GAUTRAIN CONSTRUCTION
SOUTHERN VIADUCT SECTION
PRECAST CONCRETE YARD

PROJECT MANAGEMENT
RAMP
VRESAP SITE VISIT
ON THE COVER
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Leading geotechnical solutions company Esor, part of the JSE-listed Esorfranki group, recently completed the excavation of Gautrain pier foundations consisting of six 7-m-diameter shafts at depths ranging from 21 to 29 m. The Centurion contract, awarded to Esor by Bombela Civils Joint Venture (CJV), was valued at approximately R34 million.

According to Esor senior contracts manager Anton Naude, the main challenges of the contract were the dolomitic conditions, the necessity of blasting in a built-up area close to busy offices and the presence of ground water as three of the six shafts are founded below the water table.

Dolomitic conditions are never easy to work in and demand expert geotechnical investigation and solutions that take into account the highly variable nature of the rock.

Esor’s solution was to sink the shafts using drill-and-blast methods while supporting the sidewalls using soil nails acting in conjunction with fibre-reinforced gunite and mesh reinforcing.

“The challenge of shaft sinking in dolomite is that the rock formation is decidedly uneven as one goes down, with dolomitic formations such as pinnacles, cavities and large boulders (known as “floaters”) of different sizes occurring fairly randomly until solid bedrock is found,” says Naude.

This unevenness is caused by dolomite’s susceptibility to dissolution, which results from the percolation of subsurface water. During this weathering process a residuum known as “wad” – a very soft material which is easily eroded or compressed and is unsuitable for the founding of Gautrain pier foundations – is left behind.

Esor’s contracts manager Jonathan Day explains: “Wad is very weak and tends to fall out during excavation, resulting in overbreak of the shaft sidewalls. This overbreak must then be filled up behind welded mesh and covered with gunite.”

A good understanding of the underlying geotechnical structure is a prerequisite for a job of this nature. One has to know as accurately as possible how much blasting will be required, the position and extent of cavities within the underlying strata, and the depth to suitable bedrock onto which the shaft can be founded.

Bombela CJV did this preconstruction work themselves; five percussion-drilled probe holes were sunk per shaft, to a maximum depth of 50 m, while measuring the penetration rate, thrust pressure, torque, air pressure and air flow. This, coupled with borehole radar surveys, provided an accurate rendition of the position of cavities, the presence of variable rock and the depth to suitable founding rock.
“So, before we began excavation we knew fairly accurately where the cavities were, where the protruding rock was, at what depth the bedrock was and the relevant penetration rates,” says Day.

A critical factor in the construction of shafts of this nature is to decide where the bottom of the shaft is. The toe of the shaft has to be socketed a minimum of 1 m into hard rock free of any wad material and open fissures greater than 0,5 mm. The designed depth of the shaft is predetermined by the information gathered from the geotechnical investigation and then confirmed once the specified depth has been attained.

Once the bedrock has been reached, a series of five percussion-drilled probe holes are sunk 12 m into the rock below the base of the shaft. “We record penetration rates and take drill chip samples of the rock at intervals of 1 m to check the consistency of the rock,” says Jonathan Day. The holes are then water-tested and pressure-grouted.

This is known as primary grouting. Based on the grout volume takes of each of the primary holes, secondary holes may be drilled, water-tested and pressure-grouted in the vicinity of any primary holes that had high grout volume takes. In some cases even tertiary and quaternary grouting may be required. The grout fills any fissures or voids below the base of the shaft and seals off any groundwater, affording dry conditions below the water table in which to work.

The grouting results, including the percussion-drilling penetration rates and drill chip samples, are then used to determine the suitability of the rock for founding the base of the shaft.

Once this is complete, the shaft is handed over to the client, in this case Bombela CJV; they cast a reinforced-concrete base plug and the 0,5-m-thick permanent shaft lining. The steel cage of the base plug is fixed on surface and then lowered onto the base of the shaft. This requires a minimum shaft diameter of 7,0 m. Conversely, to prevent excessive wastage of concrete while casting the permanent shaft lining, a maximum shaft diameter of 7,2 m is allowed.

On completion of casting the permanent shaft lining, and backfilling the shaft with compacted soil, a concrete cap is then cast on top of the shaft. This forms the foundation onto which the viaduct pier is constructed.

Another significant challenge was working near the Momentum Life building...
which contains a significant number of computer servers and other sensitive equipment. Here Esor had to be extremely cautious of the vibrations caused by blasting the rock just below ground level.

To achieve this Esor used a device called a “boulder buster”. This device uses a cartridge filled with explosives to create a pressure impulse, which is directed into the rock via the machine’s barrel which, in turn, is placed into a predrilled hole filled with water. When the device is fired, the resultant impulse is transferred by the water into the rock and causes the rock to break.

Regular blasting techniques using explosives were used in the shafts that were not close to sensitive buildings.

The six shafts were completed by the end of May 2009 and handed over to Bombela C JV for construction of the permanent shaft lining and backfilling. In total, 154 m of shaft were excavated and temporarily supported.

Esor has for more than 30 years been a stalwart in the provision of geotechnical services to the South African construction industry. Its claim of being the only specialist company in the country able to offer the full range of geotechnical services is vindicated by the wide range of skills it possesses which, in addition to shaft sinking, include piling, diaphragm walling, lateral support, pipejacking, grouting, percussion drilling, guniting, underpinning, dewatering, marine works and pile integrity testing.

INFO

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Creating wealth in the new South Africa – an engineering approach

IN CONSIDERING THE question of how to create wealth in a country, engineering training immediately directs us to seek precedents from which we can learn.

Consideration of those countries in the world where there are high standards of living and limited or no poverty immediately shows that there is a single common thread, namely effective and efficient use of productive technology to gear the capabilities of the people of the country. Switzerland, Germany and Japan are examples that spring to mind. It is very evident from visits to these countries that their technology is well designed, well implemented, well maintained and well operated. Things “just work”.

So, one finds that the effective application of technology is the cornerstone of creating wealth in any country – it is the only proven recipe.

Random social experiments that jeopardise the reliability and sustainability of technology are fundamentally at odds with any prospect of a vibrant economy, particularly when those policies involve diverting the money that should be going to infrastructure development and maintenance to rewarding people on the basis of criteria that cannot be linked in any way to wealth creation.

Reflection on this principle immediately indicates that South Africa is going in the wrong direction as far as technology is concerned. It does not matter what technology or methodology one considers, be it roads, sewerage, water supply, electricity supply, electricity distribution, health care, education, justice, etc., in all cases once-reliable and sustainable infrastructure and capability is progressively degrading as a consequence of failed experiments such as outcomes-based education, the abolition of the death penalty, lack of maintenance as in the case of roads, water supply, electricity distribution, etc.

Irrespective of where the money and manpower have gone that have resulted in this widespread degradation of technical capability and assets, it has been inappropriately applied.

If South Africa truly aspires to create sustainable wealth, we need to cease redirecting funds from infrastructure, whether it be hard infrastructure such as roads or soft infrastructure such as health care professionals, teachers, engineers, etc. More critically, we urgently need to stop driving experienced personnel at all levels of economic activity out of the country on the basis of their skin colour. Reverse apartheid coupled with a lack of recognition of the vital role of knowledge and experience in creating wealth is rapidly crippling the capacity of South Africa to create sustainable wealth.

We stand at a watershed – can government refocus to retain and grow technical capability and infrastructure or will it persist with a policy of endowment at the expense of sustainable wealth creation? Redistribution does not create wealth, it redistributes wealth away from those with the proven ability to create wealth. If remuneration does not relate directly to productive input that creates more value than it consumes, then the economy will progressively degenerate. Arguably, there was indeed a need to level the playing field, but the problem now is that the playing field is no longer level: it is tipping in the opposite direction, towards national impoverishment.

Spending money on elaborate infrastructure, such as prestigious soccer stadiums, does not create wealth. Instead it simply creates an illusion of wealth but does not sustain productive activities which create material value that significantly exceeds the cost of inputs.

ENGINEER AGAINST FAILURE

In the light of what I have written above and what follows, I would like to position myself. I am first and foremost an engineer – my passion is engineering and solutions that work. I first started designing and making things when I was about six years old and I have been doing this ever since.

When I was three my father was erecting a trellis. A steel pole slipped from his hand and split my skull, leaving me with a deep-seated abhorrence of failure and a love of solutions that are elegant and work reliably and dependably. This grounding has been pivotal in shaping my career.

Having graduated at Wits in 1976 as a civil engineer, I went on to do a PhD in construction materials and then found myself having to learn how to use computers in order to process my laboratory test results.

I soon discovered an aptitude for the strategic value-adding application of computers in business and in 1989 set out on my own to bring the “disciplines of engineering” to the information technology industry.

I rapidly became aware that I placed strong emphasis on preventing failure, but it took me about ten years to realise that as engineers we are not trained to design bridges to stand up, we are trained to design bridges not to fall down. This is a fundamental difference in approach that I have applied to all aspects of my work as a management consultant and strategist.
"Engineer systems and solutions NOT to fail" has become the cornerstone of my approach and is as fundamental to the design of business strategies and national policies as it is to businesses.

Until we formulate strategic business plans and national policies not to fail, we will continue to find that failure is rampant as it is evident in many areas of South African wealth-producing and maintaining economic activity today.

This worldview leads me to the conclusions presented above – as a nation we are going in the wrong direction – we are doing things that are destroying the sustainability of wealth creation wholesale, instead of doing all that is required to preserve and grow our capacity to create wealth.

As a nation, we are not talking about failure and how to prevent it, and so it is rampant, whether in the form of a blowout after hitting a pothole, a rundown hospital with doubtful hygiene, children who leave school ill equipped to compete on the global playing field or electrical transformers silently decaying as a consequence of an invisible lack of maintenance. Just below the surface of our apparent prosperity lies a mountain of technical neglect that is destined to dramatically damage our economy in the near future.

I am passionate about success through implementing exceptionally high-value strategic concepts cost-effectively and timeously to achieve success by preventing failure. This is missing from the current South African economic and technology arena.

**STRATEGY DEFINED**

What is strategy? Strategy is the essence of why an organisation (or nation) exists and how it thrives – the right things as determined by the customers (or voters). Tactics is doing things right.

Our focus should be on determining the right things to do, using technology and methodology to support exceptionally high-value outcomes and then doing them right. We must prevent failure at all costs – failure is always more costly than doing it right first time, provided that there is a valid and valuable value proposition to start with. Spending millions on number plates with microchips consumes value.

If we do the right things well, the organisation or nation will thrive, if we do them not so well we will survive, but if we do the wrong things, the organisation will die and it is only a matter of how quickly. If we do the wrong things well, our organisation or nation will die fast, or else it will die slowly.

South Africa is approaching a tipping point where our failure to maintain core infrastructure and invest in core technologies and methodologies is almost certainly going to slip into the die-fast quadrant – we are getting very good at spending money in ways that do not create sustainable value, and even better at driving out experienced people and making those who remain feel unwelcome.

The implementation of strategy has a time dimension. A strategic plan is not a forecast or a goal, it is a trajectory of change, the path to success or failure. It always follows an exponential curve; this is directly comparable to the trajectory of change of direction of a motor vehicle, aircraft, ship, etc. It starts out tangential to the current direction and slowly changes direction if the hands on the steering wheel are constant. Inconsistent steering leading to constant changes in direction will prevent change from occurring, and change that takes place too rapidly will become unstable. The car will roll, the ship capsizes or the aeroplane stalls.

South Africa is in a situation where it is seeking to change too much too fast and instability is now rapidly approaching. Instability in policy in various areas is also crippling the capacity to plan effectively and to execute plans that deliver lasting sustainable value and wealth creation.

In my journey of discovery into the factors that cause failure in information technology investment, concentrating particularly on the implementation of large business information systems (enterprise resource planning – ERP – systems), I have come to understand the critical factors.

More recently I have come to understand that the same factors cause failure of all technology investments, including electricity supply failure, failure of road maintenance, etc. and that these factors are in fact an indication of failure in strategic capability – the ability to visualise a future state and achieve it.

