary measures are required*." thereby treating developments on D4 land as a Category 4 structure. The standard, however, went further to require that site characterisation and design on D4 dolomite land should be undertaken by a Competence Level 4 geo-professional, that the design and precautionary measures should specifically address and effectively mitigate the risks. Furthermore, the development proposals must be peer reviewed by an independent, similarly qualified geo-professional. These provisions are regarded as sufficiently stringent to prevent indiscriminate development of D4 land but still practical enough to permit special projects to proceed in a properly controlled manner.

The approach advocated by the revised code is in line with the process followed on the Gautrain and, even before the standard was published, its provisions were already being adopted by the Council for Geoscience for new projects on D4 land. The prohibition on housing development on D4 dolomite land as contained in the NHBRC Home building manual still stands, and correctly so in the Candidate's mind.

14.4 Candidate's Involvement

The working group on Parts 1 and 2 of SANS 1936 met regularly at the offices of the Council for Geoscience in Pretoria. There were a number of strong characters on this working group from the Geotechnical Division, SAIEG and the Council for Geoscience. Many impassioned and well-informed debates took place among its members. The Candidate attended some of its meetings, mainly at the invitation of the working group. The Candidate's main involvement was to assist in resolving deadlocks where consensus could not be obtained within the working group and by suggesting ways of resolving particularly divisive issues. In particular, he was responsible for suggesting ways of resolving the three contentious issues listed in Section 14.3 above.

Work on Parts 3 and 4 proceeded in a totally different fashion. Here, the work was done mainly by the National Department of Public Works with input from their Dolomite Working Unit which comprises representatives from the National Departments of Water Affairs and Public Works, local authorities and the Council for Geoscience. Here, the Candidate had no involvement in the initial drafting but was responsible for the review of the drafts. He then held a series of meetings with the National Department of Public Works aimed mainly at achieving consistency between all four parts of SANS 1936 and ensuring that the recommendations contained in Parts 3 and 4 were practical and achievable.

Once the Committee Drafts of all four parts had been voted on, the Candidate was responsible for editing the codes in line with the comments received and liaison with the working groups on the changes made. After receipt of public comment on the DSS documents, the Candidate was again responsible for editing the standards to respond to the issues raised by the public. Parts 1, 2 and 4 required relatively minor changes but comments regarding the ability of local authorities to comply with some the requirements of Part 3 had the potential to send the standard back to the drafting stage. The solution proposed by the Candidate was to introduce the concept of reasonable practicability similar to that used in the Occupational Health and Safety Act, and to relax some of the requirements to make them more achievable. The Department of Public Works accepted these suggestions with the knowledge that the code will have to be reviewed in three years' time in line with SABS policy.

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15. OTHER CODES AND STANDARDS

Mention has already been made in Sections 4.2 and 6.3 of the Candidates involvement with the Lateral Support Code and the code of practice on the Safety of Men in Trial Holes. Other areas where the Candidate has contributed to codes and standards are described below.

15.1 SABS SC 59P Standards

Apart from the SANS 1936, five other standards were allocated to SC 59P as listed in Section 13.3.2 above. Of these, SANS 10400-H (Foundations) and SANS 634 (Geotechnical investigations for township development) required only minor amendments which were handled by Prof Gerhard Heymann and Dr Dave Buttrick respectively. Amendments to the remaining standards are described below.

15.1.1 SANS 633

The original title of SANS 633 was *Profiling, and percussion and core borehole logging in South Africa for engineering purposes*. The feeling of the SC 59P committee was that the initial draft of SANS 633 was incomplete. In addition, it did not reflect best practice in the profession where there are already a number of authoritative references on the subject of soil and rock profiling and core logging that enjoy the same status as codes of practice. It appears that the main reason for drafting the SANS 633 standard in the first place was to provide guidance specifically for the investigation of dolomite land.

The revision of this standard was undertaken by a working group consisting mainly of SAIEG members with the Candidate representing the interests of the Geotechnical Division. The working group reduced the scope of the standard to cover only soil profiling on dolomites and percussion chip logging. Thus, rather than conflicting with the existing reference documents, the standard now complements them. The title of the standard was changed to *Soil profiling and rotary percussion borehole logging on dolomite land in Southern Africa for engineering purposes*.

The standard was approved and published in August 2012.

15.1.2 SANS 2001-BE3

SANS 2001-BE3 *Repair of sinkholes and dolines* is part of the SANS 2001 series on Construction Works. It was reviewed by a working group of dolomite practitioners led by the Candidate.

A number of changes were made to the standard particularly to the methods of sinkhole repair. The standard was also simplified by removing the description of the materials to be used in sinkhole repair, electing instead to refer to existing standards and test methods applicable to road construction materials.

This standard was approved and published in March 2012.

15.1.3 SANS 2001-DP8

SANS 2001-DP8 *Pipe Jacking* is also part of the SANS 2001 series. It was received by a working group consisting mainly of geotechnical contractors in the pipe jacking market led by Dr Nicol Chang of EsorFranki. As was to be expected, a number of practical changes were made to the standard to reflect best practice currently being applied in this field.

This was one of the first of the SC 59P standards to be completed, being approved and published in November 2011.

15.2 SANS 517 Light Steel Frame Building

A light steel frame building consists of walls, frames and roof trusses manufactured from cold-formed, light-gauge galvanised steel sections. Exterior cladding can consist of a single skin brick wall or fibre cement board, fixed to the wall frames. Services (electricity, plumbing, etc.) are installed in the wall cavity created by the light steel frames, as is the insulation material. Gypsum board, fixed to the light steel frame, is typically used for internal wall cladding and ceilings. (SASFA, 2012)

In 2006, the South African Light Steel Frame Building Association (SASFA), an organisation affiliated to the South African Institute of Steel Construction, commenced work on a *Code of Practice for Light Weight Steel Framed Buildings (LSFB)*. In 2007, they requested the Candidate to provide assistance with drafting a section of the code on foundations. In January 2007, the Candidate commenced drafting of what was to become Chapter 8 of the SASFA Code (SASFA, 2007). In 2009, this code was published as a SABS national standard *SANS 517:2009 Light Steel Frame Building*. Apart from editing to fit the SABS format, the Candidate's original work has been incorporated into this standard in totality.

The foundation section of the standard was drafted with a view to maintaining compatibility with existing South African practice particularly with regard to site classifications and foundation types. The NHBRC's Home Building Manual (NHBRC, 1999) and the SAICE code of practice (SAICE, 1995) were used as reference documents together with Australian standard AS 2870-1996 *Residential slabs and foundations – Construction*. The intention was simply to extend the specification of foundations given in the NHBRC Manual and the SAICE code to cover light-weight buildings.

