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Klip-Tite

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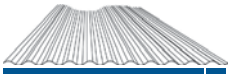
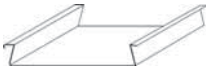


GRS- Corrugated



GRS- Bond Lok

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Rhodes must fly

I don't care whether Rhodes falls (or has fallen). It makes no difference to me. I don't care whether he walks or stands under pigeon pooh. The narrative is a chaotic concoction of the literal over the allegorical, garnished with the ultimate spice of mob mentality. A potion brewed for mass seduction for all the wrong outcomes. The sooner we make the distinction between the physical, the allegorical, and the demands to become a mature democracy, the sooner we will get over ourselves and start doing some serious nation building. Until then, Rhodes can do the twist if he wants to – I'm happy to reserve a seat and buy popcorn.

But I would prefer to invest the same enthusiasm in my family, my work and contributing to winning that elusive cricket world cup. That's the real issue here and a worthier cause – to play in the premier league and to win the world cup. How do we get Rhodes to fly?

Our dramatic exit from the world cup depressed the soul of the cricketing patriot in me. But to the receptive, learnings emanate out of every suffering. Singular heroic exploits and individual pockets of excellence are inadequate to bring the coveted trophy home. Not even an AB de Villiers cracking the ball like its Diwali is going to drag me out of bed in the small hours of the morning – not any more. I'll catch the highlights on the Blitz channel.

We need an integrated strategy that engages all our relevant people and resources, irrespective of age, colour, creed, and historical or contemporary affiliations. We must summon the wisdom of our senior cricketers whom we idolised in the 1990s, as well as the knowledge of the leading sports universities, psychologists, scientists and physicians, to make our boys lean, mean flying machines, and to play with the flair and abandon of schoolboys playing backyard cricket, but on the premier league stage.

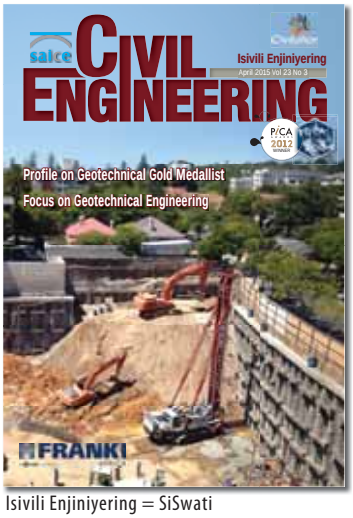
Next we need the contribution of the business sector in relation to sponsorship, investments in research and research institutions, partnering with government to subvent remuneration, implementing of new technologies at ground level, and cricket development initiatives that will give all aspiring Proteas a fair and equal chance to one day wear the green and gold and to feed the first class league with world class cricketers. It's the nature of business to make hay while the sun shines. We can therefore count on the private sector to respond. Kudos to the current sponsors who cashed in, in the most recent world cup – business has time and again risen to the occasion when the opportunity and climate to participate were engaging.

This brings me to the politicians and the department of sports. The last thing we want from the politicians is to collude with the administrators on who to pick for the squad – please no SMSs, no WhatsApps and no telephone calls. Their role is to influence and facilitate the networks through policies and appropriate officials working in efficient government departments to enable the sporting environment to be engaging. Their job is to empower the free market to connect with one another, so that the cricketers and cricket administrators can focus on cricket without having to worry about offending the side imperatives of the regime of the day.

It would be wonderful to have the minister of sport be a healthy individual, educated in the realms of academic and actual international sporting experience rather than *intellectually bankrupt* rhetoric.

Then we need a brilliant plan, whose ultimate goal is to win the world cup. If we do this, Rhodes will fly, just as he did in the 1993 world cup, when he tripped on his shoe laces to spectacularly run out the beefy Inzamamul Haq. This reminds me of that one final ingredient – lady luck. Gary Player summarised the meaning of luck beautifully when he said: "Well, the harder I practise, the luckier I get." If we get all of the above right, our boys will have very little to distract them from practising.

If they are enjoying the game – Rhodes will fly, indeed. ◻



Isivili Enjiniyering = SiSwati

ON THE COVER

Over the years Keller's Franki Africa has performed landmark geotechnical work in southern Africa and beyond, providing interesting, innovative and often brilliant solutions for a vast range of challenging geological conditions. Such conditions contributed to Franki being awarded and successfully completing the Citadel Basement Parking Project in Claremont, Cape Town.

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Winner of the 2009 Pica Award
in the Construction, Engineering and Related Industries category for the third year running

ON THE COVER P4

▶ The Citadel Basement Parking Project in Claremont, Cape Town, posed considerable geotechnical challenges that were successfully addressed by the Franki Africa team

Franki Africa – once again the geotechnical bastion of Claremont

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Franki Africa

once again the geotechnical bastion of Claremont

The Citadel Basement Parking Project in Claremont, Cape Town: the four-level basement measures about 50 x 50 m, and is 12 m deep at the deepest end

INTRODUCTION

Over the years Keller's Franki Africa (Franki) has performed some landmark geotechnical work in southern Africa and beyond, providing interesting, innovative and often brilliant solutions for a vast range of challenging geological conditions. Such conditions contributed to Franki being awarded the Citadel Basement Parking Project in Claremont, Cape Town.

FAMILIAR TERRITORY

Franki senior contracts manager, Jim Oldknow, says: "Having carried out numerous basement projects in the Claremont area, we are able to anticipate the conditions and create the most cost-effective solutions. Because of this we are often the recommended tenderers, or we are requested to negotiate the contract to the financial budget, as was the case with the Citadel contract."

Oldknow adds that the type of basement construction on this project posed a number of challenges with regard to both the general design and the design of the lateral support requirements. "The soils on this site consist of a 3–4 m layer of transported silty sand beneath which is high-weathered decomposing granite consisting of weak clay in the form of kaolin. This extends to a considerable depth of 20 m. A perched water table rests on top of these kaolin clays and is visible within the first 2 m from natural ground level.

"The basement, consisting of four levels, is approximately 50 x 50 m, with a depth of 12 m at the deepest wall height. One of the major difficulties on this project was the restricted access to the site, with only two access and exit points for materials, both exiting onto the very busy Cavendish and Warwick Roads in the Claremont CBD. This was exacerbated by the small footprint of the site, combined with the depth of the basement. All of this severely limited our ability to provide suitable access ramps for the heavy plant," Oldknow explains.

"From a geotechnical perspective the central challenge was that the very low shear strength of the kaolin, in which ground anchors had to be installed on this project, has a limiting effect on the maximum possible anchor loads. This, combined with the disintegrating qualities of the kaolin when exposed to ground water, results in nominal maximum anchor loads of 450 kN."

It is well known that lateral support movements can be considerable in these soils. And the perched water table certainly did not make things any easier! This had to be constantly managed to ensure that the in situ soils would not become saturated, making the work platform inaccessible or unstable. As the lateral support progressed the perched water table was caught behind the gunite arches and transported to excavation levels by drains. This required constant dewatering in order to prevent the disintegration of the kaolin in slurry/clay. Oldknow explains further: "Key to the success of the project was the creation of a suitably hard standing area and a dewatering system at the final level to provide a safe work platform for the installation of the foundation piles."

Naturally the choice of piles in any geotechnical project is critical. "Given the depth of the kaolin, Franki Driven Cast-in-Situ piles (Franki piles) were preferred to Continuous Flight Auger (CFA) piles as the more efficient solution. This was mainly because the Franki piles' founding could be achieved at a shallower depth by forming an enlarged base at the toe of the pile," says Oldknow.

He adds that the CFA piles (which were used for the soldier piling) would have to have been installed at a considerable depth given the weak shear strength of the kaolin, and would have required

the removal of the spoil material from the final basement excavation level.

SUMMARY OF SCOPE OF WORKS

- 21 000 m³ bulk earthworks
- 1 870 m² of lateral support consisting of excavation in 1.5 m stages in depth
- Soldier piling comprising 147 No 500 mm Ø CFA piles using a B125 Casagrande drill rig
- Construction of the concrete capping beam
- Trimming and construction of gunite arches via dry-mixed, air-driven guniting
- Installation of ground anchors and temporary steel walers at designed intervals
- Installation of temporary dewatering system and sacrificial piling platform
- Construction of temporary access ramp for foundation piling rig access at base level
- Installation of Franki pile 600 mm Ø foundation piles using a Franki British crawler piling rig; this rig will be removed on completion, using a 200 ton mobile crane, which operation will require the temporary closing of Cavendish Road.

CONCLUSION

Franki had a 35-week programme from when the project had been awarded on 21 July 2014 to completion on 27 March 2015, which included the lateral support, bulk excavation, dewatering and foundation piling.

"In spite of the normal Claremont winter rains, which are always challenging given the kaolin soil conditions, we are (at the time of the writing of this article) on time in terms of the anticipated completion date. In fact, we have managed to provide an earlier start for the building contractor by giving partial handover of half the completed pile layout. This allows the contractor to overlap with our contract, which provides him with a two- to three-week head start on his contract programme.

"While there have been many challenges on this project, we have been able to move ahead efficiently. The excellent relationships we have with our client, professional team and contractors have, of course, been critical to the success of the project. Franki Africa has an experienced design-and-construct team, especially in this geographical area. In the face of severe challenges we have successfully provided a lateral support design-and-construct solution, one that we have developed over the last 15 or more years in the Cape Town area," concludes Oldknow.

QUANTITY SUMMARY

- 1 870 m² of lateral support
- 147 No 500 mm Ø soldier piles
- Foundation piling 210 No 600 mm Ø Franki DCIS piles
- 21 000 m³ bulk earthworks

INFO

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One of the major difficulties on this project was the restricted access to the site

The perched water table had to be constantly managed to ensure that the in situ soils did not become saturated

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URETEK

Going above and beyond challenges set

A delighted Prof Chris Clayton (left) receiving the Geotechnical Gold Medal from Stanford Mkhacane, SAICE 2014 president

In 2014, SAICE's Geotechnical Division awarded its prestigious Gold Medal to a non-South African for the first time – Prof Chris Clayton of the University of Southampton. A familiar and esteemed figure not only in the South African geotechnical fraternity but worldwide, his first textbook, *Site Investigation*, became the reference work for the practising engineer, opening up the building and construction process in its entirety. Publishing profusely, his commitment and zeal in the geotechnical engineering field, first as a contractor and later as an academic, has generated a lasting body of knowledge. Coupled together with his generosity, adaptability, talent and hard work, colleagues and friends can only describe the soft-spoken, modest Chris as a superstar.

Born in Hemel Hempstead to the north of London, Chris's early years were spent in Nigeria – his father being in the Colonial Service. His African tie for then, however, was short-lived when he returned to boarding school in the UK.

Particularly strong in physics and "probably told by one of those people who carry out aptitude tests that I should study civil engineering", Chris did just that at the Polytechnic of Central London (now the University of Westminster). In his third year, he elected to take a course in foundation engineering, as "I was convinced that it would be a good career move because of the very small number of people opting for the course – there was only one other student in the class with me."

In 1970 he graduated with a BSc (Hons) Civil Engineering, and made three job applications, all to specialist geotechnical and site investigation contractors. While waiting to hear if he had been accepted for any of the jobs, Chris and his girlfriend Mary, to become his wife later that year in December, did a month-long driving trip through Europe as far as the Black Sea. This would be a real adventure, and apart from crossing behind the iron curtain, they survived on a meagre £40 each. Travelling, a mutual passion, would become a constant thread in their lives, seeing them visit almost every continent over the years.

ACTIVELY PURSUING DEVELOPMENT

Returning from the trip, and finding his father had accepted a job offer on his behalf, Chris went to work as a specialist geotechnical engineer in site investigation and civil engineering contracting at Nuttall Geotechnical Services Ltd. "This would be a key starting point. It was a small, high-quality company where I gained a lot of experience, not just in technical terms, but also in terms of learning how to handle people." For the next couple of years Chris worked mainly on site, but also on interpreting and reporting on ground investigations.

Working on site would also test the young Chris's mettle, as the work was carried out amidst a bitter national construction strike, with flying pickets (mobile strikers) leaving the contractors reeling with the damage done to plant and equipment. Looking back, the seemingly imperturbable Chris readily admits that: "I'm not entirely sure I was in control, but if you get chucked into the deep end, you just have to learn how to swim. I was just 24 years old and some of the people working for me on site were tough – one guy got caught running guns into Northern Ireland."

Two short years later, in 1972, he was appointed Nuttall's site agent for the final site investigation for the Brenig Reservoir in North Wales, a large contract involving the construction of two trial embankments, a borrow pit and a trial quarry where he would oversee the high-quality drilling, sampling and *in situ* testing for the project.

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After a couple of years at Nuttall Geotechnical Services, Chris approached management and asked for advice on getting a better geotechnical education. "They suggested that I get a Fulbright Scholarship, but not having heard of such a thing I didn't do anything about it. A while later I received application forms from Imperial College London for a Master's course in soil mechanics. It turned out that the MSc programme's admissions tutor was a friend of Nuttall's managing director, Dr Derek Cornforth, who himself had been a researcher at Imperial College.

"Taking the MSc course at Imperial College, I was taught by its world-leading academic staff, such as Profs Skempton, Bishop and Hoek. This and working with my fellow students (four of whom would become full professors at other universities) would set my whole career in motion." In 1973 Chris obtained his MSc with distinction.

On his return to industry he went to work for Ground Engineering Ltd, a subsidiary of the international contractor, John Laing. He carried out site investigations, construction claims and small research projects, as well as CPD lectures for structural engineers. As a senior soils engineer he oversaw a number of major ground investigations, including a large section of the M3 highway, and the UK Channel Tunnel terminal which involved a large trial cutting across a landslip area. Here he was also responsible for both the management and technical standards for the company's geotechnical laboratory.

While there Noel Simons, a University of Natal graduate (now the University of KwaZulu-Natal) and civil engineering professor at the University of Surrey, encouraged Chris to pursue a PhD. "Since I was already mixing research funded by John Laing with my day-to-day routine work, I decided to combine commercial and academic research, and started on a collaborative PhD with the University of Surrey. The basis for my PhD – the problems associated with using a weak rock (chalk) in civil engineering earthworks, came from the work I did for John Laing's civil engineering contractual claims – the 1970s saw millions of pounds' worth of claims against the government on the basis of trafficability and settlement of highway embankments."

UNIVERSITY LIFE

After being awarded his PhD in 1978, Noel Simons prompted him to apply for an academic post at the University of Surrey, and Chris was subsequently appointed as a lecturer in geotechnical engineering. Chris wryly looks back at this decision: "I thought this would be an easy way of making a living compared with being a contractor – but how wrong you can be!"

Initially course director for the new part-time industry-based MSc in geotechnical engineering, he later became a Reader and in 1992 became professor of geotechnical engineering. In 1993 he succeeded Noel Simons as Head of the Department of Civil Engineering at Surrey.

Despite the aggressive financial cuts that Margaret Thatcher's government would bring to the academic community, the hands-on Chris found a balance between teaching, research and administration.

As Prof William Powrie of the University of Southampton said in his introduction to Chris's delivery of the 50th Rankine lecture in 2010, "Chris is recognised as being foremost in research that is of practical relevance, high quality, thoughtful and careful, reflecting his background as a contractor, of which he is justifiably proud." Prof Powrie went on to say: "Chris's work has always been carried out methodically and with great care, sometimes developing new techniques and instruments such as the Hall effect transducers for use in triaxial tests, for example,

to ensure that the quality of data is something with which he feels comfortable."

As such Chris is also always the first to approach when there is a challenging job – he was, for example, an external Member of the UK's Health and Safety Executive Board that controlled the investigation of the 1994 Heathrow Express trial tunnel collapse.

As an academic, he has authored such well-known books as *Site Investigation* and *Earth Pressure and Earth-Retaining Structures*, and has written a number of industry-guidance documents for the Construction Industry Research and Information Association (CIRIA – a member-based research and information organisation dedicated to improvement in all aspects of the construction industry). He has published over 200 journal and conference papers making important scientific contributions in the areas of sample disturbance and sample design, investigation by remote sensing, *in situ* testing including such diverse areas as SPT and field geophysics, and laboratory element testing.

Of his incredible accomplishments Chris, casually pushing them aside, says: "It's just expected", and where he can help, he will, even to guiding friends' PhD research in such diverse areas as chemistry and history.

THE SOUTH AFRICAN CONNECTION

Not long after becoming HOD at Surrey, in 1993 Chris was given the opportunity to come out to South Africa to checkup on University of Surrey students who were doing a year's industry work in the country. Despite landing up in the wrong hotel in Johannesburg and spending the night listening to the sounds of semi-automatic gunfire, Chris would regularly return to South Africa.

He looks back humorously at the hotel mishap: "The next day I phoned Prof Eben Rust (of the University of Pretoria whom he had supervised as a PhD student at Surrey) and it was discovered there had been a terrible mistake with the bookings." He continues chuckling: "All was cleared up then, but it was a terrible thing to do to someone. Eben showed me a different side of South Africa the next day."

Striking up a lasting friendship with Chris, Eben asked him if he would come out for a conference that the University of Pretoria (UP) was hosting in 1994 on '*In situ* testing in geotechnical engineering' and be a keynote speaker – one of an incredible number of keynote speaker invitations Chris would agree to over the years. It would also be the beginning of close ties for the benefit of UP – since then Chris has been a visiting professor at the university. Eben elaborates: "I wanted my best students to do their PhDs in the UK to expose them to the resources, the tutors, the way of thinking and working there, because it broadens the scope tremendously and makes a huge difference."

Chris says: "This was great for me because the standard of engineering education in South Africa is excellent. I was able to cherry-pick the best SA graduates to work in my research team – over the years I've supervised ten, and we have done world-leading research together." Chris has received four Jennings Awards for the papers he has published with South Africans, and in 2006 he delivered the fifth Jennings Memorial Lecture.

WORLD-CLASS LAB AT UP

Another of the great contributions Chris would come to make in South Africa was helping to establish a geotechnical testing laboratory at UP.

In 1978 Chris had setup Surrey Geotechnical Consultants, a small consulting practice, persuading Hardev Sidhu, Chris's chief technician while in industry, to join him. Doing mainly consulting in the early years (and from working out of Hardev's garage), with focus, hard work and commitment it grew into a specialist geotechnical testing house carrying out the best geotechnical laboratory testing in the world for some of the most challenging modern engineering projects, one being Dubai's 321 m tall Burj Al Arab Hotel.

The reputation of this laboratory for carrying out thoughtful, high-quality tests which attest to Chris's approach to his professional activities, had Eben, upon seeing its facilities, immediately ask him to create a quality lab like that at UP. And with that Chris arrived at the airport with a suitcase full of lab equipment, and the Soils Laboratory at UP came into existence which still today competes with the best.

SAICE TECHNICAL JOURNAL

The support Chris would lend also expanded to the SAICE technical journal and the internationally-accredited status it enjoys today.

Chris also sowed the seeds for the geotechnical work done within Rail Research UK, which grew out of a project carried out by Hannes Gräbe (at the time of Spoornet, nowadays Prof Gräbe of UP – another South African PhD student whom Chris had supervised) on the effect of principal stress rotation on the performance of railway formations.

With Eben as the journal chairman in the early 2000s and wanting to get the publication to a level such as *Géotechnique*, he once again called upon Chris, who had served on numerous editorial panels, including *Géotechnique*'s advisory panel (from 2009–2011 he would be its editor), and who had also been founding editor of the Institution of Civil Engineers' (London) Proceedings, *Geotechnical Engineering*. With Chris's help, a new model for the ailing SAICE journal was decided on, a system the journal still uses today. As a result the journal grew from strength to strength, and Chris remains as joint editor-in-chief assisting in overall strategic matters.

FURTHER CONTRIBUTIONS

In 1999, preferring to move away from the management side of his role at Surrey to continue with research, Chris moved to the University of Southampton, becoming Professor of Infrastructure Engineering, and the collaboration with UP moved to this campus. Of this change Prof William Powrie says: "He threw himself wholeheartedly into research … He won a research contract for cyclic testing of chalk fill in connection with the behaviour of embankments on the Channel Tunnel Rail link and the eminent Professor Clayton was seen in denim jeans in the laboratory, elbow deep in chalk slurry as he worked alongside the researchers to prepare specimens for testing which became the talk of the tearoom."

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of Spoornet, nowadays Prof Gräbe of UP – another South African PhD student whom Chris had supervised) on the effect of principal stress rotation on the performance of railway formations.

Other significant research included fundamental investigation into the behaviour of methane hydrate-bearing sediments which saw the first realistic artificial sample of this sediment made in the world.

NOT ALL WORK

"The extra work required for South African commitments, though, has always been more than compensated by the friendship, travel and sport that have typically accompanied it – after UP's short conference in 1994, a visit to the Kruger followed and in 1995 I was invited by Eben to visit Sodwana – which of course meant learning to dive."

With licence in hand, Chris has since dived all over the world, the most unusual being in the shark-infested waters of the Bikini Atoll in the Marshall Islands, a location where between 1946 and 1958 nuclear tests were carried out by the United States on the reef and where only a select few ever go.

On the travel front there have been superb family trips, with South African colleagues who have become friends, to the Cape, the Karoo, the Wild Coast and KwaZulu-Natal.

His family, the apple of his eye, includes historian wife Mary, electronics engineer son Tom and art curator daughter Ellie, of whom he'd much prefer to talk about than himself.

Rebekka Wellmanns
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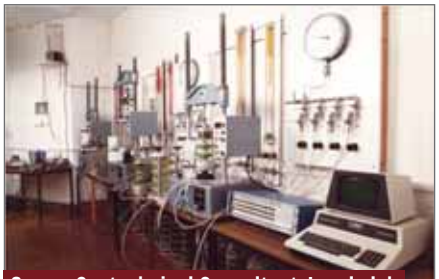
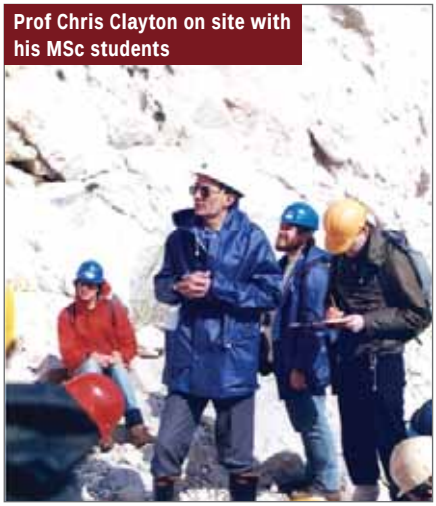

Surrey Geotechnical Consultants' early lab


Hannes Gräbe (left) and Chris (right) working on rail research


Chris diving in the Bikini Atoll (Photo credit: C D Jackson)


Prof Chris Clayton on site with his MSc students


Chris doing 1.8 m dia 500 tonne plate testing


Kind and hospitable Chris invites his PhD candidates to visit his home in Surrey where his garden barbeques are a fixture


Enjoying being amongst friends and family in the Kruger National Park in 1994. From left: Nico Vermeulen, Gerhard Heymann, Chris's wife Mary, Chris, Eben Rust, his wife Jeanne, Martin Rust and Sjanie Rust.

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Rock engineering for Boschkop intake works

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INTRODUCTION

In 2006, Schwartz Tromp (Pty) Ltd (geotechnical engineers) were engaged by the project engineer of Vaal Pipeline Consultants (VPC) to provide specialist geological and geotechnical engineering services to accompany the rock excavation for the construction of a pump chamber and intake structure sited in the Vaal Dam reservoir.

The project involved an inter-basin water transfer by 2 m diameter pipeline from the Vaal Dam, roughly 120 km across the watershed, to the power and gas-from-coal plants situated around Secunda.

The work was done on behalf of the Trans-Caledon Tunnel Authority (TCTA) for the Department of Water Affairs (DWA), as part of the Vaal River Eastern Sub-System Augmentation Project (VRESAP), and was peer-reviewed by an independent specialist.

PRELIMINARY GEOTECHNICAL ASSESSMENT

The initial rock slope and support design by VPC had been based on earlier preliminary geotechnical studies including rotary core drilling at several locations, which generally straddled the intake footprint.

It was recognised, however, and supported by the results of initial rock excavation work, that a need existed for ongoing investigation and interpretation, in parallel with and providing relevant direction to the construction process.

Construction of the intake structure proceeded in the following stages, in all but the first and last of which the author had direct involvement:

- A selected material berm – dozed into the reservoir from the adjacent hillside to facilitate cofferdam construction;
- An approximately 15 m deep rock cut in the hillside to provide the initial construction platform;
- A contiguous piled cofferdam drilled through the berm and socketed into bedrock;
- Intake excavation in stages to the base of the piling, with installation of two reinforced concrete thrust arches, and with prestressed strand anchors to resist the water load;
- Excavation of the intake void to full depth, approximately 25 m below reservoir level, and directly beneath the inside face of the cofferdam; and
- The formation of an approach channel within the reservoir, and demolition of the cofferdam – two of the last elements of excavation that followed construction and equipping of the full intake facility.

The shaft design, based on earlier exploratory drilling, assumed that competent, intact and hard to very hard rock andesite (i.e. UCS anticipated well in excess of 50 MPa) would characterise the excavation. Nominal rock bolting was envisaged, readily integrated into the excavation programme such that the design slopes and cut faces would be acceptably maintained.

However, the first construction inspections undertaken by the author presented a very different view of the rock mass response to excavation. An extremely ragged face resulted from the contractor's pre-split blasting to a 'near-vertical' profile – laid back at roughly 45° to 75° from the horizontal, as a consequence of significant wedge failures.

MODIFIED DESIGN FACTORS

It therefore became apparent at an early stage of excavation that extremes of differential weathering in the andesite bedrock, and adversely inclined discontinuities, i.e. rock mass structure, would play a significantly more important role in shaft stability than simplistic rock strength considerations.

Based on this conclusion therefore, supplementary studies of the geological setting and background data, as well as comprehensive additional (core-drilling) investigations were commenced, importantly needing to fit into the construction schedule and substantiating on-going geotechnical decision-making, a process which continued throughout the entire period of shaft excavation.

