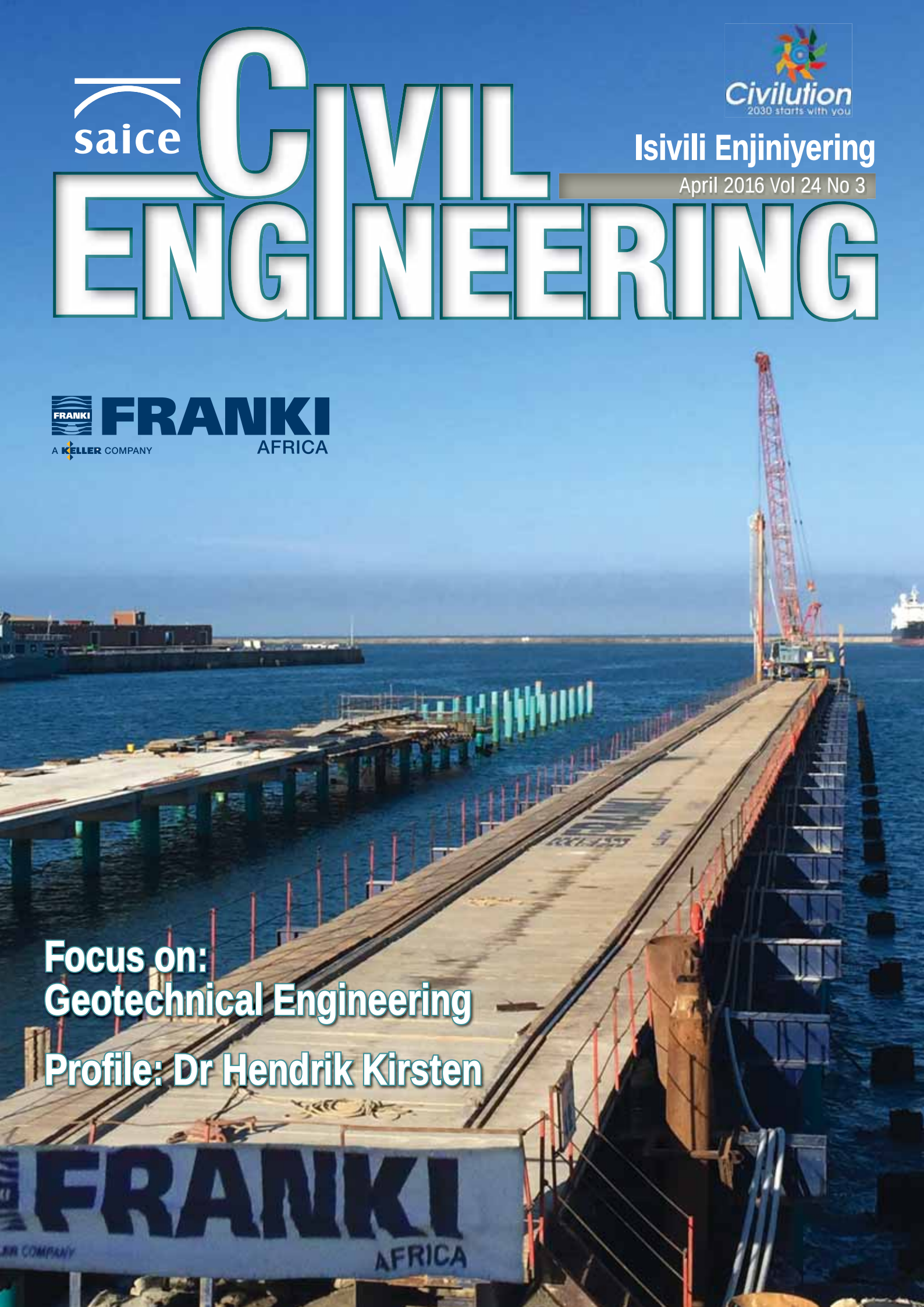
saice
CIVIL ENGINEERING
Isivili Enjiniyering
April 2016 Vol 24 No 3

Civilution
2030 starts with you



Focus on: Geotechnical Engineering

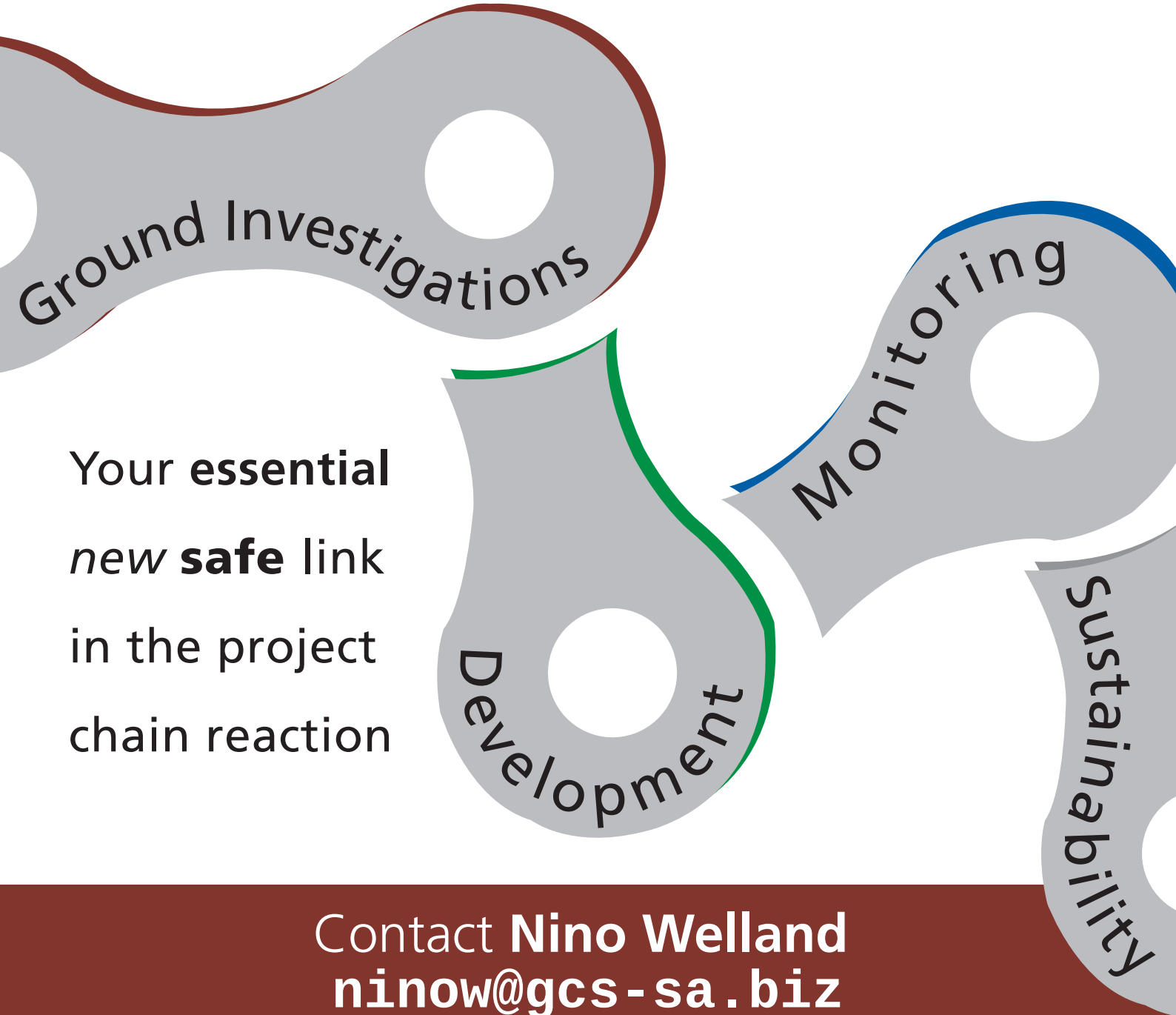
Profile: Dr Hendrik Kirsten

Introducing

GCS

Geotechnical Engineering

"The Terranauts"



Your **essential**

new **safe** link

in the project

chain reaction

Contact **Nino Welland**
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Such a time as this

"… there are many people who feel that it is useless and futile for us to continue talking peace and nonviolence against a government whose reply is only savage attacks on an unarmed and defenseless people. And I think the time has come for us to consider, in the light of our experiences … whether the methods we have applied so far are adequate." (Nelson Mandela, 1961)

IT HAS BEEN ACCEPTED in international leadership circles that the South African Constitution is arguably the best in the world. The recent ruling of the highest court in the land on the Zuma-Nkandla saga is evidence that our democracy works. Parliament has made a mockery of it, but this country – with its people, its natural heritage, its future – is worth fighting for.

I like the fact that civil engineers are an unpretentious people, and that the civil engineering profession is clothed in modesty and wisdom – that we don't aspire for pedestals to orate greatness, or pilot flashy carriages, or dress in splendour, or stand before cameras to cut ribbons. Our reputation manifests in our honest work, and our worth in the delivery of value for money.

But our friends and partners must not mistake meekness for weakness. While we may not dance to the rhythm of stampede, or chant with the reverberations of a mob, we, too, are quite capable of revolution. Our revolution is an introspective assessment that finds manifestation in the intellectual space, and its ultimate materialisation is in socio-economic wellbeing. This introspection, however, needs to find other platforms of expression, too, for evil triumphs when good people keep quiet.

This is not aimed at all public sector departments, for some have shown evidence of sterling effort, but others are deserving of rebuke.

I am fed up of our professionals being dictated to by suspect politicians and incompetent administrators on work that falls under our custodianship. We should move for all infrastructure engineering ministries at national, provincial and local government (DWS, DOT and DPW in particular) to be run by politicians who have post-graduate qualifications relevant to the built environment.

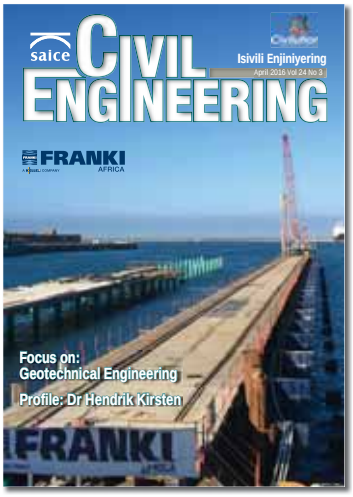
With the establishment of the NDP, the PICC and the 18 SIPs, it is clear that our leaders have adopted an infrastructure development plan for the overall strategic economic development of South Africa. As we represent the component of the private sector that is central to achieving these development aims, we want to talk to a client that is appropriately qualified, and adequately informed about the work we do. The time for a professionally registered civil engineer with 20 years of work experience to take instruction and direction from incompetent public officials, to hustle for invoices to be paid, and to deal with loop-holed procurement and SCM – is over.

We, too, have to stop selling our birthright as infrastructure engineering professionals who, having worked hard to earn our positions, relinquish our status for fear of the dynamics of master-servant relationships, rather than focusing on maintaining value for money in the delivery of projects through ethical impartation of our knowledge and scarce skills. Next time you sit in a project meeting with an incompetent client who starts to get arrogant with you, and you know that your argument is sound, make a point of sermonising your client on who the professional in the room is, who has the track record, and who is responsible for the professional risks associated with the project.

I have noticed that since President Zuma recalled Minister Pravin Gordhan to restore the financial dignity of our country, the President has been somewhat quiet in the public relations space. I can just imagine how the conversation might have gone down at the Union Buildings.

Pravin Gordhan, being a humble man, would have simply stated, "Mr President, I am honoured that you called me to assist, and I am delighted to assist." After a three-second contemplation, pursed lips and a gentle crease on the forehead, Mr Gordhan would have leaned slightly forward and said, "But Mr President, if you want me to do this job, I need to be forthright with you." After a pensive nod of approval from the distressed President, Mr Gordhan would have continued, "Sir, you have had your chance and you stuffed it up – in fact you have made a mockery of our economy. If you want me to do this job, I need you to sit quietly. Don't do anything! Sir, let me take charge and I will do my best to fix this for you."

I appeal to all civil engineering professionals to be determined in the plight of delivering value for money. Our industry is sitting on a ridge – a time of challenge and controversy. Our actions now will determine how we are remembered in history. During World War II Winston Churchill prophesied that history will deal gently with him, "... because I intend to write it!" Let us civil engineering practitioners across this country resolve to be the masters, and not the victims, of the history of civil engineering in South Africa, and take control of our profession. Our role to take charge of our profession has never been more pressing than it is today. This is Civilution. □



Isivili Enjiniyering = SiSwati

ON THE COVER

Geotechnical projects, especially on water, often demand creative solutions to complex challenges. Franki Africa's work on the conversion of a 40-ton slip into a 90-ton boat hoist jetty, and its work on the construction of two lead-in jetties for a 1 200-ton slipway, both in Port Elizabeth, are excellent examples of such creativity.

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Inserting rods on the third bay on grid-lines A and B of the 40-ton slipway in the Port Elizabeth harbour, part of the TNPA's PE Jetties Contract

FROM THE CEO'S DESK

ON THE COVER

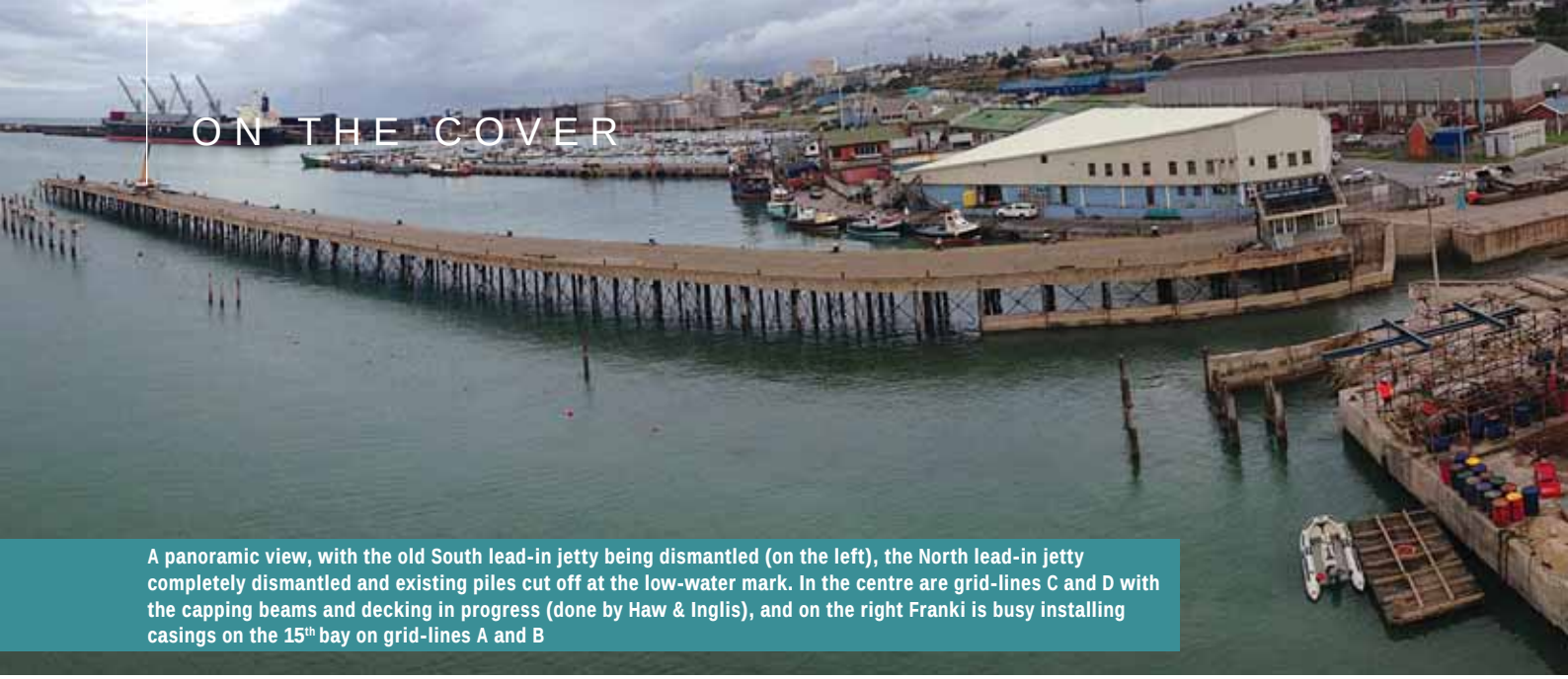
PROFILE

GEOTECHNICAL ENGINEERING

MARKET NEWS

SAICE AND PROFESSIONAL NEWS

CARTOON



A panoramic view, with the old South lead-in jetty being dismantled (on the left), the North lead-in jetty completely dismantled and existing piles cut off at the low-water mark. In the centre are grid-lines C and D with the capping beams and decking in progress (done by Haw & Inglis), and on the right Franki is busy installing casings on the 15th bay on grid-lines A and B

Franki shines on PE jetties

Geotechnical work often demands creative solutions to complex challenges. That's the nature of the game, and there are few better at it than Franki Africa, who has developed a reputation throughout Africa for their innovative and cost-effective solutions in a host of vastly divergent geological conditions. One such example is the Transnet National Ports Authority's Port Elizabeth Jetties Contract.

In September 2014 Franki Africa's Cape branch was appointed, on an alternative design, as sub-contractor to Haw & Inglis on the PE Lead-in Jetties Contract, which comprises two components:

- A 40-ton slip converted into a 90-ton boat hoist jetty, consisting of two sets of connecting jetties of 16 bays each, and
- Two lead-in jetties for the 1 200-ton slipway, consisting of the Northern Jetty (with 30 bays) and the Southern Jetty (with 39 bays).

Franki was responsible for the entire pile installation operation, while Haw & Inglis undertook the concrete deck structure in accordance with Franki's design.

According to Roy Louw, Franki's divisional director, many hours were spent strategising and planning the installation procedure and equipment that would be required. "I was concerned about the effects of vibrating through the 4.0 m seabed, drilling a 1.5 m rock socket and having a crane walking out onto the jetty before the concrete had gained sufficient strength. For this reason we finally decided to install 610 mm diameter piles using the Rotapile or ODEX method, as this would be the least risky and would also allow quicker access. I'm pleased to say that this decision certainly proved to be the correct one!"

But this meant going back to the drawing board, quite literally, for alternative design and drawings, and for the design of the single tube guide-frame. At the same time Franki requested permission to proceed with the soil investigation, as the last one had been conducted on the quay way back in 1975, and no geotechnical information was available on the lead-in jetties.

In early December 2014 the Franki crew started setting up camp, and in early January 2015 the Franki Durban team were called in to commence with a geotechnical investigation. The results were totally unexpected! The seabed was found to be 3.5 m to 6.0 m thick, with a boulder layer of 12 m to 18 m thick before bedrock was encountered. This completely vindicated the decision to opt for the ODEX piling method.

With the soils information available, Franki installed the first row of piles 19 m deep with a 1.5 m socket, just below the high-tide mark, and a 12 m test pile with a 6 m socket into the boulder layer. The test pile was tested to 3 000 kN, twice the working load, and, with a 6 mm settlement under load, the designers reviewed the original piling criteria. "With the test pile passing with flying colours, we proceeded with the pile installation, now only required to be 9 m deep below the seabed, with a minimum 3 m socket into the boulder layer," Louw explains.

Initially the piles were installed using the single guide-frame standing on the quay, as jetty sections had not yet arrived from Ghana where Franki had used them on the Ada Groynes contract. During this period – and in fact, on the entire project – the experience of those members of the Franki team who had worked on the Ada Groynes contract, proved to be invaluable.

As the team became more adept with drilling into the boulders, productivity increased significantly. They managed to complete the 30 piles on grid-lines A and B on the 90-ton boat hoist at an unprecedented rate, coming from 22 days behind schedule to only eight days.

At the beginning of August 2015 piling commenced on the Northern Jetty of the 1 200-ton slipway, and this brought new challenges to light. The cross-bracing between the piles from the original jetty, constructed at the end of the 1800s,

was obstructing the positioning of the new piles that had to be installed. A local commercial diving company was therefore appointed to dredge clear the cross-bracing and cut them out at those positions where they would interfere with the new pile positions.

Piling to the Northern Jetty was completed on 5 November 2015, 13 days ahead of schedule. The equipment was quickly transported over to the Southern Jetty, and the installation momentum was kept up. Piling to the Southern Jetty was completed on 9 March, a staggering 53 days ahead of schedule.

"This has been a monumental team effort," Louw concludes. "From the management of the complex contractual issues, to the safety management, we surpassed 80 000 accident-free man-hours. And from the management of the plant, which was kept in perfect condition throughout, to the efforts of the welding team, who ensured the reliable and regular supply of casings and jetty sections, this team has raised the bar in terms of performance."

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Drilling casings with the fixed leader on the first grid-line just below the high-tide mark

Installing a casing on the third bay grid-lines A and B

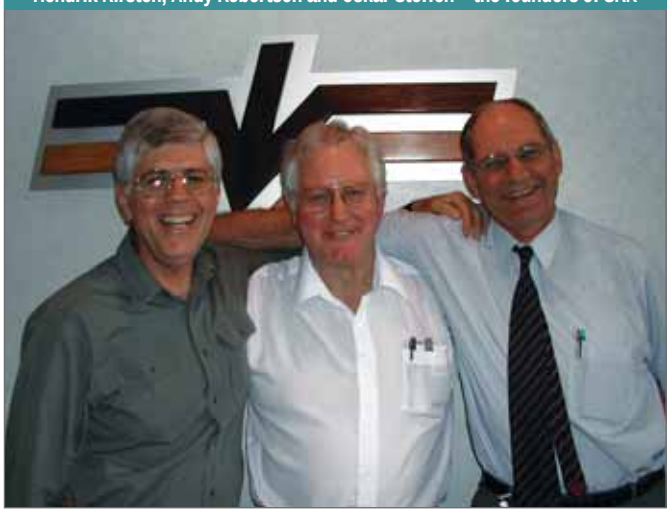
An aerial view of the casing being drilled down on the 15th bay grid-lines A and B of the slipway; note capping beams cast on grid-lines C and D to the left

Challenging the average

Dr Hendrik Kirsten, the 2015 recipient of the SAICE Geotechnical Gold Medal, has always contended that, "If you like what you do, then there's no end to you." This, and his total commitment to the pursuit of his goals, reflect how the formidable body of work he has been involved in for over 40 years – epitomising the scope and diversity that geotechnical engineering is involved in and its fertile cross-pollination of ideas – has been possible.

A delighted Dr Hendrik Kirsten (right) receiving the Geotechnical Gold Medal from SAICE COO Steven Kaplan; it was obvious at the event that he felt greatly honoured that his work had been recognised by peers in this way

Hendrik Kirsten, Andy Robertson and Oskar Steffen – the founders of SRK



LAUNCHING OF A NOTABLE CAREER

With a reputation for a searching mind, Hendrik attributes his thinking skills to the way he was brought up. "I remember my dad made a big thing of people having to use their minds, and that, I think, aroused latent abilities in us children. My mother, too, would not spoonfeed us while we were solving maths homework problems or doing Latin translations. She would assure us that solutions existed, but would let us battle it out for ourselves. As a result, my whole inclination is to use my mind as an instrument for living and being."

With nothing to commend Hendrik to the two career options of chemical engineering or civil engineering that an aptitude test later proposed, his father tossed a coin. Civil engineering won, and that was the first step towards a rewarding career which has seen Hendrik leave his geotechnical footprint in codes of practice and ground-breaking research.

After undergraduate civil engineering studies at the University of the Witwatersrand (Wits), an MSc in structures followed. Aware of his dual interest in structures and soil mechanics, Wits invited him back in 1966 to lecture in structural engineering and do a PhD. This led to an eight-year stint at the university, seven of which were spent lecturing in rock mechanics in the Mining Engineering Department, from which he received a PhD in 1986.

Undertaking consulting work during this time, it was in 1973 that he started to ponder a career outside of the university. "I was consulting on a project with Dr Andy Robertson (another Wits alumnus), and when I raised the prospect of consulting for my own account we immediately realised that, together with Dr Oskar Steffen (also a distinguished Wits lecturer), the three of us could establish a partnership. The rest is history." The three, already well-known in industry from early on, started SRK in January 1974.

As SRK's four-decade anniversary book aptly tells, this altered the trajectories of their lives. It was "three young 30-some-

thing engineers with a zeal to do things differently … and [SRK was] a magnet that attracted some of the best talents to their cause – a totally committed and hard-working professional camaraderie within a loose communal framework."

NO LOOKING BACK

"One thing I will always appreciate – in fact, I regard it as the biggest opportunity and blessing ever granted me – is that there existed a market demand for the specialist geotechnical engineering services we had to offer, and that in Oskar and Andy I had business associates of absolute integrity and trust with whom I could build a consulting practice."

SRK encapsulated the three founders' conviction that consulting is a people business, and that personal relationships are paramount. Giving people the opportunities and challenges to rise to their full potential, while cultivating a culture of innovation and competition for excellence, was also what drove Hendrik and helped SRK evolve into the dynamic practice it has become.

"We sought to create a company structure that made it possible for people to grow their own practices, be rewarded for their efforts, and yet be supported in tough times from the ruthlessness of the ever-changing consulting market place, here in South Africa and internationally.

"We knew there was commercial opportunity for SRK to provide independent consulting services in the fields of mining, geotechnical engineering and tailings disposal. The established engineering consultancies of the 1970s had in-house specialists, but it was completely novel for geotechnical engineers to establish independent businesses. The success was thanks to just fortuitously being in the right place at the right time."

The need for SRK's specialist geotechnical expertise soon came to the fore when they were appointed to design a new tailings dam at Bafokeng Mine and to investigate the reason or reasons for a breach in the embankment wall of the tailing storage facility at the mine. The failure resulted in a catastrophic slurry flow out of the impoundment, with unfortunate fatalities, major damage to infrastructure and extensive environmental impact. The risk underscoring tailings dams was clear, and this disaster *inter alia* paved the way for standards and regulations to manage tailings dam operations in South Africa, which were published some years later.

BREAKING GROUND

SRK's birth in 1974 – a practice built on high-performing, entrepreneurial individuals – was what saw Hendrik strive to continually bring the latest technologies into the business, invite leading academics in geotechnical engineering to do sabbaticals at the office and share this with the profession. This went hand-in-hand with a flow of technical papers establishing SRK as technical experts. "There was always a research aspect to many of the projects, and thus the focus on technical aspects and doing cutting-edge work."

With underground mining contracts comprising much of the work in the 1980s, Hendrik's expertise in structural engineering and the very high rock pressures in deep mines drew him into the mining side of things. However, it did not stop him from expanding the scope of his knowledge and the many interests he had within structural and geotechnical engineering, and to pursue unusual projects.

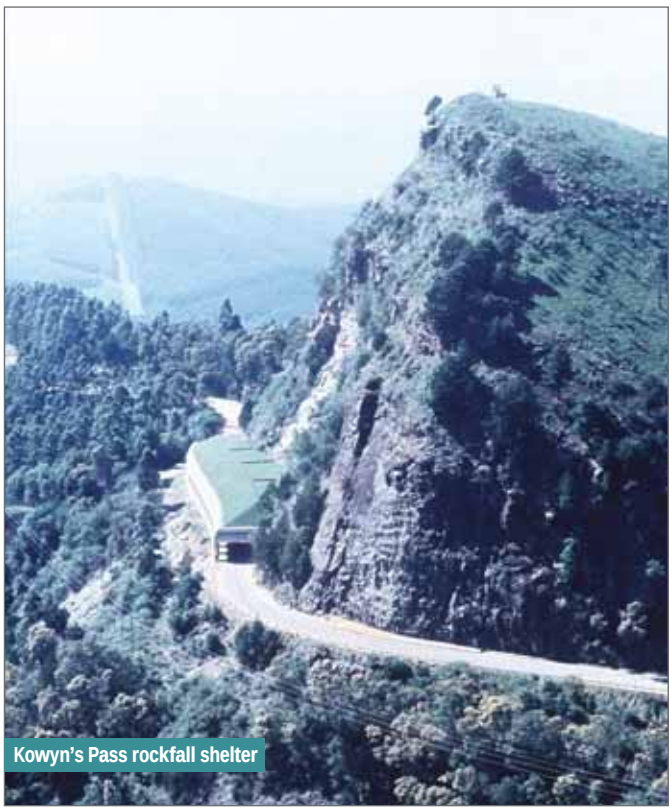
For example, from early research work on thin shells, the design solution of the Gautrain Rapid Rail Link over sinkholes was inspired (one of Hendrik's current projects includes assuring statutory compliance of dolomite land use); steel-fibre-reinforced shotcrete led to plant root reinforcement of soils; key stones in masonry vaults were found to explain the stability of jointed rock around mining stopes at great depth; the principles of mechanically stabilised earth were employed to arrest squeezing of soft ground into tunnels; and mechanical ripping of rock progressed to hydraulic erosion of soil and jet-cutting of stainless steel and ceramics.

Post-tensioned concrete u-beams supporting the Gautrain over potential sinkholes (see page 12, first paragraph)

SOME SIGNIFICANT PROJECTS

Kowyn's Pass rockfall shelter

Hendrik's appointment to this project in 1977 confirmed that the newly established consultancy was recognised for its expertise in geotechnics from the start. Loose boulders were periodically dislodged from a steep 120 m high rock slope along the road from Graskop to Hazyview. The sheer number of unstable blocks made it impossible to secure them by bolting to the bedrock. It fell to Hendrik to design a rockfall shelter across the road, much like the snow avalanche chutes in Europe. The structure comprised a pinned portal stayed against sway by horizontal anchors into the mountain. The project was a unique mixture of soil mechanics, rock mechanics and structural engineering that Hendrik pursued throughout his career.



Kowyn's Pass rockfall shelter

Hex River Tunnel

The Hex River Tunnel was another of the early projects undertaken which helped establish Hendrik and SRK's reputation. "The project involved blasting a new tunnel through the Hex River Mountains in the Western Cape for a railway line. The South African Railways engineers had compiled the tunnel design and had let the contract on the basis of what was then a standard methodology for classifying rock masses and their response to tunnelling. SRK was appointed to oversee the rock engineering aspects of the work from the start. The tunnelling did not proceed as planned. The rock did not respond in key locations to excavation as envisaged; the support needed to prevent rock falls was more expensive than planned, and the tunnel took longer to advance and cost more than estimated." A very large claim gave rise to a lawsuit on which Hendrik and SRK were the technical experts.

From this landmark South African railway tunnel project followed many more tunnel, embankment, slope and rock engineering projects, in particular participation in the Lesotho Highlands Project.

Failure of a segmented concrete pipeline in Edenvale

Another early project – the repair of a 2.1 m diameter segmented concrete pipeline for Rand Water that had failed under pressure and washed away part of the motorway in Edenvale, Johannesburg – again highlighted Hendrik's ability to solve unusual problems. The contract involved redesigning all the anchor blocks over several kilometres of pipeline, as well as redesigning the concrete spigot-and-socket joints at the bends. Later Rand Water would sell the patent rights for the joints Hendrik had designed "for a lot more than the consulting fee of R8/hour," chuckles Hendrik.

Standard Bank Building

Hendrik was also the consultant on the Standard Bank Services Centre Building – the first high-rise office building in Johannesburg that was allowed to be built on undermined ground. It was on his advice that they had purchased the land, and the foundation design helped transform the previously

Standard Bank Services Centre Building in Johannesburg

THE POWER OF **PROFESSIONAL THINKING**

HOW AN ATTORNEY IS HELPING TO BREAK THE CYCLE OF POVERTY AND ABUSE.

2012: Cindy Ross, attorney at law, is fed up with the crime, alcoholism and abuse of women and children in the Diepsloot community. Her smart solution is to create The Diepsloot Mountain Bike Academy.

Cindy gathers a support group of other concerned professionals and together they provide disadvantaged and disillusioned youth with a positive outlet of expression, through cycling, life skills programmes and enterprise development.

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unusable site on the footwall of old mining stopes into prime real estate by constructing 60 m deep mass concrete keys in the old workings. This paved the way for other developments over similarly undermined ground.

Steel-fibre-reinforced shotcrete

"De Beers' Premier Mine also financed SRK in the late 1980s to investigate whether, and under what conditions, steel-fibre-reinforced shotcrete would be equivalent to mesh-reinforced shotcrete as surface support in tunnels that are subjected to very large deformations." This was another novel problem and a big challenge. Hendrik established an industry-funded Shotcrete Working Group that took the development programme further for a number of years and showed, for example, that in some applications long polypropylene fibres were equally suitable compared to steel fibres.

This in turn led to Hendrik's ground-breaking work on the role of vegetation in making soils erosion-resistant, which culminated in the award of PhD and MSc degrees to participating members of staff.