Following a brief introduction, Section 8.2 of the standard deals with site investigation requirements and site classification. The site classification system used was identical to that in the NHBRC Manuals.

Section 8.3 deals with the selection of foundation types. It provides a table linking the various foundation types with the site class. Pad and pier foundations were added to the foundation types covered in the NHBRC Manual (strip footings, slab-on-ground and stiffened raft).

The next section of the standard (8.4) provides a series of standard designs intended to serve as deemed-to-satisfy requirements. In each case, typical details were provided, modified to suit the reduced weight of the structure and the construction details, particularly at the interface between the superstructure and the surface bed.

Section 8.5 deals with design by engineering principles (or rational design) which may be used as an alternative to the standard designs given in the previous sections. The requirements to be met by rational designs were laid down with particular attention to resistance to uplift and horizontal loads. The level of damage permitted and maximum permissible deflections are specified for the various types of cladding in common use.

Section 8.6 deals with site preparation and filling. It includes aspects such as clearing and shaping, stormwater drainage and termite control. Fill is divided into *controlled fill* below non load-bearing slabs and *engineered fill* below any load-bearing element of the structure. Material specifications and compaction requirements are laid down. In the case of *controlled fill*, practical guidance is given on material selection, moisture content control and compaction control to be applied where laboratory tests and field density

determinations are not undertaken. Laboratory tests and field density tests were specified as mandatory for *engineered fills*.

The final section deals with additional considerations including damp proofing, drainage precautions, avoidance of damage due to trees and articulation of structures.

The entire contribution runs into 21 pages of the SANS standard.

15.3 SAICE Site Investigation Code of Practice 2010

This was probably the most unique code writing experience in which the Candidate has participated.

The Geotechnical Division had been toying with the idea of writing a code of practice on site investigations for many years. The intention of the code was to set a minimum standard of acceptable practice for the execution of site investigations with two goals in mind. Firstly, it was to curb the number of sub-standard investigations being carried out, often using totally inappropriate methods of investigation. Secondly, it was intended to provide those responsible for procuring site investigations (clients, project managers, etc.) with guidance on how to go about this task and with a specification of the work required for various types of development. Like most codes written by volunteers, the drafting process dragged on for years. By the beginning of 2007, an initial draft had been prepared but with many gaps and incomplete sections. In the middle of 2007, a decision was made to tackle the problem head-on. The Division gathered together twelve of the most experienced geotechnical engineers and engineering geologists in the country and hired a hotel in the Mpumalanga area for the weekend. The group assembled on the Friday at noon and by Sunday afternoon all the required sections of the code had been written and reviewed.

The way in which this was done was by appointing a champion and a review group for each section of the code. Each champion was provided with the existing draft of the code and the framework of the new code beforehand. This allowed them to ensure that they had the right resources with them over the code writing weekend. The Friday afternoon commenced with a briefing session where questions such as writing style, format, contents and target audience were discussed. The champions and their assistants were then left to write their section of the code. Most of this drafting took place in the hotel's conference room where there was adequate table space available. Printing facilities were provided. By Saturday evening, the drafts of each section were handed over to the reviewers whose comments were worked into the drafts the following morning. Each section of the code was then presented to the entire group for final comment before packing up and leaving for home. The final drafts were assembled the following week leaving the document ready for editing.

Sadly, after this sterling performance, the code existed for years as an unpublished document. In was only in November 2008 that it was issued for a full peer review. It was finally published in January 2010 and is now available from the Geotechnical Division in electronic format, free of charge.

In this code, the Candidate was responsible for drafting the section on procurement of geotechnical investigations. This section dealt with the following topics.

- Budget and schedule — This section provided guidance on the time required for an investigation including procurement, mobilisation, safety and health requirements, land access, execution, laboratory testing and reporting. It also provided an indication of the likely cost of the investigation as a percentage cost of the works for various types of

	project.
• Selection of a consultant	Guidance on the types of establishments able to perform geotechnical investigations. The merits of various procurement methods (sole source, solicited proposals, open tender) were discussed and selection criteria (competence/experience, scope of services offered, contractual) were suggested. The various forms of appointment were presented with reference to available standard contract documentation.
• Remuneration of the consultant	The various methods of remunerating the consultant were discussed. These include time and cost, lump sum, lump sum with re-measurable disbursements and percentage fee.
• Liability and insurance	The source of the consultant's liability (typically the failure to exercise skill, care and diligence) and limitations of liability in terms of value and period were discussed. Options for professional indemnity cover were given.
• Enquiry document	Guidance was given on the information to be included in the enquiry document particularly with regard to the site and the nature of the development. A list of available technical specifications was provided.
• Legal requirements	The legal requirements for a site investigation as laid down in the Construction Regulations and the Housing Consumers Protection Measures Act were presented.

15.4 Forensic Geotechnical Engineering Handbook

For the past eight years, the Candidate has been a member of ISSMGE Technical Committee TC302 *Forensic Geotechnical Engineering* (Formerly TC40). One of the aims of this technical committee is to produce a Forensic Geotechnical Engineering Handbook.

The first two chapters of this handbook are the introduction and a chapter on data collection, both of which were drafted by the Candidate in 2007 and 2008. The introductory chapter deals with what constitutes failure, common causes of geotechnical failures and classification of distress. The chapter on data collection deals with the essential data to be collected relating to the nature of the works, the failure itself and site conditions. It also includes a section on the recording of data, attention to detail, agreements between parties, reporting and data storage.

The data collection section of the handbook was published at the Asian Regional Conference of the ISSMGE in December 2007 (Day, 2007).

As a result of pressure to complete SANS 10160, the Candidate has not been involved in the finalisation of this document which is due to be published before the next international conference in Paris, 2013.

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Part 5: CONCLUSION

In the final part of this narrative, I have taken the liberty of writing in the first person as much of what is presented is my personal view. In it, I draw together the common threads from the various contributions that have been described. I look back on the developments in Geotechnical Engineering over the past three or four decades not in a retrospective way, but to identify some of the challenges that still lie ahead.

The dissertation ends with some thoughts on what I would still like to achieve during the remainder of my working career.

16. SUMMING UP

16.1 Common Threads

There are some Afrikaans words for which there is no English equivalent. One such word is "samevatting". Somehow the words "summary" or "summation" do not convey the fullness of the meaning. This section is a "drawing together" of the topics dealt with in the previous parts of this dissertation, not in detail but as a general perspective.

At this stage, the reader may be asking *what are the common threads in the work presented in this dissertation*? In Section 1.5, I described the process that I have followed so often in my career as one of identifying a need, finding a solution and then sharing the outcome with others in the profession. The steps that form part of this process can best be described as follows.

