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Isivili Enjiniyereng

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ON THE COVER

The installation of driven tube piles is always challenging, but doing so from a floating barge with spud legs in often choppy seas requires expertise par excellence. Franki Africa admirably meets the challenge in the Port of Richards Bay

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Block 19, Thornhill Office Park,
Bekker Street, Vorna Valley, Midrand
Private Bag X200, Halfway House, 1685
Tel 011-805-5947/48, Fax 011-805-5971
http://www.civils.org.za
civilinfo@saice.org.za

EDITOR
Verelene de Koker
vdekoker@saice.org.za
Tel 011 805 5947/8, Cell 083 378 3996

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ADVERTISING
Barbara Spence, Avenue Advertising
barbara@avenue.co.za
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ON THE COVER

Water water everywhere

Franki has developed a reputation for being able to handle demanding piling jobs, and few jobs could be more demanding than the R213 million Transnet-designed Berth 208 at the Port of Richards Bay. Although driven tube piles are commonly used in a marine environment, their installation is always challenging. But doing so from a floating barge with spud legs in often choppy seas requires expertise par excellence

"WORKING IN THE OCEAN from a floating barge is one thing," says Franki technical director Gavin Byrne, "but add to that the geology of the Richards Bay area, and the difficulties of working there are compounded quite significantly."

He explains that the geological structure is characterised by recently deposited sediments, which have presented significant challenges to many of the Richards Bay structures that are located in the old alluvial river channel in which the harbour is built. He says, however, that Franki's tried and trusted expertise in driven tube piles for marine applications has enabled them to overcome these difficulties and provide their client with a world-class, cost effective piling solution.

All in all, Franki is installing 78 driven tube piles – five 800 mm diameter and seventy-three 700 mm diameter – which are cored 15 m below the seabed level, reinforced and concreted to the cut-off level of +2,05 m chart datum. The contract is worth more than R30 million. The piles are open-ended, top-driven tube piles that not only enhance the ability to penetrate hard horizons, but also help to speed up the installation process.

The process requires two Ajax cranes, which are situated on the barge, to install a guide frame first, i.e. a steel platform fixed to temporary tube piles, and once the frame is firmly installed, to pitch the permanent piles into position in the frame. An I.C.E. vibrating hammer is then placed on the pile to vibrate it down until minimal penetration is achieved.

At this point a driving helmet is placed on the pile head and, using a Franki 7-ton hydraulic hammer, the pile is driven to the required set, ensuring each pile has an adequate load-bearing capacity. The pile bore is then cored out to the required depth, and the pile is plugged and the pile shaft pumped dry. The pile is then reinforced and concreted from the barge.

At each row of piles, or bent, there are between five and eight vertical and raking piles.

Franki contracts manager, Gavin Hutton, illustrates the difficulties of working from this barge. "As this is not a jack-up barge, but a floating barge with spud legs and anchors to hold it in position, it is susceptible to any water movement. Logistics therefore become fairly complicated. All materials, including all cored materials (sand and clay) removed from the piles, which have to be taken to land and disposed of, are transported with a smaller supply barge that has to be manoeuvred with a tug as it has no motor of its own."

In spite of the challenging working conditions, the contract is running smoothly. The contract, awarded to Franki by Stefanutti Stocks, commenced in August 2008 and will be completed in July 2009.

PILING FOR MARINE STRUCTURES

In the latest edition of Franki's *A Guide to Practical Geotechnical Engineering in*

Southern Africa (Fourth Edition 2008) it is stated that piles are used for many marine applications, either as foundations to carry structural loads in poor ground conditions, or as structural members that provide both a foundation and structural element to jetties, quays, and dolphins.

According to this guide, the most suitable type of driven pile for marine structures is the tube pile used in the Richards Bay contract. This can be installed with a thin-walled casing (6 mm to 10 mm) and concreted and reinforced *in situ* to provide the permanent structural element, with a tube providing a temporary lining. The tube pile can also be installed with a thick-walled casing (10 mm to 40 mm) where the steel casing forms the primary structural load-bearing element.

Byrne explains that driven tube piles are widely used in a marine environment since they have the advantage of easily providing a supporting member to structures located in the sea. Careful assessment of the corrosive environment must be undertaken to ensure that the durability requirements are met. He adds that cathodic protection or durable coatings are generally essential to ensure long-time durability. "A wide range of pile diameters, from 300 mm for minor structures to 1 500 mm for heavy marine applications, can be installed where poor soil conditions underlie the site."

DRIVEN TUBE PILES

Driven tube piles are generally not used that extensively, mainly due to the high cost of the steel tubes. There are, however, situations where the positive features of the system outweigh the costs. The positive features can be summarised as follows:

- An extensive range of pile sizes is available.
- The system can achieve considerable depths (more than 60 m in a suitable profile).
- The pile is permanently cased and thus ideal for river and marine work.
- The pile can be installed in limited headroom.
- The pile can be installed in areas with very difficult access.
- The shaft is cast in the dry, so quality control is good.
- Noise levels are low.

ABOUT RICHARDS BAY

The Port of Richards Bay is situated approximately 160 km north of Durban. The port serves mainly the Gauteng region, Mpumalanga, and harbour-bound industries in the immediate vicinity of the port. Portnet operates two general cargo terminals (the Combi and Bulk Metal Terminals), and also the Dry Bulk Terminal. These three terminals have a total of twelve berths. Richards Bay also has six privately-operated berths. One is operated by the Richards Bay Bunker Terminal and Richards Bay Bulk Storage. The others are operated by the Richards Bay Coal Terminal (RBCT), the largest single coal terminal in the world.

Note

The technical information in this article was sourced mainly from Franki's *A Guide to Practical Geotechnical Engineering in Southern Africa* – Fourth Edition 2008.

ENQUIRIES

Gavin Byrne
Franki Africa (SA) (Pty) Ltd
011 531 2700
info@franki.co.za

1 Supply barge bringing tube pile to 'site'
2 Vibrating guide frame into position
3 Surveying pile position onto the guide frame
4 Stages in jetty pile and pile cap construction

Text Verelene de Koker
Editor
vdekoker@saice.org.za

PROFILE

Ronnie Scheurenberg: Living life to the full

By his own refreshingly candid admission Ronnie Scheurenberg, specialist geotechnical engineer, is not good at multi-tasking. He says he becomes agitated when confronted with too many tasks at the same time. However, when perusing the formidable list of projects that he has successfully completed in his career, one wonders how one person could have achieved all of that in one lifetime. The answer, says this 25th recipient of the prestigious South African Geotechnical Medal for 2008, lies very simply in doggedly and persistently focusing on one job at a time. Behind Ronnie's ready smile and relaxed demeanour is a down-to-earth, kind and purpose-driven man who gets things done while living and enjoying life to the full

EDUCATION

Born in Port Elizabeth to German-Jewish parents, he matriculated at Marist Brothers in the Friendly City. While still at school it was exposure to the *Archimedes* science magazine that sparked his interest in civil engineering. The idea of contributing to

society in the particular way that civil engineering does, thereby leaving a meaningful legacy, appealed to him. Etching out imaginary engineering plans on his wooden desk at home certainly did not endear him to his mother, but the possibilities presenting themselves from this playing around strengthened his resolve to study civil engineering!

In 1964 he graduated from the University of Cape Town (UCT) with First Class Honours in civil engineering, and was also awarded a UCT Gold Medal. He followed this up with a Master's in geotechnical engineering (soil mechanics and foundations) and water engineering (hydraulics and hydrology) from the University of the Witwatersrand in 1967.

While at UCT he was awarded the Abe Bailey Fellowship in 1963. Student leaders from various universities across South Africa, and across all disciplines, were selected for this award, and were then taken to the United Kingdom on a six-week guided tour which enhanced their understanding of the complexities of Western civilisation, and also offered opportunities for networking with fellow future leaders. At UCT Ronnie had been a leader in student affairs, including being the chairman of the UCT Engineering Society. The opportunities and experiences resulting from this award had a major impact on Ronnie's outlook and philosophy of life at the time.

WORKING CAREER

Apart from a two-year stint with a consulting firm in Israel at the start of his career, Ronnie spent almost all of his 43-year-long working life with Knight Piésold Consulting (previously Watermeyer Legge Piésold & Uhlmann). During this time he gained extensive experience in geotechnical engineering, specialising in mine residue storage facilities, earth and concrete dams, site investigations, laboratory testing (he was manager of Knight Piésold's partly owned subsidiary Civilab for four years in the 1970s), slope stability and geosynthetics. Although he worked in many parts of the world, his main work areas were southern Africa and Peru.

Ronnie was responsible for the planning, design and execution of large and small tailings disposal projects, earthfill and concrete dams, and geotechnical investigations for numerous mining, industrial, township and road developments over the years, but he considers the following projects his most significant (on all these projects he served as project engineer and/or project manager):

1 Ronnie Scheurenberg waiting in the wings at his daughter Karen's wedding in England, May 2008
2 Ronnie Scheurenberg (right) receiving the South African Geotechnical Medal from 2008 SAICE president Johan de Koker, while SW (Schalk) Jacobsz, chairman of the SAICE Geotechnical Division, looks on (photo: *Gavin Wardle*)
3 Ronnie on a site visit to the ERGO tailings dam in Brakpan, April 2005
4 Foskor Southern tailings dam near Phalaborwa, upstream of the Kruger National Park in the distance

ERGO gold tailings disposal facility, Brakpan

Here he was responsible for all the facilities associated with the disposal of up to 96 000 metric tons per day of gold tailings on a 900 ha area. This included feasibility studies, liaison with authorities, site investigations, stability and capacity calculations, design of predeposition earthworks, decants and return water dams, construction contract administration, risk analysis and monitoring of continuing deposition on the Withok and Brakpan tailings dams. Innovative features of the project included tailings deposition through hydro-cyclones using the underflow to construct the dam perimeter wall and a 40-m-high concrete decant tower in the centre of the deposit with remote control of the reclaim water outflow. This project was done for East Rand Gold and Uranium Company, then part of AngloGold Ashanti Limited.

Foskor Limited phosphate and magnetite tailings disposal facilities, Phalaborwa

Ronnie's responsibilities in this project were similar to those for the ERGO project, except that here he had to dispose of up to 60 000 and

5 000 metric tons per day of phosphate tailings and magnetite respectively. His work here furthermore included supervision of the design and construction of ancillary works, and monitoring the continuing deposition on the Selati, Southern and magnetite tailings dams.

Feasibility study for the Southern Peru Copper Corporation, Toquepala

From 1999 until 2004 Ronnie lived in Peru where, among other tasks, he project-managed the investigations and feasibility studies for the deposition of 150 000 metric tons of thickened copper tailings on the Pampa Purgatorio in the Peruvian desert in the foothills of the Andes mountains. Other major projects included tailings, infrastructure and slope-stability developments at mines such as Yanacocha, Antamina and Orcopampa high in the Andes.

Ntimbale Dam, Francistown, Botswana

In this project Ronnie acted as specialist engineer for the Botswana Department of Water Affairs, advising on the technical aspects of the dam construction – the 35-m-high RCC (roller-compacted concrete) dam and spillway, and the 20-m-high CCR (clay core rockfill) dam on the Tati River. He was responsible for the site investigations, the design of the RCC and the CCR dams, as well as for the 23 km access road, and the associated pipelines and water treatment works.

Kayelekera Uranium Mine, Malawi

Ronnie's current focus is on the Kayelekera Uranium Mine in Malawi where he is responsible for tailings storage facilities site selection and conceptual design, water balance, water management and storage dams for the Bankable Feasibility Study and the subsequent detailed design and implementation.

MENTORS AND OTHER INFLUENCES

Ronnie's career spans a period of intense developments within the geotechnical engineering environment in South Africa. One such development, which he believes

5 Antamina tailings dam starter wall, Peru, November 2000

6 The Peru Central Highway – note the steep slopes, posing major stability problems

7 Construction of the Kayelekera lined, contaminated water pond in northern Malawi, 20 km inland from Lake Malawi

8 Spillway of the Ntimbale RCC dam on the Tati River in Botswana, February 2006

is of the utmost importance, is the change in attitude of mining companies, society and government regarding tailings storage facilities – from the previous indiscriminate dumping to a regulated approach in which concern for stability and the environment has become hugely important.

Ronnie considers himself extremely fortunate to have been mentored by the likes of Prof Jere Jennings, Prof Des Midgley, Rob Williamson and Tony Brink.

Prof Jennings, whom Ronnie refers to as the giant father of geotechnical engineering in southern Africa, lectured him at Wits while he was completing his Master's. Prof Jennings' understanding of expansive clays, collapsible sands, and sinkholes in dolomitic areas had a profound influence on the young Ronnie's growth and development. Likewise, during this same period, Prof Des Midgley, the father of South African hydrology, taught Ronnie not only hydrology and hydraulics, but clear and accurate thinking and writing.

After joining Knight Piésold, Ronnie was supervised and mentored by colleagues and friends Rob Williamson, who was a recognised expert in tailings storage facilities, and Tony Brink, considered the father of engineering geology in southern Africa. Tony helped Ronnie to grasp soil profiling, and the importance of geology in the understanding of engineering problems and their solutions.

Throughout his career Ronnie has made extensive use of laboratory testing. In addition to what his aforementioned mentors had taught him, the experience and insights gained during his four years at Civilab (soils and civil engineering testing laboratory) helped him to appreciate the importance of, one, meticulous attention to detail, two, understanding the material and what the test is supposed to do and therefore not necessarily to accept computer-interpreted results at face value, and three, making sure that test samples are representative, irrespective of the level of sophistication during testing.

PROFESSIONAL INVOLVEMENT

Ronnie has been a member of SAICE since 1968 and became more involved with the Institution in the early 1980s, first as secretary and later as chairman of the Geotechnical Division. In this capacity he organised a number of seminars and conferences (over the years he has presented or published close on 30 technical papers himself). He represented the Geotechnical Division on SAICE Council, and was also the chairman of the then Production Committee (forerunner of the current Editorial Panel) dealing with magazine affairs. In 1991 he became a Fellow of the Institution, and was elected vice-president in 1999, but had to decline the position when he was transferred to Peru. He is also a member of the Institution of Civil Engineers (London) and registered with the Engineering Council UK.

Ronnie believes that volunteering time to the activities of learned societies such as SAICE is very important for professional development. He recalls earlier times when involvement in professional bodies was generally accepted by employers, and in fact encouraged, as employers recognised the mentoring role that professional bodies served with regard to their employees. However, in today's frenetically busy, results-driven civil engineering environment, coupled with the current skills shortage, young engineers do not always have time to volunteer their services. Employers unfortunately also often frown on their employees' involvement in such activities. The valuable mentoring role of a learned society such as SAICE is therefore not fully appreciated any longer. Ronnie feels that re-establishing an appreciation of this role would enhance the growth of a culture of responsibility and service delivery among young engineers and their employers.

INTERESTS AND PASSIONS

Ronnie is passionate about civil engineering and loves his job. One of his other passions is travelling, especially to places with historic relevance, and he is grateful for the travelling opportunities that his work has offered him over the years. He particularly enjoyed the time that he worked in Peru, and although he never walked the so-called Inca Trail, his work took him to many interesting Inca-linked sites in Peru. Being appreciative of good food, he also developed a palate for the peculiarly spiced Peruvian dishes. While in South America he also visited Patagonia and marvelled at its remote beauty.

Asked how he communicates engineering concepts in foreign countries to people who do not understand English, he replies that drawings often bridge a

communication gap. Also, quite often basic engineering vocabulary is surprisingly similar in different languages. Of course it does help that, apart from his home language, English, Ronnie is competent in Spanish, German, Hebrew and Afrikaans!

His other interests include reading (at the time of this interview he was reading *Fugitive Pieces* by Anne Michaels), theatre, classical music and art films, and maintaining contact with his two daughters, Karen and Lee Erica, who live in the UK.

Ronnie's philosophy is to help make the world a better place to live in, and to recognise that every person has value and therefore deserves to be treated with respect. In support of this philosophy, he currently serves on the Board of Trustees of the Rena Le Lona ('we are with you') Creative Centre for Children in Diepkloof, Soweto. This centre aims to develop deprived children by encouraging emotional expression through the medium of art (painting, music, etc). The children are also taught life and educational skills and empowered to eventually earn a living.

Back to civil engineering – Ronnie has over many years been involved in and thoroughly enjoyed training and mentoring students and young engineers from a variety of backgrounds. In the mid-1980s, for example, he was involved in a schools reinforcement programme – the Protech project – whereby civil engineers offered extra classes in mathematics and science to black children on Saturday mornings. This programme helped many a student realize his or her dreams.

CONCLUSION

For a man who is not comfortable with multi-tasking, Ronnie Scheurenberg certainly gets a lot done, not the least of which is managing to lead and enjoy a balanced life filled with fun, friends, work satisfaction and service to the less fortunate. In today's busy life that is indeed an inspiring feat! ◻

9 Ronnie at the Perito Moreno glacier in Patagonia, February 2004

10 Resting on a dugout at the edge of Lake Malawi, June 2007

11 Ronnie with his two daughters, Karen (left) and Lee Erica, at Karen's wedding in May 2008

Text: **Patrick Beales**
Associate Geotechnical Engineer
Kantey & Templer Consulting Engineers
pbeales@kanteys.co.za

Remediation of cut slopes and retaining structures in Huis River Pass

BACKGROUND

The Huis River Pass has a long history and a geological beauty that was first opened to the public in 1897 by the Divisional Councils of Ladismith and Calitzdorp. As with most of South Africa's older mountain passes, the cut rock slopes have deteriorated with time, increasing the frequency of rockfalls and the need for road closures.

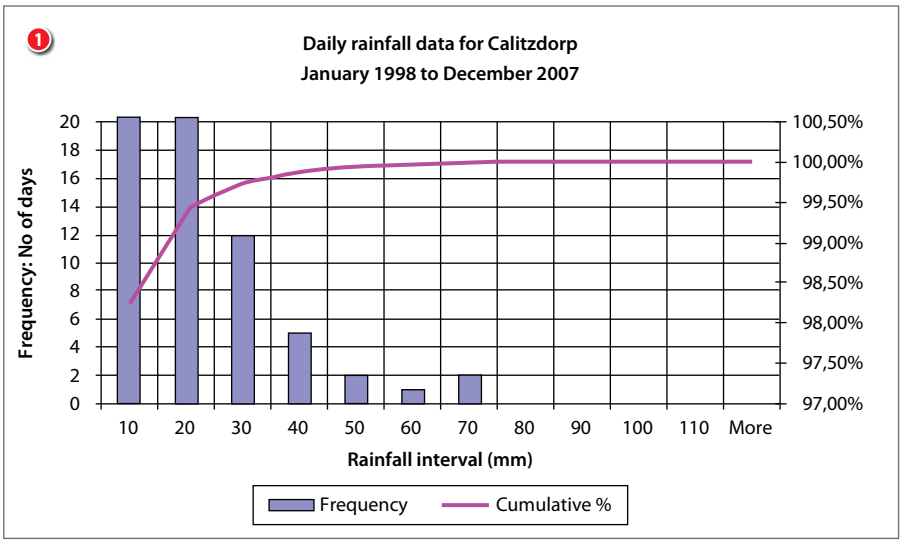
In the early 1960s, a new surfaced pass was designed by Kantey & Templer Consulting Engineers. Mr Basil Kantey, one of the founding members of the firm, played an integral part in the selection of four areas along the route where reinforced concrete retaining structures were constructed.

Nearly 50 years later, the Western Cape Provincial Administration's Roads Branch appointed Kantey & Templer Consulting Engineers (Pty) Ltd and the contractor Penny Farthing (Pty) Ltd to undertake remediation work behind the retaining walls and on selected cut rock slopes in the pass. The contract was specifically aimed at remediation, but Mother Nature ensured that the work moved towards a stabilisation application.

PROJECT DESCRIPTION

Huis River Pass is situated between Ladismith and Calitzdorp in the Klein Karoo. The rock formations in the pass are characterised by folded and contorted phyllite rock of the Kango Group. The phyllites proved to be problematic and unpredictable in the presence of unseasonal rainfall events in November and December 2007. The phyllitic rock masses are intensely folded in this area, making the orientation of bedding or jointing planes liable to slip failures and, in some localised areas, significant wedge failures.

Another important aspect of the cut rock slopes are the crests of the slopes; these typically comprise coarse transported soil (colluvium) that consists of silty phyllitic gravel and boulders of varying sizes. In some localised areas the orientation of the rock was such that varying rock masses had become detached or slid from the original bedding planes, creating oversteep crests of variable thickness. Depending on the gradient of the cut rock slope and the slope height, boulders



1 Daily rainfall data for Calitzdorp
2 Rock debris behind retaining wall (September 2004)
3 Retaining wall remediated (October 2007)

detaching from the crest were prone to a sliding, tumbling or bouncing action.

Mr Basil Kantey also recorded this unpredictable rock behaviour and his approach with regard to the retaining walls was to let the rockfalls occur and to 'contain' falling rock debris behind the walls. This 'containment' idea was again applied and modified in the form of a drapery mesh system. The remedial work behind the retaining walls involved removing the rock debris to recreate the catch reserve.

The remediation work was undertaken using well-managed and experienced barring teams, and by introducing a mesh drapery system (Maccaferri Steelgrid MO) in potential high-risk rockfall areas. The rockfall areas were rated according to the risk by the design team, as well as by the often unpredictable and unforeseen assistance of Mother Nature.

The main aim of the drapery mesh system was to control the descent of falling rock debris, irrespective of the random orientation of rock planes, the varying degrees of folding or the weathering grade of the rock. The fact that the mesh does not unravel if some wires are broken enabled a safe and adaptive system to be installed on the slopes.

PROBLEMS ENCOUNTERED

The main trigger mechanism associated with nearly all slope failures is the introduction of water to the slope. During November 2007, Huis River Pass received more than 100 mm of rainfall in one day. It is interesting to note that in the previous ten years of rainfall history in the area, a maximum one-day rainfall figure greater than 50 mm was recorded on only three occasions.

The November rainfall triggered significant rockfalls as a result of the water 'lubricating' bedding planes and jointing systems on the rock slopes. This saturation of the different planes and their random orientation meant that theoretical modelling and consideration were of little consequence, essentially implying that every rock slope had to be assessed individually (in a localised manner). The high rainfall event also oversaturated the crest material which, owing to the high

silt content, tended to remain saturated for several days and was continually recharged from higher lying areas. Upon drying, the crest material tended to "re-cement" itself in the upper exposed zones, thereby trapping the moisture under a cemented crust.

During the 2007 year-end shutdown, the pass was again subjected to another 100 mm of rainfall in a single day. This second rainfall event resulted in the catastrophic collapse of existing cut rock slopes that were already oversaturated and 'lubricated' by the November rainfall event. The daily rainfall data for the period January 1998 to December 2007 recorded at Calitzdorp are shown in Figure 1.

Although many rockfalls occurred in the pass after the November and December 2007 rainfall events, the two major rockfall areas in the pass were at the main view site and at retaining wall 4. These are discussed below.

It is interesting to note that from a modern perspective, concrete retaining walls of the type used in this rock-protection application are considered to be unfavourable and unsightly structures, and it is generally only the structural engineer or the devoted engineering geologist who would take an interest in the material behind the retaining walls in such an environment. During the contract and before the November 2007 rainfall event, approximately 8 000 m^3 of rock debris was removed from behind the retaining walls. Photos 2 to 4 illustrate the effectiveness of the retaining walls before and during the November and December 2007 heavy rainfall events. The contractor was again called on under the terms of the contract to remove a further 6 800 m^3 of rock debris from behind the retaining walls. It is probable that many road users of this pass would not comprehend the full extent of the rockfall impacts to which

4 Retaining wall after rainfall events (January 2008). Photo taken from the top of the 6-m-high retaining wall
5 View site rockfalls (December 2007)
6 Restraining mesh system showing tensioned diagonal cables

these walls were subjected, as well as how the actions taken prevented vehicle damage or loss of human life.

VIEW SITE

The slope failure at the view site consisted of a mass movement of oversaturated crest and upper slope material, which had been 'lubricated' at the soil/rock interface. Significant areas of highly fractured and randomly orientated rock were destabilised by the rainfall events and there were several close calls for construction vehicles (see Photo 5).

The stability remedial work at the view site comprised the following:

- Installation of permanent rock bolts 2,0 to 5,0 m in length, set approximately 10,0 m back from the crest of the 65-m-high slope. The view site is approximately 250 m long. The rock bolting arrangements were installed in two distinct arrangements:
  - The 3,0 m rock bolts were used predominantly as summitry and basal anchorage for the drapery mesh system, but also served to restrict the

movement of groundwater in the crest material to some extent. The rock bolt arrangement formed a rudimentary grout curtain, effectively sealing voided (washed-out) areas within the oversteep crest material.

- The 5,0 m rock bolt arrangement was installed to penetrate through as many layers of the parent bedrock layers as possible, thereby essentially preventing lubrication of the weaker bedding and jointing planes, and reducing the likelihood of the rock mass sliding.

- The drapery mesh system was enhanced to become a restraining mesh system. This involved the difficult installation of 8,0 mm steel cable over the drapery mesh, tensioned diagonally across the mesh to form a diamond shaped cable restraining system. Rock bolts were drilled on the slope at the 8,0 mm steel cable nodal points and the diamond-pattern cable system was tensioned.

In summary, 287 permanent rock bolts, 10 700 m^2 of drapery mesh and 700 m of steel cable were installed at the view site in the pass (see Photo 6).

RETAINING WALL 4

Retaining wall 4 is 90 m long and 4,5 m in height, and is situated towards the lower end of the pass (on the Calitzdorp side). This retaining wall (see Photo 3) has endured severe rock impacts and there is visual evidence that falling boulders have struck the top of the retaining wall and bounced into and over the road.

Furthermore, during the December 2007 rainfall event, it became apparent that water ingress had lubricated a rock mass in the order of 150 m^3 (1,50 m thick), resulting in this entire rock mass sliding down into and over the retaining wall. The failed rock mass was underlain by highly fractured rock with fracture or bedding planes infilled with a clayey silt. When the fracture or bedding planes were sufficiently lubricated, the rock mass slid (see Photo 7).

The original contract allowed for remedial work behind this retaining wall in terms of rock debris removal and reconstruction of the damaged wall only. The nature of the rock movement demonstrated that a further significant rock mass behind the wall was potentially unstable and needed immediate stabilisation if further damage to the wall was to be avoided. The remedial stabilisation work comprised the following:

- Stabilisation of the affected crest areas was undertaken using the same arrangement of permanent rock bolts and restraining mesh as was employed at the view site. Approximately 96 rock bolts and 4 100 m^2 of drapery mesh were installed at the crest of this retaining wall.
- The highly fractured rock layer also provided a water pathway which had the potential to destabilise further rock masses on the slope. In order to stabilise this precarious rock area, drapery mesh was placed over the affected area and steel-fibre-reinforced shotcrete was sprayed over the mesh. The steel fibres were incorporated into the shotcrete to improve its flexural strength.

7 Retaining wall 4 rockfalls (December 2007)
8 Retaining wall 4 remediated (August 2008)

- Further rock bolting was undertaken into the highly fractured rock area and the bolts were anchored into competent rock strata underneath. Weepholes were installed to prevent hydrostatic pressure build-up behind the shotcrete (see Photo 8).

Flexural strength tests were undertaken on different shotcrete samples to establish the strength characteristics of the steel fibres and to determine the contribution of the mesh in the shotcrete. The test results are shown in Table 1.

Although the testing was limited, the results indicated that the addition of the steel fibres and the mesh in the shotcrete brought about a 75% improvement in flexural strength compared with shotcrete only. An 8% improvement in flexural strength was attained in the shotcrete containing fibres and mesh in the 28-day tests. The addition of steel fibres to the shotcrete also conferred a higher degree of flexibility compared with conventional cement shotcrete application.

CONCLUSIONS

Approximately 31 000 m^2 of rockfall protection netting was installed during this contract at Huis River Pass and protection incorporated the drilling of 890 rock bolts, 9 km of steel cabling, as well as the removal of 23 590 m^3 of rock debris.

This project was marked by some unexpected events which created interesting and challenging aspects during the construction phase. Good teamwork between the employer, contractor and engineer ensured that all problems were adequately addressed.

Mother Nature is thanked for her contribution to the project team. ◻

Table 1 Flexural strength test results

Specimen description	Test age (days)	Tensile strength (MPa)
Shotcrete only	14	1,7
Shotcrete only	14	2,0
Shotcrete/steel fibres	14	3,5
Shotcrete/steel fibres/mesh	14	3,2
Shotcrete/steel fibres	21	3,5
Shotcrete/steel fibres/mesh	21	3,5
Shotcrete/steel fibres	28	3,8
Shotcrete/steel fibres/mesh	28	4,1

PROJECT TEAM

Employer Western Cape Provincial Administration
Department of Transport and Public Works
Roads Branch
Contact: M Stewart
mstewart@pgwc.gov.za
Contractor Penny Farthing (SA) (Pty) Ltd
Contact: H Terblanche
hugo@pennyfarthing.co.za
Consulting Engineers Kantey & Templer (Pty) Ltd
Contact: P D Beales
pbeales@kanteys.co.za

Text: **Ron Tluczek**
Departmental Head (Geotechnical)
BKS (Pty) Ltd
ront@bks.co.za

Risk analysis and emergency remedial works carried out along Kaaimans River Pass

In August 2006 severe floods between George and Knysna caused extensive damage alongside the N2/7 National Road. Several zones required extensive remedial works

BACKGROUND

The National Road through the Garden Route in the Western Cape is the main national economic arterial on the southern side of the country. This area is also one of the main tourist attractions in South Africa.

During the month of August 2006, abnormal weather conditions were experienced when a total of 377 mm of rain was recorded. This far exceeded the regional August average of 68 mm or the highest recorded rainfall for any month of 132 mm. The resultant floods between George and Knysna caused extensive damage to slopes alongside the N2/7 National Road and to the adjacent railway line, especially in and next to the Kaaimans River Pass. This area was subsequently declared a disaster area.

The cuttings for the original road in this area were excavated in 1948. The original road was then upgraded between 1984 and 1987. This contract entailed the construction of numerous full and half viaduct structures along the edge of the steep rock faces above either the Kaaimans River or the railway line running along the coastline. After completion of the contract, two significant failures occurred around 1993 which resulted in the loss of portions of the road surface. In both cases the failures occurred below the road after heavy rainfall. The first failure was remedied by installing a piled anchored retaining wall in order to stabilise the outer bend of the road. Remedial works at the second failure consisted of constructing a portion of full viaduct with an adjacent portion of half viaduct.

After the 2006 slope failures, and in the light of the route's history, the owner, the South African National Roads Agency Limited (SANRAL), immediately commissioned consultants to carry out a risk assessment of all the natural and man-made slopes, to make recommendations and to implement remedial measures to safeguard the travelling public.

GEOLOGY AND SITE INVESTIGATION

The route is underlain by phyllites, shales, quartzites, felspathic quartzites, greywackes and quartz schists of the Kaaimans Group. This Group represents a sedimentary sequence which has been metamorphosed to varying degrees and has been intruded by gneissic granite, granodiorite and albitite (syenite) of the Cape Granite Suite along the southern margin. The regional foliation is very well developed and is coincidental with the bedding, i.e. 30 to 40 degrees to the south-east (away from the mountainside).