Risk and quality management

With the advent of the new dispensation in South Africa in the early nineties came a drive to improve the safety in mines. This enabled Hendrik to add operational risk management to the array of consulting services that he could offer the mining industry. The skills and experiences that Hendrik acquired in this field over a number of years led to a CESA (Consulting Engineers South Africa) publication titled *Risk Management Implementation Guideline*, of which a second expanded edition appeared in 2014. It was not long after the first publication of this guideline that the related *Quality Management Implementation Guideline* was published in 2011 by CESA, also written by Hendrik.

Towards the close of the project, Hendrik in 2012 reviewed the risks of derailment of the Gautrain and the adequacy of the associated containment provisions.

After twelve years Hendrik still serves on the Quality and Risk Management Committee of CESA. He is committed to and has a passion for engineering consulting and recently participated in the presentation of the CESA 'Seminar on Practice Notes'.

The sustained quality of engineering practice in the future is a major concern to Hendrik. He believes that the public image of the profession, and the enthusiasm of students and young professionals for the profession, should be raised and the commitment of senior engineers to the profession should be proactively rekindled.

Hendrik currently serves as convenor of the WG3 Working Group responsible for the revision of Part 4 on Risk Management of the South African National Standard on the development of dolomite land, SANS 1936:2012 that will be published in 2017.

Probability-based design

Prof Milton Harr brought the power of probability-based design to Hendrik's attention during a visit to South Africa at the start of the eighties. The absence of guidelines on acceptable probabilities of failure and injury was a major challenge which Hendrik overcame without delay by publishing a set of norms based on the failure statistics for relevant engineering applications. In the

Sinkhole under single-storey dwelling

early nineties these norms were brought in line with those used in the financial and insurance industries.

This enabled Hendrik to design (on a probability basis) road bridge foundations under hydraulic action in the UK, tunnel support in mines in the USA, the post-tensioned concrete U-beams for the Gautrain Rapid Rail Link over potential sinkholes, the population density in single-, double- and treble-storey residential developments, and multi-storey office and residential buildings on dolomite land, which included the safety of the largest private hospital in the southern hemisphere on dolomite land.

Hendrik is currently engaged in the design of tailings storage facilities over ongoing underground mining operations on a probability basis.

Expert witnessing / engineering forensics

Major contractual disputes are often related to the foreseeability of adverse subsurface conditions. In some instances, the disputes are related to the responsibility of the contractor to ensure the stability and safety of temporary works, which is exacerbated when the temporary works are incorporated in the permanent works. The Hex River Tunnel was a particular case in point involving both of these issues.

Early in his career Hendrik was confronted with the inability of steel arches to protect soft ground tunnels against squeezing, which he overcame by reinforcing an annulus of material right around the tunnels with grouted cable anchors. This was met with some resistance initially, but has now become standard practice.

Hendrik has been involved as an expert witness in numerous disputes including hard and abrasive rock in quarries, road and runway excavations, submarine outfall sewers and machine-bored tunnels. He has appeared as an expert witness in a number of instances on the rippability of rock, as determined by the hardness and structure of the rock. This led to the development of his classification on the mechanical excavatability of rock.

Adverse conditions in tunnels and open-pit excavations are related to the frequency and orientation of joints in the rock which demarcate unstable wedges, widely separated joints (up to 150 mm) that give rise to block fall-out, and soft ground in shear zones that result in tunnel squeezing. Hendrik has also been involved in cases in which unstable ground has led to fatal injury.

Collapsible materials can cause severe settlement of building foundations if not identified in time and properly dealt with. Hendrik has in a number of instances been involved as an expert witness in disputes of this nature. Other disputes on which Hendrik has appeared as expert witness include shortfalls in mining production, liquefaction and breaching failures of the sand slopes of dredged harbour channels, and undue seepage into bulk earthworks excavations.

Exposure to engineering failures and the consequent resolution of the resulting damages have greatly enriched Hendrik's insight into the pitfalls in engineering judgement and decision-making. He has developed an acute sense for engineering forensics, which is often the unidentified gap between lawyers and technical experts on contractual disputes.

HIS GREATEST SUPPORT

His wife, Yvonne, has been his greatest support during the long hours and travel which were demanded of him. In the early SRK days, she would also do typing and office support on top of caring for their four young children. "Anyone who was prepared to work for free during those tough years of establishment was welcome," says Hendrik with a twinkle.

"We met at Wits, in the chemistry department where I had gone to ask for special soap for my final-year undergraduate research project. She was doing typing work, saving up to pay for teaching studies. Needless to say, I never got the soap. In fact I never even asked for the soap, but came away set for life," he quips. "We waited three years until she had completed her studies, and in 1966 we were married, which makes this year our golden anniversary."

Hendrik and Yvonne are now also the proud grandparents of their four children's seven and counting children.

ACADEMIA AND INDUSTRY

Over the years Hendrik has published 90 technical papers, and the research aspect of many of his projects led to an inseparable and critical link between academia and industry – even from the early 1980s he was involved in the development of the Rand Afrikaans University's (now the University of Johannesburg) engineering faculty.

His ability to empower others in the skills he has developed over many years saw him appointed at the University of Johannesburg as Professor of Civil Engineering from 2006 to 2008, and as Executive Dean of the Faculty of Engineering and the Built Environment in 2008.

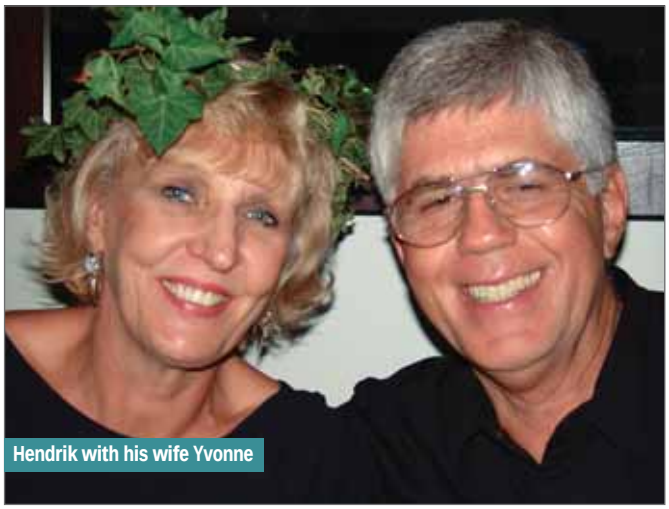
CHANGING ANGLES

"I've had a privileged career to always be exposed to and involved in novel things. One cannot wish for a better situation than to have a profession in which one is paid to do a hobby twenty-four-seven."

In 2001, however, desiring a change, Hendrik left SRK and started his own one-man consulting practice, making himself available to a wider set of clients, while being able to spend more time with his family.

Hendrik, still not retired, remains in demand as a specialist, technical expert on legal cases involving geotechnical engineering, and in supporting and expanding university education in South Africa – having truly made and still making geotechnical engineering his life's work.

"SRK still plays a unique role in my career, albeit that the relationship is completely independent. I have developed a greater awareness that one needs to continue engaging with people and the world around one, and that one needs to participate in deci-



Hendrik with his wife Yvonne

sions of which the outcome is not a known certainty. One lives and progresses by risk."

At 74 Hendrik is still registered with ECSA as a professional engineer and is actively working towards at least another five-year cycle of registration.

APPRECIATING THE ROLE OF OTHERS

Paying tribute to the many collaborations which have characterised his rewarding and successful career, Hendrik says: "One sets off with a dream to climb a distant mountain, together with some fear of not making it. The dream keeps the hope alive and the fear keeps the midnight candle burning. That is all that you are aware of, and when you lift up your head, the mountain is behind you, and you don't know how that happened and you cannot take credit for it, because somehow you were carried along by countless others."

ADVICE TO UPCOMING PROFESSIONALS

"Rest assured, you can achieve any goal that you may set for yourself, provided you stay one step ahead, but that requires that you never stop publishing your work and sharing your thoughts. Embrace the professional institutions and regulating authorities, and work together with them to serve the public towards whom you have a fiduciary obligation. In this way will you pay tribute to those who established the great profession of which you can be proud to be a member."

Rebekka Wellmanns
rebekka@saice.org.za

Hendrik has been involved as an expert witness in numerous disputes including hard and abrasive rock in quarries, road and runway excavations, submarine outfall sewers and machine-bored tunnels. He has appeared as an expert witness in a number of instances on the rippability of rock, as determined by the hardness and structure of the rock. This led to the development of his classification on the mechanical excavatability of rock.

Founding improvement using geosynthetics at the Cape Town harbour

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BACKGROUND

In 2013, Transnet National Ports Authority (TNPA) commenced with upgrading of the existing fire-fighting system of the oil tanker terminals in the Port of Cape Town (Figure 1). This included the construction of a new pump station, which would house booster pumps on the ground floor and a 250 kilolitre water reservoir directly above the pump station on the first floor. The design bearing pressure exerted on the ground by this new structure would be 150 kPa.

The pump station site is located inside the Port of Cape Town. The fill material (the placing of which started in 1965) comprised hydraulically backfilled material derived from dredging activities, and highly variable end-tipped imported material.

The objective therefore was to design a stable and effective foundation capable of handling loads from a dynamic structure (the pump station) on a highly variable, weak soil deposit.

GEOTECHNICAL CONSIDERATIONS

A geotechnical site investigation was conducted using a small-diameter rotary core barrel with standard penetration tests (SPT). The exploratory borehole was drilled to a depth of 23.5 m below the existing ground level. The schematic diagram in Figure 2 shows the subsoil conditions that were encountered during the investigation. These subsoil conditions can be summarised as follows:

- Variable fill materials in terms of composition, low consistency and thickness creating compressible soil conditions (problem soil)
- Presence of large obstacles, such as tetrahedron dollies and very hard boulders up to 1.5 m in diameter, as well as a rockfill layer at depth (quay construction), which could hamper piling installation
- Soft, variable marine deposits in the order of 6.0 m thick
- Weathered meta-sedimentary strata associated with Malmesbury rock,

Figure 1: Site location of the new pump station (Source: Google Earth)

The pump station site is located inside the Port of Cape Town. The fill material (the placing of which started in 1965) comprised hydraulically backfilled material derived from dredging activities, and highly variable end-tipped imported material. The objective therefore was to design a stable and effective foundation capable of handling loads from a dynamic structure (the pump station) on a highly variable, weak soil deposit.

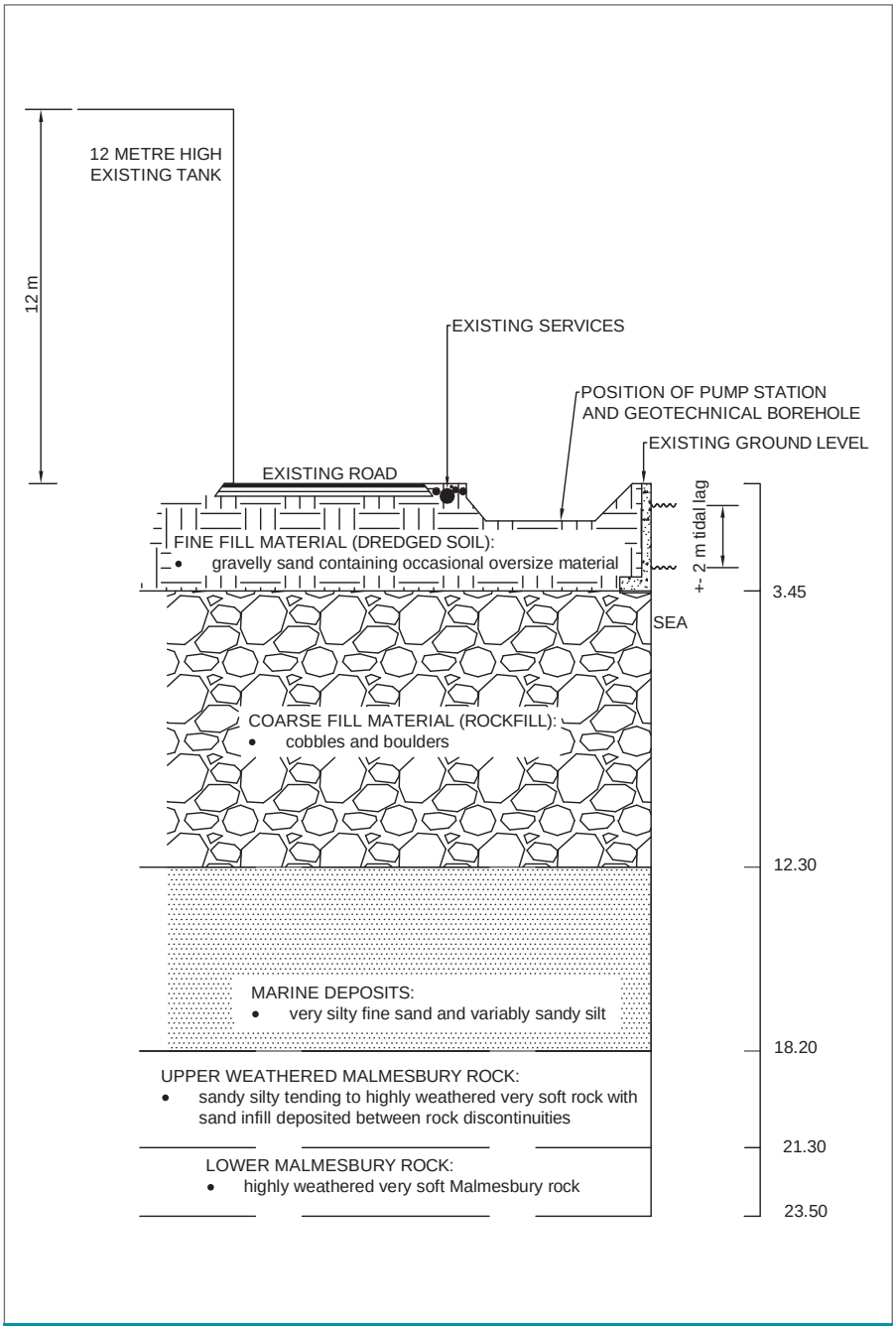

Figure 2: Schematic diagram of the subsoil conditions

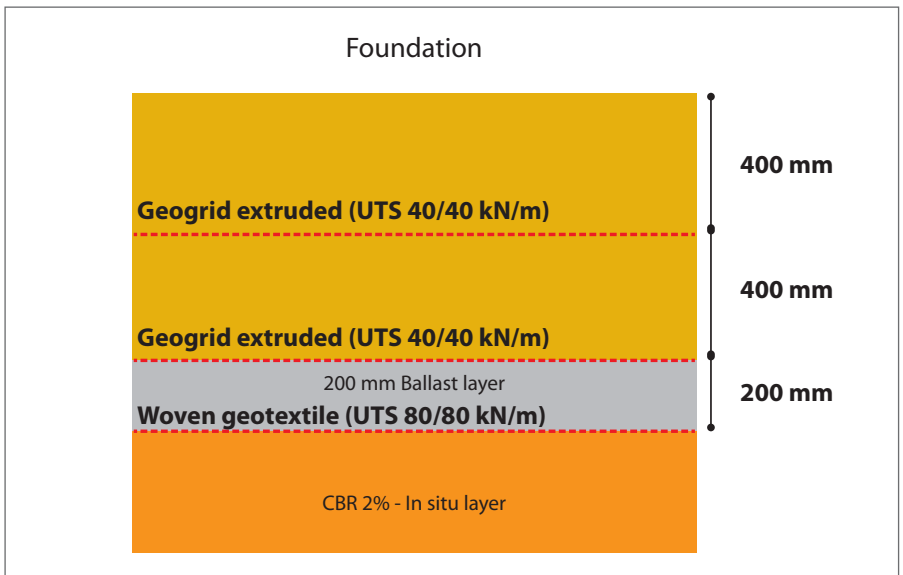

Figure 3: Section through the reinforced foundation

with occasional discontinuities filled with sand

- Significant depth to competent rock.

GEOTECHNICAL OPTIONS

Based on the subsoil conditions the following three types of foundation improvements were considered:

1. **Deep compaction or re-compaction of the fill soils:** This option was ruled out due to lateral working restraints and the potential of damaging existing infrastructure in close proximity to the site.
2. **Piled foundation:** This option would only have been viable using percussive or oscillator piling techniques to penetrate through the large obstacles and the boulder layer (rockfill). The pile seating depths were estimated at 23.5 m below the ground level into the lower competent Malmesbury rock.
3. **Geosynthetic reinforced soil raft foundation:** This option would involve the installation of geosynthetic layers between suitably engineered fill materials to improve the bearing capacity of the subsoils.

After consideration of the geotechnical conditions, construction challenges and potential solutions, the geosynthetic reinforced soil raft foundation was found to be the most viable solution for the site conditions.

GEOSYNTHETIC REINFORCED SOLUTION

The design technique is based on the distribution of vertical pressures through a geogrid reinforced layer (stiffened soil raft). The design method requires that the pressure at the top of subgrade is less than the allowable bearing pressure of the subgrade soil to provide an adequate factor of safety. This static method assumes that the vertical pressures are distributed through the platform soil layer, as outlined by the Boussinesq theory (Das 1990).

The design pressure of 150 kPa and the California Bearing Ratio (CBR) of 2% (in-situ bearing capacity of 15 kPa) were used for the design calculations. In unreinforced conditions the foundation thickness required was in the order of 2 m. However, through the use of geosynthetic reinforcements the thickness was reduced to 1.2 m, as shown in Figure 3, with one layer of woven geotextile (in polypropylene with an ultimate tensile strength of 80 kN/m in both directions) functioning

as a separator layer, and two layers of extruded geogrids (polypropylene bi-directional geogrids with an ultimate tensile strength of 40 kN/m) placed within 400 mm thick of a G5 material (TRH 14), which requires a minimum CBR of 45 once compacted to a minimum of 95% of the modified AASHTO density.

CONSTRUCTION CHALLENGES

Buried services

Attempts were made to identify the existing utility services prior to construction. However, due to the age of the port, the record drawings did not reflect the exact ground conditions upon excavation. Damage to these services could lead to safety risks for the construction team, as well as risks to the environment and commerce.

All services were therefore located by hand, and subsequently relocated to an area outside the footprint of the pump station structure. This was a time-consuming activity, because these services ranged from medium voltage electrical cables, freshwater pipelines, firewater pipelines, compressed airlines, and sleeves housing telecommunication cables (Figure 5).

Influence of the tidal zone

It was established that the bottom of the excavation, in terms of the required depth of soil improvement, fell within the fluctuating tidal zone. During low tide the area would be dry and safe to work in, but during high tide the tidal water would rise up to 500 mm above the bottom of the excavation.

Because of the saturated founding conditions, a 200 mm pioneering ballast layer, using 26 mm single-size aggregate, was placed and statically compacted underneath the engineered layers to create a stable working platform above the influence of the tidal water level.

QUALITY CONTROL

Plate load tests were undertaken on the in-situ subgrade using a 600 m plate diameter in the pioneering layer, as well as on the installed geosynthetic layers (Figure 7). The axial loads and the corresponding displacement were recorded at predetermined load increments, and the resulting data was then used to generate applied load versus deflection and subgrade modulus reaction curves (Figures 8 and 9).

Figure 4: Construction of the geosynthetic reinforced soil raft foundation

Figure 5: Locating the existing buried services proved to be a time-consuming challenge

Figure 6: Tidal influence – low tide (top) and high tide (bottom)

Figure 7: Plate load test undertaken during construction

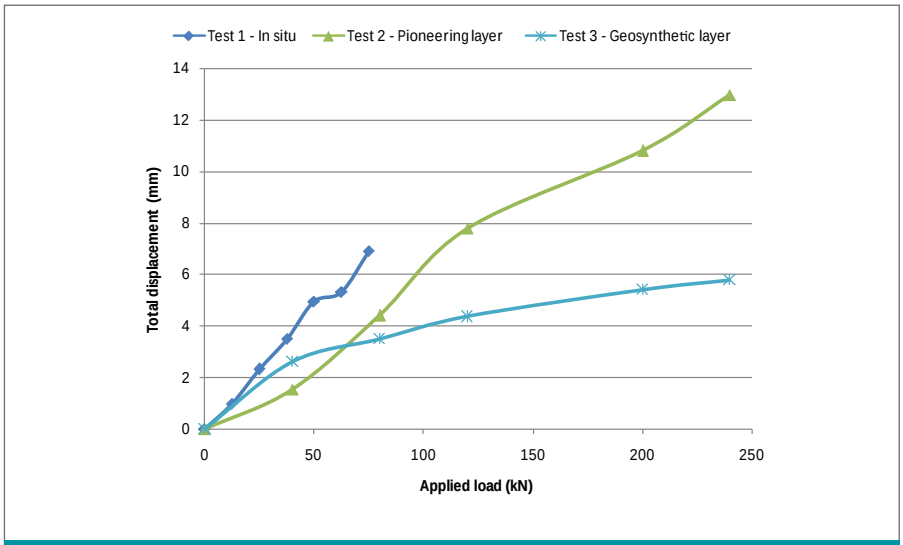

Figure 8: Applied load vs deflection

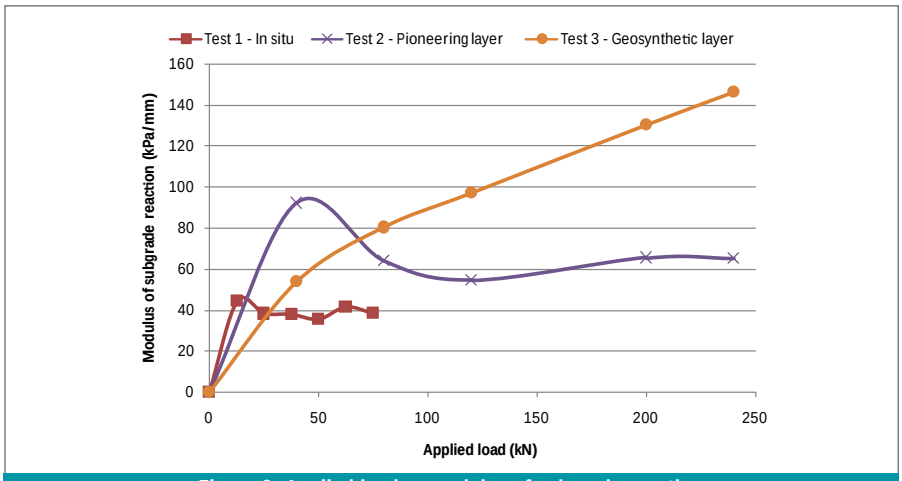

Figure 9: Applied load vs modulus of subgrade reaction

Figure 10: Booster pump station as of December 2015

The soil raft founding material (approximately 1.2 m below ground level) comprised loose, variably silty fine-grained sandy material. Test 1 confirmed that the in-situ material comprised generally low-strength soils (bearing capacity < 40 kPa), which necessitated the ground improvement. The plate loads test results at respective levels within the reinforced soil raft are presented in Figures 8 and 9.

During the construction phase the settlement of the pump station was measured on a weekly basis to record its average structure settlement. These readings confirmed that minimal settlement had occurred during the construction phases (with 95% construction completed). Following completion and leak detection testing, the structure settled uniformly and attained equilibrium with the ground.

CONCLUSIONS

The importance of an adequate geotechnical investigation and retaining the geotechnical engineer during the construction phases remain the most effective methods of managing the subsurface risks where problem soils are encountered. Sadly, in most civil engineering projects, the involvement of the geotechnical engineer tends to terminate after the investigation phase.

The installation of the cost-effective geosynthetic reinforcement improved the bearing capacity of the ground, ensuring that the structural loading conditions were met, thereby essentially eliminating the requirement for expensive piling techniques.

Although this reinforced soil raft concept is commonly undertaken overseas, this solution is not regularly considered in geotechnical applications in South Africa from the perspective of a founding soil improvement technique to develop the ability of weak soils to support structures.

The behaviour of the layers indicates that, with adequate geotechnical investigation, and assessment and design interpretation, suitable geosynthetics are capable of supporting medium-loaded structures by transferring the stresses and applied loads through the layers. This 'transfer of soil strength' is the key to designing geosynthetic reinforced foundation pads. □

PROJECT TEAM

Client: Transnet National Ports Authority
Consultant: AECOM SA (Pty) Ltd
Contractor: Stefanutti Stocks Marine

The effect of geogrid position on the integrity of clay liners

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The performance of clay liners can, even after appropriate design and construction, be compromised by differential settlement of the underlying material, should this occur. CCLs are unable to withstand tensile strains and maintain their integrity. Even at low deformation levels tensile strains are induced in the clay, which may cause cracking (Ajaz & Parry 1975). The hydraulic conductivity and strength of the clays may thus be affected by these phenomena and may create possible flow paths in the clay for contaminants such as leachates.

INTRODUCTION

Waste disposal can be implemented using a number of strategies. Landfill sites represent one type of these strategies and are specifically designed facilities built either on top of the ground or into the ground in which waste is placed, and are typically compacted and covered with topsoil (Ersoy *et al* 2013). An important factor to consider when designing a landfill site, is the need for physical separation between the solid waste body contained in the landfill and the natural ground water. This is usually achieved by installing a compacted clay liner (CCL) prior to the placement of any waste in the landfill. Liners in waste disposal sites are commonly made up of fine-grained soils such as natural clays, silty clays or clayey silts, because of their natural availability and properties. These properties include low permeability and diffusivity, high ductility, good constructability and high chemical compatibility (Rajesh & Viswanadham 2012). The prevention of possible migration of pollutants (primarily leachates) in waste into the soil, groundwater and surface water is thus the main objective of a CCL (Mohamaed & Anita 1998).

The performance of clay liners can, even after appropriate design and construction, be compromised by differential settlement of the underlying material, should this occur. CCLs are unable to withstand tensile strains and maintain their integrity. Even at low deformation levels tensile strains are induced in the clay, which may cause cracking (Ajaz & Parry 1975). The hydraulic conductivity and strength of the clays may thus be affected by these phenomena and may create possible flow paths in the clay for contaminants such as leachates. In the case of a landfill liner design this can be

severely problematic where clay has specially been chosen for its low permeability property (Da Silva 2014). It is proposed that the integrity of compacted clay liners can be improved by using geosynthetic reinforcement in the liner. The American Society for Testing and Materials (ASTM D4439 2014) defines geosynthetics as: "Products manufactured from polymeric material used with soil, rock, earth or other geotechnical engineering-related materials as an integral part of a human-made project, structure or system."

There is currently a wide variety of geosynthetics available, which include geotextiles, geogrids, geomembranes, geosynthetic clay liners, geocomposites, geonets, geopipes, geofoam and geocells (Das 2011). The study on which this article is based investigated the effect of geogrid reinforcement on the integrity of a clay liner. The use of geogrids as reinforcement material in geosynthetic clay liners is increasing rapidly. They have a net-like appearance and consist of parallel sets of intersecting ribs with large enough apertures to interlock with the soil matrix surrounding it (Das 2011).

RESEARCH PROJECT

To observe the different modes of behaviour, physical model studies were conducted on an unreinforced clay layer and three reinforced clay layers, with reinforcement placed at different depths, and subjecting them to differential settlement and a surcharge load. The four different tests were:

- Test 1 – Control test with no geogrid in the clay layer
- Test 2 – Geogrid in the bottom part of clay layer
- Test 3 – Geogrid in the top part of the clay layer
- Test 4 – Geogrid in the middle part of the clay layer.

The objectives of the experimental work were:

- To investigate the shear strain and vertical displacement of an unreinforced clay layer subjected to non-uniform settlement.
- To investigate the shear strain and vertical displacement of a reinforced clay layer subjected to non-uniform settlement.
- To investigate the effect on the location of the reinforcement within a clay layer subjected to non-uniform settlement.

Only the location of the geogrid reinforcement was changed during the experiments. All other variables were kept constant. Particle image velocimetry (PIV) (White *et al* 2003) was used as measuring technique to determine the shear strain and vertical displacement in the soil for this study.

A schematic representation of the model that was used can be seen in Figure 1. The container had openings both in the front and back to allow a water-filled cylindrical rubber membrane to pass through it. Non-uniform settlement was created by extracting water from the rubber membrane that was placed at the bottom of the model container. Water from the rubber membrane was extracted in a controlled manner during testing by means of an actuator. For the purposes of this discussion the term "volume reduction" will refer to the amount of water that was extracted from the rubber membrane. At the front of the container a 6 mm thick glass panel was installed to observe any movement for soil strain analysis, using PIV.

Typical results, as seen in Figure 2, showed that, at 50% volume reduction, a clear difference in vertical displacement of the clay layer for the various tests could be seen. However, at 100% volume reduction, the unreinforced clay layer indicated the largest overall vertical displacement, and the clay layer reinforced in the top part had the lowest overall vertical displacement, as can be seen in Figure 3.

From the PIV analysis for each test, a vector plot and a shear strain plot were obtained. Vector plots indicate the magnitude of movement of points within the clay layer. For each plot, the vector magnification factor and scales were kept constant for all the tests. Strain plots indicate the location of maximum shear strain in the clay layer. It must be noted that the *location* of the shear zone is important for this discussion, rather than the exact value that is associated with the strain. Typical results are illustrated in Table 1 (detailed results are presented by Van Zyl (2015)). Shear strain plots for Test 1-3 showed the highest shear strain values at the inflection points in the clay layer, as seen in Figure 4. Shear strain in Test 4 (centre reinforcement) appears to be spread across the layer rather than being located at certain points.

Test 3 also illustrated that it is most effective to provide tensile reinforcement in the top part of the clay layer where the reinforcement helped to reduce the tensile strains at two locations, as seen in Test 3, Figure 5(b), rather than one location, as seen in Test 2, Figure 5(a).

CONCLUSIONS

It was concluded that the inclusion of a geogrid in a clay layer improved the integrity of the clay liner. The reinforced clay layers showed better performance in terms of less vertical displacement and lower shear strain than the unreinforced clay layer. Test 3, with the reinforcement in the top part of the clay layer, showed the best performance in terms of limiting the vertical displacement and shear strain at any given percentage of volume reduction. Test 4,

with the reinforcement in the middle of the clay layer, showed similar vertical displacement than that of Test 3, but only up to 50% volume reduction. Thereafter, the displacement increased significantly. The results suggest that the overall performance in terms of minimising the settlement, and thereby preserving the integrity of a clay layer, can best be improved by placing a geogrid in the top part (three quarters from the base) of the layer.

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This article is based on Tinus van Zyl's final-year research project in the Department of Civil Engineering at the University of Pretoria (carried out under the supervision of Prof SW Jacobsz) on the influence that geogrid reinforcement has on the integrity of clay liners.

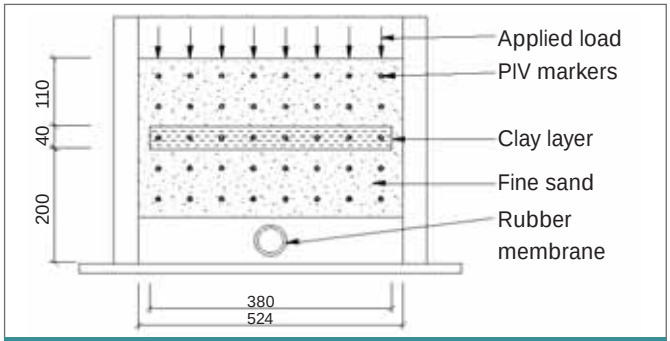

Figure 1: Model setup

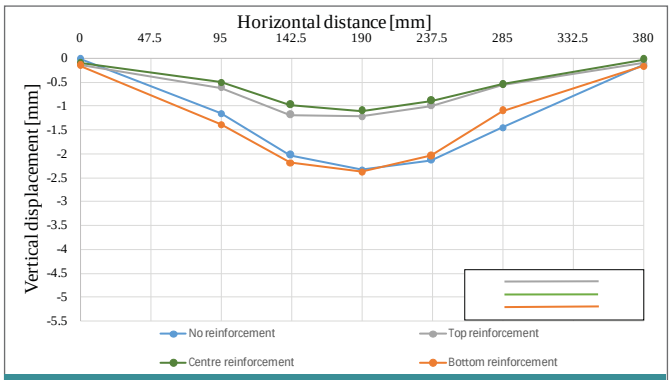

Figure 2: Vertical displacement vs horizontal distance at 50% volume reduction (Test 1-4)

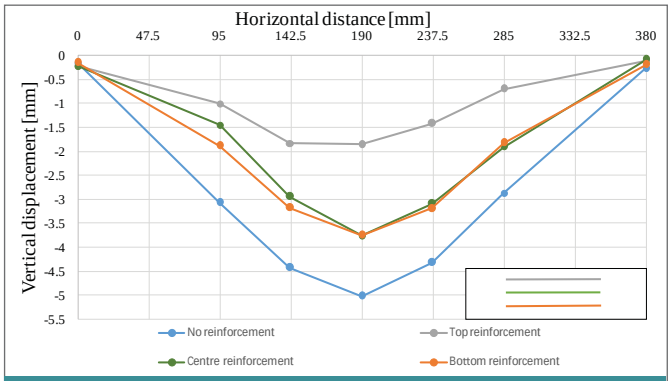

Figure 3: Vertical displacement vs horizontal distance at 100% volume reduction (Test 1-4)

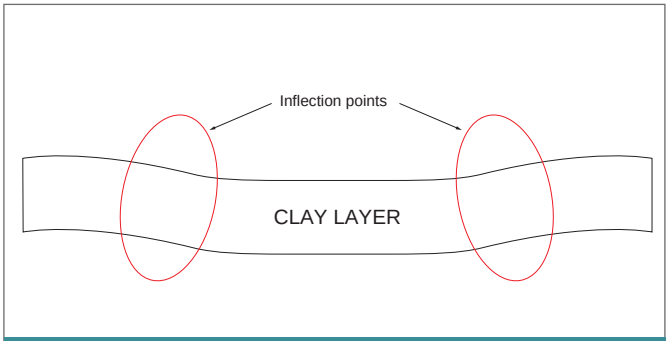

Figure 4: Approximate locations of inflection points

Table 1: Vector plot and shear strain at 33% volume reduction (Test 1)

Test 1 at 33% volume reduction	
Photo	
Vector plot	
Strain plot	

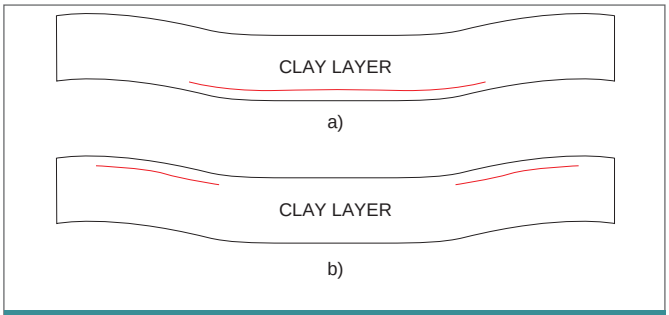

Figure 5: Location of reduced tensile forces for (a) in Test 2 and (b) in Test 3

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INTRODUCTION

Geosynthetic materials have been successfully installed as effective barriers in various types of earth and concrete structures over the past 50 years. However, as technology and research in the geosynthetics field evolved, it was found that composite liners have certain limitations, and that heat can significantly decrease the service life of geosynthetic components.