The starting point is identification of an area of engineering practice where there is room for advancement. This may be a lack of knowledge or understanding of a particular topic, the need for consolidation of existing knowledge, opportunities presented by new techniques, challenges posed by changing legislation, problems requiring unconventional solutions, the need for standardisation, opportunities for gaining knowledge from abroad, etc.

The next step is one of acquiring the knowledge that is lacking. The challenge here is to keep the outcome as simple and as practical as possible. Often, this is a painstaking process and one which can be greatly assisted by discussions with colleagues in the industry.

The motivation for these two steps can best be described as curiosity. Just what it is that causes individuals to be curious has been debated for years, as can be seen from the two quotes from Johnson and Ruskin in the opening pages of this dissertation which date back to the 18th and 19th centuries respectively.

Psychologically, two types of curiosity can be distinguished (Litman and Jimerson, 2004). Curiosity can be awakened when individuals feel they are deprived of information and wish to eliminate their ignorance. Alternatively, it can be aroused when they do not feel particularly deficient of information but would nevertheless enjoy learning something new. These two situations, which are encapsulated in the Johnson and Ruskin quotes, are known as curiosity as a feeling-of-deprivation (CFD) and curiosity as a feeling-of-interest (CFI). The former is a "need to know" experience in which feelings of tension precede pleasurable satisfaction. The latter as a "nice to know" approach to acquiring new information.

According to Litman (2005), curiosity can be classified in terms of high and low levels of *wanting* and *liking*. *Wanting* is the product of deprivation and the anticipation of the satisfaction of one's desires based on experience of similar situations. *Liking* is somewhat more complex, and may be influenced by the extent of any preceding *wanting* (e.g. weak or strong desire) and specific pleasant stimuli, such as sweetness. Litman's classification is given in Table 22.

Table 22: Classification of various types of curiosity (Litman, 2005)

		Wanting	
		Low Level	High Level
Liking	Low Level	Ambivalent disinterest or boredom (Spontaneous alternation or novelty seeking).	Need for clarification of uncertainty. (Need for cognitive closure, morbid or lurid curiosity)
	High Level	CFI: curiosity as a feeling-of-interest. "Nice to know." (Aesthetic appreciation)	CFD: curiosity as a feeling-of-deprivation. "Need to know." (Perceptual/conceptual/fluency)

Some engineers are content with simply finding a solution to the problem at hand, for example grasping onto the first equation that they can find that will yield the desired answer using the input parameters available to them. This can be equated low levels of both *wanting* and *liking*. Curiosity as a feeling-of-interest (CFI) classifies as low levels of *wanting* and high levels of *liking*. This is the engineer who appreciates the acquisition knowledge but does not feel compelled to pursue it. Curiosity as a feeling-of-deprivation (CFD), on the other hand, corresponds to high levels of *wanting* and high levels of *liking*. This is the engineer who is acutely aware of deficiencies in his or her own understanding, who feels the need to fill these gaps and derives satisfaction from the outcome. According to Litman et al (2005), the intensity of the CFD state is heightened when individuals have a strong feeling-of-knowing, i.e. the knowledge they seek is felt to be just beyond their grasp. In engineering terms, this would correspond to the engineer who is challenging the limits of what is known but is convinced that the knowledge or solution that is desired is within reach. This does not necessarily apply to new knowledge only, but also to information that may be known to others but not yet acquired by the individual.

As far as my engineering career is concerned, I find it very easy to associate with the "need to know" (CFD) type of curiosity. So often, my curiosity is aroused because I do not have the answer but am sure that it is within reach, and the tension of the quest becomes tangible. This is the high level of *wanting*. Once a solution is found, *liking* kicks in. There is much pleasure to be found in an elegant solution to a problem or finding consistency in knowledge already acquired.

The next step in the process is sharing the newly gained knowledge. Here, there are a number of factors that come into play. Firstly, the sheer joy of finding a solution demands that it be shared. The second is the confidence that whatever is given away will be returned with interest. Finally, there is again a feeling of tension or anxiety that if the new-found knowledge is not written down it will be forgotten.

Like teaching, writing of codes (or standards) is merely an extension of the sharing of knowledge. My perception of codes is well aligned with the legal status of most South African standards, namely that they are statements of good practice rather than legally binding requirements. I see codes as a framework for the well-considered application of knowledge, not a set of rules that are to be obeyed without thinking[22]. The requirements for code writing have much in common with the requirements for teaching or lecturing. Both require a thorough understanding of the subject, of the rules and the exceptions to the rules. Both require the ability to convey complex information is a simple way. Both

[22] See comments in Sections 14.3.2 and 17.2 on the dangers of overly prescriptive standards.

require accuracy and clarity of presentation. Finally, both are subject to scrutiny at the time of presentation and during future application.

16.2 A Favourable Environment

Geotechnical engineering, particularly in South Africa, provides adequate opportunities for indulging one's curiosity, some of which have been mentioned before. These include:

- There are many areas of geotechnical engineering where we do not have all the answers.
- The environment in which geotechnical engineers operate is generally receptive to innovative solutions to problems.
- The industry is not overly regulated or bound by convention.
- There a number of contractors and consultants who have worked together for years and have grown to trust one another.
- Solutions are often available that are not difficult or costly to implement.

The environment, at least among consultants, is also conducive to the sharing of knowledge. We operate in a relatively small community and many of us have our own areas of specialisation. Until recently, the availability of work has led to a spirit of cooperation rather than fighting for market share. The Geotechnical Division and SAIEG both provide opportunities for presenting new ideas and fresh insights by means of seminars, publications and evening lectures.

I have been particularly privileged to work with a company that has a very practical approach to engineering. It has a firm commitment to involvement in the profession and to continuing education. It has also supported all my "extra-mural" activities such as serving on various committees and writing standards.

All of this has contributed to what I referred to in the opening section of this dissertation as the "allure of geotechnical engineering".

16.3 Some "Fatherly Advice" for Young Engineers

If I were to offer advice to young engineers in the profession, I could do no better than to reiterate the advice I gave during my address to Engineering Graduates at Stellenbosch University in December 2008, which was:

- be confident – the future lies in your hands;
- be curious – keep acquiring knowledge throughout your career;
- be honest – both in your engineering and in your dealings with people;
- be humble – do not be afraid to ask for advice;
- be generous with your time and your knowledge;
- be inspired both by what you do and by the achievements of those around you; and
- be careful, particularly in looking after connections, in engineering and in your life.

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17. FOLLOW-UP WORK

17.1 Development on Dolomite

The publication of SANS 1936 standard (Development on Dolomite Land) in October 2012 was a vital step. The delay in publication of this standard to permit a full review by SC59P stood in the way of full implementation of the National Building Regulations. However the publication of the standard is not seen as an end in itself but as a stepping stone for further work on the development of dolomite land in the future.