The following key issues emerged during the study, which had a direct impact on the direction the construction programme took, in view of the extensive rock stabilisation measures that became necessary:

- The potential instability of the hillside cut which would tower up to 40 m above the final shaft floor;
- Significant variability (and depth) in founding material for the cofferdam piling;
- Relative instability of the shaft-head/working platform as a consequence of adverse rock structure;
- Variability in the rock mass across the (roughly 40 m diameter)

shaft footprint, including zones of complete andesite weathering to full excavation depth; and
- A non-characteristic extensive joint/fault or shear plane, uncovered in the last stages of excavation, which potentially threatened cofferdam stability.

This article describes these various elements of additional study and their outcomes, and addresses the important need (as has so often been stated before with regard to ground engineering works) for extending and integrating the geological and geotechnical modelling and interpretation into the construction process.

DATA GATHERING AND DESIGN PROCESS

Construction sequence

The modified geotechnical design had, of necessity, to mesh with construction as outlined below, with on-going monitoring, investigation and stability interpretations at essentially every stage:

- Extension of part of the construction terrace by cut-to-fill into the reservoir basin;
- Completion of the rock excavation into the hillside to a level approximately 2 m above reservoir level (due to the poor rock profile resulting from the earlier excavation works, this activity was accompanied by exposed joint surveys, inclined rotary core drilling, rock mass classifications, analysis and design of additional rock support, including rock bolting, mesh and shotcrete);
- Installation of the contiguous piled cofferdam by high-capacity piling auger, inclusive of proof-drilling of the pile–rock contacts and pressure grouting the contact zone; and interpretations of load transfer effectiveness;
- Excavation of the rock (intake) shaft, mucked out initially by excavator and ADTs, followed by kibble and crane. This was accompanied by (1) rock-bolting, mesh and shotcrete, the pattern of which was verified by continuous face monitoring on a cut-by-cut basis, and (2) pre-stressed strand anchorage of key sectors of the footprint (NOTE: A challenging aspect of excavation control proved to be maintenance of stability at the shaft-head and securing acceptably safe working conditions in the deepening shaft below);
- Installation of RC arch thrust beams and pre-stressed (strand) anchors to resist potential movement of the cofferdam piles, due to water load;
- Excavation of the link-up zone between the cofferdam inner face and main shaft excavation – this sector presented a significant continuous joint/fault/shear zone feature, the presence of which potentially threatened the stability of the piled cofferdam, discussed in detail below (see Photograph 1 taken before this feature was excavated); and
- Final excavation of the approach channel and removal of the cofferdam, following pump chamber and intake construction, which were elements of work undertaken directly under VPC supervision.

Geological setting

Desk study of available data, background geology, and interpretation of the geological setting (Brink 1979) provided the following key inputs:

- Country rocks comprise an outlier of the Ventersdorp Supergroup, a "massive accumulation of andesitic to basaltic lavas" of 2 700 Ma age.
- The project site is located close to a prominent contact between the andesite and overlying Karoo Supergroup, which are overlain by extrusive sills of dolerite. The contact zone with the Karoo is interpreted within about 250 m of the intake site.
- The weathering environment has been generally interpreted as characteristic of the regional sub-humid dry zone (with annual rainfall transitional between 500 mm and 750 mm), with occasional outcrops of sound lava, but with bedrock more generally blanketed by up to 12 m of residual soil.
- Notable exposures of the andesite, in the general vicinity of the

Photograph 1: Bird's-eye view of intake excavation – before removal of final cut beneath cofferdam

project site, confirm this transitional nature of the formation, i.e. from unweathered to completely weathered variants.

DATA OBTAINED

Intake terrace cut

Inclined rotary-cored boreholes into the partly exposed platform cut produced the results below, with core recoveries generally in excess of 90%. Throughout the process, essentially hard rock to very hard rock andesite was proven, but with significant weathered zones and closely-spaced discontinuities as reflected by the RQD values (percentage of full core pieces in excess of 100 mm in each drill run). Joint orientations were directly surveyed at this time by visual observation and conventional compass surveys:

- 0 m to 5 m into cut face – RQD: 50% to 66%
- 5 m to 10 m into cut face – RQD: 18% to 69%
- 10 m to 15 m into cut face – RQD: 43% to 56%

The joint surveys of the exposed cut face confirmed a number of adversely-inclined joint sets consistent with the earlier wedge failures which accompanied the initial cut.

Rock support at this level was necessarily designed with the ultimate objective of securing the 'vertical' shaft cut intended to extend 25 m to 30 m beneath this terrace.

The notable outcome of this initial study (verified in much of the succeeding monitoring work) was the observation that a high-strength (intact) rock mass could not be generally relied upon to characterise the shaft construction.

It became self-evident that close monitoring of all future excavation would be essential to respond to the varied conditions and to ensure safe working conditions as the excavation deepened.

Rock mass structure

From this initial work, it also became clear that additional investigation data would be essential to characterise the various sectors of the 'nominally circular' shaft footprint, with on-going interpretation to provide rational decision-making on rock support.

Conventional joint surveys were undertaken for each successive cut, as the site was progressively exposed, and the resulting stereoplots analysed to determine rock stability criteria. The failure modes predictably varied for the differing sectors of the shaft, as did the measures applied for support.

Rock mass classifications, done according to the Barton (Q-system), Bieniawski (geomechanics classification) and Laubscher (mining rock mass classification) methods, to provide a preliminary basis for judging excavation stability, were interpreted into support provisions made by visual assessment of the discontinuities and judgement based on the recommendations outlined by Barton (1974), Stacey and Page (1986), and Stillborg (1986).

Rotary core drilling

Several additional phases of rotary core drilling followed during the construction monitoring programme – some with oriented core, with logging and determination of rock mass criteria – to provide appropriate decision-making, as follows:

- Initial platform cut – inclined holes to establish behind-face conditions at depth, and to search for specific adversely-inclined discontinuities and possible deep-seated failure modes;

- Contiguous piling – vertical holes through sleeves in piles to determine the quality and to verify the integrity of the pile–rock contacts, and interpretation of contact grouting criteria;
- Main shaft excavation – inclined holes on several trajectories to determine rock quality variances, and as forward planning for sidewall support;
- Final shaft excavation up to the piled cofferdam – inclined and vertical holes with core orientation to establish joint/shear/fault zone characteristics, and design of additional rock support measures for cofferdam stability.

Intake shaft excavation

The available data, assembled from the pre-construction investigation and in-construction drilling, were analysed to provide shaft excavation parameters and support inputs.

The data was categorised into depth segments, i.e. from the construction terrace downward to the deepest point of excavation.

It was found useful, as a construction expedient, to consider the core recovery and RQD values in common elevation segments, in order to have relevance to the shaft support designs.

In spite of relatively good core recoveries being achieved over most of the 'bottom half' of the shaft footprint, the many instances of 'low RQD' values indicative of a poor rock mass, due to weathering, were notable at depth.

It was also noted, albeit on the basis of a limited sample of results, that the vertical drill holes appeared to present higher RQDs over the 'lower half' of the shaft than those recorded for inclined boreholes, which gives substance to the statement attributed to Bieniawski (Palmstrom 2005) that "RQD is a practical parameter for core logging, [but] it is not sufficient on its own to provide an adequate description of a rock mass".

In any event, the construction experience proved the validity of interpretations based on the widely varying scenarios suggested by these drill holes.

Cofferdam foundation

The cofferdam foundation, integrity and quality of pile–rock contacts, and the overall cofferdam stability condition could only be finally appreciated once the shaft excavation was substantially advanced, with the cut to the face of the cofferdam (Photographs 1, 2 and 3).

The following key elements, which indicate the andesite rock mass variability, were clearly distinguishable and were considered in greater detail:

- The remarkably located prominent joint/shear/fault plane evident in the centre of the cut beneath the cofferdam; this was identified by the geological team, as well as by independent review, as a possible shear or fault plane, consistent with the andesite intrusion. It was interpreted as having the potential to destabilise the cofferdam.
- Pile (weathered to competent) rock contacts.

The pile founding/socketing materials could now be seen totally exposed, supported partially on 'sound' or weathered andesite rock (locally completely weathered to residual soil and accompanied by notable seepage), and elsewhere on massive tooth-like (large bouldery) spheroid features (Photograph 2).

The spheroid features were interpreted as likewise leading to potential instability for the affected cofferdam piles, and were identified for specific anchorage before the final excavation.

DATA INTERPRETATION

Intake terrace cut

The rock support design was based on the following parameters and criteria:

- The key failure surface is predominantly a two-plane wedge, sliding on a completely weathered, smooth to rough and wavy joint surface, oriented slightly obliquely to the slope face and dipping adversely into it.
- The main release plane is roughly perpendicular to the slope face, sub-vertical and varies from a slightly weathered condition to completely weathered with joint gouge or filling > 25 mm (up to 250 mm observed) thick.
- Joint spacings are typically in the range 0.3 m, 0.5 m and 1.5 m.

The minimum rock support recommended, allowing for safe working in the future shaft up to 30 m below, included:

- Close-spaced rock bolting placed specifically to deal with failure from a combination of two-plane wedge and toppling modes, which could be identified from visual inspection of the initial cut of the terrace face. (NOTE: In defining the rock bolting pattern, it was considered appropriate to superimpose rock bolting positions on a photographic image which was presented as the construction drawing).
- Flexible mesh and lacing were specified as restraint for individual blocks with a tendency to loosen and fall out from within this support pattern, as a consequence of subsequent blasting operations.
- Shotcrete was applied in view of the requirement for a permanent final treatment for long-term slope protection.

Intake shaft excavation

The main shaft support recommendations were based on the following interpretations and criteria:

- The shaft rock mass was interpreted as widely varying, albeit comprising a high percentage of relatively competent and intact (hard to very hard) rock andesite, (estimated UCS > 50 MPa) but classifying as a 'poor' rock mass.
- Rock support, as is often the case, was not required to specifically stabilise deeply-seated joints and massive rock failures, but to rather create a 'nominally pre-stressed ring' to secure the excavation and prevent local 'fallout' of unfavourable wedges and blocks, which could be readily dislodged by blasting and precipitate more extensive collapse.
- The difficulty and disruption of re-establishing equipment to drill and install support once the shaft excavation had deepened, meant that decisive (albeit sometimes conservative) instructions were needed to support each cut level before deepening the shaft.
- The likelihood of groundwater seepage, driven by the high reservoir level, and the probability of erosion of (soil quality) joint filling and completely weathered pockets, and the need for pressure-relief holes installed beyond the depth of restraint.

Ground support recommendations were based on the Barton support classes 17 and 18 in respect of the 'best case' zones, and 35 in respect of the 'worst case' zones, details of which were interpreted by ongoing site inspections:

- Rock-bolting of 4 m minimum length, but up to 6 m, where stability was controlled by major joint sets;
- Shotcrete generally in the order of 20 mm to 30 mm thick, but likely to be substantially in excess of 200 mm in areas of significant over-break; and

Photograph 2: Inner face of cofferdam showing additional anchored thrust block and massive andesite 'spheroids'

Photogaph 3: Final rock cut beneath cofferdam showing discontinuities and support measures

- Mesh reinforcement to be applied in all but the 'best case' situations.

Cofferdam foundation

Additional drilling, analysis and interpretations provided parameters for stability analysis and rock support design for the cofferdam foundation to precede removal of the final rock buttress.

The outcome of these studies,and the factors noted above, resulted in the following support measures (also see Photograph 3):

- An additional thrust beam to encase and stabilise the pile bases where completely weathered andesite was identified, together with pre-stressed 600 kN strand anchors installed in like manner as previously.
- Excavation of the remaining rock buttress, to a depth of approximately 10 m below the piled cofferdam, was limited to roughly 2.5 m to 3 m cuts, each followed by a row of pre-stressed 600 kN strand anchors.
- In view of the interpreted potential for a succession of joint planes parallel to the exposed major joint/shear plane, the strand anchor installation depths were staggered alternately by half the fixed length in order to vary the rock restraint 'zone of mobilisation'.
- Provision of pore pressure relief holes to control the reduction in effective stress on potential failure surfaces.

CONCLUSIONS

The experience arising from the rock excavation for the intake provides further ample validation for the 'design-as-you-construct' approach to geotechnical engineering for major and underground rock structures, an approach that has been proposed by many previous practitioners involved in this necessarily complex activity.

Pre-construction geotechnical investigations, however well-conceived and budgeted for in terms of time and money, are ultimately and inevitably overtaken during construction, simply by the infinitely variable nature of the ground and the need to respond quickly to the needs of the project, both from a technical and safety point of view, as well as the realities of construction.

Much prior and collective experience can be mobilised to broaden the interpretation and understanding of the limited data available from any prior investigation, which is, at best, a fairly random and limited sampling of the activity under review.

By extending the investigation process into the construction programme, the necessary resources to facilitate its execution are quickly applied, interpretations made and decision-making sharpened to achieve the end result.

By definition therefore, projects of this nature need to be carefully resourced with these concluding thoughts in mind.

ACKNOWLEDGEMENTS

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Engineering geological considerations for three dams in Nigeria

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INTRODUCTION

Aurecon has been responsible for the detailed design of three large dams in Nigeria, including the necessary geological and geotechnical investigations and considerations. The locations of these dams are shown in Figure 1.

KASHIMBILA DAM

The Kashimbila Multipurpose Dam is located on the Katsina-Ala River in the Taraba State. The dam is a 32 m high, 1 245 m long composite dam. The main elements of the dam comprise a concrete spillway with integrated outlet works, a hydropower station and a clay core rockfill embankment. The storage capacity is 500 million m^3.

In broad terms, Kashimbila Dam is located on rocks of the Precambrian undifferentiated basement complex, predominantly comprising granite gneiss. These basement rocks have been intruded by younger dolerite dykes. The area of interest is located between the Benue Trough, an ancient and non-active rift system, and the Oku Volcanic Field in Cameroon. These Precambrian crystalline basement rocks on which Kashimbila Dam is located are typically stable, non-seismic areas.

The dam was initially conceptualised to act as a buffer dam in the case of natural embankment failure at Lake Nyos in Cameroon, where the failure of Lake Nyos could generate an extreme flood, endangering the population in south-eastern Nigeria. The principal aim of the Kashimbila Dam is thus to provide sufficient storage for flood absorption, should such a failure occur. During the detailed design phase the dam was optimised to also provide potable and irrigation water to surrounding towns. A 40 MW hydropower station was also incorporated.

Figure 1: Locality map showing the three dam sites (1 Karshi, 2 Otukpo, 3 Kashimbila)

The pit for the hydropower station (Figure 3) is the deepest excavation carried out at the site. As luck would have it the footprint of the hydropower station is located entirely within a large intrusive dolerite dyke. At its closest the dolerite–granite gneiss contact was about 1.5 m from the edge of the plant. Initial excavation indicated the dolerite to be fairly massive in appearance, but after blasting, prominent joint sets were evident. Consideration was given to identification of potentially unstable rock blocks and wedges, bearing in mind it was only the temporary stability that was of concern.

With the completion of the outlet works the river was diverted from its original course, finally allowing completion of the last section of the embankment. Foundation cleaning and final preparation before placing of the clay core revealed a zone of soft rock, raising some concerns. Follow-up percussion drilling was carried out to investigate

the continuity of this feature with depth, and this was also correlated with grout takes. Treatment of this weaker zone comprised additional excavation, followed by placement of dental concrete and consolidation grouting, and continuation of the curtain grouting.

OTUKPO DAM

The Otukpo Multipurpose Dam is located near the town of Otukpo on the Okpokwu River in the south-eastern part of Nigeria. The dam comprises a 22 m high, 2 460 m long clay core earthfill embankment with a storage capacity of 133 million m^3. A hydropower station has been proposed, but is not yet under construction.

Figure 2: Kashimbila Dam – foundation preparation and grouting for the final section of the embankment

Figure 3: The completed excavations for the hydropower station, Kashimbila Dam

Figure 4: Otukpo Dam – foundations comprising slaking mud rock, exposed here adjacent to the outlet conduit

The dam site is located in an area underlain by sedimentary strata of the Asata-Nkporo Group which mainly comprise shales and mudstones that proved highly susceptible to slaking (Figure 4). Intrusive dolerite dykes are recognised in the general area, and although none have been recognised at the dam site, one such dyke has provided the rip-rap. In terms of the structural geology, the sedimentary strata are sub-horizontal and dip at shallow angles between 5° and 9°. Regarding the seismic hazard, the entire country of Nigeria is characterised by a low seismic hazard, with peak ground accelerations less than 0.2 g, with a 10% probability of being exceeded in a 50-year period.

A realignment of the embankment during the design optimisation brought the embankment centre-line to within 250 m of the river downstream of the dam, allowing the spillway to be relocated, and significantly reducing the overall spillway length. There were hopes that better founding might be encountered in the area of the realigned spillway, which was associated with a small elevated area with steeper slopes. However, subsequent excavations revealed the presence of grey to dark-grey, highly to moderately weathered, very soft rock shale as foundation. Geotechnical test pitting on the spillway centre-line indicated that no suitable founding horizon occurred at a depth shallower than 6 m, and boreholes drilled did not intersect decent rock within a depth of 25 m. As a solution, the design incorporated a cascade chute spillway that was better suited to a flexible foundation, and also an hydraulic jump stilling basin to ensure that excessive scour does not take place (Wright & Van Wyk 2014). In view of the soft foundation material, it was further considered necessary for the reinforced concrete walls and floor of the spillway to be able to accommodate some relative settlement and shrinkage.

Additional test pitting during construction through the placed core material intersected reworked residual material comprising clayey silt characterised by numerous holes or mini-channels, some of which were associated with noticeable water inflows (Figure 5). Initial test pitting had not been deep enough to intersect this potentially pervious horizon. Geotechnical investigations had included shell and auger (cable percussion) drilling, which

is not conducive to retrieving and subsequent detailed description of relatively undisturbed samples. Further test pitting confirmed this 'channelled' horizon to be underlain by stiff to firm residual silty clay that was free of holes, and the depth of the core trench was subsequently revised to extend to a depth beneath this permeable reworked stratum.

KARSHI DAM

The Karshi Dam is located on a tributary of the Okwa River, just north-east of Karshi which is a small satellite town about 25 km south-east of the capital Abuja in central Nigeria. The dam comprises a 42 m high, 220 m long clay core rockfill embankment with a storage capacity of 6.7 million m^3.

The dam site is underlain by Precambrian-aged banded gneiss from the Migmatite Gneiss / Schist Complex. Satellite imagery during the initial assessment of the dam site confirmed prominent lineaments at the dam site and investigations were initially focused on confirming the nature of these lineaments which are associated with old shear zones.

Electrical resistivity traverses were conducted in order to get an initial indication of the geological profiles at depth, and to identify anomalies associated with the prominent lineaments that would be targeted during follow-up drilling. Traverses were run at the main embankment, parallel as well as perpendicular to the centre-line (covering the alignment of the outlet pipe), as well as at the saddle dam alignment. Drilling was carried out from both river banks, angled in towards the river, in order to further investigate these features at depth. Drilling angled rotary core boreholes is in itself not standard practice in Nigerian site investigation, shell and auger or cable percussion being more the norm, and a specialist drilling contractor was sub-contracted for this task.

Early-stage foundation excavations revealed the highly variable bedrock conditions, in particular alternating massive granite gneiss rock and highly sheared and fractured micaceous schists. A major shear zone of poor rock could be traced for the length of the centre-line. The decision was therefore taken at an early stage to slightly re-align the centre-line by a downwards shift of the right flank only. The benefit of this shift was to realign the centre-line from the band of sheared, micaceous schist onto the adjacent massive granite. This addressed concerns about the potential deformation of the weaker material and the influence on the clay core, and also suggested improved groutability.

The design includes a five-storey intake tower at the toe of the right flank. Fortuitously this position coincided with an area of good quality gneiss bedrock occurring at shallow depths. Expected shallow bedrock conditions in this vicinity were first suggested by the resistivity survey findings, with an exploratory borehole subsequently proving continuity to a minimum depth of 20 m.

The outlet pipe was initially planned as a straight line from the intake tower to the outlet, but poor founding conditions along the alignment resulted in some changes in this regard. In the first

instance the alignment was shifted more into the flank, in a quest for improved founding conditions associated with the effective deeper founding. This alignment was subsequently further optimised by curving the outlet pipe.

Only recently could the attitude of the main discontinuity sets be confirmed. The main set comprises sub-vertical joints that in general strike perpendicular to, or slightly oblique to, the centre-line, which is the most significant orientation in terms of potential seepage from the basin. Limited packer tests had also indicated variably pervious foundations, where the upper 20 m proved permeable in places. In order to address future concerns regarding foundation seepage, a programme of curtain grouting will be carried out. Grout holes will be angled in order to optimally intersect the prominent sub-vertical joints.

Figure 5: Otukpo Dam – close-up view of the water inflows occurring via micro-channels within the reworked residual horizon

Figure 6: Karshi Dam – view of the left flank illustrating spillway excavations in progress

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The contractor, S.C.C. (Nigeria) Limited, who is responsible for the construction of all three dams, as well as the ultimate client, the Federal Ministry of Water Resources of the Federal Republic of Nigeria, are thanked for their permission to publish this article. We further wish to acknowledge the contractor, S.C.C. (Nigeria) Limited, for their dedication and diligence with respect to all geotechnical and geological investigations, considerations and input into the design.

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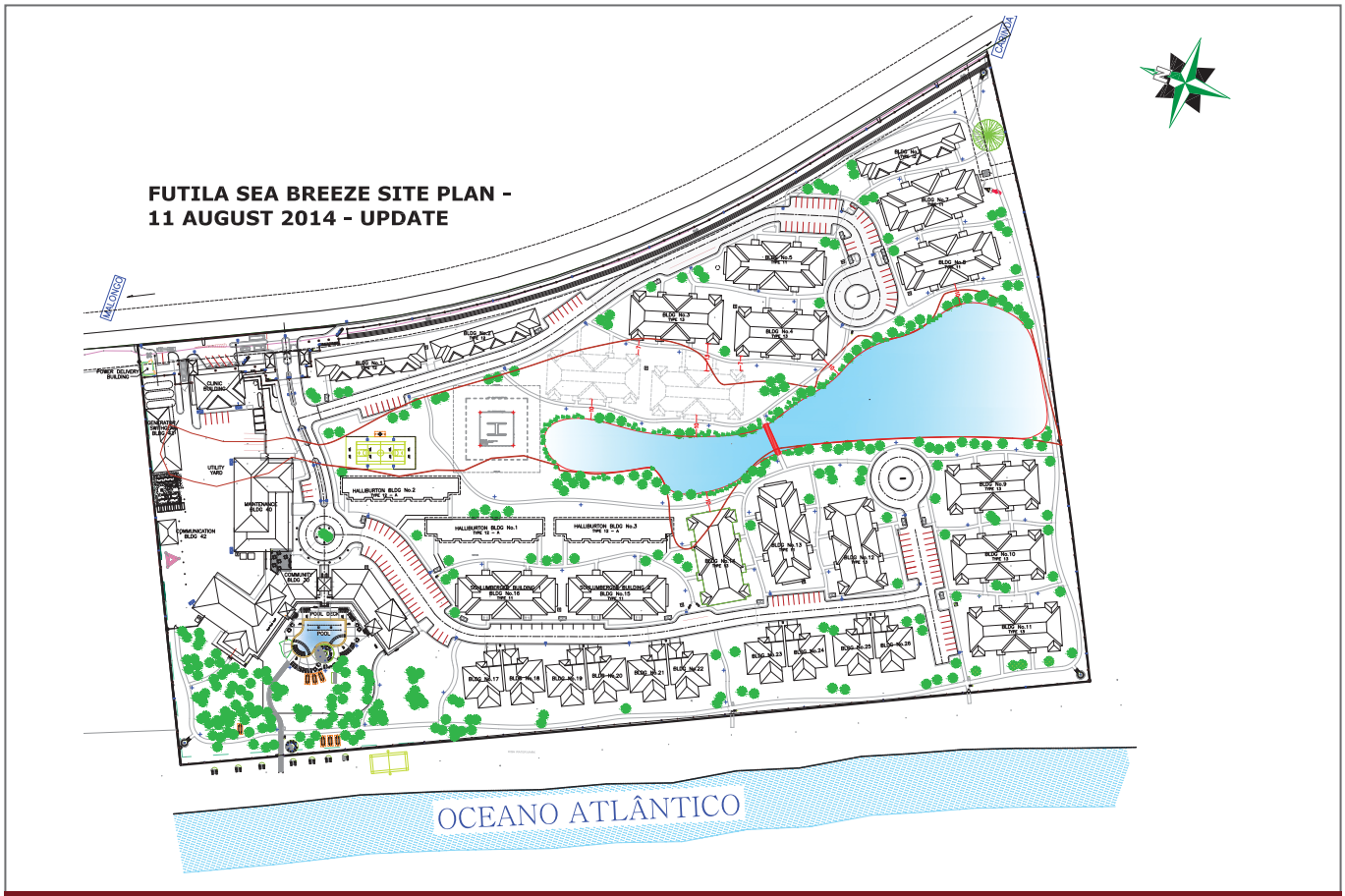

Figure 1: Site plan of the Futila Sea Breeze development in Cabinda, Angola

In situ pile testing

– a tool for design and quality assurance on site

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BACKGROUND

Innovative design approaches are often developed when something goes wrong. An unskilled and uncommitted workforce, derelict piling equipment subject to frequent breakdowns and safety hazards, variable ground conditions and significant language barriers were all concerns which alerted Aurecon's contract management team to potential quality problems at the site of the Futila Sea Breeze development, located close to the ocean in Cabinda, Angola. The development provides a staff residential complex and associated facilities for international oil companies based in Angola, and comprises a clinic, sewage treatment plant, a number of three-storey residential condominiums, helipad, play area and a community building with a swimming pool. A layout of the site is shown in Figure 1, with the wetland marshy area outlined in red.

Aurecon Ground Engineering (GE) was appointed to undertake a strict quality control and review process of the piling installation works done. In addition, an independent detailed pile design was undertaken using the results from four pile load tests performed on the site, and available ground investigation data from tests done by others in 2012.