According to the highly respected US Environmental Protection Agency, current standards do not adequately address toxic pollutant discharge, frequently resulting in toxic chemical seepage from unlined ponds and dry waste landfills into ground and surface waters. Although the Agency's concerns refer mainly to pollution caused by coal-related products, the reality is that clean water is the source of life, and hence it is critical to a sustainable future.

The worldwide survey of regulatory standards for waste management and pollution control (GRI Report No 34 of 2007: GRI's Second Worldwide Survey of Solid Waste Landfill Liner and Cover Systems) indicated that authorities prefer geosynthetic composite liners for pollution control over clay and modified soil liner systems.

COMPOSITE LINER

A composite liner can be defined as a flexible membrane liner in intimate contact with a mineral liner. The mineral liner can either be a compacted clay liner (CCL) or a geosynthetic clay liner (GCL).

The performance of such composite liners should be evaluated based on total solute transport, which considers both advective losses and diffusion of volatile organic compounds (VOCs) from the waste stream (Foose *et al* 2002).

Construction phase influences

The climatic conditions that are prevalent during construction can significantly reduce liner performance. Sunshine may induce desiccation cracking of the clay component, in particular of pre-hydrated and uncovered GCLs, as well as induce wrinkles in the geomembrane, which would lead to increased advective losses. Excessive rain on the clay component can lead to displacement of the fine fraction at the interface and resultant pervious zones, while wind, too, can displace the fine fraction, which is critical to controlling impermeability.

Hydration of the GCL component of a composite liner prior to its exposure to leachate is required. However, this hydration should take place after application of a normal load (Vangapaisel *et al* 2002). This is extremely difficult when the GCL is part of a composite liner and isolated from soil moisture by either the leakage detection system or underlying secondary liner geomembrane.

Pre-hydration by means of spraying water on the GCL immediately prior to covering with a geomembrane induces damages such as squeeze of the bentonite and preferential flow paths through desiccation cracks. The failure to pre-hydrate a GCL prior to its exposure to leachate, especially if containing hydrocarbons or salts, will result in a loss of performance.

Service life influences

The lifetime prediction of a geomembrane has been addressed by numerous authors (Sangam & Rowe 2002; Koerner & Hsuan 2003; Rowe 2005) for exposure to elevated temperature and various fluids.

These elevated temperatures significantly reduce the service life of the geomembrane, may induce desiccation cracking of underlying clay components of composite liners and increase the total solute transport.

The relatively small temperature increase in the lower range of 10°C to 35°C on a composite liner increased diffusion by 100%, and hydraulic conductivity (or advection) by 80% (Rowe 2005). Similar considerations need to be given to the drainage system performance which is affected by both normal stress resulting in intrusion, and elevated temperature-induced deformations, causing a reduction in performance of geosynthetic drainage systems.

TEMPERATURE CONSIDERATIONS

The negative effect of temperature on geosynthetic components is a topic that

attracted the attention of numerous researchers over the past decade. The seriousness of this limitation has been recognised by leading geomembrane manufacturers who are investing in the development of temperature-resistant geomembranes (Ramsey & Wu 2013).

Investigation of the thermal conductivity of GCLs showed that thermal conductivity increased with the increase in moisture content (Singh & Bouazza 2013). This may lead to the GCL acting as an insulator if it is not properly hydrated, which will cause elevated temperatures on the primary geomembrane. The moisture content of the clay component in the composite liner has a significant impact on thermal conductivity, and the leak detection or under-drainage system has significant air voids which act as a thermal barrier, with the result that heat builds up in the primary liner and accelerates its degradation, unless mitigated.

INNOVATING TO OVERCOME THE CHALLENGES

The foregoing shows that there is a need in the geomembrane industry to mitigate the effects of elevated temperature on composite liners, post-loading hydration of GCLs and the removal of VOCs to expand the performance of the conventional geomembrane installations.

Principles of the enhanced barrier system

A concept was developed which involves drawing a fluid under negative pressure through a pervious zone adjacent to the barrier, so that the fluid can be used both to cool the primary composite lining and adjacent drainage systems, and to introduce moisture to the GCL beneath the overlying geomembrane for its hydration (after placement of a normal load and prior to the risk of its exposure to leachate).

The fluid (gas, liquid or two-phase mixture) passing through the pervious zone also maintains the leak detection system at a low to zero concentration of VOCs, thus preventing their further diffusion into the adjacent environment.

The negative pressure is essential to ensure no introduction of oxygen to the waste body through a discontinuity of the base geomembrane, which could induce spontaneous combustion, depending on the composition of the contained waste. The negative pressure results in a net outward flow towards the leakage detection system sump.

This concept was verified by a range of laboratory experiments, as well as by an infield application. The results of these experiments were previously published (Gundle *et al* 2013).

Figure 1 describes the working principle of the system in a solid-waste site, specifically where all three functions of the enhanced barrier system (EBS) are utilised.

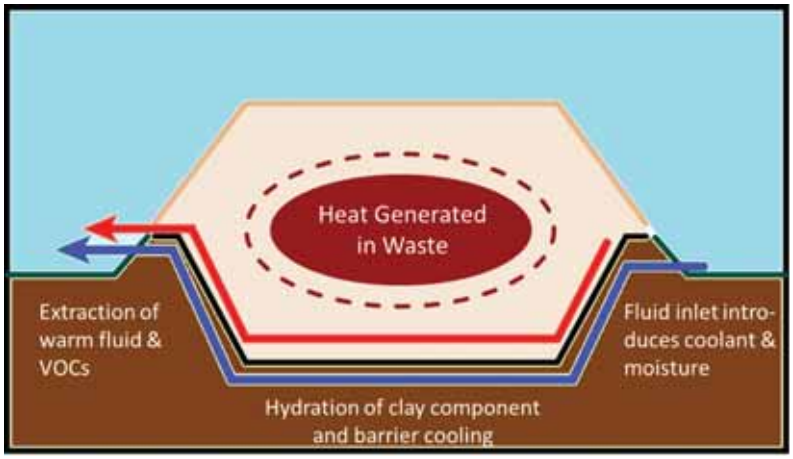

Figure 1: EBS working principle in a solid waste facility

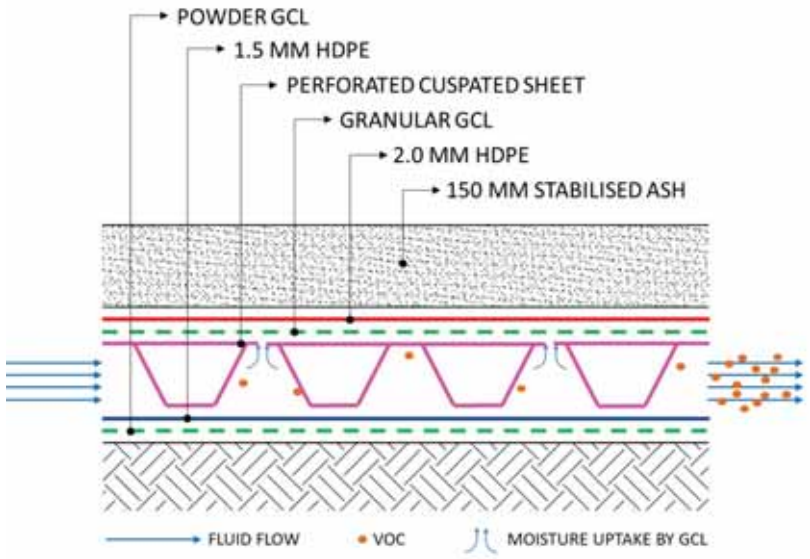

Figure 2: Barrier system and working principle of EBS

Figure 3: Earthworks preparation in progress

It is important to note that one or any combination of the three functions of the EBS can be utilised depending on project requirements.

Benefits of the EBS

The regulatory standards are becoming increasingly strict. Regulators are moving in the direction of requiring designers to address the mitigation effects of heat on a barrier installation to ensure that the required design service life is achieved. Furthermore, should VOCs be present in the containment facility, the designer has to address these and provide a solution to prevent VOCs from contaminating the subsoil.

The EBS provides designers with a practical, low-maintenance and cost-effective solution to address the requirements enforced by regulators. Even more important, it extends the barrier's service life, extracts VOCs and overcomes construction challenges regarding hydrating and maintaining the optimal hydration of the clay component – ultimately to protect the environment.

Figure 4: Lining and pipe installation in progress

Figure 5: Segmentation between compartments taking place

PROJECT DESCRIPTION

Design

The first EBS application in the world was specified by the consulting engineering firm Royal HaskoningDHV, to be installed at a hazardous waste sludge lagoon. The total solute transport analysis showed that significant VOC concentrations in the waste stream would diffuse through the contaminant containment barrier system. The site foundations and operational constraints favoured an all geosynthetic solution in which both the primary and secondary barrier would be a composite (geomembrane plus geosynthetic clay liner), separated by a leak detection system which would also be a geosynthetic product. Due to the nature of the liquid waste to be contained, it was evident that the GCL needed to be pre-hydrated, i.e. hydrated prior to exposure to waste containing hydrocarbons. Similarly the diffusion of VOCs from the liquid waste containment had to be prevented from migrating to the groundwater regime, and hence either a significant sorption layer would be required, or VOCs had to be removed from the leak detection system.

The facility is 13 hectares in footprint and 10 metres deep. The waste facility design was able to conform to the conventional double-composite liner, with an intermediate leak detection layer complying with the hazardous waste lagoon containment barrier standard (Class A Landfill in accordance with NEMWA Regulation 636, published in August 2013). This barrier design addressed seepage, but required the EBS to mitigate the risk of diffusion of VOCs and hydration of the primary GCL. Figure 2 is a diagrammatic explanation of the barrier system and the working principle of the EBS.

The most important design aspect, to ensure successful operation of the system, is that of even airflow throughout the facility. Each facility is unique, and the site-specific parameters, as well as the composition of the barrier system components, have to be taken into account at the start of a design.

To ensure even airflow through the facility, as well as to optimise the size and power consumption of the mechanical extraction fan, the facility is

divided into compartments. The width of the compartments is calculated based on the pressure drop over the corresponding flow section to ensure evenly distributed flow, as well as taking into account the width of the geomembrane sheets to ensure a practical design.

Consideration was given to the pre-hydration means of the GCLs in the primary and secondary composite liners, the rate and direction of the advancing wetting front, and the direction of potential pollutant migration. This led to the selection of different GCLs for the enclosed primary barrier and for the secondary barrier applications.

The vacuum induction system, with inlets and outlets, was designed to make use of readily available fittings and vacuum pumps.

The system is designed to induce condensation at the fluid inlet to assist with the rapid hydration of the primary barrier GCL.

Construction

The earthworks construction commenced in mid-2013, and the liner installation in November 2013. The project was successfully completed in November 2015.

Earthworks preparation

General geomembrane-lined earth dam construction principles were applied, which included engineered sloping walls and floors, with a regular slope towards the lowest point of the dam with subsoil drains, standard compaction requirements and surface finishes.

EBS installation

Standard geomembrane installation practice was followed, giving attention to specific design principles, and thermal expansion and contraction wrinkles.

Anchor trenches were adapted to accommodate the fluid extraction system. Timeous backfilling of trenches was important to prevent any localised tension in the geomembrane due to thermal expansion and contraction.

Pipe connections to the geomembranes, as well as the installation of other piping components, were of critical im-

Table 1: Cause of holes vs size of holes (Noske & Touze-Foltz 2000)

Size of holes (cm²)	Stones	%	Heavy equipm	%	Welds	%	Cuts	%	Worker directly	%	Total
< 0.5	332	11.1	-	-	115	43.4	5	8.5	-	-	452
0.5–2.0	1 720	57.6	41	6.3	105	39.6	36	61.0	195	31.3	2 097
2.0–10	843	28.2	117	17.9	30	11.3	18	30.5	36	10.7	1 044
> 10	90	3.0	496	75.8	15	5.7	-		-		601
Amount	2 985		654		265		59		231		4 194
Total	71.2%		15.6%		6.3%		1.4%		5.5%		100%

Figure 6: Electric leak detection (ELD) in progress

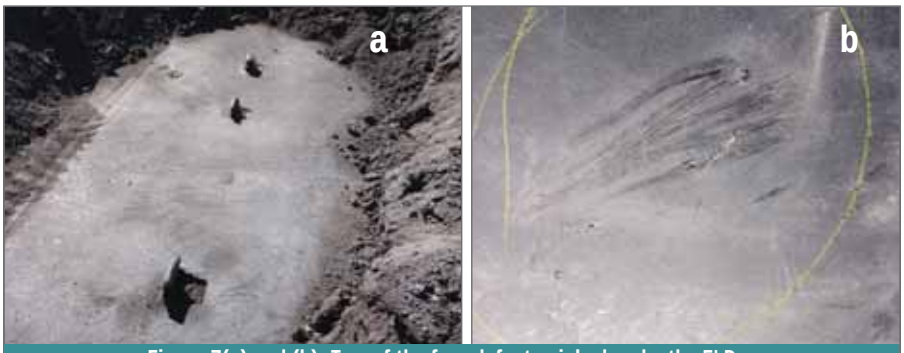

Figure 7(a) and (b): Two of the four defects picked up by the ELD

Figure 8: Airflow measurement and calibration at one of the air intakes

portance to ensure proper performance of the system and the effective separation between compartments. Particular care was taken to ensure that the installation fully met the design requirements.

Quality control

Being the first large-scale installation of this innovative technology in the world, extensive quality control and diligent construction supervision were imperative to ensuring the successful outcome of the project.

Strict quality control was enforced by auditing both the geomembrane manufacturer's facility in Germany and the pipe manufacturer's facility in South Africa to ensure that materials were supplied according to specification. Apart from this, third-party quality control testing was conducted on the various materials to ensure conformity to the project specifications.

Aquatan's participation since 2012 in the rigorous Certified Welding Technician (CWT) and Approved Installation Contractor (AIC) programmes, administered by the International Association of Geosynthetic Installers (IAGI), has considerably enhanced the company's technical and quality control skills, which were put to good use on this project.

From a construction risk management perspective the EBS part of the project consisted of five major processes:

- Earthworks preparation for geosynthetics
- Earthworks preparation for piping
- Geosynthetics installation
- Piping installation
- Capping operation.

As with any liner installation it was shown that risks of defects to barrier systems resulted from stones (71.2%) and heavy equipment (15.6%) (Table 1 refers).

As a result of this, Aquatan performed post-construction quality control by conducting an electric leak detection (ELD) survey on the completed facility, utilising the dipole method.

Despite the fact that there were spotters present during the process of capping the liner, four defects were picked up by the ELD process. All four defects were repaired (see Figure 7). Had it not been for the ELD procedure, the performance of the barrier and the EBS would have been compromised.

After construction the facility was filled with hazardous waste sludge and the leak detection system monitored for performance. This confirmed the exceptional value of the EBS in improving the post-construction performance of contaminant containment barrier systems.

CONCLUSION

As a result of effective collaboration between all project stakeholders, innovative engineering, quality workmanship and stringent construction supervision, the world-first enhanced barrier system (EBS), an innovative technology in the geosynthetics industry, was successfully installed on this 13 hectare hazardous waste sludge lagoon. The EBS successfully performs its function to hydrate and keep the GCL in a hydrated state, as well as preventing diffused VOCs from negatively impacting the groundwater regime. Figure 8 shows the airflow measurement at one of the air intakes located along the perimeter of the facility.

REFERENCES

References are available from the editor. □

Geosynthetics – from product to technology in road rehabilitation

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INTRODUCTION

Before the launch of the new *South African Pavement Design Method* (at the time of writing), extensive research was undertaken to evaluate current road pavement material performance (design versus long-term actual performance) in the development of new materials. However, very little interest was shown in a technology which has been proven to benefit pavements by an unequalled value of up to 10 times normal traffic load, thereby allowing a reduction in layer thickness of up to 50%. This means a Category C (ES-0.01 to 0.1) road becomes a Category B road. Applying this technology greatly reduces the cost of construction where a G1 or a cement-stabilised material would have been used.

This technology, which is known as geosynthetics, has a record of more than 30 years of proven results and efficiency in practice. Some new materials currently used in roadworks do not have proven records over such a time span. Although currently classified as materials or products, it is increasingly believed that geosynthetics in fact represent a new technology. Geosynthetics technology is the result of thorough research, field tests and calibration towards the development of a strongly-based formulation for designing.

Geosynthetics technology was recently successfully used in a road rehabilitation project currently under construction near Glentana in the Southern Cape.

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MR348 – MORRISON ROAD REHABILITATION NEAR GLENTANA

Kantey & Templer Consulting Engineers were appointed by the Road Network Management Branch of the Western Cape Government to design and supervise the rehabilitation of Main Road 348 near Glentana. The investigation of the road revealed that major deep-seated deformation/settlement had taken place over certain sections. A geotechnical investigation, using dynamic probe super heavy (DPSH) testing, indicated the presence

of a deep (up to 8 m in certain locations), soft, low-strength subgrade. SPT N-values as low as 1 were recorded in certain locations, due to penetration generally occurring under self-weight of the equipment with no drop weight activation required, indicating a very poor subgrade.

From test pits the subgrade material was classified as sand containing decomposed organic material.

Various pavement rehabilitation options were investigated during the design stage of the project. This included the possible removal and replacing of the poor subgrade material, but this option was found to be impractical, considering the depth of the poor subgrade and the restrictive environment. Other options investigated included the use of micro-piling, but, due to its high cost, this option was also not considered viable. The rehabilitation strategy that was found to be the most cost-effective was a ground-stabilisation technique using geosynthetics to reduce the overburden pressure caused by the increase in pavement thickness.

BACKGROUND OF GEOSYNTHETICS IN PAVEMENT

The first recorded projects where geosynthetics were used in pavements were in the 1930s in test sections on highways in South Carolina, Rhode Island, Montana and New Jersey. This was done by the United States Department of Agriculture in collaboration with the United States Bureau of Public Roads (forerunner of the Federal Highway Administration). From the records, it appears that these tests were very successful. The technology, however, lay dormant until the early 1970s when, in Scotland and North America, test sections were constructed to supply information on how geotextiles would perform in roadways. There was a growing interest in this application by geotextile manufacturers. The first design procedures were published by John Steward in the 1970s, followed by Giroud and Noray in 1981. In the early stages the main function of geotextiles was to protect good-quality imported material from being contaminated by poor-quality in-situ material. Without geotextiles, the contamination of the imported material in reality resulted in a percentage increment of its design thickness (Figure 2).

With the development of geosynthetics, such as woven geotextiles, and in particular geogrids characterised by high stiffness, a new concept was introduced – the reinforcement of soil, where the geosynthetics were able to cater for tensile strength development in soil. This resulted in the soil, in combination with the geosynthetic, being able to sustain higher stresses, enhancing the mechanical properties of the soil up to four times more, as shown in Figure 3 through a triaxial test on silty sand.

The inclusion of geosynthetics reinforcement therefore allowed a reduction in the thickness of structural layerwork (thus shallower box cut profiles), increased design life and reduced overall construction time (Figure 4).

The structural integrity of the imported layers is increased by the mechanical bonding between the soil particles and the geogrids. Both gravel and surfaced roads can benefit by the introduction of geogrids. Different design strategies should, however, be considered, such as the Leng-Gabr design method for gravel roads where the influence of the geosynthetics is based on the stiffness of the geotextile or geogrid, and taking into consideration the interlocking effect.

Figure 1(a): DPSH testing on road subgrade

Figure 1(b): Subgrade consisting of sand containing decomposed organic material

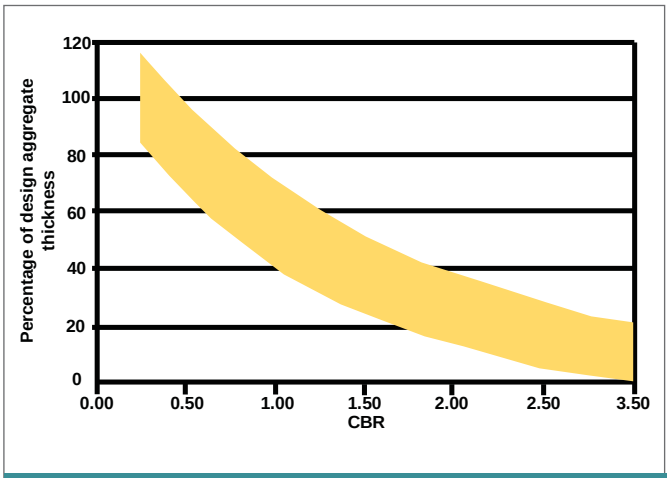

Figure 2: Aggregate loss due to weak subgrade

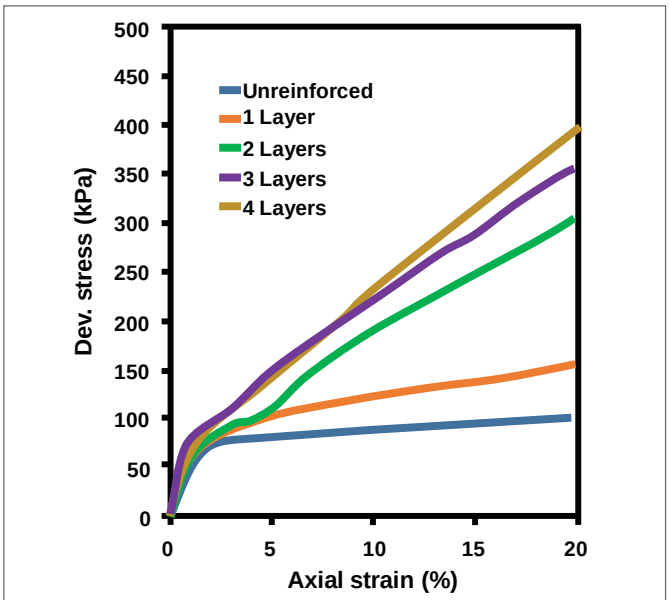

Figure 3: Triaxial test on reinforced silty sand

THE MODIFIED AASHTO 1993 INCLUDES GEOGRIDS

For surfaced roads, the formula determining layer thickness for AASHTO 1993 was modified to include the performance of geogrids.

The structural contribution of MacGrid EG geogrids on a flexible pavement system can be quantified by the increment of layer coefficient in the aggregate base and subbase as follows:

$$SN = a_1 \times D_1 + LCR \times a_2 \times D_2 \times m_2 + LCR \times a_3 D_3 m_3 + \ldots$$

where LCR is the layer coefficient ratio.

The LCR value is determined based on the results from laboratory testing on flexible pavement systems, with and without geogrid:

$$LCR = \frac{SN_r - \alpha_1 * D_1}{SN_u - \alpha_1 * D_1}$$

The SNs (structural numbers) of the reinforced and unreinforced sections are both evaluated under the same pavement conditions, i.e. same base course depth, subgrade CBR (California Bearing Ratio) and rut depth. In Figure 5 LCR values are shown versus the CBR of the subgrade applicable to Maccaferri MacGrid EG, obtained by interpolation of curves available in literature.

The LCR graph shows the improvement in the subgrade CBR when a geogrid is introduced. For high-quality soils (high CBR values) the improvement is constant. However, when the soil has a low strength (CBR less than 5), the improvement to the soil as a result of geosynthetics increases.

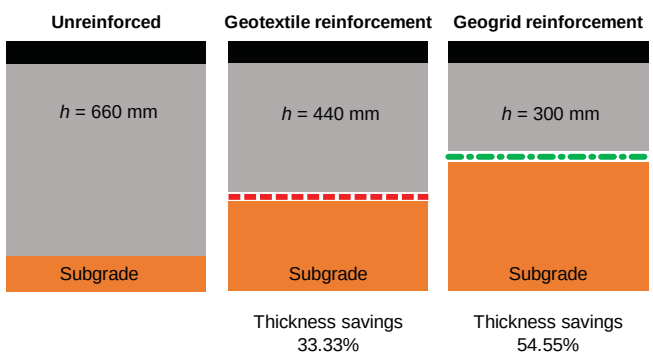

Figure 4: Reduction in layer thickness using geotextiles and geogrids

THE DESIGN OF MAIN ROAD 348 NEAR GLENTANA

One of the main design criteria was to maintain an undisturbed stress state in the soft, poor subgrade material to avoid deformation and resultant failure.

Traditional design, using SAMPDM and considering a Category B road as per TRH 4 with an ES3 (3 million ESAL), resulted in a total pavement depth of 1.2 m as shown in Figure 6.

Subsequently the AASHTO model was calibrated to match the SAMPDM in an unreinforced scenario (no geogrids present) in order to have a consistent design (not included in this article). Note that the 250 mm C3 subbase was replaced by 2 x 150 mm G4. Table 1 refers.

The results from the model are shown in Figure 7 where two geogrids were placed, one in the G7 and one in the G4 base, reducing the excavation from 1.2 m to 0.7 m, resulting in a no-stress variance in the soft layer (which would have failed due to the overburden pressure caused by the extra layer thickness), as well as maintaining the same road surface level, which was paramount, due to the main intersections and road annexures.

THE CONSTRUCTION OF MR348 NEAR GLENTANA

Construction commenced in March 2014. Stormwater reticulation along the edges of the road prism was upgraded before

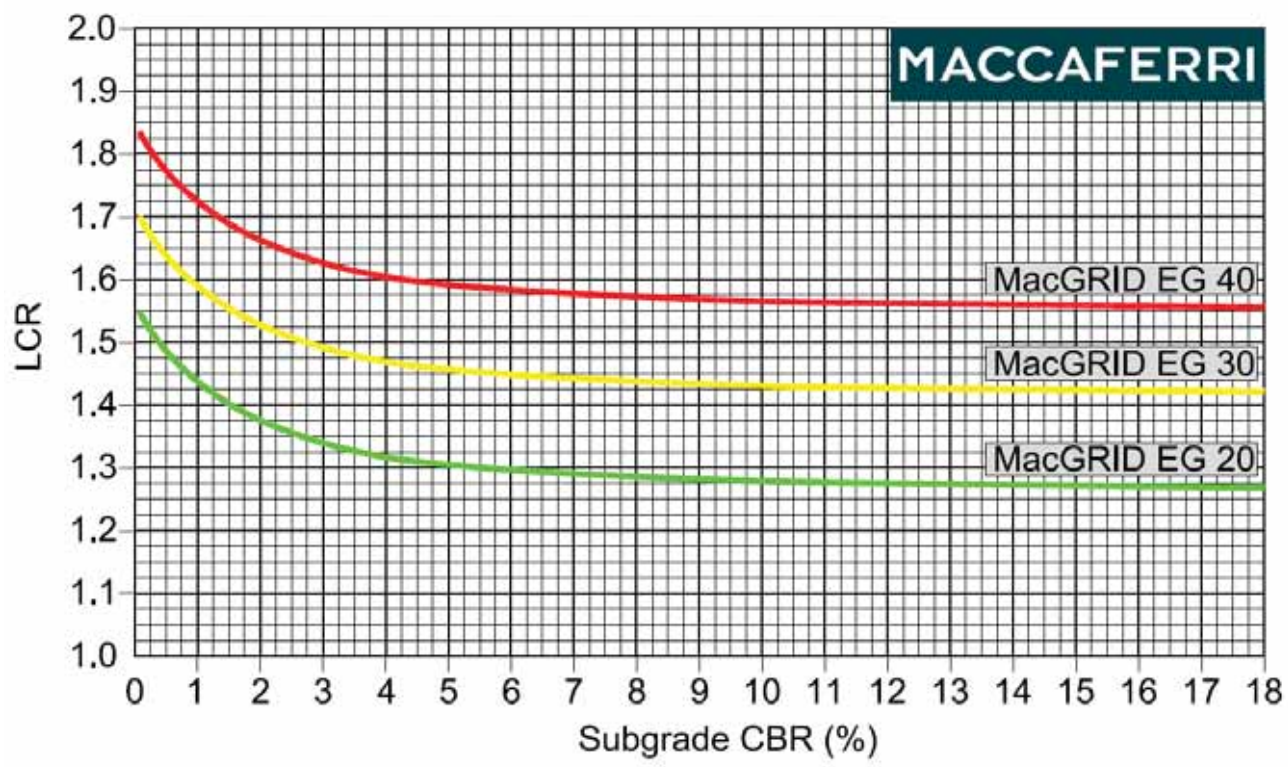

Figure 5: Layer coefficient ratio for Maccaferri EG geogrids vs subgrade CBR

Table 1: AASHTO parameters

Layer	Classification TRH 14	AASHTO	
		Layer coeff – *a*	Drainage coeff - *d*
Surface layer	not considered		
Base layer	G2	0.18	1
Subbase layer	G4	0.16	1
Selected subgrade layer	G7	0.06	1
Subgrade	CBR 0.5		

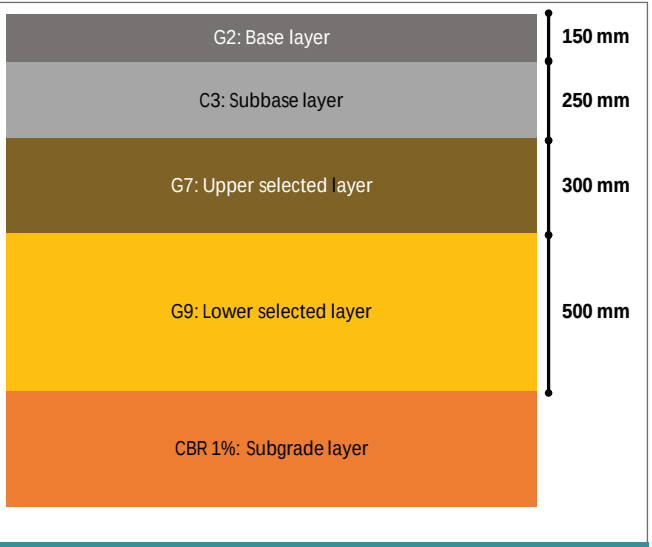

Figure 6: TRH 4 pavement structure

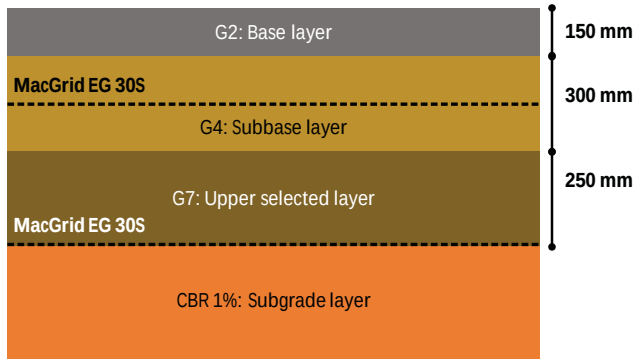

Figure 7: AASHTO design improved with geogrids

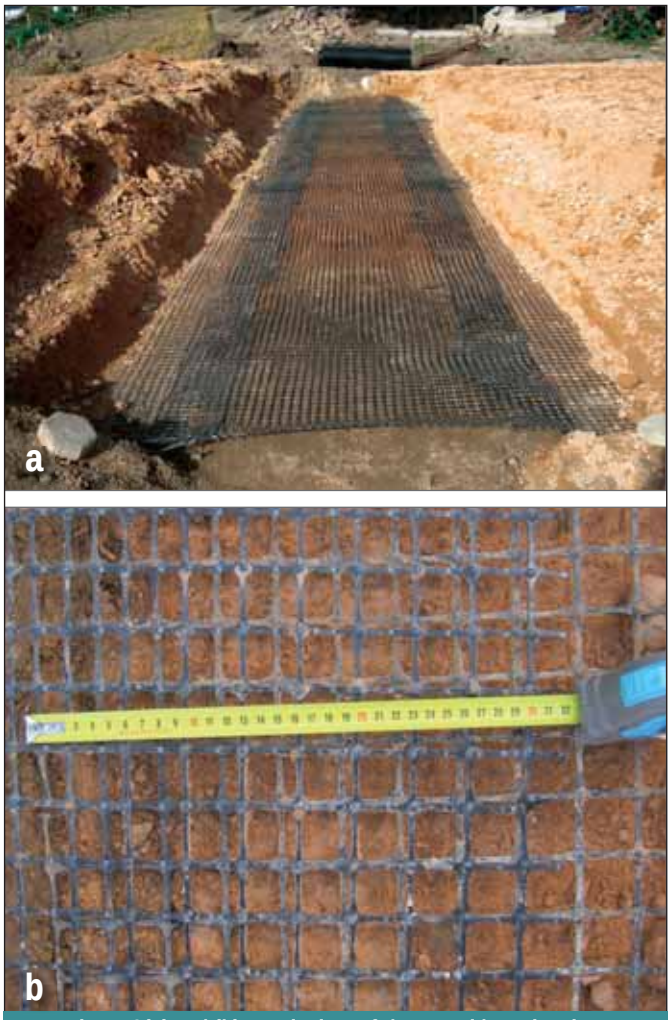

Figure 9(a) and (b): Particulars of the geogrid overlapping

Figure 8: First geogrid placed beneath the G7

Figure 10: Project successfully completed

proceeding with the layerworks. Due to traffic constraints, the contractor was initially only allowed to work half widths, but he requested access to the full width to reduce the construction time. The request was accepted by the client, and traffic was redirected through the municipal area. By constructing in full width, the contractor was able to lay down the geogrid in one operation, minimising jointing and effectively using only three roll widths to cover the road prism. The first geogrid layer was placed on the road box cut and then covered with a G7 subgrade (Figure 8).