A multi-faceted approach to the site investigation was used which included in situ inspections, aerial surveillance, remote sensing, the excavation of exploratory trenches and the drilling of rotary core boreholes to determine the underlying stratigraphy and to install inclinometers for future monitoring of ground movements.

The results from the high-resolution aeromagnetic survey identified several probable and possible faults. Although all these faults traverse the roadway in the study area, none, except one at the mouth of the Kaaimans River, seem to have any influence on the stability of the adjacent slopes.

RISK ANALYSIS

Several failure mechanisms were observed within the study area. On the soil slopes these included erosion, hill creep, and translational and circular failures. In the rock mass, these mechanisms included planar, wedge and toppling failure, as

well as ravelling. The predominant failure modes were identified as erosion and translational slides in soil, with wedge and planar failures in the rock masses.

The ground characteristics along the route through the study area varied in terms of geology, topography and the type of slope failure mechanism that occurred. To provide a means of assessing these varying conditions, the route was divided into zones. Each zone represented an area that was expected to display the same behaviour, i.e. it would have similar geology and topography but, more importantly, it would display the same type of slope failure mechanism.

Each zone was then analysed according to the slope failure mechanism(s) prevalent in that area. A probability of failure analysis was carried out at each of the analytical sections along the route. A probability of failure analysis, amongst others, was used as it is able to evaluate the sensitivity of more than one design parameter and its overall contribution to slope stability. In addition, the slopes were classified on the basis of consequence of failure, as described below.

- *Severe consequence of failure.* Failure of these slopes would result in severe damage to property and possible loss of life. Severe damage would necessitate partial or complete closure of the National Road in order to safeguard the travelling public. The major failures experienced after the August 2006 floods fell into this category.
- *Moderate consequence of failure.* Failure of these slopes may result in moderate damage to property. Possible loss of life is not foreseen. Moderate damage would result in partial closure of the National Road, necessitating immediate maintenance with minimal road closure foreseen. The erosion "failures" experienced after the August 2006 floods fell into this category.
- *Inconsequential consequence of failure.* Failure of these slopes may result in either minor damage to property or no damage at all. Minor damage resulting in excessive maintenance work is foreseen. All of the minor erosion and translational failures that occurred after the August 2006 floods fell into this category.

After the problematic slopes had been analysed and classified, a risk matrix was set up whereby the probability of failure of each slope was compared with the consequence of failure (see Table 1). From Table 1 it can be seen that there are seven (7) categories of probability of failure and three (3) categories for consequence of failure – this results in a three (3) by seven (7) matrix. By giving the categories that have the least effect the lowest rating (unity) and the most onerous category the maximum rating, one is able to develop a risk matrix as indicated below. The risk score is calculated as the product of the probability of failure rating and the consequence of failure rating. This matrix enabled the engineer to rank the slopes in order of remedial priority.

The Priority 1 slopes were identified as those with a risk score greater than 15. These slopes have the highest potential for failure, coupled with the highest consequence of failure. As a result of this combination, these slopes required immediate attention in order to ensure the safety of the travelling public.

The Priority 2 slopes were identified as those with a risk score of 15 or less but greater than 7. These slopes either have a low to

1 Failure of roadway

Table 1 Risk matrix

Risk score		Rating	Consequence of failure		
			Severe	Moderate	Low
			3	2	1
Probability of failure	50% – 100%	7	21	14	7
	20% – 50%	6	18	12	6
	10% – 20%	5	15	10	5
	5% – 10%	4	12	8	4
	1,5% – 5%	3	9	6	3
	0,5% – 1,5%	2	6	4	2
	<0,5%	1	3	2	1

medium risk of failure, coupled with a severe consequence of failure, or a high risk of failure, coupled with a moderate consequence of failure. As a result of this combination, these slopes did not require immediate attention, but would require attention in the medium term to ensure the safety of the travelling public. These slopes will also require ongoing monitoring to ensure that nothing untoward is occurring.

The Priority 3 slopes have a risk score of 7 or less but greater than 3. These slopes either have a medium to high risk of failure, coupled with a low consequence of failure, or a low risk of failure, coupled with a moderate to severe consequence of failure. As a result of this combination, these slopes require ongoing maintenance and monitoring to ensure that nothing untoward is occurring. These slopes do not represent a safety threat to the travelling public.

The Priority 4 slopes have a risk score of 3 or less. These slopes have a negligible to low risk of failure, coupled with a low consequence of failure. As a result of this combination, these slopes require no attention except for some routine maintenance. No monitoring is foreseen.

REMEDIAL WORKS

Typical emergency remedial works carried out on the major failures within the study area included the following:

Composite failure at Dolphin's Point

This failure resulted in the spectacular subsidence of a portion of the National Road in the vicinity of Dolphin's Point at the mouth of the Kaaimans River. This failure was the result of at least three separate failures which resulted in a composite failure. Remedial works entailed the construction of a half viaduct to bridge the unstable area. The half viaduct is supported on piles which are founded in good-quality rock below any potential failure surface.

Planar failure in rock mass

Two major planar failures occurred along the Kaaimans River Pass. Remedial measures for the first failure consisted of clearing all slide debris and other unconsolidated overburden until the underlying contact with more sound tight bedrock was exposed. Rock bolts were installed in selected areas on the lower rock slopes to pin together in situ rock slabs. This effectively created a thicker rock slab which would be more resistant to uplift forces from groundwater pressure. Drainage holes were drilled into the rock slope to intercept groundwater and so minimise the possibility of excessive groundwater pressures occurring in the rock mass. Due to the steepness and extent of this failure, an abseiling team were used to clear the slope of all debris and to install the rock support and drainage holes.

After all debris had been cleared off the second failure slope, pattern rock bolting was installed to pin the weathered rock mass together. Mesh-reinforced shotcrete was applied to the rock face to prevent ravelling and long-term weathering of the rock mass. Drainage holes, 6 m long, were drilled into the rock mass to intercept groundwater and so minimise the possibility of excessive groundwater pressures occurring within the rock mass.

2 Planar failure before remedial works
3 Planar failure after remedial works
4 Erosion failure

Erosion failure in decomposed granite

Erosion of the decomposed granite resulted in large corestones becoming loose and sliding onto the National Road. Initially, the exposed decomposed granitic surface was flattened by using a high-pressure water jet. Pattern bolting with mesh-reinforced shotcrete (i.e. soil nailing) was installed to pin the granitic mass together, to prevent erosion and long-term weathering of the granitic mass. Due to the inaccessibility of the rock/soil interface, a special scaffold structure was erected in order to gain access to the granitic face.

CONCLUSION

All of the above remedial works have been successfully completed. There have been two significant rainfall events in the interim, and all of the slopes in the study area have performed satisfactorily without any further failures occurring. ◻

PROJECT TEAM

Owner South African National Roads Agency Limited
Consulting Engineers BKS Engineering & Management, Pretoria
Contractors Alpinist Safety Consultants (Pty) Ltd
A C Forbes & Associates
Fairbrother Drilling
Fugro Airborne Surveys (Pty) Ltd
GAP Geophysics (Pty) Ltd
Power Construction
Stefanutti & Bressan Piling (Pty) Ltd

Text Alan Parrock
Geotechnical Principal
ARQ Consulting Engineers (Pty) Ltd
alan@arq.co.za

Lateral support for cuts and fills on National Route 1, Section 21, between Lynnwood Road and Rigel Avenue

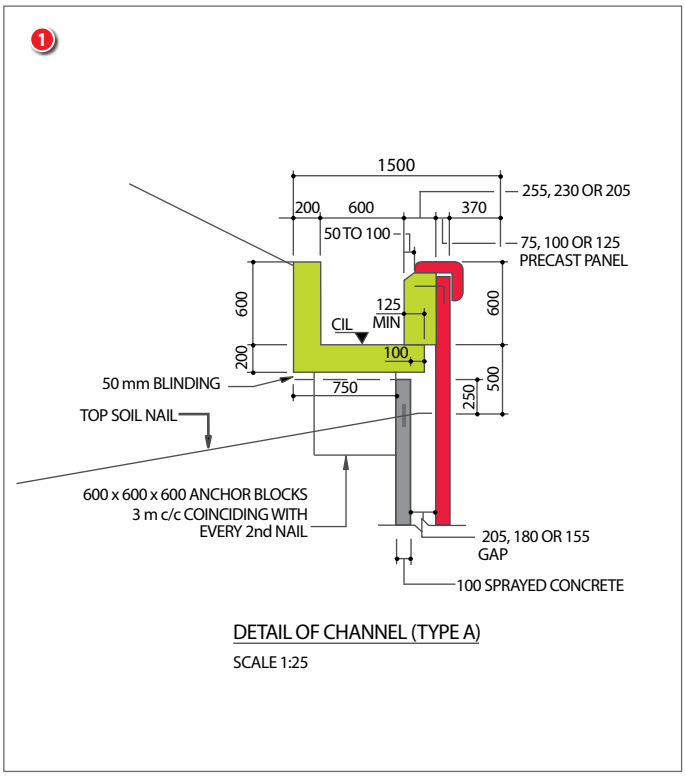
INTRODUCTION

The section of National Road Route 1, Section 21, between the Lynnwood Road (Scientia) and Rigel Avenue interchanges on the eastern outskirts of Pretoria is being widened as part of the Gauteng Freeway Improvement Project (GFIP) at present being implemented by the South African National Roads Agency Limited (SANRAL).

For the widening in cuts the options were to trim the existing slopes in a thin sliver to the top of the cut or to construct a small retaining structure at the base. Space constraints and environmental considerations precluded the sliver option and it was decided that reinforcement with soil nails would be the most cost-effective option.

For fills, the sliver option would have entailed starting at the base and building the widening to the top. This would have necessitated the widening of existing culverts beneath the route. In order to circumvent this, a small geosynthetic-reinforced retaining structure was used at the top of the fill to facilitate the widening.

This article examines the cut- and fill-structures designed and constructed on this R900 million project.



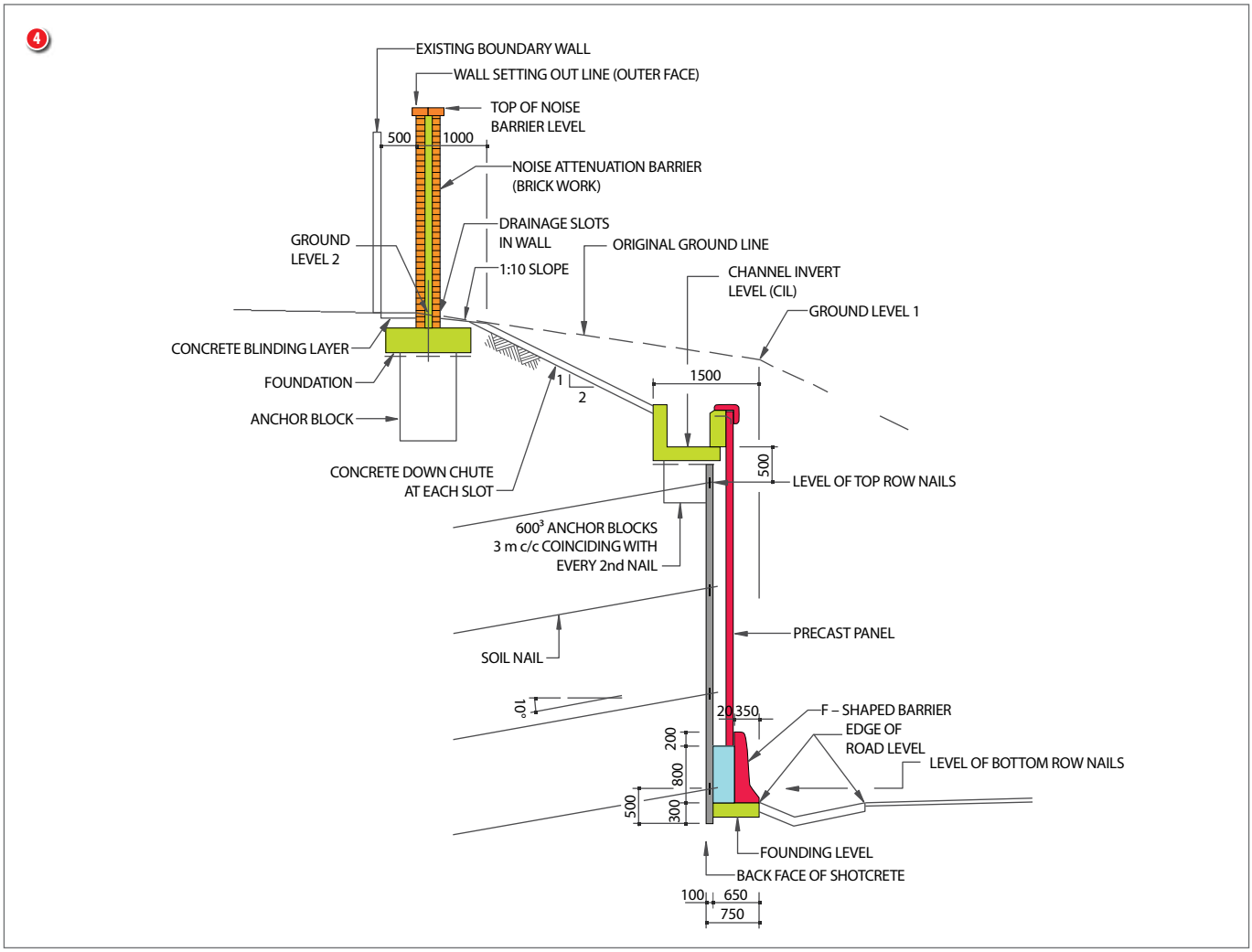
1 Detail of drain, cladding panel and capping beam

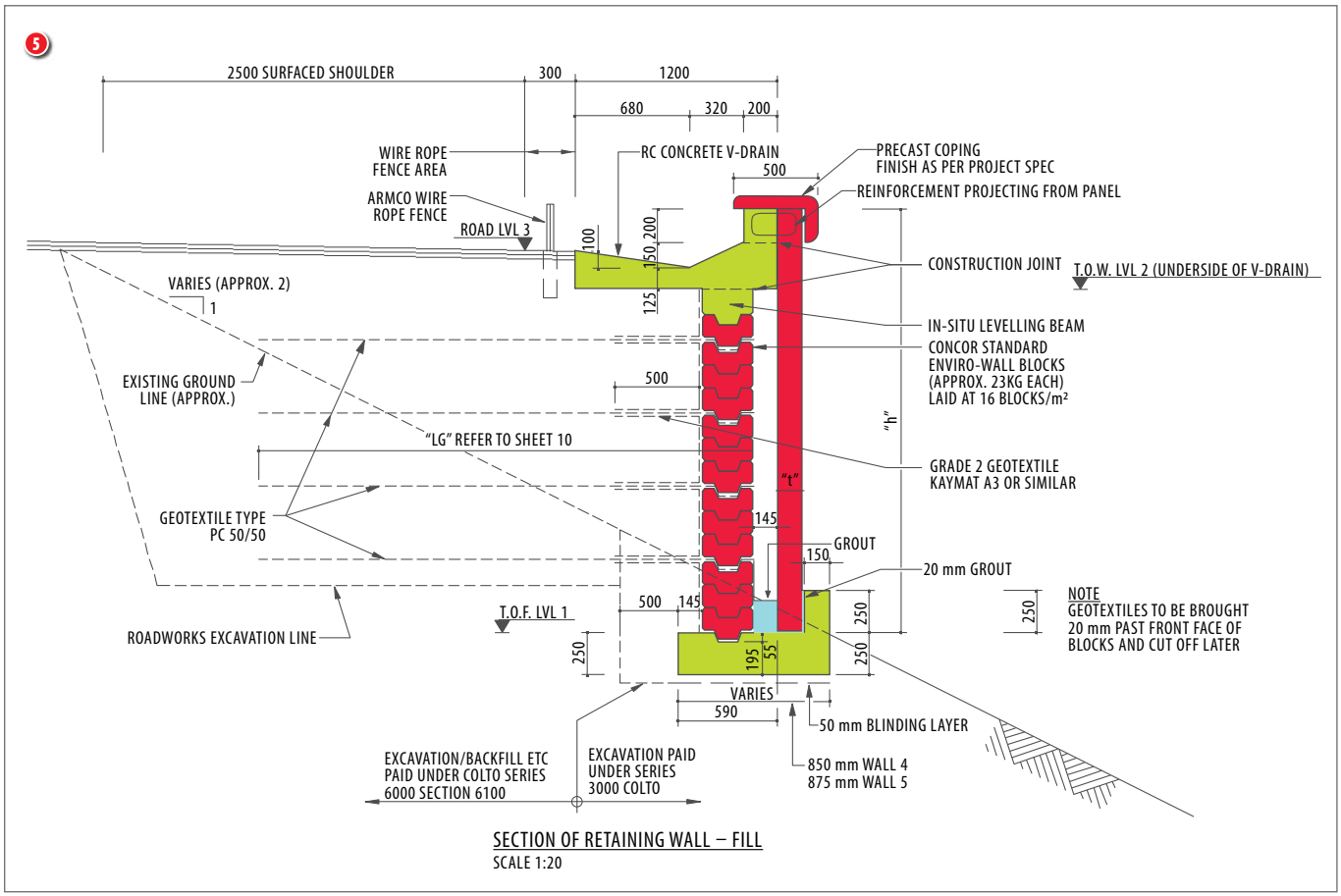
CUTS

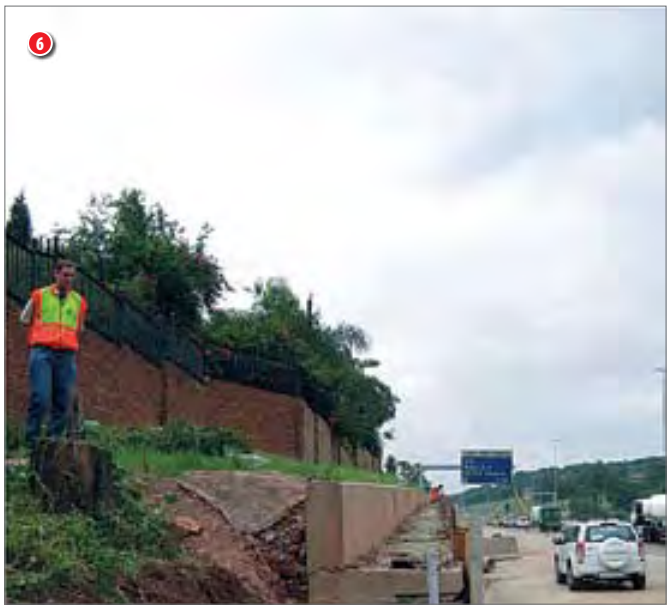
In cuts, the lateral support to vertical faces was provided using soil nails in conjunction with a shotcrete facing. The following materials were used:

- *Soil nails:* "Threadbar 500" bars which had been galvanised according to SABS ISO 1461, edition 2 2000, were used. The galvanising was to 85 µm "mean coating thickness" as per column 3 of Table 2 of the above specification. The bars were initially passivated with sodium dichromate, but when this raised health and safety questions, this was changed to Galvogaurd to counter carcinogenic considerations. The bars were installed centrally in holes of 100 mm diameter drilled using rotary or percussion techniques. The drilled holes were filled with 25 MPa of grout mixed from cement and water only. The consistency of the grout was flowable during installation.
- *"Spider":* This consisted of 4 x 640 mm long Y16 bars, welded into the form of a cross so as to form a square of 50 x 50 mm at the central intersection point. It was not necessary to galvanise this item as it is embedded in the shotcrete facing.

2 Erecting panels on cut walls
3 Cut wall nearing completion
4 General arrangement of cut wall and noise barrier
5 General arrangement of fill walls
6 Drain
7 Fill wall prior to cladding







- *Head plate and nut:* galvanised
- *Drainage layer:* The layer behind the shotcrete was of Flownet Type DN3, 150 mm wide, wrapped in Bidim A4. This drainage layer was shaped into a V at the bottom end for discharge into a 120 mm long, 50 mm ϕ Class 6 uPVC pipe, which is plugged on the soil end by affixing with binding wire a piece of Bidim A4 geosynthetic fibre.
- *Mesh reinforcement:* This complies with the requirements of clause 6302(b) of COLTO (1989).

CONSTRUCTION

Soil nails were installed as excavation proceeded. The face of the cutting was initially machine excavated, but final trimming was done by hand. Once the lines and levels to the tolerances of +0 mm (i.e. towards the outside of the face) to –50 mm (i.e. into the soil face) had been attained, the drainage strip was applied, followed by a flash coat, 25 mm thick. The holes for the soil nails were drilled through the flash coat and the grout pumped into each hole from the base of the hole so as to ensure that the complete hole was filled with grout. When the grout overflowed from the face of the hole, the soil nails were installed with spacers to ensure that they were centrally located within the holes. Where the soil face was very stable, this procedure was altered to suit the contractor's requirements in that the soil nail and grout were in some cases installed prior to the application of the flash coat.

The mesh reinforcement was installed next, such that the central portion of the 200 x 200 mm hole in the mesh was over the soil nail.

This was followed by the "spider", head plate and nut, which effectively fixed the mesh in place.

Shotcrete, 75 mm thick, was then applied, such that all mesh reinforcement, the "spider", head plate and nut were contained

8 Practically completed fill wall

with sufficient cover within the shotcrete. The surface was finished to GTW standard. This consisted of floating the trowelled surface with a wood float to produce a smooth, even surface free from trowel marks.

CLADDING, PROTECTION, SURFACE DRAINAGE AND NOISE BARRIERS

The soil-nailed face provided the basis for cladding with precast planks, an F-shaped barrier at the base to prevent damage by errant vehicles, and a storm water channel and noise barrier at the top. Some construction drawings are shown in Figures 1 and 4, and Figures 2, 3 and 6 are photos of the near-finished product at this point in time.

FILLS

Here a geosynthetic-reinforced concrete block retaining structure was constructed, as detailed in Figure 5.

A 50 kN high-strength, low-strain geosynthetic was used as fill reinforcement in conjunction with a commercially available concrete block and a selected backfill material. Specialist contractors, well versed in the construction of these walls, were appointed as subcontractors for this task. In order to attain uniformity of appearance between cuts and fills, this structure was also equipped with precast cladding panels. Figures 7 and 8 show the near-finished product. ◻

CREDITS

Client South African National Roads Agency Limited (SANRAL)
Contractor Basil Read on phase 1 with specialist subcontractors Geo Compaction Dynamics (GCD) for the cut walls and Friction Retaining Systems (FRS) for the fill walls. The BRCD joint venture is the contractor for phase 2
Consultant DCA – A joint venture comprising DEC, Civil Concepts and ARQ

Text Dr Fritz Wagener
Fritz Wagener & Associates
wagenerf@telkomsa.net

Sinkholes in the Bapsfontein dolomite water compartment caused by dewatering

The Bapsfontein dolomite water compartment, located east of Pretoria on the Delmas road, is being dewatered at an alarming rate and is resulting in widespread instability in the area in the form of sinkholes, depressions or cracks. Many boreholes in the area are drying up and this is depriving inhabitants of their source of potable water. The question that needs to be answered is: Why are a few users in the area authorised to pump more water from the aquifer than its sustainable yield?

1 Sinkhole at Bapsfontein on 28 January 2004
2 Aerial view of depression with satellite sinkholes
3 Aerial view of irrigation at Clear Water Farms

INTRODUCTION

Bapsfontein is a small settlement, in a fertile agricultural area, located 60 km south-east of Pretoria at the crossing of the R50 road between Pretoria and Delmas and the R25 to Bronkhorstspruit. Geologically, the site is underlain by Malmani dolomite of the Chuniespoort Group, Transvaal Supergroup. The dolomite stretches from the Rietvlei Dam near Pretoria in the north-west to beyond Delmas in the east, a distance of some 60 km, and more than 40 km to Springs in the south. Immediately north of Bapsfontein, the dolomite is capped by rocks of the Pretoria Group and in the south by the younger rocks of the Karoo Supergroup. In the study area the dolomite is partly overlain by windblown sands, Karoo strata and cherts of the Rooihoogte Formation.

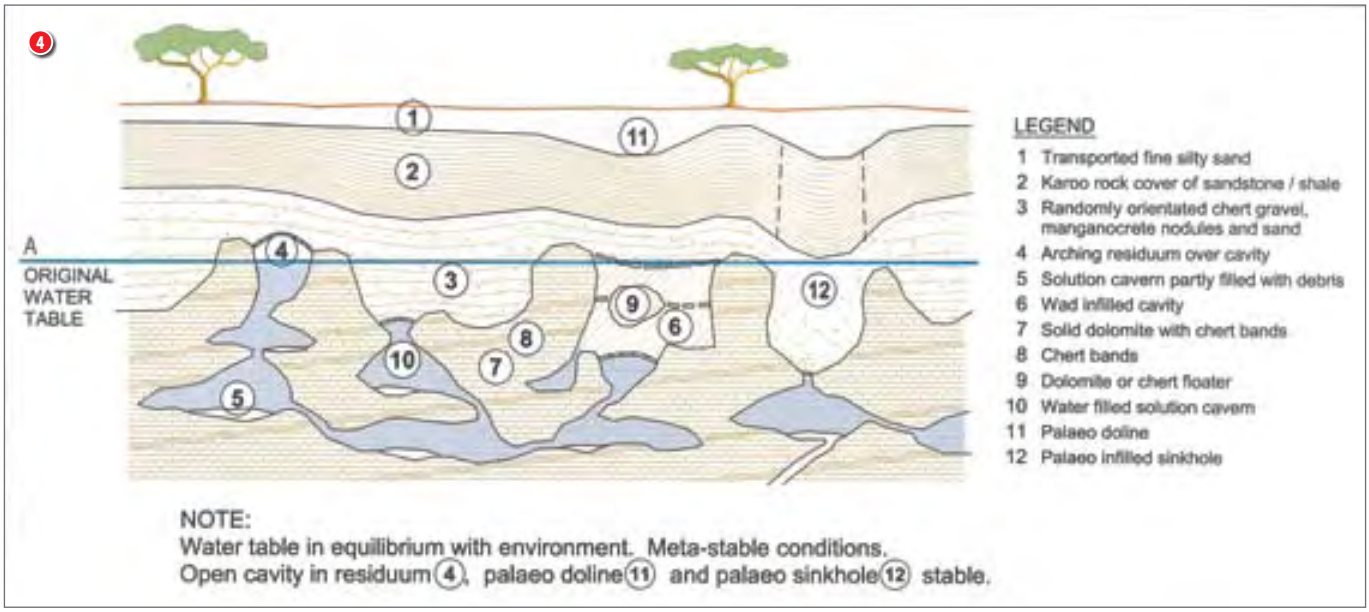
An investigation into the stability of the area was undertaken following the development of a large sinkhole on 28 January 2004 next to a surfaced road and a few hundred metres from an informal settlement. Initially, the size of the sinkhole was approximately 30 m in diameter by 15 m deep. An aerial view of the sinkhole is shown in Figure 1.

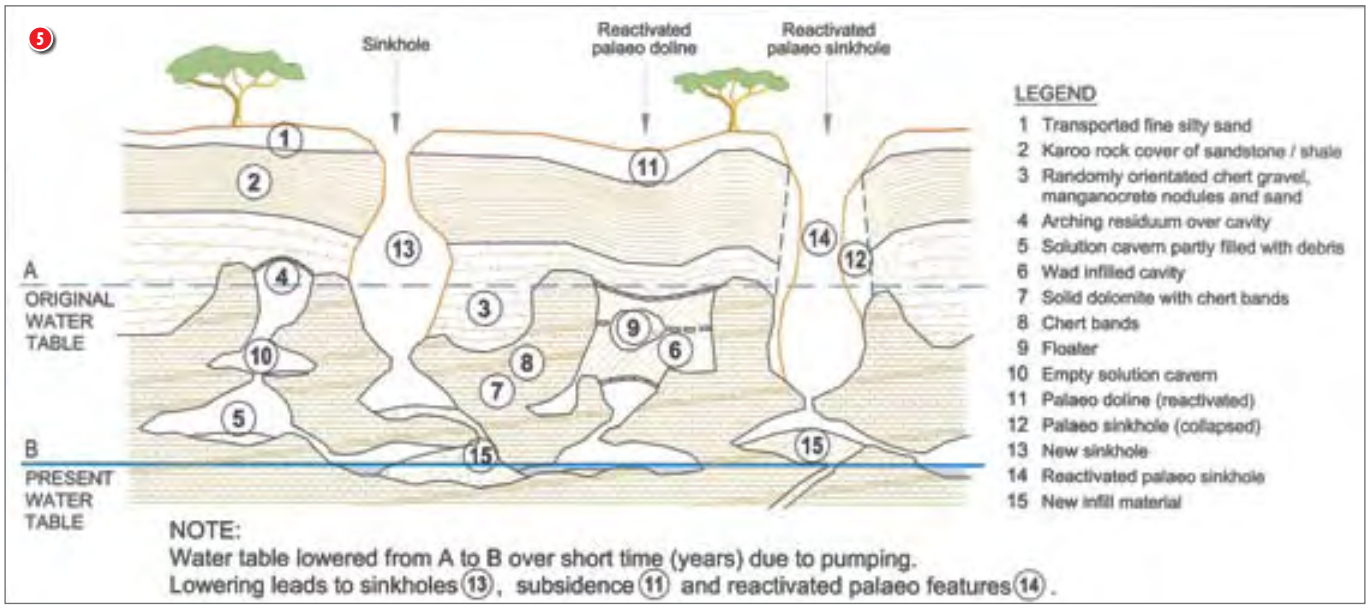
An inspection of the area around Bapsfontein in the weeks following the incident revealed a further 18 sinkholes, depressions or cracks in various stages of development. An aerial view of a doline or depression with satellite sinkholes developing along its perimeter is shown in Figure 2.

Most of the instability noted occurred in a fairly densely populated area with a number of agricultural holdings, a densely populated informal settlement, a primary school, the former Bapsfontein Hotel, chicken, mushroom and irrigation farms, and various surfaced and unsurfaced roads. Intensive irrigation farming is being practised in the immediate area and as far afield as Delmas, with many centre-pivot installations irrigating maize and vegetable crops. Figure 3 shows the intensity of irrigation immediately west of Bapsfontein.

MECHANISMS OF SINKHOLE AND DOLINE FORMATION

It is appropriate to give a brief description of the mechanisms of sinkhole and doline formation. A sinkhole is a feature that occurs suddenly and manifests itself as a hole in the ground. A doline is an enclosed depression or subsidence that usually occurs gradually. Both can occur in a dewatering or non-dewatering situation.





4 Typical soil profile over dolomite before dewatering

5 Typical soil profile over dolomite after dewatering

6 Ekurhuleni Bapsfontein dolomite: geological section of informal settlement and residual gravity along Section A-A

The typical soil and rock profile encountered in the Bapsfontein area is shown in Figures 4 and 5. The dolomite is covered by an intermittent layer of Karoo rocks comprising sandstone, shale and dolerite. This layer is draped over a much older, unevenly weathered dolomite surface and can vary in thickness from zero to tens of metres.