These factors are (percentages indicate relative contribution to failed projects):

a. **Technology mythology** (30%). We do not need to resurface the roads, they will just keep working; we can set any policy we like and it will work, etc. There is lack of understanding, inadequate knowledge and experience, etc.

b. **Lack of executive custody and inappropriate policies** (20%). “This is MY project and I accept full responsibility for the outcome” = “I have abolished the death penalty and you can hold me accountable if your loved one is murdered.”

c. **Lack of strategic alignment** (15%). This means lack of a clear definition of the essence of why the organisation exists and how it thrives.

d. **Lack of an engineering approach** (12%). This means lack of a systematic, thorough approach designed to prevent failure and thereby achieve success.

e. **Poor information management** (10%). “We do not really know how to measure the results of what we are doing and even if we have the measurements, we do not know what to do with them.”

f. **People/soft issues** (8%). Here we are dealing with human adaptability versus wisdom and competence, which equates to the synthesis between relevant knowledge and relevant experience. Each person is a complex composite of knowledge and experience. There is an exponential trajectory of value-creating knowledge and experience. It takes about 40 to 50 years from birth to form an engineer or other high-level professional who can conceptualise and execute projects that create high-value sustainable wealth and even then very few are able to do this. It will take more than 80 years of appropriate activity to achieve demographic parity with regard to high-level engineers, medical specialists, educators, etc. in this country if we plan and execute to prevent failure. This is not happening; we are driving out the very people who are the only ones who know how to do this.

There are three worldviews according to Marco Blankenburgh:

* **Guilt and innocence** – the North American and European culture

* **Honour and shame** – Asia, the Middle East, some of South America

* **Power and fear** – some of Africa, Asia, some of South America

Understanding the differences and tensions between these worldviews is vital to understanding South African politics and thereby to understanding how to prevent failure within the South African context.

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take another 20 to 50 years for outcomes-based education to deliver on expectations, simply because of the magnitude of the human-change impact. However, the worst thing South Africa could do now would be to scrap outcomes-based education and start another social experiment. We are committed to OBE, now we must MAKE IT WORK!

In order to achieve success we must first prevent failure, then we must manage towards success. The critical factors for success are:

a. Executive custody and policy (25%). Leaders at all levels MUST be held accountable. If the leaders of major US banks had faced 20 years on a chain gang for their actions, they would have conducted themselves differently.

b. Strategic architecture (18%). There must be a clear description of the desired future state of the organisation that is practical and achievable and is designed not to fail.

c. Strategic alignment (16%). A clear and detailed specification of the journey is needed.

d. Business integration and optimisation (14%). This is the aspect of how human beings integrate with the technology and methodology to create sustainable value, i.e. the management of the change process.

e. Classic project management (12%)

f. Information management (10%)

g. Technology/methodology (5%). The technology or methodology is almost irrelevant until all the other issues have been dealt with. Yes, there must be reliable technology or methodology, but that in and of itself does not create value. People using technology create value.

When one reaches a point where technology is starting to become a visible issue, such as potholes in roads, power supply failures and other technical manifestations of problems, then it is time to realise that the first six factors are NOT being adequately addressed.

The ‘load shedding’ of 2007–2008 was first and foremost a failure of executive custody and no national executive even offered to resign as a consequence of inappropriate policy decisions. The blackouts were also a failure in terms of strategic architecture, failure to have a clear view of the future state of the country and also a failure of strategic alignment: there was no clear plan of the journey towards the future state.

On the basis of this analysis, the fact that technology and methodology failure has reached epidemic proportions in every sector of the South African economy indicates that dramatic collapse is imminent.

Drastic measures are needed to alert government and business to these harsh realities and to mobilise initiatives which need to be every bit as bold as those that tore down the walls of apartheid in the years preceding the elections in 1994.

CONCLUSION

The application of these principles within a context that places appropriately high value on knowledge and experience, irrespective of skin colour, is vital if South Africa is to avoid slipping catastrophically into the abyss of technology and methodology failure that looms large before us.

NOTE

Dr James Robertson is an independent management consultant specialising in corporate strategy development and implementation, and strategic application of information technology.
This set of six articles covers the design, hydraulic, geotechnical and structural challenges associated with the construction of seven viaducts in the southern section of the massive and visionary Gautrain project. The authors wish to acknowledge the kind permission given by the Gauteng Provincial Government and the Bombela Concession Company to publish these articles. The authors also wish to advise that the views and opinions expressed are solely those of the authors and not those of the Province or Bombela.

RAIL TRANSPORT IN South Africa will enter a new era with the completion of the Gautrain Rapid Rail Link. The project will provide safe, efficient and reliable domestic and airport train services to commuters in Gauteng and will fulfil an important role in social regeneration and economic development. Following an international tender process, the Gauteng Provincial Government awarded the project to the Bombela Consortium consisting of Bombardier, Bouygues Travaux Publics, Murray & Roberts and Strategic Partners Group. The project is a public-private-partnership (PPP) and, as well as building the new railway, includes a 15-year operating and maintenance period. The operations will be led by RATP – operators of the Paris Metro. Gautrain Management Agency is the responsible Client / Authority.

The rapid rail link joins O.R. Tambo International Airport with Sandton and Rosebank in the southern section, with the northern leg continuing from Marlboro north to Hatfield in Pretoria. As a PPP project, the Gautrain’s design and construction contract involved very short lead times between the production of the drawings and concrete being laid in the ground.

To achieve this, designers and expertise from within South Africa and from all over the world have converged on Gauteng to create what is, among other things, a technological legacy. This transfer of savoir faire is in fact one of the key objectives of the project: fostering the ability not just to build, but also to coordinate and adopt technology on a scale never before seen in South Africa. Thus, the result is not just a rail system, but the freedom to continue...
making the same progress in the future without foreign impetus.

Currently among the top five largest construction projects in the world, the Gautrain incorporates a variety of engineering content:
- 80 km of modern-gauge railway
- 10 million m³ of earthworks
- 15 km of tunnel
- 10 stations, of which three are elevated and three are underground
- 10 km of viaduct
- 50 road or rail bridges

Therefore the Gautrain project is packed with interesting and exciting facets from stations, tunnels and massive earthworks to a diverse assortment of bridge structures including precast segmental viaducts. It is this latter element of the project that this article concentrates on, in particular the precast segmental viaducts of the southern section of the project.

PARTIES INVOLVED IN THE SOUTHERN VIADUCT SECTION

The Vela VKE design team were appointed as subconsultants for the design of the southern section viaducts; this included 4,3 km of viaduct and two stations, with the platforms being integral with the viaduct.

The Vela VKE team consisted of various consultants:
- Vela VKE Consulting Engineers: Tenderer and team leaders – Cape Town and Durban offices, as well as structural, civil (limited involvement) – Cape Town and Johannesburg offices
- Electrical (advisory) – Pretoria office
- Pöyry: Structural design – Taiwan
- Nyeléti Consulting: Structural design (BEE) – Pretoria
- ARQ: Geotechnical – Pretoria
- Exigent environmental: (advisory) – Pretoria

As the team was split between different cities, and even countries, this presented some logistical challenges for information transfer and final product delivery.

BOMBELA CIVILS JOINT VENTURE

Coordination of the subconsultant team’s design and document submission was handled by the Bombela Joint Venture team in the following hierarchical structure:
- The Project Manager and the
  Construction Manager
- The Chief Design Manager
- The individual Design Package
  Managers
- Document Control Centre
- Subconsultants

CONSTRUCTION TEAM

Bombela Civils Joint Venture

The construction of the project was split between the ‘casting yard’ where approximately 3 500 box girder segments, 5 300 parapet panels and 22 500 linear metres of precast, pre-tensioned M-beams were produced, and ‘production’ where individual teams worked on the construction at the individual viaduct sites. The civil works, including excavation, piling, foundations and substructure construction, were handled by Bombela, and the viaducts were erected, stressed and finished by VET (a part of VSL Internationals).

THE ELEVATED VIADUCTS

Construction choice for the viaducts

Various alternative forms of construction were considered for the viaducts during the preliminary design phase, namely the incremental launching method, balanced cantilever construction, cable-stayed bridge, precast segmental and cast in situ.

The final decision taken by Bombela was to construct all the main viaducts (with the exception of the John Vorster Bridge which has large spans resulting from a very skew crossing) using the same construction method. The precast segmental construction method was chosen. Although this form of construction was an untried method for

South Africa, it is commonly used in many other parts of the world. It was chosen primarily to achieve a consistently high standard of viaduct finish and for the ability to complete construction within the time constraints imposed on the project.

The decision was further refined by choosing to build the box girder segments using the long-bed construction principle to produce straight viaduct spans of variable length (span lengths vary between 22 and 54 m).

The variation in the horizontal alignment was to be accommodated by modifying the cantilever lengths and ‘kinking’ the deck alignment at the piers. The vertical alignment was accommodated similarly by kinking the alignment at the piers and having a variable-depth ballast to smooth out the curve.

STRUCTURAL CONFIGURATION

The viaducts comprise multiple-span, simply supported, precast, prestressed segmental concrete box girder decks, with hollow-box section piers of reinforced concrete and spread footings or piled foundations (see Figure 2).

The typical span of centre line of pier to centre line of pier is 44,0 m, but the actual span varies from viaduct to viaduct and is constrained by both the agreed alignment for the project and the acceptable radius of curvature for the construction equipment. Piers vary in height between 6,5 and 23,8 m and the spread footings/pilecaps are in general 6,5 x 6,5 m x 1,5 m deep.

The station platforms both at O.R. Tambo International Airport and at Rhodesfield are supported directly off the prestressed concrete box girder viaducts (see Figures 26-33 in section 6 – Superstructures).

1. The Gautrain route
2. Longitudinal section through typical viaduct box girder
3. Part longitudinal section along deck
Viaduct design considerations

THE DESIGN AND CONSTRUCTION of the Gautrain viaducts are being dealt with in the accompanying articles in this edition. However, the viaducts have a number of features that make them unusual. These include:

- Rail traffic travelling at up to 170 km/h with a high level of passenger comfort required
- Simply supported long spans with associated dynamic response and rail-structure interaction with continuous rails
- Precast segmental construction with only internal grouted prestressing and external prestressing crossing the construction joints
- Viaducts designed according to Eurocode

This article deals with specific design considerations faced by the Vela VKE team brought about by the more unusual aspects of the project.

EUROCODE

During the process of refining the design, it was identified that Eurocode was more suited to the design of a rapid rail long-span precast segmental viaduct than the conventional South African design code TMH7 which refers extensively to the British BS5400 bridge design code. During the application of Eurocode, the differences between various codes were investigated. The following are some of the issues and differences that need to be considered when adopting Eurocode.

The advantages of Eurocode are that it deals with a broader variety of train loadings, and deals in some detail with the dynamic behaviour of rail bridges and rail-structure interaction where the tracks, being continuous over the expansion joints, affect the behaviour of the deck and the stress in the tracks.

The disadvantages of Eurocode are firstly that it depends on National Annexes to specify certain design factors and local conditions. The structure of the code is also extremely cumbersome for bridge designers in that one part of the code deals with building structures, another part modifies this to deal with bridge structures, the appendices then expand on and alter the design requirements for more vulnerable bridges and, finally, the National Annexes modify all of these documents.

The history of Eurocode (being a compromise between countries with vastly different approaches to design) does mean that it is quite complex, offering several different ways of dealing with a concept. National Annexes and application guides and commentaries need to be referred to when dealing with many aspects of Eurocode.

The limit states of design are not as simplistic as in TMH7. States are explained more explicitly, such as loss of equilibrium, internal failure, etc. The approach of ‘robust design’ is used to limit catastrophic failure due to unforeseen events.

Load combinations in Eurocode are more realistic than in TMH7. Successive reduction factors are applied as the probability of a combination decreases. This avoids some of the unrealistic longitudinal load combinations of wind, temperature, seismic effects, braking, etc. on tall structures given in TMH7. Earthquake loading is treated as an ultimate load and not as a

\[ Q_{vc} = 40kN/m \]

\[ Q_{vc} = 125kN \]

\[ 125kN \]

\[ 125kN \]

\[ 125kN \]

\[ Q_{vc} = 40kN/m \]

Key

(1) No Limitation
service load as done by TMH7. This makes sense given the low probability of significant seismic events in South Africa.