Particular attention was given to the jointing of the geogrid. A minimum overlap of 300 mm was required to ensure that the tensile forces in the geogrids would be transmitted through the layerworks (Figures 9(a) and (b)).

The second geogrid layer was placed between a lower and upper G4 layer. A G2 base course was then constructed and sealed, and the road opened to traffic. In total, 40 000 m^2 of geogrid were placed in the layerworks. Most motorists will use this road oblivious to the fact that it has not been constructed by conventional methods, and that the road prism was constructed by a more efficient method, saving time and cost.

Subsequently many more projects across the country were completed where geosynthetics technology 'saved' the situation – from reinforcement of subgrade and subbase, to basal reinforcement of high soft-soil embankments, to asphalt reinforcement where enhancing the life of an overlay prevented reflective cracking.

GEOSYNTHETICS – PRODUCTS OR TECHNOLOGY?

The versatility of geosynthetics has been controversial, despite increasingly rigorous research and the improvement of the product and its application in less than 30 years. Traditional engineering tends to view the concept as 'too good to be true'. The answer is probably somewhere in the middle, as it will take time to digest the incredible results that geosynthetics have achieved so far. Research has in fact proved that this concept is not being used to its full potential yet. It is certainly true that a geosynthetic material – be it a geogrid, a geotextile or a mat – is a product that, on its own, will not create any interest. It needs to be supported by research, field testing and analytical calculations. From that perspective it is therefore not just a product, but real current technology, which is amazingly applicable to many engineering fields, such as landfills, water retention, walls, erosion control, coastal protection, and of course roads, as has been illustrated in this article. ◻

Due to traffic constraints, the contractor was initially only allowed to work half widths, but he requested access to the full width to reduce the construction time. The request was accepted by the client, and traffic was redirected through the municipal area.

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Shear beauty of the Franschhoek Pass

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SUMMARY

The Franschhoek Pass (R45) is one of South Africa's iconic mountain passes, serving as a gateway to the Overberg. The year 2013 saw numerous cases of slope instability along the route due to unseasonably high rainfall. In response, SMEC South Africa (Pty) Ltd was tasked by the Western Cape Department of Transport and Public Works with repairing damaged drainage infrastructure and road surfacing, implementing erosion-mitigation measures and stabilising a progressive, deep-seated slope failure on the lower western flank of the pass. The solution involved the use of an unconventional slope stabilisation method synchronous with the surrounding environment (Figure 1).

INTRODUCTION

The history of the Franschhoek Pass, which is a vital link between the small towns of Franschhoek and Villiersdorp, stretches back to the early 18th century when a route over the Middagkransberg was sought by Lord Charles Somerset,

Figure 1: Overview of the primary slope stabilisation works set against a backdrop of the Franschhoek Valley

the governor of the Cape Colony from 1814 to 1826. Completed in 1825, the Franschhoek Pass is one of South Africa's oldest roads, along which South Africa's oldest operational bridge traverses Jan Joubert's Gat. Legend has it that a route over the Middagkransberg was cut into the mountain side following a popular elephant migration path, dubbing the route *Oliphants Pad* for almost 150 years.

Whether accurate or not, it was these cuttings that were a major cause for concern almost 190 years on, following unseasonably high rainfall in the region during the latter half of 2013. Surficial instability in the form of ravelling and rock falls was prevalent, resulting in a breakdown of the road drainage infrastructure, severe scouring, and, in a gully on the lower western flank, a deep-seated slope failure.

Remediation measures to reinstate the safety of the route and mitigate the risk of future instability-related problems were required. These comprised:

- The stabilisation of a scoured embankment (Figure 2) by means of an anchored gabion wall
- Steel meshing of several cuttings to mitigate future ravelling
- Reinstating and improving sections of the route's road drainage, including dissipater structures, culverts and debris barriers
- Shear reinforcement, by means of soil nails and high tensile strength steel wire mesh, of the progressive deep-seated slope failure along the lower western flank (the primary focus of this article).

AN UNCONVENTIONAL APPROACH

In the design for the remediation of the deep-seated failure on the western flank, the need to avoid the use of unsightly concrete anchoring blocks and retaining walls required an unconventional and multi-faceted approach, as outlined below (and illustrated in Figure 3).

- Optimisation of the slope geometry to reduce the slope's gradient, while not compromising the slope's stability through disturbance to the toe of the slope – this resulted in a shallow lower slope and a steeper upper slope.
- Tecco® high-tensile strength steel wire mesh, four times stronger than normal steel wire mesh, was used together with large steel anchor plates on the lower portion of the slope where anchor loads were greatest.

Figure 2: Unseasonably high rainfall resulted in rock falls, erosion and scouring along the Franschhoek Pass in November 2013

Figure 3: A soil nail and high-tensile strength steel mesh configuration was used to stabilise the lower reaches of the slope undergoing a progressive, deep-seated failure. Dyed sprayed concrete and soil nails were used on the upper reaches. Erosion control mats and 12 m inclined purpose-made subsoil drains were used to control seepage

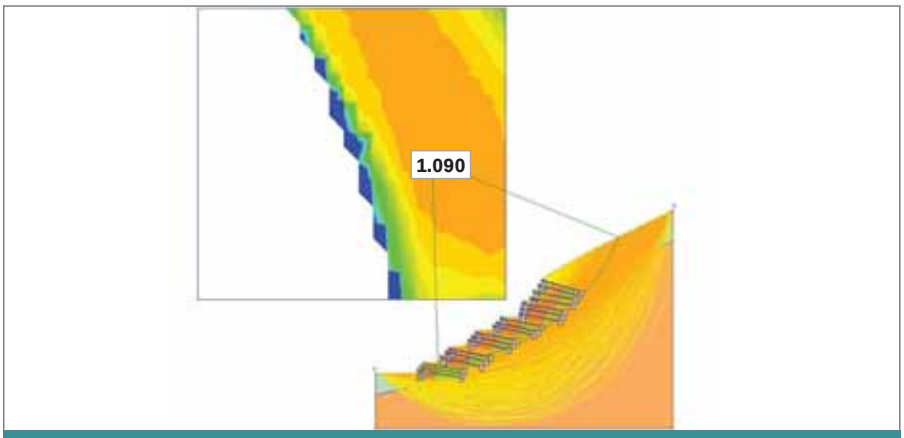


Figure 4: A combination of limit equilibrium and finite element method modelling techniques were used to evaluate overall slope stability, as well as the structural capacity of the soil nails and high-strength steel mesh based on different soil nail spacing, diameter and lengths, groundwater conditions and external loads; this was carried out in accordance with SANS 10160

- A hydro-seeded erosion control mat was placed between the subsoil and high-strength steel mesh to minimise erosion and to aid in the timely revegetation of the slope.
- On the upper, steeper portion of the slope, sprayed concrete was required. By using various dyes the mottled effect closely matched the surrounding soils, thus providing an aesthetic, pseudo-natural finish.
- The surface and subsurface drainage was improved, thus reducing pore-water pressures acting on the failure plane. To this effect 12 m deep perforated-pipe-and-geotextile drains were inserted at regular intervals at several different levels into the slope at an inclined angle. The surface drainage was improved by providing a cut-off trench over the crest of the slope, and by providing benches to the slope with drainage channels on each step to collect and rapidly discharge the collected water.

In order to ensure that the mesh-and-soil-nail solution worked as intended – by distributing the high anchor loads anticipated into the mesh – detailed analyses of both the geotechnical and structural elements were required. This was particularly important, as this system is conventionally used for shallow superficial failures and thus would result in over-stressing of the mesh if poorly designed. In addition, as the load deflection properties of the mesh are specific to the site conditions, nail spacing, mesh-soil interaction and the associated yield stresses, detailed geotechnical and structural analysis were required to ensure that the design would function as intended.

The geo-structural components were modelled utilising Limit Equilibrium (LE) and Finite Element Method (FEM) software to ensure the integrity of the chosen system, and that each component integrated with the overall geotechnical solution. The nails were spaced and sized according to these results. Results of one of the Ultimate Limit State (ULS) analyses are illustrated in Figure 4.

Thereafter, structural FEM modelling of the mesh was undertaken to evaluate the yield characteristics, considering the specific soil interface at specific applied loads, whilst varying the spike plate sizes and nail spacing.

This in-depth numerical modelling facilitated the application of an established surficial rock support system to that of a global soil instability problem. However, this was not without its challenges, most notably the numerical modelling of matric suction and partially saturated soil conditions – a common phenomenon in decomposed quartzite, granite and talus slopes – some details of which are presented in the following section.

SOME NUMERICAL MODELLING CHALLENGES

Groundwater is one of the main factors responsible for slope failures. However, quantifying its influence accurately can be challenging. During extended periods of rainfall, the groundwater table rises, leading to an increase in pore-water pressure and an associated loss in the shear strength of soils. The converse is true for completely dry soil fabrics. However, this behaviour relates to just two extreme limiting conditions of a soil.

The traditional soil mechanics approach was inadequate in describing the behaviour of the slope failure, due to its partially saturated nature and the central role that matric suction plays in such scenarios.

The effect of matric suction is depicted graphically in Figure 5 in the extended Mohr-Coulomb plot whereby

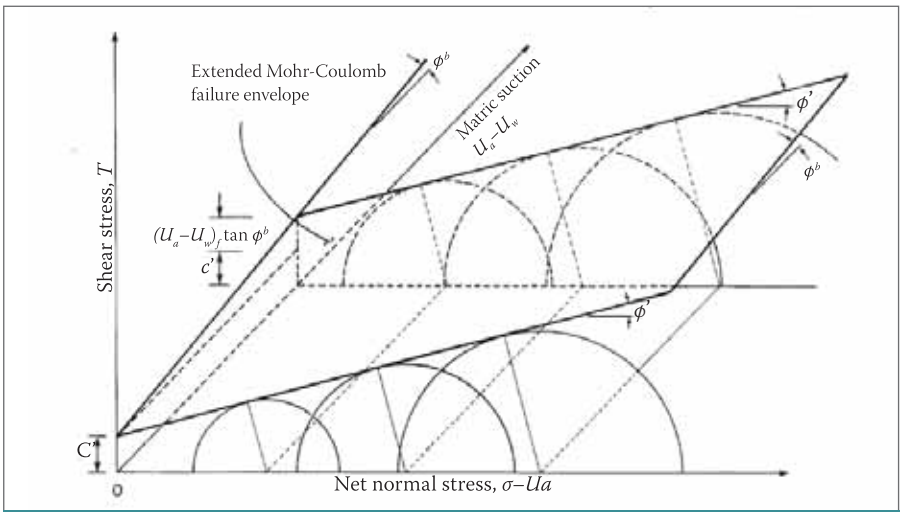


Figure 5: Extended Mohr-Coulomb failure envelope for unsaturated soils (Fredlund 2012)

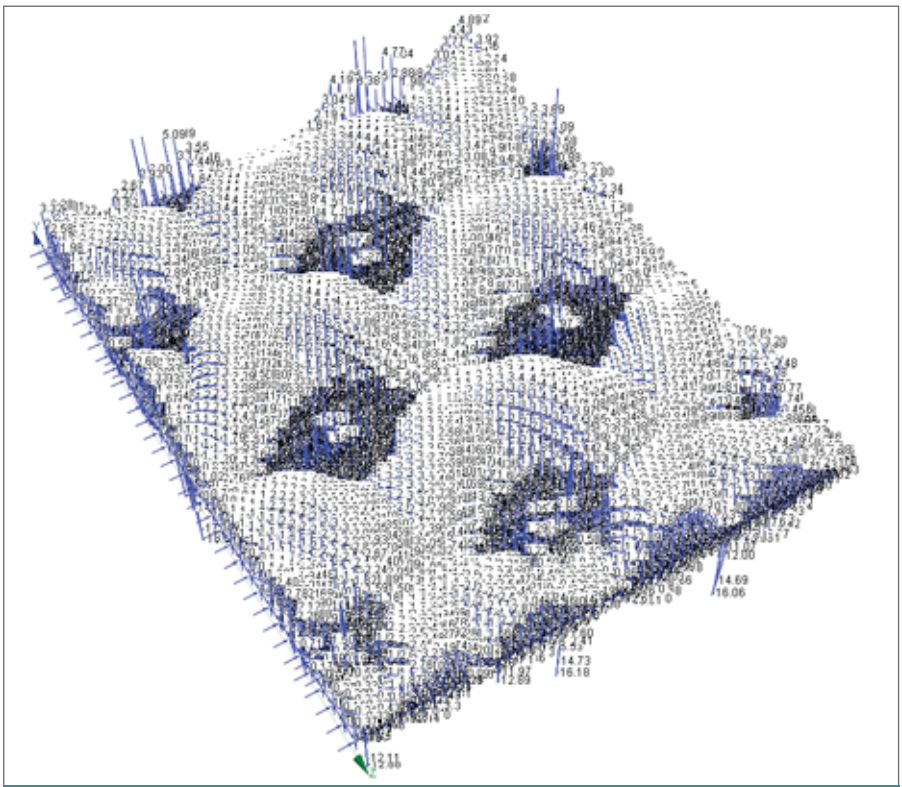


Figure 6: Structural FEM analysis incorporating pressures from the geotechnical FEM analysis

$(u_a - u_w) \cdot \tan(\phi^b)$ results in an increased apparent cohesion and associated increase in shear strength. u_a, u_w and ϕ^b denote the respective atmospheric and pore-water pressures, and ϕ^b the unsaturated shear strength angle. This influence is often ignored during design due to the nature of u_w to fluctuate, through infiltration of groundwater, over the service life of the slope.

Based on the results of back analyses and sensitivity evaluations, the drainage measures implemented on the remedial slope were designed specifically to minimise infiltration of groundwater. Following this, various FEM seepage models, incorporating soil nails and mesh, were constructed to assess the effect of the drainage measures on the water table. Thereafter matric suction functions, incorporating the Soil-Water Characteristic Curve (SWCC), were used in the FEM analysis to assess the increase in stability due to negative pore-water pressures. The slope geometry was optimised further to reduce the effect of overburden pressures exerted on the high-tensile strength steel mesh and to ensure an extended drainage profile (Figure 5).

After assessing the effect that matric suction has on reducing the lateral pressures, the high-strength steel mesh was modelled in structural FEM software to assess the moments that would develop in the mesh, as shown in Figure 6. This was an iterative process which involved revisiting the geotechnical FEM models and changing the nail spacing to reduce pressures, and the associated moments, to acceptable levels. In other parts of the world the mesh has been doubled up where moments cannot be controlled sufficiently.

DRAINAGE INFRASTRUCTURE AND ANCILLARY WORKS

A number of stormwater inlet and outlet structures were damaged during the flood events. These were redesigned as follows, firstly to protect them from theft and vandalism, and secondly to make the standard provincial design purpose-suited to the particular challenges faced on the project:

- The inlet allowed by a standard polymer grid was too small and clogged easily. Once clogged, the water would overshoot the inlet, causing significant scouring of the road shoulder and side drains. This was overcome by redesigning the inlet with a narrow slot, and offsetting this within a channelled collector dish.
- Furthermore, rock falls and ravelling in the vicinity of the stormwater inlets were deemed to pose an elevated risk to the functioning of the inlets, as debris can overwhelm the inlet. Keeping the inlets open and operational was considered key to the functioning of the stormwater drainage system and minimising damage after heavy rainfall events. Subsequently, at identified inlets, the slopes above the inlets were stabilised and or protected from erosion, either through the provision of gabion-type structures, or more commonly by providing localised steel mesh.
- Dissipating structures were constructed in regions of concentrated flow to prevent scouring of inlet and outlet

structures and the subsequent undermining of the road pavement.

- Where sprayed concrete was necessary, this was coloured and shaped to blend in with the natural slope.

Retaining walls, with the primary purpose of preventing rock debris from infringing the road, were also required. These were cladded with packed sandstone rock on the road side to match the surrounding mountainside and the existing historic dry stone walling along the route. However, there existed a risk that the cladding could delaminate, especially after an impact. The cladding was therefore monolithically cast into the retaining wall.

Figures 7(a) to (c) illustrate a few of the drainage measures and other works implemented along the pass.

CONSTRUCTION CHALLENGES

Due to the urgent nature of the project, the contract was let out on the basis of a preliminary design for which certain variables would need to be confirmed once the contractor had established himself on the site. This emergency project thus required on-site design inputs during construction, with regular oversight or advice from specialist design engineers to assist in the on-site problem-solving. This was all overseen by the project manager/engineer who collaborated and kept in regular contact with the client to ensure that his interests were also met.

Notwithstanding this, a number of construction challenges were faced:

- With access to the face of the slope being difficult, the design solution was tailored for the use of light drilling equipment which could be operated by hand, using rope access and harnesses on the slope. This equipment had been developed by the contractor, Pennyfarthing (Pty) Ltd, on previous projects, and could therefore be tailored for use on this particular slope geometry and soil nail dimensions.
- As part of the hydro-seeding operations, small pockets were excavated into the face of the slope where seeds and topsoil could be placed, as the excavated slope was not an appropriate growing medium. The slope was subsequently hydro-seeded before a biodegradable erosion control mat

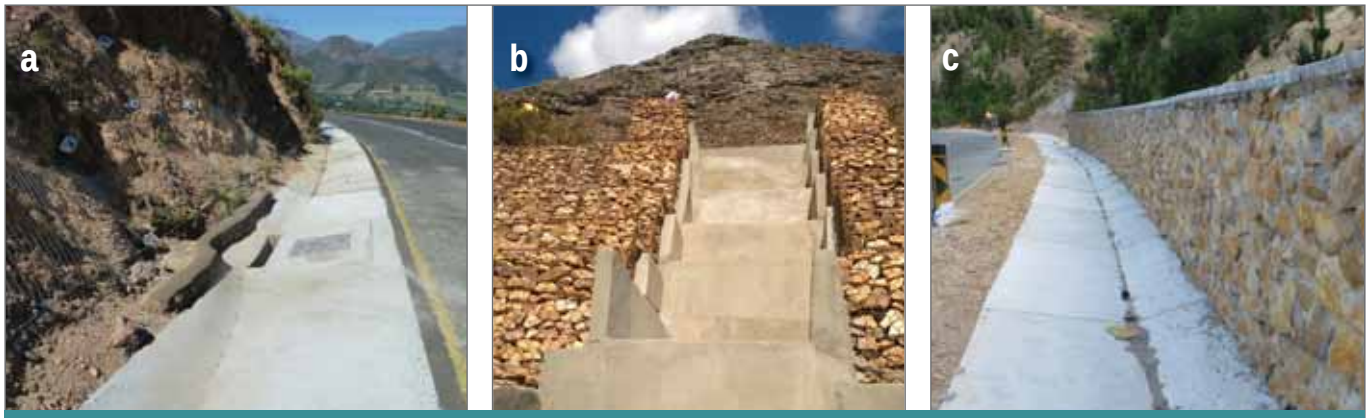


Figure 7(a), (b) and (c): Improved culvert inlets, energy dissipaters, and stone-clad retaining walls were a few of the measures put in place to improve the drainage of stormwater along the route

Figure 8: Tailored drilling plant and installation methods were used to ensure a safe and efficient construction sequence for the soil nails

was placed over the seeds, followed by the mesh.

- In one instance a natural gully on the slope needed to be protected from erosion, requiring geotechnical stabilisation. The high-tensile strength steel mesh and erosion control mat was not suited to high concentrated flows, so this was integrated subsequently with a bolted down reno-mattress to achieve a strengthened mesh facing and the flexible erosion solution offered by the reno-mattress.

Figure 8 illustrates the challenging working conditions, the novel drilling techniques and the plant that had been developed.

RISK MANAGEMENT

The project is highly visible and recognisable, and the design involved the use of a number of innovations in order to effectively mitigate, at a reasonable cost, a proportion of the risk associated with the frequent failures. Ultimately, as with most underground and geotechnically related projects, the solution cannot be both practical and risk free without becoming too robust and uneconomical. A gradation system was created whereby slope stabilisation and other interventions were focused on higher-risk and/or high-frequency areas.

The design team formed an integral part of the construction monitoring, thereby ensuring that designed elements were suited to the conditions, as some of these, due to the nature of the project, only became evident once areas were cleared or opened up during construction.

The final solution was a direct product of the quality of the design process and the adaptability thereof to suit conditions on site, coupled with rigorous quality assurance monitoring. This attests to the value of maintaining close contact between the geotechnical design team, the site personnel and the contractor throughout construction.

CONCLUSION

Geotechnical problems are inherently high in uncertainty, requiring solutions that are effective, but adaptable. The stabilisation of a progressive slope failure and drainage improvement on the Franschhoek Pass illustrated this clearly, and showed that a multidisciplinary design team and design approach, integrated with the contracting party throughout construction, can reduce this uncertainty, resulting in the delivery of a holistic solution.

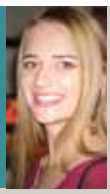
ACKNOWLEDGEMENTS

The authors would like to thank the Western Cape Department of Transport and Public Works, notably Mr Llewellyn Truter, for kindly allowing the publication of this article. George Kustner, project manager, Fernando Pequenino, geotechnical lead, and Johnny Neethling, engineer's representative, are also acknowledged for their valuable contribution to the delivery of this project. ◻

PROJECT DATA

Client: Western Cape Department of Transport and Public Works
Engineer: SMEC South Africa (Pty) Ltd
Contractor: Penny-farthing (Pty) Ltd
Value of works: R30 million

Port Louis Ring Road – design approach for post-failure stabilisation



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BACKGROUND

Over the past six years Mauritius embarked on major road and traffic infrastructure upgrade projects as part of solutions to improve road user safety, alleviate traffic congestion, boost the industry and facilitate economic growth, with added social and environmental benefits. Positive population growth and an increasing number of road users required construction of dual-carriageway roads capable of accommodating high volumes of traffic. Massive capital investments and acquisition of land were required for these projects, ultimately leading to the planning and alignment of these roads over very challenging topographical, hydrological and geological terrain and environments. The Ring Road Phase 1 project was constructed during 2010–2013, and comprises a 4.9 km dual-carriageway, and one large bridge over the St Louis River, with access roads to industrial and retail areas and the National Convention Centre. Terrain elevation and road alignment required the construction of several large cuts and fills. Space restrictions justified the implementation of large mechanically stabilised earth walls (MSEWs) in lieu of traditional fills.

In early 2014, cracks appeared on the northbound carriageway, followed by the collapse of a 15 m high MSEW portion of the fill (Photos 1 and 2). Observations pointed to a textbook slope failure with "slip at the lip" and "bulge at the toe", indicating that deep-seated movement had occurred. This article summarises the geotechnical investigation conducted to identify the failure mechanisms, the design of remedial measures, and the knowledge and understanding gained for use in future projects.

INITIAL EVALUATIONS

An evaluation of the MSEW, assisted by a finite element analysis, was conducted by

Photo 1: Initial cracking of the road surface on the northbound carriageway

Photo 2: Ultimate failure of the mechanically stabilised earth fill

ARQ (Pty) Ltd in April of 2014 to determine the cause of this failure:

- From this evaluation, it was concluded that the internal stability of the MSEW was satisfactory, and that failure had occurred behind and below the reinforced soil block.
- It was deemed possible (and probable) that a low shear strength seam, or soft, highly plastic strata were instrumental in the failure. These soft, highly plastic strata were commonly encountered in borehole profiles.
- Further testing and analyses were required to define the cause and mechanism of failure.

FIRST STAGE TESTING

Initially 25 boreholes were drilled independently and four viable samples were sent for triaxial testing. Sampling during drilling was conducted without full-time supervision of an engineering geologist or geotechnical engineer. Triaxial test results from the initial investigation indicated some inadequacies – cohesion was, for example, reported to be zero (it was considered highly unlikely that clay material would not exhibit any cohesion), the majority of Skempton's B parameters were < 0.95, and the initial slope of the stress path was 1:1, indicating that the sample was tested unsaturated.

GEOLOGICAL AND TOPOGRAPHICAL CONSIDERATIONS

Figure 1 shows the locality of the site along the Ring Road in Pailles taken from Google Earth®. The alignment follows the mountain and valley, and necessitated construction of several large cut-to-fill and valley-type fills. Via cursory inspection it crosses alluvial deposits along the St Louis River, other natural water courses and underground water seepage zones.

General geology

The majority of the site falls within the base of the Port Louis-Moka mountain range. The general geology of the site consists of variable degrees of weathered basalt, deeply weathered in some areas, overlain by a mixture of scree, talus, hillwash and colluvium, with particle sizes ranging from large boulders to a combination of both rounded and angular cobbles, pebbles, gravel and silt. Soil derived from completely weathered in-situ basalt is present as high plasticity clay with very low shear strength. Alluvial deposits are present on the valley floor. Founding material is therefore highly variable in size, shape, character and in terms of its engineering properties, presenting a considerable challenge for both designer and contractor.

SUPPLEMENTARY GEOTECHNICAL INVESTIGATION

In August 2014, ARQ conducted a supplementary investigation to obtain reliable geotechnical parameters and geologically accurate cross sections in the failed portion of the fill to be used in the design of remediation work. The investigation comprised the drilling of some six rotary core holes via triple tube to ensure maximum core recovery and minimum sample disturbance. A total of 24 standard penetration tests (SPTs) were conducted and 34 undisturbed cylindrical samples were obtained, of which 21 viable samples were air-freighted to South Africa for triaxial

Figure 1: Site locality and topography

and materials testing. Preservation and transportation of the samples were conducted according to ASTM D 4220-5.

Based on engineering parameters (strength and deformation characteristics), the profile within the failed zone was divided into four main horizons, namely:

- Imported/backfill FILL material
- Transported COLLUVIUM
- RESIDUAL basalt
- Basalt and/or breccia ROCK.

Laboratory results

Results of the triaxial tests were analysed in detail, such that representative shear and deformation parameters were deduced (see example in Figure 2).

This technique enabled any deviations from ideal to be determined, as well as representative Mohr-Coulomb, Duncan and Chang (1970) and Duncan *et al* (1980) hyperbolic parameters, to be derived.

FINITE ELEMENT EVALUATIONS OF FAILURE MECHANISM

Before remedial measures could be designed, the mechanism of failure needed to be identified and clearly understood. Figure 3 shows a section through the MSEW and underlying strata.

The yellow line shows the approximate zone of failure, occurring more or less parallel to the bedrock within the completely weathered basalt. The shape of the failure zone is therefore a combination of circular and planar, which confirms the initial observations made on site.

FINITE ELEMENT EVALUATIONS FOR REMEDIAL WORK

The failed area was interrogated and five representative sections were derived from an interpolated 3D model. Each of these sections was subjected to the Phase 2 Version 8 finite element software and analysed over some 13–16 construction steps which represented the envisaged construction process. The key parameters observed were:

1. Strength reduction factor (SRF). The aim was to reach an SRF = 1.5 for serviceability limit state (SLS) condition.
2. Pore pressure coefficient ru-value, which was assumed as 0.15 in the initial serviceability limit state analysis, but was then upped to 0.25 and combined with a seismic acceleration of 0.2 g in an ultimate limit state (ULS) case. The aim was to reach an SRF > 1.01 for ULS.
3. Axial force in the upper anchor position.
4. Tensile force in the high-strength geosynthetic.
5. Maximum moment in the pile.

The above are depicted in graphical format in Figures 4-1 to 4-4, for one of the cross sections.

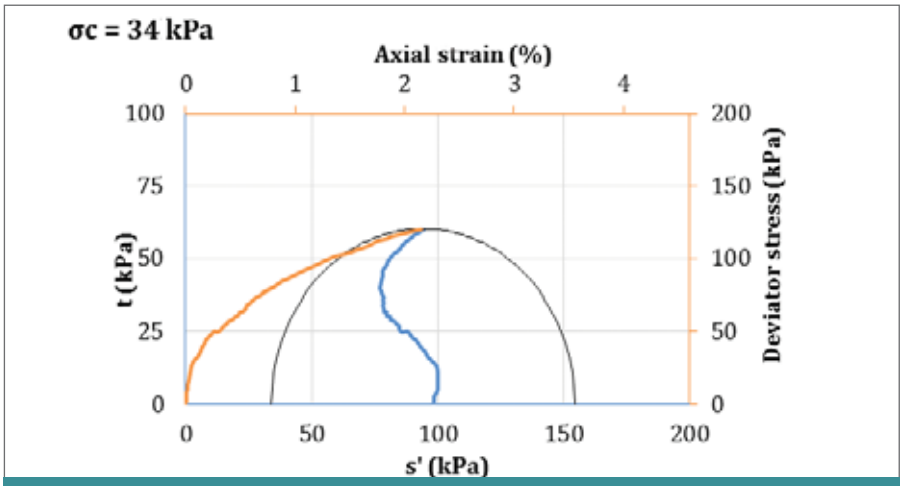

Figure 2: Graphical depiction of a typical triaxial test with (a) deviator stress/strain, (b) stress path and (c) Mohr circle

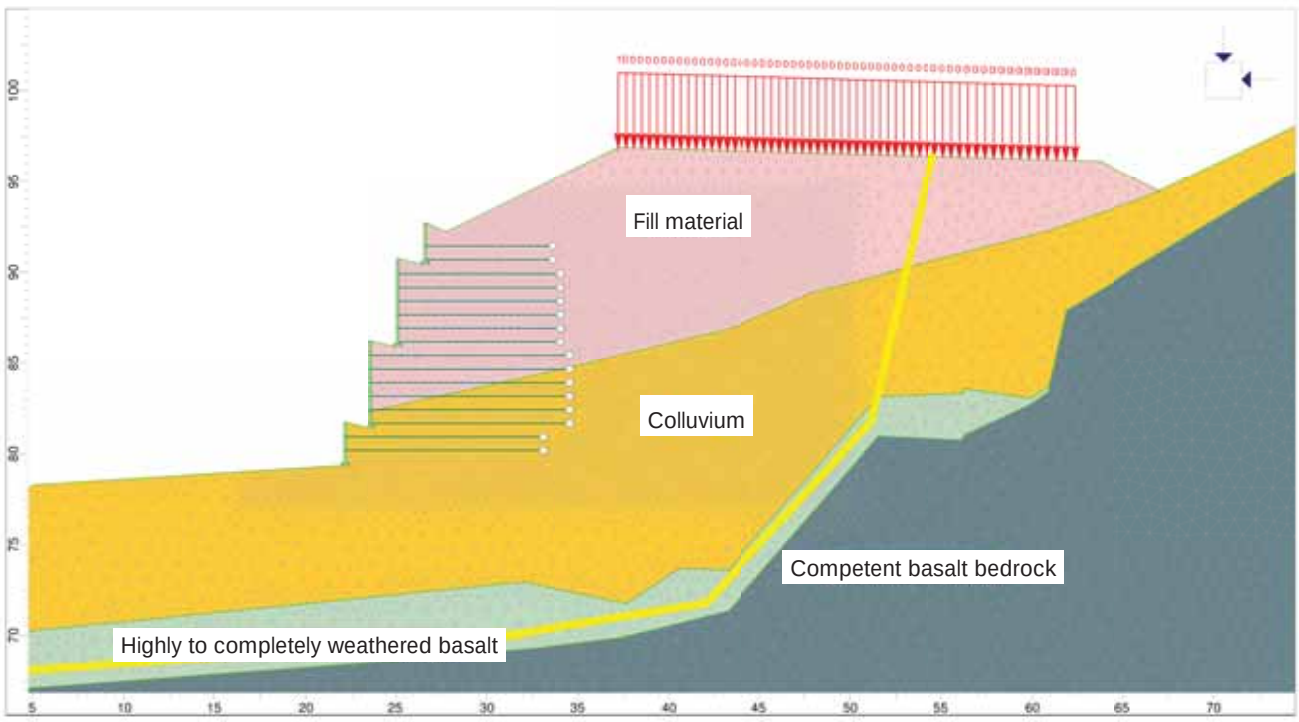

Figure 3: Cross section through mechanically stabilised earth fill prior to failure

RELIABILITY CHECK

Here the method of Duncan *et al* (2014) was used to determine a reliability index (RI). Twelve cases were evaluated, which required some six hours of computer time. This was, however, conducted as a batch process overnight, freeing up day usage. The results obtained can be summarised as follows:

RI (normal distribution) = 2.17
RI (log normal distribution) = 2.65
Probability of failure = 0.42%

Evaluation using this technique indicates that the log normal (LN) distribution is more correct, indicating (from Phoon 2008) "Average Reliability". It is interesting to note that work on the new RSA geotechnical design procedure assigns a number near 3 as a desirable RI. Although 2.65 is a little low, the redundancy generated by multiple piles, a robust capping beam and multiple anchors is deemed to add significantly to reliability.