The chert-rich dolomite rock weathered in geological time to chert gravel and boulders, as well as wad (mainly MnO_2) in a matrix of loose sand. This residuum fills hollows and grikes (subsurface valleys) as shown in the figures. Solution caverns developed in the "solid" dolomite during the same period.

The original water table in Figure 4 at "A" is relatively high, with water, wad and other debris filling most of the caverns. The only exception would have been the air-filled cavity in the residuum. In the absence of large-scale human development, such an area would be in equilibrium, with possibly a few subsidences or sinkholes occurring over time.

With the large-scale pumping taking place from the dolomite aquifer at Bapsfontein, the water table has been lowered from "A" to "B" in a relatively short time (years) and the area has become unstable. The extent of instability depends on factors such as the size of the caverns exposed above the water table, the thickness and properties of the residuum over the dolomite, the original level of the water table and the rate of pumping from the aquifer.

Following the lowering of the water table, the instability will continue for a long time. Development in the area with infrastructure will cause the concentrated ingress of water into the subsurface, initiating new instability or extending existing instability. If pumping from the aquifer continues at such a rate that the water table is lowered even further, the instability can be expected to increase.

The Karoo cover over the erodible dolomite residuum at Bapsfontein is sometimes thin or absent and in such cases the site is more sensitive to subsurface erosion, with resulting instability.

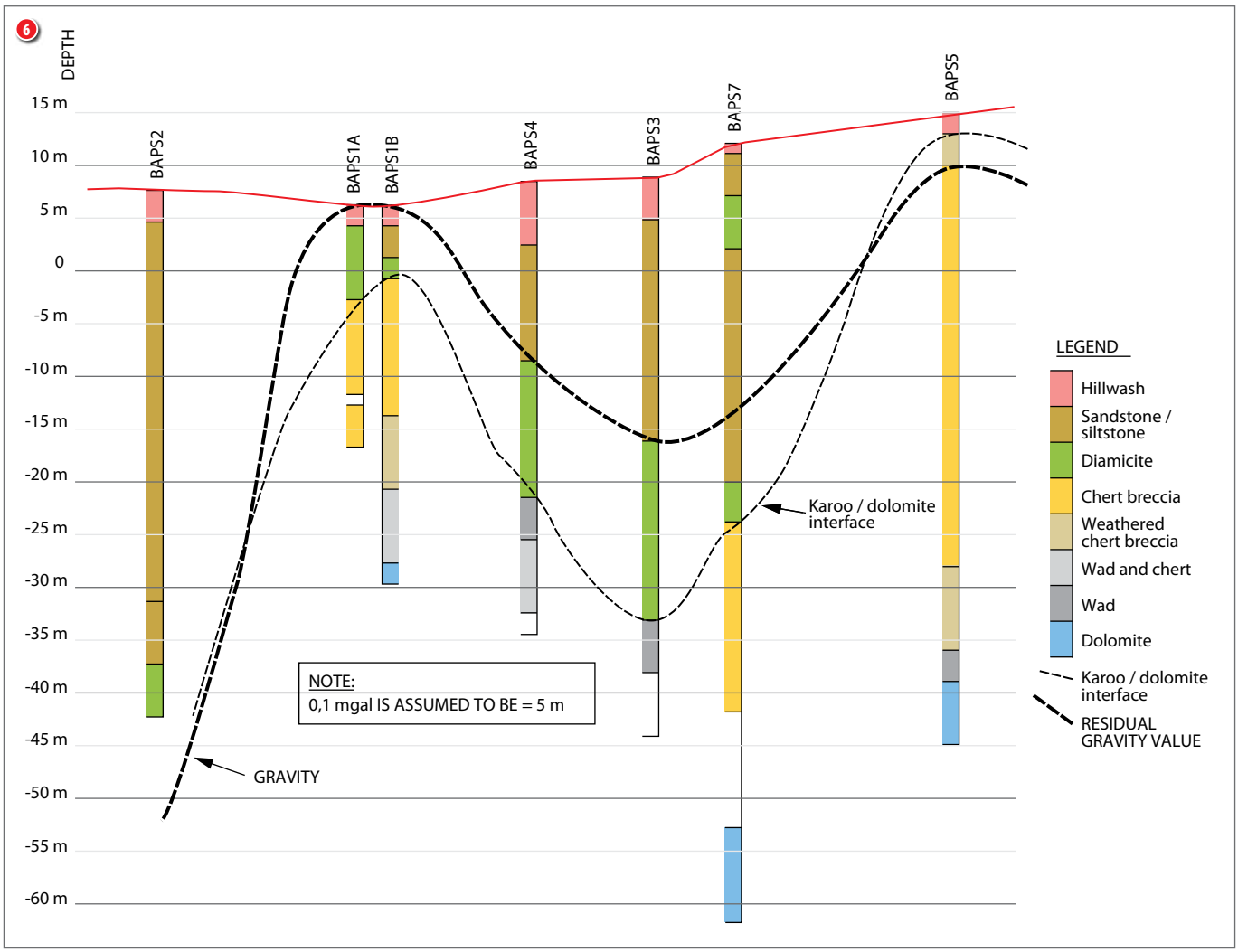
INSTABILITY IN THE AREA

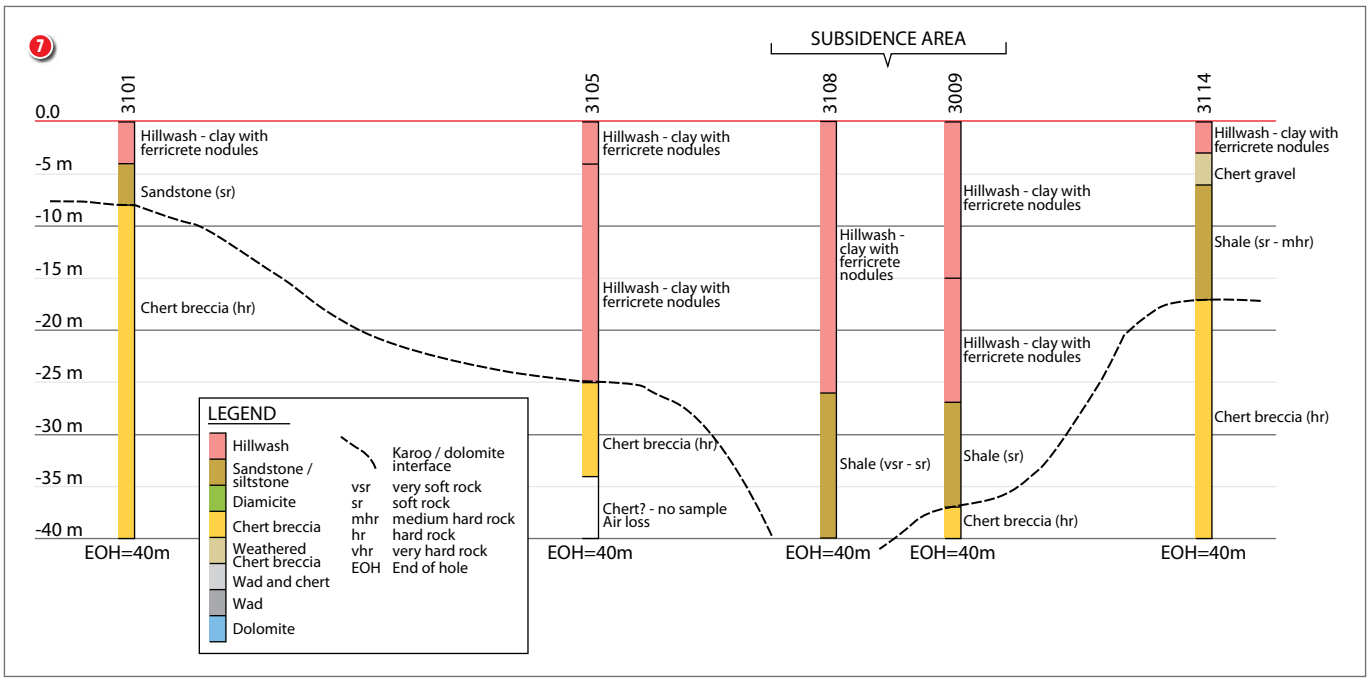
The sinkholes, depressions or cracks encountered in the immediate area around Bapsfontein were inspected in detail

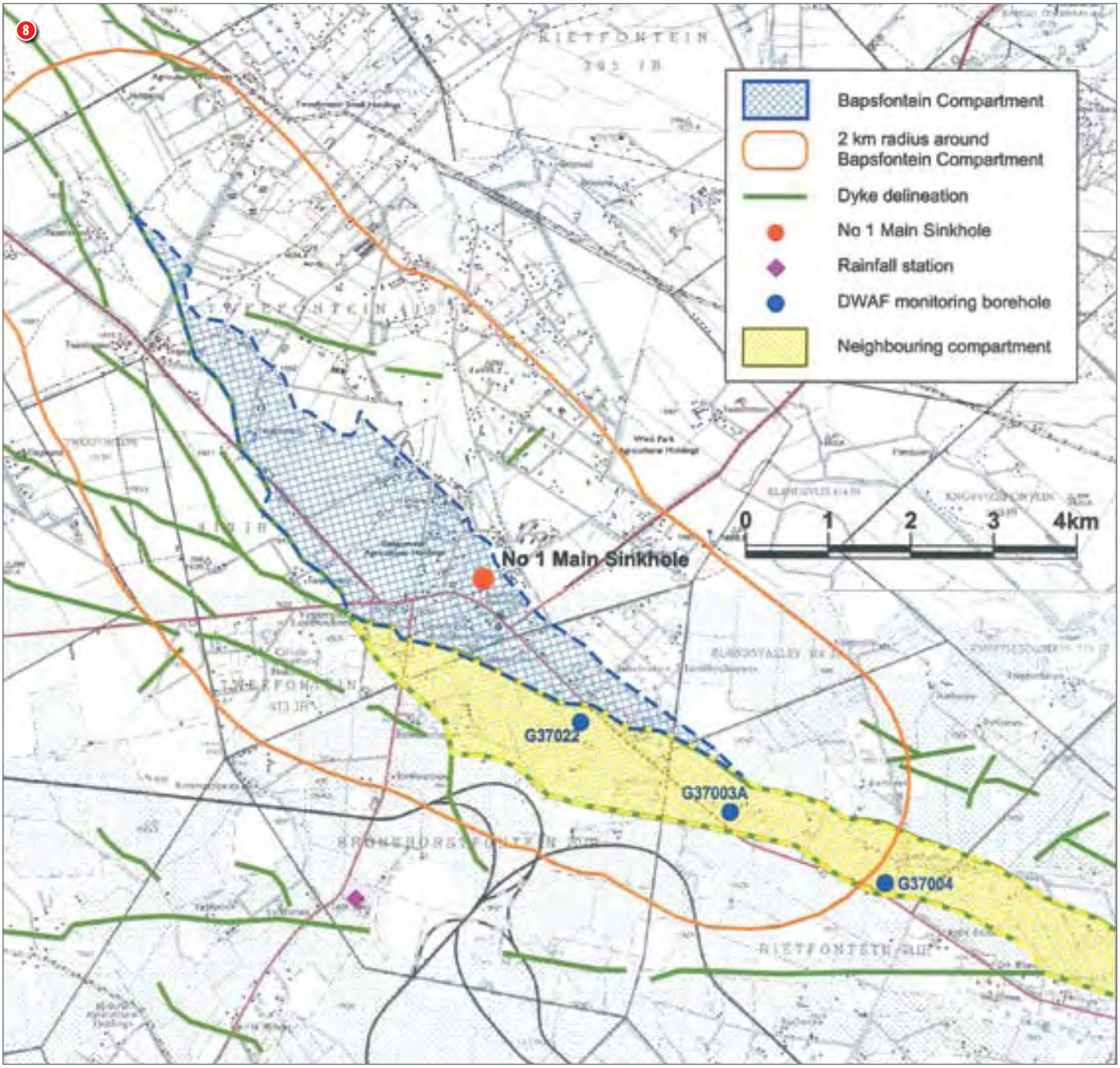
following the development of the large sinkhole in January 2004. Discussions with the residents revealed that instability in the area has been ongoing for some considerable time and has increased since the 2000s. Some of the features are described below:

- The most dramatic sinkhole is the one that developed on 28 January 2004. It is next to a surfaced road and 500 m west of the informal settlement. The surface in this area is covered with a relatively thin alluvium, followed by weathered Karoo siltstone or sandstone to a depth of about 15 m. The sinkhole started as a shallow depression next to a minor stream and upslope of a partially blocked culvert. Children were seen swimming in the water-filled depression a few days before the sinkhole formed. The mechanism responsible for the development would have been cracking in the Karoo capping caused by a lowering of the water table. The water in the stream backed up against the culvert and infiltrated the cracks, causing subsurface erosion. This would have resulted in the formation of the sinkhole at the surface. The mechanisms are illustrated in Figures 4 and 5.
- A large doline (depression) is shown in Figure 2. It is elliptical in shape with axes of 80 m and 100 m respectively. The area had subsided by about 2 m, with cracks along the perimeter up to 4 m, deep at the time of the inspection in January 2004. The depression is located at the entrance to a large poultry farm and not more than 100 m north-west of a primary school in the informal settlement. It developed following dewatering in an area underlain by a thick layer of compressible material such as wad. A feature of such a depression is that storm water flows into the perimeter cracks, causing subsurface erosion which results in satellite sinkholes. By January 2005 a satellite sinkhole on the western side had developed into a sinkhole 10 m x 12 m in plan and 9 m deep.
- A depression at the intersection of a local road with the R25 is located just east of the informal settlement. It was drilled in 1993; a geological section is shown in Figure 7 and discussed in the next section. According to local residents, it was repaired but settlement continued, resulting in a significant bump in the road by early 2004. The depression had been filled and resurfaced by May 2005.
- A 200-mm-wide crack extended through the informal settlement. Small sinkholes (1 to 2 m deep) have apparently developed along the crack, but have been backfilled by residents. Borehole BAPS 7 was drilled in the vicinity of the crack and encountered a cavity or loose zone above "solid" dolomite at a depth of 54 to 65 m. The water table was not encountered in the borehole.
- A crack was observed in an open field about 400 m south-east of the depression in the R25 road. In February 2004 this crack was about 5 m long. By July 2005 it was 30 m long, 3 m wide and a few metres deep.

Following the inspection of the instability in the area, it was decided to concentrate the geotechnical investigation in and around the informal settlement. The geohydrological investigation was done over a much wider area as described in the next section.




7
SUBSIDENCE AREA
3101
3105
3108
3009
3114
0.0
-5 m
-10 m
-15 m
-20 m
-25 m
-30 m
-35 m
-40 m
Hillwash - clay with ferricrete nodules
Sandstone (sr)
Chert breccia (hr)
Hillwash - clay with ferricrete nodules
Hillwash - clay with ferricrete nodules
Chert breccia (hr)
Chert? - no sample Air loss
Hillwash - clay with ferricrete nodules
Shale (vsr - sr)
Hillwash - clay with ferricrete nodules
Hillwash - clay with ferricrete nodules
Shale (sr)
Chert breccia (hr)
Hillwash - clay with ferricrete nodules
Chert gravel
Shale (sr - mhr)
Chert breccia (hr)
EOH=40m
EOH=40m
EOH=40m
EOH=40m
EOH=40m
LEGEND
Hillwash
Sandstone / siltstone
Diamicite
Chert breccia
Weathered Chert breccia
Wad and chert
Wad
Dolomite
Karoo / dolomite interface
vsr very soft rock
sr soft rock
mhr medium hard rock
hr hard rock
vhr very hard rock
EOH End of hole


8
Bapsfontein Compartment
2 km radius around Bapsfontein Compartment
Dyke delineation
No 1 Main Sinkhole
Rainfall station
DWAF monitoring borehole
Neighbouring compartment
0
1
2
3
4km
No 1 Main Sinkhole
G37022
G37003A
G37004

FIELDWORK UNDERTAKEN

Geotechnical

The geotechnical investigation comprised a gravity survey at a nominal station spacing of 30 m in the high-risk areas, such as the informal settlement, the school and a subsidence zone in the R25 road. The gravity information was used to obtain an indication of the subsurface bedrock profile (ridges and troughs) and to select positions for percussion boreholes.

Seven percussion boreholes were drilled in the informal settlement, varying in depth from 23 to 74 m. An idealised geological section from the south-western to the north-western corner of the area investigated is shown in Figure 6. The Karoo/dolomite interface is shown on the cross-section, as well as a plot from the residual gravity information. Of significance is the variation in depth to weathered dolomite, from 5 to 40 m below surface, and the thick layer of Karoo rock in the two grikes encountered on the site. A relatively thick layer of wad and wad/chert was found immediately above the "solid" dolomite.

In four of the seven boreholes, cavities or semi-cavities were encountered below the chert breccia with accompanying rapid penetration, air loss and poor or no sample recovery. In one borehole a cavity/semi-cavity was found from 54 to 65 m, i.e. a feature 11 m thick. "Solid" dolomite was found at depths varying from 34 to 65 m.

A further six percussion boreholes were drilled by others to investigate cracking in the R25 road immediately east of the informal settlement (Knight Hall Hendry and Associates 1993). A geological cross-section was prepared through the boreholes and is shown in Figure 7. The section shows a palaeo-sinkhole where the depth to residual dolomite suddenly increases. The subsidence and cracking in the road is concentrated on the palaeo-sinkhole, which was reactivated due to the lowering of the water table in the area.

Geohydrological study

The Council for Geoscience (CGS) used aero-magnetic and other information to delineate the Bapsfontein and other groundwater compartments in the area. This information was used to define the study area of 67 km², which included the Bapsfontein compartment of 9 km² and 2 km beyond. The study area is shown on Figure 8. Also shown in this figure is a groundwater compartment, immediately south, and of similar size to the Bapsfontein compartment.

Fairly complete monthly water level data from 1985 to 2005 are available for the three borehole locations shown in Figure 8, in the southern compartment. The information available for this compartment was used to calculate the hydraulic parameters for the Bapsfontein compartment, which included a water balance or reserve determination.

From available satellite images and with assistance from the Department of Water Affairs and Forestry (DWAF), it was established that dolomitic groundwater irrigation in the vicinity of the study area (a 7 km radius from the R25/R50 intersection) had increased substantially since 1991. From 1991 to 1994 there had been a five-fold increase in irrigation under centre-pivot, as detailed in Table 1 (Nel 2004).

The year 1998 in Table 1 is of significance since the National Water Act was promulgated in that year. The new act requires that permits be obtained for water use, including irrigation from underground sources.

The water level for the three boreholes in the southern compartment, shown in Figure 8, was plotted against

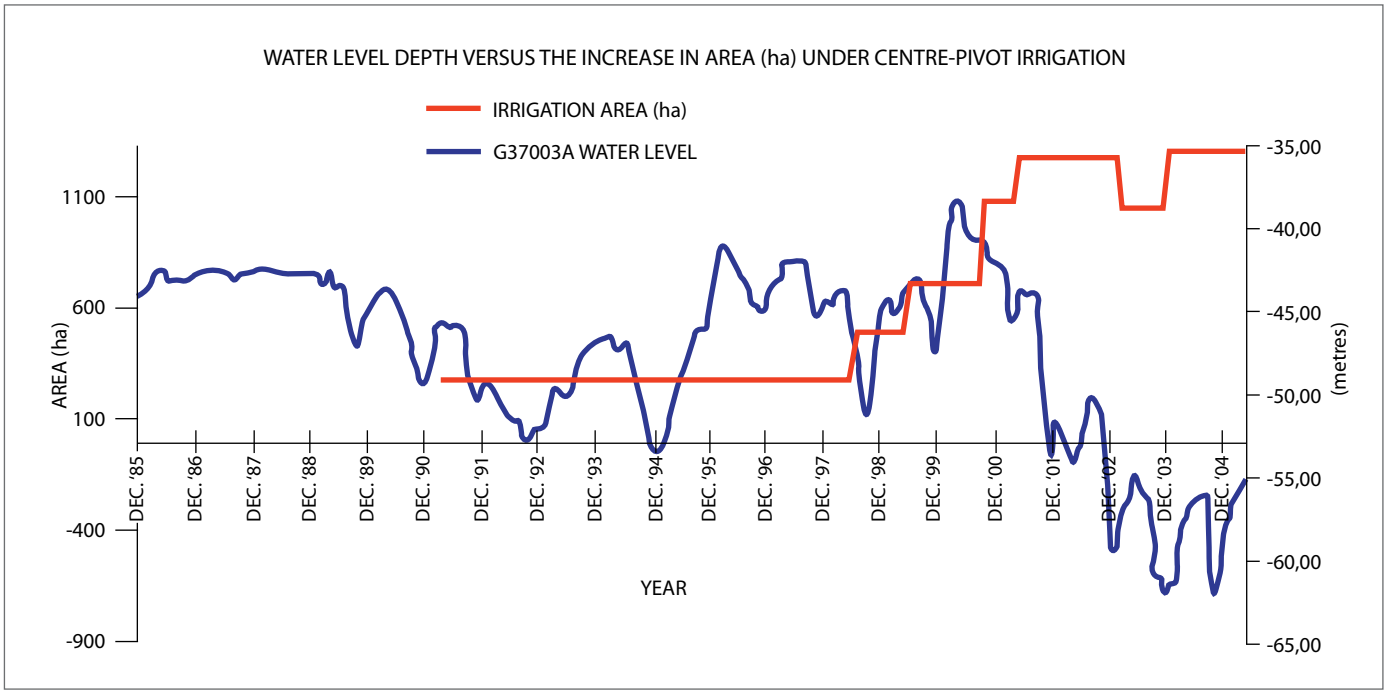
7 Ekurhuleni Bapsfontein dolomite: idealised geological Section B-B at Road R25

8 Bapsfontein and southern compartments showing study area

9 Water level depth vs area (ha) under centre-pivot irrigation (after Nel 2004)

Table 1 Increase in centre-pivot irrigation since 1991

Year	Number of pivots	Area irrigated (ha)
1991	8	273
1998	18	389
2001	37	966
2004	49	1 290



the area under centre-pivot irrigation by Nel (2004). The plot of borehole G37 003A is shown in Figure 9. The water level in this borehole was found to be fairly constant before 1989, showing only small seasonal variations. As pumping increased, the seasonal changes became larger, and between 1999 and 2004 a 20 m lowering of the water table was encountered at this borehole, corresponding to a large increase in centre-pivot irrigation over this period.

The hydrosensus comprised a survey of 153 private-user boreholes in the 67 km² study area. Of these boreholes, 138 were drilled into the dolomite aquifer and 15 into the Karoo aquifer overlying the dolomite. In addition, the information from the DWAF monitoring boreholes, dating back to 1985, was used. The difference in water level on either side of the dykes that delineate the Bapsfontein compartment further confirmed the boundaries of the compartment. A number of boreholes drilled in the dolomite, ranging in depth between 75 and 140 m were reported to have dried up. The same was found for the shallower boreholes drilled into the Karoo aquifer.

The boreholes that are considered to tap the Bapsfontein dolomite aquifer were estimated to abstract between 2,3 and 2,5 million m³/annum, as shown in Table 2. There are four major users, namely Clear Water Farms, C du Plessis, Top Turf Lawns and the informal settlement.

Clear Water Farms are registered under the DWAF Water Authorisation and Registration Management System (WARMS – information from Hobbs Khulani Joint Venture 2004) for 20 boreholes with an authorised abstraction volume of 2,7 million m³/a and C du Plessis, with four registered boreholes, is authorised to abstract just over 1,1 million m³/annum. Most of the Du Plessis water is abstracted from the southern compartment. This information shows that the two major users responsible for 96% of the water pumped from the Bapsfontein compartment abstract less water than authorised by the WARMS system.

The recharge and other hydraulic parameters for the southern compartment, for which fairly complete water level data are available, were calculated using the Exel Recharge Programme by the geohydrologists.

By making use of the parameters from the southern compartment and assuming a 10% recharge (due to numerous sinkholes in the Bapsfontein compartment), the groundwater reserve for the latter compartment was calculated and is shown in Table 3. This table indicates a negative water balance for the Bapsfontein dolomite water compartment of 1 400 000 m³/a.

Table 2 Summary of water use from the Bapsfontein compartment

Major users	Area irrigated (ha)	Water use (m³/a)	% of Total
Clear Water Farms	400	2 000 000 (to 2 250 000)	87,3
C du Plessis	287*	210 000	9,2
Top Turf Lawns	40	44 000	1,9
Informal settlement	–	32 000	1,4
Other users	–	4 000	0,2
Total		2 290 000	

*Not all water from Bapsfontein compartment.

Table 3 Groundwater reserve determination for the Bapsfontein compartment

Aspect determined	m³/a
Recharge	600 000
Groundwater inflow into area	500 000
Groundwater outflow from area	0
Subtotal	1 100 000
Groundwater abstraction including basic human need of 40 000 m³/a	2 500 000
Deficit	1 400 000

10 Relationship between Karoo overburden thickness, depth to water table level and sinkholes (June 2005)

Assuming that the safe yield from a small compartment with no natural discharge equals the recharge, then the safe yield for the Bapsfontein compartment is about 600 000 m^3/a. With an estimated abstraction of 2,3 to 2,5 million m^3/annum, an overutilisation of 3,8 to 4,2 times the safe yield is taking place at present in the Bapsfontein compartment. The above statement is graphically illustrated in Figure 9.

DISCUSSION

The water table levels measured in the Bapsfontein area over a considerable time clearly show that dewatering of the dolomite is taking place. This is graphically illustrated in Figure 9 where water level information measured over 20 years in a DWAF borehole is plotted against the increased area under centre-pivot irrigation in the area. In this borehole the water level dropped by 20 m, from 40 to 60 m, below ground level between 1998 to 2004. In this period the area under centre-pivot irrigation increased by 3,3 times. In the Bapsfontein compartment the water has been drawn down between 80 m to 120 m plus below ground level, resulting in many of the shallower boreholes in both the Karoo and dolomite aquifers drying up. This has resulted in many inhabitants in the area being deprived of their potable water where water was not being supplied by the authorities at the time of this study.

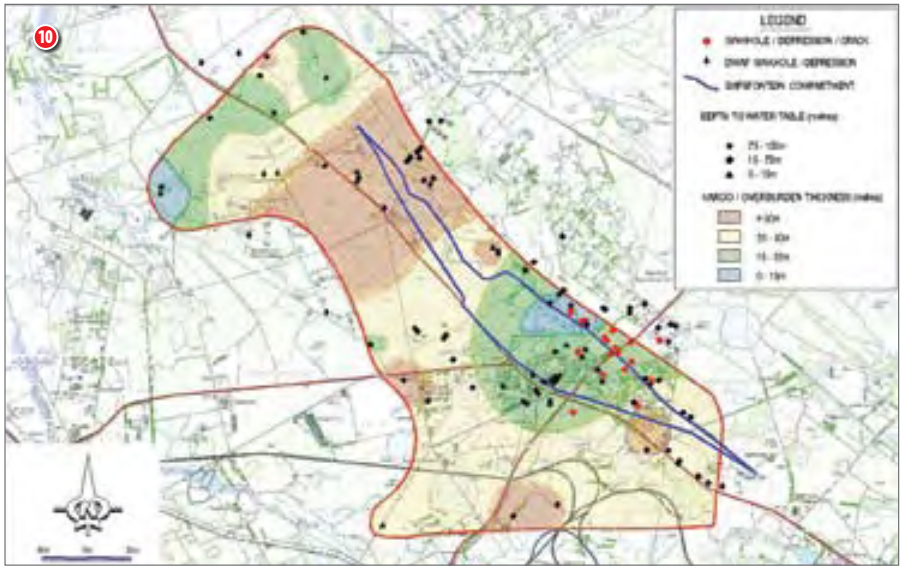
The dewatering is the direct result of excessive pumping from the dolomite aquifers in the area for centre-pivot irrigation. The area under centre-pivot irrigation increased nearly five-fold between 1991 and 2004. The 9 km^2 Bapsfontein compartment is being overutilised in the order of four times the safe yield of the compartment. If this trend continues, the dolomite water compartment will soon be destroyed.

The two main users of the Bapsfontein compartment water are responsible for 96% of the extraction. It is important to note that they use less water than authorised by the DWAF WARMS system.

In addition to depriving the inhabitants of their potable water, the lowering of the water table is causing widespread instability in the area. An inspection near the informal settlement shortly after the development of the sinkhole on 28 January 2004 revealed a further 18 sinkholes, depressions or cracks in various stages of development in the immediate vicinity of Bapsfontein. If the dewatering continues, the instability will increase as illustrated in Figures 4 and 5.

In studying the logs of the DWAF monitoring boreholes and the percussion boreholes drilled for this investigation, it was observed that the entire study area is covered by Karoo rocks. This is contrary to the geological map of the area, which shows that only 50% of the area is covered by Karoo rocks. The thickness of the Karoo rocks varies from less than 25 m around Bapsfontein to over 100 m towards the north-western boundary of the study area. In addition, the Karoo rocks thin out to less than 25 m west of the north-western boundary.

By combining the depth to water table with the thickness of the Karoo rocks over dolomite, it was found that the sinkholes or depressions around Bapsfontein are concentrated in an area where the Karoo cover is 25 m or less and the water table has been lowered to as much as 120 m below ground level (Figure 10). Not only is the area unstable, but the ground water is also prone to pollution from pit latrines in the densely populated informal settlement when instability occurs.



CONCLUSION

The Bapsfontein dolomite water compartment is being dewatered at an alarming rate due to excessive extraction from the aquifer. The extraction is estimated at four times the recharge rate. It is important to note that the two main users, responsible for 96% of the extraction, are using less water than authorised.

The dewatering is causing:

- widespread instability in the area in the form of sinkholes and dolines, endangering lives and damaging infrastructure
- the drying up of boreholes, depriving the inhabitants of their source of potable water
- pollution of the groundwater from pit latrines and contaminated storm water where instability occurs in densely populated areas

The author is of the opinion that this is cause for grave concern and requires intervention at the highest level of the DWAF and other authorities. The question that needs to be answered is: Why are a few users in the area authorised to pump more water from the aquifer than its sustainable yield?

ACKNOWLEDGEMENTS

- The Client, Ekurhuleni Metropolitan Municipality, in whose area Bapsfontein falls.
- Louis van der Walt of Jasper Müller and Associates who conducted the geohydrological studies.
- Rick Piovesan of Jones and Wagener who assisted the author with the Bapsfontein study.

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Text: **Allan Parrock**
Geotechnical Principal
ARQ Consulting Engineers (Pty) Ltd
alan@arq.co.za


Gerrie van Jaarsveld
Engineering Geology Principal
Rockland Geoscience (Pty) Ltd
gerrie@rockland.co.za


Ian Deale
Engineering Geology Principal
Rockland Geoscience (Pty) Ltd
ian@rockland.co.za

Medupi Power Station project: Quality control procedures for foundations

INTRODUCTION

One of the components of the Medupi project is a coal-fired power station being constructed by Eskom at Lephalale, Limpopo province. The station has a planned total installed capacity of 4 800 MW, which will be delivered through six units with an individual capacity of 800 MW each. When completed, it will be the fourth-largest coal-fired power station in the world. Some 400 foundations, ranging in size from a few square metres to elements of hundreds of square metres, are at present being constructed as part of the project (see Figure 1).