Towards the end of the working group's deliberations on the contents of the new code, it became clear that various working group members believed that rational analysis of the risk of subsidence is the preferred route. Such analysis would be based on a geotechnical model of the site and on an examination of the record of subsidence both in the area and in similar geological / geotechnical situations. The statistical basis for the definition of low, medium and high risk was also being questioned[23], the feeling being that the definition of a tolerable hazard was not sufficiently stringent. Rather than delay the publication of the code to debate these issues further, it was decided that SAIEG and the Geotechnical Division should continue to research and debate these issues in preparation for the first revision of the code in three to five years' time. This message was conveyed to the Geotechnical Division AGM in November 2012. Hopefully it will become a priority of the re-constituted SC59P (Geotechnical Construction Standards) which will be under new chairmanship from the beginning of next year.

One of the benefits of allowing rational assessment and design as an alternative to the prescriptive measures of the early drafts of the code is that this will encourage further research into methods of investigation, assessment, design and soil improvement techniques on dolomite. Already, we are seeing an increased interest in research into the mechanisms of sinkhole formation from the research group headed by Professor Jacobsz at the University or Pretoria. Plans are underway to use the recently commissioned geotechnical centrifuge at the university for this purpose. This was illustrated by the risky decision to conduct a live demonstration of the formation of a sinkhole at the official opening of the centrifuge in front of about 100 invited guests and university dignitaries on 13 June 2012.

In my opinion, the database of sinkholes compiled by the Council for Geoscience (CGS) presents enormous opportunities for research on the subject of dolomite related subsidence. Their current database, which contains about 2 000 entries mainly from the south of Pretoria, is currently being augmented by data from other databases. SANS 1936-3 now requires the reporting of sinkholes to the CGS in a prescribed format. This database and information from the Department of Public Works should be made freely available to researchers and the results of such research should be published. This would go a long way to dispelling some of the myths about dolomite such as the perceived dangers of developing on wad and the high risk of sinkholes posed by shallow pinnacle dolomite profiles (A'Bear and Richter, 2011).

[23] In SANS 1936-1, a tolerable hazard is one where the number of events experienced is less than 0,1 events per hectare per 20 years (preferably tending to zero per hectare), i.e. a return period of an event occurring on 1 ha of more than 200 years. This is regarded as a "low" inherent hazard. A medium inherent hazard is a return period of 20 – 200 years and a high hazard equates to a return period of less than 20 years.

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17.2 Expansive Soils

There can be no doubt that the 1960's – 1980's were the heydays of South African research into problem soils in general and heaving clay in particular. Much of this work was spearheaded by the National Building Research Institute at the CSIR. Internationally, the worldwide interest in the subject was reflected by the activities of the ISSMGE Technical Committee on expansive soils (TC6) on which I served in the early 1990's. Since then, much has changed. The NBRI has ceased to exist in its original format. The ISSMGE's technical committee on expansive soils has been disbanded.

Should this be seen as a sign that the challenges surrounding expansive clays have been effectively dealt with? My opinion is that we have the tools to address many of the issues surrounding expansive soils including predication of heave, design of foundations and protection of the homeowner. However, these tools are not being used effectively. This is clear from the number of professional indemnity claims being lodged against engineers for heave damage and of cases brought before the ECSA Investigating Committee relating to the cracking of buildings.

To improve this situation, there are a number of actions that need to be taken by the industry:

- the standard of laboratory testing needs to be improved, even if this does result in an increase in the cost of testing;
- available methods of heave prediction that take account of some of the fundamental factors that influence the behaviour of the soil but are ignored by the Van der Merwe method need to be re-introduced into common use;
- measurement of soil suction should be revived as a basic field test for the prediction of heave movements; and
- designers should once again be made aware of the fundamental principles involved in the design of structures that will perform acceptably on expansive soil profiles.

This is another instance where I believe the introduction of prescriptive measures such as those contained in the NHBRC Home Building Manual has the unintended consequence that geotechnical investigators and foundation designers do not apply their minds to the problem in the same way as we did in the past. There seems to be a perception that application of the rules will achieve an acceptable outcome and that little else needs to be done.

17.3 Lateral Support in Surface Excavations

The existing code of practice on lateral support in surface excavations has served us well for close on 25 years. However, it was written at a time when limit states design in geotechnical engineering had not even been considered for use in South Africa and when soil nails were still a novelty.

At the Geotechnical Division AGM in November 2012, it was announced that a committee is to be formed to look at revising certain sections of the code. These will include the sections on design of soil nails, testing of soil nails and anchors and case histories of wall movements (Appendix F) . It was also suggested that the review should take account of the provisions of overseas codes such as BS 8006-2 and BS 8081.

The possibility of re-writing the sections on design of lateral support in a limit states format is also being considered. However, it appears that use of limit states design for soil nailed walls is somewhat problematic and that CEN TC250 SC7, the Eurocode 7 sub-committee, is actively looking at this situation at present (Bond, 2012).

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17.4 Pile Design and Construction Practice

Since the withdrawal of SABS 088:1972 on piled foundations, South Africa does not have a pile design code. The only piling code still in existence is SABS 1200F. At a meeting called by the SABS in July 2012 (SABS, 2012), it was resolved that the SABS 1200 series of standards would be phased out and replaced by the SANS 2001 (Construction Works) series. To date, no SANS 2001 standard has been written for piling.

Following the publication of the initial batch of standards allocated to SC 59P, it is my intention to resign as chairman of this committee. The recommended candidate for this position is a member of the piling fraternity. Hopefully, the new chairman of SC 59P will provide the impetus to proceed with the drafting a new piling code.

See also Section 17.6 below.

17.5 Soil Profiles not Amenable to Small Scale Testing

In November 2012, I presented a keynote address to the 3rd African Young Geotechnical Engineers Conference in Cairo on large scale field tests. The presentation stood in sharp contrast to the many highly-theoretical papers presented by the young engineers and drew a lot of attention. It is clear that there is a place for such tests in the geotechnical industry.

There are two exciting developments taking place at present. The first is that the instrumented backfill at Grootegeluk Mine (see Section 7.4) continues to produce valuable data. Construction on top of the fill has now commenced and surface monitoring points have been established to supplement the data being obtained from the hydro-profiler at depth within the fill. To date, the period of monitoring is too short to draw any meaningful conclusions regarding the rate of creep settlement. Hopefully this will improve during the next twelve months when the effect of the rate of placement of the fill becomes less marked and the predicted creep rate becomes more linear on a log-time scale.

The second development is our appointment by Eskom to investigate the effect that settlement of backfill in a pit at Kriel Mine will have on the performance of a proposed ash dam to be constructed partly on the backfilled pit. This investigation will include the construction of a 20m high trial embankment on the fill. The intention is to install settlement monitoring devices at various depths within the existing fill in addition to carrying out further hydro-profiler measurements on the top of the fill below the trial embankment. There are also plans to saturate the fill once the embankment is complete to simulate and measure the effects of collapse settlement.