These numbers indicate a very high reliability index, or a very safe structure.

HAND CALCULATION CHECKS

In order to assess correctness of finite element calculations, an order of magnitude check was made on (a) force in anchors and (b) moments in pile. These sanity checks indicated fairly good compliance, leading to a verification of the correctness of the finite element model.

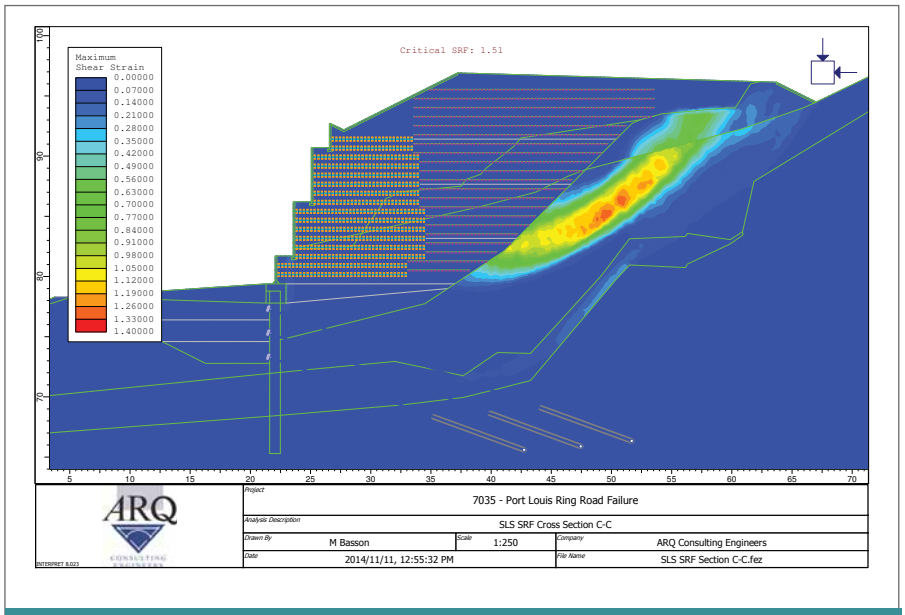

Figure 4-1: Critical SRF in the serviceability limit state (SLS)
(Here it is desired that the SRF should be 1.5. This condition is thus satisfied, if only just.)

BETTERMENT FOR FUTURE PROJECTS

- During conceptual design, consider the influence of the following characteristics on the final site selection: topography (like mountainous terrain), geological features, variability of and problem soil conditions, hydrological and geo-hydrological factors.
- Conduct a risk assessment on identified hazards.
- Perform appropriate type and level of investigations, based on the risks identified and the category and size of development. It could include from a desktop study or site walkover, to a more detailed investigation including in-situ or laboratory testing.
- More than one alternative should be considered for final site selection, together with careful consideration of the outcomes of the initial investigation.
- Depending on the scope of the initial investigation during site selection, conduct a well-planned investigation and allow for the presence of a specialist on site to oversee testing and sampling.
- Allow for specialist(s) input and oversight during construction, as it is not always possible to identify all ground conditions from the investigation, and ensure quality and accountability of the final product.
- Never underestimate valuable information and insight into site conditions that can be gained by consulting the local community.
- Depending on the project, allow for monitoring to identify any deviation from expected outcomes following completion of construction.

There is a famous saying amongst geoscientists: "You will pay for a geotechnical investigation, whether you have one or not", i.e. apparent savings at the start of a project may well lead to considerable effort, delays and additional cost later in the project cycle. A lesson identified, only changes into a lesson learned once a change in personal or operational behaviour is made.

As civil engineers we must ensure due diligence, albeit via geotechnical, topographical or hydrogeological investigations for both large and small infrastructure projects. Not only is it our duty and responsibility, but our communities and society will be stronger for it.

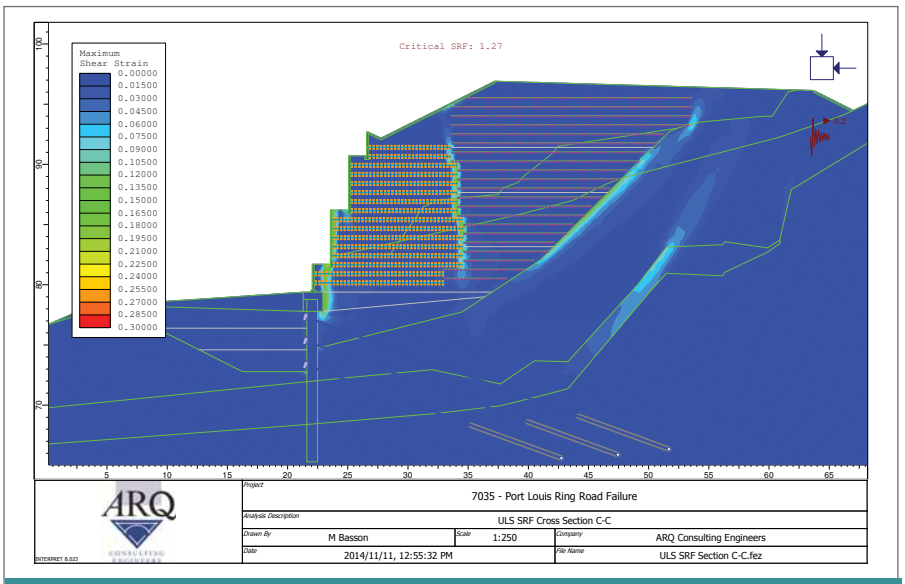

Figure 4-2: Critical SRF in the ultimate limit state (ULS)
(Here it is desired that the SRF should be > 1.01. This condition is thus also satisfied.)

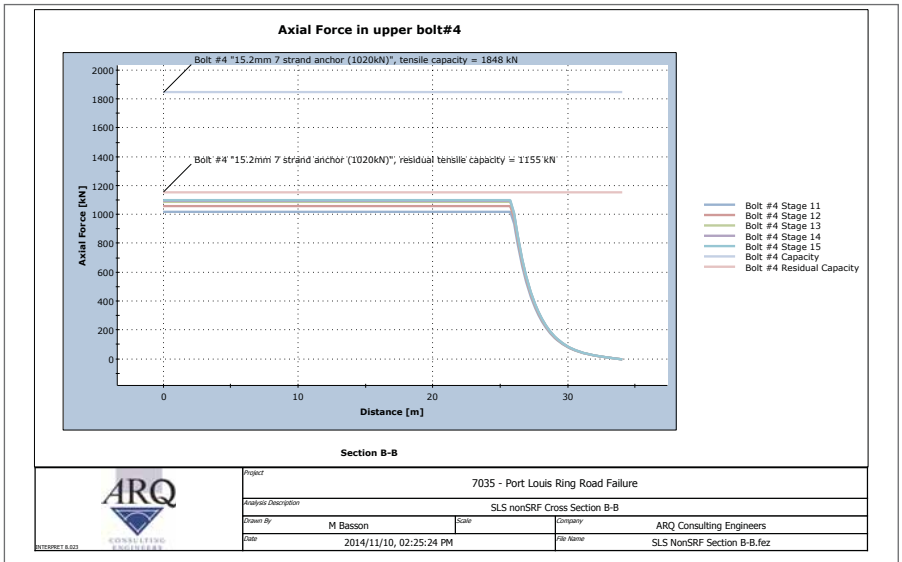

Figure 4-3: Forces in the anchors
(The anchor/bolt force should be < 65% of the ultimate value = 1 200 kN. This condition is thus satisfied, as all predicted forces are < 1 200 kN.)

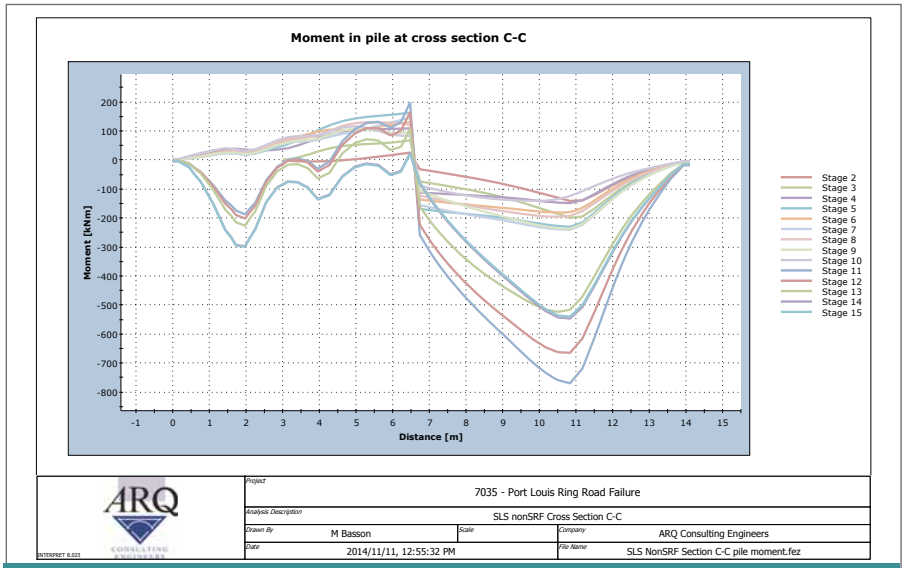

Figure 4-4: Moments in the pile
(The maximum moment in the pile should be < 1 200 kNm for the steel which is detailed as 18 Y32. This condition is thus satisfied.)

SPECIAL ACKNOWLEDGEMENTS

For their support and assistance, sincere appreciation and gratitude go to:

- Fellow geotechnical engineer and colleague Katy O'Brien of ARQ
- Our client and main contractor, REHM Grinaker: in particular Mr Nazir Korimbocous, Mr Tawfick Raymode and Mr Anwar Ramdin (MD).

REFERENCES

The list of references is available from the editor.

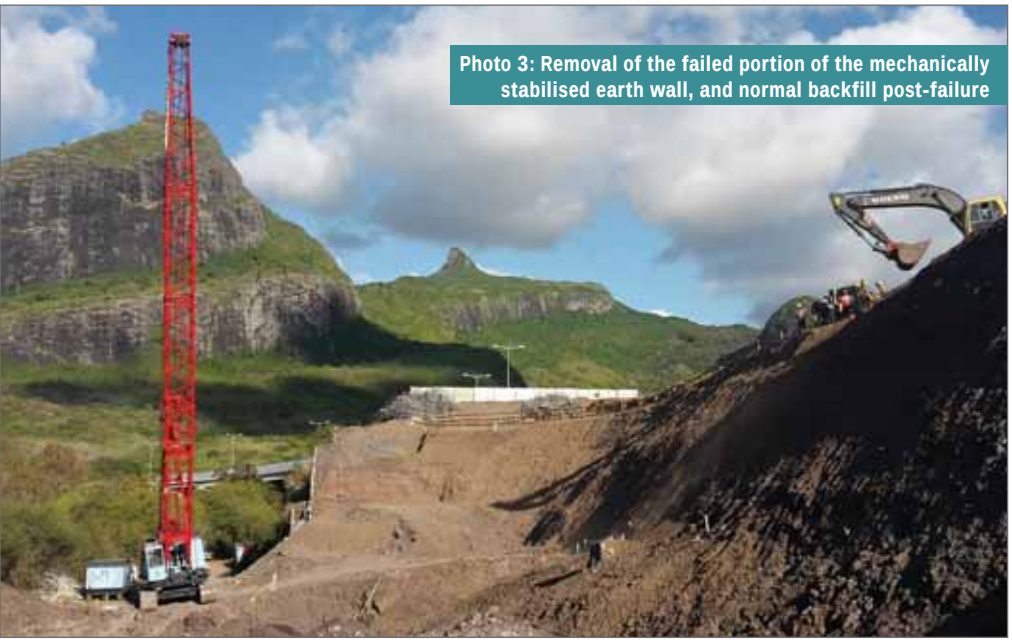

Photo 3: Removal of the failed portion of the mechanically stabilised earth wall, and normal backfill post-failure

Photo 4: Construction of the pile guide beam

N2 upgrade – taming the uMdloti

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The first phase of the upgrade of the N2 Freeway Section 26 has seen extensive geotechnical works. A feature of this phase of the project included the widening of the dual uMdloti River viaducts. The viaducts are founded on 45 m deep piles that required subcontractor Keller-Franki to import a massive Bauer BG28 rig for the project. The project also included over 2 km of fabric-reinforced mechanically stabilised earth walls and 2 km of cut retaining walls. This article discusses some of the more interesting facets of the geotechnical components of the project.

BACKGROUND

The freeway expansion project involves the widening of the N2 from eMdloti Interchange to Tongaat Plaza, for the South African National Roads Agency SOC Ltd (SANRAL). The approximately 10 km section has been widened by adding two lanes to each carriageway at a cost of R390 million. The project forms part of SANRAL's strategic development of the N2 north-coast route, which has become necessary due to increased traffic volumes.

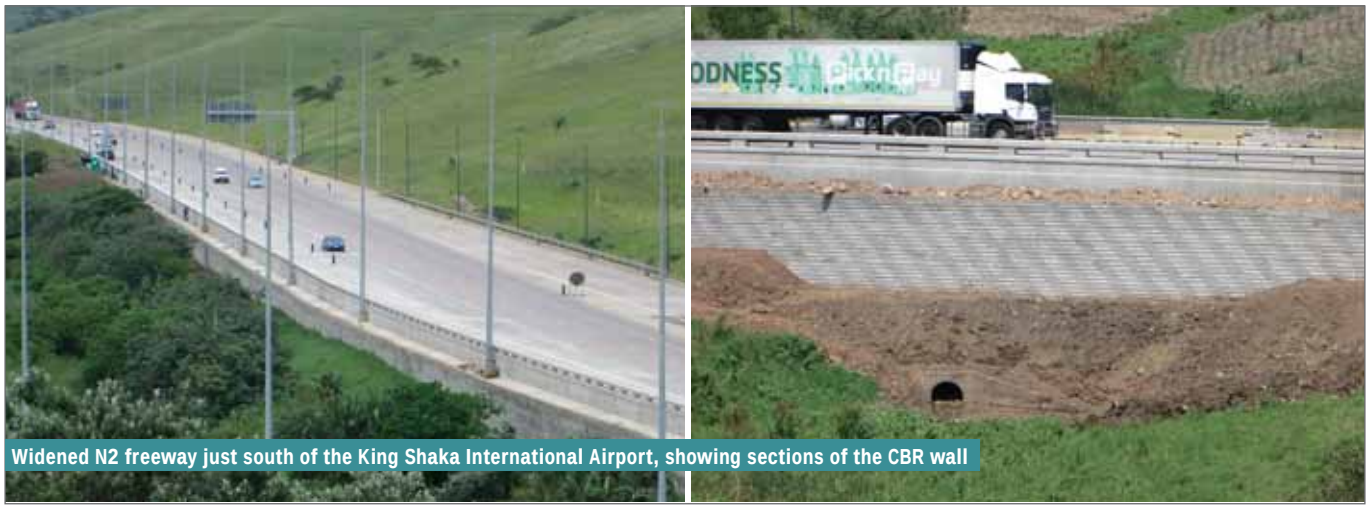
GEOTECHNICAL INVESTIGATIONS

Although investigations for this phase of the project were limited, it is necessary to recognise that there are two obvious differences between an investigation of a project which comprises the upgrading of existing infrastructure and that of a greenfield development:

- Firstly, there is potentially a wealth of information comprising previous investigations, as-built drawings and reports which can be studied to develop an understanding of ground conditions and the design of the existing infrastructure.
- Secondly, there is the performance history, sometimes formalised through maintenance records, or simply by observations, which can be useful in understanding how the infrastructure has behaved given its design and the prevailing ground conditions.

The process of obtaining, sorting and studying this information can be cumbersome and implies a significant amount of work to be done during the planning stages of the project, in advance of any investigative site works.

However, depending on the quality of the information, the planned investigative works can be curtailed. Additionally, that



Widened N2 freeway just south of the King Shaka International Airport, showing sections of the CBR wall

which is done can be focused on particular geotechnical problems or project optimisations. The geotechnical engineer would thus need to target the investigations on aspects where:

- there are significant changes to the infrastructure and/or imposed loads
- new or alternative construction methods can be considered
- less conservative designs can be adopted
- the impact of new construction adjacent to existing infrastructure needs to be considered
- information obtained from historic records is unclear or inadequate
- performance history of existing infrastructure is poor.

UMDLOTI VIADUCTS – FOUNDATIONS

A significant feature of this phase of the project was the widening of the 281.5 m long dual viaducts over the uMdloti River using the incremental launch method (ILM). The structural design included a deeper deck section with a continuous cast in-situ stitch to the existing structure. This implied that the new widened deck attracted more load and was more sensitive to settlement, which needed to be accommodated in the foundation design.

The foundation design was further complicated by challenging geological conditions. The uMdloti River is characterised by the presence of a deeply incised paleo-channel which has since filled with various sediments after the recovery of sea levels after the last Ice Age. Factors such as the nature of the material in suspension and the velocity of the river contribute to the type of material that was deposited. Materials encountered, including running sands, boulders and Hippo Muds, are typically highly variable and very weak, and can present a challenge to piling.

Foundations for the viaducts comprised 4 x 900 mm diameter permanently cased screwed-in-casing-augered piles (SICAPs). The SICAPs were well suited to this project and provided a number of specific benefits. Depths up to 45 m were achieved using the oscillator attachment, installation was possible under high water table conditions, and the pile was suited to the difficult soil profiles encountered without collapse of the pile annulus.

The pile shaft was constructed by driving an open-ended casing into the ground by means of a casing drive adaptor on the Kelly Bar. An oscillator attached to the piling rig was utilised when depths exceeded approximately 20 m. The soil on the inside of the casing was then augered and removed, whilst the side walls were supported by the temporary casing.

INCREASED SHAFT STRESS

Although the final pile configuration did not differ from that utilised on the existing piers, the additional load and stricter settlement criteria required shaft stresses on piles to be increased from an estimated 8 MPa (on existing) to over 11 MPa. In so doing, the same pile configuration could be used, thereby eliminating the need for a fifth pile at each pile cap, resulting in an estimated saving of some R2 million.

The design loading of a pile is a complex relationship between the quality and reliability of the pile itself (affected by factors such as piling method, strength and integrity of concrete, and quality control and supervision) and the geotechnical design and geological conditions (such as rock/soil strength and integrity, soil-structure interaction and stress-strain behaviour). This influences geo-structural design aspects such as foundation settlement, rotation and stiffness, which ultimately affect the design of the superstructure.

Piling on the outside of the existing dual viaducts over the uMdloti River, which were doubled by two additional lanes

Due to the poor soils, emphasis was placed on the formation of rock sockets which need to be embedded in hard, but very highly fractured, rock. In the design of rock sockets, the more favoured empirical methods relate the end bearing and shaft capacity of the socket only to the strength of the rock itself. However, authors such as Rotehberg (1976) and Pells (1981) have shown that when the bedrock is fractured, these methods can incorrectly estimate the capacity. Thus methods which consider settlement criteria with due consideration of the rock quality (fracturedness) are preferred.

Keller-Franki's BG28 at work on one of the 45 m deep SICA piles

Demonstration of permanent casing reinforcement and CHSL tube configuration

In addition, to achieve the increased shaft stress, the piles were provided with a 5 mm thick permanent casing and constructed using 45 MPa concrete. The permanent steel liner was provided to protect the wet concrete from flowing groundwater and soil influx. In the long term, the liner would also protect from debris impact and chemical attack from coastal conditions.

SANRAL's construction manual advocates that when piles are raked, a thin-walled casing must be used, as the soil face would be prone to collapse during extraction of a temporary casing. However, the casing can also fulfil a structural function, and the pile can be designed as a composite element allowing larger shaft stresses to be placed on the piles. The casing thickness, when purely used to prevent influx, would need to be some 3 mm thick for the uMdloti site, which would in all likelihood corrode in 130 years. Casings on the site were, however, much thicker, providing additional capacity where it was required.

Each pile was also equipped with 4 x 80 mm diameter steel tubes through which concrete integrity cross-hole sonic logging (CHSL) and base integrity core testing were conducted under direct full-time supervision.

RETAINING WALLS

Two final aspects of the project which are worth noting are the approach fills and cuts to the uMdloti viaducts.

A prominent V-shaped valley, with embankment heights of up to 15 m, occurs in the median just north of the uMdloti River. Widening of the road without some form of retaining structure would imply narrow sliver construction along the side of the embankment, and would also imply the encroachment of works into a local stream, which was an environmental concern.

The investigations confirmed that the embankments had been constructed with Berea Red sands. Below this fill, weathered products of either shale or sandstones were present, followed by the respective rock. A geosynthetically reinforced concrete block reinforced wall (CBRW) was utilised to support the widened freeway. The walls were designed with assistance from ARQ Consulting Engineers. The final structures comprised two CBRW walls, each about 0.5 km long on the northbound carriageway, up to 3.5 m in height, constructed on top of the existing embankments of 15 m.

On the southern approach to the uMdloti viaduct, the N2 goes through an area where a large amount of cut had to take place to accommodate the widened road. Adjacent to the northbound carriageway (NBC), a 25 m deep cutting occurs in dolerite, while adjacent to the SBC a 10 m deep cutting occurs in dolerite and shale.

A detailed study of the NBC cutting was undertaken by Davies Lynn & Partners in 1983 during construction of the existing freeway following two slip failures. The report concluded that, due to unfavourable joint orientations, a pre-

existing failure plane and the risk of a high water table developing in the cut, this should be cut back at approximately 1:3.

Additional investigations were undertaken to determine whether the cut toe could be stabilised using a vertical retaining wall. Investigations showed the upper portions of the cut to comprise very poor silty and clayey soils of residual dolerite. These soils were poor both in terms of their shear strength and their suitability for use in engineered layerworks. Below this hard rock dolerite occurred along the toe of the cut. The dolerite varied from a dense weathered gravel to a highly fractured hard rock dolerite. No ground water was encountered.

The good quality interlocking rock with occasional gouge implied that a soil nail wall could be utilised, which resulted in a R10 million saving in earthworks. Two rows of nails of approximately 6 m length each at 1.5 m spacing were utilised, with provision being made for some hollow self-drilling injection anchors where more highly fractured dolerite was present. The soil nail wall was cladded with a precast panel to provide an aesthetic façade to the cutting.

CONCLUSION

By obtaining and studying previous records, an understanding of ground conditions and the structural design and performance of the existing infrastructure was developed. The benefits were that the ground conditions were well understood and the investigations could be focused. As an example, a deep cutting previously cut back at 1:3 was provided with a conventional, vertically-installed soil-nailed retaining wall, resulting in a considerable cost saving in earthworks.

At the same time, for the piling to the uMdloti viaducts, little additional investigation was necessary, but the design could be optimised by using contemporary design theories in close collaboration with structural engineers. The increase of design shaft stresses and loading resulted in the elimination of a fifth pile at each of the piers.

However, caution must be exercised when adopting such an approach. This can only be done where there is adequate quality control and direct full-time supervision. It also requires continued involvement of the design teams throughout construction. Piling data and integrity testing from each pile were analysed by the design team during construction before approvals were provided, and, where necessary, remedial actions were recommended and implemented.

ACKNOWLEDGEMENTS

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Caution must be exercised when using increased design shaft stresses on piles. This can only be done where there is adequate quality control, direct full-time supervision and continued involvement of design teams throughout construction.

PROJECT TEAM AND STATISTICS

Client: South African National Roads Agency SOC Ltd (SANRAL)
Consultant: SMEC South Africa (Pty) Ltd
Contractor and subcontractors: Group Five Construction, with Keller-Franki as piling subcontractor
Project value: R390 million (2015, excluding CPA)

Artist's impression of the Rea Vaya Sandton BRT cable-stayed bridge over the M1

Foundation design and construction challenges at the Rea Vaya Sandton BRT cable-stayed bridge

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INTRODUCTION

The M1 cable-stayed bridge, currently under construction near the Marlboro Drive off-ramp in Sandton, forms part of the Rea Vaya Bus Rapid Transport (BRT) network, one of the largest projects ever undertaken by the City of Johannesburg. The bridge, which is built in partnership with the Johannesburg Development Agency (JDA), will provide vehicular and pedestrian access from Sandton, across the M1, towards Alexandra. Michael Pavlakis & Associates were appointed to carry out the geotechnical investigation and the design of the pile foundations for the bridge.

THE BRIDGE

The 271 m long structure extends from Katherine Street in Sandton, across the M1 highway and on to Lees Street in

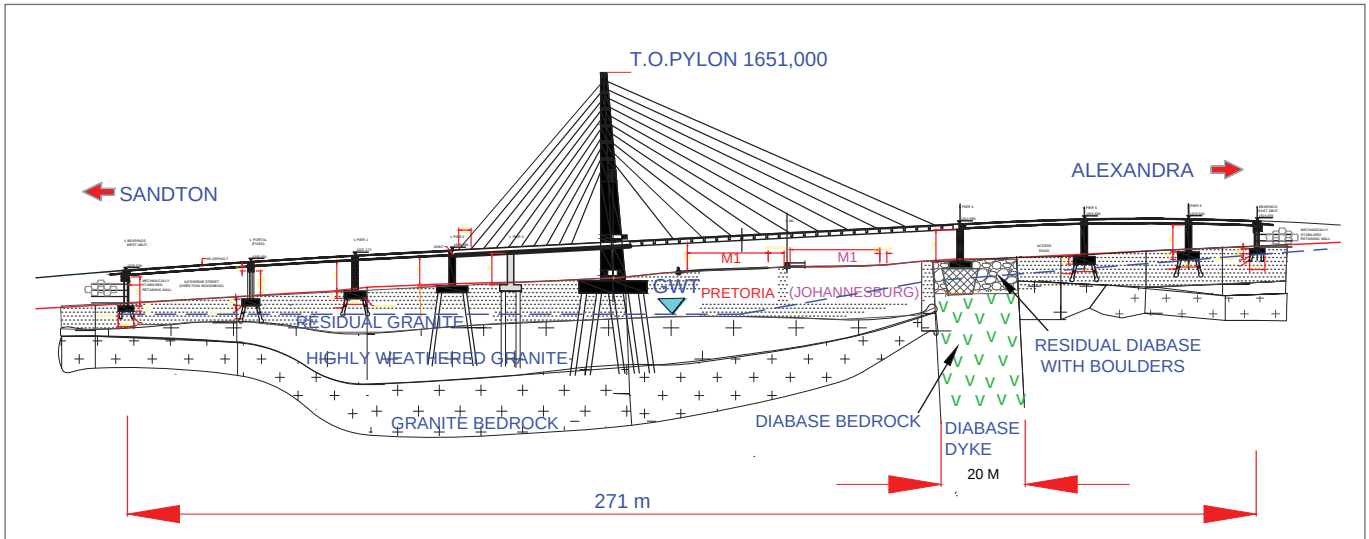


Figure 1: Longitudinal section of the bridge with underlying geology

Wynberg, and incorporates eight spans, as shown in Figure 1. The cable-stayed section comprises a main span of 83 m crossing the M1, and a 39 m long back span provided with a central tension pier. Most remaining spans vary in length from 25–30 m. A 50 m high slightly inclined concrete pylon, comprising dual columns each having a base length of 6 m, carries the cable-stayed section. Mechanically stabilised earth walls form the transition from the abutments to the roads on either side of the bridge. Two ramp bridges provide pedestrian access from either side of the highway to the 3 m wide pedestrian section of the bridge deck, which has a total width of 13.35 m over the freeway.

GEOTECHNICAL INVESTIGATION

A two-stage geotechnical investigation was carried out, with the second stage related to a more extensive study of the pylon and back span piers – as the original bridge design envisaged the construction of a 'normal' bridge. Further drilling work was carried out at the early stages of construction. The exploratory work included:

- The drilling of a total of 15 rotary core diamond drill boreholes down to depths varying from 15–35 m, as part of a geotechnical investigation that included other bridges along the route.
- Two 800 mm auger holes drilled to 'refusal' of a relatively weak Soilmec auger rig. Refusal near the pylon occurred at 15.1 m.
- Standard penetration tests (SPTs), uniaxial compressive strength tests with strain measurements and point load index tests on rock core.
- Laboratory index, consolidation and shear box tests on undisturbed and remoulded soil samples, and chemical analysis of groundwater samples.