Despite the load combinations being more realistic, they are more complex than in codes such as TMH7, BS5400 and AASHTO. This is due to the attempt to treat load combinations as a single probabilistic equation, as opposed to other codes that consider specific combinations. For South African application, we should consider adopting a simplification of the Eurocode load combinations.

One of TMH7’s durability and appearance requirements is to control concrete crack widths under full service loads. Eurocode only controls this under permanent loads, which is in line with the latest thinking on durability. This approach has a significant impact on the reinforced concrete beams and on the top slabs of box-girder-type decks where, according to TMH7, it is necessary to double the amount of reinforcement when the exposure condition is within 5 km of the sea in order to reduce allowable crack widths from 0.2 to 0.1 mm.

The equations used in TMH7 to predict crack widths are more conservative than those in Eurocode, with TMH7 crack widths being wider by 40% or more. The implication is that TMH7 will require 40% or more reinforcement to satisfy crack widths.

Shear design in Eurocode is more flexible and less complicated than in TMH7. A strut-and-tie truss analogy is used where the angle of the compression strut can be varied between 45 and 22°, the former allowing a thinner web and the latter less shear reinforcement. For prestressed sections, a simple enhancement is allowed. This is much simpler than TMH7’s complex handling of Class 1, 2 and 3 prestressed sections. A crucial weakness is the definition of z – the inner lever arm. For a section with axial compression, z will decrease, requiring more shear reinforcement – this is not logical. Furthermore, z is meaningless in a Class 1 prestressed section where there is no tension in the section. The National Annexes need to deal with this explicitly, setting z equal to 0.9 d or even 0.85 d as some research has shown that the former is not always conservative. For the Gautrain, we have used z associated with the ultimate bending capacity, as opposed to the applied ultimate bending moment, to deal with these anomalies.

The dynamic behaviour of bridges, especially footbridges and railway bridges, is comprehensively and clearly dealt with. The answers we obtained were realistic and agreed reasonably well with actual behaviour on some footbridges constructed recently.

The British bridge design code BS5400, on which TMH7 is based and which TMH7 refers to for composite design, structural steel design, bearing design, etc., is being withdrawn in 2010. TMH7 and other South African codes referring to BS5400 will therefore have to be modified. Moving to Eurocode will have distinct advantages in that it is supported by a vast amount of ongoing research – it is hoped there will be good literature on the application of the code. Adopting the British National Annex also makes good sense given our history of using British Standards. However, for bridges it would be worthwhile to write a South African version of Eurocode for classic concrete bridge types and then refer to Eurocode for the more complex and less common structures. This would avoid the constant paging backwards and forwards between three to six different documents just to apply something as simple as a reinforced concrete beam bending calculation. Adopting Eurocode would mean that South Africans could more easily design bridges in the rest of the world and that the rest of the world could more easily design bridges in South Africa. With our tendering system, we may end up with all our bridges designed in India and China!

In modifying or replacing TMH7 we also need to consider documents and codes that refer to TMH7. Some documents that come to mind are client documents such as the railways’ SATS design code, the SADC codes, SANRAL documents and bridge design documentation from various provinces and municipalities.

**DESIGN CRITERIA**

Early in the design process, design criteria were established and agreed to by all parties, namely the client, various parties in Bombela Joint Venture and the viaduct designers. Some of the items covered in the design criteria are discussed below.

- The rolling stock is supplied by Electrostar with a design effective speed of 170 km/h.
- The main design standards are from Eurocode, with COLTO as the standard specifications. British Standards, French railway standards and project specifications are also referred to.
- For concrete to decks the cube strength is 50 MPa, with concrete cover of 30 mm on external surfaces and 25 mm on internal surfaces.
- The structural steel includes the new grade S355 NL to EN 10025-3.
- Prestressing has been 15.7 strand (150 mm²) with a characteristic strength of 1860 MPa, in line with EN practice. Traditionally, a lower strength has been used in South Africa. The allowable stress
after wedge pull-in is 75% of ultimate capacity as opposed to TMH7’s 70%. These two changes resulted in less prestress.

- Deck loading was typically 155 kN/m for dead load at mid-span and 124 to 164 kN/m for superimposed dead load consisting of ballast mats, ballast, catenary, rails and sleepers, etc. Traffic loading was 0.5*LM71 train loading consisting of a 500 kN locomotive 6.4 m long and 40 kN/m carriages with a dynamic factor applied. Typically, 38% of the total service load was dead load, 35% superimposed dead load and 26% train load with 1% due to other effects.

- Other loads include traction and braking, centrifugal forces, nosing, wind, temperature, differential settlement, etc.

- Derailment and collision loads were decreased by installing containment rails across the viaducts. The position of a derailed train is shown in Figure 2.

- Creep and shrinkage were calculated according to Eurocode. The calculations consist of a series of equations which are much easier to apply in a spreadsheet or FEM (finite element method) analysis than TMH7’s tables and graphs. The effect of creep, namely the possibility of the deck ‘hogging’ upwards and the ride quality of the train being changed, had to be considered in the design. The ballast may have to be relevelled during the life of the decks.

- Specific load combinations are given, simplifying the complex approach adopted by Eurocode.

- Movements and deflections, especially accelerations, are limited according to Eurocode. What is significant is that the acceptable vertical acceleration of someone sitting, as in a train, is higher than that of someone standing, such as on a station platform. The dynamics of a deck with a station platform attached are therefore more critical than those of the same deck without a station platform.

- No tension was allowed across the glued deck-construction joints under traffic plus temperature load. This proved to be the load case that determined the amount of prestress. The locked-in effects of reverse temperature gradient were especially onerous for the design of the deck prestressing.

### DYNAMIC ANALYSIS OF ROLLING STOCK

During the analysis of the various spans, the deflections under train loading and the natural frequencies were calculated. Some of these are reproduced in Table 1.

Based on the behaviour of the deck, Eurocode then had requirements as to whether a full dynamic analysis needed to be done. As can be seen in Table 1, spans of up to 44 m did not require a further dynamic analysis, whereas longer spans did require one.

Where full dynamic analysis of rolling stock was required, the FEM program ALGOR was used. This had been successfully used on the Taiwan High-Speed Rail. The process was as follows:

- Model the deck as a series of beam elements of equal length.
- Calculate the first four to six natural vertical frequencies.

<table>
<thead>
<tr>
<th>Deck length</th>
<th>Max. bearing rotation (radians)</th>
<th>Max. midspan deflection (mm)</th>
<th>Vertical natural frequency (Hz)</th>
<th>Torsional natural frequency (Hz)</th>
<th>Dynamic analysis required?</th>
</tr>
</thead>
<tbody>
<tr>
<td>54 m</td>
<td>0.0009</td>
<td>11</td>
<td>2.1</td>
<td>8.5</td>
<td>Yes</td>
</tr>
<tr>
<td>44 m</td>
<td>0.0005</td>
<td>5</td>
<td>3.2</td>
<td>10.7</td>
<td>No</td>
</tr>
<tr>
<td>34 m</td>
<td>0.0002</td>
<td>1.6</td>
<td>5.6</td>
<td>14.3</td>
<td>No</td>
</tr>
</tbody>
</table>

![Containment rail restricting derailed train](image_url)
For a given speed and train configuration, calculate the dynamic response as the train moves across the deck. This is done by dividing the time taken by the train to cross the deck into a number of time steps. A time step is typically a fraction of a second and is smaller than 1/20th of the time taken for an axle to travel the length of one element, or 1/20th of the highest natural frequency one is interested in. For each successive time step, the train is moved into its new position and the dynamic response is calculated based on the dynamic behaviour at the end of the previous time step and the modification to that caused by the current time step, using modal superposition. Special pre- and post-processors were written in Turbo Pascal to assist in this process. The maximum deflections, velocities and accelerations are then extracted.

The process is repeated for various train configurations and speeds up to the design speed and from this the worst vertical acceleration can be determined. Figure 3 gives the vertical acceleration in m/s² against different train speeds in m/s for a 54 m deck. As can be seen, at 170 km/h (47 m/s) the acceleration is 1.5 m/s². This is within code requirements. It is also worth noting that the typical minimum vibration of 1.29 m/s² is caused by the first mode of vibration (first natural frequency). As speed increases, other modes of vibration add to the acceleration with the first major resonance occurring at 58 km/h (16 m/s). Maximum vibration of 1.68 m/s² occurs at 187 km/h (52 m/s). This is 31% higher than that predicted by static loading. After this, the train is travelling too fast for the deck to respond dynamically and the vibration decreases.

**CREEP IN THE DECKS**

In terms of passenger comfort, the additional vertical deflection in the deck due to creep once the tracks have been installed is significant. Excessive creep could cause additional vertical acceleration over and above that caused by normal dynamic response. This would lead to the ballast having to be relevelled earlier than necessary.

When the deck is initially prestressed, the bottom goes into significant compression and the top into low compression, causing the deck to bow upwards. With creep, the bottom will shorten more than the top, causing the deck to bow up even more. Once the ballast and tracks have been installed, the compression in the bottom of the deck decreases and the compression in the top increases, causing the deck to deflect downwards as would be expected.

What now becomes interesting is the behaviour of the creep in the bottom of the deck: we have a decrease in load leading to creep relief. Eurocode treats this creep relief no differently from normal creep, whereas other literature gives creep relief as being much lower than normal creep – typically down to 60% of normal creep. The implication of this is that Eurocode predicts some ongoing upward deflection, whereas creep models with creep relief give significantly higher deflections. This led to interesting discussions between the designers and the checkers, with spreadsheets and calculations travelling backwards and forwards. (No voices were raised excessively during this process.) The more conservative approach was then used to predict final deflections. It will be interesting to see what happens with the deck deflections over time – will Eurocode adequately predict the long-term behaviour?

**RAIL–STRUCTURE INTERACTION (RSI)**

With simply supported decks on tall piers, each expansion joint at each pier will open and close under acceleration and
braking and under temperature loading. This will in turn induce stresses into the tracks, which are continuous over the expansion joints. This has two significant consequences:

- Stresses in the rails must be within acceptable limits to avoid failure due to fatigue.
- Loading on piers will decrease as the loads are spread via the rails to more piers.

Central to RSI is the behaviour of the track fastenings. Idealised, they behave elastically up to a certain value and then they slide at a constant force. Figure 4 (p 14) shows their behaviour.

To model this interaction, the FEM program RM was used as it had non-linear spring elements to model the track fastenings. A typical extract from the FEM model is given in Figure 5, showing the non-linear springs between the rail elements and the deck elements.

Traction and braking, as well as thermal actions, were modelled and envelopes for the maximum rail axial load, bending moment and deflections were extracted. The results of the maximum axial load for Viaduct 1 are shown in Figure 6.

As would be expected, the maximum axial load is at the expansion joints, decreasing towards midspan.

The results of the RSI for Viaduct 1 are given in Tables 2 and 3.

**CONCLUSION**

The design of the Gautrain viaducts presented an interesting challenge to the VelaVKE design team, necessitating extensive discussions, references to design codes, Internet searches, scouring of literature and comparative calculations. It had moments of extreme frustration with the obtuseness of some parts of Eurocode, compounded by the relatively tight deadlines for some parts of the work. Now that the design is complete and checked, and most of the construction of the viaducts on the southern section is almost complete, there is a sense of relief among the designers. However, as with almost every bridge we design, next time there are some things we will do differently.