The observations from these two projects will add considerably to our knowledge on the long term (creep and collapse) settlement of pit backfill.

17.6 Limit States Design in Geotechnical Engineering

The publication of SANS 10160-5 in 2010 marked the end of an 18 year “journey” from the time the Geotechnical Division took the decision in 1993 not to write a geotechnical design code but to follow the development of the Eurocodes. I have been privileged to be involved in every step of that journey including the 1995 Limit States Design Symposium, the 1998 National Loading Conference, participation in the ISSMGE Limit States Design technical committee, the development of a new loading code and finally the drafting and publication of SANS 10160-5 *Basis of Geotechnical Design and Actions*. One of the

Division's intermediate goals has been achieved, namely providing the means whereby South African geotechnical engineers can use limit states design on a routine basis and we can, at last, harmonise our designs with those of our structural colleagues.

There are, however, many further goals that I hope to achieve. The first is to facilitate a decision by South African geotechnical designers whether to adopt EN1997 or to write a South African geotechnical design code modelled on the Eurocode. The Geotechnical Division AGM in November 2012 was reminded of their commitment to make such a decision in 2013. The second is to encourage further code calibration studies such as the work by Dithinde (2007) on model uncertainty factors for piles.

I get the sense from recent comments by piling contractors that the publication of SANS 10160-5 and the likely withdrawal of SABS 1200:F has re-kindled the idea of writing a South African piling code which was, as will be recalled from Section 9.1, what started the ball rolling in 1993. In my mind, two codes are required rather than one. The first should be an addition to the SANS 2001 (Construction Works) series and should deal with the practical aspects of pile installation including issues such as concrete placement methods. The second should be a South Africa pile design code or a section of the South African geotechnical design code dealing specifically with pile design. The former will probably be the responsibility of SC59P (Geotechnical Construction Standards) while the latter should be undertaken by TC98/SC 006 (Geotechnical Design Standards).

17.7 Other Codes of Practice

Mention has already been made of possible further developments regarding the codes of practice on lateral support and development of dolomite land.

With the trend towards continued harmonisation with international standards, there is also scope for combining the various "informal" standards on soil and rock profiling and logging and the newly published SANS 633 into a single document. Such a document should preferably be compiled as a joint effort between SAIEG and the Geotechnical Division. The compilers of the document should consider taking the best of South African practice and aligning it with ISO 14688:2004 and ISO 14689:2003.

In the latter half of 2012, there have been some interesting developments surrounding the SAICE Site Investigation Code of Practice. As indicated in 15.3, one of the reasons for introducing this code was to provide guidance to those responsible for procuring site investigations on the required scope of the investigation for a particular type of project. Unfortunately, the code has failed to meet this expectation. Many of the investigations being carried out at present, particularly for commercial and residential developments, fall well short of the requirements of the code. This is partly due to the competitive nature of this sector of the industry and the failure to adequately specify the requirements of the investigation.

That could all change if the NHBRC continues on its current path. In October 2012, the NHBRC refused to enrol a multi-storey residential complex being developed by an international investor on the grounds that the geotechnical investigation failed to meet the requirements of the code. My advice to the developers was that the NHBRC were fully within their rights, their concerns were justified and that the best action would be to carry out the additional investigation rather than challenging the ruling. The additional investigation was carried out within a period of 7 days and approval by the NHBRC was obtained. The delay, however, is reported to have cost the developer an estimated R150 000 per day. I fully support the stance adopted by the NHBRC. The requirements of standards should only be circumvented in cases where there are sound reasons for

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doing so and not as a result of commercial pressures combined with inadequate specification of investigation requirements.

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18. PLANS FOR THE FUTURE

18.1 SABS TC98

The first challenge that lies ahead is the work of SABS TC98. One aspect of this challenge is chairing this new technical committee at a time when many new design codes are being developed. The second aspect centres on the writing of a South African Geotechnical Design code by the TC98/SC 006 subcommittee. Whatever the Geotechnical Division decides regarding the adoption of EN1997 or writing a South African geotechnical design code, there will be lots of exciting work to be done. If the decision is to adopt EN1997, the emphasis will be on the compilation of a South African National Annex. Writing a South African geotechnical design code will present far wider challenges.

A South African geotechnical design code will have to deal with aspects not adequately covered by Eurocode 7 such as problems associated with partially saturated soils including heave and collapse. It should draw not only on the Eurocodes but also on developments in Australia and Hong Kong in particular, countries that find themselves in much the same situation as South Africa.

Whichever of the two options is selected, it will be necessary to revise SANS 10160-5 and remove the geotechnical design aspects from this part of the standard.

18.2 Work with ECSA

The second area in which I would like to continue working is with the Engineering Council of South Africa. I have served on the Investigating Committee of ECSA since 2006. During this period, I have headed the task team entrusted with defining roles of Assessors and Experts on the committee and with revising the Code of Conduct for Registered Persons. I also served on the committee for the drafting of ECSA codes of practice.

There are two items in particular that I would like to see through to conclusion. The first is the completion of the Codes of Practice, where initial drafts have already been produced for Geotechnical and Structural Engineering. These codes have been “put on the back burner” for the time being with the intention of developing an umbrella code covering all sub-disciplines of civil engineering. The sub-discipline codes (e.g. geotechnical and structural engineering) will then become annexes to the umbrella code.

The second item is the resolution of the deadlock between the Engineering Council and the Council for the Built Environment (CBE) regarding the Identification of Engineering Work. In terms of Section 26 of the Engineering Professions Act, ECSA is obliged to identify the type of engineering work reserved for professionals registered in terms of the Act and the overlaps with other professions, and to submit recommendations to the CBE in this regard. In order to accommodate the wide variety of engineering work and the number of disciplines involved (e.g. civil, electrical, mechanical, etc.), ECSA has chosen to define engineering work in generic terms based on the characteristics of the work, the type of work, the functions performed and the competencies required (ECSA, 2005). The CBE has rejected this approach and is insisting on a detailed itemisation of engineering work in all disciplines. This has led to a deadlock between ECSA, the CBE and other professions registered under the “Professions Acts” (Project Managers, Architects, Quantity Surveyors, etc.). Certain of these professions are now laying claim to aspects of engineering work which engineers have performed for years.

In an attempt to resolve this deadlock, I was invited by ECSA to serve on an "overlaps committee" that would attempt to define the issues and resolve the impasse. The first thing that became apparent was that the other professions are all "single discipline" professions and are project-orientated. So much was evident from their approach to the definition of tasks, which was clearly associated with various phases of a project from inception to close-out. There are, however, many aspects of engineering work that are not project-orientated, for example environmental studies, geotechnical investigations, infrastructure maintenance, etc. If the scope of the overlaps committee can simply be limited to project work in the context of the built environment, the solution becomes relatively simple. However, this alone will not solve the problems associated with the identification of engineering work in general, as required by the Act. Much additional work is required and I am keen to contribute where I am able.