In order to ensure the adequacy of the foundation rock, each foundation is subjected to a geotechnical mapping procedure to

1 Foundation cleaning in foreground and construction of Unit 6 air-cooled condenser columns in the background
2 Final cleaning of foundation footprint using compressed air

ensure that it meets the foundation design criteria. This is especially important in establishing potential blast-induced degradation of the foundations due to rock blasting. Given the topography of the site and the geometry of the foundations and services, significant rock blasting has been undertaken during the bulk earthworks phase of the project. This article details the design requirements and illustrates the mapping procedures employed on site.

PROCEDURE

Following the completion of blasting and mucking operations to the planned foundation levels, individual foundation footprints are demarcated in such a way that the height of the area is known. A preliminary evaluation is carried out by the Engineering Geologist (EG) or Geotechnical Engineer (GE) based on a visual examination. If the EG or GE is of the opinion that the site is *not* suitable, appropriate remedial measures are communicated to the production personnel and fractured and weathered material is removed.

If the preliminary evaluation is positive, the area is finally cleaned via pick, shovel and broom work, accompanied by high-pressure air jetting (Figure 2).

The edges of the area to be mapped are agreed on with the production personnel who demarcate these areas physically on site by means of pegs, stakes or paint markings.

Two lines are selected diagonally across the block at approximately right angles to each other. The tape is laid along

these lines and the attributes of each and every joint, natural as well as blast-induced, crossing the tape are recorded as per the requirements of the *Geotechnical Mapping of Foundations Record*. These attributes include the following:

- Distance along tape (m)
- Joint description, including dip (°), dip direction (°), joint type and length of joint (m)
- Waviness of joint, including roughness and amplitude (mm)
- Infill description, including thickness (mm), type and consistency
- Estimated strength or hardness of rock (based on the AEG-SAICE-SAIEG Classification for Rock Logging, 2001).

The mapped area is sketched to illustrate the location of major features, including joints, blastholes and topographic steps, with notes on where rock core samples are to be taken. Individual photos are taken of each discontinuity recorded, as well as panoramic photos providing an overall view of each foundation footprint. Rock cores are tested for Uni-axial Compressive Strength (UCS) and Point Load Index (PLI) (Figures 3, 4, 5 and 6).

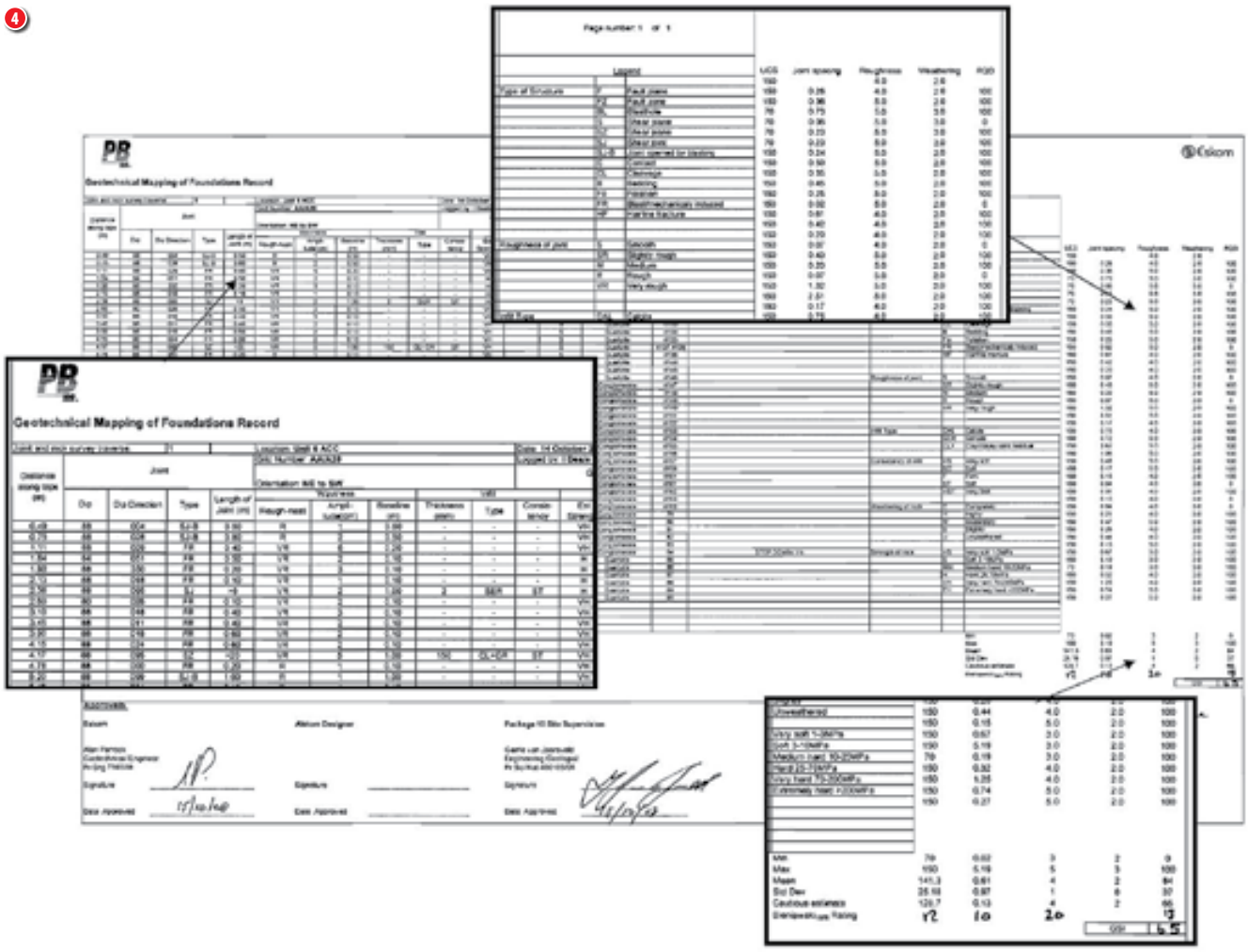
The data collected during the geotechnical mapping are used to perform a rock mass classification. This classification allows numerical analysis through assigning values to descriptive terms and utilising other numerical data after some manipulation

ANALYSIS

The data collected during the geotechnical mapping are used to perform a rock mass classification. This classification al-

3 Rock cores being retrieved from foundation footprint using core drills
4 Example of a typical *Geotechnical Mapping of Foundations Record*
5 Hand sketch of foundation footprint

4



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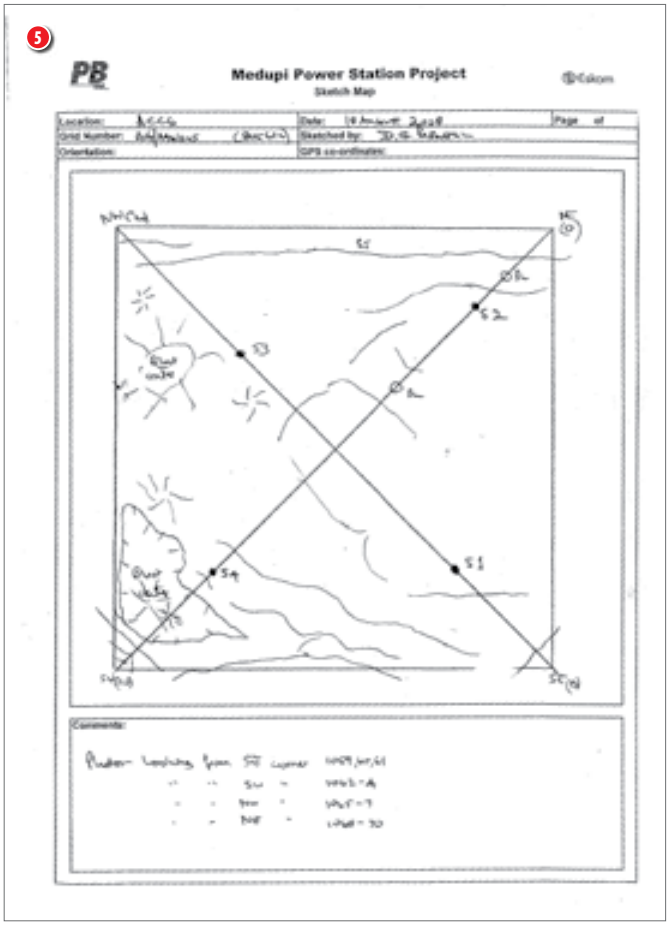

lows numerical analysis through assigning values to descriptive terms and utilising other numerical data after some manipulation. Mean, standard deviation and cautious estimate values are assigned to UCS, joint spacing, joint roughness and Rock Quality Designation (RQD) such that an estimate of the Geological Strength Index (GSI) may be made.

Bieniawski's (1976 and 1989) Rock Mass Rating System (RMR), calibrated against Barton et al's (1974) Q-System and Laubscher's Mining Rock Mass Rating System (MRMR) (1994), are used to derive an estimate of Hoek and Brown's (1988) GSI using well-established relationships between the various systems.

The results from the UCS and PLI tests are used to verify the strength of the rock estimated during geotechnical

mapping. Measured orientations of the discontinuities are plotted on a stereo net (Figure 7) and rose diagram (Figure 8), depicting the orientation of major joints or fractures.

INSPECTION

Each week a trip to the site is undertaken by the GE and the data that have been prepared by the various mapping engineering geologists are reviewed. In addition, each base is visually inspected before the GSI value is calculated by hand to ensure a modicum of personal contact.

DESIGN CRITERIA

The following design criteria were established:

- Minimum allowable GSI = 44
- Cautious Estimate (CE) of rock hardness > 50 MPa
- CE of weathering classification to be "slightly weathered" or better
- CE of RQD >50
- CE of joint spacing > 200 mm
- CE of joint roughness > "Medium"

APPROVAL

If all facets satisfy the above design criteria, the foundation is approved by the GE, co-signed by the Site Supervisor and Design Engineer, for levelling or placement of blinding concrete.

PERMISSION

Eskom's permission to publish this article is acknowledged with thanks.

Each week a trip to the site is undertaken by the GE and the data that have been prepared by the various mapping engineering geologists are reviewed

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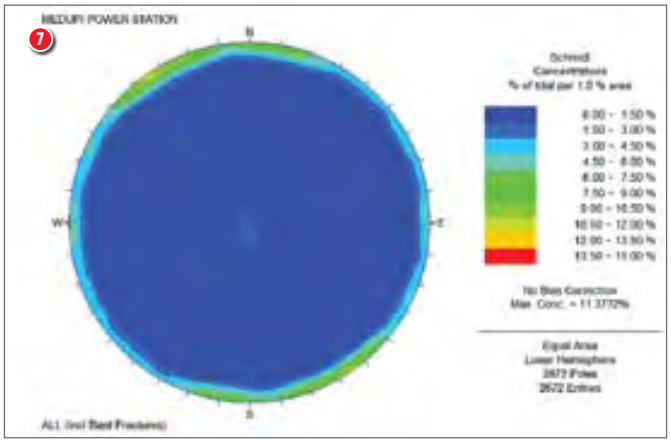
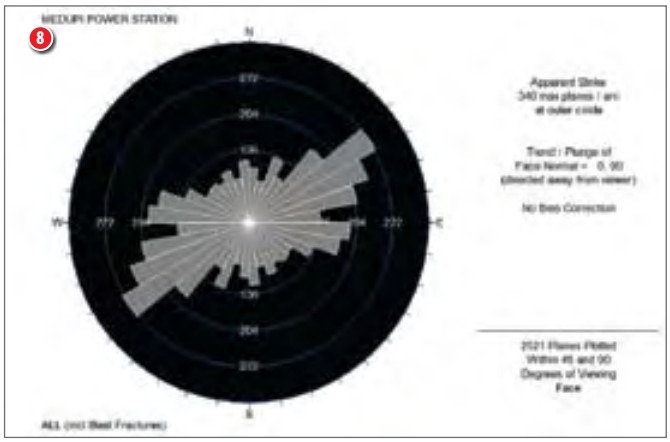
6 Photo showing foundation footprint

7 Stereo net presenting joint or fracture orientations

8 Rose net depicting strike directions of main joint or fracture orientations





Text **Trevor Green**
Verdi Consulting Engineers
trevor@verdicon.co.za

Integration of quality control and base improvement in auger piles

INTRODUCTION

Given the correct soil conditions, open-hole auger piling cast in situ remains one of the simplest, quickest and cheapest piling methods available. This pile type is commonly used in South Africa as it is ideally suited to the partially saturated soils and relatively shallow rock encountered over much of the country. These advantages are, however, diminished by the uncertainties inherent in auger piles with respect to the transfer of load from the base of the pile to the in situ soil or rock.

With newer, more powerful auger rigs now available, it has become economically feasible to install auger piles into much stronger and more competent rock than was previously possible. To fully utilise the capacity available in this rock, a methodology is required to ensure a clean contact at the base of the pile. Since the traditional method of hand cleaning and inspecting the base of the pile is virtually impossible from a health and safety perspective, alternative methods are required which will both guarantee this "clean" contact and maintain the simple and economical nature of auger piles.

This article explores potential methods of improving the performance of such piles, with regard to compression, tension and shear, with specific reference to piles recently installed for the main events stadium for the 2010 FIFA Soccer World Cup in South Africa (i.e. the Soccer City Stadium, see Figure 1). Using experience gained from this and other recently completed piling projects, a range of integrated quality control and base-improvement methods are proposed which may further improve the value of auger piles cast in situ.

CURRENT LIMITATIONS OF AUGER PILES

Although auger piles have many advantages, they do suffer from two distinct limitations:

- The sides of the auger hole are unsupported while it is being drilled, and are therefore prone to collapse, particularly in loose soils or below the water table.
- The design of the piles is complicated by uncertainties with regard to the transfer of load from the base of the pile to the in situ soil or rock.

Typically speaking, open-hole auger piles should not be used in ground that is prone to collapse or below the water table. Therefore the first limitation is generally avoidable, provided a thorough geotechnical investigation is completed before piling.

The second limitation is much more complex. Even in reasonably stable soil or rock profiles, slight collapse of the sides of the auger hole is possible, and not all the loose material at the base of the hole, generated by the augering process, can be removed using mechanical means (i.e. cleaning using the auger flight or a cleaning bucket). As a result, it is current design practice to downrate the end bearing component of auger piles significantly or, by adopting an even more conservative approach, to

ignore the end bearing completely.

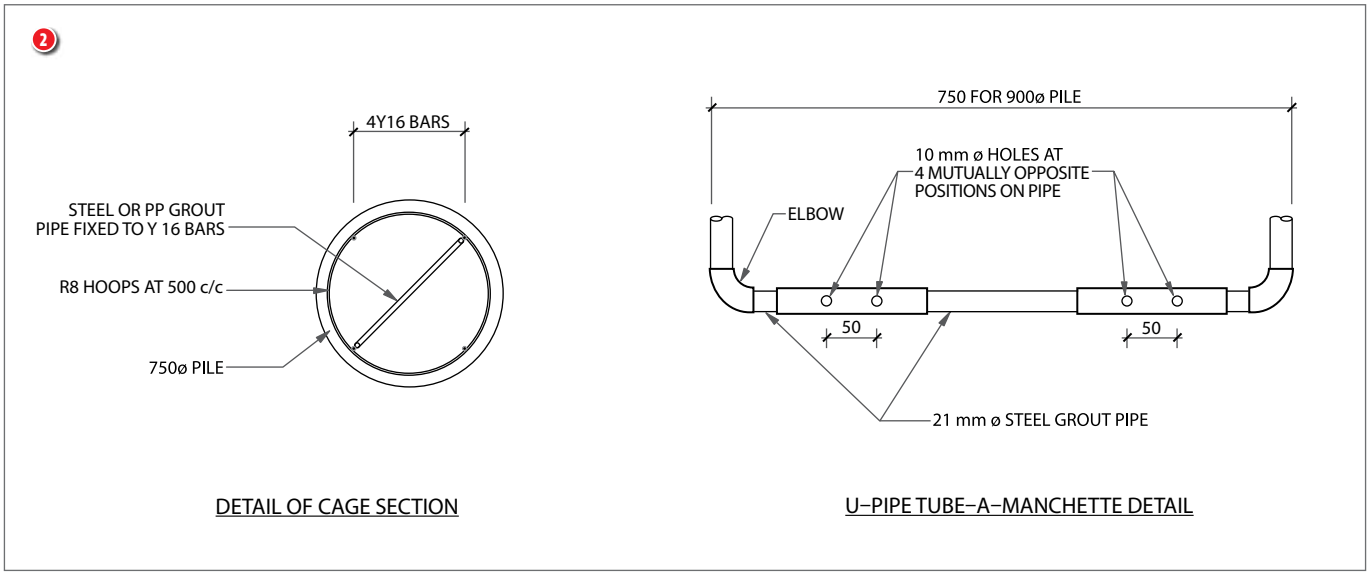
Byrne et al (1995) state that "The safe working loads of auger piles founded on competent material can be calculated using a shaft stress of 3 MPa for pile diameters less than 600 mm and 6 MPa for pile diameters of 600 mm and greater." The assumption is that piles of larger diameter can be cleaned carefully by hand. They go on to state that the maximum shaft stress for auger piles socketed into rock may be increased to 8 MPa, provided the socket and the base of the pile are cleaned out and inspected to ensure the competency of the rock at founding level.

Amendments to the Occupational Health and Safety Act (Act 85 of 1993) and more recently the Construction Regulations (2003) have all but prohibited the practice of sending workers to the bottom of an auger hole to clean and inspect the base. Although it can still be done, it requires a written assessment by a professional engineer or engineering technologist. This is impractical where a large number of piles are required, and would expose the professional engineer or engineering technologist to excessive professional liability.

If the piles are not cleaned by hand, reducing the allowable shaft stress on a pile to only 3 MPa would significantly reduce the effectiveness and economy of this pile type. The use of modern cleaning buckets in certain conditions has been moderately successful, and can allow the maximum allowable

1 Aerial view of the Soccer City Stadium taken towards the end of 2008

2 Typical details showing tube-a-manchette grouting system at pile base



shaft stress to be increased to 4 MPa or even 5 MPa. However, given the much more powerful auger rigs now available, capable of augering into rock strengths greater than 10 MPa, designing piles that are loaded only to 4 or 5 MPa would seem overly conservative.

This is especially true given that the shaft of the pile, from a structural perspective, can carry significantly more load. Again according to Byrne et al (1995), the ultimate axial load, N, applied to a pile should not exceed:

$$N = 0{,}4\, f_{cu}\, A_c$$

Therefore the allowable shaft stress in a pile (using a factor of safety of 1,50) is limited to:

$$\sigma_{shaft} = 0{,}4\, f_{cu}/1{,}50$$

In its simplest form, this means that the allowable shaft stress in a pile should be limited to 27% of the concrete strength; this equates quite neatly to BS8004 which limits the average compressive stress in the pile shaft at working load to 25% of the characteristic cube strength at 28 days. Using this criterion and typical characteristic concrete strengths of 30 or 40 MPa, the allowable shaft stress of the piles can be pushed as high as 7,5 or 10,0 MPa, far in excess of current design practice.

Another major constraint in fully utilising the end bearing resistance of auger piles is the basic strain incompatibility that exists between the side shear resistance and the end bearing. In soils, the development of the side shear resistance can be fully developed within displacements equivalent to 0,5 to 1,0% of the shaft diameter, while displacements equivalent to 10 to 15% of the shaft diameter are required to fully mobilise the end bearing. Thus the end bearing requires between 10 and 30 times more displacement to develop than the side shear.

Even if such large displacements were permissible, it can be further argued that these greater displacements would be detrimental to the side shear resistance. Strain incompatibility is less of a problem when the pile is founded on rock, but once again a clean contact is required between the base of the pile and the rock.

Another major constraint in fully utilising the end bearing resistance of auger piles is the basic strain incompatibility that exists between the side shear resistance and the end bearing

PROPOSED METHOD OF BASE IMPROVEMENT

The challenge is therefore to provide a means of ensuring a clean contact between the base of the pile and the in situ soil or rock. Two methods are proposed for improving the performance of auger piles. The application or choice of these methods will depend on the unique circumstances of the project being considered. Base-grouting, however, will typically be used for simpler piles (i.e. piles of smaller diameter acting mainly in compression). Steel tubes on the other hand, will be used for more complex piles (i.e. piles of larger diameter required to resist tension and shear forces, in conjunction with compression loading).

Base-grouting (simple piles)

The base-grouting process entails the installation of a grout-delivery system, typically attached to the reinforcing cage and lowered into the hole before the pile is cast. The shaft of the pile is constructed

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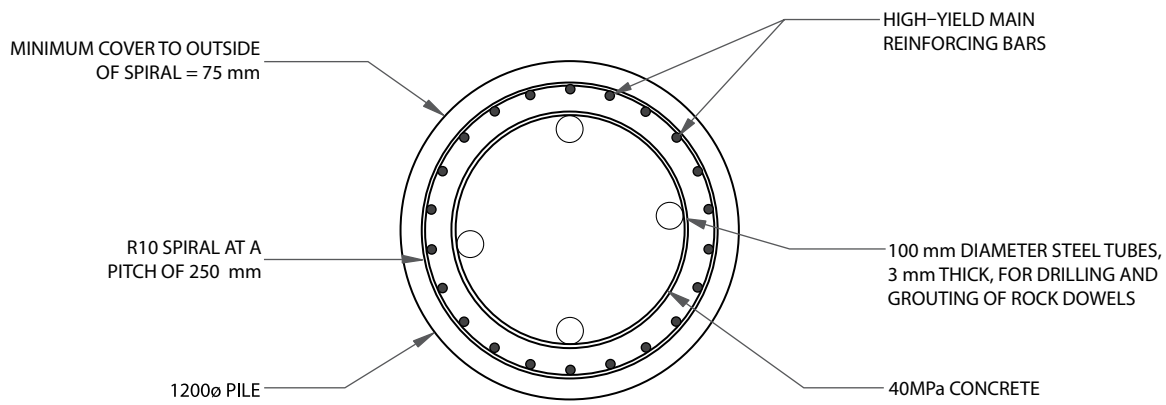


as usual and the grout is injected through the base of the pile at high pressure once the concrete shaft of the pile has reached sufficient strength. The density and compressive strength of any loose material at the base of the pile are significantly improved through this application and any voids are filled with grout.

This method of improvement is best suited to piles of smaller diameter (i.e. piles with diameters between 450 and 900 mm), and those piles acting in simple compression. A number of different systems are available for delivering the grout to the base of the pile. One such method is illustrated in Figure 2.

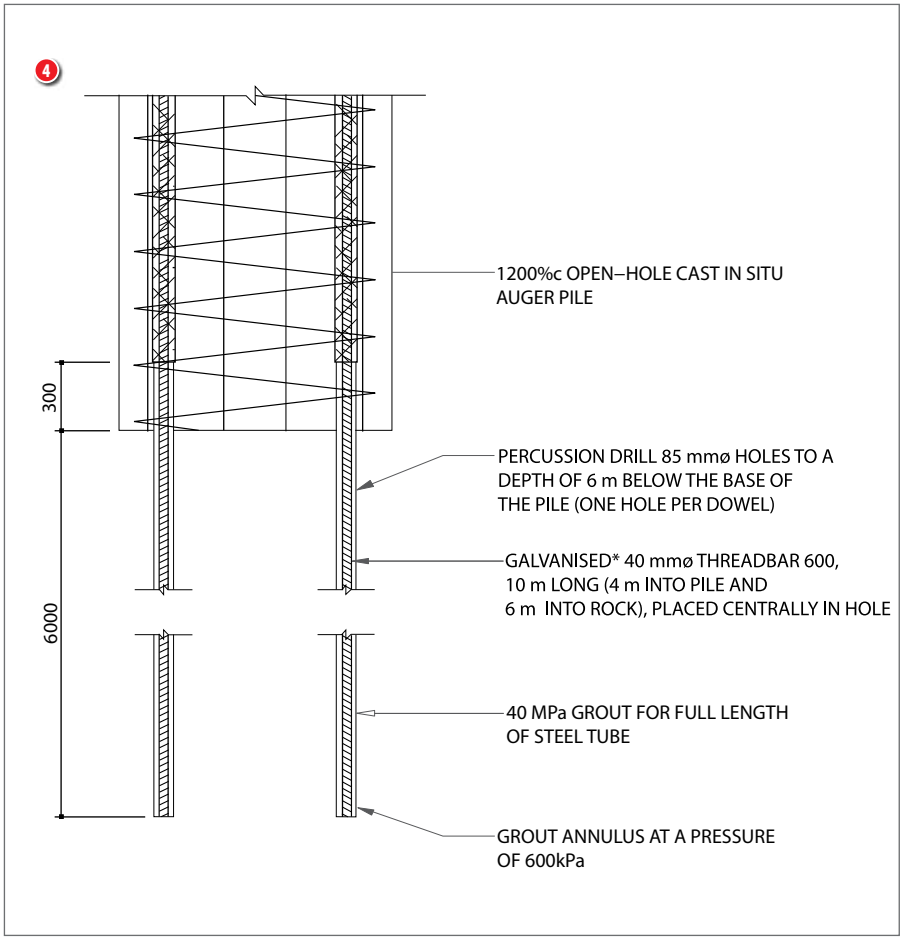
Essentially, the grout is pumped through the pipes from the top of the pile, and out the tube-a-manchette at the base of the pile. The flow of grout is continued until either a certain pressure is reached, generally between 2 and 3 MPa, or a certain volume of grout has been pumped. If the required pressure is not reached, the grout pipes are flushed and the grouting procedure is repeated at a later date. Movement of the head of the pile is also monitored to ensure that the pressure at the base of the pile does not exceed the side shear resistance of the pile shaft. The total volume of grout required to achieve the desired pressure is recorded, providing an empirical measure of the success of the means originally used to clean the base of the pile.

The use of base-grouting has a number of advantages. Firstly, any deficiencies in the cleaning process are addressed by improving the material at the base of the pile. Secondly, by applying the grout under pressure, the base of the pile is effectively preloaded, thus greatly improving the compatibility between the side shear strain and the end bearing. Lastly, the base-grouting process can be used as a form of quality control, assessing both the effectiveness of the cleaning process and, by using a high-pressure grouting system, even testing the side shear resistance of the pile.

It should also be noted that the use of base-grouting is relatively inexpensive in comparison with the cost of the pile itself. For this relatively small additional cost per pile, the pile capacities

3 Typical section through a pile showing the steel tube arrangement

4 Typical detail showing dowel installation through the base of the pile



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can be increased from 4 or 5 MPa up to 7 or 8 MPa, thus providing significant savings for a piling project.

Steel tubes (complex piles)

For larger piles (i.e. piles with a diameter >1 200 mm), the use of base-grouting is no longer appropriate. For such large pile diameters, it can no longer be assumed that the grout will penetrate through the entire area beneath the base of the pile. In addition, piles of a larger diameter require an even more stringent quality assurance process and are often required to carry tension and shear loads in conjunction with conventional compression loads.

When the pile is being installed, a number of steel tubes, 100 mm in diameter, are placed in a circular arrangement into the augered hole, along with the steel reinforcing. These tubes may be attached to the main reinforcing cage or placed in a separate "inner" cage as shown in Figure 3, and extend from natural ground level to approximately 300 mm above the base of the pile. Once the pile has been cast, these tubes then provide access to the base of the pile which can be used for a number of applications, as will now be discussed.

Quality control

The tubes can be used to conduct cross-hole sonic integrity testing on the piles, to check for voids or anomalies in the concrete. In addition, using a rotary core drilling rig, a core can be extracted that will incorporate the base of the pile shaft, the pile/rock interface and the rock directly below the base of the pile. By examining the pile/rock interface, an assessment can be made as to the success of the cleaning process and samples can be extracted of the rock below the base of the pile to determine the strength of the rock.

Base improvement

Should the interface between the pile and the rock prove inadequate, the steel tubes then provide access for base-grouting the pile. The voids provided by the steel tubes are extended using percussion drilling and air is used to flush out any loose material below the base of the pile. Grout is then pumped at high pressure to the base of the pile. Given the much stronger configuration and the fact that only a portion of the pile base can be grouted at a time, much higher pressures can be applied than are used for traditional base-grouting.

Tension and shear dowels

Utilising the voids created by the steel tubes, a percussion drill can be used to drill through the base of the pile and into the rock below. Steel dowel bars can then be grouted into the rock, providing additional shear and tension capacity for the piles. A typical dowel arrangement is shown in Figure 4.

CASE STUDY

The Soccer City Stadium in Johannesburg, formerly known as the

5 Preparation for one of the pile caps of the 12 shafts
6 & 7 Steel tubes and pile reinforcing on one of the shaft piles

When the pile is being installed, a number of steel tubes, 100 mm in diameter, are placed in a circular arrangement into the augered hole, along with the steel reinforcing

FNB Stadium, is at present being upgraded at a cost of R2,1 billion. This stadium is earmarked to host five first round matches (including the opening match), one second-round match, a quarter final and the final of the 2010 World Cup. The new stadium will have an official capacity of just over 90 000, making it the biggest stadium ever to be used for the Soccer World Cup.

Aside from completely renovating the existing stadium, the boxes and upper seating, formerly only on the western side of the stadium, are being extended around the entire perimeter. In addition, a circular steel roof is to be added, giving the stadium the distinctive calabash shape envisioned by the architects. Due to its round shape, the roof will be largely independent of the stadium structure and will be supported by 12 reinforced concrete "shafts". These shafts and the loads generated by the roof have resulted in exceptionally high loads being transferred to the foundations of the shafts, supported by auger piles of 1 200 or 1 500 mm in diameter.

Although many of the piles carry large compressive loads, perhaps the most daunting aspect of the pile design was the exceptionally high tension loads. By way of an example, a single shaft foundation is required to carry up to 13 000 kN of tension, combined with 6 000 kN of shear and 125 000 kNm of moment. Given the limited space, it was only possible to install a maximum of 12 piles per shaft foundation with the result that some piles were expected to carry up to 5 800 kN of tension.

Unfortunately, the rock strength and depth across the site is highly variable, and therefore only limited capacity can be safely generated in the rock socket. In order to accommodate these massive loads, it was decided to anchor the piles into the sandstone bedrock using dowel bars installed through the base of the pile. Between four and eight 100 mm steel tubes were cast into the pile to a depth of 300 mm above the pile/rock interface (see Figures 5, 6 and 7).

Once the concrete had reached sufficient strength, these holes were extended into the rock below using percussion drilling techniques. Y40 Threadbar 600 dowel bars, 10 m long, were then grouted 6 m into the rock, with 4 m projecting into the pile, generating just over 500 kN of tension capacity per dowel.

The steel tubes were also used to conduct cross-hole sonic logging testing and to check the pile/rock interface.

CONCLUSIONS

Restrictions imposed by new health and safety legislation have limited the efficiency and potential capacity of open-hole auger piles cast in situ. This can be overcome through the use of either base-grouting or steel tubes to maximise the capacity generated by any single pile.