---

**Table 2 Additional rail stresses due to RSI for Viaduct 1**

<table>
<thead>
<tr>
<th>Additional rail stresses [MPa]</th>
<th>Max $\sigma$ (tension)</th>
<th>Allowable limit</th>
<th>Exceeded?</th>
<th>Min $\sigma$ (compression)</th>
<th>Allowable limit</th>
<th>Exceeded?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traction and braking</td>
<td>50</td>
<td>92</td>
<td>OK</td>
<td>-37</td>
<td>-72</td>
<td>OK</td>
</tr>
<tr>
<td>Temperature</td>
<td>39</td>
<td>92</td>
<td>OK</td>
<td>-26</td>
<td>-72</td>
<td>OK</td>
</tr>
<tr>
<td>Combined temperature, traction and braking</td>
<td>68</td>
<td>92</td>
<td>OK</td>
<td>-64</td>
<td>-72</td>
<td>OK</td>
</tr>
</tbody>
</table>
A full hydrological and hydraulics analysis at each of the viaduct river crossings was undertaken and evaluated to identify the potential impacts of the viaduct and to design remedial measures that would minimise the predicted hydraulic and environmental impacts. Both the short-term impacts associated with the construction phase and the resulting long-term conditions were assessed. The analyses focused on changes in river flood levels and erosion potential along the river channel and on scour at the viaduct piers and abutments.

The design process included:
- Review of catchment hydrology
- River hydraulics modelling, including hydraulic roughness determinations and flood line analyses
- Evaluation of the impacts of the proposed viaduct
- Assessment of possible remedial measures
- Design of remedial measures, specifications and quantities

HYDROLOGY
The design flood was specified as the 1:100 year return period flood or, for catchments larger than 15 km², the 1:200 year return period flood.

Design flows were calculated using a number of hydrological calculation methods, and a flood value envelope determined to quantify sensitivities. The methods included:
- Rational Method and Alternative Rational Method
- Standard Design Flood Method
- Statistical methods - these were used where flow-gauging weirs that had a sufficiently long historical record existed along the river course (Jukskei River only)
- Regional Maximum Flood Method - this method was used to determine the 1:100 and 1:200 return period floods

RIVER HYDRAULICS MODELLING
In order to evaluate the effect of the proposed viaduct on the watercourse, it was necessary to create a hydraulic model of the river to gain as accurate an understanding of the present and future conditions as possible. River modelling commenced with the generation of baseline models of the existing conditions. The models were calibrated according to known data obtained from previous flood measurements, as well as from on-site and aerial photograph interpretation. Future conditions for the viaduct and proposed remedial measures were then included in the models and design modifications and impacts on the environment assessed. Baseline, viaduct and remedial measures models were created. RiverCAD™ was used to generate the hydraulic models, which in turn were built up from the 3D contour and digital terrain model (DTM) data provided.

The primary objectives of the hydraulic analyses were:
- Confirming that the works would not materially change the present river flood levels
- Where flood levels were increased, designing countermeasures to ensure that the river flood levels prior to construction of the works were not materially exceeded
- Evaluating the impacts on wetlands with regard to changes in inundation depths, erosion and sedimentation potential
- Determining an envelope for each of the hydraulic design parameters (lower and upper boundary values) by running the hydraulic models for the 1:5, 1:10, 1:20, 1:50, 1:100 and 1:200 return period flood envelopes
- Determining equilibrium slope and channel widths, as well as bed shear stress limits, using geotechnical information in conjunction with the above hydraulic data

DESIGN APPROACH
The designs were carried out in accordance with the procedures set out in the South African Road Drainage Manual and other procedures and guidelines developed by the USBR, USACE and USFHA. Important parameters used in the designs were bed shear, Froude number, flow velocities, riverbed and riverbank characteristics, constituent particle size distributions and the Plasticity Index of the bed material.

HYDRAULIC SOLUTIONS CONSIDERED
Three primary zones of potential impacts were identified: the zone below the viaduct...
(piers and abutments), the upstream and downstream riverine zone (river channel and riverbanks) and the border zone consisting generally of developed urban areas.

**Pier and abutment scour**
Measures considered included:
- River training walls immediately upstream and downstream of the piers to reduce turbulence around the piers and to improve river flow lines
- Riverbed protection around the piers to limit depth of scour
- Energy-dissipation structures to reduce free energy levels at the piers
- Riverbank widening and protection where piers could act as obstructions to normal flow
- Abutment erosion protection, which typically consisted of rock rip-rap protection placed along the toe of the fill up to the design flood level

The computations of scour around piers and through bridge structures were done in accordance with the recommendations of the South African Road Drainage Manual.

**Riverine zone erosion protection measures**
These included:
- **Rock rip-rap linings.** Rock used as rip-rap consisted of durable rock with some grading to improve particle interlock. Typical rock sizes utilised were $D_{50} = 150$ mm for flow velocities of less than 2 m/s and $D_{50} = 500$ mm for flow velocities of less than 5 m/s. The minimum rip-rap lining thickness was specified as $3 \times D_{50}$. Adequate cut-off toe and flank designs were key features of the design. All in situ material interfaces were lined with filters.
- **Groynes.** These were used to limit riverbank erosion or to redirect flows away from sensitive areas where flow capacity was not a constraint. Groynes were considered a better solution than formal riverbank linings as the natural processes of erosion and sedimentation would still take place but with limited damage and subsequent natural rehabilitation.
- **Vegetative stabilisation measures.** Where bed shear stresses were sufficiently low, grassing was used to stabilise problematic areas and to provide erosion protection during floods. Non-invasive cultivars with strong root systems were specified.

**Dedicated energy-dissipation measures.** Apart from rock rip-rap protection and groynes, limited use was made of dedicated dissipation structures such as weirs and stilling basins.

**Monitoring and rehabilitation.** Apart from normal landscaping and grassing rehabilitation measures, the hydraulic analyses in some cases indicated that no additional measures were required. In order to ensure long-term environmental compliance, these cases, as well as the cases where remedial measures have been implemented, will be subjected to a monitoring and rehabilitation programme which will form part of the project operations and maintenance management plan.

**Border zone**
In some cases the hydraulic sizing of nearby existing structures was done to flood design criteria (defined by the relevant owner or authority) that were lower than the requirements for the project which were, depending on the size of the catchment, the 1:100 or 1:200 year return period flood. Scenarios covering the present structure and a future new or upgraded structure were included in the modelling and resulting design parameter envelopes.

**OTHER ISSUES**
Existing and future external influences were addressed through the project operations and maintenance management plans. These included:

- **Site observations at Viaducts 01 and 11**
- **Piers under construction at Viaduct 03**
Loss of conveyance or changes in flow patterns due to lack of maintenance (removal of accumulated sediment, debris, etc) in the river reaches outside of the project area

Failure of existing river structures upstream or downstream of the viaduct which could impact on the performance of the works in the project area

Existing erosion damage along the river-banks and structures was dealt with by documenting such damage for future reference and by undertaking proactive repairs where this was necessary inside the hydraulic zone of influence of the viaduct.

ENVIRONMENTAL CONSIDERATIONS

The design and subsequent construction activities were required to comply with the regulations and additional requirements listed in the relevant licences and permits issued by the following authorities:

- EIA Regulations of the National Environmental Management Act (Act 107 of 1998)
- GDACE permits and approvals
- DWAF (now DWEA) permits and licences
- Requirements of the South African Heritage Resources Agency
- Wayleaves from Gautrans and the Johannesburg Roads Agency
- Permissions from the various other City of Johannesburg agencies
- Requirements of the EIA and EMP

Of particular interest were the requirements for wetlands in the zones of impact:

- All wetlands within the project area were to be delineated and wetland boundaries clearly shown on layout plans.
- A wetland professional was appointed to confirm the extent of wetlands in accordance with the DWEA wetland-delineation guideline document.
- No construction activities were allowed to take place within any wetland boundary.

MATERIALS

A set of material specifications was developed which made provision for the maximum use of locally available materials. Gabion baskets and mattresses were required to be galvanised and PVC-coated to improve design life. It was anticipated that it would be possible to source suitably sized rock (granite) from the tunnel excavations or from nearby cuttings excavated in rock.

Natural filters were used in conjunction with geofabrics where possible. Geofabrics were used on all interfaces between soil and erosion protection structures to prevent the washing out of fines behind the structures. Tensile strength, punch-through resistance and fabric-clogging characteristics were important parameters considered in the selection of the geofabrics.

Cement mortar grouting of plain rock structures was specified where free energy levels were excessive and where reinforced concrete structures would be unsightly, uneconomical or impractical.

CONSTRUCTION

The Bombela Joint Venture’s temporary works layouts were reviewed to ensure that such works did not interfere with planned river protection works and create additional erosion and environmental problems. Forming part of the site rehabilitation works, river erosion and pier scour protection works were undertaken after construction of the viaduct. Where reinforced concrete training walls were required, these were integrated into the pier construction programmes.

Design validation during construction comprised a review of method statements to ensure that the proposed construction procedure did not violate any of the design assumptions, field checks during construction and end-product inspections to ensure that the works complied with the specifications.
**INTRODUCTION**

The seven southern section viaducts, with a total length of some 4.3 km, traverse a variety of geological units. Preferential weathering in the upper reaches of the profile necessitated that two main founding solutions be adopted: piles and spread footings. The spread footing founding depth varied from 3 to 7 m while that for the piles approached 30 m in places. Table 1 details the foundation attributes.

**DETAILS**

The pile cap and spread footing foundation size was standardised at 6.5 x 6.5 x 1.5 m, while pile diameters were fixed at 1.5 m, except for Pier 41 on Viaduct 15/3 where piles 900 mm in diameter were used to penetrate the parking lot slabs at O.R. Tambo International Airport. Four piles per pier were used in all cases, with maximum serviceability loads of some 11 MN ensuring pile shaft stress of just more than 6 MPa. For the spread footings, the maximum bearing stress was limited to around 1 200 kPa.

Capacity verification of both piles and footings was carried out according to the requirements of Eurocode 7: Geotechnical Design and checked using conventional calculation methods for bearing and pile capacity.

**QUALITY ASSURANCE**

The correct founding stratum for spread footings was ensured by inspecting and signing-off each and every base. Serviceability stresses under operating conditions were limited to around 1 200 kPa.

The integrity of all piles was checked using cross-hole sonic logging (CHSL) techniques. Five ducts were installed, equally spaced around the inner circumference of the reinforcing steel, and the CHSL conducted between successive pairs of holes to establish internal pile quality. The condition of the concrete–rock interface at the tip of those piles which carried significant load in end-bearing was, in addition, checked by drilling through the base of the CHSL.

**Table 1 Foundation attributes**

<table>
<thead>
<tr>
<th>Number</th>
<th>Length</th>
<th>Piers</th>
<th>Abutments</th>
<th>Geology</th>
<th>Allowable limit</th>
<th>Exceeded?</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>226</td>
<td>4</td>
<td>2</td>
<td>Granite</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>635</td>
<td>12</td>
<td>2</td>
<td>Granite</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>4</td>
<td>745</td>
<td>16</td>
<td>2</td>
<td>Granite</td>
<td>14</td>
<td>2</td>
</tr>
<tr>
<td>11</td>
<td>227</td>
<td>4</td>
<td>2</td>
<td>Granite</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>13</td>
<td>398</td>
<td>9</td>
<td>2</td>
<td>Greenstone</td>
<td>9</td>
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</tr>
<tr>
<td>14</td>
<td>630</td>
<td>14</td>
<td>2</td>
<td>Granite/Greenstone</td>
<td>10</td>
<td>4</td>
</tr>
<tr>
<td>15/1</td>
<td>820</td>
<td>21</td>
<td>1</td>
<td>Greywake/Agglomerate</td>
<td>0</td>
<td>21</td>
</tr>
<tr>
<td>15/2</td>
<td>450</td>
<td>16</td>
<td>0</td>
<td>Greywake/Lava</td>
<td>0</td>
<td>16</td>
</tr>
<tr>
<td>15/3</td>
<td>168</td>
<td>4</td>
<td>0</td>
<td>Greywake/Lava</td>
<td>0</td>
<td>4</td>
</tr>
<tr>
<td>Totals</td>
<td>4 299</td>
<td>100</td>
<td>13</td>
<td></td>
<td>42</td>
<td>234*</td>
</tr>
</tbody>
</table>

* This comprises 228 x 1 500 mm diameter piles and 6 x 900 mm diameter units
tube and through the base of the pile, and extending the core some 2 to 3 m beyond the base. Examination of the core enabled a decision to be made as to the required recourse. If soft or weathered zones were absent, the CHSL tubes were grouted. If there were soft zones present, these were removed by water pressure washing and the voids thus created were grouted under fairly low pressures.