18.3 Work with SAICE

An area in which I would like to carry on working is the Journal. I have been a member of the Journal Editorial Panel for the past 10 years. During this time, the journal has changed significantly and is now listed with the Institute for Scientific Information (ISI). This has a significant impact on the credibility of the Journal and the willingness of South African academics to publish their work in the Journal. I appreciate all the hard work by a dedicated team of SAICE employees that goes on behind the scenes to ensure that the Journal is produced on time. I would also like to carry on working as an assessor and referee of papers submitted for publication.

Another area of SAICE's work that I wish to continue supporting is the adjudication of the Awards for Excellence where I have served as chairman of the adjudication panel in recent years.

18.4 Work with the Universities

The two year period in my early career spent lecturing at the University of Natal was one of the most fulfilling times of my life. There is no doubt that this set me on the path that I have tried to describe in this dissertation, one of seeking to understand a problem and then communicating that understanding to others.

My work with the Universities has continued in a number of forms. I serve on the Civil Engineering Advisory Committee of the University of Pretoria and have, in the past, filled the same role at the University of the Witwatersrand. I am an external examiner for undergraduate and post graduate students at these universities and deliver occasional lectures to post graduate students at the universities of Stellenbosch, Witwatersrand, Pretoria and KwaZulu-Natal. As a company, we assist the universities with "real life" data for final year design projects and we now run the SAICE "100 for 100" bursary scheme launched as part of their centenary celebrations from our office. We also donated the structural design of the containment structure for the newly constructed centrifuge at the University of Pretoria. This, together with generous construction support from various civil engineering companies enabled the university to buy a bigger centrifuge that they would otherwise have been able to afford.

I would like to continue my work with the universities, particularly in the field of post graduate research. At present, we are working with the University of Pretoria on suggesting and refining research proposals for centrifuge experiments, particularly in the field of dolomite related subsidence. I have also been in discussion with the University of Johannesburg on the possibility of two Masters theses on the design of concrete block

retaining structures and on the quality of laboratory testing data from South African geotechnical laboratories.

18.5 Maintaining a Balance

Probably the biggest single challenge that I will face in the future is maintaining a balance in life. Retirement is something that people plan for and look forward to. Sadly, many couples do not live to enjoy the fruits of their labours together.

In the address I gave to the Engineering Graduates at Stellenbosch University in December 2008, I pointed out that great care is required in the design and maintenance of connections. This is because they play a critical role in ensuring the functionality of the product and the fact that they are often brittle and fail without warning. Examples were given of the Large Hadron Collider (September 2008), the Boston Bridge (August 2007) and the Nicol Highway excavation (April 2004), all of which failed as a result of defects in connections. Similarly in life, one needs to pay attention to the connections with one's community, friends and family. I would be wise to heed my own advice.

18.6 Reference

ECSA (2005) Framework for the Identification of Work for Persons Registered in terms of the Engineering Profession Act, 2000. IoEW Steering Committee, Nov. 2005.

Stellenbosch University http://scholar.sun.ac.za

Appendix A

ABRIDGED CURRICULUM VITAE

ABRIDGED CURRICULUM VITAE

PETER DAY

Date of Birth:	2 May 1953
Nationality:	South African
Education / Qualifications:	BSc Eng (Civil) *Cum Laude* (University of Natal, 1975); MSc Eng (University of Natal, 1980).
Family:	Married to a farmer / microbiologist. Two sons (a civil engineer and a wine maker).
Employers:	
1976	Department of Water Affairs – Natal
1977 – 1978	University of Natal – Researcher, Junior Lecturer
1979 – Present	Jones & Wagener Consulting Engineers, Geotechnical Engineer and Chairman of the Board.
Awards:	
2010	Honorary Fellow of the SA Institution of Civil Engineering.
2005	South African Geotechnical Medal.
2002	Fellow of the SA Institution of Civil Engineering.
1991	J.E. Jennings Award (SAICE Geotechnical Division) for contribution to the 1989 Code of Practice on Lateral Support in Surface Excavations.
1981	SAICE prize for Best Short Paper or Technical Note published in the Transactions during 1980.
1981	SAICE prize for Best Graduate Contribution published in the Transactions during 1980.
Professional Affiliations:	
SAICE	Member (1982), Fellow (2002), Honorary Fellow (2010)
ECSA	Professional Engineer (1982)
	Member on Investigating Committee (2006-). Code of Practice Steering Committee (2009-). Identification of Work Overlaps Committee (2012-)
ISSMGE	Vice-President for Africa (2001-2005).
	Chairman of Technical Committee TC23 on Limit State Design (1997-2001).
	Member of Technical Committees on Professional Practice, Limit States Design, Underground Excavations; Forensic Geotechnical Engineering.

Geotechnical Division	Secretary (1988-1992). Chairman (1993-1994). Treasurer (1995-2003).
SABS	Chair, Subcommittee SC59P: Geotechnical Codes (2009-). Chair, Technical Committee TC98, Structural & Geotechnical Design Standards (2012-). Chair, Subcommittee SC98F: Geotechnical Design (2012-)
SAICE & Geotechnical Division	Chairman of organising committee: • Pile Design and Construction Practice (1988) • *XII* ISSMGE African Regional Conference (1999) • Seminar on Limit State Design in Geotech Engineering (1995) • S.A. National Conference on Loading (1998).
Cement and Concrete Institute	Alternate Board Member representing the Built Environment Professions.

Areas of Expertise:

Geotechnical investigation; lateral support and slope stability; construction on problem soils; design of foundations; insurance and contract dispute investigations; drafting codes of practice.