GROUND CONDITIONS

The ground investigation that had been conducted by a separate contractor in 2012 comprised 20 boreholes using a hollow auger rig with Standard Penetration Testing (SPT) at 1.5 m depth intervals. The site is underlain by recent beach deposits (Cenozoic-Pliocene-Quaternary). These mostly sandy conditions vary between fine clayey sands to coarse sands, typically occurring as loose sand at surface and steadily becoming denser with depth. Medium dense and dense conditions typically occurred approximately 5 m below natural ground level. However, there was a variation in consistency across the boreholes, as is shown in Figure 2 which provides a plot of the SPT distribution with depth. Although it was not possible to accurately map varying zones of consistency to specific locations on site, soft zones at particular elevations could be identified, as shown in Figure 2. These soft zones were present particularly in boreholes located close to a marshy area towards the centre of the site, and were identified as potentially problematic during pile installation with regard to pile borehole collapse and side friction. The groundwater level across the site was approximated to be 0.5 m below ground surface.

FULL-SCALE STATIC LOAD TESTS FOR DESIGN VERIFICATION

Aurecon's design appointment was for the structural design of three buildings located within the marshy wetland area of the site. The marshy wetland area was, however, characterised by standing water and overlain by approximately 2 m of high-plasticity, organic sandy clay. The area was not trafficable when wet, and due to the shallow water table, it would be quite difficult to over-excavate to remove the clayey soil and replace it with inert material. On Aurecon's advice, the three buildings were relocated, and the parking areas, helipad and play area were moved to this difficult section.

As part of the design verification process for the greater site, the contractor conducted full-scale pile load testing

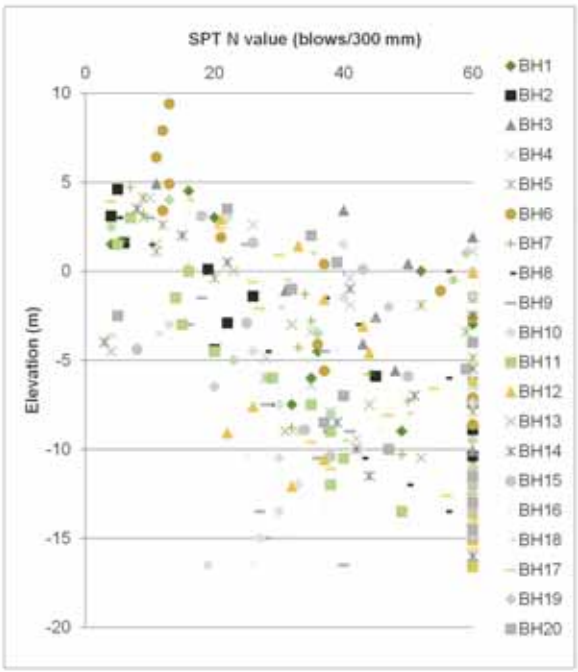


Figure 2: SPT blow counts (N-value) plotted with true elevation (Max SPT N-value is 60)

Figure 3: (a) Piling rig (Jintai GPS-15) and (b) drill bit used for test pile installation

Figure 4: Test pile installation

In addition, using the back-analysis approach, the failure mechanism in the failed test pile could be understood in greater detail. A fit of the achieved settlement indicated that the predicted shaft:end-bearing load ratio was already 83%:17% which, based on the results of the other tests, indicated a situation where shaft side shear capacity was fully mobilised and the pile was resisting any additional load from there on in end-bearing.

whereby the test piles were to be installed as bored piles using the same equipment planned to be used for construction of the 1 200 planned piles. The purpose of the four test piles was to confirm the assumptions of the overall site design with regard to ultimate bearing capacity and allowable pile capacity under working load, with the hypothesis that 4 500 kN of ultimate load capacity would be reached in a combination of shaft and end-bearing load capacity. The working load design was based on a factor of safety (FoS) approach utilising an FoS of 2.6. The pile tests also needed to confirm the installation methodology to ensure repeatability of installation.

The full-scale load testing was undertaken according to ASTM standards (ASTM D1143-81 1994). During pile load testing the strategy had been to load single 600 mm diameter reinforced concrete piles, 16 m long in compression to a maximum load of 4 500 kN. The equipment used to install the piles utilised a derelict Jintai GPS-15 rig, which was totally reliant on the performance and actions of the piling rig operator. The pile installation process as proposed by the contractor consisted of the following steps:

1. A 600 mm diameter borehole was drilled under bentonite slurry, using a temporary steel casing, to a depth of 16 m below pile cut-off level. The bentonite had to be cleaned as it circulated, but during the test pile installation process the contractor could not control the cleaning process effectively yet.
2. Upon reaching the desired depth, the drill was lifted out of the borehole and the reinforcement cage was placed into the hole using a crane and manual labour. The reinforcement cage was fitted with a 600 mm outer diameter steel casing (approximately 0.5 m long) to protect the upper portion of the pile and to provide a firm loading area for the test (Figure 4).
3. After centralising the reinforcement cage, a concrete funnel was assembled. This comprised a steel pipe section that was fitted to a funnel-shaped element where concrete was poured into. The pile and funnel system was lowered into the hole before concrete was poured.
4. Concrete was poured into the funnel, displacing the bentonite in the hole and creating the pile. After removing the temporary casing, the pile was allowed to cure for a minimum of 28 days. The average cube strengths for the four test piles ranged from 42.8 MPa to 45.9 MPa.

The load test frame and kentledge constructed for each test pile comprised 500 tons of steel reinforcing stacked onto a level surface and bound together.

ANALYSIS OF TEST PILE RESULTS

The approach used to analyse the test pile data gave insight into the behaviour of the pile and its utilisation of shaft versus bearing resistance in reaching ultimate load and failure. The analysis of the test pile data consisted of a two-pronged approach:

1. The ultimate vertical compression capacity of the pile was predicted using the method described in Brown *et al* (2007).
2. The method estimates both the shaft capacity (with likely maximum mobilised side shear) and the end-bearing capacity. For shaft capacity, two methods were used, namely the Federal Highway Administration (FHWA) method or the method by Coleman and Arcement (2002). Both were considered during the back-analysis of the test piles.

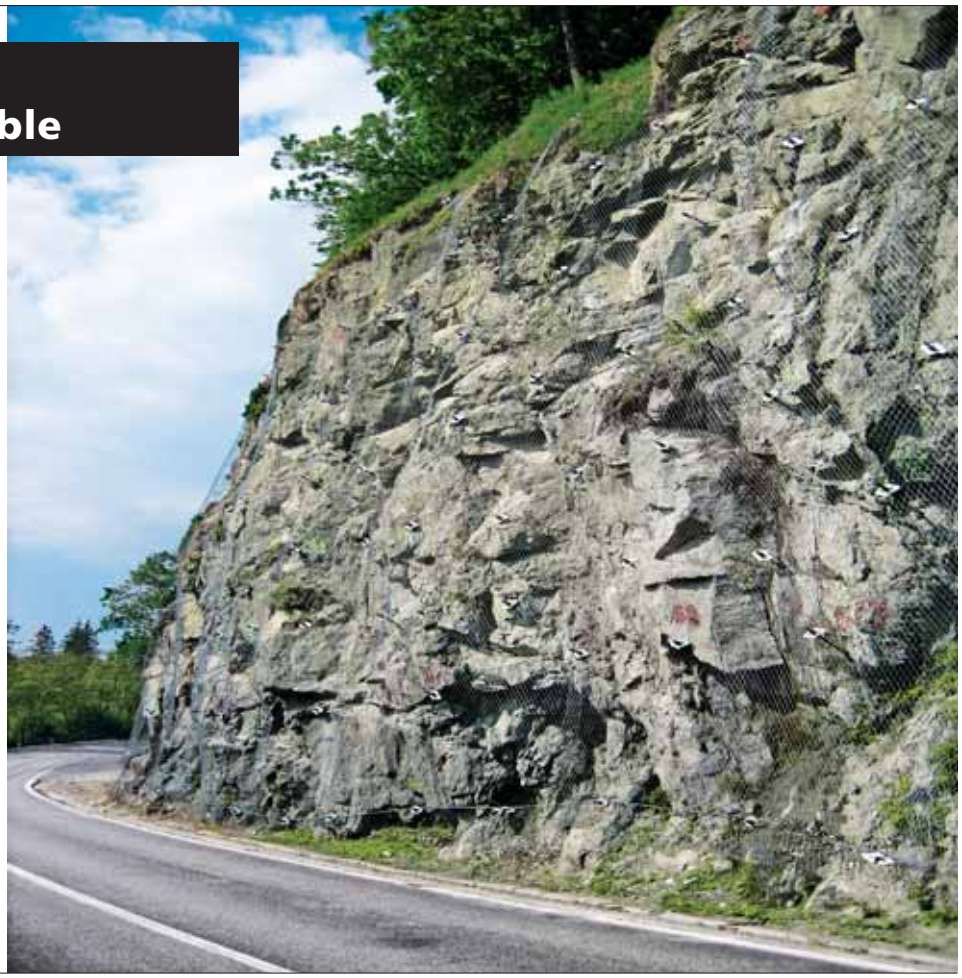
3. The settlement results of the pile were back-calculated using the method presented in Das (1995). This method requires estimation of the soil modulus along the length of the pile, the modulus at the toe of the pile and the mobilised side-shear at a particular point in time. Although the method is based on elasticity, the mobilised condition (taking into account non-linearity of the soil) is provided by the combination of SPT estimation of soil modulus using CIRIA 143 and the iteration of mobilised side shear. Values of mobilised side shear were iterated until the predicted and observed pile settlements were similar. The vindication of the settlement calculation was when the mobilised side shear value approaches the maximum side shear value at ultimate load capacity, calculated using Brown *et al* (2007) in the first stage of the back-analysis. By using the pile settlement measured at each load sequence as a target, the distribution of end-bearing and shaft load was estimated.

DISCUSSION OF FULL-SCALE LOAD TEST RESULTS

Good agreement was obtained between measured values and back-analysed values, to such an extent that the ultimate failure load and development of load between shaft and end-bearing could be defined. One test pile failed and the approach also allowed the back-analysis of this pile in order to assess the likely failure mechanism observed.

The test piles were considered representative of the piling installation process to be used by the contractor. The combination of methods proposed by Brown *et al* (2007) and Das (1995) provided a good estimate of test pile conditions, using the ground information from the closest boreholes to the test piles. As a realistic estimate the method proposed by Brown *et al* (2007), in combination with estimating the maximum side shear using Coleman and Arcement (2002), provided the closest estimation of ultimate load capacity. Ultimate load capacity of the piles varied between 2 252 kN and 2 813 kN. The difference between the expected loads was attributed mostly to the variable conditions at the base of the pile (affecting end-bearing potential) and the mobilisation of side shear. The ultimate load was achieved at settlements of approximately 2.1% to 3.5% of pile diameter (typically 12 mm to 21 mm).

The maximum side shear estimated during loading varied between 59 kPa and 81 kPa, and was believed to be mainly a result of the repeatability of the installation process, and not so much due to the variability in ground conditions. At ultimate load the back-analysed shaft:end-bearing load ratio achieved values ranging between 84%:16% and 78%:22%. The approach allowed the assessment of this ratio for loads less than ultimate. At loads of about 1 126 kN (approximately 50% of ultimate) the load ratio in all three tests was 99%:1%, which meant that the piles carried the load primarily in shaft friction. At 1 687 kN (approximately 75% of ultimate) the test



piles were estimated to carry between 3.5% and 13% of the load in end-bearing.

In addition, using the back-analysis approach, the failure mechanism in the failed test pile could be understood in greater detail. A fit of the achieved settlement indicated that the predicted shaft:end-bearing load ratio was already 83%:17% which, based on the results of the other tests, indicated a situation where shaft side shear capacity was fully mobilised and the pile was resisting any additional load from there on in end-bearing. By the time the next load increment of 940 kN was applied, base failure occurred (with the estimated available maximum end-bearing being 743 kN). The cause of the failure in side shear followed by base failure is believed to be due to the collapse of the side wall and the prolonged opening of the borehole, which may have caused softening of the borehole wall. Upon research into the construction of this pile, it seemed that the contractor had re-drilled the borehole and delayed installation. In addition, it is speculated that during the drilling operation a soil bentonite 'smearing' may have formed that significantly reduced the sidewall friction. These were very significant conclusions, as they pointed to the importance of quality and repeatability of pile installation.

REFINEMENT OF PILE DESIGN

Using the back analysis results of the four test piles, Aurecon GE developed the pile design to be used for the three buildings. Subsequently, this design was used for all the piles to be constructed on the site. The back analysis provided a firm basis for estimating a safe working load for the piles and a factor of safety of 2.0 could be used. A lower-bound design line utilising all the available borehole data on site was used to estimate Young's Modulus. The resultant pile design comprised a 600 mm diameter, 16 m long bored pile installed under bentonite with an ultimate load capacity of 2 252 kN and a safe working load of 1 126 kN. Settlement at working load was estimated to be 7 mm.

The pile installation was paramount to the repeatable and successful installation of a load-bearing pile, as was illustrated effectively by the failure of one test pile. The pile operator had total autonomy and control as to how the pile was constructed. To drive repeatability of installation, Aurecon GE thus advised the contractor as follows:

1. The piling plant had to be replaced by suitable, reliable, new and quality-maintained plant.
2. Only trained staff would operate rigs and should not be changed for the duration of the contract.
3. The process used of circulating the bentonite into a soil sump where soil was allowed to settle out of the bentonite/soil mix was not acceptable. Bentonite shall adhere to the specification such as BS8004 or similar and shall be properly quality-controlled.
4. To reduce the possible effect on the pile–soil interface, the pile installation process had to be streamlined and delay in concreting had to be avoided.
5. Caving of the borehole within the soft zones observed in the ground profile was highly likely. Installation of a temporary casing was advised. In addition, necking of the pile after concreting was identified.

PILE QUALITY ASSURANCE

Aurecon GE continued to assist when the piles that were installed on site were observed to be constructed to a substandard quality. At first, material residue inclusions were observed in the centre of the reinforcement cage in a number of piles, as shown in Figures 5 and 6. When the piles were cut back to reach what appeared to be sound concrete, this at times occurred at depths a lot greater than pile cut-off level (Figure 7). Based on these observations, the extent of the ma-

Figure 5: Material residue inclusions observed within centre of reinforcement cage

Figure 6: Another example of material residue inclusions observed within centre of reinforcement cage

Figure 7: The length of pile that had to be cut back to reach sound concrete

terial residue inclusions throughout the length of the pile, and thus the integrity of the piles, were questionable.

As a quality assurance method, the contractor used dynamic Pile Integrity Testing (PIT) to test the integrity of each installed pile. The method employed a 'hammer' to induce a shear wave into the pile. The PIT results for each pile would then be used to assess the integrity of each pile. Two different sub-contractors had been used to undertake PIT on different buildings within the development. The method of portraying the results differed between the sub-contractors and at times it was difficult to identify a typical 'good pile' from which anomalies on other piles could be identified. Aurecon GE thus requested that the two sub-contractors both undertake separate PIT on the building where the most defective piles were observed, as a calibration test between the two.

INTERPRETATION OF PIT RESULTS

Using the results submitted by both sub-contractors, Aurecon could utilise two separate graphical PIT outputs for one pile to analyse the integrity of the specific pile. The analysis first involved the identification of a typical 'good' pile response. This is shown in Figure 8. The sub-contractors had identified possible defective piles based on their own interpretation and had defined typical defects which could be associated with the observed anomalies; these consisted of necking, segregation, material residue inclusion, cracking in the top of the pile or a broken pile. The 'good' pile response curve was compared to the response curves of other piles to identify potential anomalies within the response. Based on this, Aurecon could assess and justify piles that were identified as being defective, as well as identify additional piles that could potentially be classed as being defective. An example of this assessment is shown in Figures 9 and 10. Figure 9 shows what can be interpreted as possible necking in the top 3 m to 5 m length of the pile. Figure 10 shows an isotropic reflected signal and a multiple reflection in the top 4 m length of the pile, which could be associated with necking, segregation, material residue inclusion or a broken pile.

An indication of the pile length can be obtained from the PIT results. However, this interpretation is unreliable and should not be used as an accurate means of measuring installed pile length. The PIT signal as shown in Figure 8 inflects above the x-axis at the base of the pile length due to the change in density between the concrete and in situ soil. This is used as an indication of pile length. As observed in the PIT results, a number of piles were installed to a reduced length from the design length of 16 m, with pile lengths ranging from 13.7 m to 15.6 m.

As part of the interpretation of the PIT results, anomalies identified along the length of a pile were compared to the borehole log located closest to or in the footprint of the respective building. It was observed that anomalies identified as potential necking of the pile occurred at depths where soft zones within the ground profile occurred or at interfaces between soft zones and stiffer zones. Anomalies interpreted as 'a reduction in pile diameter after a thickening in first pile metres' were interpreted to be associated with casing removal.

In response to Aurecon's assessment of the integrity of each individual pile, one sub-contractor undertook another round of PIT on the identified potentially defective piles. The sub-contractor then gave a more in-depth interpretation of the PIT results. This in particular was in response to Aurecon's concern with regard to the extent of necking perceived in the piles. Aurecon was uncertain about the level of detail in terms of the extent of necking that could be interpreted from the PIT results. However, notwithstanding the sub-contractor's clarifications, Aurecon assumed that the reduction in diameter still provided sufficient cover to the reinforcement. This was based on the anomalies occurring below the water table, hence there would be low oxygen levels present and the potential for corrosion of the reinforcement was considered low. The small moments exerted on the respective piles also supported the interpretation that some of the piles associated with 'necking' anomalies were satisfactory.

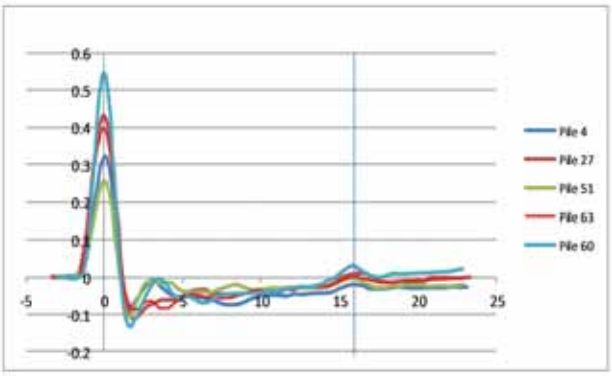


Figure 8: A typical 'good pile' response utilising PIT response curves from piles

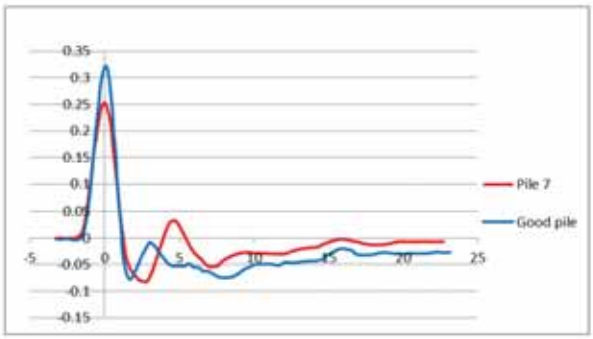


Figure 9: Defective pile showing possible necking in top 3 to 5 m of the pile

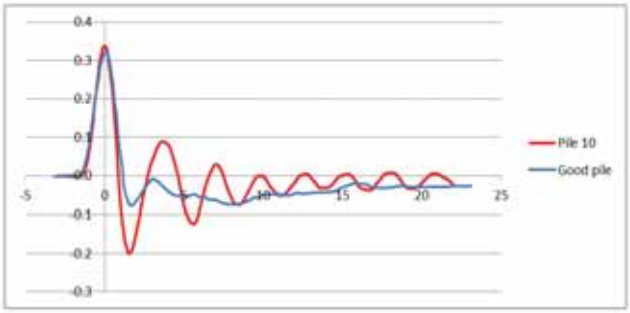


Figure 10: Isotropic reflected signal and multiple reflection in top 4 m length of pile could be associated with necking, segregation, material residue inclusions or a broken pile

PROBLEMS ENCOUNTERED WITH THE PILES

In response to Aurecon's initial assessment of pile integrity, core drilling was undertaken on a number of piles to investigate identified anomalies associated with concrete contamination and cracking. This, however, would not be able to investigate anomalies associated with necking and pile diameter variations.

Concrete contamination observed in the piles discussed above (Figures 5, 6 and 7) could be due to contaminated air-lifted bentonite being used for the construction of the piles. This would affect the displacement properties of the bentonite. Bentonite should be cleaned prior to reuse in the support of the auger drilling for construction of the piles. At the time when the defective pile tops were discovered, two piling contractors were operating on site and fingers were being pointed implying that one contractor had supplied dirty bentonite to the other.

The core results were assessed along with the borehole log and PIT results. An example of the assessment is shown in Figure 11 where cracks within the pile, which were identified as the possible causes of anomalies observed in the PIT signal, are shown on the core log. The core logs were used to understand the anomalies identified in PIT signals. Concrete discolouration and contamination were observed in other core logs.

The piles installed to reduced lengths were assessed individually. The pile capacity for the shorter pile was determined based on the design approach discussed earlier, however using the borehole located within the footprint of the respective building, as opposed to using a design based on all the boreholes throughout the site. The design was based on shaft capacity alone (ignoring the contribution of base capacity). This pile capacity was compared to the load imposed on each individual pile. In most instances, the shorter piles were satisfactory.

Figure 11: Core drilling examples showing possible causes of anomalies

REMEDIATION MEASURES FOR DEFECTIVE PILES

For defective piles, foundation intervention was required. Aurecon worked with the on site team to develop solutions for each individual pile scenario. Since the installation of any additional pile could attract an off-balance load, this had to be kept in mind when proposing a solution. Additional piles were installed, as well as ground beams in some instances, to rectify defective pile scenarios. Following the poor quality of pile installation observed for one building, piles installed subsequently for other buildings were tested using PIT and found to be satisfactory and to the correct length. This showed that the contractor was working to an improved quality level and following proposed installation procedures.

CONCLUSION

This article details an approach whereby pile load test results are used to offer a deeper understanding of pile behaviour in situ under working and ultimate loading conditions, and how these results can be used to give a more cost-effective, safe design. By using the presented back-analysis approach, the following was achieved for each test pile:

- The estimated ultimate load capacity.
- The estimated working load capacity for design. For the design a factor of safety of 2.0 was used. This lower factor of safety was based on the fact that four test piles would provide information required for design.
- The estimated shaft:end-bearing carrying ratio and therefore the mechanism of load carrying for design.
- Focus could be put to issues of installation that may adversely affect the design of the piles.

The pile load tests also highlight the importance of using reliable, quality-maintained plant and skilled plant operators to ensure repeatable pile installation. Pile Integrity Tests (PITs) are a useful means to control quality of piling installation works on site.

ACKNOWLEDGEMENTS

The authors would like to thank Mr Henry Burgess and Mr Vitor Martins of Aurecon Angola for providing insight into operations on site, as well as constant communication from site. The authors also wish to thank Mr Carlos Pereira de Almeida of Servicab, South Africa, for his support of the article and for allowing the sharing of data about the Futila Sea Breeze development.

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The list of references is available from the authors on request. ◻

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Tensar TW1 system used on major roads contract in Ballito

The Tensar TW1 System was developed as an alternative to traditional retaining wall options and has been used extensively in Europe and elsewhere internationally. The system has very recently been introduced into the South African market through Kaytech Engineered Fabrics. This article discusses the use of the system on a major road upgrade contract in South Africa, Ballito Drive, which was one of the first and largest applications of the system locally to date. Project engineers, SMEC South Africa, were closely involved in the detailed design of the system and realised a number of cost benefits on the project by using the Tensar TW1 system.

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INTRODUCTION

Ballito Drive is situated about 40 km north-east of Durban. The scope of works entailed the widening of a two-lane, single carriageway to a three-lane, dual carriageway. Due to the undulating topography of the site, earth retaining structures had to be built to bring the extra lanes to level. The lane widening had to be constructed within the road reserve to eliminate encroachment into existing developments. In order to reach this objective two near-vertical mechanically stabilised earth (MSE) walls of 11 m and 5 m, covering a total length of over 400 m and 2 000 m^2, were proposed.

MSE walls, in broad terms, consist of fill material with horizontal layers of reinforcing elements which may take the form of sheets, grids, strips or meshes. The reinforcing elements, which are either metallic or polymeric, are capable of sustaining tensile loads and the effects of deformation or soil strains developed in the fill, part of which is transferred to the clad face through some form of positive connection.

A number of proprietary MSE products are currently in the market and, following a competitive tender process, the winning tender for construction of the project (by Afriscan Construction), included the use of the Tensar TW1 system. The project consulting engineers were satisfied that the system would meet the technical requirements, and were subsequently closely involved in the de-

Figure 1: Installation of crash barrier on 11 m high Tensar TW1 wall at Ballito Drive

tailed design of the system. The intricate design needed to ensure that the system complied with internal and external stability and project technical requirements. A benefit of the system was that lower quality fill, which was more readily available and less expensive, could be utilised, as the grids provide greater coverage and soil adherence than other systems on the market, and the product is also not prone to degradation or chemical attack by natural soils.

A further benefit to the system was that adjacent landowners were satisfied that the appearance of the split-face blocks would provide a pleasing aesthetic appearance which would complement the local architecture.

TENSAR TW1 SYSTEM

The system, developed by Tensar International in the United Kingdom, comprises the specially designed TW1 block, combined with high-density polyethelene (HDPE) grid mats – known as Tensar uniaxial geogrids – that are attached by a special connector into the blocks and extend horizontally to secure and reinforce the fill, thereby turning the whole structure into a monolithic mass.

The positive connection to the cladding or split-block face is an important attribute of the system and allows it to be used on near-vertical walls exceeding 7 m, which is the present maximum height attainable with dry stacked block retaining systems available locally.

Internationally maximum-tiered wall heights of 60 m have been achieved using the TW1 system, with a height of 22 m in Fujairah, UAE, being the maximum for a single-tiered wall.

GEOTECHNICAL INVESTIGATION AND DESIGN

As the Ballito project was one of the first of these walls in the country, the design of the wall was a close collaborative effort between Kaytech, Tensar and SMEC South Africa. SMEC undertook the final design checks to ensure overall stability of the system, and compliance with project specifications and local codes. These included integration of the system with the new roadway and the New Jersey barriers along the top of the wall, as well as taking cognisance of the overall geotechnical conditions.

The geotechnical investigation of the site revealed the site to be underlain by mudrock of the Karoo Supergroup, overlain by Tertiary to Recent sediments. At the location of the MSE walls the site was underlain by thick coastal dune Berea deposits, and bedrock was present at depths exceeding 30 m.