**DESIGN VERIFICATION**

Predicted deflections at the serviceability limit state were made for each critical pier on each viaduct. These generally varied from 6 to 13 mm. Monitoring of the piers under construction loads has produced deflections from 1 to 6 mm. It is reasoned that the final measured values will be very close to those predicted.

**CHALLENGES**

On this project, not only was each and every pier and abutment position core-drilled to establish the in situ profile, but pressure-meter testing was also carried out in selected holes at various depths. This was supplemented by laboratory testing of soft and hard materials retrieved as cores, as well as testing of the water quality from these holes. In some cases the process was supplemented by test pits and auger holes.

This information was distilled into a comprehensive geotechnical report, with the recommendations for founding having been discussed at length with Bombela Joint Venture before finalisation of the desired system.

For the design, the foundation system at every pier position was subjected to checking against 12 critical load cases. As mentioned above, two design systems were used: the as-specified one complying with Eurocode 7 (E7), and a supplementary check against normally used criteria. This was necessary as E7 utilises ultimate limit state design, whereas the familiar systems normally employed in South Africa are based on serviceability criteria. Much paper detailing the design was generated in this process.

In another deviation from the norm usually applied in South Africa, field verification was carried out by a third party employed by Bombela and not by the designer. This often led to heated debate but necessitated that the full construction-verification procedure be either included on the drawing (which was preferred) or detailed in a project specification.

The design process conducted by the Vela VKE, Pöyry, Nyeleti, Exigent and ARQ teams was ably controlled by Josh Mattheis of Bombela, who acted as the link between the two. Our thanks go to him for remaining calm in sometimes very difficult situations.

Although the train has not yet traversed the route, many of the viaducts are now nearing completion for all to see and marvel at. The foundations (as is the norm) are unfortunately hidden from view. It must, however, be remembered that these elements (the piles and the spread footings) provide the practical link between what we consider the ‘esoteric’ (the structure) to the ‘soul’ (the earth). Long may they live as durable elements!
INTRODUCTION
The substructures of the viaducts were designed and detailed by Nyeleti Consulting. Close liaison between the geotechnical engineers (ARQ) and the superstructure designers (Vela VK and Pöyry) was required in the detailed design of the substructures. The design forces for the substructures were obtained from a global analysis which was carried out for each viaduct by the superstructure designers.

TYPICAL LAYOUT
Abutments
The abutments are of solid-walled reinforced concrete construction, perched in the fill at each end of the viaduct. Two different launching girders were used to construct the decks. These launching girders had two different heights which resulted in two types of abutment being required (Type I abutment is 7,125 m high and Type II abutment is 6,175 m high). The abutments (Figure 1) comprise spread footings, solid return walls and a solid breast wall which retains the fill behind and accommodates the viaduct expansion joint, as well as the approach slab. The spacing between the top of the abutment bearing plinth and the soffit of the deck is 500 mm in order to accommodate the equipment required for bearing replacement. The front face of the abutment is closed off with a brick wall and contains an access door for maintenance purposes.
Piers

The piers are of reinforced concrete construction, founded on either spread footings or piled foundations. The dimensions of the typical spread footing / pile cap are as shown in Figure 2.

Piers vary in height (between 6.5 and 23.8 m) with a basic 4 by 3 m hollow cross section (Figure 3). The wall thickness is constant within the height of any pier (with a minimum thickness of 300 mm) and there are no openings through the pier wall except for the drainage and earthing ducts.

The pier reinforcement and formwork were designed and detailed to accommodate three typical lifts in the construction process:

- A standard upper lift for all viaduct piers
- An intermediate lift that could be replicated to achieve the required height for the pier stems
- Unique lower lifts (at the base of individual pier stems) which, in addition to the intermediate and upper lifts, make up the required heights for individual pier stems

Pressure due to the concreting of the pier stems was taken up by the circular belts incorporated within the formwork design (Figures 4, 5 and 6). No tie rods or anchorages were used.

Although the walls of the pier stems were designed to a thickness of 300 mm, the inner shutters were designed in order to accommodate wall thicknesses varying from 300 to 500 mm.

Access platforms used for the pier construction formed an integral part of the formwork design and were incorporated into the pier formwork fabrication. In addition, bottom lift formwork was also used as upper lift formwork.

The tops of the piers expand into a solid reinforced concrete cap to accommodate the four bearings for the...
two simply supported decks, as well as a recess for maintenance access. The piers were also designed to accommodate the drainage pipes used to transport storm water from the deck to the ground.

The general arrangement of pier tops (including bearing positions) has been specifically planned to ensure:
- Adequate edge distance to eliminate spalling
- Sufficient space for the hanger brackets of the launching girder
- Sufficient space for the bearing replacement jacks
- Sufficient working space for the maintenance crew for general maintenance to the deck, piers and drainage system, as well as for bearing replacement

The arrangement is shown in Figures 8, 9 and 10.

The reinforcement arrangement in the pier tops has been planned and detailed with due consideration to addressing the following:
- Clashing reinforcement
- Adequate space to place concrete
- Adequate space to compact the concrete with immersion vibrators
- Clashing in the congested area around the bearings

Piers within 8 m of a travelled way, which are not protected by a concrete barrier, have been designed to withstand impact loads. Piers immediately adjacent to travelled ways, such as in a median or traffic island, have been filled with mass concrete to 2 m above ground level in order to satisfy the local effects of impact loading.

**FOUNDING CONDITIONS**

**Abutments**

A ‘one solution fits all’ approach was chosen for the abutments on this portion of the project with a choice having to be made between two completely different foundation concepts (Figures 11 and 12):

**Option 1:** An abutment supported directly on rock (either by way of piles or directly bearing on hard rock) will be rigid in its vertical behaviour and consequently presents a ‘hard edge’ to the viaduct approach fill. The approach fill will settle over time and the abutment (being rigid) will maintain its position.

The settlement \( \Delta \) is made up of both primary and secondary components from the substrata. The settlement in the substrata takes place during construction, but the settlement in the approach fills takes place partially during construction and most of the remainder during the 4 to 5 months after the fill has been placed.

**Option 2:** Perched abutments are constructed on a fill that has already been placed and compacted and where a large part of the settlement has taken place. The abutment will gradually settle together with the remainder of the fill settlement, consequently having no ‘hard edge step’ at the back of the abutment.
Instead of the abrupt step at the abutment in Option 1, the settlement is taken up over the whole span from the ‘rigid’ adjacent pier to the abutment. Although the ballast will ‘soften’ the effects of the settlement difference expected in Option 1, Option 2 will provide a significantly better riding quality for the train and will contribute to better construction conditions with reduced maintenance. Option 2 was therefore the preferred solution.

**Piers**

The foundation design was based on the findings of a geotechnical investigation and resulted in piers supported on either spread footings or piles (Figure 13).

**ANALYSIS AND DESIGN**

A global analysis of each viaduct was conducted using RM, a specialised bridge design package developed by TDV in Austria. The analysis was conducted for all
load cases and combinations as specified in the general design criteria for the viaducts. The substructure reactions were extracted from the results of the global analysis.

The global analysis did not make provision for geometric imperfections and pier slenderness. The reactions extracted from the global analysis were therefore adjusted as specified in the relevant Eurocodes to take these second-order effects into account.

Pile group interaction was investigated during the research stage and found to be negligible; it was subsequently ignored in the detailed design.

A simplified beam element model was set up for each pier to accurately represent the founding conditions on site accurately. Strand7 was used as the structural analysis program to determine the design forces in the piles (Figure 14).

The model consisted of piles modelled as three-dimensional beams and a pile cap modelled with a series of rigid links. The soil interaction of the piles was modelled with horizontal springs spaced at intervals of 1 m for the full length of each pile. The horizontal spring stiffness assigned to these springs represented the lateral support from the in situ material. The end bearing conditions of the piles were modelled as pinned horizontally and vertically, supported by a very stiff spring (spring stiffness ranged from 400 to 1200 kN/m).

Scour, if relevant, was modelled by removing the horizontal springs on the piles to the anticipated depth of scour.

The piles were designed to accommodate the calculated design forces. It is of interest to note that the short piles (length < 3 x diam.) resulted in high design forces. All short piles had to be dowelled into bedrock to accommodate the applied horizontal forces at the bedrock interface. These dowels were installed after the pile construction process through the cross-hole sonic tubes. However, the short pile alternative was in most cases preferred to a deep spread footing. Apart from a shorter construction period obtained by piling, deep excavations would have presented potential problems to other services, roads or structures that were close by.

**INTERESTING CHALLENGES ENCOUNTERED**

During the construction phase of the project, several challenges were experienced.
General challenges included:
- Flow and flood conditions of river crossings
- The installation of temporary or permanent earth-shoring work
- Accommodation of construction loads
- Coordination between various consultants (Vela VKE in Cape Town, Pöyry in Taiwan)

A number of other aspects of the substructure component contributed to the challenges experienced on the project (Figures 15, 16 & 17).

‘Non-standard’ pier foundations
All pier piled foundations consist of a four-pile configuration, with the exception of one pier (P41) which is located within the existing double storey parking garage of O.R. Tambo International Airport, where six piles were used. These piles (which are exposed for the first 8 m) are sleeved with a 10 mm thick permanent steel casing. The main purpose of the steel casing is to protect the piles during construction.

Pier 41 is designed to enable independent functioning between the parking garage and viaduct structure. Portions of the two slabs had to be demolished in localised areas to make allowance for the pile cap and piles.

Reinforced concrete edge beams were constructed around the slab openings to reinstate the structural integrity of the existing floor slabs.

Redesign of revised pile layouts
During the construction phase it was required that certain pile caps be redesigned due to the following reasons:

Clashing of piled foundations with existing services
In one instance, the pile cap configuration needed to be revised due to a clash with an existing Telkom manhole containing fibre optic and other cabling. Discussions between all relevant parties led to the conclusion that the option of relocating the manhole and the cables was not viable. The proposed spread footing foundation design was therefore adjusted to accommodate the existing

16 Longitudinal section through Pier 41 foundation
17 Transverse section through Pier 41 foundation
18 Revised pile cap configuration with repositioning
19 Revised pile cap configuration supplementing condemned piles 1
20 Revised pile cap configuration supplementing condemned piles 2
manhole containing the optic fibre and other cables by changing to a piled solution with a larger pile cap to straddle the existing manhole (Figure 18).

**Condemned piles**

Due to construction problems identified through cross-hole sonic testing, some piles were condemned. These piles were not capable of fully supporting the design loads and it was therefore necessary to install additional piles to accommodate the total design load. As a result, new configurations for the pile caps (incorporating additional pile caps and in some cases enlargement of the pile caps) had to be analysed and designed (Figures 19 and 20).

**CONCLUSION**

Whilst all the viaduct piers have the same cross section, the varying heights, varying founding conditions and various obstructions that had to be avoided led to many different solutions. The design team, working as a cohesive unit, successfully met all the challenges of this great project, in the process expanding their level of expertise.
VIADUCTS IN GENERAL

The successful design, manufacture and construction of the viaduct superstructure with its precast segmental format was the key to the timeous completion of these sometimes vast structures. Reaching the final solution required extensive planning and interaction between the design subconsultants (VelaVKE), the Bombela Joint Venture supervisory team, the casting yard where the segments were to be manufactured, and VET, the viaduct erection specialists.

With more than 3,000 deck girder segments being required on the whole project, attention to weight limitations, reinforcement quantities and sheer practicality in detailing for ease of manufacture determined whether the project would be a financial success or not.

Although the process of manufacturing and constructing these simply supported viaduct spans may in general appear straightforward, the attention to detail required to ensure that all contingencies had been addressed dictated the need for a detailed set of rules for the design, manufacture and erection of the spans. Some of the construction constraints and tolerances imposed on the viaduct construction are listed below.

Construction constraints imposed by the viaduct design
- Construction joints are to be perpendicular to the deck soffit. The deck ends are to be vertically perpendicular to the deck soffit and transversely square to the orientation of the pier.
- Segments are to be matched-cast in a precast yard using the long-bed method, starting with the end segments and closing with a centre segment.
- No de-bonding agents are to be used.
- No more than two segments may be stacked on top of each other.
- Segments (28 days old and older to limit creep deflection) are supported on top of the underslung gantries, which are supported on the piers.