Base-grouting should be used for smaller piles, improving the material at the base and preloading the pile base to improve the compatibility between the displacements necessary to generate the side shear resistance and the end bearing resistance. Steel tubes can be fitted in piles of larger diameter, which can be used to facilitate quality control, improve the conditions at the base of the pile or install dowels to improve the tension or shear resistance in the pile.

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Text **Des Lange**
Technical Manager
Maccaferri Southern Africa
des.lange@maccaferri.co.za

Effective performance monitoring
of reinforced soil and other related structures

Maccaferri Southern Africa has maintained its place as a leading solution provider in the broad field of civil and environmental engineering. The range of solutions includes: mass gravity gabion retaining walls, reinforced soil embankments, slope stabilisation, erosion control, bank protection, channel lining, scour protection, wetland rehabilitation, rockfall protection and basal reinforcement

1 Location of the ramp inside the pit at the Potgietersrus platinum mine

MOST OF THE PRODUCTS developed and supplied by Maccaferri for use in these solutions have a combined purpose, namely structural integrity and enhancement of the environment. It follows then that if the design, the installation of the product and the vegetation are adequate and maintenance is carried out as recommended, the intended purpose of the solution will be realised. In certain applications of this type, effective establishment and development of the vegetation or other naturally formed external covering will lead to improvement of both the structural characteristics and the aesthetic appearance.

These solutions do not incorporate the common construction materials such as timber, concrete or steel, and neither are they based on standard or typical civil engineering construction procedures. As

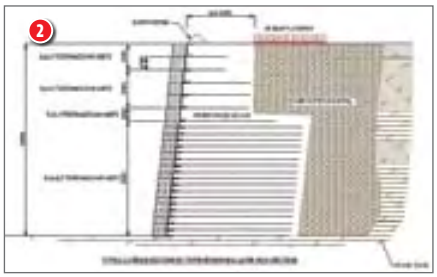
a result, it is essential that both the client and the team from Maccaferri supplying the solution agree on some form of plan to monitor the performance of the project during the construction stage, as well as during the working life of the structure. The monitoring that takes place during construction will largely ensure that the appropriate methods and specifications are complied with.

This article focuses on the importance of effective client participation during planning, design, construction and post-construction stages. Case studies of projects in which client involvement was positive are given to illustrate the processes followed and the lessons learned.

POTGIETERSRUS PLATINUM MINE

A Terramesh™ System retaining wall and embankment were constructed in place of an existing ramp formed by the in situ ore-bearing rock in the open pit of this Anglo Platinum mine (Figure 1). Two of the most important requirements of the design were the ability of the structure to withstand the effects of blasting in the adjacent rock formation over a 10-year service life and the provision of a stable ramp with a 10% gradient to support the 390-ton dump trucks.

The 1,0-m-wide rock-filled gabion-type face of the near-vertical retaining wall in this Terramesh™ System had a maximum height of 15 m and a length of 150 m (Figure 2). The structural fill, consisting of waste crushed rock, was reinforced with mesh panels, 10 m long, at 0,50 m vertical intervals. The wall and ramp were constructed over nine weeks at a cost at least 40% lower than that of alternatives, such as a reinforced concrete retaining wall or a reinforced earth structure with precast concrete face panels (Figure 3).



Blast-control methods

Trial blasts adjacent to the site were carried out to determine the limits for the charge mass, frequency, peak particle velocities (PPVs) and safe distance from the structure. These trials enabled the project team to decide that blasting should not be carried out closer than 8,0 m from the base of the wall, and a limit of 320 mm/s was set for the PPV; this limit can be compared with the safe limits for other situations as shown in Table 1.

Monitoring programme

Blasting and excavating of the pit below the level of the Terramesh™ System wall took place after the wall had been completed, from 9 January 2001 to 20 March 2003. Displacement of the wall and development of cracking in the surrounding rock on the slope were monitored using laser optical measurement and accelerometer units installed in the face.

Performance of the structure

In some instances the particle velocities were as high as 2 000 mm/s and the recorded, accumulated vertical displacements of three targets on the wall at the end of the period were 110, 165 and 240 mm respectively. The gabion wall and reinforced soil embankment proved to be adequately flexible and durable in this situation.

Conclusion – Potgietersrus platinum mine

This project proved to be a good representation of a novel ramp-construction method, together with sound risk-management techniques for open-pit mining.

MARIKANA PLATINUM MINE

Maccaferri SA (or African Gabions as it was then known) produced a design in January 2005 for a vertical tip wall to enable access to a 20-m-high loading bin at the primary crusher at Marikana platinum mine (Figures 4 & 5). The Terramesh™ System was proposed for the main and wing walls. The wall was

Table 1 Safe PPV limits for various situations

Application	Velocity limit (mm/s)
Green concrete	3 – 5
Water-retaining structures	12
Masonry buildings	50
Concrete block structures	200
Concrete walls	250
Rockfall in unlined tunnels	300
Cracking in rock	650

founded 7 m below the original ground level, on approximately 6 m of stiff sandy silt (or alluvium). Below this alluvium, the material comprised stiff clayey silt (residual norite) with very soft anorthosite at 16 m depth. The presence of an expansive layer of material at the base of the wall meant that excessive settlement and deformation were expected. Future dewatering operations planned by the mine would also add to the total and differential settlements. An independent geotechnical investigation showed that as much as 200 mm of settlement could be expected in the short term.

Plan to monitor settlement

The tip wall and the reinforced soil embankment took 13 weeks to construct. During this process, there was careful supervision to ensure that the reinforced layers were placed and compacted to within specification and that the correct vertical alignment of the rock-filled mesh units in the face was maintained. A total of eight survey targets were bonded to the face of the wall at strategic positions so that movement of the wall could be carefully monitored. Maccaferri decided that precise observations would be done on a yearly basis for the first five years of the service life to determine any settlement, and undertook to fund this exercise independently.

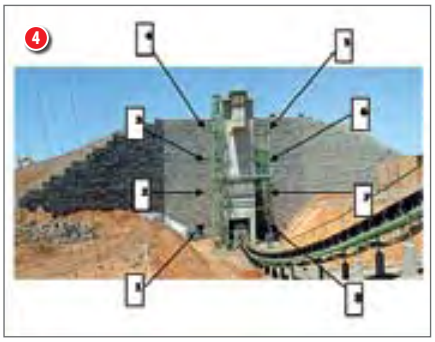
Survey results

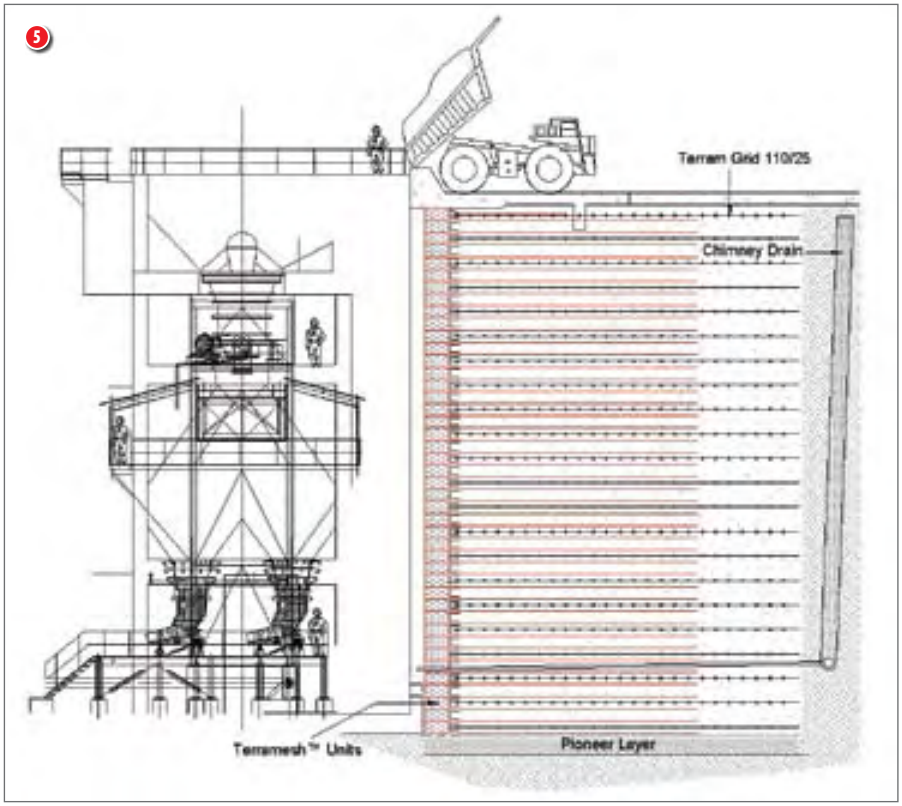
The first year's results showed that the wall had settled by at least 160 mm from its initial horizontal position. In addition to this, as expected, the face of the wall had rotated in the vertical plane to the extent that the wall had moved at least 40 mm away from the crusher. In other words, this rotation measured 0,12° backwards. The final results at the end of the five-year period showed that the progressive total settlement had reached an average of 221 mm, with a rotation of 80 mm or 0,19°. The graph in Figure 6 illustrates this trend and also shows how the rate of consolidation of the underlying soil formation is "flattening out".

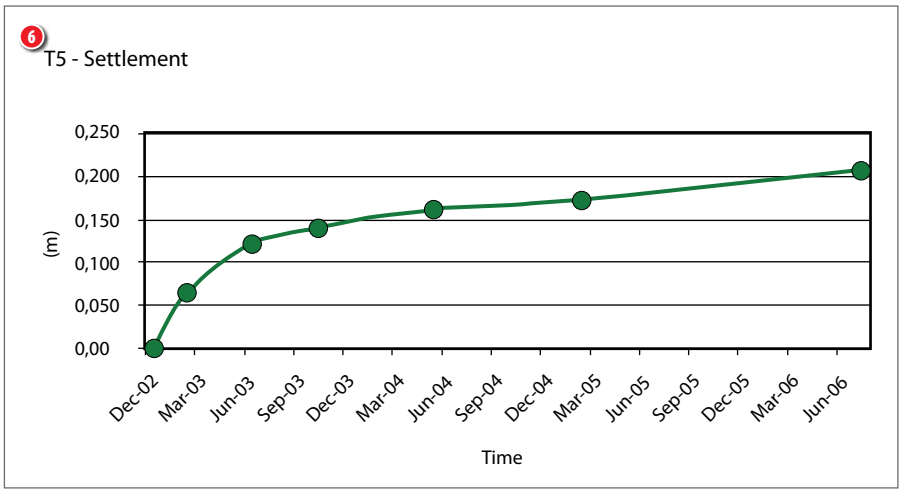
Conclusion – Marikana platinum mine

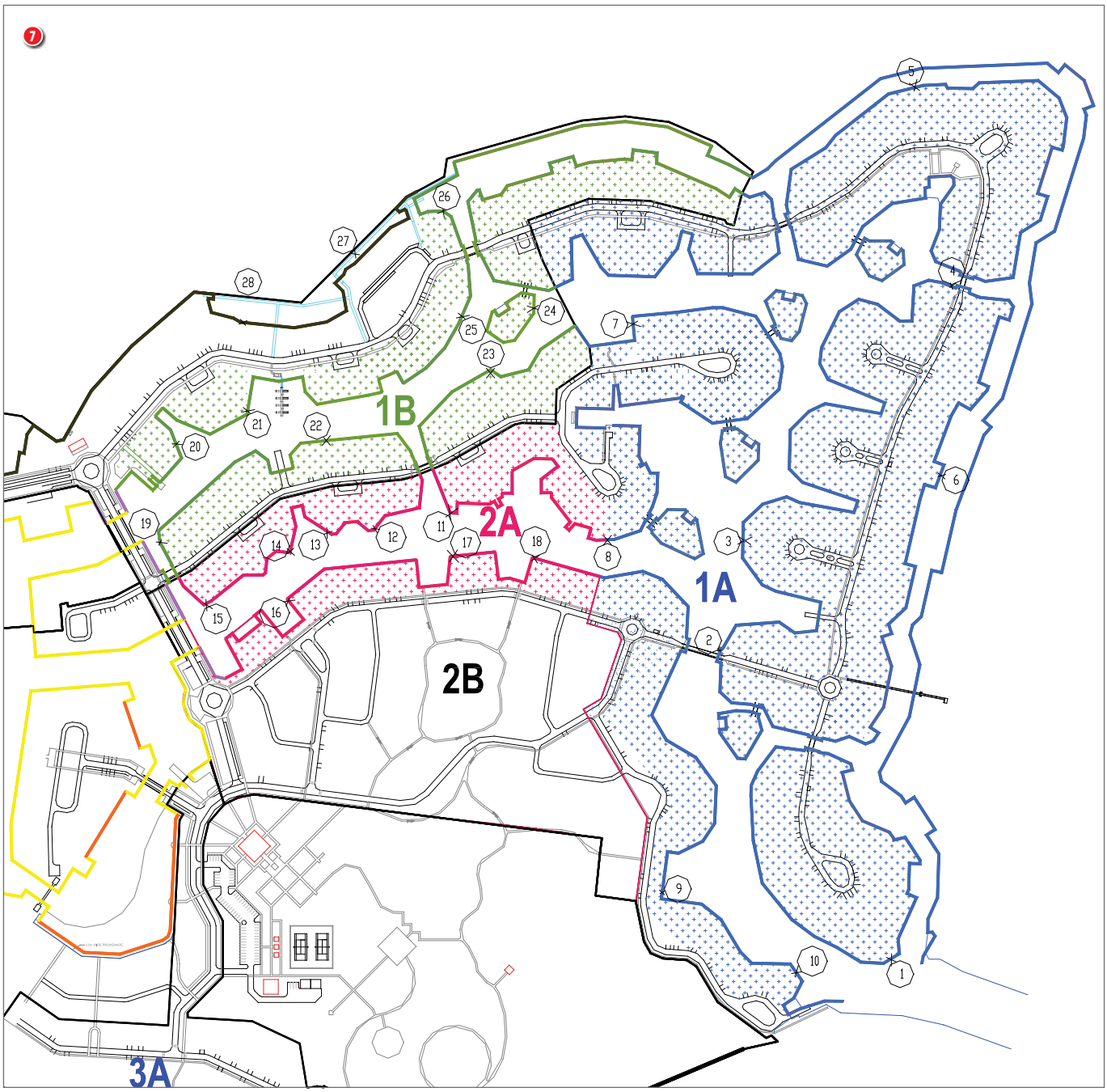
The structure and its foundations performed as expected, although the final settlement is greater than originally predicted and appears to be continuing, albeit at a much reduced rate. Maccaferri is investigating the possibility of agreeing

2 Cross-section through the 15-m-high Terramesh™ retaining wall
3 View of the Terramesh™ System ramp during construction
4 Front elevation of the tip wall
5 Cross-section through the Terramesh™ System tip wall
6 Graphical representation of settlement of the wall over five years









on an extension to this monitoring programme with the client.

THESEN ISLAND MARINA

The development of the Thesen Island Marina residential and commercial area in one of the most environmentally sensitive areas in the country required careful attention regarding the protection of 15 km of pristine waterfront property around the Knysna Lagoon.

Search for an appropriate solution

Thorough research, investigations and site trials were carried out by the planners Chris Mulder Associates Incorporated (CMAI), in conjunction with consulting engineers Arcus GIBB. It was decided to consult Maccaferri for their expertise in such situations and a unique set of solutions was generated, which incorporated their hexagonal woven mesh products in the form of Reno mattresses, gabions and Terramesh™, together with appropriate vegetative techniques. By integrating these versatile products effectively with a sound design philosophy, Maccaferri showed that they would be capable of achieving and maintaining the strict standards and conditions required on this project, which were based on the following criteria:

- Long-term durability in aggressive marine surroundings
- Structural stability under tidal and loading conditions
- Harmonisation with the environment
- Non-rigid structural behaviour

- Uncomplicated installation procedures
- Promotion of the establishment of vegetation (Figures 8 & 9)

Suitability of the proposed solutions

Gabions, Terramesh™ and Reno mattresses are flexible structures capable of adapting to changing field conditions and profiles without fracture. The former two are used for bank protection and retaining walls, and the latter for channel lining applications. The residual, long term strength of any product used for bank retention under aggressive marine environments is crucial for the success of such developments. In this project the long-term durability of the double twisted steel mesh depends solely on the protection offered by the PVC to the zinc-coated steel wire. It was made clear to the client that the PVC coating on the wire in this environment would have to be carefully protected throughout the life cycle of the product.

Product performance agreement

Maccaferri SA entered into a formal agreement with the client for the purpose of monitoring the behaviour and subsequent condition of the works on an ongoing basis. This agreement was carried out in two stages: the first was for five years, funded fully by Maccaferri, and the second was for as long as necessary after that (with a limit on the time span), funded by the client.

The agreement involved annual visits to the site by a recognised professional to make observations and extract samples of material from sacrificial sections (built in for this purpose) to be tested in the laboratory, followed by a comprehensive report on the state of the works and any changes taking place (Figures 10 & 11). In terms of the agreement and where necessary, Maccaferri will carry out remedial work on damaged or excessively weathered sections after consultation with the client.

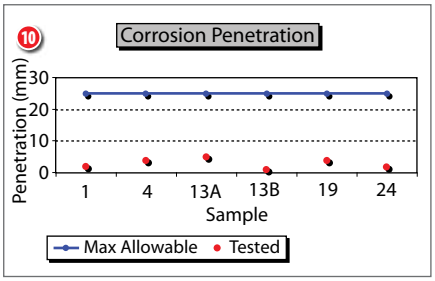
Record of performance to date

The work on this project commenced in January 2001. Stage one of the Product Performance Agreement ended in 2007 and during this period performance was according to expectations. Stage two commenced at the end of 2008 and in early 2009. Positive results have been observed and investigations will continue.

GENERAL CONCLUSIONS

Maccaferri SA has many other examples of projects of this nature in which it has been involved. Each project in its own unique setting will pose a different set of problems requiring appropriate solutions. Our experience and related documentation show quite clearly that the greatest success has always been achieved in situations where the client has actively engaged in a partnership with us and assumed joint responsibility for monitoring the performance of both the products used and the serviceability of the structure formed. ◻

7 Layout of Thesen Island Marina
8 Preparing for vegetation
9 Vegetated slope
10 Measurement of corrosion penetration
11 Sample under test



Text Graham Howell
Principal Geotechnical Engineer
SRK Consulting
ghowell@srk.co.za

Innovative geotechnical solutions for mining

Geotechnical expertise has been put to the test by the substantial capacity enhancement of Konkola mine through the Konkola Deep Mining project near Chillilabombwe in Zambia by Konkola Copper Mines (KCM) Plc. Innovative solutions had to be applied while sinking a new No 4 Shaft from surface through subsoil (lateritic) and incompetent rock to a considerable depth

1 Early days – view of the bank level prior to shaft sinking

THE PROJECT IS UNDER construction and due to become operational up to mid-shaft level some time in 2009, when the new area will start producing. After completion of the bottom shaft to full depth, a final annual ore production rate of 7,5 million tons is expected to be achieved – equivalent to 210 000 tons of copper a year. Vedanta Resources Plc is the major shareholder of KCM, which currently produces 150 000 tons of copper a year from its Nkana smelter and SE&EW. Upgrades at this facility and commissioning of the new smelter at Chingola will allow for a future 500 000 million tons of copper production.

At 82 m tall, the headgear at No 4 Shaft is believed to be the highest structural steel headgear in the world, and weighs over 1 600 tons. This weight, combined with challenging ground conditions, required a made-to-order solution that demanded close co-operation between client, contractor and consultant.

On the consulting side, SRK designed a foundation that was not, like most, on the surface, making it unique for this and a number of other reasons.

The challenge arose from a hard rock layer at about 25 m below surface that would resist penetration by an auger drill piling rig. These piles needed to create a tightly packed inner ring to provide lateral support, as well as four sets of six outer piles to support the headgear legs.

The headgear was designed to be supported by piles and the piles were intended to go through the soil and be socketed in the soft rock at 25 to 30 m. Because it was thought that there would be problems with the soils, an inner ring of piles was installed for lateral support and outer piles for bearing the load.

The piles were inserted close together (just 500 to 600 mm between each), which controlled any potential movement of the ground under load.

DEALING WITH SOIL AND SOFT ROCK

The soils at Konkola are typical subtropical soils. They are very weathered

down to about 25 m. Proper rock only starts to appear at deeper levels and even then the rock is often quite soft.

Another problem in the region is that groundwater is widespread; it can cause particular problems at the contact point between one type of material and another. A mitigating factor is that the area has extensive underground workings as it has been mined for decades. A network of underground pumps and pipes continually channels water into nearby rivers and has lowered the water table to a considerable depth.

The top 25 m of soil is not strong enough to support the headgear which, when operational, will exert a force of up to 10 000 kN – or 1 000 tons – through each of its four legs. The first 5 or 6 m is particularly treacherous from an engineering perspective, consisting of "potentially collapsible" material. This is made up of loosely packed granules which are cemented to each other at their contact points, but when it gets wet, it can collapse under load.

Even small amounts of water from leaking storm water or sewage drains can precipitate such a collapse of ground. For a shaft that is eventually going to reach a depth of 1 500 m, the forces at the headgear on surface are going to be severe, and clearly needed stronger material for securement.

PILING ON THE PRESSURE

The piles themselves needed a strong grounding and they were to be driven deep into competent rock to ensure their effectiveness. Limitations in the equipment capacity would prevent sufficient penetration, however, so an alternative way of securing the piles was required.

Using an unconventional technique of "undermining" the piles, a strategy was designed for contractors to break adits into the rock under the piles; this allowed the creation of a spread footing to secure the headgear, in the form of a concrete "ring beam" some 2,5 to 3 m thick.

The adits were worked one quadrant at a time: the rock was removed, the bottom of the piles exposed and the cavity filled with concrete.

The implementation of the new plan took about a month and a half and went smoothly. Apart from providing a solution to the immediate challenge, the innovation had kept this aspect of the construction moving so that it did not hold up the project as a whole.

SHARING SKILLS AND EXPERIENCE

To provide value-added engineering solutions like this, South Africa must continue to train and nurture enthusiastic and high-quality engineers. The problem is that there are simply not enough engineering graduates coming through the ranks of our tertiary institutions.

2 Exposed lateral support piles at lower ring beam level

3 View of the shaft and stage during the construction of the lower ring beam

An important way that SRK contributes to the career development of its own experts is by constantly encouraging them to share exciting ideas and techniques among themselves. At regular "Young Engineers' Think-tanks", each of the engineers is able to give a short presentation on a special innovation that he or she has made in the field. This information is then stored on a database that can be searched by keywords.

The trouble is that all kinds of useful innovations are made on projects, but few people get to hear about them. The whole idea of the think-tank is to foster everyone's interest and to have a lively discussion about clever ideas that have been tried in the field.

The key is to develop well-trained engineers into confident experts who can become great problem-solvers by learning from their own experience and the experience of others. At SRK, we believe in career-long learning and the importance of individual advancement alongside the growing expertise of the company as a whole.

SUMMARY: KONKOLA DEEP MINING PROJECT

Owner: Vedanta Resources and Konkola Copper Mines (KCM)

Aim: To increase the ore production capacity of the Konkola copper mine from the current 2 Mt/a to 7,5 Mt/a and extend the life of the mine by 22 years or more

Investment: Currently at least US$1 billion

Time frame: Completion scheduled for up to 1 010 mL by 2009 and to full depth by 2010

Shaft work: No 4 shaft is a new production shaft (close to the existing No 1 shaft) to be sunk to 1 490 m. The No 1 shaft will also be deepened to this level and will act as a service shaft for the Konkola Deep Mining project. A new pump chamber with infrastructures will be created at 1 390 mL. Additional ventilation and pipe shafts are also being constructed ◻

Text: **Richard Puchner**
Jones & Wagener Consulting Civil Engineers
puchner@jaws.co.za

Geotechnical investigation for decline shafts

The geotechnical conditions of a site often have a large influence on the final design and position of a shaft, or of any structure for that matter. This article examines the methods used to investigate and acquire information for the assessment of the ground conditions and presents two selected case studies. An understanding of the site's geology and geomorphology is of great importance to the geotechnical assessment

BACKGROUND

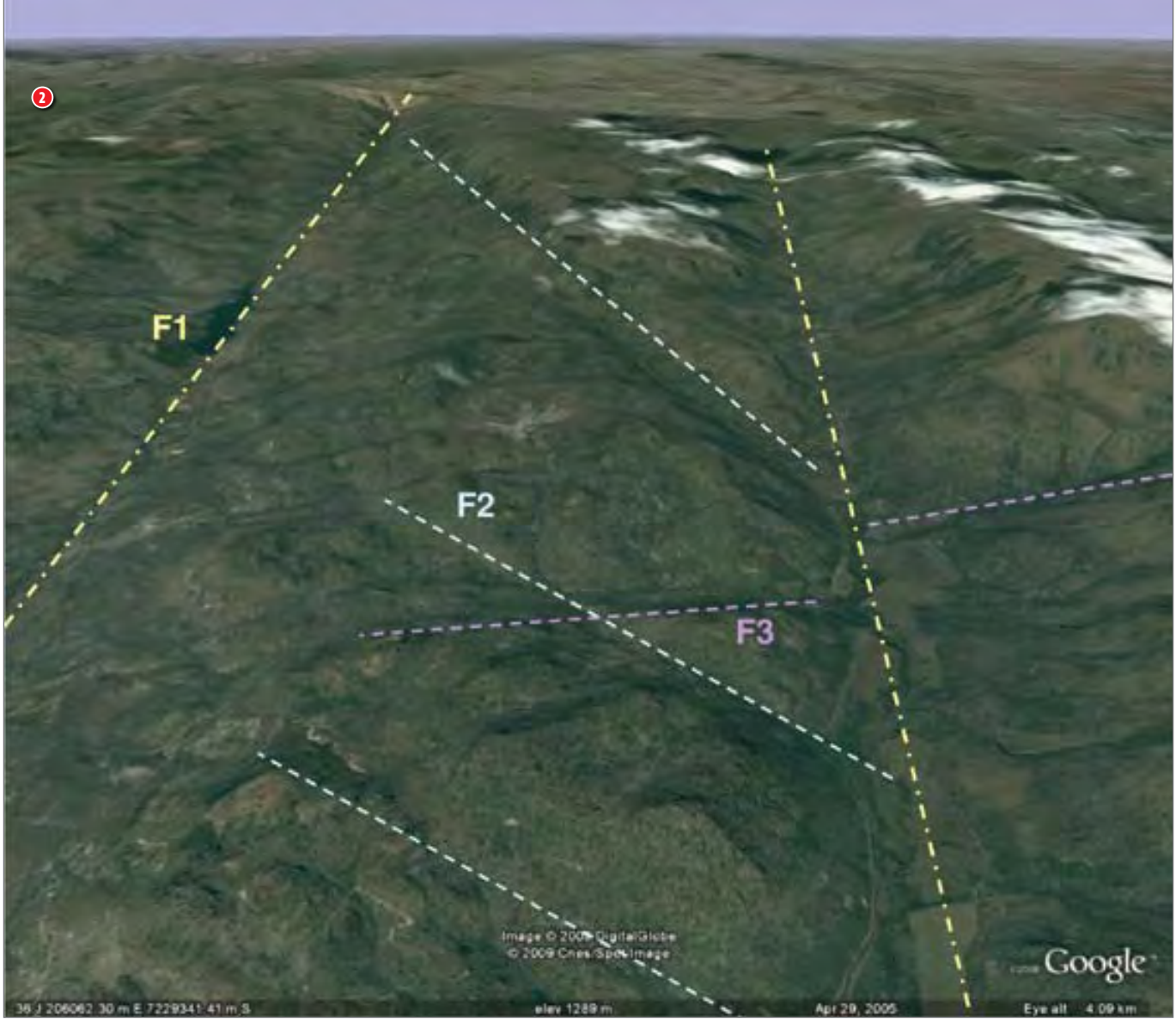
In many cases, rather than selecting a position which is best suited to start tunnelling, the location of a mine shaft is dependent on the underground mining plan. When projected back to surface, the ground conditions may not be ideal for the development of a shaft. Excavation of a boxcut through the upper soils and weathered bedrock provides access to a point, or portal, where tunnelling may commence safely. Determining the correct portal position is important for project-costing purposes and requires thorough geotechnical investigation.

OBJECTIVE OF A GEOTECHNICAL INVESTIGATION FOR SHAFT CONSTRUCTION

The objective is to provide an indication of the ground conditions before the design and the construction are undertaken. To obtain the necessary data, geotechnical investigations should be carried out in two phases: a preliminary phase and then a detailed investigation.

The preliminary investigation should aim to provide an overview of the subsurface conditions across a large area at a relatively low cost and in a short period of time. An estimation of the depth to bedrock, the extent of weathering and basic soil characteristics is obtained during this phase of the investigation. If any parameters are found to be unsuitable for the construction of the shaft, e.g. extremely thick soils or deeply weathered bedrock, then an alternative area can be investigated in a relatively short space of time and at a relatively low cost.

Once a suitable area has been located, or if the shaft position cannot under any circumstances be moved, a detailed investigation is required to provide additional parameters regarding soil properties, rock mass quality and the structural or geological regime of the site. These parameters are used for



1 Entrance to the 2,1-km-long decline shaft at Nchwaning Mine near Kuruman
2 Using Google Earth to provide an initial overview of the site topography and structural geological setting of a site

the final design of the shaft, including determination of the portal position, boxcut depth, sidewall angles and shaft-support requirements.

INVESTIGATIVE METHODS

A number of investigative methods are available, the selection of which may depend on several factors. They include the budget, the geological and geomorphological setting, accessibility on site and the phase of investigation.

Preliminary investigation

Methods used in a preliminary investigation generally include desk studies, site inspections, non-invasive techniques and, at times, invasive techniques. A desk study provides an initial overview of the site in terms of the geological and geomorphological setting. Data may be obtained from sources either published (e.g. literature and maps) or unpublished (e.g. old exploration borehole logs and previous geotechnical reports). A preview of the site using Google Earth is very useful in providing a preliminary overview of the site in terms of topographical and structural geological setting (Figure 2).

A site inspection will help confirm interpretations made during the desk study. Observations of the site topography and accessibility will be useful when planning the next phase of the investigation. Mapping of outcrops will confirm the geology and, together with topographical information, may provide an indication of expected weathering depths. Measurement of joint orientations (from outcrops, or from within excising excavations) will provide further indications of the structural regime of the area. For all mining projects, exploration borehole cores should be available. These provide additional information regarding the geological and geotechnical setting of the site.

Following the site inspection and desk study, the investigation will require obtaining further information via either invasive or non-invasive techniques. Invasive techniques include test pitting (Figure 3), large-diameter hole augering (Figure 6) and borehole drilling by either percussion or rotary core (Figure 4). These methods provide an indication of soil profile, depth to bedrock, depth of weathering, rock quality and water table depth. Access and space for testing equipment may be limited when the site is built up, or if it is in an ecologically or archaeologically sensitive area. Invasive techniques are also generally costly and time consuming.