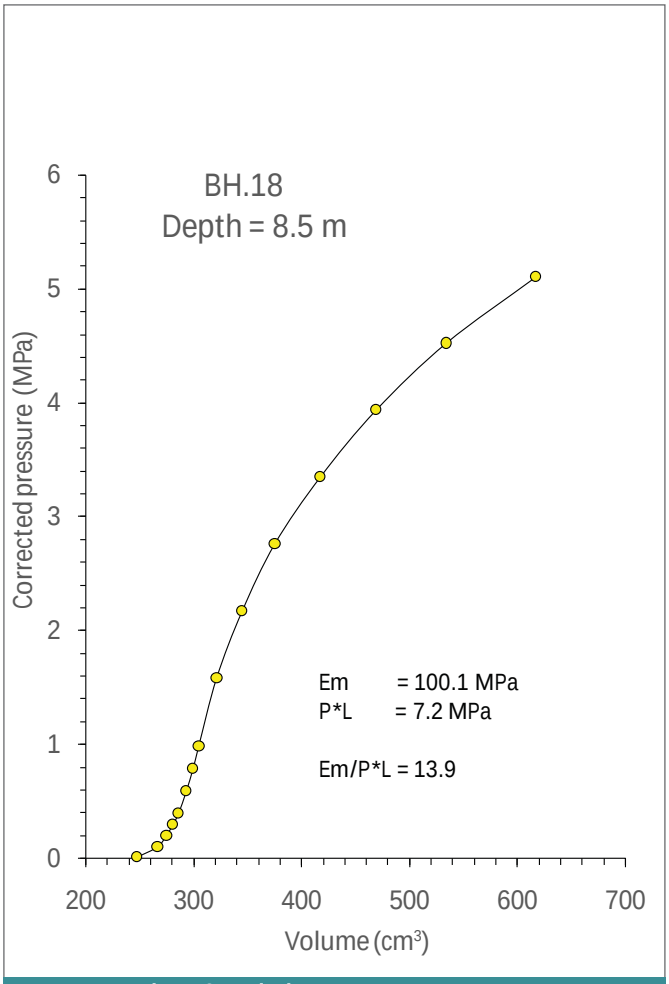
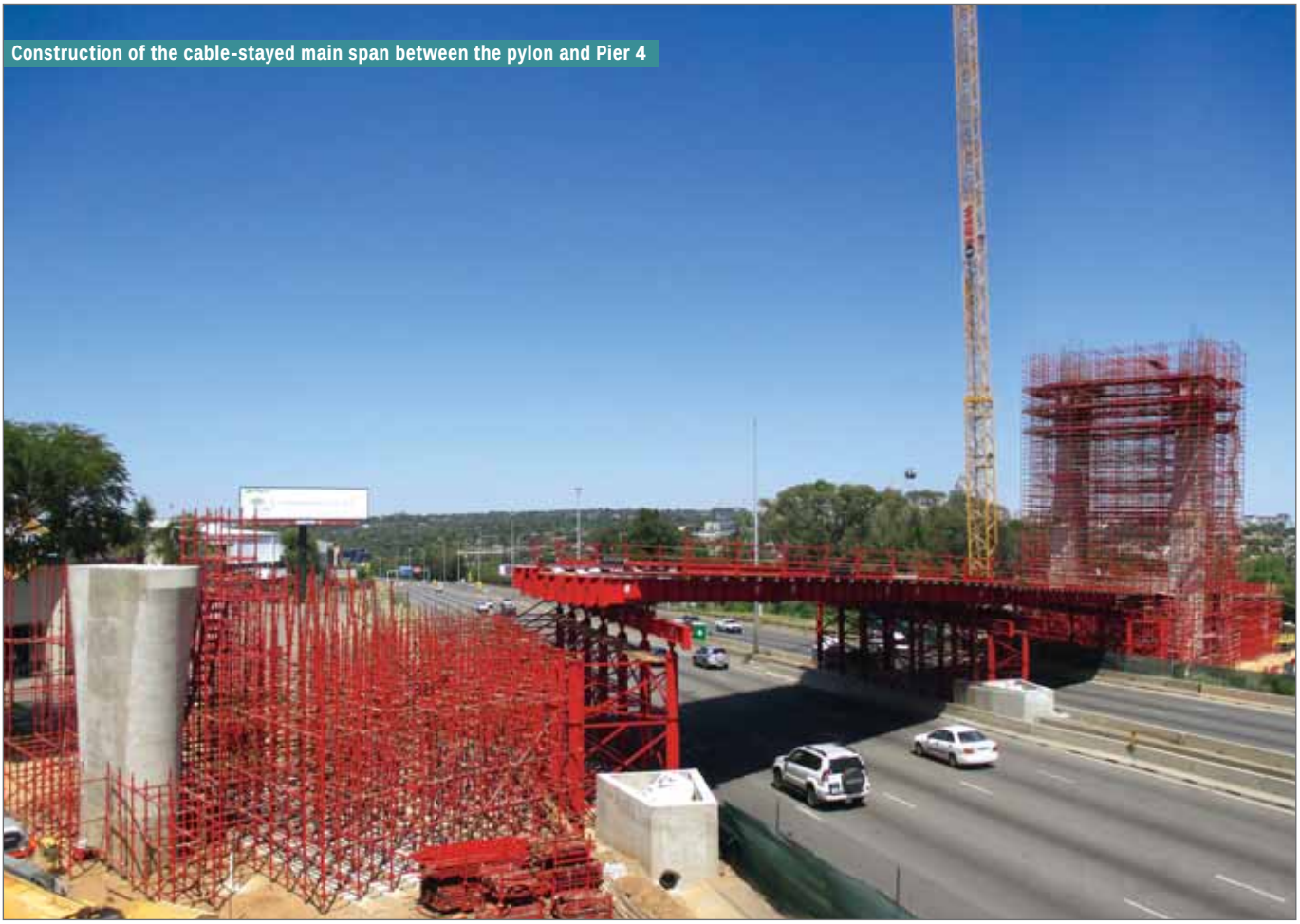


Figure 2: Typical pressuremeter test curve



Construction of the cable-stayed main span between the pylon and Pier 4

- A series of Menard pressuremeter tests (PMT), carried out mostly at 1 m depth intervals within the pylon and back span section of the bridge for a more accurate appreciation of the engineering behaviour of the various formations encountered. The tests were carried out using the latest pressuremeter equipment.

Typical results of a pressuremeter test carried out within very dense residual granite, tending towards very soft rock, at the depth of 8.5 m, are shown in Figure 2. The pressuremeter modulus E_m is related to the stiffness of the soil or rock tested while the net limit pressure P*L is a parameter that can be used to compute shear strength and bearing capacity.

GROUND CONDITIONS

The simplified geology along the bridge route is shown in an idealised geological section in Figure 1.

The site is situated within the 3 200 million-years-old Johannesburg granite-gneiss dome, part of the Basement Complex, at an average elevation of 1 600 m AMSL, in a region of annual water surplus. It occupies part of a gentle rise that begins at Sandspruit to the west (where another bridge, part of the same project, is under construction) and reaches a maximum elevation near Louis Botha Avenue, from where it falls eastwards towards the Jukskei River.

The granites would thus be expected to be deeply weathered, and they are indeed so, with the residual soils, comprising orange-brown silty and gravelly sands (mostly ancient soils that probably formed on the African erosion surface over 100 million years ago), extending down to depths of 6–8 m. Typical strength parameters from shear box tests are listed in Table 1. Below these depths, the granite rock has undergone extensive differential weathering, which has resulted in large variations in the subsoil conditions along the bridge route. Thus, the west abutment and Pier 1, as well as Pier 6 and the east abutment, are underlain by competent granite bedrock of medium-hard rock or hard rock quality at the depths of 6–11 m, while within most of the remaining bridge route, including the cable-stayed section, severe weathering has taken place down to average depths of 23–25 m,

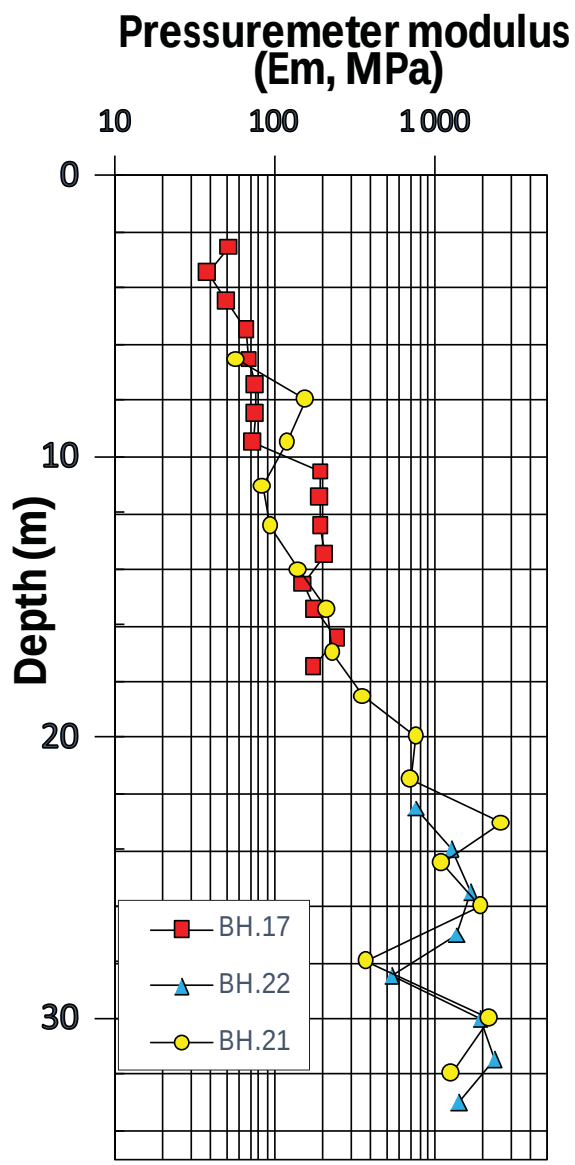


Figure 3: Some pressuremeter test results showing variation of pressuremeter modulus with depth

Table 1: Shear box test results on undisturbed and remoulded samples of residual granite

Borehole no and Auger hole no	Depth (m)	Dry density (kg/m³)	Moisture content (%)	Cohesion C' (kPa)	Friction angle ϕ' (degrees)
Undisturbed samples					
BH.17	3.05	1 814	7.6	66.5	28.6
BH.19	3.25	1 781	17.7	42.4	27.1
Remoulded samples					
AH.K1	4.0	1 355	12.6	18	30
AH.K1	8.0	1 487	8.3	22	30

Table 2: Typical results of uniaxial compressive strength tests on weathered granite rock core

Borehole number	Depth (m)	Density (kg/m³)	UCS (MPa)	Secant modulus E (MPa) (at 50% UCS)	Strain at failure (%)
BH.18	8.7	2 250	0.1	0.3	0.13
	11.3	2 150	2.8	0.7	1.04
	14.9	2 400	7.4	0.2	0.85
	17.7	2 330	1.6	0.2	0.80
	20.8	2 380	4.9	0.2	1.60

where the competent, slightly weathered or unweathered granite rock is found. Hard, coarse pegmatite bands are present in places, mostly in the pylon area. The results of uniaxial compression tests on weathered granite rock core, showing the significant variation of strength with depth in a single borehole, are shown in Table 2. The groundwater table is present mostly within the depth range of 3–8 m.

A narrow diabase dyke that intruded the granites, was identified during construction at the position of Pier 4, near the eastern edge of the M1. The pier is situated in an area that was covered by the old pedestrian bridge access ramp, and some 8–10 m east of the exploratory borehole drilled, with the edge of the dyke being only a few metres away from the nearest borehole. Trial holes and an additional rotary core borehole drilled at the centre of Pier 4 indicated that the diabase comprised a clayey-sandy silt containing large 300–1 500 mm very hard rounded boulders (weathering spheroids), extending down to the level of the hard diabase bedrock at a depth of 9 m. Pressuremeter tests were also carried out in the borehole to determine the engineering properties of the residual diabase.

DESIGN CHALLENGES

Although the decomposed granite was mostly dense within a few metres from the surface, in certain sections it was found to be less competent and medium dense down to depths of 3–5 m. The heavy imposed foundation loads, and bridge performance requirements, necessitated the use of pile foundations. The design challenges included the following:

- The proximity of the pylon to the edge of the highway, requiring the minimisation of the size of the foundation; associated excavation depths would also have to be minimised to avoid possible destabilisation and/or cracking, and possible disruption of the M1.
- The presence of the high groundwater table, making it difficult to manually clean the base of large diameter piles, despite the slow flow rates.
- The highly variable depths to competent bedrock, and the variable engineering characteristics of the residual granite.
- The presence of soft rock bands and occasional spheroids within the less competent very soft rock granite, which caused 'refusal' of the drilling rig at geotechnical investigation stage, at the depth of 15–18 m, above the proposed founding level of the pylon and some piers.
- The presence of hard rock pegmatite bands causing further difficulties, especially in the construction of rock sockets.
- The diabase intrusion, which affected the foundation of Pier 4 and some foundations of the eastern pedestrian ramp bridge; the residual diabase contained large hard boulders (spheroids) 'floating' within the relatively incompetent decomposed diabase, making it difficult to pile through to bedrock with usual piling equipment.

THE PYLON

The pylon foundation consists of a single pile cap 12.6 x 16.2 m in size, supported by a pile group comprising 20 bored (augered) cast-in-situ piles to resist the large imposed pylon loads. The 1 200 mm diameter bored piles were socketed a minimum of 2 m into the hard granite bedrock or very hard pegmatite at the depths of mostly 25–27 m.

All piles were drilled by a Casagrande B180 auger rig, while the sockets were constructed by a purpose-made core bucket using the same rig. Approximately one half of the piles intersected very hard pegmatites at depth, making it difficult to form the rock sockets, each of which often took more than a day to complete.

The large-diameter shafts, which carry serviceability limit state axial loads of 7.5 MN each, provide the necessary stiffness and robustness to limit pylon foundation settlement and tilt movements to acceptable levels. They also served to minimise the foundation size and to limit excavation depths. Even so, the edge of the pile cap is only some 5 m distant from the edge of the M1. The piles were provided with moderate rake of 1:8.

In carrying out pile group analyses, much reliance was placed on the results of the pressuremeter tests (PMTs), which provided a near-continuous record of the soil/rock engineering properties, from the surface to a depth of 33 m (information from the SPT tests was limited due to early 'refusal' at depths of 5–8 m, while, due to the presence of joints and fractures, much of the weathered rock core was not suitable for laboratory testing). These were also particularly useful in establishing the resistance of the piles to large applied horizontal loads, as the test models this condition directly.


The installation of pylon bored piles


Exposing and trimming 1 200 mm bored piles for the pylon

An important issue was the cleaning of the base of the pile shafts to ensure adequate end bearing, which would serve to increase the factor of safety against overall bearing failure to the required value. Although groundwater seepage rates were very slow, the long duration of the pile-forming activities allowed the water to enter and fill the pile holes up to the groundwater table. Pumping of the water and manual cleaning of the pile holes, with personnel working inside a steel casing, was considered, but this was deemed to be difficult and unsafe at the depths under consideration. It was therefore decided to clean the base of the pile holes by means of airlifting. This worked well, due to the high groundwater table, as evidenced by the probing of the pile hole bottom after airlifting. All pylon large-diameter shafts were airlifted and concreted immediately afterwards by tremie. They were also provided with 50 mm diameter steel tubes to enable concrete quality checking by cross-hole sonic logging.

BACK SPAN AND INTERMEDIATE PIERS

The back span and intermediate piers are supported on 1 050 mm diameter bored piles founded at a depth of 20 m. They are required to resist a total uplift force of 18 MN (SLS), in addition to other substantial imposed loads. Tensile restraint is achieved by installing stressed rock anchors through 200 mm diameter steel sonic tubes installed within the piles. The anchor-fixed length was installed within the hard rock granite below the depth of 24 m. The anchors incorporated 32–40 mm diameter SAS950 threadbars, provided with double corrosion protection.

REMAINING BRIDGE PIERS AND PEDESTRIAN RAMPS

All the remaining piers, as well as the abutments, were supported on pre-drilled 610 mm diameter driven cast-in-situ (DCIS) piles. Pad footings were also possible, but these would have resulted in large and deep excavations in Katherine and Lees Streets, effectively necessitating the closure of those roads. Consideration was also given to using other pile types, including augered cast-in-situ and continuous flight auger (CFA) piles, but these were not practical or economical due to the following reasons:

- The variable subsoil conditions and difficulties in controlling the depth of founding, unless taken to 'refusal', which would have been uneconomical for certain areas
- The high groundwater table, the possibility of local pile hole sidewall collapse and the difficulty of cleaning the base of the auger piles
- The pressure of alternating hard and soft layers to large depths, and occasional boulders which could cause premature refusal of the proposed relatively small diameter piles in the case of both auger and CFA piles.

Each of the DCIS piles were founded at the fixed depth of 8 m below natural ground surface. They were founded within dense to very dense gravelly sand (residual granite), and occasionally granite rock, to carry a serviceability limit state axial load of 2.1 MN. The piles were provided with a 1:4 rake, which assists in stiffening the pile cap.

PIER 4: DIABASE INTRUSION

Pier 4, which is situated near the eastern edge of the M1, is underlain by a 20 m wide diabase dyke striking in a roughly north-south direction, perpendicular to the bridge centre line. The dyke contained an abundance of very large, hard, up to 1 500 mm spheroids/boulders which prevented the installation of the DCIS piles. The boulders were embedded within firm, mostly saturated clayey silt and extended to hard diabase bedrock at a depth of 8.5–9.0 m. A suitable piling rig to penetrate the boulders would be costly and would take months to organise. Consideration was also given to supporting the pier within the residual diabase, but this was rejected due to the random occurrence of the boulders, which could give rise to significant foundation tilt, depending on their configuration. Pemissible foundation settlements would also be much smaller than values that would normally apply, as the adjacent pier was piled and would not settle significantly. Therefore, any settlement that the Pier 4 foundation would undergo, would be mostly differential.

An alternative was investigated, comprising excavation to bedrock and mass filling with weak concrete to a suitable level, where a normal concrete footing could be constructed. This would, however, require a steep excavation, as the centre of the pier was only 15 m from the edge of the highway. Strength data for analysing the excavation was obtained from the pressuremeter tests performed in a borehole drilled at the centre of the


Near-completed pile group for pylon


View from the western abutment in Katherine Street

base, which indicated an undrained shear strength of 79 kPa, and 2 SPT tests (N-value = 16–17).

A drained analysis, based on estimated strength parameters, indicated that the excavation would be unstable if left standing for a significant period, bearing in mind the high water table at that position. An undrained analysis, however, based on the measured strength data, indicated a short-term factor of safety of 2.3 for a benched excavation cut at the steep angle of 70° below the depth of 2 m, and an average slope angle of about 60°, as shown in Figure 4. It was therefore decided to proceed with this solution, on condition that the excavation would be strictly and continuously monitored for stability.

During the final stages of excavation, a local collapse occurred, associated with the high moisture content and relative looseness of the trench backfill, linked to an old sewer pipeline. The sewer line was present at the edge of the excavation closest to the highway, and could not be diverted. The overall excavation, however, proved to be stable, with the undamaged sewer line visible at its northern edge. The entire operation was completed, and the weak concrete mass fill was placed, within a period of three days. The deeper section of the excavation (below 5 m) and the pouring of the mass concrete fill took two days, with the second day stretching to close to midnight, by which time the remaining excavation for the pier footing was much shallower and safe in the longer term.

PILE TESTING, SONIC LOGGING AND ANCHOR PULL-OUT TESTS

Taking into account the importance of the structure, a comprehensive programme of static pile load tests (maintained load tests) was implemented in order to confirm design parameters. This comprised the testing of two bored trial piles 900 mm and

Large boulders were encountered within the Pier 4 excavation


Mass filled excavation for Pier 4

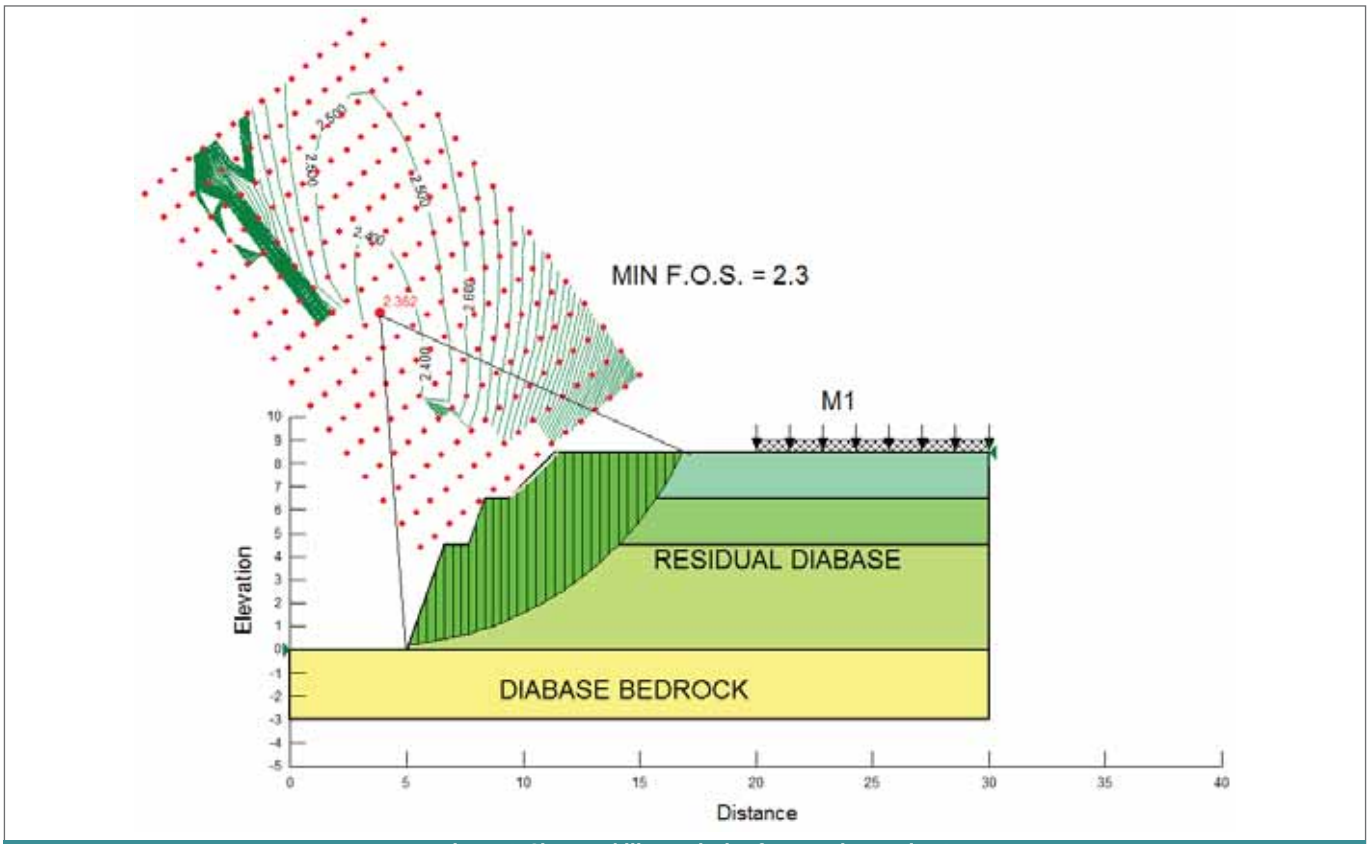


Figure 4: Slope stability analysis of excavation at Pier 4

1 200 mm in diameter, with the latter loaded to 15.7 MN (just over twice the working load), and of three DCIS trial piles along the route of the 271 m long bridge. The measured load-settlement behaviour of the piles proved to be close to that predicted, validating the design parameters.

Two working DCIS piles were also tested up to 150% of working load, and the results also proved to be satisfactory.

Sonic logging was implemented as part of the quality assurance process. The test proved reliable, picking up a fault in one of the pylon piles (highlighted by the contractor before testing) very

Static load testing of a bored pile

close to the suspected problem depth.

Three 40 mm diameter thread bars were grouted within the granite rock within the depth range of between 23–30 m and subsequently tested to their ultimate tensile strength. The results confirmed the high frictional bond developed between the non-shrink cement grout and the hard granite rock.

CONCLUSIONS

This article highlights some of the geotechnical challenges associated with building in the Basement granites in the Sandton area, and the importance of an adequate and fit-for-purpose geotechnical investigation. The principle that rock becomes less weathered and more competent with depth does not necessarily hold (at least within the normal founding depth range), nor the assumption that if the ground conditions are the same at two relatively close positions, these same conditions will apply between them. The pressuremeter testing work carried out, in combination with other routine SPT and laboratory tests, enabled a firm appreciation of the engineering behaviour of the soils and rocks underlying the site, leading to more economical and reliable foundation design. Experienced contractors, coupled with close supervision during construction, and synergy between all parties in working towards the common goal, serve to ensure quality of construction. An allowance for a comprehensive geotechnical investigation programme ensures safe and economical foundation design, and minimises construction difficulties and expensive delays.

ACKNOWLEDGEMENTS

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Stability of wind turbine foundations – accounting for gapping and eccentric loading

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INTRODUCTION

In the last two decades, the global development model has shifted to idealise sustainable development, as well as to promote 'green' and renewable ideals. South Africa, following the example of other more developed nations, has tackled these ideals by beginning to supplement its energy production infrastructure with more sustainable alternatives and, with the exception of its planned new nuclear scheme, has committed to a reduction in non-renewable production methods over time. This was highlighted in the country's 2012 Integrated Energy Plan, where the National Energy Regulator committed to investing 1 600 MW a year in renewable energy infrastructure, of which half is dedicated to wind energy and the creation of wind farms. With this increase in scope, understanding the complexities of turbine engineering has never been more important for South African engineers.

Foundation designs for these structures pose a series of complex challenges, due to the unique loading characteristics that turbines inherently possess, as well as the wide variety of soil conditions that exist within the South African wind energy corridors. One of these challenges is to provide a safe approximation of the bearing capacity, which must account for the extremely large gravity and moment loads that the structure experiences during normal operating conditions. This leads to very large eccentricities, which can often destabilise a footing to the point of failure. This article introduces and discusses the various turbine loads and how they can be theoretically accounted for in the design of a conventional gravity footing for a wind turbine structure.

LOADING ON TURBINES

To fully appreciate the complexities of the structural and geotechnical design of wind turbine foundations, it is critical to understand the origin of turbine loading and the number of forces that must be accounted for in design. A turbine, at an idealistic level, could be considered a concentrated mass supported by a very long, slender column, which is exposed to very large lateral loads and dynamic actions throughout its lifetime. Besides the large self-weight of the structure and the mechanical components that the structure houses, turbine design is mainly influenced by the aerodynamic forces caused by wind. These can be treated as both static and dynamic loads caused directly by airflow and its interaction with the turbine structure, most notably the blades of the turbine. The intensity and type of load is governed by a number of factors, including average wind speed, turbulence of the air flow, rotational speed of the rotor, air

density, the shape of the rotor blade, and any interactive effects between the blade and the air flow, such as drag.

Ultimately, these loads can be reduced to a combination of two simple forces and two moments, based on direction and magnitude of action. Each resultant load or moment is provided by the manufacturers after they have undertaken numerous models and practical tests for their specific turbine type within its operating limits. Many turbine manufacturers produce their own specification and loading documentation that note a certain sign and symbol convention in order to communicate the loads experienced. Figure 1 highlights one of these representations, as presented by Vestas (2011) in their documentation. The loads highlighted include the following:

- F_{res} – a lateral load acting through the hub of the turbine due to aerodynamic forces
- F_z – a vertical downward acting load typically consisting of weights of the structural elements
- M_{res} – a moment caused at the base due to F_{res} acting at hub height
- M_z – additional moments caused due to rotation of the turbine, subsidiary aerodynamic effects and other loading.

With the F_{res} force often exceeding 600 kN under normal operating conditions, combined with tower heights of approximately 100 m for the 3 MW turbines that dominate the South African energy market, design moments (M_{res}) are generally within the range of 60 000 kNm (Vestas 2011). For the new larger 8 MW turbines currently being erected in Europe, which extend to heights of 150 m, the design moments can eclipse 100 000 kNm. With a gravity foundation design, the structure

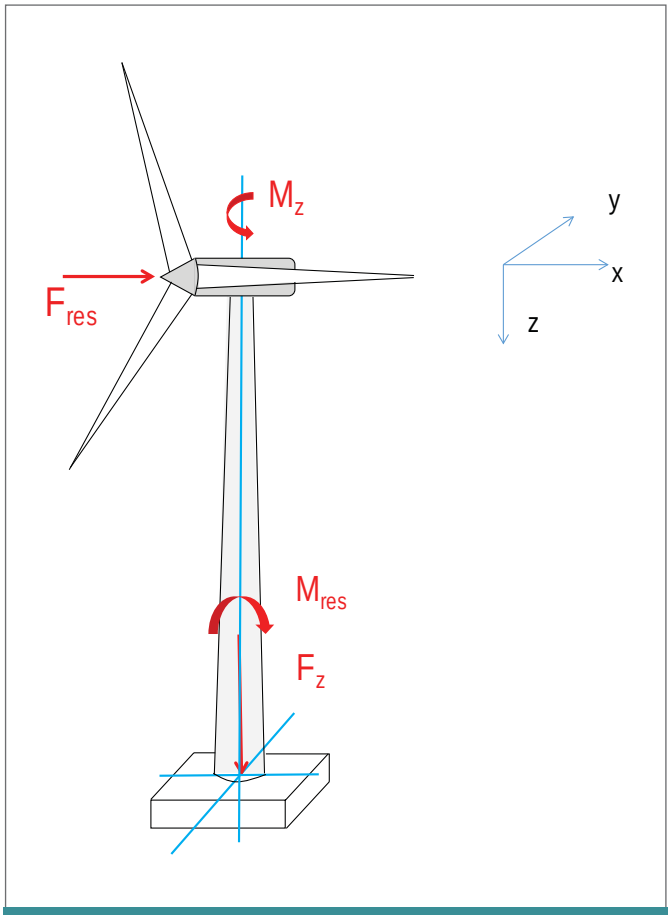

Figure 1: Simplified annotation of loading as applied to wind turbine structure (Source: Mawer 2015)

Table 1: Simplified Vestas V112 3 MW turbine load cases (Source: Mawer 2015)

LOAD CASE 1			
Normal conditions			
M_{res}	M_z	F_{res}	F_z
[kNm]	[kNm]	[kN]	[kN]
66 700	–353	695	–4 590
LOAD CASE 2			
Extreme conditions			
M_{res}	M_z	F_{res}	F_z
[kNm]	[kNm]	[kN]	[kN]
85 100	1 551	1 031	–4 500
LOAD CASE 3			
Serviceability conditions			
M_{res}	M_z	F_{res}	F_z
[kNm]	[kNm]	[kN]	[kN]
49 100	731	554	–4 620

Table 2: Eccentricities calculated for each load case (Source: Mawer 2015)

ECCENTRICITY (m)		
LC1	LC2	LC3
1.40	1.79	1.03

is stabilised against sliding and overturning by the large mass of the foundation base. However, it must also not be so large or so heavy that it exceeds the bearing capacity of the supporting soils. Typically in foundation engineering, designers account for base moments by making use of Meyerhof's effective area method, which assumes a reduced contact area between the supporting soil and the base by adopting the theory that the moment causes the vertical loading applied through the centre of the base to act eccentrically. With such high moment loads, eccentricities can become very large, which in effect dictate the need for extremely large base sizes. This, coupled with the prevention of gapping (the temporary uplift of the footing base during operation), as well as limiting settlements and meeting manufacturers' soil stiffness requirements, poses a complex design challenge.

An additional consideration to be given attention is the different load cases that apply to wind turbine structures. Load combinations in turbine design apply to the IEC 61400-1 (2005) which stipulates load scenarios that require consideration in all the crucial operating conditions of the turbine, i.e. parked, start-up, normal operation, abnormal operation, shut down, etc. This list is often reduced by turbine manufacturers into specific loading conditions for each necessary design check, such as overturning, sliding, bearing capacity, etc. However, in principle, the design dimensions to be used must apply to worst load case, as theoretically this will produce the worst eccentricities. The worst loading that a turbine experiences is generally in an extreme weather condition when wind speeds have increased beyond the acceptable operating window and the turbine has been parked to avoid damage to the internal mechanics. In this state, the F_{res} and M_{res} loads are up to 20% larger than that experienced during normal operating conditions. While these exaggerated loads will not occur for extended periods of time over the lifetime of the structure, it is important that the overturning resistance and the bearing ability of the supporting soils are able to safely handle the extreme wind conditions.

For the purpose of this explanation, Table 1 presents three load cases that were identified by author Mawer (2015) to illustrate the principles behind the foundation design for large eccentric loading. Load Case 1 (LC1) can be identified as the normal operating condition, Load Case 2 (LC2) can be considered the extreme weather condition and LC3 is for the serviceability state requirements. These loads are specific to Vestas V112 3 MW turbines which are widely used in South Africa.

DESIGNING FOR ECCENTRIC LOADING

The majority of representations for Meyerhof's effective area method in geotechnical literature are presented for square or rectangular footing shapes, which are arguably the most common in everyday construction. However, for wind turbines, circular bases are the most common shape adopted, due to the fact that turbine rotors are able to rotate around the tower axis to account for changing wind direction. The resulting effective area reduction is slightly more complicated for a circular foundation, but follow the same principles as suggested by Meyerhof.

The DNV/Risϕ (2002) publication, titled *Guideline for the Design of Wind Turbines*, presents an adaptation of Meyerhof's effective area method specifically for circular bases subjected to large moments, and is the basis of the method described below. After taking into account the assumed mass of the foundation base, using an initial estimate of the foundation radius, the eccentricities of the vertical loading can be calculated using Meyerhof's approach. For an assumed 21 m diameter footing, and the loads stipulated in Table 1, the eccentricities presented in Table 2 can be calculated. These are then used in the calculation of the effective area as per the DNV/Risϕ (2002) method.

Circular foundation

In contrast to a square or rectangular footing, which has the effective area reduced to a smaller rectangle with a new eccentric centre, a circular foundation is generally represented best by an ellipse centred laterally at a spacing of e from the foundation centre (Figure 2). The effective area is then calculated by:

$$A_{eff} = 2[R^2 \arccos\left(\tfrac{e}{R}\right) - e\sqrt{R^2 - e^2}] \quad (1)$$

where:
R is the radius of the footing, and
e is the eccentricity calculated using $e = M_{res}/(F_z+V)$

where:
V is the mass of the footing.
with effective dimensions of (as shown in Figure 2):

$$b_e = 2(R - e) \quad (2)$$
$$l_e = 2R\sqrt{1 - (1 - \tfrac{b_e}{2R})^2} \quad (3)$$

As an ellipse is a shape that is usually hard to work with in design, a rectangle with the same effective area as the ellipse is

assumed, centred over the same centre as the ellipse (at a lateral spacing of *e* from the foundation centre). This shape and these dimensions are then used in design. The new dimensions are referred to as the effective breadth and length given in Equations 4 and 5.

$$l_{eff} = \sqrt{A_{eff}\frac{l_e}{b_e}} \quad (4)$$

$$b_{eff} = \frac{l_{eff}}{l_e} b_e \quad (5)$$

The results of these calculations are the effective dimensions highlighted in Table 3, with Load Case 2 presenting the most critical effective area, as expected.