The design of the MSE walls was based on South African National Standard SANS 207: 2006: *The design and construction of reinforced soils and fills*, which provides guidance applicable to the design of reinforced walls.

A reinforced soil structure must be checked for both external and internal stability. External stability considers sliding, bearing/tilt and overturning of the MSE block. Internal stability considers not only the essential checks of failure against pull-out of the geogrid and failure against rupture, but also a number of ancillary checks, including compressive failure of the blocks, block rotation and bulging and connection failures.

The type of geosynthetic reinforcement selected must take into account the soil properties of the reinforced, retained and foundation materials. These soil properties contribute to determining the tensile strength, stiffness requirements and spacing of the geogrid. The geogrid will only be able to withstand the tensile forces once attached to the facing and once normal stress is applied to its length. The ultimate tensile strength of the geogrid is factored, giving rise to the calculated long-term design (LTD) strength which is provided and discussed in detail in the manufacturer's design guidelines.

Critical sections were analysed for internal and external stability. These were modelled and checked with the proprietary design software packages from Tensar International. The overall stability of the wall was checked using geotechnical finite element software, Phase 2[8].

Modelling material properties for the membrane and interface elements in a finite element model can be problematic, especially considering that the geogrid's pull-out resistance is derived from friction generated on the soil-reinforcement boundary,

Figure 2: Tensar Polyethylene RE560 uniaxial geogrid

which in this case is not continuous and has perforations. This is further complicated by the fact that the in-soil stiffness of a geogrid is stiffer compared to the in-air stiffness under which the geogrid is tested. An approximation thus needed to be made. The reinforcing layers were simulated using liner elements which had capacity to resist tensile forces, whilst the interface between the soil and liner was modelled with zero thickness interface elements, as per the recommendations of Potts (2005).

DESIGN OPTIMISATION

A key consideration in the design was to optimise the use of lower-quality fill material, whilst simultaneously minimising the amount of lateral support required in cutting back and benching into the existing roadway, i.e. the back excavation slope. Limited space was available for the 11 m high wall, which restricted the length of the strips to 7 m. At the same time it would be beneficial to the project if Berea sands could be utilised. However, by using the lower quality fill, strip lengths would need to be increased, which implied either increased cut or the use of a near-vertical back excavation slope requiring the use of shotcrete and ground anchors or nails.

After a number of iterations, the final design for the 11 m high wall comprised the use of 7 m long strips, a granular (COLTO G6) backfill for part of the height and a 1 m thick granular soil-raft foundation. No lateral support was thus required, and conventional benching into the existing fill was utilised. For the upper 3 m of the 11 m wall, and for the 5 m high wall, Berea sand was used throughout.

CONSTRUCTION

Some of the further benefits of the Tensar TW1 system are that it is labour-intensive, and also eliminates the need for cranes and other heavy lifting equipment. The TW1 block is also manufactured locally by Remacon, a Tensar licensee for the manufacturing of that specific block.

In utilising a new system a number of challenges were experienced during construction. This included the setting of the base block, which is key to achieving the final face inclination of 86°, compaction criteria, stormwater control, and the use of labour not experienced in building these walls. However, these were quickly resolved through close collaboration between the contractor, consultant and supplier. The Kaytech and Tensar teams were able to provide technical assistance to the contractor and the consultant's supervising team with regard to installation, testing standards and quality control and assurance.

CONCLUSION

This project has showcased the level of knowledge and experience required to design and construct a Tensar TW1 mechanically stabilised earth wall, and this has been a major achievement for Kaytech, as this is the first wall of such a size to be constructed in South Africa. The TW1 system provides a number of benefits over other block and other MSE systems, including the effective connection between block and geogrid, a near-vertical face inclination, locally manufactured blocks, aesthetic appeal and labour-intensive construction, eliminating the use of heavy lifting equipment. ◻

PROJECT STATISTICS

Client: KwaDukuza Municipality
Contractor: Afriscan Construction
Supplier: Kaytech Engineered Fabrics and Tensar International
Design and Supervising Engineer: SMEC South Africa
Construction Value: R45 million (walls and fill R8.5 million)

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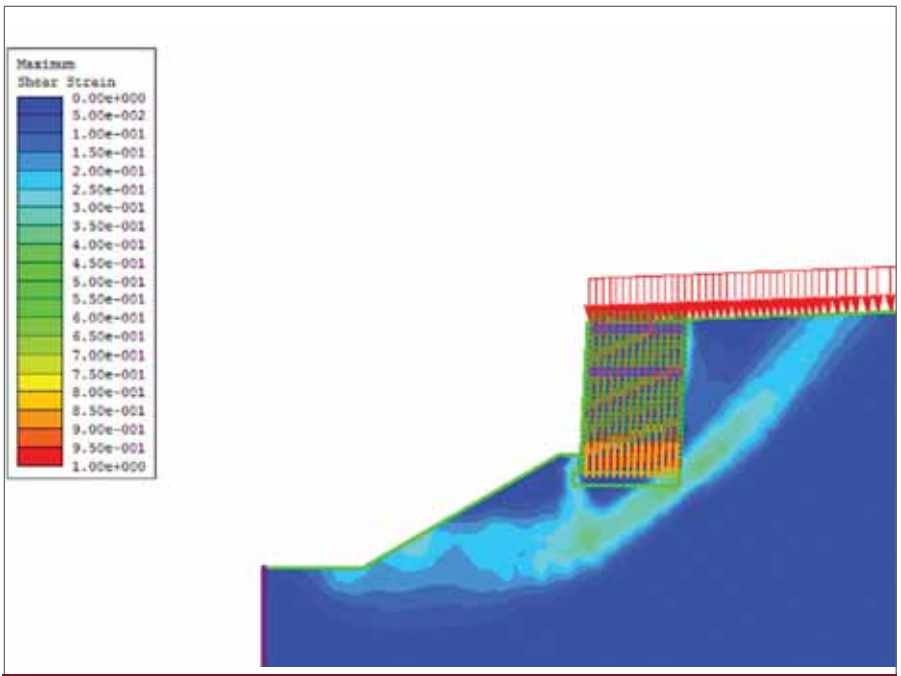

Figure 3: FEM model showing development of strains in soil behind and within the MSE

Figure 4: Locally manufactured Tensar TW1 blocks on a 5 m high wall section

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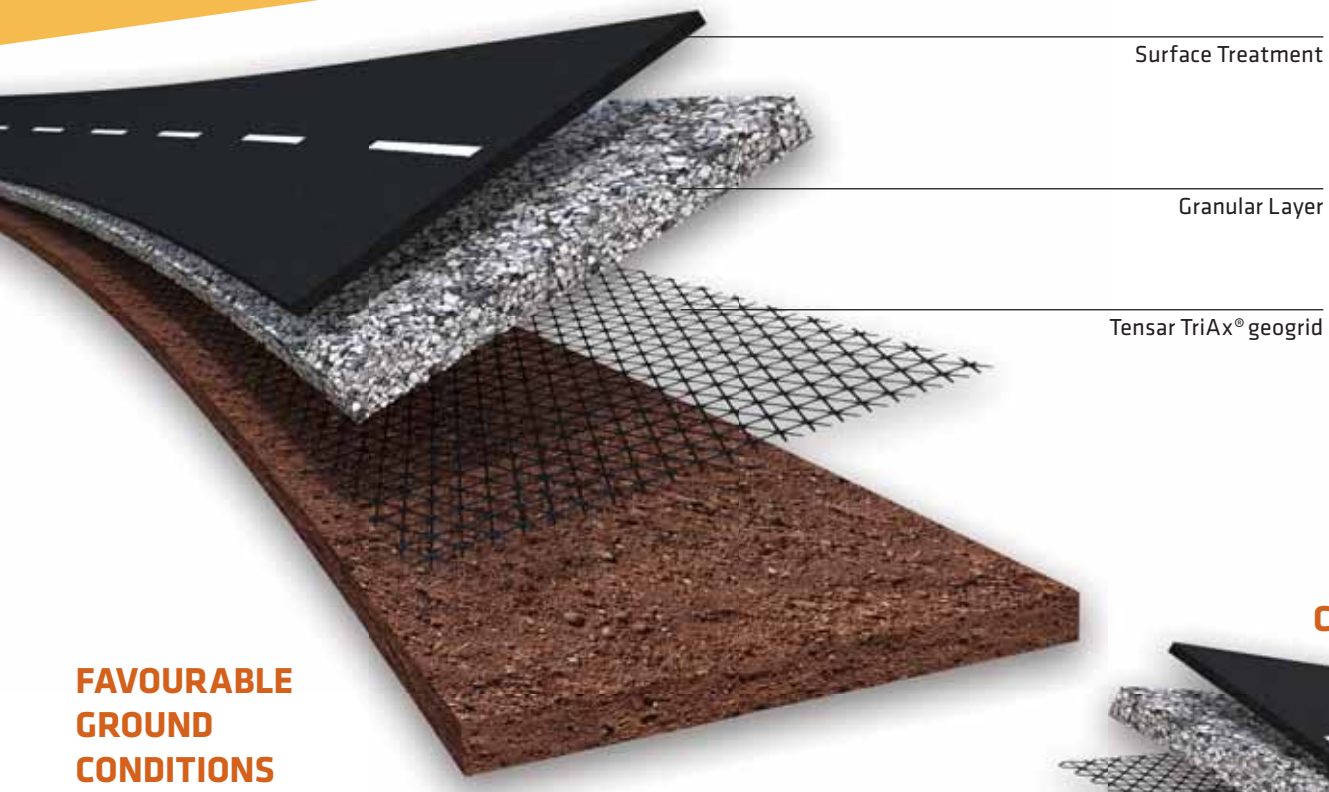

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96 Mill Point Road development lateral support

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INTRODUCTION

The proposed development at 96 Mill Point Road, South Perth, Western Australia, comprises five basement levels and 22 above-ground levels. Groundwater was encountered some 2 m below natural ground level (NGL). This combination of features presented an interesting case study. ARQ Consulting Engineers was approached to conduct a preliminary and detailed design for the lateral support and foundation of the development. This article discusses the findings of three design options which were evaluated, analysed and compared.

GEOTECHNICAL INFORMATION

The soil profile underlying the site comprised alluvial sand, Guildford clay and residual Tamala limestone. Tamala limestone bedrock was encountered at 24 m to 26 m below NGL.

Material properties relevant to the design are included in Table 1. Where possible these material properties (such as permeability, unit weight and Young's Modulus) were obtained from a geotechnical investigation report conducted by CMW Geosciences (Pty) Ltd.

DESIGN COMPONENTS

Excavation depth

The building would comprise five basement levels, founded on a piled raft of estimated 2 m thickness. The designs incorporated excavation to a depth of (5 x 3) = 15 m.

Table 1: Material properties used in the analyses

Material name	Unit weight (kg/m³)	Young's Modulus (MPa)	Tensile strength (kPa)	Friction angle (°)	Cohesion (kPa)	Permeability (m/s)
Alluvial sand	1 800	60	0	34	0	1×10^{-4}
Guildford sandy clay	1 900	40	0	20	20	5×10^{-5}
Tamala limestone (medium dense/dense)	1 900	80	0	36	3	1×10^{-4}
Tamala limestone (very dense)	1 900	100	0	40	20	1×10^{-4}
Dewatering well (clean sand)	1 800	40	0	33	0	1×10^{-3}
Pile material	2 400	21 000	1 500	45	6 000	1×10^{-9}
Jet-grouted raft	2 200	18 000	1 500	45	6 000	1×10^{-6}

Table 2: Anchor properties used in the analyses

Anchor	Equivalent bolt φ (mm)	Young's Modulus (GPa)	Tensile capacity (kN)	Residual tensile capacity (kN)	Pre-tensioning force (kN)
2 strand anchor	21.5	20	528	352	414
3 strand anchor	26.3	20	792	528	620
4 strand anchor	30.5	20	1 056	704	825

Secant pile wall

Secant pile walls were deemed necessary to provide lateral support to the excavation, while impeding ground water flow into the excavation. The secant pile walls were to comprise alternating 'soft' and 'hard' piles, where the 'hard' piles (30 MPa reinforced concrete piles) were to be drilled through the soft piles (5 MPa unreinforced piles).

To ensure sufficient stiffness, 750 mm diameter piles, installed to a depth of 27 m, were selected, thereby ensuring that they were socketed into the limestone formation which underlies the site. The piles were installed at 600 mm centres, therefore with a 150 mm overlap, ensuring water-tightness.

Anchor

Pre-tensioned low-relaxation ground anchors were used in the design. Anchoring was to take place through the 'soft' piles at 1.2 m centres. The fixed and free length of the anchors were arranged so that:

a) Compliance with SAICE's Lateral Support Code of Practice (1989) was adhered to;
b) Fixed lengths were placed so as to intercept regions of high shear strains, which may manifest as regions of possible slip failures; and
c) Fixed lengths were sufficiently long to withstand the envisaged pull-out forces.

The anchors considered in the design comprised 2, 3 and 4 strands, and 15.2 mm diameter low-relaxation strand with an ultimate tensile strength of 265 kN.

The ground anchor properties used in the models are provided in Table 2.

Dewatering

All designs were developed to ensure that:

- The phreatic surface would be some way from the base of the excavation at all times. This would enable normal construction operations to continue unhindered within the excavation.
- Due to the cut-off effect of the long piles into the underlying limestone formation, the drawdown of water level outside the site would be practically eliminated. This would ensure that minimal settlements were induced outside the site with practically zero effect on services, roads and adjacent structures.

Design options

Three design options were considered and analysed. All options involved a secant pile wall constructed to a depth of 27 m on the perimeter of the excavation, comprising alternating soft and hard piles. Four rows of ground anchors were installed with varying numbers of strands and pre-tensioning forces. The variations in each of the design options are provided below:

- ***Option 1*:** A multi-point dewatering system comprising 750 mm dewatering wells installed in a 15 m grid to a depth of 21 m below natural ground level.
- ***Option 2:*** A 2 m thick jet-grouted raft (JGR) installed from -17 m to -19 m below natural ground level. To prevent uplift of the raft under the action of the approximately 15 m of water pressure, 750 mm diameter piles were installed through this raft in a 5 x 5 m grid to create a piled raft foundation. These piles were installed from ground level, but only concreted from -15 m to -27 m. It had to be ensured that the volume of concrete used was only sufficient to fill the auger holes to a depth of 15 m below NGL. In this way it was not required to break the piles down during the excavation process, as the tops of the piles would be encountered at the envisaged excavation base.
- ***Option 3:*** A 3 m thick jet-grouted raft installed from -24 m to -27 m below natural ground level. 750 mm diameter piles were to be installed from -15 m to -24 m below natural ground level, such that the base of the piles would rest on top of the raft. The installation of the piles was to take place prior to excavation.

FINITE ELEMENT EVALUATION

The above design options were assessed via a final finite element evaluation. The finite element analysis was conducted using Rocscience's Phase^2 finite element software. The staged construction of a typical cross section of the excavation was utilised to conduct the analysis. Initially, a seepage analysis was conducted on the model, followed by a stability analysis.

The models

A typical cross section of the excavation was modelled and analysed. Each model was compiled using staged construction. Figures 1, 2 and 3 show the plain strain models of each design option, as described above, during the final construction stage.

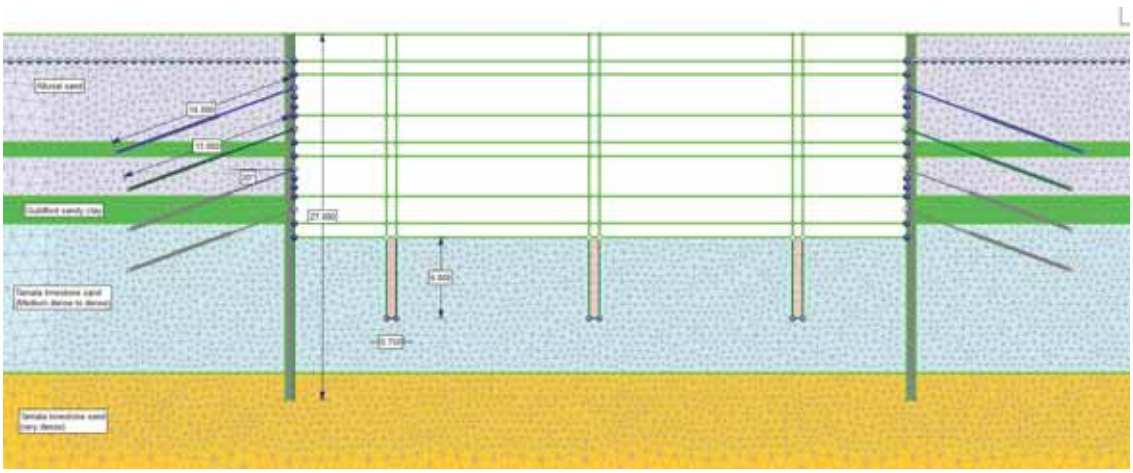

Figure 1: Dimensioned model of Design Option 1

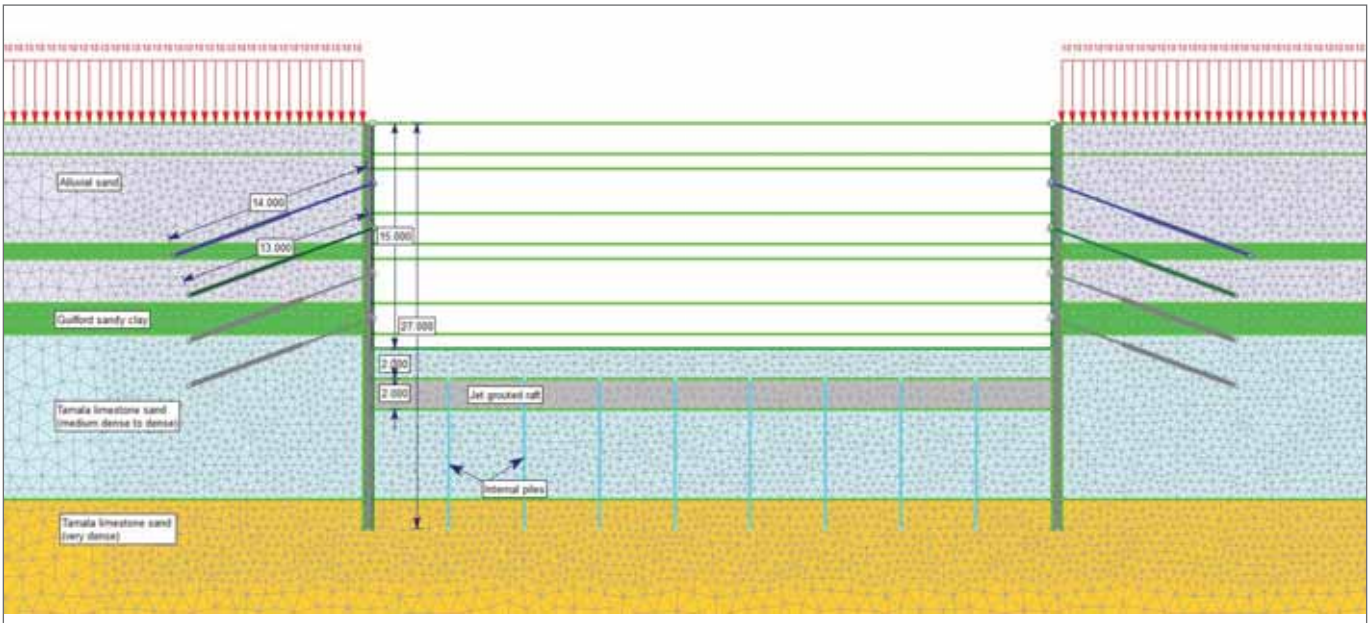

Figure 2: Dimensioned model of Design Option 2

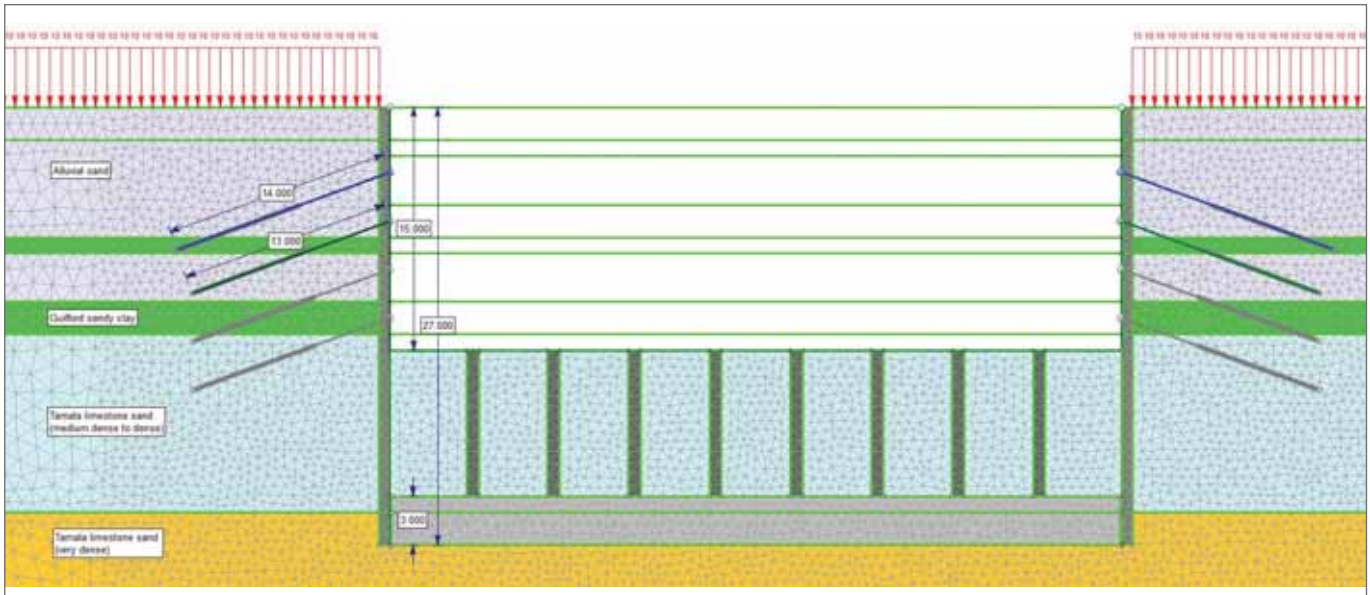

Figure 3: Dimensioned model of Design Option 3

Load cases

Two load cases were applied to each model. It should be noted that Load Case 1 constituted the Serviceability Load State (SLS) of the model, while Load Case 2 constituted the Ultimate Load State (ULS). For Load Case 1 (SLS) a surcharge of 10 kPa was applied to the ground surface surrounding the excavation during the final construction stage. For Load Case 2 (ULS) a surcharge of 10 kPa was applied to the ground surface surrounding the excavation during the final construction stage, and seismic coefficient of 0.1 was applied to the model during the final construction stage to model a seismic event characteristic of the area.

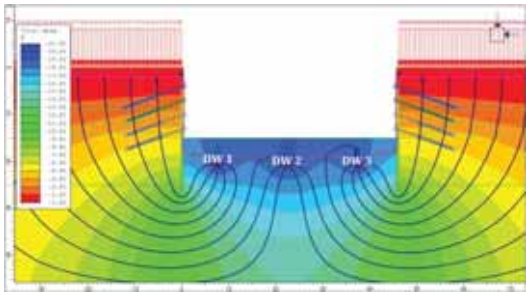

Figure 4: Position of phreatic surface during the final construction stage of the model

Factors of safety

According to the *Lateral Support in Surface Excavations Code of Practice* (SAICE 1989), the generally accepted factor of safety (FoS) against circular slip failure or wedge type failure is at least 1.5 for permanent structures and 1.25 for temporary structures.

It is ARQ policy that the stability of a structure is assessed for both serviceability and ultimate load cases. For a temporary structure under ultimate loading conditions, an FoS of at least 1.01 must be calculated, such that the structure is just able to withstand the ultimate load case.

In addition to the stresses, strains and displacements within a model, Phase^2 is able to determine the Strength Reduction Factor (SRF) of a model. This SRF is comparable to a conventional FoS, and will be used as such for the following analyses.

Table 3: Flow calculated from each of the drainage wells

Construction stage	Flow (l/s)					
	Excavation bottom	Left pile wall face	Drainage Well 1	Drainage Well 2	Drainage Well 3	Right pile wall face
1	--	--	0.849	0.636	0.819	--
8	2.00×10^{-3}	9.51×10^{-5}	0.457	0.282	0.335	9.44×10^{-5}

RESULTS OF ANALYSES

The following subsections discuss the results of both the seepage and stability analyses completed.

Design Option 1

Seepage analysis

The seepage analyses for all design options were conducted assuming that, prior to the commencement of construction, the phreatic surface was encountered 2 m below ground level. Figure 4 provides the results of the seepage analysis during the final stage of construction. The total head contour plot is displayed in the image, while the purple line indicates the position of the phreatic surface. The dark blue flow lines indicate the directions of flow within the model.

Table 3 provides the flow required from each of the drainage wells in order to ensure that there would be no build-up of water in the excavation. The dewatering well positions are indicated in Figure 4. These flows are provided for both Construction Stage 8, the final stage, and Construction Stage 1, the stage in which the maximum flows were calculated.

Estimated flows from the entire excavation were 5.2 l/s and 2.5 l/s for Construction Stages 1 and 8 respectively.

Stability analysis

Table 4 provides a summary of the stability analysis results.

Although the factors of safety produced exceed the required minimums, the main concern for this option was that examination of outcrops of Tamala limestone which occur over many areas of Perth, indicated that the permeability of this formation may in fact be many orders of magnitude greater than that assumed in the analysis.

If this were the case, the required extraction rate may rise alarmingly to values as high as 200 l/s. As the maximum flow into stormwater drainage conduits permitted by the Perth Authorities is 8 l/s, the high risk attached to this solution is evident. This is the reason for this solution not gaining preference in the evaluation.