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Appendix B

LIST OF PUBLICATIONS

PETER DAY
INDEX OF PUBLICATIONS

Listed by Year

PAPER 57	Fanourakis G.C. Day P.W. Grieve G.R.H	Oct 2012	The effects of placement conditions on the quality of concrete in large-diameter bored piles. Journal of the South African Institution of Civil Engineering, Volume 54, Number 2, October 2012
PAPER 58	Day P.W.	Nov 2011	SAICE's interaction with the South African Bureau of Standards. Civil Engineering, November 2011. SAICE, Midrand. pp60-61.
PAPER 54	Fanourakis G.C. Day P.W. Grieve G.R.H	October 2011	The effect of site practices on the integrity of large diameter bored piles. 14th Pan-American Conference on Soil Mechanics and Geotechnical Engineering and 64th Canadian Geotechnical Conference, Toronto, 2-6 October 2011.
PAPER 55	Breyl J. Wardle G. Day P.	July 2011	Theoretical evaluation of the influence of cohesion on lateral support design. 15th African Regional Conference on Soil Mechanics and Foundation Engineering, Maputo. 18-21 July 2011.
PAPER 53	Day, P.W.	June 2011	Managing poorly quantified risks by means of national standards with specific reference to dolomitic ground. International Symposium on Geotechnical Safety and Risk. Munich, 2 & 3 June 2011.
PAPER 52	Day, P.W. Jaros, M	June 2010	Application of Eurocode 7 to the geotechnical design of a stadium roof support arch in South Africa. XIVth Danube-European Conference on Geotechnical Engineering. "From research to design in European practice". 2 – 4 June 2010, Bratislava, Slovak Republic
PAPER 45	Schuppener, B. Bond, A.J. Day, P.W. Frank, R. Orr, T.L.L. Scarpelli, G. Simpson, B.	March 2009	Eurocode 7 for geotechnical design – a model code for non-EU countries? 17th International Conference Soil Mechanics and Geotechnical Engineering, Alexandria, Egypt. October 2009.
PAPER 48	Day, P.W. Retief, J.V. Dunaiski, P.E.	2008	An Overview of the Revision of the South African Loading Code SANS 10160. Chapt. 1.1. Inst. For Structural Engineering, Stellenbosch Univ., S. Africa.
PAPER 47	Day, P.W. Retief J.V.	2008	Part 5: Basis of Geotechnical Design and Actions. Background Report to SANS 10160, Chapt. 5.1. Inst. For Structural Engineering, Stellenbosch Univ., S. Africa.
PAPER 44	Day, Peter Jacobsz, Sw	Apr 2008	Are we getting what we pay for from geotechnical laboratories? Civil Engineering, April 2008, Vol. 16 No. 4.

PAPER 46	Day, Peter	2007	Krebs Ovesen's Legacy to South Africa: A harmonized basis of design code. Spirit of Krebs Ovesen Session – Challenges in Geotechnical Engineering. XIV European Conference on Soil Mechanics and Geotechnical Engineering, Madrid 2007.
PAPER 43	Day, Peter	Dec 2007	Forensic Geotechnical Engineering Investigations: Data Collection. TC40 Workshop on Forensic Engineering. 13th Asian Regional Conference ISSMGE, Kolcuta, Dec 2007.
PAPER 32	Day, P.W.	March 2007	The effect of Free Fall Placement of Concrete in Large Diameter Bored Piles. SAICE Geotech Div. Pile Design & Construction Practice 6-7 March 2007, Midrand.
PAPER 59	Day, Peter	2006	An Engineering Perspective on Brink's Engineering Geology of Southern Africa. Spine of a Dragon, Contributions on ABA Brink (1927-2003) – South Africa's pioneer of engineering geology. Kleio Publishers, Johannesburg. Chapter 9, pp 127-133.
PAPER 29	Day P.W.	2006	Geotechnical Engineers and the Construction Regulations. Journal of SAICE, Volume 48, No. 4, December 2006.
PAPER 30	Vermeulen N.J. Day P.W.	2005	Dewatering at the Port of Ngqura: A case study. Proc 16th ICSMGE, Osaka, Japan.
PAPER 17	Day P.W.	2005	Issue 1: Long term settlement of granular fills. Academic-Practitioner Forum, Proc 16th ICSMGE, Osaka, Japan.
PAPER 17	Day P.W.	2005	Issue 2: Development of Advanced Constitutive Models – should these proceed? Academic-Practitioner Forum, Proc 16th ICSMGE, Osaka, Japan.
PAPER 50	Day P.W.	Oct 2004	South African Code of Practice. Symposium on Earth Pressures and Retaining Structures, SAICE Geotechnical Division. October 2004, Pretoria.
PAPER 25	Wardle G.R. Day P.W.	Dec 2003	Settlement predictions and monitoring of heavy industrial structures at Saldanha Bay, Cape West Coast, South Africa. 13th African Regional Conf. on Soil Mech. and Geotech. Eng., Morroco.
PAPER 22	Orr, T.L.L. Matsui, K. Day,P.W.	Apr 2002	International practice with regard to geotechnical investigation methods and the determination of parameter values. IWS Kamakura, Japan.
PAPER 36	Day,P.W. Wardle, G.R. Modishane, T.L.	Oct 2001	Pre-loading to reduce settlement below heavy industrial structures. SAICE Ground Improvement Seminar, Midrand.
PAPER 35	Day P.W.	June 2001	Case Histories and Lessons from Failures. Zimbabwe Institute of Engineers Congress, Harare.
PAPER 56	Day P.W. Wardle, G.R. V.D. Berg J.P.	Nov 2000	National report on limit states design in geotechnical engineering: South Africa. International workshop on limit states design in geotechnical engineering, Melbourne Australia, 18 November 2000. ISSMGE Technical Committee TC23.

Compact Disc	Krebs Ovesen, N. Day, P.W. (Editors)	Nov 2000	LSD 2000 Proceedings of International Workshop on Limit State Design in Geotechnical Engineering. Melbourne, Australia.
PAPER 10	Day, P.W.	1999	Braced excavation – deformation and displacement of walls. Proceedings of International Symposium on geotechnical aspects of underground construction in soft ground (IS Tokyo 99), General Report, Session 4, p43 – 52. Balkema, Rotterdam.
PAPER 19	Brink, D. Day, P.W. Du Preez, L.	Oct 1999	Failure and remediation of BulBul Drive Landfill: KwaZulu-Natal, South Africa. 7th International Symposium on Waste Management and Landfill, Sardinia.
PAPER 21	Day, P.W. Kemp, A.	1998	South African Loading Code: Past, Present and Future. Civil Engineering, Volume 7, No 4. SAICE.
PAPER 26	Day, P.W.	1998	Interface Testing of Geomembranes and its application to Landfill Design. SAICE Geotechnical Division / SAIEG 1999 Lecture Course.
PAPER 16	Day, P.W.	Sept 1997	Limit State Design – A South African Perspective. Discussion Session 2.3, XIV International Conference on Soil Mechanics and Foundation Engineering, Hamburg.
PAPER 5	Day, P.W. Wardle, G.	Oct 1996	Effect of Water on Settlement of Opencast Pit Backfill: Case Histories of Investigation and Performance. SAICE Seminar on Hydrology of Made Ground, Johannesburg.
PAPER 8	Day, P.W.	Oct 1996	Geotechnical Engineers and the Occupational Health and Safety Act. SAICE Journal, Volume 38, No. 3.
PAPER 13	Day, P.W. Harbuz, H.	Jun 1996	Foundation Investigation, Selection and Design. Symposium of Industrial Civil Engineering, Rand Afrikaans University, Johannesburg.
PAPER 15	Day, P.W.	Sept 1996	Implementation of Eurocode 7 in South Africa. Eurocode – Towards Implementation, Institution of Structural Engineers, London.
PAPER 27	Day, P.W. Wardle, G.R. Krone, B	Dec 1995	Design, Construction and Performance of Deep Basement Excavations in South Africa and Zimbabwe. Eleventh African Regional Conference on Soil Mechanics and Foundation Engineering, Cairo.
PAPER 51	Wagener, F. Day, P.W.	1995	Chapter 7: Site investigation and geotechnical information. Construction Law and Related Matters. Ed Philip C Loots. Juta & Co, Cape Town. pp 241 - 256.
PAPER 7	Day, P.W.	Mar 1994	Safety of Men Working in Testpits and Auger Holes. Requirements of Occupational Health and Safety Act (85, 1993). Safety and Health in Industry, Construction and Mining. The New Legislation. LGI, University of Pretoria. Bifsa Conference Centre, Midrand.
PAPER 14	Day, P.W.	Jan 1994	Factors Influencing the Movement of Retaining Structures. Plenary Session C, 13th International Conference on Soil Mechanics and Foundation Engineering, New Delhi.
PAPER 4	Howie, C.T; Scheele, F; Day, P.W.	Jan 1994	Soldier Pile Analysis using Non-linear Beam-Foundation Theory. Proceedings 13th International Conference on Soil Mechanics and Foundation Engineering, New Delhi,