1. Viaduct 03 erection in progress
2. Fabrication of the Type 1 launching girder
3. Typical connection between launching girder segments
4. Pier brackets installed on first pier
The segments are to be glued together and stressed with temporary prestressing, the wet glue acting as a lubricant, as well as a waterproofing sealant.

The tapered keys, 20 mm deep, will assist in lining up the segments as they are stressed together.

The thickness of glue after temporary prestressing will be approximately 0.3 mm. (The tensile strength of the glue is not taken into account for design purposes.)

Sufficient internal and external prestressing must be applied at the deck ends for the span to be self-supporting before the segment loader can be moved over the deck and before the deck can be lowered onto the temporary bearings.

Construction tolerance allowance

Pier construction inaccuracies were corrected by placing the bearing plinths in the correct position and to the correct level.

The recesses for the bearing dowels, in the bottom plinth and the deck soffit, allow ± 30 mm of movement in any direction, giving a further adjustment in the deck positions of ± 60 mm.

The cast length of each deck was sized to allow for shrinkage, prestress shortening and a glue thickness of 0.3 mm per joint such that on installation of the deck, the expansion joints are 50 mm at the narrowest position. Under creep and shrinkage, the joints will open such that at the time of the installation of the expansion joints, the joints will be wider.

CONSTRUCTION OF THE SEGMENTS AND THE ELEVATED VIADUCTS

General description of method

The viaduct deck consists of precast segmental spans that are simply supported on the substructure. Precast segments are manufactured in a casting yard and delivered to the erection site on trailers, either to a point below the span of erection or along the completed viaduct.

The process of manufacture, erection, stressing and finishing of the viaducts can be split into three main components:

- The launching girders – their fabrication and their installation by launching them onto the first span
- The precast segments – their manufacture, transport and erection
- The fabrication of the precast segments and their stressing together into an integral structural unit capable of
supporting itself, the additional dead loading and the high-speed train itself. This process is briefly described alongside by way of a series of photographs with appropriate descriptions.

The launching girder

The launching girder system chosen by the Bombela Joint Venture for the Gautrain Project is of the underslung type whereby the individual precast segments are supported by the twin girders underneath the side cantilevers of the box section.

Time constraints dictated that three sets of launching girders were used on the project. Figure 2 shows the Type 1 girder.

The launching girders, which were designed in Singapore by VSL TCAA Singapore and manufactured in China, were delivered to the Gautrain site in containers.

The girders, which were prefabricated in China into transportable sizes (to fit in the containers), were fabricated on the approach fill behind the viaduct abutments.

Specially designed launching girder pier brackets manufactured in China (Figure 4) were installed by stressing a pair of brackets together on either side of the viaduct pier heads.

The abutments were constructed in two stages. The first stage comprised the spread footing, allowing the launching girders supported by the ULRS (universal lowering and rolling system) seated on the abutment footing to be launched over them and the second stage completed the abutments so that the approach fill could be placed.

The overall length of the launching girders is slightly more than the length of two spans of the viaduct to allow for stability during launching.

**PRECAST SEGMENTS – THEIR MANUFACTURE, TRANSPORT AND ERECTION**

The key segments on each span are the two end segments, or 'segment on pier' (SOP) as they are known. These segments, which carry huge forces from
the prestressing, and the bearings, being the heaviest and most complex in shape, warranted not only their own special formwork mould, but also the largest amount of planning and discussion time in preparation. The reinforcement required to transfer all the forces had to be honed down to a practical arrangement that not only could be fabricated easily, but would also allow the concrete to be placed and compacted effectively.

Precision in the fixing of the reinforcement in the casting yard was of paramount importance, requiring specially manufactured jigs to ensure accurate placing. Although the first few segments took a huge amount of time to complete, with repetition and the well-known principle of ‘practice makes perfect’ the casting yard was finally churning out these reinforcement cages at three per day (Figure 6).

Reinforcement jigs for fixing the steel
High-precision reinforcement cage jigs were manufactured for each segment reinforcement type to ensure consistent and accurate placement of reinforcement.
Once the reinforcement cage is complete, it is lifted out of the jig using a horizontal steel frame spreader beam for even load distribution and transported to a storage place or lowered directly into the waiting external formwork mould (Figures 7 and 8).

Once the reinforcement cage is in place, the prestressing ducts are fixed and the internal formwork has been appropriately positioned, the segment can be concreted.

Order of casting
The end segments of each span (the SOPs), which have their own special formwork as mentioned above, are cast first as they form the start of the match cast process for all the spans. Once they have been concreted and reached sufficient strength to be moved, these SOPs are either moved to storage or directly to the ‘long casting bed’ where they are appropriately positioned and the adjacent internal segments are cast against them. The casting process proceeds from the ends of the span towards the centre, with the final segment being match cast against the two segments on either side of it.

Figure 9 shows the formwork stripping of a segment that has been match cast on the long bed form against a previously cast segment.

This new segment, once appropriately cured and cleaned down, will then be used as the ‘stop-end’ for match casting the next segment.

All segments are appropriately marked with both their segment numbers and their span numbers and viaduct numbers, each segment having its own unique position in the project.

Once the segment has served its match casting purpose, it can be moved to its storage place in or around the casting yard.

Installing the segments on the launching girder and applying temporary prestressing (Figures 10 – 20)
A segment loader supported by the previous span or, as in the case of the construction of the first viaducts, a mobile crane, is utilised for lifting the segments onto the underslung launching girders which are supported on purpose-made brackets connected to the pier tops.

Segment trolleys are used to deliver and align the segment on the launching
girder. The segments are lifted above deck level and lowered onto the jacks positioned on the four segment trolleys. Each trolley is equipped with a support jack and a transverse hydraulic jack that can move the segment transversely ± 50 mm. All movement of the segment longitudinally is also carried out using the trolleys. The segment trolleys, together with the segment, are driven along the launching girder close to their final position. The segment weight is then transferred from the trolley jacks to the segment support jacks. The above procedure is repeated until all the segments have been located close to their final positions. The gap between two segments can be as low as 0.1 m for segment gluing, aligning and joining. The segment is aligned vertically, transversely and longitudinally using the trolley jacks, care being taken to see that the gaps are completely closed after gluing.

Temporary prestress is then applied, providing an even stress of approximately 1.0 MPa.

The sequential procedure of ‘segment alignment, load transfer from the trolley jacks, gluing and temporary prestressing’ is repeated until all the segments have been joined together and are in their final position.

Permanent prestressing and load transfer
The permanent prestressing has to be installed and stressed before the span can be lowered onto the piers (Figures 21 and 22).

Stressing of the permanent prestressing deflects the span upwards. The loading on the launching girder is redistributed at the same time as the prestressing is applied. The deflected shape of the launching girder changes as well, with consequent changes to the load on the temporary segment support jacks.

The way that the loads are distributed depends on the procedure described above and on the particular deck being erected. It is also constrained by detailed calculations carried out by VET, the installation engineers.

The span weight has to be transferred progressively from the launching girder to the temporary span jacks while stressing the permanent prestressing. Care has to be taken to avoid any overloading of the jack supports and consequent damage to the segment cantilevers.

Span alignment
This operation is performed only once the load has been transferred to the span jacks. The span is not necessarily correctly aligned after the load transfer from the launching girder and needs to be corrected using the span jacks. The span jacks are used to adjust the span vertically, transversely and longitudinally. Vertical span jacks are located under the pier segment and are equipped with a sledge/sliding plate system.
Final prestressing
Stressing of the remaining (if any) permanent prestressing cables is done at any time after the span has become self-supporting, provided a special stressing sequence has not been specified – as in the case of station spans where some of the prestressing may only be allowed after the installation of the final permanent loads, such as ballast rails, canopy, etc. (see Figures 24 and 25).

STATION VIADUCTS
The station structures supported from the viaduct spans at Rhodesfield, Centurion and O.R. Tambo International Airport are unique to the Gautrain Project. Although there are examples of stations supported on bridge decks around the world, using precast concrete struts to support the station deck is thought to be a first (see Figures 26 and 27).

Design Development
Along the way there have been some significant challenges. One of the major ones was the need to erect the station structure over the live electrical lines at Rhodesfield. A design that would allow quick erection without significant temporary works was therefore important. Connecting the struts to the different types of precast deck segment with a single simple detail was a further challenge and needed careful review and consultation.

As always, the issues of economy and durability were key drivers in the design development. In the final analysis, concrete was Bombela’s material of choice, given
the resources at hand in their extensive precasting yards. However, it was also recognised that the stations are a very visible part of the project with which commuters will come into close contact. The design team therefore wanted to create an aesthetically pleasing structure that would reflect positively on the project.

**Design of stations**

The reinforced concrete strut is an offset Y-shape that independently supports the precast station slabs (Figure 28). Its base is attached to the web of the deck with a steel connection unit including a stainless steel pin joint. Each pair of struts, symmetrical about the deck section, is laterally connected by a prestress cable that stresses the struts against deck cantilever. The prestressed connection provides lateral restraint only, with the pin transferring the struts' axial and shear loads to the deck section.

The struts support a precast concrete platform slab which in turn supports the station's structural steel canopy. All other elements, such as the ballast walls, are connected to the struts.

**Shape of the struts**

Significant axial loads, shears and bending moments develop in the struts under the applied actions of commuters and the station roof structure. The challenge was to develop a concrete section that did not appear too heavy and bulky in the context of the bridge deck and the station canopy roof. In the end, a T-section was developed with a tapering flange depth. As the bending moments in the section increase, the flange section deepens. The maximum depth neatly coincides with the anchorage point for the prestressing cable. The result is an efficient structure with a light appearance. The outer sides of the strut are vertical to simplify the precasting operation. The details of the tapering flange depth and tapering web sides give the strut its character. Because of the number of struts constructed, a lot of time was spent in reviewing the strut geometry, the formwork and the lifting operations during erection.

**The deck connection**

The pinned connection at the base of the strut quickly became the focus of much of the design and development effort. The implications of the pin seizing over the structure's 100-year design life were serious. Any transfer of moments from the strut would
overload the deck section’s webs. To make sure this did not happen, a pin design more common to the lifting arm of an excavator was developed. A stainless steel pin sheathed in a synthetic material was detailed. This then connected an 80 mm mild steel spade between two 40 mm forks. This detail eliminated the risk of the pin seizing due to corrosion. The potential for bi-metal corrosion was avoided by using the synthetic sheath and neoprene washers on the pin cover plates.

The pinned connection is bolted to the side of the webs via a 40 mm connection plate welded to the forks (Figure 31). A second, thinner plate was cast into the webs, together with shear blocks of different lengths. The reason for the variation in the shear blocks’ lengths was the prestressing ducts in the web. In each deck segment the duct positions obviously varied. The connection detail, however, had to suit every segment. In the end, the design team used 3D CAD to model the decks’ segments for each of the station spans, including the prestress ducts, to check the chosen configuration graphically. Each shear stud was threaded internally to receive the M36 HSFG that joined the connection plates. The design team had to do some in-depth research in calculating the thread length necessary for connecting Grade 8.8 HSFG bolts to a mild steel block. The method was eventually found in BS3642:2007, ISO metric screw threads.

The need for tolerance during the erection sequence was emphasised throughout the design development process. To this end oversized holes were used on the connection plates. This meant that the shear was transferred through friction rather than a mechanical connection. The interface of the connection plates was therefore critical and machined surfaces were specified to ensure adequate surface contact between the two plates.

**The station deck**

The station deck is made entirely of precast slabs that are connected using halving joints. The vertical setting out of these slabs, however, needed careful consideration.

One important function of the deck slab was to support the columns for...
the station roof structure. To maintain the roof’s transverse stability, full fixity had to be provided at the base of the columns. It was found that a pair of struts was necessary to restrain the applied moments. This meant that the roof structure’s column spacing of 8 to 9 m dictated the position of the strut pairs.