Table 4: Summary of stability analysis results for both load cases for Design Option 1

Parameter		Load case	
		Serviceability load state	Ultimate load state
Maximum anchor forces (kN)	2 strand anchor	416	427
	3 strand anchor	620	633
	4 strand anchor	825	825
Maximum moment within pile wall (kN.m)		269	276
Maximum horizontal displacement of pile wall (mm)		23	55
Maximum vertical displacement at base of excavation (mm)		82	82
SRF		1.57	1.30

Table 5: Flow calculated from each of the drainage wells

Construction stage	Flow (l/s)			
	Left pile wall surface	Excavation base	Right pile wall surface	Total for entire excavation
8	1.262×10^{-4}	0.237	1.247×10^{-4}	9.5

Table 6: Summary of stability analysis results for both load cases for Design Option 2

Parameter		Load case	
		Serviceability load state	Ultimate load state
Maximum anchor forces (kN)	2 strand anchor	410	420
	3 strand anchor	603	615
	4 strand anchor	825	825
Maximum moment within pile wall (kN.m)		613	721
Maximum horizontal displacement of pile wall (mm)		13	42
Maximum vertical displacement at base of excavation (mm)		90	94
Maximum vertical displacement of jet-grouted raft (mm)		85	87
SRF		1.60	1.20

Table 7: Flow calculated from each of the drainage wells

Construction stage	Flow (l/s)			
	Drainage Well 1	Drainage Well 2	Drainage Well 3	Total for entire excavation
8	1.278×10^{-4}	0.151	1.257×10^{-4}	6.1

Design Option 2

Seepage analysis

Table 5 provides the calculated flows occurring from the base of the excavation as well as through the secant pile walls.

Stability analysis

Table 6 summarises the results of the stability analysis.

Design Option 3

Seepage analysis

Table 7 provides the flow calculated from the base of the excavation, as well as both of the pile walls, during the final construction stage.

Stability analysis

Table 8 provides a summary of the stability analysis results.

Load Case 1

Figure 5 shows the contour plot of the maximum shear strains within the model during the final stage of construction. It can be seen that a maximum shear strain of some 1.5% was calculated at the outer edges of the excavation base.

Load Case 2

A contour plot of the maximum shear strains within the final construction stage of the model is provided in Figure 6. A maximum strain value of 3% was calculated. A clearly asymmetrical strain distribution was initiated by the application of seismic activity.

An SRF of 1.06 was calculated for the model, as shown in the contour plot in Figure 7. This is slightly greater than the required minimum and indicates that the structure is able to withstand the applied loading.

Table 8: Summary of stability analysis results for both load cases for Design Option 3

Parameter		Load case	
		Serviceability load state	Ultimate load state
Maximum anchor forces (kN)	2 strand anchor	412	427
	3 strand anchor	608	620
	4 strand anchor	825	825
Maximum moment within pile wall (kN.m)		721	1 016
Maximum horizontal displacement of pile wall (mm)		21	80
Maximum vertical displacement at the base of the excavation (mm)		70	80
Maximum vertical displacement of the jet-grouted raft (mm)		35	36
SRF		1.25	1.06

CONCLUSION

Design Option 1 provides adequate margins of safety. However, groundwater flow into the excavation is not inhibited in any way, resulting in flows which require continual pumping. The risk in this option is that the flow may exceed that permitted by the authorities.

Table 9: Required number of reinforcing bars for each design option

Design option	Reinforcing bars per hard pile
1	5 Y32s
2	14 Y32s
3	19 Y32s

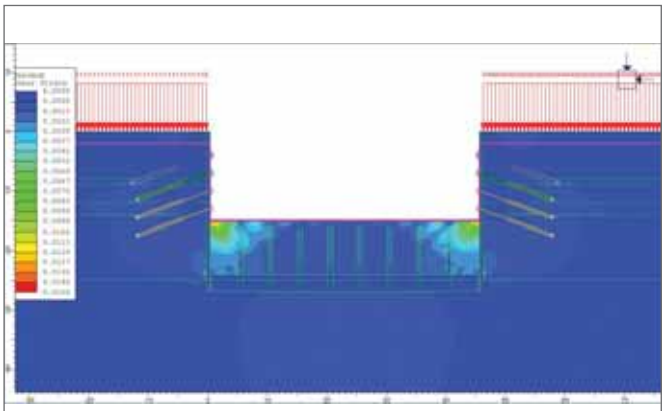

Figure 5: Contour plot of maximum shear strain within model for Analysis 1

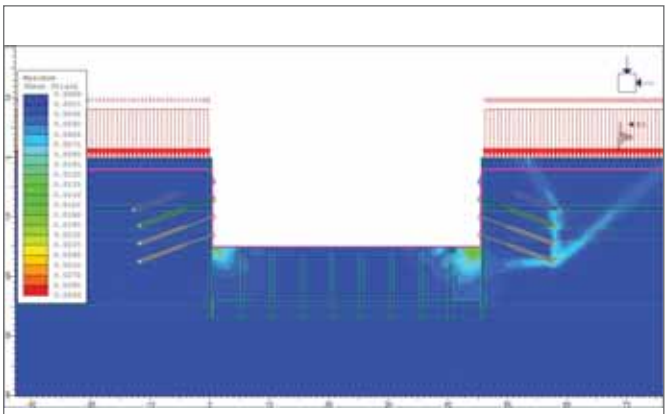

Figure 6: Contour plot of maximum shear strains during the final stage of construction for the model under ultimate load conditions

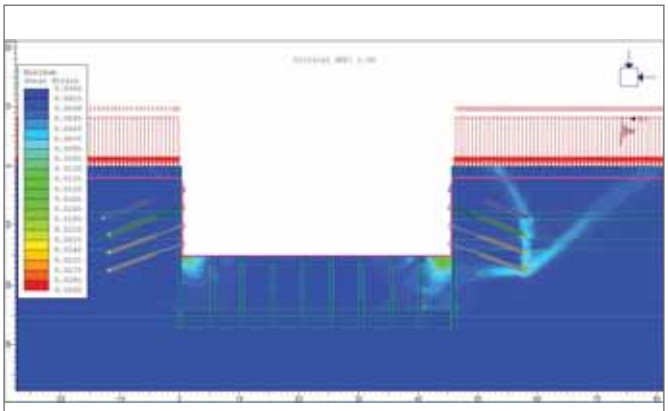

Figure 7: SRF output for the model under ultimate load conditions

Design Option 2 also provides the required factors of safety, but large uplift displacements were calculated at the base of the excavation. These in turn translate into unacceptable tensile stresses in the JGR.

Design Option 3 exhibited the required factors of safety while maintaining acceptable strain/displacement values for both the serviceability and ultimate load conditions. This is therefore ARQ's preferred option. Large moment values were calculated in the pile walls adjacent to the jet-grouted raft. For the piles to sustain these moments, some 19 Y32 reinforcing bars must be incorporated in the hard piles. Table 9 provides the number of reinforcing bars required to withstand the moments in each design option.

Of the three design options considered, Design Option 3 was found to be the preferred option as it provided a suitable, effective solution.

REFERENCES

Proposed Building Development, 96 Mill Point Road, South Perth, WA Geotechnical Investigation Report. 18 July 2014. CMW Geosciences (Pty) Ltd.

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Road construction on expansive or low strength subgrades

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INTRODUCTION

Road construction can be quite daunting at times. This is exacerbated when constructing on an expansive or low strength in situ material or subgrade. This note briefly explores ways to deal with such challenges.

PROJECT DESCRIPTION

A 20 km long access road is proposed for a mining development in northern KwaZulu-Natal. The geology of the area consists of basalt, dolerite, shale and sandstone. Due to chemical weathering (Weinert N-value of 5), some of these materials will cause challenges in the subgrade for road layerworks as they will typically weather to clay.

On investigation it was found that the in situ material in places exhibit plasticity indexes exceeding 25, and some sections are underlain by clay layers in excess of 3 m of moderate to high activity. Much variation was observed.

IMPACT ROLLING

In order to counteract this low quality and variation, impact rolling is envisaged as a good solution. The various strata would be handled as follows:

- Test sections of 120 m long and 5 m wide, which reflect the different substrata conditions along the proposed route, are envisaged, especially at those sections where weaker material had been identified through test pit investigation, and later CBR and indicator laboratory tests.
- Certain fixed measuring points would be demarcated along the test section and a datum height, for measuring purposes, created with wooden poles and removable builder's line at 20 m intervals longitudinally and three central knots transversely. This leads to a total of 7 x 3 = 21 measuring locations.
- A high-energy impact roller would be used to complete five passes over the section, where after the total deflections would be measured at the knots and subtracted from the previous measurements. The deflection after each set of five passes would then be recorded. The average height drop should then be calculated.

This process should be repeated systematically until the average deflection is less than 2.5 mm per five passes or < 0.5 mm per pass. The total amount of roller passes needed to reach the limit criteria signifies the optimum amount of passes required for the section to be adequately compacted. It is important that the tractor unit must be sufficiently powerful to conduct the test at a speed which varies between 10 and 18 km/hour.

The above will ensure that all sections, albeit that they exhibit vastly different attributes, are compacted to a comparable stiffness.

COMPACTION AT EQUILIBRIUM MOISTURE CONTENT (EMC)

In order to ensure that large volume changes do not occur in the underlying material, it is envisaged that they will be compacted at EMC.

According to Emery (1988), the moisture content of the subgrade material below a pavement structure migrates with time to an equilibrium value. This EMC differs from the optimum moisture content. Should compaction of the subgrade then take place at optimum moisture content, the moisture content will simply increase/decrease towards its EMC value. This change in moisture content will cause swelling or contraction of the material.

It is imperative that construction of the upper layers of the structure should take place immediately before any of the moisture in the subgrade is lost due to evaporation. This EMC may be calculated from the following formula:

$$EMC = 0.45\,(Liquid\ Limit^{0.7})\left((P_{0.425})^{0.3}\right) + 5.29\left(Ln(100+I_m)\right) - 29.5$$

where:

I_m – Thornthwaite's moisture value for the area
$P_{0.425}$ – Percentage material passing 0.425 sieve

It should also be noted that this EMC is above optimum for most cohesive material. The compaction equipment thus causes a rise in the pore water pressure in the material.

To combat this rise in pore water pressure, the construction process is divided into two halves. One half is conducted in the afternoon, allowing for the excess pore water pressures to dissipate through the night. The final passes are then completed the following day, allowing for some 12 hours for excess pore water pressures to dissipate.

Through this process, swelling or contraction due to an increase or decrease in moisture content can be minimised, enhancing the expected lifetime of the access road.

CONCLUSION

A process of dealing with construction on an expansive or low strength subgrade was detailed in this article, describing the optimum number of roller passes which take into account the characteristics of the material in a broad sense, and constructing at equilibrium moisture content to decrease the risk of movements due to fluctuating moisture content. ◻

Predictable granites? Not really

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INTRODUCTION

The Alice Lane Phase 3 project comprises a multi-million rand development located in the heart of Sandton, the richest square mile of land on the African continent. The site is situated on the corner of Alice Lane and 5th Street, just opposite Sandton City and the Sandton Sun Hotel. The development, owned by Pivotal and developed by Abland, covers a total area of approximately 5 400 m^2. When completed, this will comprise an office building with seven basement levels and a total of 20 storeys above ground, with 35 000 m^2 of office space.

In order to realise this vision, the original Standard Bank building had to be demolished, and an excavation with a total volume of 110 000 m^3 and a maximum depth of 20.1 m was excavated for the basement parking structure. The lateral support contract (with a value in excess of R11 million) was awarded to Terra Strata Construction on a design and construct basis, who employs the services of Verdi Consulting Engineers for design. By the end of February 2015, the lateral support was approximately 90% complete.

Although the geotechnical conditions were believed to have been well understood at the start of the project, due to information obtained during the completion of the neighbouring buildings, it became apparent that the granites were highly variable and that continuous monitoring of the excavation was required during construction. The unfavourable jointing of the rock, which was identified midway through construction, resulted in considerable additional analysis and tweaking of the lateral support. This emphasised both the value of the observational method and the dangers posed by limited geotechnical information.

In order to realise this vision, the original Standard Bank building had to be demolished, and an excavation with a total volume of 110 000 m^3 and a maximum depth of 20.1 m was excavated for the basement parking structure.

GEOLOGY

As with all major developments, one of the key aspects affecting the design and construction of the lateral support is the underlying profile and geology of the site. The excavation is within the granite of the Johannesburg Granite Dome, which typically comprises residual granite underlain by massive rock with little or no discernible structure. The Johannesburg Granite Dome has been intruded by mafic dykes striking approximately in a north-south direction across the dome. A map created by Anhaeusser (1973) indicates the site as being underlain by a homogeneous, medium-

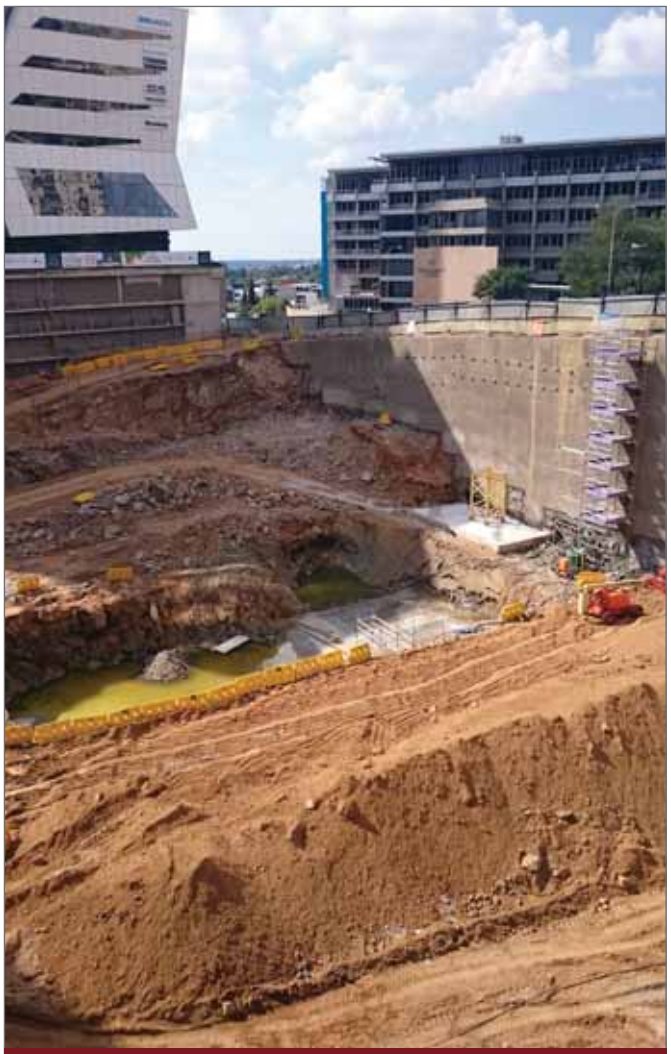

Figure 1: Alice Lane Phase 3 – development of the basement excavation

Figure 2: Alice Lane Phase 3 – another view of the basement excavation and lateral support

Figure 3: Dipping rock and dolerite dyke

Figure 4: Spheroidal weathering within the dolerite dyke

Figure 5: Contact between granite and dolerite

Figure 6: Dipping rock

grained, grey granodiorite. Also, according to this map, the site is located on the south-western boundary of a localised shear-zone.

At the time of the design of the lateral support, the only geotechnical information available comprised a geotechnical investigation report for the adjacent site and drawings showing the proposed elevations of the lateral support indicating rock levels. Limited geotechnical investigation was conducted due to the presence of the existing Standard Bank building which caused access constraints on the site.

During the initial stages of the excavation, a soil horizon comprising transported material underlain by silty sand and residual granite with an approximate combined thickness of between 5.0 m and 11.0 m was expected. However, as the excavation progressed, much shallower rock levels were encountered and a sudden transition from soil to rock occurred across most parts of the site.

The following is a brief description of what was typically encountered:

The granitic rock mass encountered is highly jointed and has been intruded by a large dolerite dyke, approximately 20 m thick and striking in a northeast-southwest direction.

The dolerite rock mass can be described as blocky, with the blocks being generally equidimensional to rectangular. Block sizes range from approximately 200 mm x 200 mm to 300 mm x 800 mm in size. Zones of very blocky material are present along contact between the dolerite and the granite material, with significant ravelling already being evident in these zones. The granitic material to the northwest of the dyke appeared to be less jointed compared to the material to the southeast of the dyke. Zones of spheroidal weathering were encountered throughout the dyke face ranging from 0.5 m to 1.0 m in diameter.

Given the unexpected structure, which was identified in the rock for three of the faces, it was deemed necessary to conduct further analyses to assess the impact that the structure of the rock would have on the overall stability of the rock mass and the lateral support.

ANALYSES

A discontinuity survey was conducted to evaluate the stability of portions of the rock face to determine the orientation of all discontinuities, the roughness, waviness, persistence and condition of the discontinuities.

The data gathered from the discontinuity survey was incorporated into the computer programme DIPS, in order to generate a stereographic projection of the discontinuities encountered on the three elevations. A kinematic analysis was generated for each potential mode of failure encountered across the elevations in order to assess their stability conditions. Based on the stereographic projections of the discontinuities, a number of potential modes of failure were identified for each elevation, and these were analysed to determine the stability conditions. For failure to occur, a number of geometric conditions need to be satisfied (Hoek & Bray 1981):

Planar failure

- The plane on which sliding occurs must strike parallel or nearly parallel (within approximately ±20°) to the slope face.
- The failure plane must 'daylight' in the slope face. This means that the dip must be smaller than the dip of the slope face.
- The dip of the failure plane must be greater than the angle of friction of this plane. An angle of friction of 45° was assumed for the granite.
- Release surfaces must be present in the rock mass to define the lateral boundaries of the slide. Alternatively, failure can occur on a failure plane passing through the convex 'nose' of a slope.

Wedge failure

- Considering two discontinuities form a wedge, Plane A and B, where Plane A is the flatter.
- The discontinuity intersection must 'daylight' in the slope face.
- The intersection must be inclined at an angle greater than basic friction on either of the discontinuities.
- The dip direction of the face and the discontinuity intersection must be within 20° of each other.

Toppling failure

- A toppling failure analysis was carried out utilising the DIPS programme which is based on the requirements for toppling failure by Goodman (1980).

The DIPS programme identifies the orientation of each discontinuity and plots the pole of each discontinuity on the stereonet. A kinematic analysis for each failure mode is then applied to the stereonet, which creates a critical zone (represented by the red zone on the stereonet in Figure 7). The poles of the respective discontinuities that lie within the red zone, are identified as critical poles, and failure may occur along these discontinuities.

A summary of the findings for a typical portion of slope is presented below:

A total of 8% of the discontinuities measured satisfied all the requirements for planar failure to occur. Toppling failure is not expected to be a problem on this particular portion of slope, with only 5% of the discontinuities measured satisfying the requirements for toppling failure. However, should the lateral limit be

increased from 10° to 15°, the percentage of discontinuities satisfying the requirements increases to 13%. The slope contained a significant number of critical intersections which meet all the requirements for wedge failure. The most critical failure mechanism seems to be the failure of small slabs.

It was clear from a simple visual inspection of the slope, and borne out by the kinematic analysis, that the slope face is unstable, with failure of blocks of limited extent being problematic.

The subsections of Figure 7 are typical examples of the output that was obtained for the Elevation 6 analyses.

IMPACT ON THE PROJECT

The results of the analyses revealed that the unusual structure of the granite rock mass and the dolerite intrusion would negatively impact the overall stability of the excavation. The visual inspections and analyses revealed the presence of unstable joint sets which are not typically encountered in granitic rock.

As a result, the design of the lateral support had to be revised to cater for the potential instability. This was done by identifying critical rows of rock bolts and increasing their length sufficiently to ensure that enough of the unstable blocks were bolted together by the proposed rock bolts to produce a stable rock mass. Figure 8 shows a typical section that was generated for the revised design of the lateral support.

CONSTRUCTION CHALLENGES

A number of challenges were identified and overcome during the construction of the basement for the Alice Lane Phase 3 development. The most significant challenge was the shallower than expected rock level which affected a range of different aspects of the basement construction.

As a result of the shallower rock, more drilling and blasting were required to excavate the basement to the final level. The additional drilling and blasting resulted in some delays to the earthworks programme, which in turn affected the lateral support programme. The quality, hardness and structure of the rock adversely affected the trimming of the rock face. Due to time constraints on the project, a decision was made to remove the rock through blasting rather than carefully pecking the face. This resulted in significant overbreak occurring and shotcrete thicknesses in excess of 350 mm in some areas. The blast designs had to be conducted to very strict peak particle velocity limitations due to the presence of the lateral support around the blast zones, as well as the proximity of the surrounding buildings.

In addition, as with most sites located in commercial hubs, the amount of space available on site and the access to the site were major challenges during construction. This required good communication and effective programming.

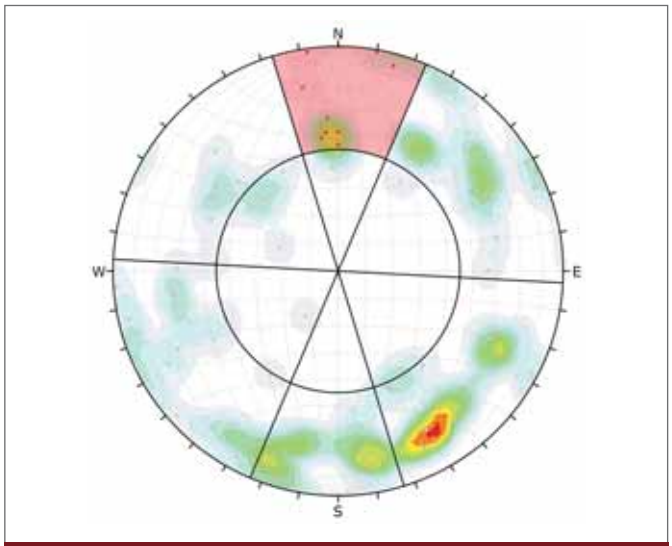

Figure 7(a): Planar failure

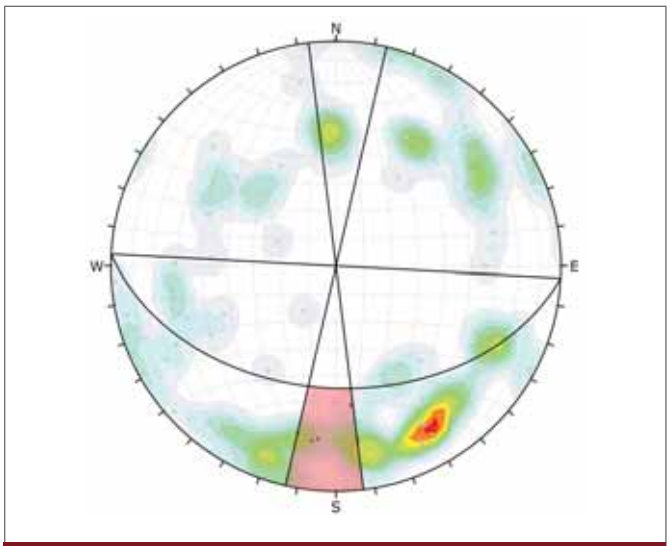

Figure 7(b): Toppling failure

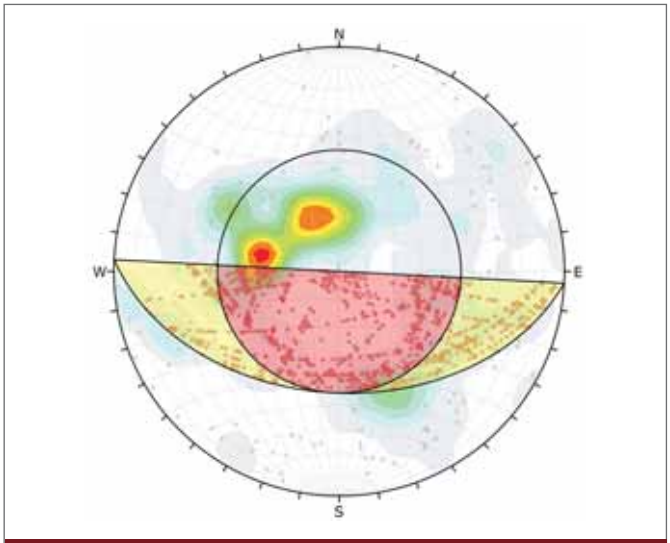

Figure 7(c): Wedge failure (intersections)

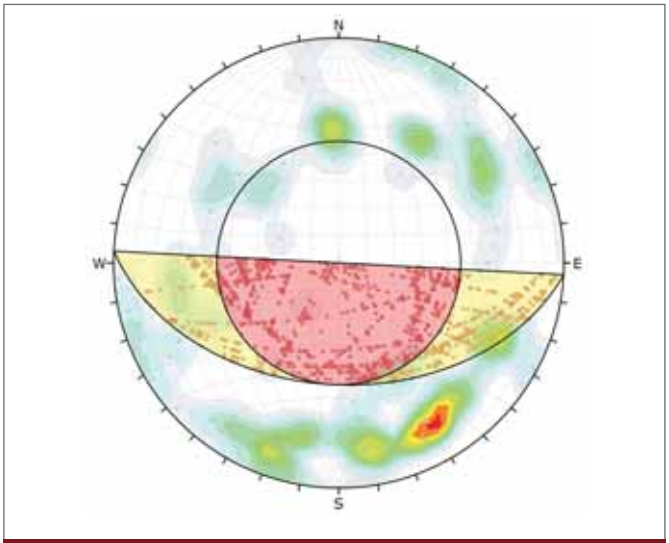

Figure 7(d): Wedge failure

CONCLUSION

The Alice Lane Phase 3 development is a major construction project currently under way in the heart of one of the most affluent cities on the continent. As evidenced by this project, even though the geotechnical conditions were thought to be well known and understood (due to experience gained from geotechnical investigations during the completion of the neighbouring developments), geotechnical conditions cannot always be predicted and site-specific geotechnical investigations are required in all cases.