Non-invasive techniques may be better suited for a preliminary investigation. These generally require less operating space and are relatively quick and cost-effective, which is a plus when the site might be moved and the investigation has to be redone. Non-invasive techniques include geophysical testing (e.g. seismic, gravity, resistivity or magnetic) and DPSH

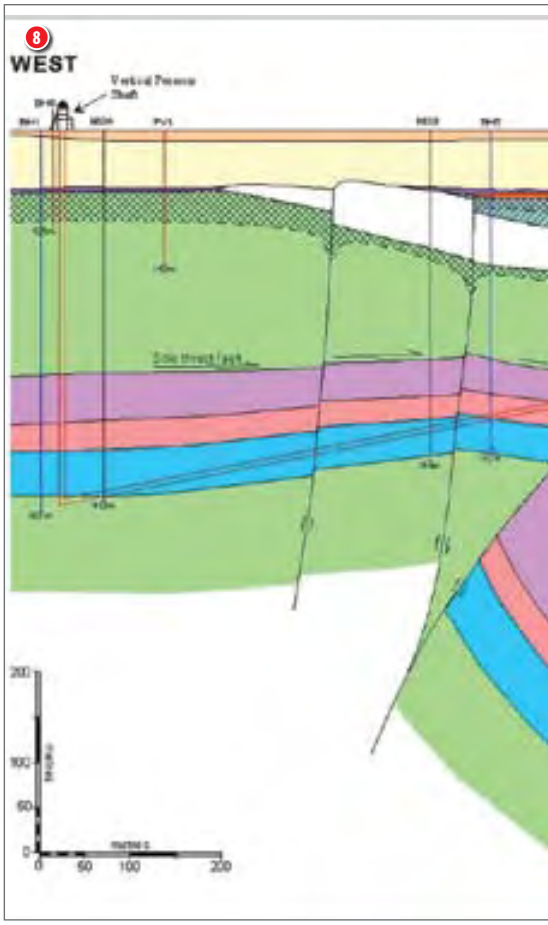
(dynamic penetration super heavy) testing. Although it is invasive in that it means physical penetration of the probe into the soil, DPSH testing does not require much surface space (Figure 5) and is relatively quick and cost-effective. However, these non-invasive techniques do have their limitations: they provide only a general indication of depth to bedrock and structural setting. Selected invasive methods may have to be used to assist in acquiring the basic information needed to conclude the preliminary phase of the investigation.

Detailed investigation

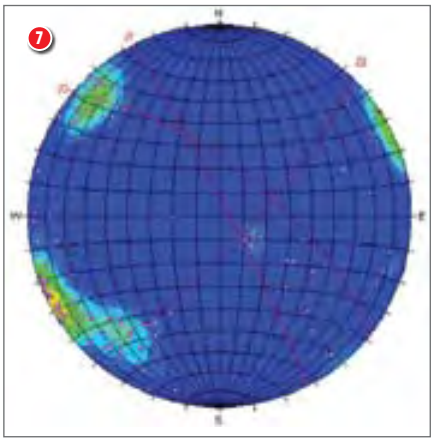
In order to obtain as much information as possible regarding the soil and rock properties for the final design of the shaft and boxcut, invasive field-testing techniques are required. For soil, a profile of the near-surface soils is best obtained via profiling of test pits or auger holes. Laboratory testing of the soil samples taken during this process provides an indication of the soil characteristics for design of the boxcut side walls and angles. An indication of whether the material can be re-used, e.g. in roads or terraces, is also obtained.

If the soils are deeper than the reach limit of an excavator or auger rig, assessment of these deeper soils may be done by rotary core drilling. Although recovery of soils is difficult, relatively undisturbed Shelby samples of soil may be obtained and SPT (standard penetration testing) may be carried out to provide an indication of soil consistency.

Once through the soils, rotary core drills provide a relatively accurate depth of the soil-bedrock contact, the depth of weathering within the bedrock and the rock mass quality. Rock strength can be determined by subjecting the core samples to either PLI (point load index) or UCS (unconfined compressive strength) testing. Logging of the core, which includes a detailed description of the rock, its weathering and joint conditions, also



3 Test pitting
4 Inclined rotary core borehole drilling
5 DPSH penetrometer testing
6 Large-diameter auger hole profiling
7 Stereonet plot of the joint orientations and main joint sets
8 Geological long section of the Nchwaning decline shaft



requires measurement of core recovery and RQD (rock quality designation). These are all used to calculate the rock mass quality. Two commonly used methods are Barton's Q (Barton et al 1974) or Bieniawski's rock mass rating (RMR) (Bieniawski 1989). The calculated values provide a rating of the rock mass quality for tunnelling and enable the support required to be selected.

Inclined rotary core drilling may be required when it is suspected that vertical joints have a large bearing on the overall stability of the decline shaft or boxcut excavation. Orientation of the core allows measurement of joint orientations which, when plotted onto a stereonet (Figure 7), provides an indication of the predominant joint sets. From this, potential sidewall and hanging wall failures can be identified and the boxcut design can be modified to allow for safer sidewall angles and/or sufficient support.

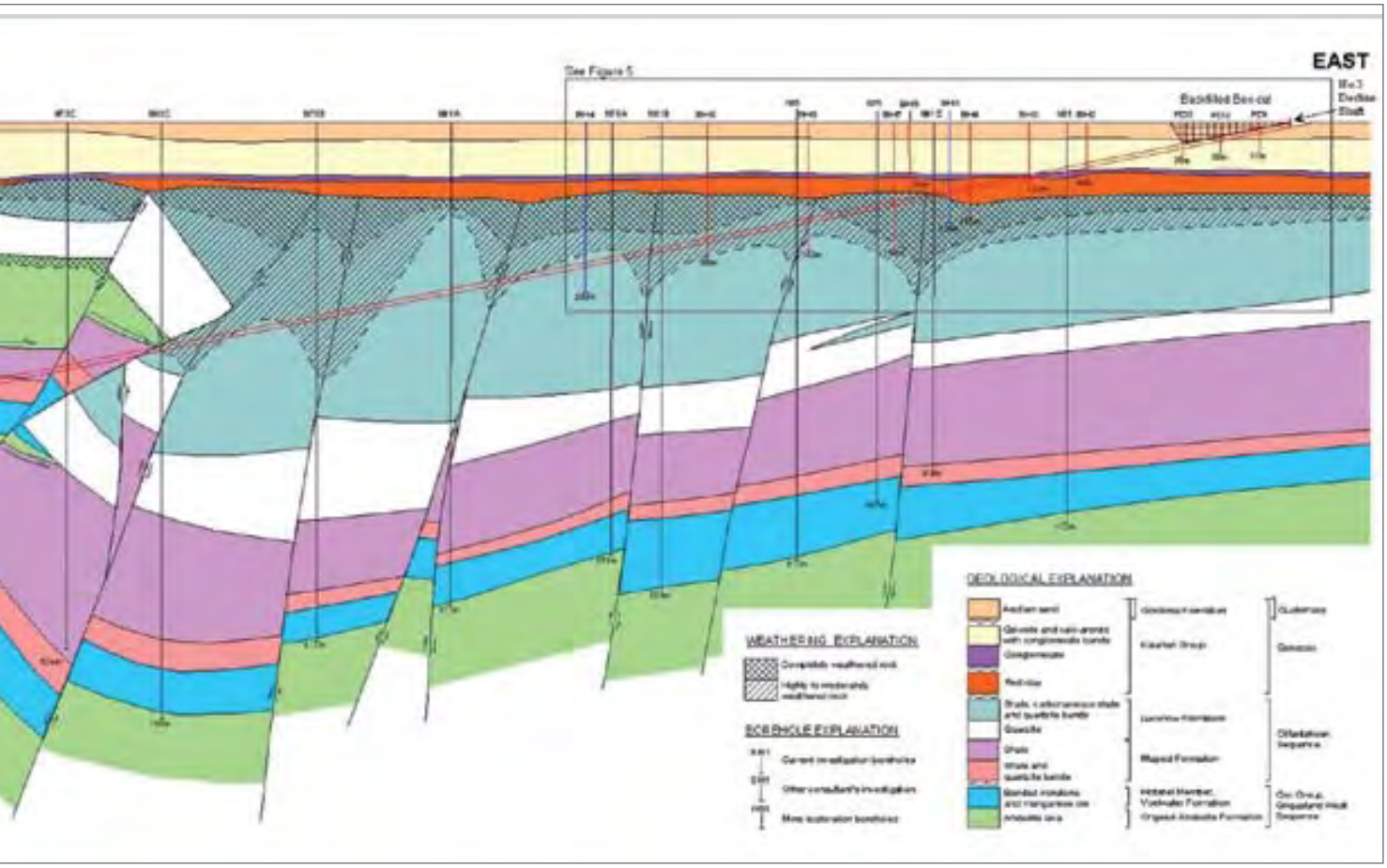
If groundwater is suspected to be present, piezometers installed into the boreholes will confirm the presence of a groundwater table and whether it will have an effect on the sidewall stability. Lugeon (or packer) testing at regular depth intervals during core drilling will assist in determining the degree of rock fracturing and openness of joints. If an aquifer is suspected to be present, pump-out testing of a larger diameter percussion borehole will help in estimating the anticipated volume of water that may enter the shaft during construction.

CASE STUDIES

Nchwaning manganese mine (see Figure 1)

Situated north of Kuruman in the Northern Cape province, Nchwaning mine lies well within the semi-arid Kalahari. The ore body is approximately 500 m below surface and requires a shaft to access it. A decline shaft was constructed to provide vehicular access and to allow conveying of ore to surface. The shaft went through a vertical thickness of 20 m of collapsible aeolian sands, 30 m of calcretised sands and gravels, and 30 m of thick, highly expansive clay. This was then followed by up to 80 m of weathered shale. Altogether more than 800 m of challenging ground conditions were encountered.

Since the area is covered by the Kalahari sands, the landscape is flat and featureless with no outcrops to map. Preliminary investigation included a review of old exploration



9 The mountainous terrain of the Der Brochen project near Steelpoort

borehole logs and the creation of a geological section (Figure 8). With this interpretation of the subsurface horizons, percussion boreholes were drilled and DPSH tests carried out within the proposed boxcut area. This provided an indication of the sand's consistency and confirmed the depth at which calcretisation was competent enough for tunnelling to start.

Detailed investigation included the drilling of rotary core boreholes through the upper sediments into the weathered shales. Analysis of the cored samples allowed the determination of the rock mass quality. Pump-out testing of the aquifer above the clay unit provided an indication of the amount of water that might be encountered during tunnelling. Monitoring of the piezometers installed within the boreholes showed that the local water table had been drawn down while the tunnel passed through, and that there were no immediate concerns regarding further water ingress.

The data obtained from the investigations were used to design many aspects of the decline shaft. These included the boxcut sidewalls, the depth at which the tunnelling machine could start, the point at which conventional drill-and-blast techniques would take over, the length of the precast concrete lining and the amount of rock bolting, mesh and shotcrete needed.

Der Brochen platinum project

Unlike the topography of the Kalahari, the location of the Der Brochen project is in a mountainous area south of Steelpoort in Mpumalanga (Figure 9). The project area has two main platinum-bearing reefs, the Merensky and UG2 reefs. Both span 25 km on strike and 8 km on dip. It was planned that underground access would be from either side of a mountainous ridge. Three adits and a gathering haulage were required to provide horizontal access to an underground network from the one side of the ridge and two decline shafts from the other side.

Since the positions of the adits and shafts were fixed by an underground plan and were required to enter on reef, the aim of the investigation was to determine the portal positions where tunnelling could start safely.

Preliminary investigations included the study of published literature and geological maps. Review of exploration boreholes, detailed geological, structural and topographical maps provided

an initial indication of the expected ground conditions.

Geological and joint mapping provided an indication of preferred portal positions. Initial drilling of vertical rotary core boreholes within the preliminary phase confirmed the depths of weathering and the likely portal position. It was found that the pyroxenitic Merensky reef, which lies within more competent anorthositic and noritic units, is preferentially weathered. Where the mountain slope is less steep, weathering was found to be more extensive. Based on these findings, some of the adits were moved to reduce the size of the boxcut required to get to the tunnelling start point. The detailed investigation included inclined and orientated core drilling to further define the rock mass quality for boxcut side slope design and initial tunnelling support requirements.

CONCLUSION

A number of methods are available for the investigation of decline shafts. Although using of all of them would provide an impressive assessment of the ground conditions, a number of aspects restrict their use. These include budget, surface and subsurface conditions and restrictions. A typical geotechnical investigation for a decline shaft would be carried out in two phases. A preliminary investigation would determine the ideal location of the shaft, while the detailed investigation would provide the information required for final design and construction. Although the above case studies provide examples of sites that are extreme opposites, ground conditions can never be assumed to be similar, even though the shafts might be a few metres apart from one another on the same reef outcrop.

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Text Dr Phil Paige-Green
Chief Researcher
CSIR Built Environment
ppaigegr@csir.co.za

Dr Dave Turner
Division Manager: Pedology & Mineralogy
ARC – Institute for Soil, Climate and Water
dturner@arc.agric.za

The preliminary identification of **problem soils for infrastructure projects**

This article discusses the application of the information currently available on the land maps and soil types for South Africa and their accompanying material data, originally compiled to provide information for agricultural purposes, as a cost-effective preliminary indicator of the common problem soils for civil engineers. Such information is extremely useful for the planning, feasibility and preliminary design stages of road construction and other civil engineering projects

INTRODUCTION

The development of any infrastructure project requires a good knowledge of the material and founding conditions at the proposed site. This information is often obtained only after the feasibility and planning, and sometimes even after the preliminary design, have been done. Between 1972 and 2002 the land types of South Africa were comprehensively mapped over the whole of the country for agricultural purposes. Individual land types are based on having a "marked degree of uniformity with respect to terrain form, soil pattern and climate" (ARC-ISCW 2007a). It should be noted that the land type maps fulfil the same intention, although the method differs, as the requirements outlined in TRH2 (NIRR 1971, NITRR 1978). Although mapping of the land types was carried out at a scale of 1:50 000, the maps are readily available in electronic or hard copy formats at a scale of 1:250 000 (ARC-ISCW 2007a). Each map is accompanied by a wide range of soil descriptors and properties conventionally used for agricultural purposes, as well as certain selected engineering properties, e.g. Atterberg limits (Netterberg 2001). Many of the soil properties that can have a bearing on their performance as subgrade or foundation materials for development, or possibly even their use in construction, are available or can be roughly estimated for each land type provided.

A slightly different version of the soil maps based on broad pedological groupings (more than the land types of the ARC-ISCW) is presented by Harmse & Hattingh (1985). The soils identified on the accompanying map (1:6 000 000 scale) can, however, be used in the same general way as those described in this article, although less detail relating to the individual soil types is provided in the text of Harmse & Hattingh.

Although the mapping (and associated soil analyses in the case of the ARC-ISCW data) has been carried out with the purpose of indicating the potential of the soils for agricultural use, this information can prove very useful for engineering purposes. The value (and shortcomings) of this information was highlighted by Brink et al (1982). Fanourakis (1991, 1999) carried out research into the assessment of soil properties for civil engineering applications from the pedological data provided in the ARC land type maps and profiles. Netterberg (2001) also highlighted the potential application of soil science in civil engineering. The findings and recommendations of these researchers have, however, never been extended or implemented to any extent in the civil engineering industry in South Africa.

This article discusses the application of these maps and their accompanying soil data as a preliminary indicator of the common problem soils for civil engineers. Aspects such as material thicknesses and depths, and the classification, mineralogy and properties of each of the materials, the presence of problem soils (expansive, dispersive, saline, etc.) and certain basic road engineering properties (e.g. soil strength) can be quickly found to assist with optimum site planning and cost-effective field investigations. The maps are potentially useful for providing a number of other useful geotechnical properties as well, but these are not discussed here as they have been identified separately (Paige-Green & Turner 2007).

Although the overall results should be used only as preliminary indicators for planning projects, they allow the typical types of problems that can be expected, as well as issues such as the likely availability of construction materials, to be identified during the early stages of projects. This information can have a significant impact on the final project design.

SOUTH AFRICAN LAND TYPE MAPS

Detailed or semi-detailed soil maps would be a first choice as information sources in civil construction projects but, with the exception of certain metropolitan areas of the Western Cape and Gauteng provinces (ARC-ISCW 2007b), such detailed soil maps are seldom available. However, land type maps, with complete coverage of South Africa, provide a useful alternative as a soil information source. The fundamental mapping unit of the South African land type maps is based on a combination of the topography, climate and soil pattern. Twenty-eight Broad Soil Pattern Groups were chosen as the basis for the soil mapping which, when combined with the topography and climate, gives the land type. Soils are classified according to *Soil classification: A binomial system for South Africa* (MacVicar et al 1977) and reported in the soil and terrain inventories as Soil Forms and Series. Modal profiles have been described which represent the dominant soils of most land types and samples of each horizon taken for analysis. It should be noted that the binomial system has subsequently been revised and Soil Forms and Families are now used (SCWG 1991). The Soil Family criteria have not yet been applied to the maps.

The maps are printed as 1 x 2 sheets at a scale of 1:250 000 with accompanying memoirs and are available from the Agricultural Research Council (ARC) – Institute for Soil, Climate and Water (ISCW) in electronic or hard copy formats. They can also be accessed directly on the Department of Agriculture website (www.agis.agric.za/agis). The soil analysis test results are provided in land type memoirs (ARC-ISCW 2007a) or through the ARC-ISCW Soil Profile Information System (ARC-ISCW 2007c). Land type soil data are provided as inventories, of which there are some 7 200 comprising about 65 000 soil components. The memoirs include about 2 500 modal profiles for which comprehensive soil analysis data are available for the various horizons. Examples of a portion of a land type map and a typical extract from the accompanying memoir are given in Figures 2 and 3.

In addition to this conventional soil property information, data related to the topography, climate and parent geology are available. Land type modal profile data (point data) have been extrapolated using

Table 1 Information available from land type soil and terrain inventory data and its potential application to geotechnical engineering

Available (and useful) soil information	Potential application or indication
Terrain information: Slope classification and definition (slope angles, shape and lengths of terrain units, terrain unit as a percentage of the land type)	Indicative of possible areas for borrow pit and quarry location
Mechanical limitations to ploughing	Excavatability, stoniness, soil thickness
Areal percentage of rock outcrop	Potential excavation problems, possible quarry materials
Soil form	Nature of the general soil properties, including surface organic matter, internal soil drainage, presence of temporary or permanent wetness, general soil mineralogy and expansiveness, soil permeability and stability to erosion, and nature and composition of underlying materials
Soil series	A more detailed explanation of the properties as expressed in the soil form, but extended to additional properties, including base status and cation exchange properties as possible indicators of soil stability, clay content and dominant sand grade as indicators of texture. Also indications of depth of soil, excavatability (MB), general soil properties and hardpan (plinthite) materials
Depth-limiting material	Subgrade materials, location of rock or hardpan
Clay content and soil texture	Soil type
Geology	Potential problematic material types – dolomite, shale, basic crystalline rocks, etc

spatial modelling techniques to illustrate the spatial distribution of soil property values over South Africa. Maps illustrating, for example, the distribution of soil pH, cation exchange capacity and the potential for erosion (predicted soil loss) are accessible on the AGIS (Agricultural Geo-Referenced Information System) website as the Natural Resource Atlas (AGIS NR Atlas 2007). Used in combination with land type survey data, these maps and data sources provide very useful information for the road design engineer.

INFORMATION AVAILABLE

Essentially, the printed maps and memoirs provide the basic data. A much wider range of interpretive maps is provided on the website and the two sources of information are discussed separately below.

When using land type information, an appreciation of the underlying variability of natural materials and the scale of the published information source must be maintained. Although the mapping units are clearly differentiated on the maps, significant differences in the depth and nature of the soils can occur over relatively short distances. A strong materials or engineering geological background with some experience in soil mapping, profiling and terminology is thus essential to gain maximum benefit from the use of this information in infrastructure engineering.

Printed maps

Besides the general topo-cadastral information provided, the maps identify the broad soil patterns in an area. Other than general properties, such as the presence of vertic (swelling) or thin soils, this alone is of little use to the engineer. However, when the topo-cadastral information is combined with the information on the soil and the terrain inventory data for the land types and the modal profile data provided in the memoirs, significant useful insight into the materials in the area can be obtained. The information contained in the soil and terrain data sheets is mostly spatial (area and depth) and that applicable to and useful in geotechnical engineering is summarised in Table 1.

It should be noted that the information provided by the maps and accompanying documents is based on the upper 1 200 to 1 500 mm of the soil profile, unless hard saprolite or rock is reached before this. Although this is not necessarily deep enough for all geotechnical work, it is certainly a good indication of the likelihood of potential problems and the presence of deep soils, and is highly applicable to light engineering works such as foundations for houses, light structures, roads, canals and reservoirs (Jacobs & Van Huyssteen 1996). For foundations for large structures, the normal drilling programmes will still be necessary. However, knowledge of the "surface" soil conditions can often assist in optimising the location of test pits and drilling sites.

The contents of the modal profiles are equally, if not more, useful. The information included in these profiles is summarised in Table 2. The soil profile descriptions used in pedological profiling differ from those conventionally used in soil engineering (Jennings et al 1974), although they do include moisture, colour and soil type. Descriptors for soil structure and consistency are provided but differ from conventional soil engineering definitions.

The particle size distribution results have been determined on a small 100 g sample and are thus representative only of the soil matrix. However, together with the percentage retained on the 2 mm screen (where provided) and the soil description in the modal profiles, they do indicate the general nature of the materials. Fanourakis (1991) compared the results of the soil gradings provided in the memoirs with those from conventional geotechnical testing and concluded that the engineering grading characteristics could be estimated from the pedological grading to a significant degree of reliability. Fanourakis (1991) also attempted to estimate the plasticity characteristics from the pedological properties with mixed success. However, Atterberg limits have been determined on many of the profile samples and, although not published, are obtainable through the ARC-ISCW Soil Profile Information System.

Soil colour can be a useful indicator of its properties (Jacobs & Van Huyssteen 1996). Red soils (e.g. Hutton Form) are usually the result of hematite and are indicative of well-drained, highly weathered soils. Soils that have a yellow-brown colour usually contain goethite, which is indicative of higher rainfall areas with a cooler climate. Grey soils are often indicative of an excess of soil water.

A strong materials or engineering geological background with some experience in soil mapping, profiling and terminology is thus essential to gain maximum benefit from the use of this information in infrastructure engineering

Web-based maps

The land type maps on the website provide the basic soils information but do not have the narrative data and test results given in the memoirs in an easily accessible form. It should also be noted that these maps use the revised soil patterns, with grouping of certain patterns and thus less homogeneity of the soils within the pattern. These maps have advantages in that the scale can be altered at will such that users who do not have access to geographic information system software can make on-screen adaptations to the map scale. Various other useful layers (e.g. roads, rivers, farm boundaries and climatic data) can be added or removed as required. Engineers regularly requiring the land type information should consult the ARC-ISCW concerning the original information source (ARC-ISCW 2007a).

The website does contain a number of small-scale derived maps published as the Natural Resources Atlas (ARC-ISCW 2004), indicating potentially problematic materials. Although derived for agricultural purposes, the following maps can provide a general in-dication of the possible local problem conditions:

- soil susceptible to water erosion
- soil susceptible to wind erosion
- potential shifting sands
- potential for soil regeneration if badly eroded
- soils with textural contrast
- soils with poor or impeded drainage
- swelling clays
- saline and sodic soils

A typical example of the map provided for swelling clays is shown in Figure 1.

USE OF THE MAPS

In order to obtain maximum benefit from the maps, they should be used in a systematic and structured way, in conjunction with other conventional information. The proposed site or route should initially be plotted on the map and its relation to the topography, surface water and the broad soil patterns established. Particular note should be made of problems in this regard that could be avoided by realigning the route.

Once the route alignment has been confirmed as being the optimum, closer inspection of the broad soil patterns should be made. The fewer the soils being traversed, the more likely it is that the material will be reasonably consistent and less of a preliminary field inspection and subgrade centre-line survey will be required. A more detailed assessment of the soil types that the road will traverse can then be made. The soil types alone will provide useful information.

Reference to the memoirs can then provide detailed information on the local geology, terrain, materials and their properties in terms of the problem soils discussed below.

In order to obtain maximum benefit from the maps, they should be used in a systematic and structured way, in conjunction with other conventional information

Table 2 Information available from modal profile data

Profile site description
Soil form and soil series
Water table
Occurrence of flooding
Surface rockiness/stoniness
Erosion (type and classification)
Underlying material
Weathering of underlying material
Profile soil description
Profile depth of layers
Profile depths, soil fractions and clay mineralogy
Soil analyses
Organic carbon
Cation exchange capacity (CEC)
Exchangeable sodium or exchangeable sodium percentage (exchangeable Na*100/CEC)
pH
Resistance or electrical conductivity of saturation extract (EC)
Particle size distribution – clay, silt and sand contents
Soil fertility properties: phosphate, Ca, Mg, K, CEC soil acidity values

Expansive soils

Vertic soils are currently defined by having a Plasticity Index (PI) > 32 (SCWG 1991). The lower limit for very high heave identified by Van der Merwe (1964, 1975) was a weighted PI of 32%. Thus for fine materials with 100% passing the 0,425 mm sieve, the limit is the same. For materials containing increasing quantities of material coarser than 0,425 mm, the weighted PI will decrease correspondingly. Any material identified on the land type map as a Vertic soil should, however, be considered as likely to have expansive properties. These land types include soils of the Rensburg and Arcadia Forms and should be noted early in the project (feasibility and/or planning stage), with the potential for looking at relocation or realignment of the facility where possible.

Soils with a Melanic A horizon are defined as those which, among other attributes, have a PI of less than 32%. There is thus a good chance that those soils in the higher range (particularly if the majority of the material is finer than 0,425 mm) could also be sufficiently expansive to give problems, and soils classifying as such should also be assessed in more detail.

Other soil forms, however, not defined by their Vertic or Melanic topsoil horizons but by subsoil horizons with high clay contents, high base exchange status and which contain smectite clays, also need to be assessed for their expansive properties. This information can be obtained from the profile descriptions and layers containing these types of soil warrant detailed investigation at the design stage.

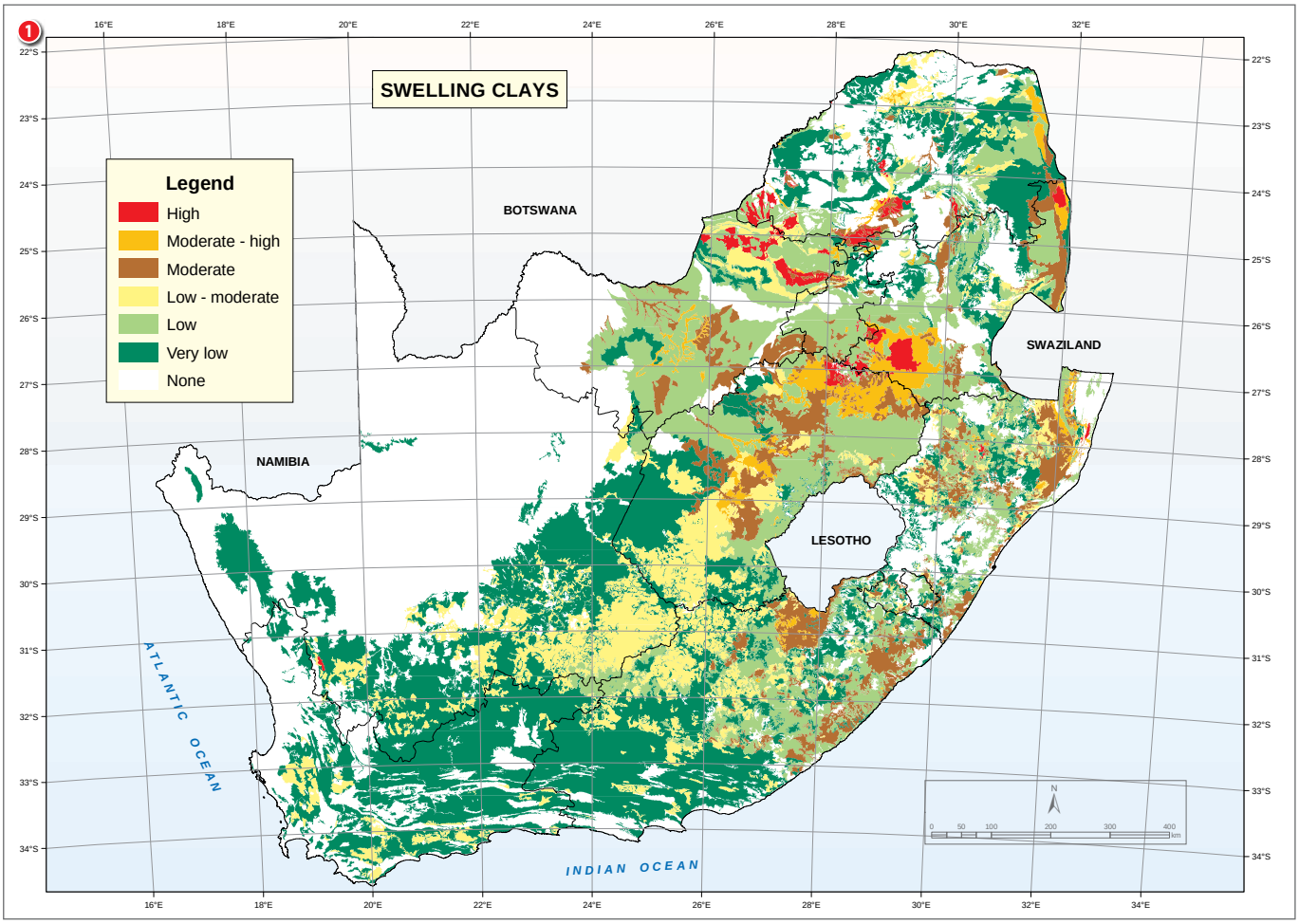
Dispersive soils

Dispersive soils can lead to catastrophic failures of earth embankment dams, as well as severe distress of road embankments. These soils contain clays that are high in sodium such that the individual particles repel each other to a stronger degree than the Van der Waals forces of the particles attract each other. Thus soils with high exchangeable sodium contents are normally considered to indicate potentially dispersive soils. Although there is still considerable debate regarding the positive identification of dispersive soils, the use of the exchangeable sodium percentage (ESP = Na+/cation exchange capacity*100) can certainly give a first indication of potential dispersive problems. It is suggested that the presence of an ESP greater than 10% should be regarded as a warning (Elges 1985) and additional testing should be carried out on such soils. Gerber & Harmse (1987) state that the limit changes with the clay mineralogy, as indicated by the CEC, but that all soils tested with ESP values higher than 15% are highly dispersive.

The determination of the cations in the saturation extract of the soils has been carried out for soils with resistances of the saturated paste of less than 460 Ω. From these, the sodium adsorption ratio (SAR), calculated as SAR = Na/√[0.,5(Ca+Mg)], can be calculated and materials with values of greater than 2 (Bell & Walker 2000) should be investigated further for potential dispersivity. Unfortunately, only a few of the samples have had their saturation extracts tested.