The main concern in setting out the struts was that the arrangement should appear balanced. It was not possible to locate a strut on certain segments, such as the end segment and the deviator segments, because they were too heavily reinforced to accommodate the strut connection. The possible permutations were therefore limited. After much iteration, the struts were set at 2.5 m spacing on adjacent panels (Figure 32). The roof columns were then positioned at varying points between them, giving some flexibility in the setting out.

**Erection of struts**

During the first stage of the erection sequence the struts are prestressed in pairs against both sides of the deck section. Although stress bars were considered for the prestressing operation, prestress strands were chosen in the end (Figure 33). Although they require more effort in the erection phase, the design team felt that their ductility would give the desired robustness during the installation process.

One interesting design requirement was the allowance for additional axial load in the strut due to the shortening of the cantilever sections under stressing. The behaviour of the strut under various stages of loading was carefully checked using a 3D finite element model.

An in situ stitch between the struts and the deck, in which reinforcement from both sections overlapped and was tied together, was completed before the stressing sequence. This ensured there was no chance of cracks or recesses forming that might allow the ingress of moisture at this critical juncture. The need to thicken the deck cantilevers at each strut location caused a few challenges in the precasting of the deck segments. The solution adopted was first to box out the cantilever...
sections during the casting of the segment. Reinforcement was then added, and the widened section was cast after the segment formwork had been stripped, leaving a box-out for the in situ stitch.

CONCLUSION
The final design visible today went through a rigorous development process that involved a number of iterations. Since the Gautrain is a design and build project, the designers, Bombela Civils Joint Venture’s production units and the erection teams were able to confer throughout the product development. In addition, detailed design reviews by the Bouygues Travaux Publics head design office in Paris and Halcrow integrated hard-won knowledge from previous projects around the world. The Province design review team and the Independent Certifier (Arup) must also be recognised as partners in the design and construction process. Throughout the process the Bombela team has done an excellent job in pulling together all of the players to design and safely build an end-product that everyone can be proud of.
Gautrain precast concrete yard – achieving more with less

A QUALITY IMPROVEMENT in the production of any precast concrete product generally involves an increase in costs. So when better quality, which in this instance means strength, durability and impermeability, is achieved with a decrease in costs, the method by which this win-win situation is achieved bears close scrutiny.

The Gautrain Project’s Midrand-based precast concrete production facility has made significant gains in the MPa ratings of all the precast concrete elements it manufactures, while having simultaneously achieved considerable cost savings, especially on cement usage. Moreover, a third gain, that of productivity, came as an unexpected extra. All of this and more was achieved within the context of a 100-year lifespan requirement, one of the world’s highest durability specifications for precast concrete.

During my interview with Cyril Attwell, chief concrete technologist of Bombela, the consortium appointed for the design and construction of Gautrain, Cyril explained how the seemingly impossible was achieved. (Incidentally, during the interview I discovered from Cyril that there are fewer than 500 qualified concrete technologists throughout the world and that a large proportion of them are South Africans. This led to another pleasantly surprising revelation, namely that South African concrete technology is world class.)

The Midrand precast yard is producing viaduct segments weighing up to 58 tons each, M-beams of up to 40 tons apiece

1 Cyril Attwell, chief concrete technologist of the Bombela Consortium, stands next to a cluster of viaduct segments prior to their final placement
2 Viaduct 3 which spans 640 m and crosses Allandale Road - thirteen spans comprising between 15 and 18 viaduct segments per span
3 A close-up of the underside of Viaduct 3 in which the individual viaduct segments are clearly visible
and several other precast elements, and on all of these precast concrete elements approximately 100 kg of cement per cube has been saved since the first 400 elements came off the production line. This saving is based on the simple fact that when production began in April 2007, more than 400 kg of cement in combination with 100 kg of fly ash per cube was used to achieve the high early strengths. Thereafter, more accurate strength-testing procedures and better mixes meant that cement usage was reduced to 300 kg with a minor 20 kg increment in fly ash usage, representing a significant overall saving.

Getting better usage out of a given quantity of cement has been the key driver at the Midrand precast yard.

“We’ve achieved this by applying a locally developed technology called ARC (advanced recrystallisation) technology. The process first saw the light of day at the Grootvlei Prison in Bloemfontein where we achieved 12 MPa in 12 hours at an average temperature of –8°C. By using ARC technology at Marlboro we are averaging strengths of 74 MPa using a water:cement ratio of 0,45 with a fly ash replacement of 30%. We are also using a 42,5 Cem1 cement which is quite unusual. Precast yards generally use a rapid-hardening 52,5 Cem1 cement at a rate of 400 kg per cube to achieve a 74 MPa rating,” Cyril said.

He continued: “The 42,5 Cem 1 is not as refined as the rapid-hardening cement. It also presents a lower surface area and lower reactivity. South African fly ash has a very high reactivity of 0,8 as opposed to European fly ashes which are generally rated at 0,4, the net result being that we are achieving more than we would have done had we been using rapid-hardening cement.”

ARC technology is based on the manner in which cement, water, aggregates and extenders such as fly ash react chemically. The mineralogy of the aggregate in this process is critical. Unlike past practice when aggregates were considered as a bulk material only, ARC uses them as a chemical component as well, and in so doing enhances the strength of the concrete substantially.

“We are using dolomitic crusher sand in combination with the granite excavated from the Gautrain tunnel. Owing to the fact that less cement is being used (somewhere between 65 and 75% of a normal precast yard), our carbon footprint has been lowered accordingly. Moreover, the water used to wash the trucks on site has a high dissolved calcium content. It is collected in sumps and is reused in the production of our concrete. It gives us an additional 4 to 5 MPa and provides further savings per cube, simply because the high-calcium-content water is being reused.”

While the saving of cement and the lowering of the carbon footprint could be simply regarded as ‘nice-to-haves’, maintaining rigorous standards and best practice quality control is considered essential to the overall success of the project. Measuring and maintaining consistent MPa ratings for every single precast concrete element is key to this process.

“When we began production we made a corresponding set of test cubes measuring 100 x 100 x 100 mm for every concrete element produced. This gave us full traceability. Cube tests were initially used for gauging early strength ratings so that demoulding could take place at the earliest possible moment. However, given that the actual segments were considerably larger than the test cubes and because of this had higher heat and strength ratings, we found it difficult to make accurate strength rating correlations between the two during the first five days of curing.

“It was for this reason that we decided to opt for a maturity test for our early-age strength requirements and to use cube testing for
the 28-day strength assessment only, which is when this test gives a more realistic reading. The maturity test is based on measuring the heat generated within each precast element and gives us a far more accurate strength reading at any given time.

“We have produced over 3 000 precast elements since we introduced the maturity system and have not experienced a single breakage since then. To put this into perspective, two breakages out of every 400 precast concrete elements is considered to be good by world standards, whereas maintaining a clean slate in the production of 3 000 elements, as we have done, is quite exceptional.

“Unlike cube tests which take place under ideal temperature conditions of about 22°C, the maturity test is conducted in conditions of far greater thermal variance. For instance, we have experienced temperatures of -8°C, and it is in extreme conditions such as these that we need to know what is happening inside each element. There is no point in testing a cube that is kept in relatively ideal conditions and then hoping that this will be representative of early-age strength. Maturity testing tells us with far more accuracy when we can strip, move or detension and this alone has enabled us to accelerate our M-beam production from the original three-to-four-day cycle to a 36-hour cycle.”

The maturity test involves inserting several thermocouples into each precast element.

“As there are more than 46 reliable maturity formulas available worldwide, we did some research as to which one best matched the maturity (temperature/time) functions of our concrete. We opted for the Nurse-Saul function which accurately measures strength during the period from 5 hours to 24 days. The reason for inserting

Concrete erosion occurs when carbon monoxide and water combine inside a concrete structure to form carbonic acid. This lowers the pH level of the concrete and when it falls below nine, the passivation around the steel reinforcing is eliminated.
several as opposed to one thermocouple into each element is be-
cause the temperature of the core of, say, a viaduct is much warmer
than that of the wing areas. So before the deviation gets too high we
apply insulation formwork composed of polyurethane foam to the
wing areas. It is simply sprayed onto the exterior of the formwork
and this gives us a boost of 5 to 6 MPa within the first 24 hours.

“The increased production rate achieved through the maturity
test has more than offset the cost of the thermocouples. One of the
reasons for this is that we no longer have to make cubes for early-age
testing, and the production bottleneck caused by this testing has
been eliminated. In fact we have actually reduced cube usage by
some 60%. Many precast yards still use cubes and I believe this is the
first time that maturity testing has been used on such a large scale.”

Permeability testing on all precast elements is another
quality requirement at Marlboro. The 100-year durability
specification requires high levels of impermeability in the
concrete. Low permeability is achieved through high strength
ratings and good curing practice.

Concrete erosion occurs when carbon monoxide and water
combine inside a concrete structure to form carbonic acid. This
lowers the pH level of the concrete and when it falls below nine,
the passivation around the steel reinforcing is eliminated. By
increasing the impermeability of the concrete, the rate of carbon
monoxide diffusion through the concrete is lowered.

Note
Photos: David Beer
THE INCREMENTALLY launched construction system is best suited for the construction of bridge decks for railways and freeways over difficult terrain where it is not possible to erect staging, such as over rivers and ravines.

The system involves the casting of sections of the bridge deck behind one abutment in a specially designed casting yard. After attaining the specified concrete strength, the cast segment is stressed, and then, using purpose-made hydraulic jacks, launched forward to clear the casting yard. The next segment is then cast and stressed to the rear of the first segment, and the two segments are then launched forward again. The whole process is repeated in a ‘sausage machine’ process until the entire superstructure has been launched into its final position.

Stefanutti Stocks Civils is one of the leading engineering and construction companies in South Africa in the incrementally launched construction technique. At present it is involved in three major projects.

THE KHANGELA BRIDGE, KWAZULU-NATAL
This composite dual-carriageway bridge structure crosses the four main electrified Metrorail lines serving the south of Durban, the eight-lane M4 Southern Freeway and six electrified Portnet lines taking the main goods lines into Durban harbour. There are three different types of bridge construction in the one structure:
- 147,30 m of incrementally launched bridge deck
- 15,90 m of cast in situ deck
- 39,10 m of precast beam deck
The project has progressed well, with the northern incrementally launched bridge deck and precast beam decks complete, and the southern incrementally launched bridge deck past the half-way mark. Construction of the main approach roads is also nearing completion.

KING SHAKA INTERNATIONAL AIRPORT INTERCHANGE BRIDGES, KWAZULU-NATAL
This project involves the construction of the main interchange off the N2 freeway, taking traffic into the new King Shaka International Airport to the north of Durban.

There are two main bridges in the interchange. The first is the overpass bridge, which is a conventional four-span, voided deck structure with a length of 86 m crossing over the busy N2 freeway. The second is the main ramp bridge, which is a seven-span incrementally launched bridge deck with a
length of 284 m crossing over the overpass bridge and the N2 freeway.

The project started well and most of the substructures are complete. The main launch yard for the incrementally launched bridge deck is currently being constructed, and staging for the overpass deck is being erected.

**INCREMENTAL LAUNCH OF BRIDGE OVER THE R21-N1 INTERCHANGE - PRETORIA**

Over the past ten years, the growth in peak-hour traffic in Gauteng has led to the inevitable requirement for an upgraded road and traffic infrastructure. In response, the South African National Roads Agency Ltd (SANRAL) proposed the Gauteng Freeway Improvement Project (GFIP), which is designed to improve the existing Gauteng freeway network and to provide additional infrastructure by May 2010.

The part of this project awarded to Stefanutti Stocks Civils is the construction of a 240 m bridge to span the N1 and R21 interchange in Pretoria. The purpose of such an extensive bridge project is to relieve the considerable traffic congestion around the subgraded access loop within the interchange, allowing access from the R21 (from the airport) onto the N1 north.

Due to great spans between the piers, and working to a height of 17 m above live traffic on the busy N1, it was decided to construct the bridge by means of the incremental launch bridge construction method.

The bridge will be constructed in 18 segments, each weighing approximately 323 tons. The sections will be “manufactured” in a casting yard where they will be launched in increments, using hydraulic jacks, into the desired final position.