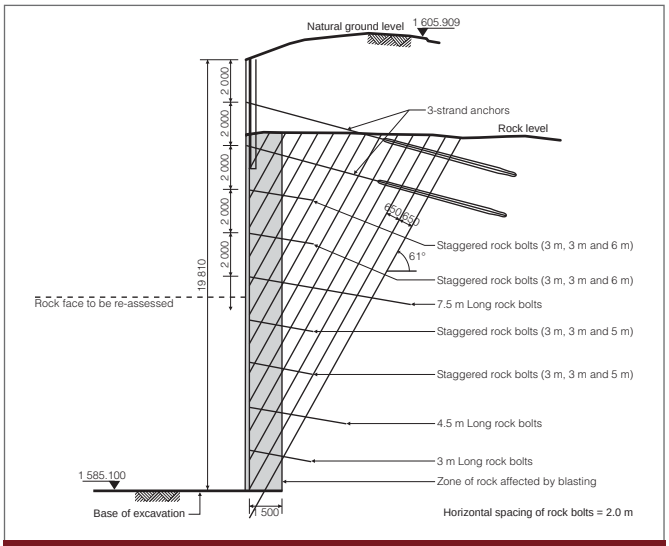

Figure 8: Typical modified section

Even though unexpected ground conditions were encountered well into the construction of the lateral support (i.e. conditions that were not described in the geotechnical investigation report), use of the observational method and a thorough analysis revealed that the design could be revised by supplementing the remaining support without having to change any of the support already installed. This solution minimised the financial and programme implications on the project.

As always with these types of projects, the interaction between the professional team, lateral support contractor, bulk earthworks contractor and blasting contractor determined the success of the project. On this specific project, the positive relationship between these parties successfully mitigated all the challenges experienced.

ACKNOWLEDGEMENTS

The authors would like to thank Pivotal, Abland, L&S Consulting and Terra Strata Construction for making the publishing of this article possible.

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Piling completed for a tower base at the Gouda Wind Farm

The role of the geotechnical engineer on site

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INTRODUCTION

On a regular basis ARQ is tasked with the responsibility of providing site supervision for geotechnical construction contracts. This article serves to highlight some of the geotechnical engineer's responsibilities on site, using a few of ARQ's recent projects as examples.

GEOTECHNICAL SITE CONDITIONS

Almost all geotechnical investigation reports are based on interpretations and conclusions which are uncertain, due to the nature of investigations. No study can investigate every risk, and even a rigorous assessment or sampling and testing programme may not detect all problem areas throughout a site. It is assumed that the conditions observed through the sampling programme are indicative of conditions throughout the site, and this is usually a reasonable interpretation. Designs are then based on information presented in the geotechnical report. It is therefore important for geotechnical engineers on site to assume the responsibility of reporting any geotechnical conditions that are not in accordance with conditions mentioned in the report.

ARQ was appointed for the design of the piled foundation for the Gouda Wind Farm in the Western Cape. ARQ was also requested to provide full-time site supervision during the installation of the piles. Unexpected conditions were encountered when four piles within a wind tower base did not intersect rock at the expected depth due to a dolerite intrusion intersecting the footprint of the foundation. The pile design and installation method were subsequently revised. Pile lengths were increased significantly in order to achieve the required pile load capacities, and temporary casings were used during installation to counter potential collapsing conditions.

A similar situation was recently encountered during the installation of soil nails for the lateral support for the Acid Mine Drainage Eastern Basin project. The site is generally underlain by residual mudstone and the lateral support was designed to resist a 7 m high vertical wall, a 3 m bench and the top 5 m battered back at 45°, with a total height of 12 m. The holes for the soil nails would have been drilled using percussion drilling techniques. This drilling method proved to be inadequate after holes collapsed and instability was noted in the unsupported wall during drilling. The installation method was changed to self-drilling anchors in order to address the above issues, and the design changed accordingly. Since percussion holes were drilled during the geotechnical investigation there was no reason to believe that the postulated installation method would not be suitable.

More than just identifying potential problems, it is also the geotechnical engineer's responsibility to report on conditions that are better than expected, and which might therefore result in cost and time savings for the client. During a recent contract where piles were installed for a bridge across the Thukela River near Sahlumbe, un-fractured very hard rock dolerite was encountered as founding material. According to the available geotechnical information, rock was expected to be no harder than 25 MPa. Uniaxial Compression Strength (UCS) tests of the

Lateral support at the Acid Mine Drainage Eastern Basin

dolerite during pile installation indicated a rock hardness in excess of 250 MPa. Rock sockets were reduced and extensive cleaning methods introduced to ensure pile load capacities were achieved by end bearing. The reduced socket lengths saved the client time and money for the installation of the piles.

QUALITY CONTROL

Quality control is an important part of any construction project, and is one of the major roles of the geotechnical engineer on site. This entails performing various quality control tests and measurements to confirm the quality of the work completed by the contractor.

CONCLUSION

Geotechnical engineers on site should confirm geotechnical conditions and make recommendations accordingly. This should be done to ensure that structures can be installed as efficiently and cost-effectively as possible, while ensuring that structures will perform as designed. ◻

Health and safety risks of geotechnical investigations in townships across South Africa

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BACKGROUND

In 2014 Aurecon South Africa (Pty) Ltd was appointed to carry out a detailed geotechnical investigation within one of the townships in Gauteng, South Africa. The appointment comprised a Phase I desk study and a Phase II geotechnical investigation including laboratory testing, dolomite study and extensive field testing.

The geotechnical investigation was conducted by the authors, both geotechnical engineers in Aurecon's ground engineering team. The work involved excavation of test pits for profiling and sampling utilising a tractor loader backactor (TLB) in confined areas amongst the informal settlements, often right adjacent to spaza shops, self-made walls and informal housing.

Typical township conditions encountered during the geotechnical investigations

This article outlines the key risks that were faced and what solutions were employed in order to mitigate them. The article also presents a risk management plan in the form of a step-by-step checklist based on the experience gained during our investigations. This checklist can serve as a starting point when planning intrusive investigations or engineering work within similar townships or environments.

SITE CONDITIONS

The site was densely populated with informal houses, and had extensive illegal overhead and underground electrical connections. At the time of the investigation, there were no reliable as-built drawings available that could give accurate indications of water and sewer service lines. This was mainly due to the sprawling informal housing which had spread rapidly since the servitude installations, and thus there was no clear indication of where services lay. Servitudes did not always fall along the road reserves, as could be expected in areas of formal housing. Thus identifying possible areas for ground investigations was complicated, and suitable locations proved to be limited.

IDENTIFICATION OF RISKS

Prior to planning the excavations it was important to obtain all wayleaves and as-built drawings. However, shortly after arriving on site, and while conducting a site walkover, it became evident that three significant health and safety risks needed to be resolved before work could commence. These risks (listed below) involved personal health and safety, as well as significant potential damage to property and/or machinery.

1. Risk of electrical shock to the operator, personnel, or local residents due to excessive illegal overhead and underground electrical cabling. These connections were unsheathed and under tension and could fall onto individuals if damaged by the TLB when operating on the site. The underground cables posed an even greater risk, as these were often only covered by a few centimetres of soil or were lying openly exposed on the surface.
2. Secondly, due to the unknown locations of services and the sprawling nature of the settlement, there was a high probability of intersecting underground servitudes. This would not only cause severe monetary damage to the infrastructure and surrounding dwellings, but could also create unrest among the local community, as service delivery protests had only just subsided in the area in the weeks leading up to the fieldwork.
3. Thirdly, and tying into the point raised above, was the risk of information not being conveyed timeously to the local community, resulting in reluctance of local community members to allow work with the TLB to continue unhindered.

MITIGATION OF RISKS

A meeting with all stakeholders was called in order to raise and discuss the possible risks. The stakeholders included the local community councillor and some of the leading community members, representatives from the local municipality, the client and Aurecon representatives. Through discussion and the combined effort of all parties, each one of the major risks was mitigated and classified according to the risk rating matrix shown in Table 1.

Table 1: Risk rating matrix

	Very likely to occur	Good chance to occur	Likely to occur	Unlikely to occur	Very unlikely to occur
Disastrous	Extreme	Extreme	Extreme	Extreme	High
Critical	Extreme	Extreme	Extreme	High	High
Serious	Extreme	High	High	Moderate	Moderate
Significant	High	High	Moderate	Low	Low
Minor	Moderate	Moderate	Low	Low	Low

Electrical shock

As per the matrix shown in Table 1, the likelihood of electrical shock was assessed as "very likely to occur" and a "critical" consequence, and was thus rated as an extreme risk.

To mitigate the risk of electrical shock, which was of utmost concern, it was decided to employ a specialist underground service detection company. It was proposed that the test pit positions be scanned prior to excavation by means of a Cable Avoidance Tool (CAT) and an RD8000 device to detect large underground current flows, and to mark up the location of potential illegal connections. We were then able to relocate the test pit positions accordingly. With regard to the overhead cables, we had to plan our route to each site carefully, and where access was not possible due to the density and low height of connections, relocate test pit positions prior to commencement of excavation.

Damage to property

This risk was assessed to have a "good chance to occur" with a consequence level considered to be "significant" and thus was given a rating of a high risk.

The risk of damage to services was mitigated through the assistance of a team from the local municipality who assigned a maintenance crew for the duration of the field work to aid with the identification of any known underground servitudes, based on their experience of working in the area. They were more than adequately equipped with tools and materials that would allow them to react and execute emergency repairs to any water or sewer servitudes in the event of accidental damage during excavations. Their presence with the Aurecon team also further assisted with public confidence that the work we were undertaking was fully supported by the relevant authorities.

Public unrest

This risk was pinned as "likely to occur" due to the recent protest action in the

area resulting from lack of service delivery. The consequence was therefore set as "serious" due to the threat of bodily harm to personnel in the event of unrest, and thus was rated as a high risk.

For mitigation of the risks pertaining to uninformed local residents, non-cooperation and unrest, three local community liaison officers (CLOs) were employed to inform and communicate with affected parties in the appropriate language at each excavation location, thereby ensuring understanding and support from the local community. The CLOs were also tasked with creating a secure work environment around excavation sites by keeping onlookers behind a danger tape line and keeping the local residents informed of our intentions and reasons for working in the area. They also assisted by talking to and getting information from the local residents about any known underground servitudes or illegal connections within their area, allowing us to adjust our test pit positions accordingly. The local community councillor also assisted by calling meetings in each affected zone to inform residents prior to our arrival at the location.

Once the above mitigation plans were implemented the project went ahead without any incidents.

RISK MANAGEMENT PLAN

It is of paramount importance to develop a safe work procedure before commencing with the field work. Based on our experience, we propose the following risk management plan to be employed when planning a geotechnical investigation in townships across South Africa:

Step 1: Determine proposed geotechnical investigation locations, and ensure that you have all available excavation permits in place, and study them in detail to ensure your locations are best suited with regard to known and existing servitudes and/or infrastructure.

Step 2: Establish contact with the local council member and ask for assistance with informing the public, and with contacting possible CLOs who could assist while working on site. Make sure to include provision for remuneration for these CLOs, as their services are well worth it.

Step 3: Make contact with the local municipality and find out if they can provide as-built or preliminary drawings which could give an indication of services on site. If they have the manpower to spare, their presence through a maintenance team or a representative on site would be invaluable. Ensure that you have all emergency contact information on hand and readily available, specifically whom you need to contact in the event of accidental damage and who will sign liability forms for the damage. Make sure to inform the responsible parties of this duty and of the fact that they will need to make a representative available to react immediately if an incident occurs. The responsibility would lie either with your company, the contractor or the client, and this should be established before commencement of work.

Step 4: If the site poses a high risk of electrical shock the services of a cable detection company should be sought, and all personnel should be inducted and informed of the risks that are to be faced on site. Make sure that all members stand at least 2 m away from the excavation and TLB during its operation. In addition, the TLB operator should be instructed that, while excavating, he/she should never be in contact with the metal/bodywork of the machine. The area must be made safe for the onlookers and residents by using danger tape to keep residents at a safe distance, especially as there tends to be a large number of small, curious children within the townships, and the operation of large excavation machinery always attracts onlookers.

Step 5: Take the time to speak to the residents/onlookers at the excavation location prior to commencement of excavation, as their insight and knowledge of the location can be invaluable; often they have lived there for many years or may well have been present during the installation of any servitudes/infrastructure. Ensure that all potential hazards are marked up with the use of biodegradable spray paint or danger tape, and that they are clearly visible to the operator. Care must be taken that, when stockpiling soil during excavation, it does not cover the identified locations of hazards.

Illegal overhead and underground electrical cabling posed a major risk

CONCLUSION

Working within townships can be an incredibly rewarding and enjoyable experience if all the required precautions have been implemented. There are significant risks that are not often faced in the usual run-of-the-mill geotechnical investigations, but if properly dealt with and managed, these can be quite readily mitigated. We hope that the above risk management plan will help towards the smooth and safe running of future geotechnical investigations in townships. ◻

The need for 'hands-on' geotechnical engineering

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BACKGROUND

From the tallest building and the largest dam or harbour wall to the most humble cottage, there is a foundation where the structure interacts with the earth. If that foundation does not perform satisfactorily the structure above it will not perform as it should, and failure to some degree will occur. It was largely the impact of a series of spectacular failures in Europe (such as the railway embankment at Weesp in the Netherlands) which led to the development of soil mechanics as a civil engineering discipline nearly a hundred years ago. One of the prime movers of this development, Carl Terzargi, set soil mechanics on a scientific footing with his insights into effective stress. Terzaghi moved from Germany, eventually settling in America, where his successes helped to set soil mechanics in high regard, and a generation of greatly respected personalities in the field of geotechnical engineering emerged. One of Terzaghi's pupils, Jeremiah Jennings, was perhaps South Africa's best known example of the giants in this field. A notable feature of this generation of soils engineers was their 'hands-on' approach. They appreciated the necessity of personally interacting with the materials involved – going down test holes to examine those materials under field conditions, collecting and testing their own samples. They also appreciated the necessity of personally monitoring the performance of their designs to check that their calculations matched reality within acceptable limits.

THE CHALLENGING SITUATION TODAY

Major changes are underway in civil engineering in South Africa, especially in the case of ordinary bread-and-butter projects. There is, in particular, an unhealthy pressure for economy and speed of construction. The replacement of SABS 1200 by SANS 2001 is symptomatic of this pressure. Responsibilities allocated to the engineer by SABS 1200 are no longer specifically allocated to anyone at all. The result is that today project managers and quantity surveyors (often with very little understanding of engineering, and with their focus on time and cost alone) are in control of many, if not most civil engineering undertakings. Soil mechanics is a major area of ignorance among these project managers and quantity surveyors. We have a situation which invites potentially serious consequences. As an example, construction of a series of schools in the Free State was planned in 2014. The project manager considered that R5 000 was adequate for a geotechnical investigation at each school. When informed that the most perfunctory investigation would cost at least twice that amount, he proposed eliminating geotechnical investigations altogether!

A notable feature of this generation of soils engineers was their 'hands-on' approach. They appreciated the necessity of personally interacting with the materials involved – going down test holes to examine those materials under field conditions, collecting and testing their own samples. They also appreciated the necessity of personally monitoring the performance of their designs to check that their calculations matched reality within acceptable limits.

Perhaps it is because geotechnical engineering deals with things hidden below the ground, where lurking problems cannot be immediately seen and where engineering solutions to those problems are soon backfilled and hidden from view, that there seems to be a growing lack of appreciation for the discipline. It is perhaps not surprising that we have geotechnical investigation being typically allocated only about 0.15% of the cost of a construction project. The geotechnical allocation for a typical government subsidy house is about R104. Such an allocation is not likely to attract the services of a geotechnical engineer with sufficient skill and experience to give a sound engineering assessment. It would hardly attract even a relatively inexperienced engineer to give much of his personal attention to the problem. Collection of samples may be delegated to a junior, who simply delivers them to a commercial laboratory. The laboratory is asked to perform a specified set of tests, of which the relevance to the project at hand may be very limited. The engineer involved with foundation design may never have seen or handled any of the soils concerned. His only contact with them may be a set of numbers produced by the testing laboratory.

Deterioration of structural integrity typical of subsidy housing projects. In the development where this photograph was taken, approximately 25% of the houses were in a condition similar to this within five years after completion. The CUT research group has examined one development were almost 30% of the houses became unfit for habitation and had to be demolished.

Heave damage to a house shortly after completion. The CUT research group examined a development where one of the houses became structurally unsound (and had to be demolished) even before it was completed. The tests performed in the geotechnical investigation had given a very misleading indication of heave potential.

THE ESSENCE OF THE PROBLEM

The essence of the problem with this approach is the uniqueness of soil as an engineering material. Most materials – concrete, steel, aluminium, etc – have engineering properties which are easily appreciated, remain reasonably constant throughout the life of a structure and can be determined to a high degree of certainty. Soils have properties which may vary enormously depending on their water content and loading history – and no two soils are identical. No two soils behave exactly the same, even if the numbers coming from some particular test on them are the same. An appreciation of soil as an engineering material requires personal involvement to a far higher degree than with any other engineering material. At the Geotechnical Research Group of the Central University of Technology in Bloemfontein (CUT) no one gets involved without doing enough basic testing on soils to develop a feel for their behaviour. Perhaps this should be standard practice for everyone concerned with engineering soils.

When one has gained a feel for soils one can begin to make a fair assessment before any tests have been done – as in the case of a CUT post-graduate student taking a lump from a sample bag and, after a brief interaction with it, declaring: "This is the stuff that nightmares are made of." He was correct, but one should not lose sight of the fact that soils can be so variable that occasionally a specimen may fool almost anyone.

A relatively brief series of testing encounters with soils should be sufficient to raise serious doubts in any engineer's mind about the suitability of some of the TMH1 and SANS 3001 methods for anything other than highway construction purposes.

Highly plastic clays in particular do not give a good indication of their properties when tested using these methods. Highly plastic material is usually rejected for highway construction, or given stabilisation treatment, so a general indication of high plasticity may be all that is required. The soils under foundations, particularly for smaller buildings, are rarely removed or given stabilisation treatment, and an accurate assessment of their properties, such as their heave potential, is required for effective design purposes. The materials used in highway construction are frequently transported, remoulded and compacted; almost all of the materials under most building foundations are not so treated. The fact that highway construction materials tests are specified for almost all foundations of light structures should be a cause for concern.

Complete reliance on nothing but the numbers coming from commercial laboratories – from tests appropriate only for highway construction purposes – may be one of the reasons for the sad state of the government's subsidy housing projects.

Many of the first wave of government subsidy houses deteriorated so rapidly that a geotechnical investigation became a specified requirement. So many of the newer houses are deteriorating rapidly that some housing department officials have raised the question: "Why do a geotechnical investigation at all?" The situation with government subsidy housing is not a good advertisement for the civil engineering profession. Solving this problem will take more than just a more sensible allocation of funds. It will need a major change in outlook.

CONCLUSION

The great pioneers of soil mechanics understood something that seems to be in danger of being lost. The current obsession with cutting costs and saving time is making that loss ever more imminent. The most advanced mathematical analysis and the simplest empirical method for predicting foundation behaviour depend critically on meaningful input parameters. Those inputs come from understanding and assessing the soils involved. To understand soils it is not sufficient to look at some numbers on a laboratory test report; one must be prepared to get down on one's hands and knees and listen to what they have to say. Soils speak in a language very similar to braille. If you do not get your fingers onto them they will not be able to tell you the things you need to know about them. □

RESEARCH TEAM

The CUT Geotechnical Research Group consists of teaching staff, civil engineering practitioners, post-graduate students and keenly interested undergraduates associated with the Central University of Technology in Bloemfontein. One of its main fields of focus is soils testing for foundation design of light structures.

The role of civil engineering materials laboratories

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RECENTLY AT A SAICE GOLF DAY in Bloemfontein, two engineers were discussing ongoing projects, touching on various aspects of services provided to them by various service providers. Their discussion was interrupted by the SAICE facilitator calling for attention to the announcements and proceedings to follow. Keen to follow their conversation, as I'd heard one of them mention the Roadlab branch that I happen to manage, I joined their table.

Eavesdropping on their conversation I learned a few interesting things about the perceptions out there concerning civil engineering material testing laboratories. I was astounded to learn that soil laboratories are generally only considered as service providers in the same way that plant-hire companies and trading stores are viewed. Samples are sent, results received, end of story …

Working with clients, and specifically consultants, I have been on the receiving end of very interesting questions, and often find myself dumbfounded by those questions. Some questions are extremely technical, and I have been pleasantly surprised by some engineers' in-depth knowledge of laboratory work, while other questions make one wonder whether those consultants have any knowledge whatsoever regarding the material testing industry.

Several consulting engineers, who are supervising road construction projects, have asked me what a CBR (California Bearing Ratio), UCS (Unconfined Compressive Strength) and/or ITS (Indirect Tensile Strength) are. Often, when given the laboratory test reports, consultants reply with, "What does this mean? Does it pass?" These are some of the most vital information items available to engineers to base their design decisions on, yet they are completely unfamiliar with it. It begs the question of whether in situ material is even considered during the design process.

Typically on road rehabilitation projects a centreline survey is conducted prior to any designs being done. It is a time-consuming task that costs a considerable amount of money, yet it seems that often this is only done to satisfy a requirement in the specification and is not even considered during the design stage.

On the same SAICE golf day there was a short discussion on where civil engineering consultants and contractors fit it. A question raised by one of the members regarding where laboratories fit in was answered by the speaker in an almost unsure tone, indicating laboratories as service providers.

The truth is, civil engineering laboratories are one of the backbones of the industry. Prior to starting my career at Simlab in Bloemfontein, I was interviewed by Mr Gerhard Verwey. He is a well-known and respected figure in the roads sub-division of civil engineering. He stressed the importance of understanding materials prior to working in a design office (a long cherished dream of mine), and appointed me in a soils laboratory to obtain the necessary laboratory experience. Why is it generally only senior engineers who see the value of understanding the materials we are constructing our livelihoods and the country's infrastructure with? Everything in civil engineering consists of materials; is it not vitally important then to understand the test reports relating thereto?

I want to categorically state that civil engineering technicians, technologists and the few engineers working in civil engineering materials laboratories across the country, focusing on soils, aggregates, concrete, asphalt, chemically blended materials, masonry, timber and steel, form a vital cog in the engineering industry.

The value of understanding materials is greatly underestimated, and working as a branch manager in this field I ex-

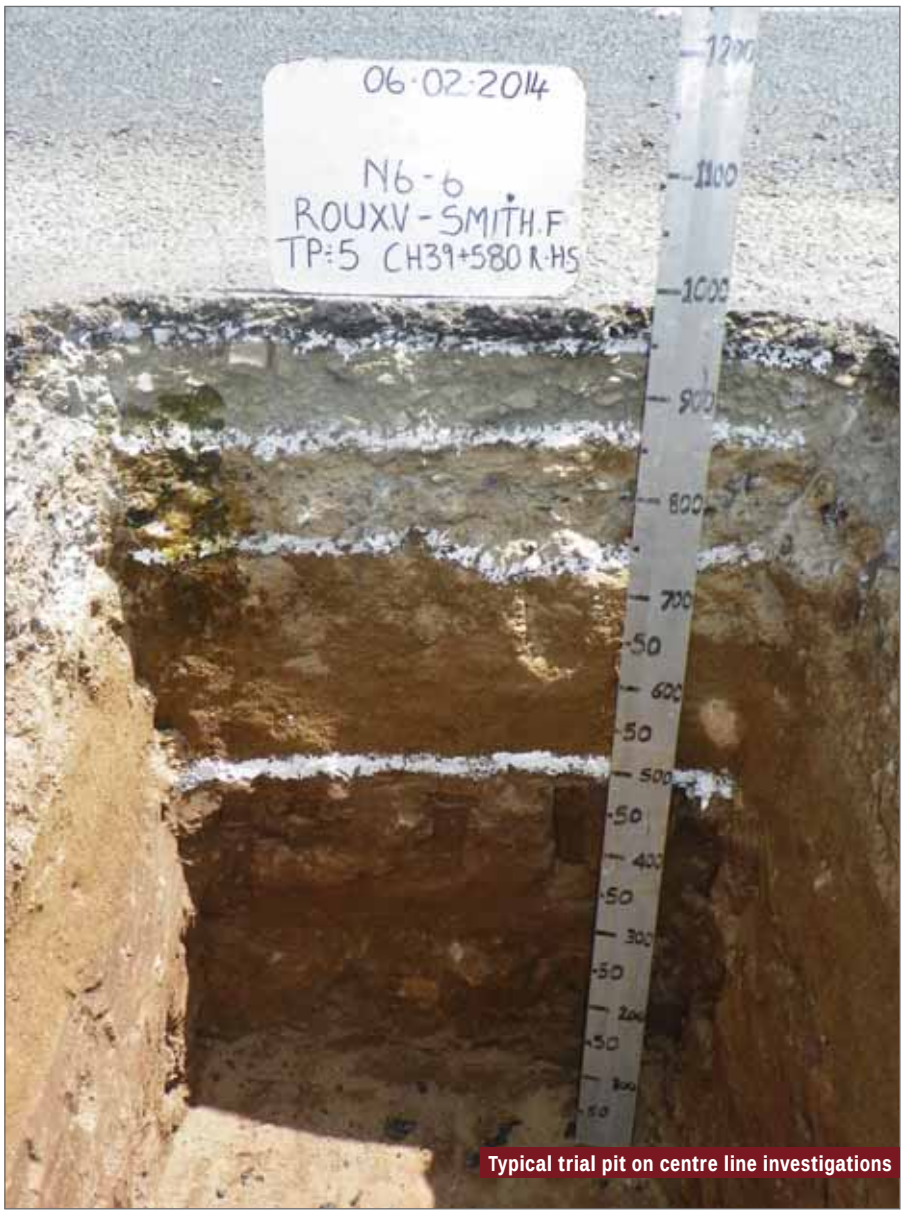
perience the perceptions about civil engineering materials laboratories. Civil engineering students have no interest in working at a laboratory, because they "want to focus on civil engineering". Why are laboratories not perceived to be part of civil engineering?

Typically the technical staff at local soils laboratories are experts on the material in their vicinity and usually possess a wealth of information on most of the relevant materials surrounding the laboratory. This information is readily available at little to no cost, so why not utilise this knowledge?

Material testers and technical personnel at laboratories have hands-on experience of most matters related to materials. Material technicians and technologists visit many construction sites and witness many types of construction methods, and can in all probability add value to any project, should their input be requested.

Working as a manager at a testing laboratory is an extremely challenging task. Often we find ourselves positioned between the consultant and the contractor. Whenever a test report is positive the consultant suspects the laboratory of favouring the contractor, while negative reports are deemed to be some mistake on the laboratory's part by the contractor. Consultants and contractors need to realise that there is absolutely no point in favouring any one side; it is imperative to obtain consistent and reliable results. There is a tendency among many consultants to police the laboratories, as we are simply not seen as professional bodies. It appears to stem from perceptions that are enduring, due to misinformation fed to young civil engineers, technicians and technologists coming through the ranks.