Extremely eccentric loading

Before bearing capacity calculations can be attempted, it is important at this point in the design process to assess the implications of the eccentricities calculated. Allowable limits for eccentricities in foundation designs are commonly considered as B/6, because any eccentricity larger than this will begin to cause uplift at the extreme edges of the footing perimeter. Uplift – also referred to as gapping in terms of wind turbine structures – is in some circumstances permitted in the extreme loading case, but often limited to 25%. The majority of turbine manufacturers and design guidelines, however, suggest that 0% gapping should be allowed for. When uplift has been allowed for, a further bearing capacity check is required, particularly when the eccentricity is considered "extremely eccentric".

The point at which this occurs is when the eccentricity exceeds the ratio of 0.3B. At this point it is possible for bearing failure to occur beneath the unloaded area of the footing. DNV/Risφ (2002) suggests that this scenario is checked by conducting an additional bearing capacity calculation given by Equation 6. In the case that the eccentricity is less than B/6, it automatically follows that this further bearing capacity check is not needed.

$$q_d = y' b_{eff} N_y s_y i_y + c_d N_c s_c i_c \,(1.05 + \tan^3\phi) \quad (6)$$

with: $i_c = 1 + \frac{H}{V + A_{eff} \cdot c \cdot \cot\phi}$

and: $i_y = i_q^{\,2}$

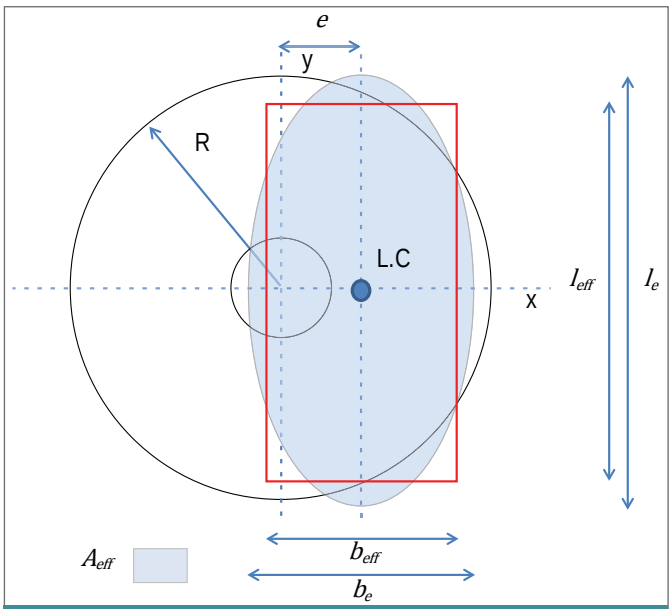

Figure 2: Calculation of effective area of circular-shaped gravity footing (Source: Adapted from DNV/Risφ 2002)

GAPPING

Gapping can be described as the tendency for wind turbine foundations to experience uplift at the heel due to the footing being too small to manage the high moment loads applied to the structure. Both the DNV/Risφ (2002) and Eurocode 7 (2004) make no mention of allowances for gapping of footings, with only the IEC 61400-1 (2005) stating that the prevention of overturning must be ensured under normal operating conditions. The effect of this is that all data surrounding gapping considerations comes directly from the manufacturer's technical directions and specifications.

It is generally understood that uplift would have a negative effect on the stability of the structure. However, under strict control, the loss of contact area has been found to have a number of notable benefits to geotechnical design, including limiting permanent settlements, as well as limiting the effects of dynamic amplification due to resonance (Vestas 2011). Unfortunately most turbine failures worldwide have been due to overturning, leading most major manufacturers to suggest that little to no uplift should be designed for, even under extreme conditions, although practically a certain amount can be allowed if absolutely required.

The easiest way to prevent loss of contact with the soil is to ensure that the foundation is sufficiently sized to guarantee that little to no uplift takes place. To plan for this, the foundation is assumed to have a breadth or diameter large enough that the eccentricity of the loading is within the limit B/6 under normal operating conditions (LC1). The effect of this choice in foundation size is then assessed in terms of the extreme load case, when gapping must be ensured to occur only within certain limits defined by the manufacturer, or not at all. Vestas (2011) recommends that, if allowed for, only uplift of 25% of the footing base is acceptable to ensure foundation stability in the extreme condition. This is calculated from the extreme edge towards the main axis of the foundation and assuming an elastic soil pressure distribution.

General Electric (2013) recommends that the gapping design criteria should comply with the Germanischer Lloyd Wind Energy GmbH IV Rules and Guidelines Edition 2003, 6.7.6.3 Part (3) and Part (4). From this guideline, 100% contact area between the foundation and soil during normal operational loading is required, and for the extreme load case, at least 50% contact must be maintained.

Table 3: Effective area and dimensions for circular foundations

Load case	Radius (m)	Area (m²)	A_{eff} (m²)	b_e (m)	l_e (m)	l_{eff} (m)	b_{eff} (m)
1	21.0	346.4	287.85	18.21	20.81	18.14	15.87
2	21.0	346.4	271.71	17.43	20.69	17.96	15.13
3	21.0	346.4	303.26	18.94	20.90	18.29	16.58

An additional problem that is experienced when gapping has been allowed for in design, is that it directly affects the stiffness of the soil, which can be mobilised in rocking. The k_θ value must be recalculated for a new rotation angle that has been factored in to account for the loss of contact area. Equation 7, adapted from Vestas (2011) can be used, where C_M is spring stiffness of soil in rotation:

$$k_\theta = \frac{M}{\phi} = \frac{M}{\frac{(\sigma_{max} - \sigma_{min})}{B_{eff} \cdot C_M}} \qquad (7)$$

It must be ensured that, in the extreme load case, the soil stiffness degradation due to this lower rocking stiffness will not increase the risk of differential settlements. It is vital that the turbine manufacturer's technical guidelines are consulted to assess how their specific turbines should be treated for the allowance of uplift, or whether it should be avoided under all loading conditions.

CONCLUSION

The complex loading associated with wind turbine structures poses an extreme challenge in assessing bearing capacity for wind turbine gravity foundations, particularly when taking large eccentricities due to high inherent moment loads into account. The DNV/Risφ (2002) method simply accounts for the effective area for circular footings and prescribes for additional bearing capacity checks for extremely eccentric loading. Gapping criteria and their effect on turbine footing design are also extremely important for the stability of the structure, with particular emphasis on the fact that 0% gapping should be allowed for in normal operating conditions. Gapping can be permitted under extreme conditions, although this should be subject to the manufacturer's specific guidelines, and stiffness effects should be accounted for.

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Slope failure on Lusaka-Chirundu Road in Zambia

Social geotechnics

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THE FOLLOWING DEFINITION of geotechnical engineering is suggested by a popular search engine: "Geotechnical engineering is a civil engineering discipline that is concerned with building on, in, or with soil and rock. Geotechnical engineers design dams, embankments, cuts, foundations, retaining walls, anchors, tunnels, and all other structures directly interacting with the subsoil, both onshore and offshore." [(1)] I assume that this definition would generally be accepted by most geotechnical engineers as an adequate definition of geotechnics. However, as Victor de Mello pointed out, we are first human beings, then civil engineers, and then specialists [(2)]. Therefore we need to be aware of the society we serve. This article sets out to answer this question by reviewing various news articles dealing with issues of geotechnics.

SURVEY

Over a one-week period (22–28 July 2015) a class of roughly 100 non-engineering second-year students at the University of the Witwatersrand (Wits) were instructed to find a news article pertaining to any aspect of geotechnics. Each article was read and analysed based on keywords (Table 1) and the following questions:

1. Which geotechnical engineering problems are addressed?
2. Which social aspects are associated with the geotechnical problem?
3. Is an expert or source quoted for technical aspects?
4. From which geographical region is the article?
5. When was the article published?

Although limited, the review carried out here is more than sufficient to get an overview. A more rigorous analysis of the articles can be the subject of future study.

Table 1: Keywords used to analyse papers

Aspect	Keywords
Geotechnical	Dam failure, Demolition, Earthquake, Erosion, Excavation, Floods, Foundation failure, Liquefaction, Mining, Problem soils, Research, Road damage, Settlement, Sinkholes, Slope stability, Soluble rock, Tailings dams, Tunnels
Social	Airport closure, Environmental damage, Evacuation, Fatalities, Financial loss, Inconvenience, Legal, Property damage, Relocation, Road closure, Safety
Expert	Department of Civil Engineering, Department of Geology, Geo-engineer, Geological Engineer, Geological Survey, Geologist, Geotechnical Engineer, Soil Engineer, Soil Scientist

OVERVIEW

A quarter of the articles submitted were written within the week assigned to acquire articles, half were written in 2015 and the earliest were written in 2001. The geographical distribution of articles was consistent with internet use (Figure 1), with over half the articles from North America and just under a quarter from Africa.

The most common word found in the titles of the articles was "collapse", typifying the content of most stories. The most common geotechnical problem raised by the articles was slope stability (Figure 2), followed by sinkholes and problem soils. In three quarters of the cases, an expert source is cited for technical aspects.

Of the social aspects, legal issues (Figure 3) were discussed most. Legal issues included arbitration, building codes, court cases, government debates, government inquiries, litigation, policy-making or political debate. A third of the articles cited either fatalities or safety issues. This is to be expected, as many dealt with the collapse of a structure. Closure of transportation infrastructure (roads, airports and railways) was also a significant social implication. More articles dealt with property damage and financial loss than environmental damage. A tenth of the articles highlighted that evacuation of people was required, but permanent relocation was only necessary in half as many cases.

PERSPECTIVE

A more nuanced perspective can be obtained by comparing articles dealing with similar geotechnical problems but within different social settings.

Apartheid dolomite

In the latter half of 2011, two articles were written on sinkholes, one in Gauteng, South Africa [3] and the other in

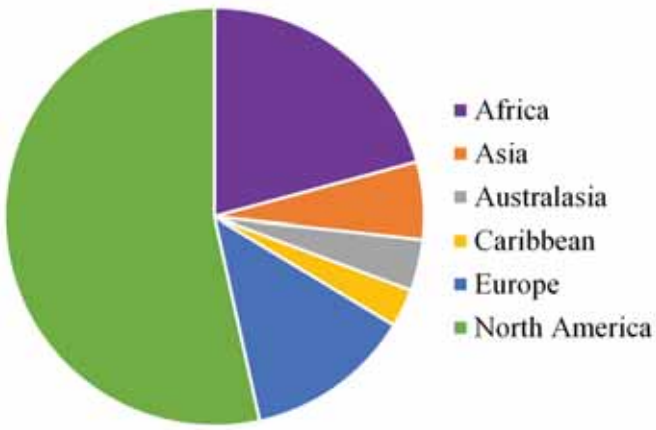

Figure 1: Geographical distribution of articles

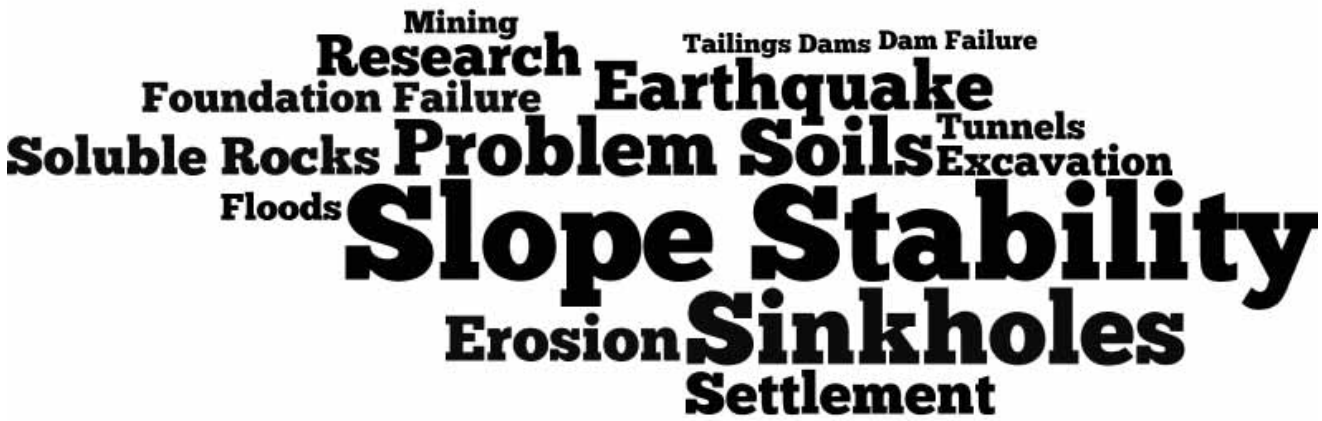

Figure 2: Common geotechnical problems raised by news articles (font size corresponds to frequency)

Figure 3: Social aspects associated with articles on geotechnical engineering (font size corresponds to frequency)

Florida, United States [4]. The main themes of these articles contrast the social settings of the two problems. The theme of the article in Gauteng addressed how best to protect vulnerable citizens from sinkholes, whereas the theme of the Florida article focused on how vulnerable people can make the most of insurance payouts.

Common to both articles is that engineering solutions can be used to mitigate the risk of sinkhole damage. However, in the case of the Gauteng citizens, limited access to finance in order to mitigate risk resulted in government bodies seeking to relocate them. The fact that more affluent citizens, literally across the road, were not required to move, presumably as their more robust structures were deemed to be at a lower risk, was seen as discrimination based on race and class. Whereas, in Florida the homeowners, in some cases, were receiving insurance payments of up to ten times their property value for mere cracks. Payments, in many cases, were not used to repair damage and mitigate future risk. The situation was worsened by collusion between lawyers and contractors, and litigation between parties. Government bodies, in this case, were seeking to put in place legislation to regulate insurance claims better and ensure that repairs were done.

These two stories highlight how completely different social issues can arise surrounding similar geotechnical problems, and these are often the most difficult to solve.

Deadly landslides

Landslides in urban areas are deadly events that can claim the lives of residents and cause extensive damage to property. From a geotechnical engineering perspective, classifying areas as high risk is not a difficult task. Costs associated with classifying areas, in many cases, would offset property and financial losses. Two articles, one from Japan in 2014 [5] and another from the United States in 2015 [6], highlight both technical and social difficulties around implementing landslide hazard maps. Both countries have attempted to implement landslide hazard maps, but the work to implement these has been hampered by shortages of funds and relevant staff. In both cases these delays were questioned following a devastating landslide.

Hurdles to implementing hazard maps are not merely due to funds and personnel not being available. In Japan, for example, there has been resistance to zoning as it leads to a devaluation of property and land being deemed unsuitable for construction. Similar concerns in the United States have led to State Legislatures shutting down hazard-mapping initiatives. This highlights that socio-political factors can often have an overriding influence.

Classifying areas as being at risk may be easy, but predicting when landslides will occur in order to permit timely evacuations, is a much harder task. In the case of the landslide in Japan [5], warnings were issued an hour after the event. This delay was blamed on the warning criteria that were not able to take into account extreme localised downpours. This is a potential area for research.

Infrasonic weapons

The final comparison presented here is between two inquiries into collapsed buildings. The two collapsed buildings were a guesthouse in Lagos (Nigeria) and a multi-storey apartment block in Morvant (Trinidad and Tobago). The guesthouse collapse was not finally attributed to foundation failure, but to structural deficiencies [7], although initial discussions considered the possibility. Slope failure is being considered as the main cause of failure for the apartment block collapse [8, 9]. Although collapse was not finally attributed to geotechnical problems in these two cases, media coverage of the two inquiries highlighted the importance of engineering ethics.

In the case of the guesthouse, absurd notions of infrasonic weapons were proposed as possible causes for the collapse [10]. Presumably to try and add scientific rigour to this proposition, an academic paper was published on the possibility [11]. The journal in which the paper appeared (*International Journal of Scientific Engineering and Research*) is a predatory journal of questionable repute. The article itself has entire paragraphs plagiarised from websites that, borrowing a biblical expression, can best be described as those that "creep into households and take gullible persons captive".

In the case of the apartment block, discourse was much more realistic. Although there was prior evidence of slope failure, the actual site investigation was not deemed comprehensive enough to fully quantify the risk [9]. The geotechnical engineer in question admitted failing to adequately perform his duties [8]. However, it was also apparent that a review of ground conditions carried out by another practitioner was withheld by the contractor. This inquiry is ongoing.

The SAICE code of ethics requires that members take personal responsibility for work completed, and that work be delegated to competent persons. Admittedly, taking responsibility for mistakes is not easy, but this is a basic tenant of engineering practice. Understanding failures is a key means for engineers to broaden their competence.

CONCLUSIONS

Whilst geotechnical engineering is ostensibly about building structures safely with, on or within soil and rock, the interaction of these structures with society is equally important. Failure of these structures can result in considerable social disruption. These range from fatalities to inconvenience, legal wrangling to financial loss, and closure of transport infrastructure to environmental damage.

Social issues do not merely arise following the failure of a structure. They can also arise when trying to mitigate risk. While it may seem prudent to warn or relocate people due to ground risks, such as dolomite or landslides, these warnings can be ignored. The cost to mitigate risk, be it social or financial, does not seem justified in many cases, even when money is made available.

Engineers have a duty to serve society with integrity, applying suitable skills, sound judgement and technical innovation to solve problems. When our solutions fail, we need to take the responsibility and not create smoke screens around the real issues. Exploring the reasons for failure can prevent mistakes in the future. Acting without integrity does a disservice to our profession, as it portrays us in a bad light.

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Variance of geotechnical properties and implications for the selection of design values

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Published results of investigations of variance in geotechnics are understandably scarce. The first study on soil variability was undertaken by Lump (1966), and only in the late 1990s was there renewed investigation. A seminal study was undertaken by Phoon and Kulhawy (1999), which investigated the variances of each parameter in terms of the coefficient of variance (CoV) for various techniques, such as triaxial testing, cone penetration testing (CPT) and standard penetration testing (SPT).

INTRODUCTION

Historically, geotechnical engineering has had little use for standard statistical analyses, due to the high intrinsic variability of the ground, small data sets and often zero replication, which render statistical analyses of geotechnical data rather meaningless.

This has had the undesired effect that geotechnical practitioners are often unaware of the enormous variation associated with a chosen value of a geotechnical parameter, often described as the "characteristic" (Eurocode 7) or "moderately conservative" / "cautious estimate" parameter (British Standards).

Published results of investigations of variance in geotechnics are therefore understandably scarce. The first study on soil variability was undertaken by Lump (1966), and only in the late 1990s was there renewed investigation. A seminal study was undertaken by Phoon and Kulhawy (1999), which investigated the variances of each parameter in terms of the coefficient of variance (CoV) for various techniques, such as triaxial testing, cone penetration testing (CPT) and standard penetration testing (SPT) (summarised in Table 1). Bond and Harris (2008) listed some other common parameters and their associated CoVs for UK data (shown in Table 2).

It was also found by Phoon and Kulhawy that the CoV was generally higher for sands than for clays, and that even results from 'elegant' techniques, such as CPT, exhibit a remarkably high variation, and that its recorded variation is also strangely similar to that of the 'cruder' SPT.

An intriguing observation was that for many parameters – such as the undrained shear strength (S_u), the angle of friction, plasticity index, liquidity index and relative density – the variation decreases with an increase in the value of the parameter, i.e. an inversely proportional relationship between variation and strength. This observation is summarised in Figure 1 (for the angle of friction to illustrate the principle).

It is therefore evident that from a low to a high strength, the intrinsic variation is not constant, and distinctly nonlinear.

Many studies have since been undertaken using probability theory and Bayesian theorem (Wang *et al* 2015). Most of these methods seem to be useful for academic research purposes, but are rather irrelevant for practitioners in industry, as they do not seem to offer any tangible advantages for real designs.

The aim of this study was to investigate the variation of in-situ techniques on various South African sites and to compare the findings with published values. The objective was to ascertain, using a boundary value problem, whether current limit state designs, such as Eurocode 7 and SANS 10160-5, accommodate this variation.

INTRINSIC VARIATION OF STRENGTH

Intrinsic variation in the ground is in two components, namely horizontal and vertical. On a uniformly stratified deposit it can be assumed that the horizontal component is kept to a minimum. For this study two sand deposits and a residual clay were selected. These are of estuarine (Isipingo Formation) and aeolian (Gordonia Formation) origin, and a clay stratum of residual norite of the Rustenburg Layered Suite. Cone penetration testing (CPT) and dynamic probe super heavy (DPSH) probing were carried out on the sand strata, and dynamic cone penetrometer (DCP) measurement on the residual norite.

Table 1: Variation of geotechnical parameters (after Phoon & Kulhawy 1999)

Test type	Parameter	CoV (%)
Triaxial	S_u	10–55
Cone Penetration	q_c	20–60
Vane shear	S_u	10–40
SPT	'N'	25–50
Pressure meter	E_{PMT}	15–65

Table 2: Variation of geotechnical parameters (after Bond & Harris 2008)

Parameter	CoV (%)
$\tan\phi$	5–15
c'	30–50
S_u	20–40
m_v	20–70
γ	1–10

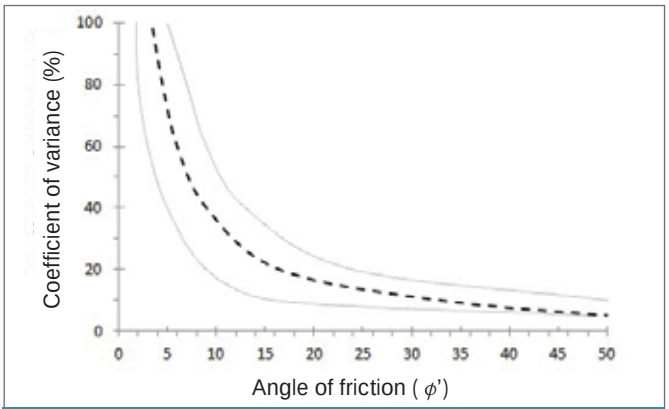


Figure 1: Example – mean angle of friction and its variance

The CoV was calculated for each depth increment using the following relation:

$$CoV(\%) = \frac{\sigma}{\mu} \times 100$$

where σ = standard deviation, and μ = mean.

It is assumed that only vertical variance is recorded and that this enables the determination of the variance of each investigative technique, and the variance with each respective soil strength increment. The ground strength gradually increases with depth, hence increase or decrease of variance with respective soil strengths can be measured.

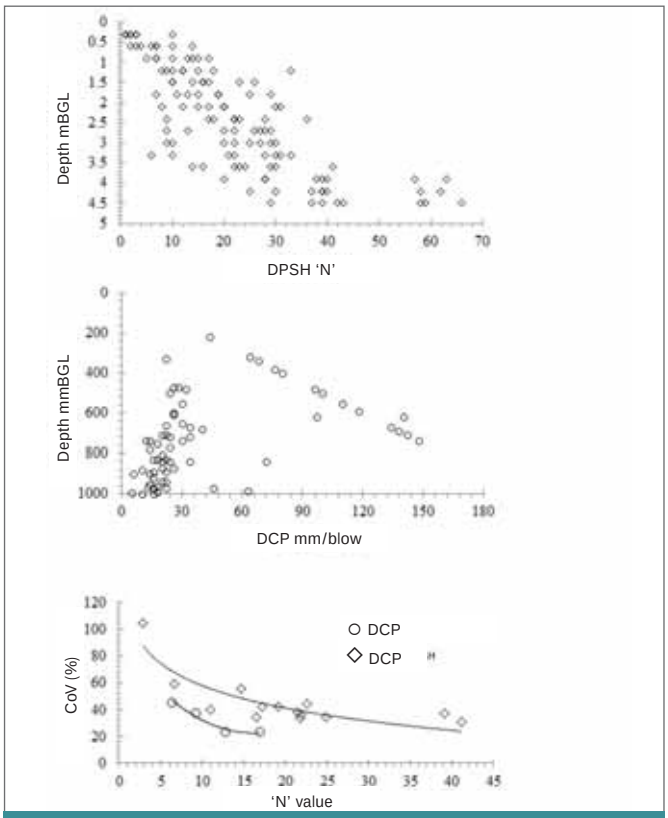


Figure 2: DPSH and DCP values with depth and the variance per consistency of each

Site 1: Gordonia Formation

DCP, DPSH and SPT probings were carried out on a site in the Northern Cape on aeolian sands. The SPT probings were not numerous enough, so only the DPSH data was used for the study of variation. The DCP and DPSH data and the 'N'–CoV relation are presented in Figure 2.

It can be seen that the variance does follow the same trend as the literature values, as shown in Figure 1, but are higher. The DCP was correlated to 'N' values using the relation of Brink (1982). The DCP's variance is lower than the DPSH's variance.

Site 2: Isipingo Formation

CPT, DPSH and SPT probings were carried out on estuarine sands with some intercalated clay strata in KwaZulu-Natal. The CPT was related to 'N' values using the relation suggested by Robertson (2014):

$$\frac{\left(\frac{q_t}{p_a}\right)}{N_{60}} = 10^{(1.1268 - 0.2817 I_c)}$$

where q_t = normalised tip resistance, P_a = 100 kPa, N_{60} = energy normalised 'N', and I_c = soil factor.

Fixing a value for the I_c factor is problematic for the 2–9 m depth, as much of the borehole information is unavailable. The available borehole logs record a sand stratum, whilst the CPT

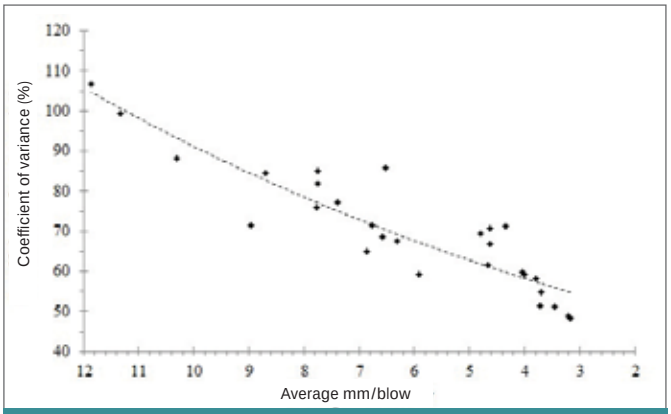


Figure 4: DCP values with depth and the variance

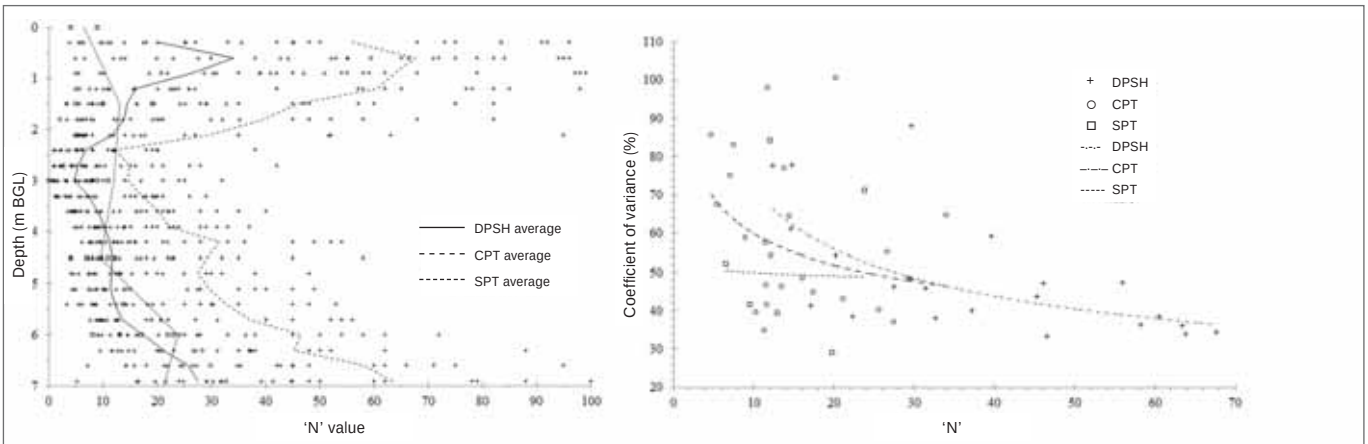


Figure 3: CPT, DPSH and DCP values with depth and the variance per consistency of each

Table 3: Parameters for each case

Case	Average CoV (%)	Range in 'N'	Range in friction angle (°)	Range in E (MPa)*
'N' = 20	60	13–32	31–37	13–32
'N' = 40	45	28–58	35–43	28–58

*For the Young's Moduli, the ratio of 1 'N' = 1 MPa was used (Clayton 1995)

probings calculate a 'clay' stratum (Robertson & Campanella 1983). Due to the effect of wash boring, which tends to wash out fines, even the borehole information may not represent the actual ground accurately. An average I_c value of 2.53 was selected from calculated values, and it also gives comparable 'N' values with those from the SPT. The results are shown in Figure 3.

It can be seen that there is remarkable correlation between the CPT and DPSH. The variance of even an elegant technique such as the CPT is high for 'N' values less than 40. The DPSH's variation is the lowest.

Site 3: Residual Norite of the Rustenburg Layered Suite

To ascertain whether fine-grained strata exhibit the same behaviour, as well as whether the size of a data set can affect the CoV, the CoV was determined for a site near Tshwane, where 35 DCP soundings were carried out to a depth of 1.5 m on completely weathered norite, recorded as stiff clay in test pits. The DCP has the advantage that a large data set can be generated at low cost. The CoV of this large data set is presented in Figure 4.

SUMMARY OF FINDINGS

In summary it can be stated that:

- In-situ testing techniques show the ground's intrinsic variation to be non-linear and with high scatter of data at low strengths. The CoV does not seem to decrease with a large amount of data.
- The implication of these results is that input parameters have much higher variation at low consistencies than at higher consistencies. This could have implications for limit state design in the case where parameters are factored.
- Variance is technique-dependent, with the SPT having much higher variance than the DPSH and DCP. Therefore the designer should take cognisance of the variance associated with each technique used to determine the design value.

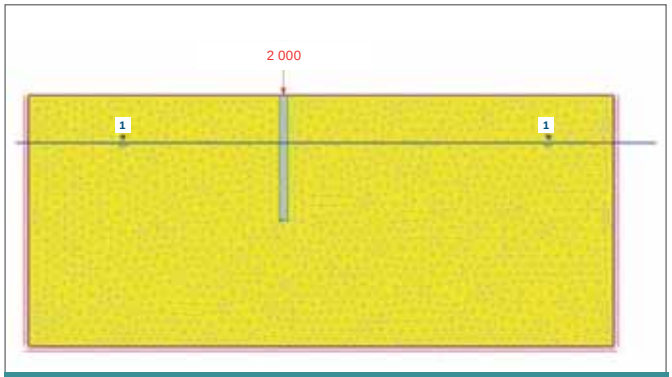


Figure 5: Finite element model for pile settlement

Table 4: Pile settlement analyses for the two cases

Case	φ' (°)	E' (MPa)	Settlement (mm)
'N' = 20	31	13	210
	33	20	161
	37	32	117
'N' = 40	35	28	133
	37	40	104
	43	58	76

SIMULATION OF PILE SETTLEMENT

To simulate the effect of variation of input parameters, a boundary value problem was used to test the effect of variance. Pile settlement was simulated using Rocscience's Phase 2 finite element program with a thick sand stratum. The relation between friction angle and 'N' value was determined using Peck *et al* (1974). Two cases were considered, shown in Table 3.

The pile was modelled as a 15 m long, 1 m diameter cast in-situ auger pile, and a loading of 2 000 kN was applied. For each case the upper- and lower-bound values were used as input parameters to determine a range of pile settlement. The outline of the model is shown in Figure 5.

The results of the analyses are given in Table 4.

It can be seen that the range of settlement due to the intrinsic variation of the input parameters is much more at a lower consistency.

Current limit state design codes, such as Eurocode 7 (2004) and the SANS 10160-5:2011, have partial factors for earth materials (material factors) which are used to derive a design value and are calculated to determine whether a particular limit state is satisfied:

$$X_d = \eta \frac{X_k}{\gamma_m}$$

where X_d = design value of geotechnical parameter, η = conversion factor, normally taken as unity, X_k = characteristic value or 'cautious' estimate, and γ_m = partial factor for the material.

The suggested values of the codes for the partial material factors are fixed between 1.25 and 1.4, irrespective of soil strength.