			Volume 4, 1937 – 1402.
PAPER 6	Day, P.W.; Schwartz K	Jan 1994	National Report on Codes of Practice and Authoritative Reports on Braced Excavations in Soft Ground, South Africa, Africa Region. International Symposium on Underground Construction in Soft Ground, New Delhi.
PAPER 33	Day, P.W.	May 1992	Determination of Settlement Parameters for Opencast Pit Backfill by means of Large Scale Tests. Proc. Conf. on Construction Over Mined Areas. SAICE, SANGROM, SAIEG, AEG.
PAPER 42	Day, P.W.	Aug 1990	Site Investigations for River Bridges. 10th Annual Transportation Convention, Pretoria.
PAPER 3	Day, P.W.	Aug 1990	Observed Movement and Remedial Measures to Northern Face of an Excavation in Jeppe Street, Johannesburg. SAICE Lateral Support Code Launching Conference, Johannesburg.
PAPER 9	Day, P.W.	Jun 1990	Design and Construction of a Deep Basement in Soft Residual Soil. Proc ASCE Speciality Conference on Design and Performance of Earth Retaining Structures, Cornell University, Ithaca NY.
Published by SAICE as a book.	Saice Geotechnical Division Subcommittee	1989	Code of Practice, Lateral Support in Surface Excavations. SAICE, Geotechnical Division, 1989. Awarded J.E. Jennings award for best SA geotechnical publication in 1991.
PAPER 1	Day, P.W.	Nov 1989	Design of Raft Foundations (using Lytton's Method). Proc. Course on Design of Stiffened Raft Foundations, CSIR, Pretoria.
PAPER 11	Day, P.W.	Apr 1988	Reinforcement of Cast in situ Piles. Proc. of Course on Pile Design and Construction Practice, Geo Div, SAICE, Johannesburg.
PAPER 31	Day, P.W.	Oct 1997	Prediction of Settlement. SAICE Geotech Div. Lecture Course Programme. TWR.
PAPER 49	Fritz Von M. Wagener & Peter W. Day	1986	Construction on Dolomite in South Africa. Environ Geol Water Sci Vol 8, Nos. 1 / 2, 83-89.
PAPER 37	Williams, A.A.B.; Pidgeon, J.T. Day, P.W.	1985	Expansive Soils: State of the Art. The Civil Engineer in South Africa, Vol. 27, No.7, pp 367
PAPER 24	Day, P.W. Wagener, F. Von M.	1984	Investigation Techniques on Dolomites in South Africa. Proc. 1st Multidisciplinary Conference on Sinkholes, Orlando, Florida. pp 153-159.
PAPER 23	Wagener, F. Von M. Day, P.W.	1984	Construction on Dolomite in South Africa. Proc. 1st Multidisciplinary Conference on Sinkholes, Orlando, Florida. pp 403-413.
No copy available	Nicholaysen, L.O. Day, P.W. Hoch, A.	1984	On the Stress Changes within a Porous Elastic Sphere during Consolidation and their importance for the Supply of Power to the Gravitational Dynamo. Terra Cognition Vol. 4, No. 2, pp 239.

No copy available	Nicholaysen, L.O. Day, P.W. Hoch, A.	1982	Loss of Deep Mantle Carbonic Reduced Volatiles during N Polarity, Storage of these Volatiles during R Polarity. Lunar and Planetary Inst. Topical Conference, Alexandria, Minnesota. October 9 – 12th. LPI tech report 83-01. pp 119-121.
PAPER 34	Day, P.W. Wates, J.A. Knight, K.	1981	Skin Friction on Underslurry Piles. 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm. pp 689-692.
PAPER 28	Day, P.W.	1981	Properties of Wad. Seminar on Engineering Geology of Dolomite Areas. Pretoria 1981. pp 135-147.
PAPER 38	Day, P.W.	1981	Dumprock and Chert Gravel Mattresses. Seminar on Engineering Geology of Dolomite Areas. Pretoria 1981. pp. 256-260.
PAPER 41	Day, P.W. Wagener, F.	1981	A Comparison and Discussion of Investigation Techniques on Dolomites. Ground Profile No. 27, July 1981.
PAPER 2	Day, P.W.	1980	Geometrical Properties of Unsymmetrical Sections by Line Integration. Civil Engineering in South Africa, No. 6 Vol. 22, June 1980, pp 159-162. (Awarded SAICE awards for Best Graduate Contribution and Best Short Paper or Technical Note published in the Transactions in 1980).
MSc Eng Thesis	Day, P.W.	1980	The Application of Three Dimensional Consolidation Theory to the Movement of Ground Around an Underslurry Pile. MSc Eng Thesis. University of Natal.
PAPER 60	Day, P.W.	1978	Four Undergraduate Courses in Soil and Rock Mechanics. Lecture notes compiled for lecturing at the University of Natal (1977 and 1978).
PAPER 39	Day, P.W.	1977	An Outline of Consolidation Theory and a Simple Numerical Solution to the Biot Equation. Pulse, Vol. 224, pp 26-34, University of Natal.
PAPER 40	Day, P.W.	1976	Line Integral Method of Section Analysis. Journal of the Students' Engineering Society "Pulse", Vol. 23, pp 27-29, 1976. University of Natal.