If you have the time to visit a soils laboratory close to you, use that time to do so and enlighten yourself. Consider it an investment into your future. Understanding the tests will aid you in understanding the test reports. Understanding the test reports should inevitably lead to better, more durable road designs, which should ultimately lead to less potholes. If you are unsure about which tests need to be conducted for whatever purpose, ask the laboratory personnel. The chances are they've done similar work before on several occasions and should therefore be familiar with all the technical requirements. ◻



Typical trial pit on centre line investigations

Typical samples from trial pit

The importance of practical skills in geotechnical courses

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Another important aspect is to balance the available time between helping the students learn concepts and understand theory, and spending enough time in the laboratory to provide them with practical laboratory skills.

THE GEOTECHNICAL world has expanded in the last decade and now involves much more than the basic concepts expounded by the founders of engineering, such as Terzaghi, Peck, Kovacs and Holtz. Today underground resources require new perspectives in terms of development, which in turn needs advanced knowledge and skills in geotechnical engineering. Therefore an active learning approach and hands-on laboratory experience must be a key component in any undergraduate course. This not only enhances the understanding and application of theory, but also the knowledge retention of the student.

The undergraduate geotechnical laboratory must therefore create the opportunity for civil engineering students to investigate soil behaviour in a controlled experimental setting, so that hands-on knowledge of practical geotechnical testing is acquired. Even though undergraduate courses differ widely with regard to names, content and degree of difficulty, all of them have to focus on students' need to be able to understand sampling, measurement of properties, and data interpretation.

Another important aspect is to balance the available time between helping the students learn concepts and understand theory, and spending enough time in the laboratory to provide them with practical laboratory skills. Something else that must be taken into consideration is that, with the wide variety of sub-disciplines in civil engineering, the majority of students taking the required undergraduate geotechnical courses might not become geotechnical engineers. Most likely they will become general civil engineers. However, it is still extremely important that these civil engineers must understand basic geotechnical tests to interpret the accuracy of information contained in site investigation reports, and most importantly, be able to interpret and understand geotechnical recommendations needed for doing their own designs.

The most common laboratory experiments performed by students usually include the following:

- Moisture/water content
- Atterberg limits
- Specific gravity
- Particle size distribution
- Compaction
- Permeability
- Direct shear
- Consolidation

It is important that staff conducting these tests are experts themselves in these fields and in the different testing procedures. In the laboratory students are typically guided through a series of experiments using a laboratory manual under the supervision of staff. These laboratory manuals are usually the ones that are used in the private sector (e.g. AASTHO, TMH1 and ASTM) or simplifications of these standards. This counts in favour of the student who wishes to do his/her experiential training in an accredited laboratory. However, students sometimes consider the standard manuals difficult to understand, especially if the equipment used differs from that in the manual. To solve this problem a supplementary manual can be compiled by the technical staff to assist the students with experi-

ments not matching the actual manual. Other complementary materials that can be used are videos, slide shows, workshops and field trips.

Cognisance must also be taken of the use of sophisticated modern automated equipment versus the manual devices still commonly used in many commercial laboratories. Acquiring automated equipment can be beneficial to the learning process, as this speeds up long laboratory testing such as direct shear testing, oedometer testing and tri-axial testing, but should not replace manual testing.

It is also important to look at the benefits of test simulation software versus real testing. Software development permits students to simulate the testing process in a virtual environment. The advantages are that the virtual tests can be a substitute when laboratory equipment, expertise, and/or time to perform an actual test are unavailable. One disadvantage is that students first need time to learn how to use a complicated computer program that may only be used in one or two courses. Therefore it is important for staff to determine which software programs are used in the private sector and be certain that the computer program can be used in other subjects, and future geotechnical engineering courses. Most undergraduate courses include assignments as part of the student's practical mark to gain examination admittance. An important aspect is that these assignments must be examples of real projects to enable the student to do real geotechnical work. Made-up imaginary assignments are not beneficial and do not add value.

An aspect that is very often overlooked is the advantage of involving research staff and/or off-campus practitioners. A well-seasoned and experienced practitioner/researcher can be an important resource, and can therefore be of extreme value to students, as well as to full-time staff who did not have the opportunity to practise engineering for a substantial period of time before joining academia.

This approach will also help to change the private sector's perception that engineering education does not prepare graduates adequately for practice.

Another aspect to bear in mind is that ECSA (the accreditation body for engineering education) regards the combining of practical experience and teaching by introducing relevant applications and real design problems into the curriculum, as very important. Of great value is the chance to set up linkages with the private sector, thereby creating cooperative activity opportunities for graduates, and the potential for collaborative research programs.

However, there will always be limitations to what can actually be achieved in the teaching laboratory. The biggest constraints are time and costs. How much time needs to be spent on laboratory work versus theoretical concepts, bearing in mind, too, that time spent in the laboratory includes planning/setup, actually doing the experiments, and then cleaning up and storing the equipment. Furthermore, facilities are limited by space, equipment availability and equipment cost.

In summary, civil engineers rely significantly on the soil and rock properties developed through field and laboratory tests for their designs. Therefore, all civil engineers should have a proper understanding of the different methods of field tests, sampling techniques, various types of samples required to perform geotechnical experiments, and analyses to interpret and understand geotechnical concepts needed for doing their own designs.

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Students engaging in a practical session in the geotechnical laboratory

Seasoned practitioner adding value to students' laboratory experience

Geotechnical research at Wits

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Conventional wisdom is that if one cannot condense one's research into a few words then one does not know what one is pursuing. To this end, the following article has been written to briefly highlight some of the research activities that are ongoing at the University of the Witwatersrand.

INTERNAL EROSION

Seepage-induced erosion of earth dams, particularly long-term suffusion, can unwittingly jeopardise dam safety. A loss of finer particles can take place within internally unstable (or non-self-filtering) materials when a critical combination of stress and hydraulic load develops. The phenomenon is termed *suffusion* if an equilibrium is reached in which the stresses are maintained despite the loss of finer particles, or *suffosion* if the soil fabric destabilises. Various geometric (or grading) criteria are available to determine whether a particular material is self-filtering, although these tend to be conservative. Recent discreet element

Figure 1: General arrangement of test setup

Figure 2: Close-up of variedSB

modelling (Muir Wood *et al* 2010; Shire *et al* 2014) has shed light on the associated volume and strength changes, and the interactions between soil fabric, stress and hydraulic load, as fines are lost. However, these models are experimentally untested.

To this end, the vertical axis restrained internal erosion direct shear box (variedSB) has been developed at Wits to investigate the phenomenon. Vertical axis restraint has been incorporated to enforce rotational restrain of the top platen (Shibuya *et al* 1997; Lings & Dietz 2004). The top platen and bath have been modified so that flow can take place through the sample to cause particle loss. This will allow various internally unstable materials to be tested at different densities and stress levels, and the volume and strength changes to be quantified. Photographs of the device are given in Figures 1 and 2. As the equipment is non-standard, tests on silica sand are underway to refine the device and establish appropriate test procedures.

MECHANICAL CHARACTERISATION OF TAILINGS

The steady state line (SSL) of a soil is defined as the locus of all points in void ratio-stress space at which a soil mass deforms under conditions of constant effective stress, void ratio and velocity (Been & Jefferies 1985). Soil liquefaction susceptibility is known to be closely correlated to where the soil plots in void ratio-stress space with respect to its SSL. As such, determination of the SSL is an essential step when considering the liquefaction susceptibility of any soil mass. Tailings storage facilities (TSFs) are known to be susceptible to liquefaction, which is why research is currently underway at Wits to better understand how the characteristics of tailings affect their SSL. Several researchers (e.g. Cho *et al* 2006; Rahman & Lo 2008; Gang 2013) have reported the effects that characteristics such as particle shape, the coefficient of uniformity, and fines content have on the SSL. However, there is still no agreement on which index parameter could be most suitably used to capture the combined effect of these characteristics. The current research effort aims at contributing to bridge that gap in our understanding of the SSL.

Figure 3: Automated triaxial testing equipment

Triaxial testing is the method most commonly used to determine the SSL. For this purpose, the Geoffrey Blight Soils Laboratory at Wits recently purchased a fully automated triaxial testing machine (Figure 3). The installation and calibration of the equipment are currently underway and will allow triaxial testing to the most modern standards. Additionally, two manual Wykeham-Farrance loading frames have been refurbished and fitted with transducers to allow triaxial testing with automated data acquisition. Although the current focus is on the triaxial testing of tailings, the improved understanding of the SSL is expected to be applicable to a wide array of non-plastic soils.

RAPID PILE TESTING

In South Africa a smaller proportion of foundation piles are load tested than in many other countries. This is because the methods of load testing that are quicker than the traditional static load test (i.e. the 'dynamic' and 'rapid' tests – possibly ten piles per day) are not much used. One set of electronic equipment for the dynamic test is in occasional use, with the measurements interpreted overseas, but the rapid test has never been used in South Africa.

Research is under way at Wits to devise equipment that will enable the rapid test's pattern of loading to be applied to piles that are also being tested by the static load test. The objective is to compare the results of the two tests and so gain confidence in the interpretation of the rapid test into behaviour under static load. A further objective is to design and build relatively inexpensive equipment to carry out many rapid tests on working piles in a day.

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8th SAYGEC cultivated at Spier Wine Estate, Stellenbosch

The conference was overseen by our godfather, Ken Schwartz, who gave us valuable feedback on the papers that were presented. After each presentation session Ken provided the delegates with a short summary thereof. Offering not only technical inputs and asking thought-provoking questions, he provided us with numerous interesting titbits of his time spent studying and practising with the greats of our industry, such as Professors Jere Jennings, Geoff Blight, Ken Knight and Tony Brink, to name a few. In so doing he challenged us as young engineers in many ways about our future and the future of our profession. We are very thankful to him, his wife Marion and son David for attending, and the time and hard work he put into making this venture a great success.

FOURTEEN YEARS after SAICE hosted the 4th YGE in Stellenbosch, 2014 saw the return of the much anticipated event to the 'City of Oaks', aka the 'Eikestad'. The 8th South African Young Geotechnical Engineers Conference (SAYGEC) was hosted by the SAICE Geotechnical Division at Spier Wine Estate from 17 to 19 September last year. In keeping with the venue, the theme was *Cultivating the future of Geotechnics*. The triennially hosted event provided ample opportunity for interaction among delegates from different geo-related institutions between and after conference sessions. Geotechnical engineers, engineering geologists and rock mechanic engineers from SAICE, SAIEG and SANIRE under the age of 35 (at the time of abstract submission) were invited to submit papers. The event was attended by 114 delegates and 60 papers were presented, covering a wide range of topics.

The conference was overseen by our godfather, Ken Schwartz, who gave us valuable feedback on the papers that were presented. After each presentation session Ken provided the delegates with a short summary thereof. Offering not only technical inputs and asking thought-provoking questions, he provided us with numerous interesting titbits of his time spent studying and practising with the greats of our industry, such as Professors Jere Jennings, Geoff Blight, Ken Knight and Tony Brink, to name a few. In so doing he challenged us as young engineers in many ways about our future and the future of our profession. We are very thankful to him, his wife Marion and son David for attending, and the time and hard work he put into making this venture a great success.

Needless to say, the social activities were great fun, such as a team-building evening of drumming followed by a sit-down dinner. The wine blending and tasting event especially was a highlight as teams competed to win over the vintners at Spier with their own unique blends. The SAYGEC is always loads of fun, where new acquaintances are made, and where young professionals learn from one another in an environment that allows speakers to present to a more forgiving audience.

The Division would like to congratulate Luis Alberto Torres-Cruz and Jean Potgieter for winning the best paper and best presenter awards respectively. Courtesy of the sponsorship from the Geotechnical Division, they will be attending the international YGEC in Seoul, Korea, in 2017.

The following is a summary of the SAYGE 2014 award winners:

- Best paper: Luis Alberto Torres-Cruz (University of Witwatersrand)
- Runner-up best paper: Valencia Kuppusamy, Ashmita Boodoo (Golder Associates)
- Best presenter: Jean Potgieter (Jones & Wagener)
- Runner-up best presenter: Ryan Freëse (SMEC SA)

The Division would also like to thank the organising committee, which comprised young people from the industry, for their time and effort, as well as RCA conference organisers for all their hard work in making the event such a success.

Finally, the Geotechnical Division of SAICE sincerely thanks the following companies for their sponsorships:

- Franki Africa (platinum sponsor)
- SMEC (gold sponsor)
- Geotechnical Division (prizes for best paper and presenter to attend iYGEC)
- Steffanutti Stocks
- Crossman Pape & Associates
- Aveng Grinaker
- Jones & Wagener
- Bauer
- Kaytech
- Geo-explore store
- All the exhibiting companies.

Marie Basson
Young Geotechnical Engineer
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Best paper winner: Luis Alberto Torres-Cruz (University of Witwatersrand) receiving his award from Marie Basson of the Geotechnical Division for his paper titled *Assessment of two methodologies for evaluating the liquefaction potential of tailings storage facilities using the CPT*

Best presenter winner: Jean Potgieter (Jones & Wagener) for a paper titled *Effects of compaction on lateral earth pressure*

Ken Schwartz (left) and delegates at the Franki Africa exhibition stand (platinum sponsor)

Delegates engaging in some fun wine blending activities

The winning wine blending team

Geopile Africa expanding the footprint of the TRM ductile piling system

TRM piles being installed for a sewage treatment works in Port Elizabeth; note close proximity of piling works to existing structures

Specialist foundations and piling contractor Geopile Africa is busy expanding the footprint of the TRM ductile piling system by performing many exciting and challenging projects all around South Africa.

TRM PILING SYSTEM POPULAR IN THE EASTERN CAPE

In Port Elizabeth, TRM piles were chosen and installed for a sewage treatment works, as well as for an ACSA (Airports Company South Africa) warehouse. One of the challenges at both sites was that the piling works had to be done in very close proximity of existing structures, in some cases only 40 cm away. TRM piles are low-displacement prefabricated piles, and by driving them using powerful high-frequency hydraulic hammers, the peak particle velocity was below 1 mm per second, a low level of vibration which most structures are unaffected by. At the sewage treatment plant there were very fragile large diameter sewer pipes running at full capacity, so any other displacement piling system would have been a risky choice. But, due to the minimal vibrations caused during installation, all of the TRM piles were successfully installed without causing any damage whatsoever. In addition the saturated and potentially contaminated soils were simply displaced by the full displacement TRM piling system, completely eliminating all associated risks and possible high handling and disposal costs.

In Alice, TRM piles were successfully chosen and installed for the abutments of a river bridge, and in East London for a shopping centre and several three-storey residential blocks.

Access was one of the challenges in all three of these projects – in Alice the site was located in a rural area with limited space for site platform preparations, and

the access and working space at both the East London sites were also limited. However, the advantages of the TRM piling system were again displayed and the compact piling rig excavator was able to successfully complete all pile installation works, showing great versatility.

ADVANTAGES OF TRM PILING SYSTEM DEMONSTRATED IN DURBAN

In Durban, the challenges faced by Geopile Africa on a particular site were even greater. Not only were the access and site working areas extremely narrow, but in addition, the piling rig had to be tracked between taxis parked along a narrow street, and then the piling works had to be performed adjacent to a fully operational railway line. The client personally came to witness Geopile Africa's piling rig being manoeuvred through the streets and onto the site, and commencing the installation of TRM piles. He was impressed that it took less than one hour for Geopile Africa to achieve this from the time of arrival of the low-bed transport outside. A large static load test was also performed at this site, with the test pile located next to the railway line. The client was impressed by the very positive load test result which was achieved with limited geotechnical information available about the bedrock underlying the upper saturated alluvial materials.

TRM PILING AT MINES AND PROCESSING PLANTS

TRM piles have been chosen and installed at numerous mines and processing plants all around South Africa. New structures at a manganese mine in the Northern Cape were successfully supported on TRM piles, while in Middelburg piled foundations for new processing plant transformers and other heavy equipment have been constructed on TRM piles. Several complex and challenging projects were also completed in the coal mining sector, at large coal mines where Geopile Africa's compact piling rigs again have demonstrated their versatility to the customers. Whether the customers are dealing with manganese, coal, chrome, or anything else, is of little importance to the TRM piling system, with which Geopile Africa continually manages to find innovative, fast and sure solutions for its customers.

ONGOING TRM PILING WORKS AT VARIOUS SOLAR PROJECTS

In addition to the selection of projects mentioned above, Geopile Africa has also been busy with ongoing TRM piling works for various solar projects located in the Northern Cape. These are all large-scale projects, each requiring tens of thousands of piles connecting the foundations of the solar structures down into the shallow bedrock. High quality and productivity are the main benefits offered from using the TRM piling system, which is unaffected by the unstable Aeolian sand layer and which easily penetrates the abrasive Calcrete before penetrating solidly into the underlying medium-hard bedrock. TRM piles can be designed and are guaranteed for up to 100 years' service life, which is also attractive to customers who may have plans for electricity generation beyond their current 20–25 year licence. In addition, Geopile Africa is currently working on piled foundation solutions for PV projects and is planning to launch a complete turnkey system, including the design, supply and installation of the TRM piles/posts plus the PV support frame structures.

Rob Marsden, CEO of Geopile Africa explains, "Our aim is to make the job of the solar panel erector as simple and as fast as possible by offering the customer a complete turnkey service to get projects off the ground using TRM piles/posts and including the PV panel support structure. In this way Geopile Africa carries the risks and complexities that the customer will otherwise continue to face due to the challenging and highly variable geotechnical conditions of the Northern Cape".

100% TECHNICAL COMPLIANCE AND CUSTOMER SATISFACTION

At every project Geopile Africa checks to make sure that the customer is totally

The piling rig manoeuvring itself through taxis parked on both sides of the narrow access street in Durban


TRM piles have been chosen and installed at numerous mines and processing plants all around South Africa

satisfied with the performance and service provided. Geopile Africa is happy to report that in every case so far the customer has expressed complete satisfaction with our service and performance. In particular, the safety, simplicity and speed of the TRM piling system have been appreciated by the customers, as well as the versatility and numerous other benefits (no mess, no pile head trimming, etc) that the TRM piling system provides. Dr Ruan Swart of Royal HaskoningDHV kindly said, "I'd always found piling to be such a big, slow, messy and complicated procedure until I witnessed Geopile Africa in action with the TRM piling system, which was just so fast and easy – what a pleasure!"

▶ INFO

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The versatile TRM can work comfortably in restricted space

This series of articles is presented in the author's personal capacity and does not reflect the views of his employer.

Behaviour change at household level in the water sector

This article is the fourth in a series on the economic pricing of services and the beneficial effect this could have on the economy and the everyday lives of people. The first three articles appeared in the October 2014, November 2014 and March 2015 editions of *Civil Engineering*.

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Braaiing your money by not fixing leaks!

INTRODUCTION

This article will argue that, even where there is a clear economic incentive through the tariff to reduce water consumption, if the consumer remains unaware of what is reasonable consumption, it will not result in the necessary behaviour change in being proactive about fixing leaks, reducing excessive garden watering and getting teenagers to take shorter showers. Paying for water and not using it effectively is the same as putting it on the braai and cooking it without any meat.

The article will show that even where consumers receive regular, accurate monthly bills, this holds true, which means where there is inaccurate or non-existent billing and credit control from a municipality (sadly a common occurrence) there is no hope of trying to change consumer behaviour.

Using the City of Johannesburg as an example, the current tariff for domestic use, on a monthly read meter, is depicted in Table 1. This results in an increasing cost of water as shown in Table 2.

From the author's own experience it could be argued that in a house of four people, a consumption of between 10–15 kilolitres per month is not unreasonable.

Whilst it might be argued that an amount of R30.73 plus VAT is unreasonably low and not cost-reflective, when the sanitation charge is taken into account, which is a fixed monthly charge, the amount appears reasonable for the

package of services provided (as per Table 3). In fact, the structure of the tariffs is such that if all other things are equal, there is a clear incentive to opt for a sanitation solution that can be managed on site by the household. In the next article this point will be debated further.

The question that needs to be asked is why, when there is such a clear financial incentive to conserve water, is it so common to come across households using in excess of 50 kl per month, and why do they not take action to find the cause and rectify the problem? Since often this is water that the consumer is paying for, but not using in any effective manner, it is the equivalent of putting R6 000 of your money annually on the braai, burning it and not even having cooked meat to enjoy afterwards.

The author will argue, from his personal experience of helping friends and colleagues, that this situation is due to a combination of bad practice at househoulds, municipalities and within the plumbing profession. The key to unlocking this problem is empowerment at household level to get consumers to proactively manage their water consumption.

FIVE CASES IN POINT

Although the general principle is that any leak before the water meter is for the municipal account and anything after is for your account, Lunghisa's experience shows that this is not always the case.

Lunghisa's water bill suddenly tripled, but she swore that there were no leaks in the house. She went to the municipality to complain, but they simply sent her away saying that it was her meter reading. After another month of high water bills Lunghisa then noticed that water was appearing on the surface next to her drive, which was a sure sign that there was a leak in the pipe. The problem, however, was a leak in the meter box between the meter dial and the stop cock. So, although the water had passed the dial, it was still in the municipality's property. Armed with that information Lunghisa was then able to go back to the municipality, and get them to come out and replace the meter.

This can be contrasted with another colleague who also received a high water bill and swore that it could not be his as the meter was covered and had not been read. He had already lodged a complaint with the municipality before requesting assistance from the author. The first check, which involved uncovering the meter and checking the reading against the bill, showed that it was in fact his bill. A quick check also revealed that the outside toilet was overflowing, and then he remembered leaving the hose on overnight, an act which can easily result in 10 kl flowing through the meter. Now put yourself in the municipality's position of trying to distinguish between legitimate queries and where basic checks have not been done.

Toilets overflowing is a very common problem, and in a third colleague's house it was found that all three toilets were overflowing. In fact, one could clearly hear the toilets when entering the house, but the colleague was oblivious to what the sound meant.

A fourth colleague was more aware and realised that his water consumption was too high, so he called out a plumber to try and find the leak. The recommendation from the plumber was to replace the entire pipe from the meter to the house – a distance of over 40 m. The cost was so high that he balked at this. However, when the author was at the house he noticed what appeared to be an overflow pipe continually running; according to this colleague, the plumber was unable to trace where it was coming from. It took five seconds to work out it was the overflow from an old-fashioned toilet, and fix it. Thus armed with the knowledge to trace leaks the colleague was also able to pinpoint a second leak outside the kitchen door. In the end the result was a much reduced water bill without the need to replace the entire pipe.

The fifth example is a complex comprising 60 units in Montgomery Park, Johannesburg. The author was approached after the monthly bill in the complex went from R33 000 per

Table 1: The current water tariff for domestic use in Johannesburg, on a monthly read meter

Tariff	Per kilolitre, per erf, per month (excl VAT)
First 6 kl	R0.00
In excess of 6 kl up to 10 kl	R6.18
In excess of 10 kl up to 15 kl	R9.97
In excess of 15 kl up to 20 kl	R14.06
In excess of 20 kl up to 30 kl	R18.46
In excess of 30 kl up to 40 kl	R19.67
In excess of 40 kl	R24.21

Table 2: Cost of water

Cost of water	Excl VAT
6 kl	R0.00
10 kl	R24.72
15 kl	R74.57
20 kl	R215.17
30 kl	R399.77
40 kl	R596.47
50 kl	R838.57

Table 3: Water and sanitation bill from the City of Johannesburg

Johannesburg Water Water & Sanitation	VAT 4270191077	Sub - Total	Total Amount
(Reading period = 2015/01/27 to 2015/02/23 = 28 days)			
Meter readings and consumption: Meter no GRA8166 start reading 177.000			
and end reading 187.000 = 10.000 KL - Actual Reading			
Daily average consumption 0.357 KL			
Charges for 10.000 KL are based on a sliding scale for a 28 day period			
Step 1 5.520 KL @ R 0.0000 Step 2 3.679 KL @ R 6.1800 Step 3 0.801 KL @ R 9.9700		30.73	
Extended Social Package Grant		0.00	
Sewer monthly charge based on Stand size 495 m2 (Billing Period 2015/03)		229.78	
VAT: 14.00%		36.47	296.98

month (R550 per household) to R96 000 per month (R1 600 per household). Here it was clear that the reticulation was rotten, as the unmetered fire reticulation (non-revenue water to Johannesburg Water) had already required extensive repairs. One year later neither Johannesburg Water nor the complex had taken any action, resulting in major losses for both of them. Given the demographics in the complex, a monthly bill of R5 000 per month would have been reasonable, meaning that the complex over the past 12 months had paid out over R1 million for water that had passed through its meter, but that had not been used for any constructive purpose whatsoever.

LESSONS DRAWN

A number of lessons can be drawn from this last example.

Firstly, if the households in the complex were happy to pay R550 per month, then increasing tariffs to reflect the true cost of supplying water should not be difficult if it is matched with a reduced consumption (i.e. the monthly bill remains the same). Once consumers understand what a reasonable consumption is and how they can trace leaks themselves, it will empower them in their dealings with the municipality and plumbers. From the municipality's point of view, empowered consumers will assist them in providing an efficient and effective service.

Secondly, in order to reduce abstraction from the resource, both non-revenue (the fire reticulation) and revenue water (the domestic reticulation) need to be targeted. In the case of the Vaal River System a 15% reduction is the target and it can be concluded, from all the above examples, that this can easily be achieved.

Thirdly, a more proactive approach needs to be taken by the water service providers in helping consumers fix the leaks. The problem in the complex was that the rules governing the Body Corporate did not allow them to borrow the money to fix the problem. However, if a special tariff could be introduced by Johannesburg Water for the period it takes to pay back the cost of fixing the problem (in the above case combining both the fire and domestic reticulation into a single reticulation with individual household metering) then everyone would gain.

However, all the other examples show that, at a household level, once the household was aware of how to resolve the problem it was relatively easy to fix, either themselves or with their municipality. The primary driver in all cases was money. So, for behaviour change to happen, municipalities must accurately bill their consumers on a monthly basis, enforce credit control and, if possible, notify the consumer when consumption patterns change.

How many more R6 000 braais must South Africans have before they can enjoy the meat? ◻

... once the household was aware of how to resolve the problem it was relatively easy to fix, either themselves or with their municipality. The primary driver in all cases was money. So, for behaviour change to happen, municipalities must accurately bill their consumers on a monthly basis, enforce credit control and, if possible, notify the consumer when consumption patterns change.