The results required to determine ESP and SAR (when available) are provided in the soil profile data and need to be calculated for each material. However, a cursory inspection of the sodium and

1 Example of swelling clay map provided on AGIS website



resistivity results (high values of sodium and low values of resistivity are immediately recognisable compared with the typical values of most materials) will usually indicate whether the data needs to be processed further.

Should any dispersive soil problems be suspected, it is recommended that the battery of tests (CEC versus ESP, crumb test, SAR, pinhole test and total dissolved solids versus percentage sodium) be determined as suggested by Bell & Walker (2000), and the potential dispersivity of the material rated accordingly.

Collapsible soil

Soils with high single-sized fine sand contents, silty sands or clayey silts (e.g. deep Regic soils, some residual granites, arkoses and "dirty" sandstones, and various other residual or transported materials) indicate potentially collapsible materials. Fine silty or sandy soils with high feldspar and/or kaolin contents in drier areas may also be potentially collapsible.

It is noted (SCWG 1991) that certain soils in the E horizon are prone to having a high dry strength but fail under load when wet. These are thought to resemble the "fragipan" soils of other classification systems – a descriptive term implying their collapsibility.

Netterberg (2001) has recorded that most of the highly weathered ferralitic (clay fraction SiO_2/Al_2O_3 molecular ratio < 1,3) and fersialitic (clay fraction SiO_2/Al_2O_3 molecular ratio 1,3 to 2,3) soils are likely to have collapsible grain structures. These would include the widespread dystrophic to mesotrophic, fine sand and silt-textured Hutton and Clovelly soils. They are readily recognised by their uniform red or yellow-brown colours respectively and their weak or apedal structure.

Saline soils

The presence of saline soils under roads and structures can result in significant problems. These materials have a high soluble salt content, of which sodium often comprises only a modest proportion, but can result in the crystallisation of soluble salts in the top of base courses in roads, leading to loosening of the compacted base course and, beneath road surfacings, leading to blistering and cracking of the bituminous seals. The materials discussed in this section are predominantly chlorides and hydrogen carbonates, with the soluble acid sulphates being discussed separately in the following section. The latter are generally more common in soluble slat problems in southern Africa, although a number of chloride problems have been reported (Roads Department 2001).

Salts can originate from the in situ natural soils beneath the structures, as well as from introduced material or saline construction water. This discussion refers only to the former situation.

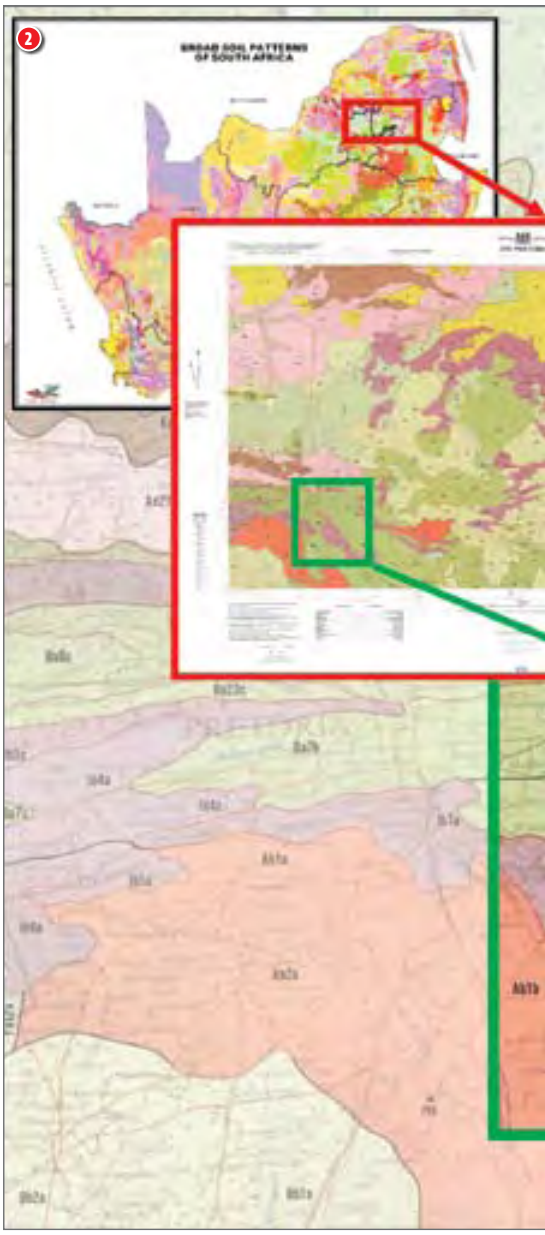
The soil profile information provides an indication of the electrical resistance of many of the soils. In the conventional road engineering context (and application to other infrastructure developments), the identification of possible soluble salt problems is based on the pH and conductivity of the materials. The pH and the electrical resistance (which is the inverse of the conductivity) are provided in the soil profile data (ARC-ISCW 2007c). Although the techniques and the fractions tested in conventional pedological and engineering investigations differ, which could mean some differences in the results, a general assessment of any potential problems with respect to saline soils can be obtained. As a first indication, where the electrical resistance is less than 460 Ω and the pH is greater than 7,0, the materials should be investigated further for their likelihood of presenting soluble salt problems. The limit is loosely based on that for pavement layers (selected subgrade to base) proposed by Netterberg (1979), but is probably equally useful for in situ subgrade materials (Roads Department 2001), especially for thin pavement structures.

Materials containing acid sulphate

Natural soils containing acid sulphates are of limited extent in South Africa, and are probably located in peat lands that, on drainage, produce deleterious acidity, as well as in semi-arid to arid areas prone to high evaporation (Roads Department 2001). Materials derived from unconsolidated rock may contain sulphides or weatherable sulphate minerals that are potential sources of acidity, or hydrated sulphate minerals with negative impacts on stabilised layers and concrete structures and the potential for soluble salt damage to road structures. Similarly, acid waters derived from certain sulphide containing materials can attack chemically stabilised soils and concrete structures. Initial recognition of the soils is probably through the wet lowland positions with high organic matter, while that for the materials derived from unconsolidated rock is via their mineralogy. No reference is made in the land type memoirs to any of the anions, particularly sulphates, that can cause damage to concrete or to soluble salt problems. However, it is recommended that for materials with a resistivity of less than 700 Ω, at least qualitative testing for sulphate should be carried out, followed by determination of the acid-soluble sulphate content if the qualitative test proves positive. The limits described by Netterberg (1979) can probably be applied to cement- and lime-stabilised layers to prevent acid damage to the stabilisation products.

Compressible soils

Soils with poor or impeded drainage indicate potentially compressible materials, subject to significant settlement under



road and/or traffic loading. When more than 40% of any land type consists of soils classified as Champagne (Ch), Rensburg (Rg), Willowbrook (Wo), Katspruit (Ka), Kroonstad (Kd), Estcourt (Es), Longlands (Lo), Wasbank (Wa), Cartref (Cf) or Lamotte (Lt), the potential for wetland compressible soils should be noted.

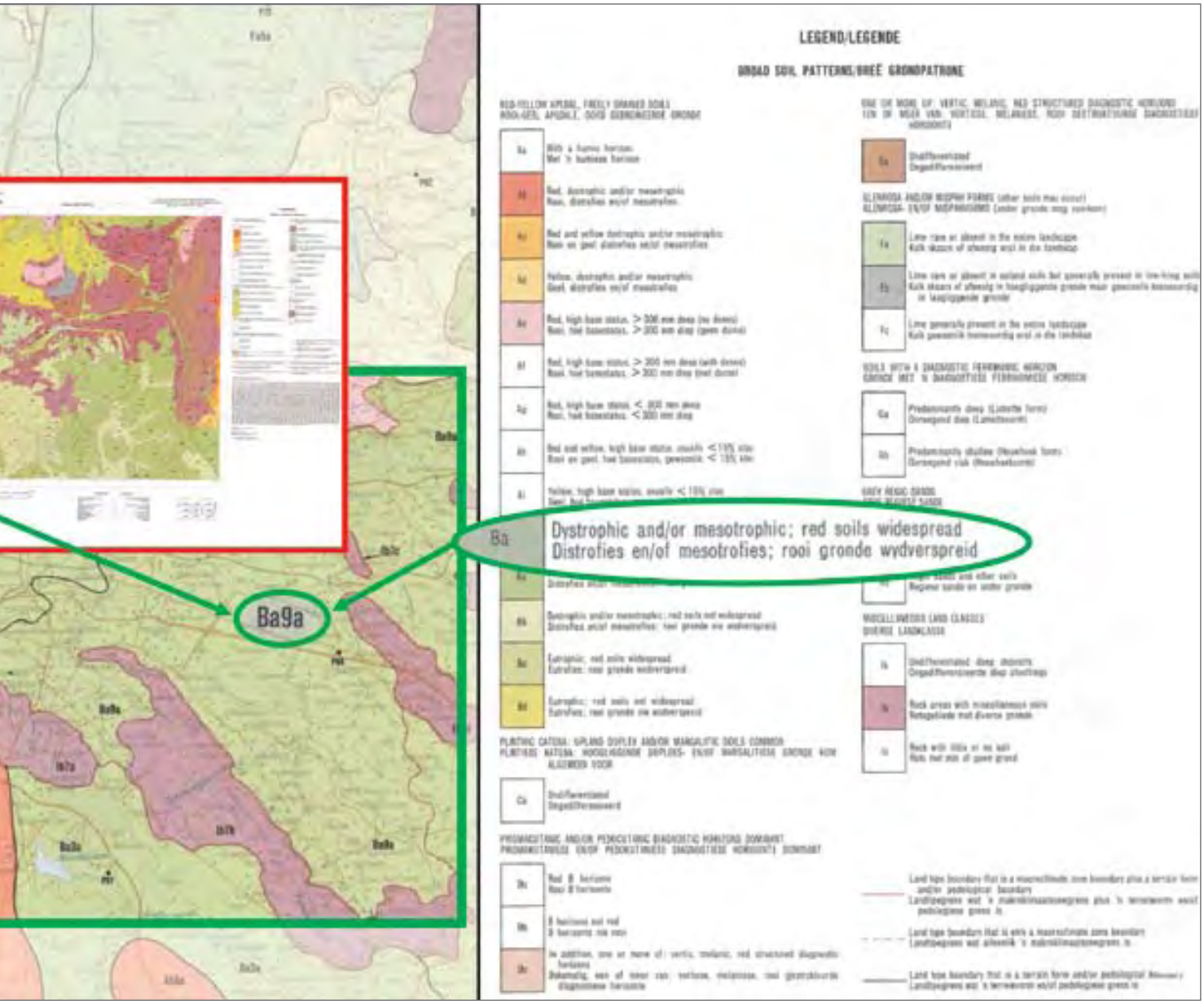
These materials require extreme care during the construction of embankments over them as their shear strengths are low and they have a high potential for settlement which, depending on their thickness, can require carefully designed countermeasures.

Dolomitic soils

Soils developed on dolomite have unique problems that are best identified from standard geological maps. Development on dolomite requires special investigations, carried out by specialists in dolomitic terrain investigations. Infrastructure development on dolomite will usually have identified the potential problem and need for expert investigation early in the project and the need for soil maps is thus redundant in most cases.

Soils prone to liquefaction

Soils with high silt contents or high clay-sized components (< 0,002 mm), but which are predominantly quartz with minimal clay minerals, can indicate materials potentially prone to liquefaction. These materials lose shear strength rapidly as a result of vibration, which causes a temporary increase in pore water pressure. Although South Africa is not a particularly earthquake-prone area, the possibility of liquefaction adjacent to mines and heavy industrial areas cannot be excluded.



2 Overview example of the 1:250 000 Land Type Map 2528 Pretoria (1 x 2 size). The Land Type Ba9 and its legend description are highlighted, while the background location is visible. A Broad Soil Pattern map is inset

GENERAL

Information regarding a range of other potential problems and material properties can also be gleaned from the soil maps. This includes:

- presence of a high water table
- presence of organic material (interferes with stabilisation)
- depth to bedrock and/or excessively large stones
- excavatability
- potential material sources

A detailed and knowledgeable examination of the properties provided with the land type maps should provide a lot of useful information regarding the depths and qualities of the soils or materials and should be used as often as possible. It should, however, be noted that the information on the maps and accompanying data is of a fairly general nature (considering the inherent variability of geological materials). It should be used as a preliminary indicator and should not replace a proper engineering geological investigation and laboratory testing. It can, however, be beneficially used to optimise the field investigation and minimise the field and laboratory testing required.

CONCLUSIONS

Information currently available on the land type and soil maps for South Africa can be invaluable for the identification of problem subgrade soils and for the preliminary route location of new roads and other infrastructure (e.g. pipelines and railways). This cost effective information source offers potential throughout the project planning cycle and is currently underutilised by engineering geologists and engineers. It must, however, be accompanied by sound engineering geological experience, examination and practice in order to achieve the optimum benefits.

ACKNOWLEDGEMENTS

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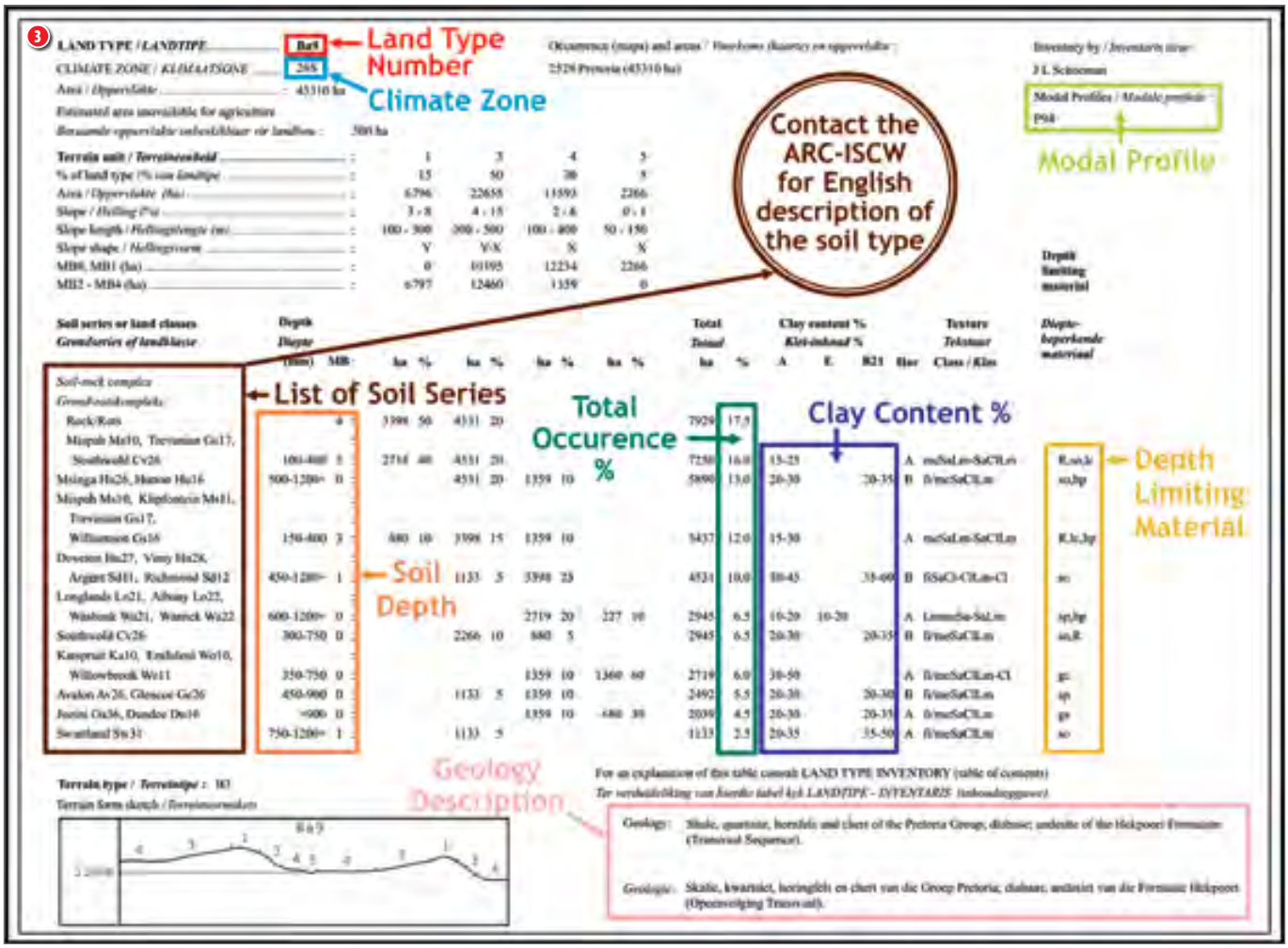
Note

The list of references is available from the editor. ☐



3 Overview example of the technical Land Type Inventory Ba9 (Ba = plinthic catena: dystrophic and mesotrophic soils; red soils widespread) listing terrain units, soil series, selected soil properties and a Modal Profile Number for associated laboratory analyses

Text: **Hennie Barnard**
Africon
hennieb@africon.co.za

Young engineers get a chance to

AS A FINAL YEAR STUDENT at the University of Pretoria I had the privilege last year to write a paper for the 5th South African Young Geotechnical Engineers Conference (YGEC). The conference took place at the Camelot Conference Centre in Hillcrest, Durban, from 20 to 22 August 2008.

The YGEC was a great opportunity for young and upcoming geotechnical engineers in South Africa to become part of the geotechnical network, to become familiar with best practice and to meet other people working in the field of geotechnical engineering. It also gave me the opportunity to publish a paper and to improve my presentation skills. The idea was that everyone who attended would write a paper and present it themselves.

In all there were nine technical sessions, all chaired by young engineers. The most satisfying part for me was presenting my own paper (entitled *The performance of a new ceramic stress cell to measure the total stress in a granular material*) and improving my interpersonal skills during the course of the conference.

Every evening we had a lot of fun socially. The first night was the "meet and greet evening" at which everyone got to know each other in a relaxed atmosphere with snacks and drinks, together with a local performer to provide entertainment. On the Thursday night we all attended a medieval banquet in Camelot Castle; this was definitely one of the highlights of the conference. Bernie Krone (CEO of Esor) was in charge as the king, with the chairperson of the organising committee, Michelle Theron, as his lovely queen. Every lord and lady attending the banquet had to pick a

1

network at the YGEC in Durban

medieval costume to wear for the evening. Much shouting, dancing and eating took place in the castle that night and we all enjoyed the medieval theme. Unfortunately, I had to catch an early flight back to Pretoria on the Friday and wasn't able to attend the final, very interesting site visit to the Moses Mabhida Stadium which is under construction in Durban for the 2010 FIFA World Cup competition.

The winner of the Best Paper was Richard Puchner of Jones & Wagener Consulting Engineers, and the winner of the Best Presentation was Trevor Green of Verdi Consulting Engineers. They will both be sponsored by SAICE's Geotechnical Division to attend the next International Young Geotechnical Engineers Conference. This is planned to take place in parallel with the 2009 ISSMGE International Conference to be hosted in Alexandria, Egypt.

I want to encourage all young engineers and engineering geologists to attend the next YGEC in 2011. The committee of the Geotechnical Division is also planning several events for 2009, one of which will be a course on Problem Soils aimed specifically at young engineers. ◻

1 YGEC participants in front of the Camelot Conference Centre
2 Jacobus Breyl, left, runner-up: best presentation; Trevor Green, winner: best presentation; Richard Puchner, winner: best paper; and Dr Michelle Theron, chairperson of the organising committee
3 Nicol Chang, left (Franki Africa), Mark Laughton (ARQ), Michelle van den Berg (ARQ) and Eulane Heukelman (ARQ) enjoying the medieval banquet

Text: **Redmond Taggart**
RDH Taggart and Associates
office@taggarts.co.za

Building on the theories of our forebears

We live in a world of superfast desktop computers and words like dxf and pdf (if these are indeed words) are part of our everyday conversation. Designs are all carried out using sophisticated computer modelling programs and even in our so-called "paperless society", hundreds and thousands of drawings are produced using advanced CAD programs and high definition plotters. Engineers and technicians who graduated in the last 10 to 12 years will have no knowledge at all of what it was like in the pre-computer era. This is going to be the first of a series of articles describing how designs were carried out not that long ago without the benefit of computers or even electronic calculators

IT IS ABSOLUTELY AMAZING to see what our predecessors achieved without the help of these modern electronic aids. In fact, almost all of the theories and methodology we use today were formulated long before the monitor screens appeared on office desks. I will give you two examples. One was invented by a gentleman, a Flemish gentleman, called Geradus Mercator who saw the need for maps on which angles and distances as shown on the flat map would accurately indicate the same angles and distances on the round surface of the world. The geodetic projection system that he worked out is the basis of all the geodetic systems that we now use. The amazing thing is that Geradus Mercator did all this between 1535 and 1594 – over 400 years ago! Another was Leonhard Euler who, amongst his other achievements, formulated the transition curve in the late 1700s, long before there was a need for it, which was about a hundred years later when the railway systems started to be developed.

Believe it or not, computers existed in those far-off days, but they were not machines, they were people, mathematical whizzkids of the Middle Ages and later, who were recruited from the universities to help do the computations that allowed people like Newton to carry out their work.

Another early genius was a guy called John Napier who, in 1614, discovered the logarithmic scale and, by the turn of that century, the logarithmic slide rule was in common use. Therefore, from 1700 onwards, engineers were able to use slide rules and logarithmic tables to do all their calculations and this remained the case up until the early 1970s – 270 years later. Did you know that, although wonderful engineering works were carried out, the name "Civil Engineer" was not used until the late 18th century and it is thought that the word "Civil" was used to differentiate the profession from military engineers? However, up until the mid-1930s, there was often very little distinction made between civil engineers and architects; in fact they often had a common university degree. It was only after graduating that they had to qualify in their chosen professions. Many did both and I suppose the fact that their names were therefore followed by so many letters, listing their qualifications, led to the invention of the long envelopes that we see today! But seriously, from medieval and even ancient times the so-called architects of structures like cathedrals, monuments, bridges, etc had to be both engineers and architects at the same time. Did I mention the Tower of Pisa? Well, we all make mistakes and,

like the Titanic, if there had not been a big problem with its foundations, nobody would have heard of it!

It was not only in the fields of theoretical scientific research that our computer-illiterate forebears excelled, but also in the advancement of material technology. Brunel's early bridges were made of cast iron. Later, wrought iron came into fashion and then, of course, in the mid-19th century (only 150 years ago) steel came into common use. Today, people boast about the modern invention of composite materials being successfully used as construction materials as if it was something unique to the late 20th century. Reinforced concrete, a composite, has been used for over 100 years!

At university, I used to curse the day that people like Terzaghi, Reynolds, Chezy, Manning and others were born. Wouldn't our degrees have been much simpler if we hadn't had to study and try to understand their amazing theorems, formulae and equations? All of their research was carried out using experiments and the genius of advanced mathematics, some aspects of which they had to invent themselves.

If we look at the works of these people, we may ask what advantages computers have given us? Up until the mid-1970s, a major tool for the analysis of structures was the iterative Hardy Cross method. Although the theory was known, it was practically impossible to use multiple simultaneous equations or matrix analysis. To be fair, larger firms had access to mainframe computers from the early 1960s, enabling them to carry out finite element and matrix analyses, but their use was so cumbersome and expensive that they could only be used for really complicated structures. The theories that even these computer programs used, though, had been worked out decades or even centuries before. It was only from the early 1980s that personal computers changed all that and analyses could be carried out in the blink of an eye, allowing multiple configurations to be analysed.

Another of the millions of examples is in the geometric design of roads. In the olden days this was so cumbersome that one had to get it right the first time. Now it is easy and one can play with horizontal and vertical alignments at will to produce optimum designs. On the other hand, the art of road building has hardly changed at all. One of the huge advantages of computers is that now we have the capacity to build computer models of almost anything – from A380 Airbuses to simple commercial structures. This allows infinitely more flexibility in design and cost-saving efficiencies.

I will end this prelude with a philological thought. Our science, the science of civil engineering, is probably the most inexact science of them all. Our non-computer-literate ancestors studied the properties of materials under load, physical phenomena of fluid flow dynamics and the stability of soils, to name but a few, and formulated equations which fairly accurately model their behaviour and allow us, in this computer age, to carry out safe, economical and successful designs. The fact remains that the materials we deal with can vary dramatically and we base designs on huge assumptions, but the wonder of it all is that by using the theories of our forebears, the wonderful code of procedures and our own experience and judgement, we seem to get it right almost all of the time. ◻

CONJET ASSISTS WITH CHANNEL TUNNEL REPAIRS

TRAIN SERVICES THROUGH the Channel Tunnel linking Britain and France were scheduled to be fully restored in February 2009 following several months of repairs after a major fire in the tunnel on 11 September 2008. A lorry caught fire on a Shuttle train carrying heavy goods vehicles bound for France through the 50-km-long northern tunnel. The fire, which spread to other vehicles, raged for about 16 hours. The extreme temperatures of up to 1 000°C caused extensive damage to about 600 m of the 7,6-m-diameter tunnel's concrete lining, approximately 11 km in from the French entrance.

The Channel Tunnel operator, Eurotunnel, awarded the repair contract, estimated to be between €50 and 60 million, to a consortium of Freyssinet, Eurovia Travaux Ferroviaires (ETF) and Vinci Energies. Freyssinet was appointed to look after the civil engineering works, while ETF would be responsible for track and overhead wire restoration, and Vinci Energies would be responsible for other equipment.

1 *The total area of removal of the fire-damaged concrete with the Conjet hydrodemolition robots was about 9 500 m² over approximately 600 m of tunnel, including about 350 m² directly above the fire where the damage was most severe*

The technique of hydrodemolition, which uses high-pressure water jets to remove concrete from various structures, was specified as the method for removing the fire-damaged concrete. Freyssinet subcontracted the concrete removal to the specialist hydrodemolition and industrial cleaning contractor Philip Lassarat.

For the hydrodemolition part of the repair, Philippe Lassarat hired four Conjet hydrodemolition robots and pumps, together with a team of experienced operators and a project manager from Rotterdam-based Doornbos Equipment, a specialist in the rental of ultra-high-pressure water jetting and vacuum equipment. To meet the tight schedule, Doornbos had to work three shifts round the clock and opted to use two Conjet 364s and two Conjet 322s. Conjet supplied specially modified banana-shaped feedbeams, which carry the jetting nozzle, to match the curvature of the tunnel wall.

"We had to remove the damaged C45 concrete just from the walls and roof to a depth of 30 mm and managed this at an average of 650 m^2/day," says Doornbos project manager Uwe Clausen. "The total area was about 9 500 m^2 over approximately 600 m (length) of tunnel, including about 350 m^2 directly above the fire where the damage was most severe. We removed all the fire-damaged concrete in 14 working days using four robots. We had to work non-stop round the clock in three 10-hour shifts, which included an hour in and an hour out at shift change to get through all the security." The robots were supported by 350 and 400 kW pumps operating at pressures up to 1 000 bar and flows of up to 240 litres/min.

After the removal of the damaged concrete, Philippe Lassarat followed on, repairing and replacing any fire-damaged reinforcing before spraying on the new concrete lining. The various services and utilities were then replaced before the tunnel was handed back to Eurotunnel to restart a full service of the Shuttle trains on 10 February 2009. Eurostar, the company that operates the passenger trains running through the tunnel, anticipated that services would be fully restored on 23 February.

▶ INFO

Lars Göran Nilsson
conjet@conjet.com
www.conjet.com

PIPEHAWK'S NEW *e*-Spott GPR SYSTEM MAXIMISES REINSTATEMENT TESTING EFFICIENCY

With a long history of achievements in the design and manufacture of specialist technology products, PipeHawk Plc is best known for its Ground Probing Radar (GPR) systems which are used around the world for the tracing and mapping of buried utility services. The most recent of these is the e-Spade™ range launched in 2008. Now, with the launch of its

latest system, the *e*-Spott™, the company has turned its attention to meeting the needs of the roads sector.

Designed and built in collaboration with Utsi Electronics Ltd, which is also long established in GPR design, the *e*-Spott™ system provides a fast and reliable method of testing the total thickness of bound layers to the subbase interface or the thickness of surface concrete, without having the major disadvantages of traditional coring methods, such as disruption to road traffic and immediate damage to newly laid surfaces.

The small, lightweight *e*-Spott™ unit has been designed to work in areas of limited access, such as around parked vehicles and other obstacles, and can be easily carried and handled by a single operator. Obtaining a reading is very fast, with each measurement taking less than 30 seconds per spot using a simple one-touch system.

Commenting on the new product, Erica Utsi, Managing Director of Utsi Electronics, said: "This cooperation marks a new departure for both companies. Combining our respective strengths and experience has been key in the development of this innovative and practical testing tool."

For PipeHawk, Alan Jones, Technical Sales Manager, said: "The arrival of *e*-Spott™ brings a new way of establishing the depth of the bound surface layer in a road that is efficient, cost-effective and minimises disruption to traffic flow in the process."

With its small size, light weight, easy operation and a high productivity factor, it will become an essential piece of kit for the roads inspector or utility contractor.

1 *The new e-Spott™ GRP top layer thickness measuring unit*

CUTTING THROUGH MINE DUMPS TO GET TO 2010

THE NEW FOUR-LANE N17 link between Nasrec and Soweto is making good progress towards meeting the completion deadline at the end of 2009. The link is part of the future N17 and forms an additional new link road between Soweto and Nasrec where the 2010 Soccer World Cup stadium is being constructed.

The Nasweto Highway, as it is known, was tendered as a 'Design and Build' by SANRAL (South African National Roads Agency Limited) in order to reduce the design and construction time. The project was awarded to the VelaVKE-Group Five Joint Venture. Design commenced in December 2007, and construction was under way by January 2008.

The project starts at the Soweto Highway on the western side and links to Nasrec Road in the east. In addition to this new link, the project also includes the rehabilitation and widening of 3 km of Nasrec Road, including a traffic circle facility at the N17 – Nasrec Road intersection.

One of the key features of the project is a new multi-span bridge over the N1 using precast beams to minimise disruption to traffic on the N1. These beams have a depth of 2 m and are constructed adjacent to the N1 in a casting yard, transported and then crane-lifted into position.

A further key feature is the new four-lane underpass bridge under the main Soweto

1 *Four-lane bridge in preparation for jacking under rail embankment*
2 *Rail strapping in preparation for bridge jacking*

rail line. This structure will be the largest pre-constructed structure in South Africa and will be jacked into place through the rail embankment.

As far as the road works are concerned, cutting through mine dumps and making effective use of the 'slimes' in the earthworks presented their own challenges. Strict environmental management requirements were incorporated into the design to ensure that contaminated runoff from the mine dumps would be contained in retention ponds and only 'clean' runoff channelled into the local streams.

Other key features include extensive protection and relocation of bulk water supply pipelines, high voltage power lines, oil and gas pipelines and slimes pumping mains.

▶ INFO

Dawie Erasmus
Vela VKE Consulting Engineers
031 240 7300
erasmusd@velavke.co.za

TOWARDS CLEARER HAND SIGNALS FOR CRANE OPERATORS: SANS 10296

FROM THE TOP OF a crane, all the driver often has to go by are the hand signals given to him from someone far below. It follows that improving the clarity of these hand signals is important as this can help prevent misunderstandings, wrong use of the crane, and the hazardous conditions that can cause accidents and injuries. That is why the recent publication of the latest edition of SANS 10296 (full title *Hand signals used with cranes and with lifting and suspended equipment*) is important as it helps to standardise these very important hand signals.