In the case of friction angle, the design friction angle (ϕ'_d) is related to the characteristic friction angle (ϕ'_k) by means of the material factor (γ_m) as follows:

$$\phi_d = \tan^{-1}\left(\frac{\tan\phi}{\gamma_m}\right)$$

If the average values of Table 4 are taken as 'characteristic', design values are arrived at using a material factor of 1.25 as per SANS 10160-5 and Eurocode 7, and settlements of the pile can be determined. This is presented in Table 5.

For the lower consistency case, the use of the material factor appears over-conservative, but as Figure 3 clearly shows, the CoV can reach up to 100%, and the settlement prediction using a 'mean' value will still be correct.

However, it is clear that the same value for the material factor (1.25), irrespective of ground strength, yields an over-conservative prediction of settlement at a high consistency.

DISCUSSION OF FINDINGS

Taking cognisance of variance for partial factors

Even an elegant technique such as the CPT exhibits high variation in homogeneous, stratified ground which is also distinctly nonlinear – high variance at low ground strength and low variance at high strengths. The observed variance for South African sites does appear higher than for the published values for overseas sites. For inhomogeneous ground, the variance is thought to be much higher.

It is also clear that each investigative technique records a different intrinsic variance. The DPSH technique does appear superior to the SPT, as it has lower variance. This variance may also be due to the ground being disturbed prior to the SPT sounding being carried out. The sensitivity of the CPT may account for the high observed variation, and the comparison to the cruder techniques may not be correct. Particularly if the designer only has SPT data, taking cognisance of its inherent high variability at low consistencies is essential.

Table 5: Settlement analyses using a material factor of 1.25

Case	ϕk' (°)	ϕd' (°)	E' (MPa)	Settlement (mm)
'N' = 20	33	30	10	225
'N' = 40	37	33	20	161

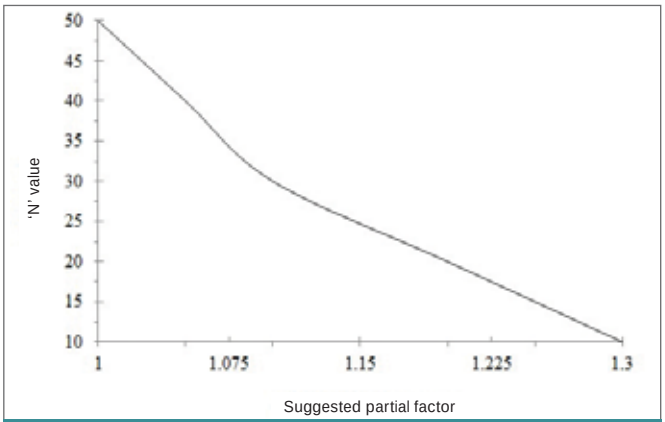


Figure 6: Suggested change to partial material factor in relation to consistency

The suggested values of the codes for the partial factors range between 1.25 and 1.4, irrespective of soil strength. This study shows that variance can range from 20–100% for the same ground using the same test technique on homogeneous ground.

It can be assumed that the use of the current values for the partial factors may lead to over-conservative designs at high ground strength. This is especially likely to occur when the designer has little ground data at hand.

It also shows that accurate predictions can be obtained using mean or average values for the characteristic values.

The variances found in this study suggest that, for a high strength material ('N' > 30), the partial factors can be decreased to 1.1, until verification is obtained from case studies. The suggested partial factors per strength range are presented in Figure 6.

Taking cognisance of each technique's effect on variance

It is also clear that each investigative technique records a different intrinsic variance. The DPSH technique does appear superior to the SPT, as it has lower variance. This variance may also be due to the ground being disturbed prior to the SPT sounding being carried out.

The sensitivity of the CPT may account for the high observed variation, and the comparison to the cruder techniques may not be correct. Particularly if the designer only has SPT data, taking cognisance of its inherent high variability at low consistencies is essential.

CONCLUSIONS

- The intrinsic variance of ground materials was found to be high and also varies with strength in a non-linear way. This trend was previously observed for nearly all geotechnical properties. The magnitude of the observed variance of various in-situ techniques on various South African sites suggests a higher variance than that for the overseas case studies.
- The observed trend of variance is similar on soils of different consistencies and composition, and, unexpectedly, even large data sets do not show a decreased variance.
- Variance is strongly technique dependent.
- As this study was carried out for uniform stratified ground, the variance of inhomogeneous ground is likely to be much higher.
- The findings of this study have implications for limit state design codes where fixed partial factors are used for materials, irrespective of ground strength. The current specified values for the partial factors for ULS and SLS are to be used, irrespective of ground strength. This may result in significant over-design on high strength materials and under-design at low ground strengths. It is suggested that materials are not factored at all, such as for Design Approach 1 Combination 1 or DA2. Good correlations have been achieved in France (using DA2) between predicted and measured deflection, using 140 pile tests (Burlon *et al* 2014). This suggests that, when geotechnical parameters are left unfactored, the predictions are accurate. This is particularly true if the data set is small.

REFERENCES

The list of references is available from the editor. ◻

SAICE Geotechnical Division News

Trevor Green
Chairman
SAICE Geotehnical Division
trevor@verdicon.co.za

THE YEAR 2016 IS ALREADY WELL UNDER WAY, and shaping to be a very active one for the Geotechnical Division. Aside from the traditional Jennings Lecture and a short-course on piling, both presented by Dr Mark Randolph in March, we are also looking forward to the 1st Southern African Geotechnical Conference in May, and several fascinating evening lectures that have been confirmed for the latter half of the year – these include presentations on dolomite by Dr Fritz Wagener, lateral support by Ken Schwartz, and a comparison of compaction techniques by Alan Parrock.

1ST SOUTHERN AFRICAN GEOTECHNICAL CONFERENCE

In early May this year anyone looking for a geotechnical engineer or engineering geologist to do their bidding is going to find it a rather fruitless search.

Following a lack of large-scale geotechnical conferences in South Africa for many years, this year is all about moving things up a gear. The 1st Southern African Geotechnical Conference takes place at Sun City on 5 and 6 May, and the response of the professional community to this has been such that plenary sessions are being planned so that all the accepted papers can be presented.

The conference will incorporate the Geoff Blight Memorial Lecture, with Prof Andy Fourie flying in from Australia to present the lecture. I was fortunate to be among the students to be lectured by both Prof Blight and Prof Fourie at Wits, and this is sure to be an occasion to remember.

The conference is being held under the auspices of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE), and the President, Secretary and the Vice-President for Africa have all been invited to attend.

As most engineers would have already guessed, the conference is only being held at Sun City due to its proximity to the Pilanesberg Alkaline Ring Complex. This is one of only three alkaline volcano complexes in the world. At over 2 000 million years old this is one old lady! At its peak the volcano was over 7 000 m in height, i.e. approximately 1 000 m higher than Mount Kilimanjaro (5 895 m). Now *that* would have been something to see! Obviously, the proximity of the conference to the casino and vertical water slides is purely coincidental.

17TH AFRICAN REGIONAL CONFERENCE

The ISSMGE African Regional Conference (ARC) has been held in South Africa only four times in the past – the 1st in 1955 in Pretoria, the 4th in 1967 in Cape Town, and the 6th and 12th in Durban in 1975 and 1999 respectively. So, despite South Africa having the majority of ISSMGE members in Africa, we have not hosted an ARC for over 15 years.

In 2015 Dr Denis Kalumba, from UCT, successfully presented our bid (at the ARC held in Tunisia) to host the 17th ARC in Cape Town (visit www.geotechnicaldivision.co.za for details). A great deal of work was put into preparing and presenting the bid, and thanks must go to all those involved.

The 7th African Young Geotechnical Engineers Conference will be held over the same period, so both young and not so young will have a forum at the conference.

MEMBERSHIP

The Geotechnical Division currently has 547 members of whom 495 are in good standing. All those who think they may be on the 'not-so-good' list should contact the SAICE Membership Department and query their membership status.

It is also worth noting that, at 536 members (out of a total of 945), we are by far the biggest member of the ISSMGE in Africa (with Egypt following at 108).

We really are quite an active division and there has been a big push recently to improve communication with our members. If you are not receiving regular news and updates, please e-mail Luzaan Hamel (luzaan@verdicon.co.za) and she will include you on the mailing list.

DIVISION AWARDS

At the Geotechnical Division's Annual General Meeting held towards to end of last year, the following awards were made:

Barry van Wyk Award

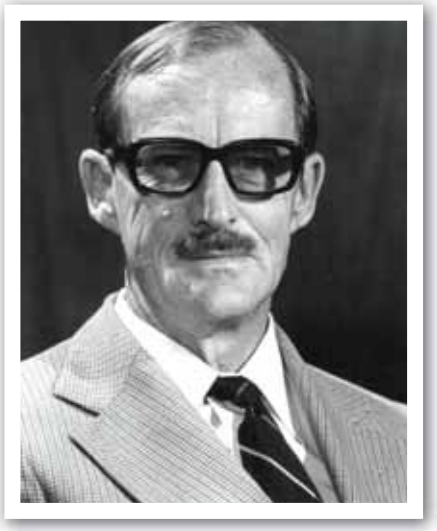
The award is presented annually to the author of the best final-year dissertation on a geotechnical topic at a South African university. David Marx from the University of Pretoria was the 2015 recipient for his dissertation *The tensile strength of unsaturated sands*.

JE Jennings Award

The JE Jennings Award is presented annually to honour the late Professor JE Jennings and the outstanding role played by him in the development of geotechnical engineering in South Africa. The award is presented to the author/s of a meritorious publication relevant to geotechnical engineering in South Africa published during the previous year, either in South Africa or elsewhere. The 2015 recipients were Profs SW Jacobsz, Wynand Steyn and Elsabé Kearsley for their publication *Centrifuge modelling of ultra-thin continuously reinforced concrete pavements*. It was published in the *Proceedings* of the 8th International Conference on Physical Modelling in Geotechnics 2014, which was held in Perth, Australia. □

David Marx (right) receiving the Barry van Wyk award from SAICE COO Steven Kaplan

Prof Elsabé Kearsley (left) and Prof SW Jacobsz, recipients of the 2015 JE Jennings Award, with SAICE COO Steven Kaplan (the third recipient, Prof Wynand Steyn, could not attend the event)



Dr AAB (Tony) Williams
1926 – 2016

DR AAB (TONY) WILLIAMS passed away peacefully at home in Johannesburg on 4 February 2016, just short of his 90th birthday, leaving his wife Veronica (whom he married in 1953), four children and six grandchildren.

Tony was born in Kokstad and attended school at Highbury and Michaelhouse. He joined the Navy during World War II, serving on a frigate out east.

He studied civil engineering at the University of the Witwatersrand where he obtained the degree BSc (Eng) in 1948 (and a PhD in 1976).

After three years in bridge and road construction with the then Transvaal Roads Department he went to Imperial College, London, where he obtained a DIC in soil mechanics. In 1954 he joined the National Building Research Institute (NBRI), then under the directorship of Jere Jennings, to work on unsaturated soils, where he also first met Tony Brink. He later spent ten years at the National Institute for Road Research as Head of the Soil Mechanics Division before returning to the NBRI. Tony retired from the CSIR as Head of Structural and Geotechnical Engineering in 1989, but continued acting as a consultant until final retirement in 1993.

A feature of Tony's research had always been an appreciation of the forces of nature when dealing with soil conditions in southern Africa, and the necessity of 'whole engineering'. Towards this end he advocated a multidisciplinary approach to work, drawing in people like Tony Brink.

He authored or co-authored some 63 publications covering the influence of climate on soil behaviour, field characteristics of South African soils, soil profiling, mapping and data storage, the role of engineering geology, soil suction measurements, slope stability (including slimes dams), foundations and roads on active clays, settlements on low-density residual and transported soils and the value of the then new impact roller, and the in-situ properties of fissured clays.

He is perhaps best known for his co-authorship with Jennings and Brink of their 1973 *Revised guide to soil profiling for engineering purposes in southern Africa* (which was adopted as an industry standard), his 1982 book, *Soil survey for engineering*, with Tony Brink and Tim Partridge, the state-of-the-art report with Terry Pidgeon and Peter Day on expansive soils for the 1985 Symposium on Problem Soils in South Africa, and the recognition that the shear strength along existing slickensides was close to the residual strength of the mass (typically about 50% of the peak mass strength). The importance of the latter is of course crucial for design locally, particularly regarding slopes, in contrast to the common overseas practice of using peak strength for analysing 'first time' slides.

What is less known is his compilation of the computerised *Bibliography of South African Geotechnical Publications*, covering the period 1947–1976, which also incorporated a keyword-in-context search and retrieval system.

Tony received a number of awards for his work, including a SAICE Best Paper Award in 1962 (with Basil Kantey), the JD Roberts Award for Building Research in 1980, the South African Geotechnical Medal in 1991, and the Gold Medal of the South African Institute for Engineering and Environmental Geologists in 2005. He was also due to receive the SAICE Medal for Meritorious Research at the 2016 Jennings Lecture – the official notification of this arrived in the mail on 4 February, and he read it with quiet enjoyment on the day of his passing.

Tony, who was a Fellow of SAICE, formerly served on Council, and on the committees of the Pretoria Branch and Geotechnical Division, as well as on the organising committees of several Regional Conferences for Africa on Soil Mechanics and Foundation Engineering. In addition, he contributed to many Division lecture courses, represented SAICE on the SABS 0161-1980 Code of Practice for the design of foundations for buildings, and chaired the development of the 1980 Code of Practice relating to the safety of men working in small-diameter vertical and near-vertical shafts for civil engineering purposes.

Tony was also a keen sportsman (squash), and an enthusiastic conservationist and ornithologist.

We salute a man who was a true gentleman, quiet in his manner, generous with his knowledge, honest in his assessments, and kind with his criticism.

Dr Frank Netterberg
With inputs from Fritz Wagener, George Dehlen and Peter Day
fnetterberg@absamail.co.za

Bryan Edward Tromp
1949 – 2016

BRYAN TROMP'S LIFE ebbed away peacefully on the night of 17 January 2016 following a protracted struggle of nearly two years with cancer, during which time, in spite of increasingly unbearable pain, painful treatment and diminishing mobility, he never lost his sense of humour and enthusiasm for life and his family, as he continued to press on towards the goal of defeating this insidious enemy.

Bryan's last two years developed into a special time of close family relationships with his wife Helen, their sons Brett and Warren, their spouses and five grandchildren, during which time also he came to know the Lord in an even more meaningful way which, I have no doubt, made bearable his final months.

Bryan was born in Pietermaritzburg and the family spent his early years in Bulawayo in the former Rhodesia. He completed his schooling at Roosevelt Park High in Johannesburg, before going on to Wits University where he obtained his Civil Engineering degree in 1971 under the tutorship of eminent engineering educators of the era, including Professors Jennings, Ockleston and Midgeley.

His professional career began with the Johannesburg City Engineers Department, then moved on to Gough Cooper Construction, developing an early interest in matters geotechnical, combined with housing, and on to the (then) fledgling geotechnical engineering practice of Steffen, Robertson and Kirsten (SRK) in 1977, at which time he also met Ken Schwartz, with whom he formed the practice Schwartz Tromp and Associates (STA) in 1980.

Bryan was registered as a Professional Engineer in 1981 and soon became recognised for his skill in all aspects of geotechnical engineering and engineering geology.

STA developed into a geotechnical consulting practice of great repute, professionalism and integrity over the succeeding 15 years. When Ken left the practice in 1995, Bryan approached me with a view to merging our respective geotechnical practices, while retaining the STA brand, and so began a roughly 20-year association during which I got to know Bryan well as professional engineer, business colleague, entrepreneur, considerate employer, engineering mentor, family man and close friend.

Bryan's main professional interests of relevant geotechnical site investigation work and design of special foundations for the civil engineering and building industries, were encapsulated in his published works and appointment to technical committees formed to establish Codes of Practice for foundations for residential units, and structural and serviceability assessment criteria for housing units, and his later appointment to the NHBRC peer-review panel for dolomite-related works.

In spite of the 'modern tendency' to reduce many engineering problems to numerical analysis and mathematical modelling, I knew Bryan to generally reduce most problems first to simple hand-calculations, from which to derive first-order estimates, before accepting the computer-generated results.

And this was his great strength – a down-to-earth, hands-on geotechnical specialist, together with a well-developed 'feel' for the engineering properties, based on the well-established southern African soil profiling criteria, established by the likes of our eminent geotechnical predecessors, Jere Jennings, Tony Brink and Tony Williams.

Bryan's consideration for all our staff members was always exemplary, as was his mentorship of young engineers and geologists into the so-called 'black art' of geotechnical engineering. His business acumen came to the fore in our negotiations which led to STA being successfully absorbed into the global engineering and environmental practice of Golder Associates in 2009, where we continued to operate as their all-Africa geotechnical arm.

Apart from family, Bryan's great loves were cricket, and his dogs – the most beautiful Golden Retrievers which accompanied him to the office most weekends, often terrorising the other members of the practice hierarchy who dared to invade 'their master's space', and sometimes also accompanied us as our security detachment on remote site investigation operations. Our fun forays onto the golf course, sadly too few in number, without exception resulted in much golf being played and much territory being visited.

In commemorating the passing of a great friend and fellow professional engineer of the highest ethics and integrity, our thoughts turn to Helen, Brett and Warren, their spouses and grandchildren, as they grieve over their most special one – Bryan Tromp.

To Almighty God be the glory for a life lived well.

Alastair Morgan
Former business partner, fellow engineer and friend
alastair@geoid.co.za

JG AFRIKA:
Engineering a new future, developing together

SOUTH AFRICA remains the beloved country; a land of great opportunity. So, too, the continent of Africa.

One engineering and environmental consultancy has pledged its commitment to the country, its love for the continent, and its dedication to a transformed and democratic Africa. To express this commitment, and in celebration of its continued independence, the firm, formerly known as Jeffares & Green, embarked on a rebranding exercise at the end of 2015.

Now known as **JG Afrika,** the company and its staff are excited about the message they are sending – a message that tells the world that Africa has a lot to offer. "Our name change speaks to our commitment to being proudly South African. We want to make a bold statement that we are locally owned and managed, and plan to remain so. The company has a rich heritage and history in Africa. We are very excited about the future and remain committed to our beloved continent," confirms Phakamile Ngqumshe, Director and Johannesburg Branch Manager.

The inclusion of 'JG' in the company's new name denotes its acknowledgement of and appreciation for its history, while 'Afrika' indicates its independence, its love for the continent, and is a nod to the traditional spelling of 'Africa'. "This is most obviously represented in our first democratic National Anthem, *Nkosi Sikelel' iAfrika*. With this name, we will show the world that we are true to our African roots, while remaining unique and maintaining our independence," believes Ngqumshe.

"The brand development started with the selection of a new name, and after much research the selected options were presented to our staff and a vote held. We really enjoyed the process of evaluating the naming options and involving our staff," says Paul Olivier, Managing Director of JG Afrika.

Paul Olivier
Managing Director of JG Afrika

Phakamile Ngqumshe
Director, JG Afrika, and Johannesburg Branch Manager

The firm announced its new name to clients in February 2016, and launches the new brand throughout Africa in April.

"The brand identity was developed and designed with a purpose – to remember the company's history, to reflect its ethos and project its future," says Olivier. "The logo's icon is representative of man-made, engineered, symmetrical lines. These lines are contrasted with organic shapes which represent the environment (green) and water (blue), denoting the environmental sphere of JG Afrika's services. The design and name incorporate the three pillars of the company's ethos – experience, quality and integrity – while displaying fresh, innovative thinking."

The JG Afrika personality is perfectly portrayed through the new brand colours, being blue and green. In addition to the environmental connotations of these colours, they are associated with trust, dependability, strength, peace, growth and health. These characteristics reflect the company's culture.

"In planning for 2016, part of our goal for the new year was to sustain the advancement and success that we have achieved for the past 94 years. Over this period, the company has progressed and evolved to keep pace with fluctuations in demand, the industry and customer requirements. To remain relevant, this must be a continuous process," says Olivier. "As such, a strategy plan was meticulously devised to take JG Afrika to the next level on all fronts."

As the African proverb goes: *If you want to go quickly, go alone. If you want to go far, go together.* This is the basis of JG Afrika's long-term plans. "Together we will continue to grow, learn and develop, with a focus on continuous improvement. The time has come to look to the future and to align our corporate identity with our diverse expertise, our modern approach and the great future Africa has as a growing continent," concludes Olivier.

▶ INFO

www.jgafrika.com

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SAICE Training Calendar 2016

Course Name	Course Dates	Location	CPD Accreditation Number	Course Presenter	Contact
GCC 2015 (Third Edition)	9–10 May 2016	Cape Town	SAICEcon16/01869/19	Benti Czanik	cheryl-lee@saice.org.za
	12–13 May 2016	George			
	23–24 May 2016	Upington			
	26–27 May 2016	Bloemfontein			
	6–7 June 2016	Mafikeng			
	9–10 June 2016	Polokwane			
	13–14 June 2016	Kimberley			
	28–29 June 2016	Nelspruit			
	16–17 August 2016	Pretoria			
	6–7 September 2016	Midrand			
GCC 2015 and GCC 2010 Differences	31 May 2016	Durban	SAICEcon16/01890/19	Benti Czanik	dawn@saice.org.za
	20 June 2016	George			
	27 July 2016	Cape Town			
	1 August 2016	Port Elizabeth			
	22 August 2016	Polokwane			
	29 August 2016	East London			
Project Management of Construction Projects	19–20 July 2016	Midrand	SAICEcon15/01754/18	Neville Gurry	cheryl-lee@saice.org.za
	13–14 September 2016	Durban			
	27–28 September 2016	Cape Town			
	4–5 October 2016	Port Alfred			
	11–12 October 2016	Bloemfontein			
Technical Report Writing	30–31 May 2016	Durban	SAICEbus15/01751/18	Les Wiggill	cheryl-lee@saice.org.za
	29–30 June 2016	Midrand			
	25–26 July 2016	Port Elizabeth			
	1–2 August 2016	East London			
	5–6 September 2016	Cape Town			
	17–18 October 2016	Midrand			
Structural Steel Design to SANS 10162-1-2005	24 October 2016	Midrand	SAICEstr15/01726/18	Greg Parrott	cheryl-lee@saice.org.za
Reinforced Concrete Design to SANS 10100-1-2000	25 October 2016	Midrand	SAICEstr15/01727/18	Greg Parrott	cheryl-lee@saice.org.za
Practical Geometric Design	21–25 November 2016	Midrand	SAICEtr13/01418/16	Tom Mckune	dawn@saice.org.za
Business Finances for Built Environment Professionals	9–10 June 2016	Midrand	SAICEfin15/01617/18	Wolf Weidemann	dawn@saice.org.za
Handling Projects in a Consulting Engineer's Practice	6–7 June 2016	Midrand	SAICEproj15/01618/18	Wolf Weidemann	dawn@saice.org.za
Leadership and Management Principles and Practice in Engineering	4–5 May 2016	Midrand	SAICEbus15/01784/18	David Ramsay	dawn@saice.org.za
	8–9 June 2016	Cape Town			
	17–18 August 2016	Durban			
	14–15 September 2016	Port Elizabeth			
Water Law of South Africa	20–21 September 2016	Durban	SAICEwat13/01308/16	Hubert Thompson	dawn@saice.org.za
	27–28 September 2016	Cape Town			
	20–21 October 2016	Midrand			
The Legal Process dealing with Construction Disputes	18–19 May 2016	Mthatha	SAICEcon13/01368/16 SACPCMP/CPD/15/010	Hubert Thompson	dawn@saice.org.za
	24–25 May 2016	East London			
	31 May–1 June 2016	Port Elizabeth			
	26–27 July 2016	Polokwane			
	2–3 August 2016	Nelspruit			
	23–24 August 2016	Bloemfontein			
Earthmoving Equipment, Technology and Management for Civil Engineering and Infrastructure Projects	20–22 July 2016	Midrand	SAICEcon15/01840/18	Prof Zvi Borowitsh	dawn@saice.org.za
Sanitary Drainage Systems for Buildings	27 May 2016	Durban	SAICEwat12/01103/15	Vollie Brink	dawn@saice.org.za
	10 June 2016	Midrand			

SAICE / Induna Training Services

Course Name	Course Dates	Location	CPD Accreditation Number	Course Presenter	Contact
Comparing Construction Contracts	29–30 July 2016	Cape Town	SAICEcon15/01855/18	Lydia Carroll	dawn@saice.org.za
	8–9 September 2016	Durban			
FIDIC Contracts and Agreements – A Practical Approach	10–11 May 2016	Midrand	SAICEcon15/01774/18	Kevin Spence	dawn@saice.org.za
	29–30 August 2016	Cape Town			
	16–17 November 2016	Durban			
Management of Claims under FIDIC	16–17 November 2016	Durban	SAICEcon15/01775/18	Kevin Spence	dawn@saice.org.za
The Implementation of the Model Services Agreement including the Role of the Engineer	23 June 2016	Midrand	SAICEcon15/01858/18	Peter Glass	dawn@saice.org.za
New Engineering Contract	27 May 2016	Durban	SAICEcon15/01856/18	Lydia Carroll	dawn@saice.org.za
	8 July 2016	Port Elizabeth			
FIDIC Claims and Dispute Resolution	27 June 2016	Cape Town	SAICEcon15/01857/18	Peter Glass	dawn@saice.org.za

SAICE / South African Road Federation (SARF)

Asphalt: An Overview of Best Practice	5–6 July 2016	Durban	SAICEtr15/01806/18 SARF15/5001/18	J Onraet	sybul@sarf.org.za / tshidi@sarf.org.za
Assessment and Analysis of Test Data	25–26 May 2016	Durban	SAICEtr15/01805/18 SARF14/0001/17	R Berkers	sybul@sarf.org.za / tshidi@sarf.org.za
Concrete Road Design and Construction	29 June 2016	Cape Town	SAICEtr15/01802/18 CSSA-N-2013-08	B Perrie, Dr P Strauss	sybul@sarf.org.za / tshidi@sarf.org.za
	20 July 2016	Durban			
	20 September 2016	Gauteng			
	27 September 2016	Port Elizabeth			
Pavement Rehabilitation by Recycling / Bitumen Stabilisation	14–15 June 2016	Gauteng	SAICEtr15/01810/18 SARF15/0004/18 SAICEtr15/01807/18 (Assignment) SARF15/0041/18	Prof Kim Jenkins, D Collings, K Louw	sybul@sarf.org.za / tshidi@sarf.org.za
	10–11 August 2016	Bloemfontein			
Construction of G1 Bases	18 July 2016	Cape Town	SAICEtr15/01809/18 SARF14/9103/17	E Kleyn	sybul@sarf.org.za / tshidi@sarf.org.za
Stormwater Drainage	6–10 June 2016	Cape Town	SAICEtr15/01808/18 SARF12/0107/15	C Brooker, Matt Braune, Alaster Goyns	sybul@sarf.org.za / tshidi@sarf.org.za

SAICE / Classic Seminars

The NEC Contract	6–7 June 2016	Jhb/Midrand	SAICEcon13/01448/18	Nicolette Calasse	admin@classic-sa.co.za
	13–14 June 2016	Durban			
	20–21 June 2016	Cape Town			
	3–4 November 2016	Jhb/Midrand			
	10–11 November 2016	Durban			
	17–18 November 2016	Cape Town			

SAICE / Mentoring 4 Success

Course Name	Course Dates	Location	CPD Accreditation Number	Course Presenter	Contact
Introduction to Structured Mentoring in the Workplace	24 May 2016	Midrand	SAICEbus16/01893/19	Philip Marsh / Celestine Jeftha	info@m4s.co.za
Generational Differences and Learning Styles Workshop – in the Engineering and Construction Workplace	14 June 2016	Midrand	SAICEbus16/01889/19	Philip Marsh / Celestine Jeftha	info@m4s.co.za
Behavioural and Emotional Dynamics – in the Engineering and Construction Workplace	15 June 2016	Midrand	SAICEbus16/01888/19	Philip Marsh / Celestine Jeftha	info@m4s.co.za
Foundations in Structured Mentoring in the Workplace	25 May 2016	Midrand	SAICEbus16/01894/19	Philip Marsh / Celestine Jeftha	info@m4s.co.za
Mentors Masterclass in Engineering and Construction	13–14 July 2016	Midrand	SAICEcon14/01675/17	Philip Marsh / Celestine Jeftha	info@m4s.co.za
Knowledge Mentoring in Engineering and Construction	23–24 August 2016	Midrand	SAICEbus16/01886/19	Philip Marsh / Celestine Jeftha	info@m4s.co.za
Kick-start Structured Mentoring Programme	Book on request	–	SAICEbus16/01887/19	Philip Marsh / Celestine Jeftha	info@m4s.co.za
Head Start Structured Mentoring Programme in Engineering and Construction	Book on request	–	SAICEot14/01701/17	Philip Marsh / Celestine Jeftha	info@m4s.co.za
½-day, 1-day and 2-day courses are based on a minimum of 10 participants. In-house courses and workshops are limited to a maximum of 15.					

Candidate Academy

Road to Registration for Candidates	18 May 2016	Midrand	CESA357-04/2016	Allyson Lawless	lizelle@ally.co.za
	2 August 2016	Durban			
	12 September 2016	Midrand			
Road to Registration for Mature Candidates	24 May 2016	Durban	CESA484-01/2017	Stewart Gibson	lizelle@ally.co.za
	23 August 2016	Midrand		Peter Coetzee	
	7 September 2016	Cape Town		Peter Coetzee	
	15 November 2016	Durban		Peter Coetzee	
	1 December 2016	Midrand		Stewart Gibson	
Basic Contract Administration and Quality Control	31 Aug–2 Sept 2016	Durban	CESA359-04/2016	Theuns Eloff	lizelle@ally.co.za
	9–11 November 2016	Midrand			
Getting Acquainted with Road Construction and Maintenance	6–8 June 2016	Midrand	CESA379-05/2016	Theuns Eloff	lizelle@ally.co.za
	5–7 September 2016	Durban			

Candidate Academy

Course Name	Course Dates	Location	CPD Accreditation Number	Course Presenter	Contact
Pressure Pipeline and Pump Station Design and Specification – A Practical Overview	20–21 October 2016	Cape Town	CESA376-05/2016	Dup van Renen	lizelle@ally.co.za
Getting Acquainted with GCC 2015	5–6 May 2016	Midrand	CESA377-05/2016	Theuns Eloff	lizelle@ally.co.za
	11–12 August 2016	Durban			
Getting Acquainted with Sewer Design	19–20 July 2016	Midrand	CESA378-05/2016	Peter Coetzee	lizelle@ally.co.za
	13–14 September 2016	Durban			
	22–23 November 2016	Cape Town			

In-house courses are available.

For SAICE in-house courses, please contact Cheryl-Lee Williams (cheryl-lee@saice.org.za) or Dawn Hermanus (dawn@saice.org.za) on 011 805 5947.

For Candidate Academy in-house courses, please contact Lizélle du Preez (lizelle@ally.co.za) on 011 476 4100.

Civillain by Jonah Ptak

For more engineering humour, please visit "Unreal Bridges" on Facebook and "@TheUnrealBridge" on Twitter.

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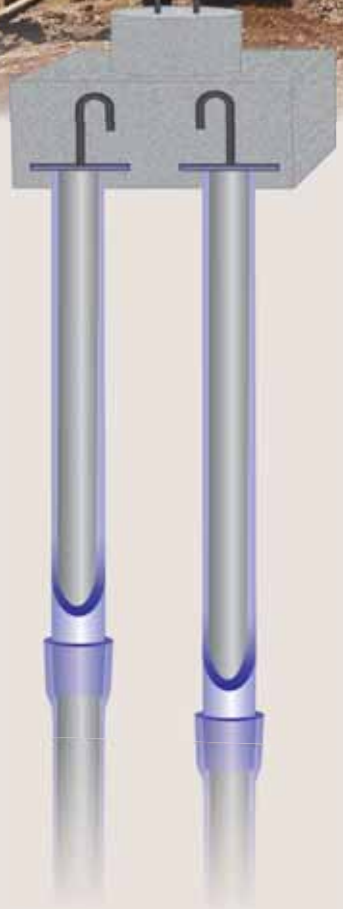
Greene Group	Mosomo
BA Engineers	Many more



- Requires **little preparation** of site working platforms
- Access to the most **challenging working conditions**, ranging from low head room (as low as 4.5m) to congested fully active sites
- **Extremely versatile and adaptable** so even if unexpected geotechnical conditions are encountered, it creates minimal disruptions to the piling programme.
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1. TRM product guarantee with limit of liability of 10 million Euros
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4. TRM piles have substantially higher corrosion resistance than steel, giving up to 100 years structural service life
5. Soil conditions are improved during TRM pile installation
6. No spoil or mess is generated
7. Vibration is minimised to below 1mm/s causing no damage to adjacent structures
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9. All pile head trimming is included in Geopile Africa's scope of works,
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