LOOKING FOR LEAKS

Step 1: Basic steps

1. In an average home with no excessive garden watering or big pool to top up, consumption (including teenagers who stand in the shower for an hour) should be between 3–4 m^3 per person per month (divide total consumption on your bill by the number of people in the house).
2. Open the meter and check that the meter reading corresponds, or almost corresponds to that on your bill. If it is widely out then it is probably a billing problem. If they almost correspond then it is a problem after the dial.
3. Check that the meter number (if you can see it) is the same as that on your bill. If it doesn't correspond, there is a big problem at the municipality.
4. If you have a new meter where the shutoff valve is incorporated in the meter box, turn it off. If the meter still runs then the leak is in the meter box and the municipality must replace the meter and refund above the average consumption.

Step 2: Checking for leaks

1. Go through the house and check that taps are not dripping or toilets overflowing (very common). On old toilets it is easy to see if the rubber seal is old and hard. With modern mechanisms, lift the ball valve (or equivalent) and if the toilet still fills, the seal needs replacing – this will cost R5 assuming you know how to do it.
2. If not leaking taps or toilets, then it is leaking pipes, or behaviour (that you don't know about). Do the following checks:
   a. Look for any damp patches on the walls, particularly where you know there are pipes going to taps, toilets, etc.
   b. Put your ear to the pipes where they come out of the ground/walls, etc, and listen for any sounds (make sure dripping taps and toilets are fixed first). If you can hear sounds then you probably have a leak. The louder the sound the closer you are to the leak, where at least you can direct your efforts to this general area.
   c. If still nothing, then the leak is probably between the meter and the house or it is behaviour. Leave the shutoff valve at the meter open and close the valve (if you have one) where it goes into the house. If the meter still runs then it is somewhere along the pipe. If the pipe is easily accessible (i.e. not buried under paving, etc) then by opening up every few metres you will eventually find the leak. If the pipe is inaccessible at that point it is often cheaper to lay a completely new pipe than trying to find a leak.
   d. If you close off and the meter doesn't run, then it is probably behavioural. Monitor the meter every day at the same time and if there is a spike in the daily reading, investigate. A hosepipe left on all day has been known to cause a 10 m^3 jump in readings.

Dispute Boards – Dispute Avoidance Role, Part 2

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This is the sixth and concluding article in the series on Dispute Boards[1] (DBs) and is Part 2 on the dispute avoidance role of a DB. It describes the need for dispute avoidance and the avoidance role of DBs in practice. In the prelude to this article (Part 1 of the dispute avoidance role of a DB) the evolution of the dispute avoidance role of DBs and the avoidance provisions found in commonly used standard form conditions of contract and DB rules were discussed. The first five articles (introductory and Parts 1, 2 and 3 on the operation of a DB and Part 1 on the dispute avoidance role of a DB) appeared in the August[2], September[3], October[4] and November[5] 2014, and the January/February[6] 2015 issues of *Civil Engineering*[7].

INTRODUCTION

It was stated in the introductory article[8] on DBs, that the DB is a creature of contract. Accordingly the contract includes the provisions which govern the DB's dispute avoidance role, and also the employer's and contractor's obligations in this regard.

As in previous articles, this article relates to standing DBs rather than *ad hoc* DBs[9], as *ad hoc* DBs are akin to adjudication and therefore do not have the advantages and distinguishing features of standing DBs[10] which, very importantly, enable them to also undertake a dispute avoidance role.

WHY DISPUTE AVOIDANCE?

Dispute resolution

Common to all the traditional models of dispute resolution is that a dispute has already arisen and its resolution is reactive, rather than proactive which would seek to avoid the dispute developing in the first instance.

As early as 1700 the poet John Pomfret (cited in Brown & Marriott 1993) wrote, "*Law-suits I'd shun, with as much studious care, as I would dens where hungry lions are.*"

In 1926, Judge Learned Hand[11] confessed, "*I must say that, as a litigant, I should dread a law suit beyond everything else short of sickness and death.*"

A fundamental prerequisite to a successful project is sound relationships within the owner, contractor and engineer project team.

The emphasis of a rights-based dispute resolution process on pronouncing right and wrong, and naming winners and losers, has a high probability of destroying almost any pre-existing relationship between the parties involved. It is virtually impossible to maintain a civil relationship once people have confronted one another across a courtroom (including adjudication/arbitration).

Aubert points out that adjudication changes simple conflicts of interest into conflicts of value. It is backward looking, actions and events are measured against a normative standard, with one disputant held to be in the right, the other in the wrong. Often the rule is that the loser pays the winner's cost, which accentuates the problem.

It is not only the process in itself, but the preparation and events leading up to an adversarial resolution process that strains relationships in an already complex and often stressful construction project environment.

Bevan (1992) states that litigation is an adversarial system, which even if its theory works in practice, the system has led to the other side being attacked in a way which precludes cooperation, let alone business association.

MacGregor (1994) says that many adjudications may miss the point by failing to meet the real concerns of the parties, thereby dividing the parties even further after one or both loses.

Control

In litigation and arbitration the proceedings are very formal and the parties have a passive role. Professional dispute resolvers more often than not take control of the process. The disputants become bystanders and spectators in their own case.

Disillusionment

Derek Bok, former president of Harvard University and former dean of the Harvard Law School, described the American system of resolving disputes as "*strewn with the disappointed hopes of those who find [it] too complicated to understand, too quixotic to command respect, and too expensive to be of much practical use*".

Harvard law professor Lawrence Tribe adds that the results do not justify the costs: "*Too much law, too little justice; too many rules, too few results*".

Cost

Brown and Marriot (1993) say about the courts in the UK that they are "*indeed like the Ritz hotel, open to all, but in reality only to those who can afford it*". Most people, including small- and medium-sized companies, cannot afford the civil justice system.

Probably the most underestimated financial cost of a high-level dispute resolution process is the cost of being diverted from income-generating productive activities, and the cost of lost opportunities. Inevitably those involved in these processes are also senior managers with concomitant loss/reduction of management leverage while engaged on the dispute.

Delays

Generally dispute resolution is a time-consuming process, often plagued by delays. The culprits include lengthy procedures (e.g. discovery), procedural sparring and point scoring, incompatible diaries of counsel, instructing attorneys, third party triers of fact, the parties themselves, party experts, etc. The list continues …

Clarity of issues

The legal process distorts reality. Not only speed and economy, but the real issues in dispute and the outcome, are obscured for all the parties involved.

Traditional models of dispute resolution

The TCCP of UTRC levels the following criticisms at [alternative] dispute resolution processes:

- Most ADR processes are employed a substantial time after the work has been completed.
- The decisions are often made by persons:
  - who have a limited background in the type of work involved; and
  - who are not familiar with the specific project.
- They start too late, take too long and cost too much.

The commonly known models for dispute resolution include adjudication, arbitration and court action, and it can be said that they (except adjudication in certain circumstances) suffer from the above afflictions.

The reality is that when the resolution processes engage gear, the damage has been done. From then on, it can be said, that it is downhill all the way as the above afflictions take hold. It is true that certain ADR processes are more expedient and cost-effective than others, the DBs being such a process. However, common to all, even mediation which is an interest as opposed

to rights-based process, the dispute has already arisen with concomitant risks and undesirable characteristics.

Is an obvious solution to the problem not then to avoid disputes altogether, as advocated by Sir Michael Latham?

DISPUTE AVOIDANCE BY DBs

While the DB concepts were originally developed with their focus only on dispute resolution (e.g. Dispute Adjudication Boards as used in FIDIC forms of contract), this aspect is now seen as utilising only one part of a DB's potential and is no longer regarded as 'best practice' within the DB world.

Fourie had the following to say at the 2012 Annual DRBF Conference in Sydney, Australia:

"The objective of the DB must be to help the parties communicate in a positive environment so that they avoid a dispute. In our experience the DB process:

1. *Is the most objective, informed, unbiased, fair and just forum that either client or contractor will ever face on any dispute.*
2. *The key is the regular DB meetings, and, if managed well, it is instrumental in helping the parties to avoid hearings (formal or informal), claims, mediation, arbitration and litigation.*
3. *If regular DB meetings are properly implemented from the start and maintained, it will enhance trust and assure good relations between the parties to the project.*
4. *It saves the project money and emotional capital."*

In particular, the *ad hoc* formation of a DB only after a dispute arises on a project, fails to utilise the real and most significant advantage of the DB – the opportunity to avoid the dispute altogether. A 'standing' DB which is in place between the parties from the project outset seems to be the only available ADR process which can deal with 'issues' as they arise, influence the project participants' behaviour, maintain good relationships between the parties and substantially reduce the probability of 'issues' escalating into 'disputes' on the project.

Graham Easton advises that recent information from current projects where a proactive approach has been adopted by the members of the DB, indicates a high degree of acceptance by both owners and contractors. The success of these DBs generally results in a 'best-for-project' outcome for all parties and the completion of large and complex projects within time and at or below budget. Although the DRBF's information in this regard is somewhat subjective, DB members appointed to these projects regularly advise the DRBF that the major part of their working time is now spent on 'dispute avoidance' activities. In some of these projects, recent statistics indicate that no disputes have ever reached the stage of a formal referral to the DB for resolution.

He further advises that the concept of dispute avoidance and prevention has received strong support within the DRBF community worldwide, particularly from the more experienced and sophisticated DB users on major public infrastructure projects. Significantly in many US and Australian projects, DBs are taking more innovative approaches to dealing with potential disputes, and the procedures being adopted reflect a far more proactive role for the DB within the governance structure of the project.

ADVANTAGES OF DISPUTE AVOIDANCE BY DBs

Proactive and cost-effective

While the ADR processes have to some extent ameliorated the issue of high costs, they are, by definition, initiated only after a dispute has arisen (as is the case with formal arbitration or litigation). Accordingly, they are reactive rather than proactive processes, in the sense that they are generally not able to influence the performance and outcome of the project in any positive way. Similarly, since the ADR process inevitably follows after the causal event(s) giving rise to the dispute, there is little or no opportunity for the contracting parties to manage the project in a way which addresses or eliminates such causes.

In contrast, the unique feature of the DB process is that, if properly utilised, it can provide a relatively low-cost, effective means of not only resolving disputes, but more importantly, preventing and avoiding disputes before they arise. This proactive role for DBs clearly differentiates the process from all other conventional ADR processes.

Communication and positive relationships

In addition to the direct benefits in project cost and time which flow from the avoidance of disputes, there are significant intangible benefits provided by the regular meetings between the DB and the senior executives from the project parties. These DB meetings provide a forum, outside the formal contractual regime, for the parties to review their performance, identify current and future problems, remove road blocks to progress and actively engage with one another in a constructive relationship.

While these collaborative aspects of DBs are important, when the inevitable issues (commercial, technical, legal) arise, it is still necessary to remind project participants that proactive issue resolution by the DB is not and cannot be a substitute for proper contract administration and management.

Further, the parties are ultimately bound by the risk environment and the contract terms and conditions they entered into, however unfavourable they may be. Within these constraints, DBs are frequently able to encourage and assist the contracting parties to deal pragmatically with the inevitable lack of perfection in any complex set of contract conditions and specifications.

Familiarity with project and participants

Because the DB is appointed early in the contract, regularly receives documents from the parties as to project status and the like, and regularly visits the site, it is familiar with the project and the parties and would have established a positive relationship with the parties. It is thus in a unique position to deal with issues before they become disputes.

Advice and opinions

Various authors[12] have expressed opinions as to the advantages of informal advice and opinions (one of the dispute avoidance tools and techniques) by the DB. These include:

- Parties are assisted in identifying the issues in dispute.
- By articulating the issues, the parties themselves improve their knowledge/understanding of the matters pertaining to the potential dispute.
- The mere fact of preparing presentations to the DB (for an opinion), may enable the staff on site to resolve their differences.
- It leads to informal discussion between the parties, which can lead to resolution.
- Informal unbiased advice/opinions from DB members, who have the confidence of the parties, can explain/clarify/put issues in a different light/perspective, which can facilitate resolution.

- If advice/opinions are given prior to involvement of legal representation it can avoid potential procedural issues, legal point scoring and potential high-jacking of the process.
- It avoids disagreements becoming disputes.
- A proactive approach by the DB has the added advantage of positively influencing project participant behaviour and improving relationships.
- If the process does not depend on decisions that are binding, there is far less danger of relationships being damaged by a failure to implement or enforce them.

DISPUTE AVOIDANCE ROLE OF DB MEMBERS

Proactive not reactive

Dispute avoidance and prevention requires DB members to take a more 'hands-on' and inquisitorial approach during the regular DB meetings, and generally during the course of the project. Such an approach requires DB members who possess a high level of professional skill and experience (together with a well-developed sense of the requirements for independence), impartiality and procedural fairness.

For example, at the first DB meeting with the parties, the DB members should ensure that they:

- Educate first-time users in relation to the role of the DB and the procedures which the DB intends to adopt;
- Emphasise that the DB's role is an integral part of the project governance and management;
- Stress that the DB's objective is to ensure a successful and 'best-for-project' outcome for all parties; and
- Seek and obtain the parties' agreement to include DB operating procedures which are specifically directed at dispute avoidance[13].

At all subsequent DB meetings, the DB members should continue to:

- Work hard to build and maintain the DB's relationship with the parties, instilling trust and confidence in the DB process;
- Focus on the early identification of issues arising within the project before they become disputes;
- Encourage a collaborative approach between the parties in relation to issue resolution; and
- Ensure that 'difficult' issues are brought forward and addressed rather than being suppressed and left to fester.

PRACTICAL STEPS FOR DISPUTE AVOIDANCE

There are several practical aspects of a DB's work which need to be considered when maximising the DB's potential for dispute avoidance and prevention.

Early involvement of the DB

It is important that the tripartite agreement between the DB and the contract parties should be finalised and signed concurrently with the execution of the main contract (at contract commencement) or at least within 56 days thereafter. By this means, the DB is appointed and empowered from the start of the project. Experience in this regard has clearly demonstrated that the early involvement of the DB is extremely important when the primary object of the DB is dispute avoidance. However, early appointment in itself is of little real benefit unless it is accompanied by several early meetings of the DB with the contract parties, in order to set in place the

procedures to be followed and to encourage effective relationship building with the parties.

Further, experience with 'design/build' contracts has also shown that the design phase of the project is the single aspect which generates potentially serious differences of opinion, and, if not resolved before construction commencement, can lead to long-running and serious disputes. The design phase of a major design/build contract may extend over six to twelve months before any significant construction work commences. It is nevertheless critical for the DB to be fully involved through this early period. The DB would then be in a position to assist the parties in any areas of ambiguity, inconsistency or reasonably interpreted requirements for performance or design standards, enabling pragmatic solutions to design-related issues.

Adjustments to DB procedures

An important proactive step for a newly-formed DB is to review (or in some cases, draft) the DB procedural rules (usually provided in the tripartite agreement) which will apply to the project. Even when the DB is purportedly bound by a set of standardised procedures (such as in the FIDIC form of contracts)[14], there are significant benefits to be gained by discussing and, if appropriate, adjusting the DB procedures for that particular project with the agreement of the parties. This is a good way of introducing a proactive *modus operandi* into an otherwise silent set of DB procedural rules. It also enables the DB, at the start, to establish a cooperative relationship with the contracting parties and, where necessary, to define its role in influencing the project participants' behaviour.

'Without Prejudice' status of DB meetings

It is, of course, a necessary part of the DB process for a DB's formal recommendation or decision (in relation to a dispute) to be issued 'with prejudice', so that it might be relied upon by the parties within the framework of their contract. It may also be required as evidence in any subsequent proceedings. However, it is uncommon for there to be any express contractual provision which defines the status of other DB-generated documents, such as DB meeting minutes, reports prepared for the DB or notes of discussions between the parties and the DB. Thus an issue sometimes arises as to the nature and legal status of this documentation.

Further, at routine DB meetings, the 'best practice' protocol now being used by many DBs is to actively encourage both contracting parties to speak freely and to frankly discuss current issues that are foreseen. The parties provide a detailed report (usually jointly) to the DB on the status of the project, site work, programme claims, delays, disputes, matters of concern, potential issues and the like. The DB could also request one or both contracting parties to provide a specific report or position paper on a particular issue, either out of session (copied to the other party) or at the next DB meeting.

To encourage frankness and openness in all these communications between the parties and the DB, international experience is that such DBs now often implement a procedure whereby the information and documents noted above, as well as the reports of the DB site visits, are afforded a 'privileged' or 'without prejudice' status. This status does not, of course, apply to documents produced by or exchanged between the parties in the normal course of business, nor to claims or disputes that are formally referred to the DB for a decision. Nevertheless, internationally there is little doubt that the 'without prejudice' umbrella under which the DB meetings are conducted is a significant contributor to the dispute avoidance objectives and success of the DB.

Other steps towards dispute avoidance

Other important protocols and procedures which have been adopted by DBs, leading to successful dispute avoidance and prevention, include:

- Ensuring that the 'right' people from each party attend all DB meetings – in this regard, the attendance by senior 'off-site' executives is an important requirement;
- Implementing an issue-tracking procedure, whereby issues identified by the parties (or the DB) are specifically listed, actioned and reported upon at each DB site visit;
- Calling for special reports or presentations in relation to specific, identified or potential risk areas such as the design, the specifications, the construction programme or the contractor's planned methodology;
- Acting as a facilitator of party meetings/workshops, convened to discuss or resolve particular issues; and
- The use of an advisory opinion from the DB as a means of providing an independent, expert view on an issue for consideration and negotiation by the parties, before the issue escalates into a dispute and formal referral to the DB for a decision.

CONCLUDING REMARKS

As evidenced above, there is little doubt that avoiding disputes makes good commercial and business sense. Standing DBs are in a unique position to successfully undertake a dispute avoidance role. The members of such DBs are to be carefully chosen to

ensure they subscribe to the avoidance role of DBs, and for their skills and experience in this regard.

NOTES

1. A generic term denoting various types of dispute boards.
2. Introductory.
3. DB Operation, Part 1 – appointment of the DB members, composition of the DB, its powers, remuneration and termination.
4. DB Operation, Part 2 – the site visits, site visit meetings and reports and activities between site visits.
5. DB Operation, Part 3 – the powers of the DB, formal referral of disputes to the DB, hearings and DB decisions and their enforcement.
6. DB Dispute Avoidance Role, Part 1 – evolution of the dispute avoidance role of DBs and the avoidance provisions found in commonly used standard form conditions of contract and DB rules.
7. Van Langelaar, A 2014. Dispute Boards – An Introduction. *Civil Engineering*, 22(7): 72–73; Dispute Boards – Operation, Part 1, *Civil Engineering*, 22(8): 60–64; Dispute Boards – Operation, Part 2, *Civil Engineering*, 22(9): 52–55. Dispute Boards – Operation, Part 3, *Civil Engineering*, 22(11): 62-66. Dispute Boards – Avoidance Role, Part 3, *Civil Engineering*, 23(1): 69–74.
8. Van Langelaar, A 2014. Dispute Boards – An Introduction. *Civil Engineering*: magazine of the South African Institution of Civil Engineering, 22(7): 72–73.
9. A DB which is only called on when a dispute has arisen and the matter is to be referred to adjudication. The *ad hoc* DB may be appointed at the outset or once the dispute has arisen, subject to the provisions of the relevant contract.
10. See Van Langelaar, A 2014. Dispute Boards – An Introduction. *Civil Engineering*, 22(7): 72–73.
11. A United States judge and judicial philosopher.
12. For example Nigel Grout in his paper "Obtaining Informal Advice and Opinions from the Dispute Board", presented at the 2nd DRBF Regional Conference held 10–11 June 2010, Bucharest; Totterdill, B W 2006. FIDIC users' guide, *A practical guide to the 1999 red and yellow books*. London: Thomas Telford; Easton, G 2014. The Role of Dispute Boards in Dispute Avoidance. A paper delivered at the DRBF Regional Conference and Workshop 27–28 February, Johannesburg.
13. Which would include a protocol for seeking and providing advice and opinions.
14. Which, for example, does not include procedures for seeking and providing advice and opinions, although provided for in the contract.

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SAICE Training Calendar 2015

Course Name	Course Dates	Location	CPD Accreditation Number	Course Presenter	Contact
GCC 2010	15–16 July 2015	Cape Town	SAICEcon13/01359/16	Neville Gurry	cheryl-lee@saice.org.za
	22–23 July 2015	Midrand			
	25–26 August 2015	Port Elizabeth			
	22–23 September 2015	Bloemfontein			
	29–30 September 2015	Midrand			
	7–8 October 2015	Mafikeng			
	14–15 October 2015	Durban			
Technical Report Writing	25–26 May 2015	Bloemfontein	SAICEbus12/01067/15	Les Wiggill	cheryl-lee@saice.org.za
	16–17 July 2015	Midrand			
	3–4 August 2015	Durban			
	24–25 August 2015	Cape Town			
	3–4 September 2015	Port Elizabeth			
	22–23 October 2015	Midrand			
Practical Geometric Design	7–11 December 2015	Midrand	SAICEtr13/01418/16	Tom McKune	dawn@saice.org.za
Reinforced Concrete Design to SANS 10100-1:2000	3 June 2015	Durban	SAICEstr15/01674/18	Greg Parrott	cheryl-lee@saice.org.za
	23 June 2015	Bloemfontein			
	30 June 2015	Upington			
	2 July 2015	Midrand			
	12 August 2015	Kimberley			
	16 September 2015	Port Elizabeth			
	27 October 2015	Midrand			
Structural Steel Design Code to SANS 10162:1-2005	2 June 2015	Durban	SAICEstr12/01158/15	Greg Parrott	cheryl-lee@saice.org.za
	22 June 2015	Bloemfontein			
	29 June 2015	Upington			
	1 July 2015	Midrand			
	11 August 2015	Kimberley			
	15 September 2015	Port Elizabeth			
	26 October 2015	Midrand			
Business Finances for Built Environment Professionals	28–29 May 2015	Midrand	SAICEfin15/01617/18	Wolf Weidemann	dawn@saice.org.za
	12–13 November 2015	Midrand			
Handling Projects in a Consulting Engineer's Practice	25–26 May 2015	Midrand	SAICEproj15/01618/18	Wolf Weidemann	dawn@saice.org.za
	9–10 November 2015	Midrand			
Leadership and Management Principles and Practice in Engineering	9–10 September 2015	Durban	SAIMechE-0543-02/15	David Ramsay	dawn@saice.org.za
	30 Sept–1 Oct 2015	Cape Town			
	14–15 October 2015	Bloemfontein			
	28–29 October 2015	Port Elizabeth			
	4–5 November 2015	Midrand			
Water Law of South Africa	10–11 June 2015	Cape Town	SAICEwat13/01308/16	Hubert Thompson	dawn@saice.org.za
	12–13 August 2015	Durban			
	9–10 September 2015	Bloemfontein			
	14–15 October 2015	Upington			

Course Name	Course Dates	Location	CPD Accreditation Number	Course Presenter	Contact
The Legal Process dealing with Construction Disputes	20–21 May 2015	Kimberley	SAICEcon13/01368/16	Hubert Thompson	dawn@saice.org.za
	28–29 July 2015	Midrand			
	25–26 August 2015	Upington			
	16–17 September 2015	Durban			
	21–22 October 2015	Port Elizabeth			
	4–5 November 2015	Bloemfontein			
Earthmoving Equipment, Technology and Management for Civil Engineering and Infrastructure Projects	22–24 July 2015	Midrand	SAICEcon12/01177/15	Prof Zvi Borowitsh	dawn@saice.org.za
	27–29 July 2015	Durban			
Sanitary Drainage Systems for Buildings	13 July 2015	Durban	SAICEwat12/01103/15	Vollie Brink	dawn@saice.org.za
	20 July 2015	Port Elizabeth			
	17 August 2015	East London			
	25 November 2015	Midrand			
Water Supply and Drainage for Building Systems	14 July 2015	Durban	SAICEwat13/01393/16	Vollie Brink	dawn@saice.org.za
	21 July 2015	Port Elizabeth			
	18 August 2015	East London			
	26 November 2015	Midrand			

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Candidate Academy Course Schedule 2015

Course Name	Course Date	Location	CPD Accreditation Number	Course Presenter	Contact
Road to Registration for Candidates	28 May 2015	Johannesburg	CESA-357-04/2016	Allyson Lawless	margie@ally.co.za
	30 July 2015	Durban			
	2 September 2015	Johannesburg			
Road to Registration for Mature Candidates	6 July 2015	Durban	CESA-484-01/2017	Peter Coetzee	margie@ally.co.za
	5 October 2015	Cape Town			
	7 December 2015	Durban			
Road to Registration for Mature Candidates	20 July 2015	Johannesburg	CESA-484-01/2017	Rob du Preez	margie@ally.co.za
	8 October 2015	Johannesburg			
	3 December 2015	Johannesburg			
Basic Contract Administration and Quality Control	22–24 July 2015	Cape Town	CESA-359-04/2016	Theuns Eloff	margie@ally.co.za
	14–16 October 2015	Durban			
Getting Acquainted with Road Construction and Maintenance	22–24 June 2015	Johannesburg	CESA-379-05/2016	Theuns Eloff	margie@ally.co.za
	19–21 August 2015	Durban			
Getting Acquainted with Basic Pressure Pipeline Design	10–11 June 2015	Johannesburg	CESA-376-05/2016	Dup van Renen	margie@ally.co.za
	5–6 November 2015	Cape Town			
Getting Acquainted with GCC2010	4–5 June 2015	Cape Town	CESA-377-05/2016	Theuns Eloff	margie@ally.co.za
	6–7 August 2015	Johannesburg			
	5–6 November 2015	Durban			
Getting Acquainted with Sewer Design	7–8 September 2015	Durban	CESA-378-05/2016	Peter Coetzee	margie@ally.co.za
Getting Acquainted with the Fundamentals of Tendering and Procurement	9–10 July 2015	Cape Town	CESA-395-06/2016	Theuns Eloff	margie@ally.co.za
Getting Acquainted with Geosynthetics in Soil Reinforcement	2–4 June 2015	Durban	SAICEgeo14/01627/17	Edoardo Zannoni	margie@ally.co.za
	7–9 October 2015	Cape Town			

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