This standard is also being considered as the basis for an international standard. The standards writer, Pierre Terblanche, explains the key issues in his answers below.

Why was the standard written?

The previous edition of this standard came up for revision, and the national technical

committee decided to standardise it for the lifting industry. Coincidentally, at a meeting held in Seoul, Korea (2006) ISO TC 96, *Cranes* enquired about the need for an international standard. As South Africa was then busy revising the standard, we volunteered to draft it as a proposal. The newly published SANS 10296 is therefore currently being considered as the basis for a new international standard.

Why is the standard important for industry?
As the use of all types of lifting equipment has become more common in industry, the need has arisen for communication when using this equipment. Without standardised hand signals, miscommunication can lead to hazardous conditions, including accidents and injuries. On big construction sites, operators or workers from foreign countries are often using this equipment. Without standard hand signals, the same hazardous conditions could arise.

Will the standard be referred to in legislation?
Yes. Driven Machinery Regulations 18 of the Occupational Health & Safety Act are at present in the process of being amended. All applicable SANS standards will be incorporated into a schedule attached to the revised regulations. SANS 10296 will definitely be included in this schedule.

What are the key provisions of the standard?
The contents of the standard are structured around the following basic signals:

- Start and stop (4.2)
- Selection of hoists (4.3)
- Vertical movement of hook (up/down) (4.4)
- Movement of hook and booms (forward/back/left/right) (4.5)
- Movement of crane (4.6)

A wall chart presenting all the signals is being developed.

Who will need to purchase or use the standard?
Absolutely everybody involved with lifting equipment, i.e. users, owners, operators, training service providers, LMIs, etc.

▶ INFO

Irma Loots
Manager: Services and Systems Standards – SABS
lootsi@sabs.co.za

TO PURCHASE THE STANDARD
Standards Sales at the SABS
sales@sabs.co.za

1 *C&CI's Concrete Basics booklet explains the important factors to consider when making and using concrete, mortar and plaster*

STRONG DEMAND FOR C&CI'S BASIC GUIDE TO CONCRETE

THE CEMENT & CONCRETE Institute's informative booklet, *Concrete Basics*, is continuing to prove extremely popular with requests for the free publication emanating from sources as diverse as building companies and DIY enthusiasts.

John Sheath, Marketing Manager of C&CI, says several companies in the construction equipment hire and hardware retailing industries have also requested *Concrete Basics* to use as educational handout to customers.

"The booklet, in simple terminology, explains the important factors to consider when making and using concrete, mortar and plaster. It discusses how and why choosing the best materials, using the correct concrete mix proportions, and ensuring good site practice affect the strength, durability and economy of the finished concrete," Sheath explains.

"It also describes the various processes that can be applied by the small builder, DIY enthusiast or handyman making and using concrete, mortar or plaster."

The use of sand in the concrete mix is just one example of some of the handy hints contained in the publication. C&CI says soil dug out of the garden should not be used because organic materials and/or fertilisers may retard setting, while the use of clay could cause cracking. Pit and plaster sands are too fine for concrete mix and mine dump sand may contain chemicals harmful to the concrete.

The booklet also explains why sea water should not be used if the concrete contains reinforcing steel or metal brackets and fixings: the salt causes the steel in the concrete to rust. When laying concrete floors, on the other hand, panels no more than 3 m wide or long should be laid to allow for expansion and avoid cracking failures.

Finally, and as another example, C&CI says that in bricklaying concrete bricks and blocks should be kept dry, whereas the opposite applies to burnt clay bricks.

For free copies:
C&CI Information Centre
011 315 0300
info@cnci.org.za

▶ INFO

John Sheath
011 315 0300

CMA REPRINTS MANUAL ON DRAINAGE OF CONCRETE BLOCK PAVING

THE CONCRETE MANUFACTURERS ASSOCIATION (CMA) has reprinted its concrete block paving (CBP) manual *Drainage of Concrete Block Paving* on conventional paving installations.

Aimed at architects, landscape architects, engineers and contractors, the six-sided manual provides diagrammatic illustrations on surface and bedding layer drainage.

CMA's Director, John Cairns, says good surface and subsoil drainage are essential for satisfactory CBP performance.

"Drainage should form part of the design, specification and construction phases, and factors such as rainfall, water levels and soil conditions must be taken into account at the outset.

"Tests conducted by Dr Brian Shackel of the University of New South Wales in Australia have shown that between 30 and 35% of rainfall will penetrate newly laid, untrafficked and unsealed block pavements. Therefore the drainage of bedding sand is crucial, particularly in instances where stabilised bases are used or in areas close to kerbs, edge beams and manhole inlets. This aspect is often neglected and leads to a build-up or 'boxing' of water and the progressive failure of CBP," notes Cairns.

He advises that the permeability of joints can be reduced by up to 50% with the application of a water-based acrylic sealer.

"Similarly, infiltration can be inhibited by adding 10% of lime or 6% of bentonite to the jointing sand. And a bituminous seal should be considered for all unbound base material before bedding sand is placed."

The manual provides a bedding sand grading with nominal sieve sizes ranging from 9,52 mm to 0,075 mm, and various surface drainage diagrams depicting drainage and gutter applications are included. There is also an illustration of a manhole inlet, where special attention must be paid to compaction and the possible stabilisation of backfill.

Bedding layer diagrams include sand-bed drainage details on steep slopes and three drainage pit drawings. Diagrams of CBP on a concrete slab and flownet drains are also given.

The manual includes a list of CMA paving block producer members, associate members and contractor members.

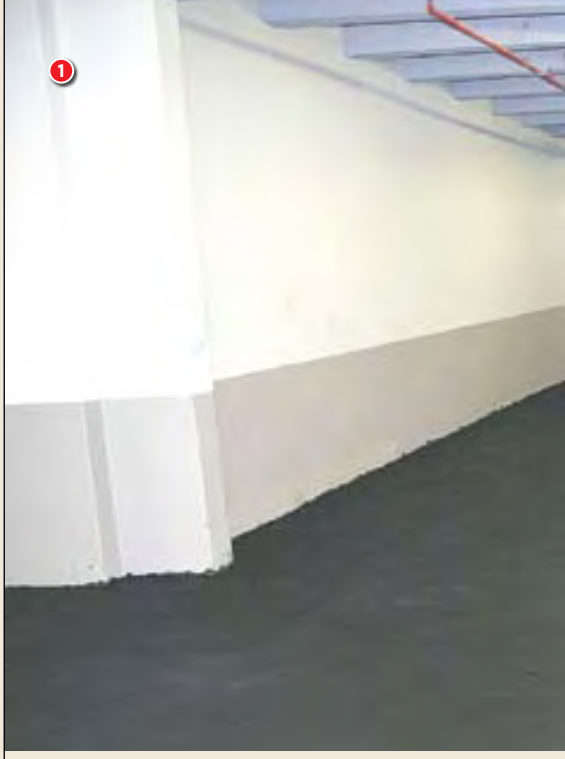
For free copies:
Pam - 011 805 6742
main.cma@gmail.com

▶ INFO

John Cairns
011 805 6742

a.b.e. HEAVY-DUTY FLOORING SYSTEM FOR SANDTON PARKING BASEMENT

A HEAVY-DUTY MASTIC flooring product from a.b.e. Construction Chemicals was specified for the extensive parking garage



at ABSA Capital in Fredman Drive, Sandton, Johannesburg.

Eric Gouws, Flooring Technical Sales Consultant for a.b.e. in Boksburg, says the need for a hard, jointless, non-dusting flooring system influenced the decision to specify abeco mastic, applied in a 12 m thickness. "The floor, furthermore, had to be resilient and able to handle heavy traffic. The product (abeco mastic) has an excellent track record in South Africa and has already been successfully applied to single areas exceeding 10 000 sq m. The basement parking area at ABSA Capital is about 2 500 sq m in extent," Gouws stated.

abeco mastic is a cold-laid bituminous mastic consisting of stone chippings, portland cement, sand and flintkote 3 binder. The flooring system – which does not require any expansion or contraction joints – has a service history of around 50 years on the local market and is suitable for areas such as warehouses, railway platforms, printing works, heavily trafficked corridors,

1 *a.b.e. Construction Chemicals' specialist flooring product, abeco mastic, was used for the flooring of the basement parking area at ABSA Capital in Sandton*

The contractor for the flooring installation at ABSA Capital was R Tingtinger Waterproofing, with sub-contractor, Phermafloor, handling the application of abeco mastic.

▶ INFO

Ian Hague
a.b.e. Construction Chemicals
011 306 9000
www.abe.co.za

textile factories, and loading bays. It dries to a dark brown colour when laid to a nominal 12 mm thickness.

Some of the other benefits of this product are the following:

- Non-slip, verminproof, dampproof, and non-flammable
- Self-healing under the heaviest of traffic loads – construction joints disappear after a few days of traffic exposure
- Does not disintegrate and can be laid over any firm, sound, clean sub-flooring such as concrete, asphalt, well-compacted crusher run, steel and timber

MASSIVE BREWERY TO BOOST FLAGGING SPIRITS

GRINAKER-LTA IS currently one of the contractors building the new Heineken Brewery in Kliprivier, south of Johannesburg, on a massive 1,5 km^2 site adjacent to the R59 freeway.

1 *Aerial view of the new Heineken Brewery at Kliprivier showing the sophisticated slab construction in the bottling area*

The Grinaker-LTA contract was awarded in May 2008 and is due for completion in August 2009.

The contract was awarded in numerous different phases, with phase 6 being the external building works, offices, security buildings, etc. The team is well advanced with the contract structures, including the brewing house, utilities and malt-treatment buildings, which will be complete for equipping in due course.

The buildings at the brewery are mainly concrete, structural steel and brick structures with steel roof sheeting. Some 30 000 m^3 of concrete will be used in the project together with 3 500 t of structural steel and two million bricks.

The 70 000 m^2 brewery is very much a standard design for Heineken. It is not certain at this stage which brand of beer will be produced at this plant as Heineken owns various well-known branded beer names, but the brewery will be capable of producing approximately 3 million hectolitres of beer per year and will provide much-needed employment for the local community.

▶ INFO

Melanie Karusseit
Grinaker-LTA Civil Engineering
011 923 5000

SUCCESS FOR KAYTECH'S QUICK4 INFILTRATOR CHAMBER SYSTEM

WHEN THE OWNER WANTED to subdivide his property in West Riding, KwaZulu-Natal, with two dwellings on the property – a house and a cottage – a problem arose as it was discovered that both dwellings discharged their individual septic tanks into the same soak pit. Consultant Michael Haviland recommended Kaytech's Quick4 Infiltrator Chamber System for the job.

The grassed site is rectangular in shape and level, but with a general slope of 1:3 in a south-easterly direction. The subsoil profile with depth consists of a silty, sandy topsoil layer, 250 mm thick, and a 600 mm layer of moist, silty sand with sandstone fragments, underlain by highly weathered sandstone.

"In accordance with the local Metro's guidelines and National Building Regulations, a soil percolation test was performed, establishing a percolation rate of 90 mm per hour. Effluent flow for the house was taken to be 900 litres per day," says Kaytech's Technical Marketing Director, Garth James. "Based on this information, the engineer decided that the existing disposal system should be retained for the cottage, and he designed a new septic tank and a 17-m-long soak pit for the house."

The new soak pit was formed by joining 14 Quick4 Infiltrator Chambers end-to-end. The design of the new Chamber System permits a 10 degree swivel at each join, allowing the construction to follow the contour of the

1 *The Chamber fitted with a Multi-Port End Cap*

land. McLeod Plumbers were the contractors involved in this project.

"The Quick4 Infiltrator Chamber System is a direct replacement for the stone-and-pipe storm water and wastewater French drainage system. One of the advantages is that, because of the greater infiltrative capacity, the Chambers require only half the space of the old-fashioned French drains," explains Garth James. "In addition, a traditional French drain tends to clog with the intrusion of surrounding soil and can result in surface settlement over time. With the Quick4 Infiltrator Chamber, inspection ports can be installed (three in this case). These ports also allow for the cleaning out of any accumulation of sludge in the soak-away system."

The system is cost-effective in that it is lightweight, easy to transport and quick to install. It is chemical and UV resistant and has a large storage capacity at 235 litres per unit. The entire base area is open to infiltration, making it very effective. Other benefits of this system are: on-site clean-up is much easier due to the elimination of stone; the system can be expanded or relocated; and it can be used with Multi-Port End Caps which allow piping to enter or exit the system from multiple directions and eliminate pipe fittings.

▶ INFO

031 717 2300
megan@kaytech.co.za

INFRASTRUCTURE DEVELOPMENT IN MISSIONVALE

THE INFORMAL TOWNSHIP of Missionvale in Port Elizabeth forms part of the wider Nelson Mandela Metropole. The area is located approximately 12 km north-west of the CBD of Port Elizabeth, bordered by Bethelsdorp Salt Pan on the western side and the old Uitenhage Road on the eastern side. The township still operates with a bucket toilet system and has communal taps for water supply, very few recognisable roads and many illegal electrical connections.

Civil engineer Percy Makofane from Sigodi Marah Martin Development and Consulting Engineers is involved in the Missionvale roads and storm water infrastructure development initiative under the mentorship of his manager, Ross Grainger.

The informal settlement currently has an estimated population of around 13 000. There is a care centre, a clinic, a primary school and a community hall to serve the residents. Plans for the new RDP housing development have now started in earnest. The newly proposed

1 *General view of the Missionvale area with the salt pan in view*

bulk storm water project, commissioned by the Nelson Mandela Bay Municipality, is one of many initiatives that will help to alleviate the poor conditions in the area.

"The residential project will provide 2 499 formal residential erven for the area, together with piped water, waterborne sanitation and roads. A formal internal storm water system will be installed to manage storm water runoff and prevent the flooding of properties, while the internal roads will be surfaced in stages," explains Percy.

A major challenge during the sewer and storm water design phase was the flat topography of the area. Sigodi Marah Martin has therefore invested in the infrastructure design software Civil Designer to produce all the preliminary designs for the roads and storm water system. The company has also completed the design for the water and sewer networks using the program.

The topography of the area is such that an adequate storm water system is critical. The proposed system will consist of both pipes and canals, leading to detention ponds. Storm water runoff from the proposed development area which reaches the berm protecting the salt mine flows to the southern end of the pan via a storm water canal where there is an evaporation area and dam, and a municipal pumping station.

"When the water levels rise above a certain level in the dam, the pumping station will discharge any excess storm water into the Papenkuils River catchment. The project team were able to determine the runoff factors after a careful analysis of the geology, the terrain, the vegetation and the land use in the area," Percy continues.

The storm water master plan makes provision for major storm events which will be accommodated using the proposed formal drainage system, runoffs via major roads and the existing natural drainage system. The ponds within the township form an integral part of the storm water management system as storm water is directed to or routed through them. The ponds temporarily detain the storm water and then release it at a controlled rate, thereby decreasing the peak flow.

Where necessary, low-lying areas or ponds required for development will be filled. The ground levels will also be artificially raised and the area landscaped in order to ensure overland flow. Eliminating some of the smaller ponds will reduce disease-harbouring stagnant water and improve environmental conditions. The bulk storm water drainage system has also been designed as far as possible to accommodate closed pipes rather than open drains in order to prevent contamination with debris.

The project is currently in its final stage of design and is likely to proceed to the construction phase by 2010.

INFO

Charles Scott
Knowledge Base
info@knowbase.co.za

A woman of substance

ON WEDNESDAY 18 MARCH this year the University of Stellenbosch conferred an honorary doctorate on Allyson Lawless, who was SAICE's first lady president in 2000, and who still serves on SAICE's Council.

This pioneering lady engineer received the degree Doctor in Engineering (DEng), *honoris causa*, for her excellence as a civil engineer and scientist, leadership as a businesswoman and being a role model for young people from all backgrounds.

Allyson Lawless qualified from the University of Natal (now the University of KwaZulu-Natal) as a civil engineer at a time when there were only a handful of female engineers in South Africa. She then went on to complete an MSc in Structural Engineering at the Imperial College in London.

After her registration as a Professional Engineer in South Africa in 1978, Allyson became a Chartered Engineer of the Engineering Council of the United Kingdom. She was only the second woman to become a Fellow of the South African Institution of Civil Engineering and the third to become a Fellow of the Institution of Structural Engineers (IStructE) in London, and of the South African Academy of Engineering (SAAE). She also served on the Executive Committee of SAAE from 2004 to 2006. Allyson has filled leadership roles in these organisations and was the International Vice-President of IStructE in 1997/1998 – the first woman to hold this position.

In addition, Allyson serves on the council of the University of Venda, and the JIPSA Technical Panel. The Deputy-President has also appointed her as an Energy Champion.

In the 1980s, her first company, Allyson Lawless (Pty) Ltd, was at the cutting edge of the development of affordable spreadsheet software for civil engineering. She has always had a lively interest in the technical skills of engineers, particularly those from disadvantaged backgrounds. After identifying large-scale skills gaps in local government, she devised a government intervention under the banner of SAICE by which retired professional engineers are reappointed in the service of local governments. Here they support (primarily black) students who need practical training or graduates who cannot find work.

After extensive research, Allyson published two books on skills development: *Numbers and Needs,* and *Numbers and Needs in Local Government*. In the latter she provides a turnaround strategy for service delivery in local government, which could change the face of this crucial sector. The findings resulted in the launching of major national initiatives, including increased funding to tertiary engineering departments.

1 Allyson Lawless with Professor Russel Botman, Rector and Vice-Chancellor (left) and Dr Frederik van Zyl Slabbert, Chancellor of the University of Stellenbosch. Allyson was one of only two people on whom an honorary doctorate was conferred by the University of Stellenbosch on 18 March 2009

Allyson has received a number of awards, among others being the 2007 winner of the Shoprite-Checkers/SABC2 Woman of the Year in the category Science and Technology, as well as being the winner of the prestigious National Science and Technology Forum category for an individual for her outstanding research in science, engineering and technology in South Africa.

All this and much more Allyson Lawless has achieved while raising two children!

▶ INFO

Marie Ashpole
Outreach Officer: SAICE
mashpole@saice.org.za

Agrément approval for precast foundation beams

THE CONCRETE MANUFACTURERS ASSOCIATION (CMA) has been given the green light by Agrément South Africa to license its precast concrete slab producer members to manufacture precast concrete foundation beams for deployment in the construction of affordable single-storey houses.

Providing they comply with the specifications of Agrément Certificate No 2008/350, CMA member companies will be able to manufacture CMA Building Foundation Beams, as they are to be known, for houses with maximum dimensions of 6,6 x 6,2 m.

CMA Building Foundation Beams offer certain advantages over foundations built in situ. They are far quicker to install and therefore more cost-effective. They are also ideal for supporting suspended flooring consisting of precast concrete slabs, thus completely eliminating the need for any 'wet' trades for foundations and ground floors.

CMA Director John Cairns says that what this means is that yet another use has been found for the multi-faceted precast slab in South Africa which, besides its prime function as a precast flooring material, is being used in applications as diverse as security and retaining walls, warehouse walls, and walls and roofs for reservoirs.

FOUNDATION BEAM SPECIFICATIONS

The Foundation Beam Agrément Certificate specifies that the beams should be 200 mm wide by 400 mm deep and have a maximum length of 6,2 m. Any deviation from this specification will not be allowed.

The Certificate specifies further that the design, manufacture and installation of precast concrete beams should comply with established manufacturing processes and codes, and these should be under the control of a professional engineer or other approved competent person to ensure that the requirements of the Certificate are met.

Moreover, the Certificate holder's quality management system needs to comply with Agrément South Africa's requirements. For instance, all the materials used in the manufacturing process are required to meet the following national standards: SANS 1083 for aggregates and sand; SANS ENV 197 for cement; and BS 5896 for prestressing steel.

Certificate holders are also obliged to ensure that the founding conditions are correctly classified and that any threat of underscour due to water erosion adjacent to the foundation beams is addressed. Adequate drainage and the prevention of water ponding close to the beams is another aspect that requires the input of an engineer.

Over and above Agrément South Africa's performance criteria for durability and quality management, the National Home Builders Registration Council (NHBRC), which states that building sites should be classified as either C or H in accordance with the Code of Practice for Foundations and Superstructures for single-storey residential masonry buildings, must also be met.

The accurate preparation of foundation trenches is another Agrément prerequisite. To ensure this, special attention must be given to the training of workers. It is imperative that foundation beams rest on the same plane and the maximum permissible variance is 15 mm.

The base of each foundation trench must also be horizontal, with the maximum allowable "out of level" tolerance being ± 15 mm. This will ensure that the maximum difference between the highest and lowest points of the top surfaces of the beams is no greater than 30 mm. Compaction of trench bases may also be required if soil has been disturbed through excavation, and in some cases concrete blinding may be required to ensure proper bedding.

Once in position, the line and level of the beams must be checked further to ensure they are parallel and that the floor slabs will rest horizontally. The foundation trenches can then be backfilled and hand-compacted in layers not exceeding 100 mm.

The exposed ends of the beams must be closed off with a stiff 1:3 cement/sand mortar trowelled flush and, once dry, waterproofed with a cementitious coating at least 23 mm thick to prevent corrosion of the prestressing steel.

CMA Building Foundation Beams are designed to provide stable support for precast floor slabs and this support should be sufficiently horizontal for the construction of walls. Moreover, the precast floor slabs must be designed to be adequately stiff to avoid deflections that could cause internal walls to crack.

Drainage and details of articulated pipe connections must meet Section 6.3 of the Code of Practice – Foundations and Superstructures for Single-Storey Residential Buildings of Masonry Construction, issued by SAICE and IStructE in 1995.

In summary, the engineer or other competent person is required to ensure the following:

- Founding conditions should be either Class C or H and be classified as such by a qualified geotechnician or a professional engineer.
- The uniformity or otherwise of the founding soil conditions must also be assessed by the geotechnician.
- The predicted differential foundation movement should not exceed 4 mm.
- Soil bearing pressures must not exceed the recommended limits as set out in SANS 10161 Code of Practice for *The design of foundations for buildings*.
- The possible effects of differential settlements in collapsible soils must be assessed.
- Matters pertaining to site drainage must be adequately addressed in order to ensure that ponding of water adjacent to the beams does not occur.

Foundations other than the precast beams must also be properly designed to support the building, and a building's superstructure should under no circumstances exceed the performance criteria of its foundation.

MANUFACTURING PROCESS

Like all precast slabs, CMA Building Foundation Beams are cast in a continuous process on prestressing beds. Reinforcing steel, comprising 2 x 4 No 7 mm diameter strands, should be symmetrical at both the top and bottom of the beam. The bed must be cleaned and lubricated prior to casting and care must be taken to ensure that the lubricating oil does not come into contact with the strands. If this does occur, the oil must be removed by wiping the steel with a cloth.

Concrete should be batched by weight on a calibrated scale and the required mix proportions should yield a minimum crushing strength of 50 MPa after 28 days and 35 MPa after 18 hours, when distressing (i.e. the cutting of the prestressing steel) takes place.

Curing of the beams is accelerated through the use of steam-heated moulds, a process during which the temperature must be maintained at between 60 and 70°C. Once poured, the concrete should be covered with polythene sheets to assist with the curing. Each production batch must be tested in accordance with SANS 5863.

The beams should be identified by means of metal tags indicating the panel mark, project name, project number, production sequence number and date of manufacture. Beams passed by the quality controller should be marked with green spray paint, and those rejected with red spray paint.

TRANSPORTATION AND SITE HANDLING

CMA-approved foundation beams should be transported to site on trucks and be loaded and off-loaded with cranes; care should be taken to ensure that the beams are safely strapped onto the truck. Building sites should be prepared to permit the trucks access and allow for the unimpeded operation of cranes. Furthermore, all aspects of the Occupational Health and Safety Act must be adhered to during transportation and site handling.

INFO

John Cairns
011 805 6742

George Donaldson
leaves a wonderful legacy

FOR MANY YEARS GEORGE was a regular visitor to SAICE National Office. He was a dedicated volunteer and contributed his skills freely to the many committees on which he served.

He was also a giant in engineering and the story of his illustrious career was told on the pages of the SAICE magazine in December 1994, when he was awarded a SAICE Honorary Fellowship.

It was then with great sadness that we learnt of his recent passing. Some time after the sad news, however, we received an unexpected and very positive letter from his estate attorneys – George had generously left R10 000 in his will to SAICE!

We are grateful to announce that we have now received the cheque and have decided to donate the R10 000 to SAICE's SPEBS bursary scheme to assist two deserving civil engineering students. To commemorate this special contribution, it was decided to name the two bursaries: The George Donaldson investment in the future of South Africa. We salute his valor and cherish his memory.

SATC 2009

THE 28TH SOUTHERN AFRICAN TRANSPORT Conference (SATC 2009) with the theme "*Sustainable Transport*" will take place at the CSIR International Convention Centre (CSIR ICC) from 6 to 9 July 2009. The Minister of Transport, who is the patron of the conference, has been invited to open the conference. The opening plenary address will be presented by Mr Randell H Iwasaki, Chief Deputy Director of the California Department of Transportation (Caltrans), USA. Daily plenary addresses will be delivered by other eminent overseas speakers.

The conference will focus on one of the major concerns of this millennium, namely *sustainable development*, which will be debated in the following sessions:

- Capacity Building
- Infrastructure
- Passenger Transport
- Port Infrastructure
- Rail, Air and Transport Logistics
- Sustainable Transport
- Traffic Engineering
- Traffic Management, Safety and Security
- Transport Planning

In addition, SAICE will have a follow-up to the one-day symposium presented at last year's conference on *Progress with Transport Operation Plans for the Soccer World Cup in 2010*.

An accompanying exhibition is also being arranged.

Contact details for further information:

Ammie Wissing - Conference Secretariat
012 348 4493
wissing@iafrica.com

SAICE Photo Competition 2009

2008 winner: 'Dawn of 2010' by Philip Bateman

Rules

1. The competition is open to the general public to submit photographs.
2. It is essential that entries portray people and/or projects in civil engineering.
3. Photographs will be judged in ONE general category only.
4. Entries must be colour prints and in A4 size. Only quality prints will be accepted. Please supply electronic copies of the print/s in jpeg format, 300dpi.
5. Please complete an entry form for each entry and supply an appropriate title & short description of each project. It is essential that the photographer's name is included.
6. Please supply details of the client, consultant and contractor involved in the project.
7. The entrant is responsible for obtaining permission for the use of the photographic material as well as subject material from the authority or project manager concerned.
8. Entries submitted by organisations must be accompanied by written consent of the photographer.
9. Permission for the reproduction of photos for any exhibition or publicity is assumed unless the entrant specifies otherwise. Due recognition will be given to the photographer.
10. No responsibility will be accepted for any loss or damage to entries.
11. Closing date: **22 July 2009**

Entry Form

saice

This section must be completed by the person submitting the photo/s

NAME_____

ADDRESS_____

_____

TEL_____ FAX_____

E-MAIL_____

PHOTO TITLE _____

DESCRIPTION_____

_____

PROJECT INFO _____

_____

PHOTOGRAPHER_____
(Name and surname of the photographer to be inserted)

This section must be completed by the photographer or the company that owns the photo.If you are not the photographer or if you are submitting the photograph on behalf of a company owning the photograph, please sign 'on behalf of'.
I hereby grant permission for reproduction and agree to abide by the rules of the competition.

Signature:_____

Please complete the entry form and send to: Private Bag X200, Halfway House, 1685. Fax: (011) 805 5971. This form is available on the SAICE website: http://www.civils.org.za/portals/0/pdf/pc/pc-entry-form.pdf

Date	Event and CPD validation number	Presenters	Contact details
2 – 5 June Durban	Project Management *SAICEbus07/00252/10*	Tony Lydall	Sharon Mugeri cpd.sharon@saice.org.za
5 – 6 May Durban 2 – 3 July Gauteng	Technical Report Writing *SAICEbus09/00427/12*	Les Wiggill	Sharon Mugeri cpd.sharon@saice.org.za
18 – 19 June Johannesburg 22 – 23 June Johannesburg 25 – 26 June Nelspruit 9 – 10 July Polokwane 13 – 14 July Mmabatho 16 – 17 July Kimberley 20 – 21 July Bloemfontein 23 – 24 July Pietermaritzburg 27 – 28 July Richards Bay 30 – 31 July Durban 3 – 4 August East London 6 – 7 August Port Elizabeth 11 – 12 August Cape Town 13 – 14 August Cape Town 17 – 18 August George	GCC 2009 *CPD number to be announced*	Les Wiggill	Dawn Hermanus dhermanus@saice.org.za Sharon Mugeri cpd.sharon@saice.org.za
11 – 12 May Cape Town 25 – 26 May Port Elizabeth 8 – 9 June Nelspruit 27 – 28 July Bloemfontein	Handling Projects in a Consulting Engineer's Practice *SAICEproj08/00404/11*	Wolf Weidemann	Dawn Hermanus dhermanus@saice.org.za
12 May Midrand	Bridge Maintenance *SAICErail09/00495/12*	Ed Elton	Dawn Hermanus dhermanus@saice.org.za
14 – 15 May Cape Town 28 – 29 May Port Elizabeth 11 – 12 June Nelspruit 30 – 31 July Bloemfontein	Business Finances for Built Environment Professionals *SAICEfin08/00405/11*	Wolf Weidemann	Dawn Hermanus dhermanus@saice.org.za
25 May Cape Town 01 June Midrand 27 July Port Elizabeth 21 September East London	Structural Steel Design Code to SANS 10162: 1-2005 *SAICEstr06/00050/09*	Greg Parrott	Sharon Mugeri cpd.sharon@saice.org.za
26 May Cape Town 02 June Midrand 28 July Port Elizabeth 22 September East London	Reinforced Concrete Design to SANS 10100-1 *CPD number to be announced*	Greg Parrott	Sharon Mugeri cpd.sharon@saice.org.za
26 – 27 May Midrand	Critical Factors in Information Technology Investment Success *SAICEit08/00346/11*	Dr James Robertson	Sharon Mugeri cpd.sharon@saice.org.za
3 – 4 June Midrand	Basics of Track Engineering SAICErail09/00496/12	Ed Elton	Dawn Hermanus dhermanus@saice.org.za
6 – 12 June Gauteng 4 – 10 July Cape Town 15 – 21 August Durban	The Application of Finite Element Method in Practice *SAICEstr06/00018/08*	Roland Prukl	Dawn Hermanus dhermanus@saice.org.za
22 June Port Elizabeth 24 June East London 26 June Durban	Ridding Stormwater of Litter *SAICEwat08/00361/11*	Prof Neil Armitage	Sharon Mugeri cpd.sharon@saice.org.za
16 July Midrand	Essential I.T. Knowledge for Business Executives *SAICEit08/00345/11*	Dr James Robertson	Sharon Mugeri cpd.sharon@saice.org.za

For more information on courses, venues and course outlines please visit **http://www.civils.org.za/courses.html** or contact **cpd.sharon@saice.org.za**