The geometry of the superstructure adds to the technicality of the project. To achieve a horizontal and vertical curve of the road surface, the design of the superstructure is based on a circle with twisted x and y axes. By twisting the axes the circle becomes an ellipse through which the global curve is achieved.

Extensive temporary works designs were carried out to cater for the powerful launching forces, the great spans
between piers and the dead weight of the superstructure in the construction phase. The works designs include a launching girder, temporary piers, structural guides and a casting yard facility capable of achieving construction tolerances of 1 mm.

The superstructure will slide along temporary bearings, prepared and installed to an accuracy of 1 mm. The construction tolerances are important due to the stiffness of the superstructure.

In addition to the tight casting cycle, an advanced early-strength, high-durability concrete is required to achieve a minimum of 35 MPa compressive strength over 60 hours prior to post-tensioning and subsequent launching of a superstructure segment.

When the superstructure has been completed, the final post tensioning will be done and the temporary bearings positioned below the webs of the superstructure will be replaced with permanent bearings positioned on single columns 2 m in diameter. Completion of the bridge is scheduled for the second quarter of 2010.
Risk Analysis and Management for Projects

INTRODUCTION
Given today’s systematic approach to project and risk management, it is tempting to believe that projects are not only fail-safe, but also executed within budget, on schedule and are of the intended scope. Unfortunately this is not always the case, and as a result many projects and companies fail due to unforeseen and unidentified risks during the entire life cycle of projects. Current methods, although advanced and thoroughly thought through, have a number of shortcomings when it comes to project risk. The consequences of these shortcomings are numerous and often troublesome.

A relatively new method which aims to overcome the weaknesses of current methods is known as RAMP – Risk Analysis and Management for Projects. The actuarial and civil engineering professions developed the method jointly.

RAMP is a wide-ranging method in which risks are given a financial value and managed accordingly. The intention is to achieve as much certainty as possible about the project’s entire life cycle, from concept to termination. The method assists with risk mitigation and provides a control system for the remaining risks.

The RAMP method is systematic and ensures that major risk issues are adequately addressed. RAMP identifies three areas of risk:
1. Risks to the health and safety of people, including personal injury and loss of life
2. Risks to the environment, including pollution, damage to flora and fauna and soil erosion
3. Risks to the project, including damage to equipment, loss of output, resultant contractual delays and penalties

These three areas have financial, schedule and scope implications which in turn determine how much time and money should be spent on reducing the risks to an acceptable level.

METHOD
The RAMP process consists of four activities which are carried out at different times in the life cycle of an investment or project:

- Process Launch
- Risk Review
- Risk Management
- Process Close-Down

The activities are further divided into several phases and process steps that are carried out at key risk review intervals or crucial stages. Risk management activities are performed between risk review intervals and are based on analysis, strategies and procedures created during risk review intervals.

Process Launch
The first task is to confirm the perspective from which the risk analysis and management is being carried out and to identify the principal stakeholders. A risk process manager is appointed who plans, leads and coordinates the entire process.

A preliminary brief on the cost, scope and schedule of the project or investment must be prepared at this stage. This includes a strategy for risk review and management throughout the lifecycle of the project. Issues that must be addressed at this stage include the purpose of the process, an analysis of the acceptable level of risk, a review of the phase-specific scope of the project,
review stages and finally the amount of time and money the client is willing to spend on risk analysis and management.

At this stage, the RAMP strategy must be communicated as fully as possible to all stakeholders, and there must be frequent involvement of the various consultants, contractors, stakeholders, investors and managers. Finally, a team must be formed from these parties. The team members will act as risk analysts during the lifecycle of the project.

**Risk Review**

In this crucial phase, the aim is to identify all significant types and sources of risk and uncertainty associated with the scope and objectives of the project. These risks will then be assessed relative to other risks, and classified and grouped for evaluation purposes. Significant or potentially significant risks are evaluated and the following options are considered for mitigating the risk:

- Reduce or eliminate the risk
- Transfer the risk
- Insure the risk
- Avoid the risk
- Absorb the risk
- Obtain better information to reduce the uncertainty about a particular risk

Risks are recorded in the risk register with the proposed mitigation option and the reason for the selection. This information is then used to create an action plan which will be used to implement action applicable to the identified risks. In addition, a risk mitigation strategy is compiled showing the action plans, and a risk account indicating the financial implications of the mitigation measures. The project value is calculated using an investment model to reflect the approved mitigation measures. The results are reviewed and re-evaluated for different risks and measured to obtain different results. This will not only indicate what mitigation measures may be omitted due to high cost and limited beneficial effect, but will also indicate the best mitigation option for high risks.

A residual risk analysis, which involves the assessment of residual risks and the appropriate mitigation measures, while also considering secondary risks and the financial implications, must be carried out. Residual risks are classified by order of significance for each economic parameter that might influence the project. Monte Carlo simulations or other techniques are applied, and the risks for each parameter are collected and recorded in the risk register.

An estimate is made of the potential impact of unforeseen and unmeasured risks, based on experience and the complexity of activities in each phase of the project life cycle. Using contingencies, the project value is recalculated for different parameters, and sensitivity analyses are performed on assumptions and estimates. The results are analysed on volatility, reliability of estimates and potential consequences of major risks. A final review is carried out to determine project viability. Eventually the client and other stakeholders will issue a formal approval to proceed with the project and the risk mitigation strategy.

A risk response plan is then compiled in which each residual risk is assigned to a responsible person. Responsibility is also assigned for various actions listed in the risk mitigation strategy. The risk response plan also includes the following:

- Plans to minimise risk and impact
- Plans to deal with specific residual risks, including anticipated ‘trigger’ events for which contingency plans will be implemented
- Contingency budgets allocated to residual risks
Finally, a risk review report will summarise the main results of the risk review phase, including the main risks, project riskiness, effectiveness of the review, problems experienced, lessons learnt and recommended improvements for future reviews.

**Risk Management**

At this stage the risk response plan is implemented in the main project and operating management processes, with responsible persons assigned to each activity. All actions are monitored and any significant changes in risk or new risks must be reported and assessed immediately.

Risk monitoring can be done by studying events, situations or trends. These trends must be systematically identified, analysed and monitored on a regular basis by scrutinising reports, letters and notes on visits, meetings and telephone conversations. The results are entered in trend schedules. Ideally, these should be considered at regular progress meetings involving key members of the management team.

As progress is made through the project life cycle, the residual risk analysis, risk mitigation strategy and risk response plan are revised and the contingency budget released as risks materialise, change or disappear. Regular risk review reports are submitted to the client and other key stakeholders during this phase.

**Process Close-Down**

During the final phase of the project life cycle, a review is done of the contribution to, and effectiveness of, the RAMP process as applied to the project. The risk process manager, in conjunction with the client, evaluates the project and compares the results with the original budget, scope and time schedule. Using risk review reports, an assessment will be made of the risks, their actual impacts and successful or unsuccessful mitigation. The results of the evaluation will be recorded in a RAMP close-down report, which will be applied to future projects.

Some projects will be terminated as soon as the initial risk review has been completed as a result of high risk and a low return on invested capital. Other projects may be terminated before the end of their expected life cycle because the risk-reward ratio is not deemed to be sufficiently attractive, and other projects will be terminated before the end of their planned life cycle because of undesired developments. The production of a RAMP close-down report as a guide to future projects is likely to be particularly valuable in these circumstances because the most critical events in the history of the project will have occurred recently.

**CONCLUSION**

Although modern project management theory makes provision for risk management as an element of project management as a whole, existing methods of risk management often fail to identify many of the risks. This can be attributed mainly to the fact that risk management forms only a small part of the project management process. A specific focus on risk analysis and risk management as a separate component of a project cannot be overemphasised. Risk process managers and project managers should rather work together to approach projects holistically and allow effective execution of all the project activities during the entire project life cycle.
ON TUESDAY 27 January 2009, 22 eager engineers boarded a bus, sponsored by the Trans Caledon Transfer Authority, to embark on a site visit organised by the SAICE Project Management Division (PMD) to the Vaal River Eastern Subsystem Augmentation Project (VRESAP).

The VRESAP project involves extracting water from the Vaal Dam and pumping it to Bosjesspruit and Trichardtsfontein Dams close to Secunda. It was instigated when Water Resource Planning showed that the Sasol complex and power stations in the area may run short of water by mid-2009.

On arrival at the VRESAP site offices, we were met by representatives from Vaal Pipelines Consultants (VPC), our hosts, who sponsored refreshments throughout the day. The contractor for the abstraction works gave us a brief safety induction course. Then it was off to see the temporary abstraction pump station and the construction of the permanent low-lift pump station. Items of interest included the substation requirements to feed power to the temporary abstraction pump station and the construction of the permanent low-lift pump station. The slab of rock at the permanent pump station excavation which is so smooth that it looks like a cast slab of concrete, and the relatively small size of the site for the permanent pump station, one of the reasons for this being the rare fauna on the koppie just above the site.
An interesting aspect of the permanent abstraction pump station is the multi-level suction pipe for feeding the pumps. This will be extended as a channel into the Vaal Dam, with the design allowing for the pump station to operate when the dam’s water level is as low as 18%. This will require underwater blasting to create the trench, which will require some very careful planning.

The water extracted from the Vaal Dam reports to the balancing dams. These dams feed the pumps housed in the high-lift pump station, from where the water is pumped to the Knoppiesfontein diversion structure. From this structure the water reports to either the Sasol water supply system (Bosjesspruit Dam) or the Trichardtsfontein Dam which feeds the Eskom power stations in the area.

The site visit then moved on to the high-level pump station. This has a control room that will be operated remotely from the Grootdraai Dam. There are four sets of pumps: each set is made up of two pumps – one fixed-speed pump and one with a variable-speed drive. The pumps pump into a steel pipeline that is 1,9 m in diameter and 115 km long, with a surge tower at the high point for pressure alleviation.

Following the site visit, we sat down for a presentation prepared by Paul le Roux, our host from VPc. The presentation concentrated on the major lessons learnt during the implementation of the project, ones that can be fruitfully applied in the inter-basin transfer projects now being considered by the Department of Water and Environmental Affairs (DWEA, formerly DWAF). This is of the utmost import-

tance to ensure that future project teams learn from the experience of this and similar projects so that the country and society as a whole benefit from technically better and more cost-effective projects.

Some facts of the project:
- The project’s objectives included:
  - Pumping 5,4 m³/s of water over a distance of 115 km in a 1,9 m diameter steel pipe to supplement the Vaal River Eastern Subsystem
  - Fully automated system required to be controlled remotely from the Grootdraai Dam

Construction of the low lift pump station
The inside of the high lift pump station
The design life of the project was 45 years with an overall availability of 94%. Training of operational staff was to be done during commissioning of the pipeline. Operational and maintenance manuals were to be included as part of the as-built records.

The project’s budget was R2.7 billion.

There were two design contracts: one for the civil works undertaken by VP c (a consortium of Goba, Ninham Shand and PD Naidoo) and the other for the mechanical and electrical works undertaken by the DWEA.

Two construction contracts were awarded: the civil structures and MEIP components to COVEC Mathe Construction (a joint venture of COVEC and Mathe Construction); the pipe supply and installation to MPC JV (a joint venture between Murray & Roberts, Group 5, WJ Pipelines and the J&J Group).

There has been a focus on health and safety. At the time of the visit:

- 10,844,400 manhours had been expended
- 80 recordable incidents had been reported
- The RCR (resource cost ratio) was 1.48
- One fatality had occurred – the project team emphasised this was one too many

Only one pipe length could be transported on each truck.

Pipe manufacturing capacity was of about 300 m/working day; pipe laying capacity was 200 m/working day.

Risk management principles were used during the project execution. Examples of this include the decision to implement the temporary low-lift pump station from the Vaal Dam due to the expected late commissioning of the permanent low-lift pump station, and assistance to consultants from the client to retain staff during construction.

FIDIC (International Federation of Consulting Engineers) Conditions of Contract were used.

In closing, the PMD wants to acknowledge the efforts and inputs of the sponsors of this visit, VPC and TCTA.
THE ‘POWeR cRiSiS’ is a well-known and daily experienced reality facing South Africa. Eskom has placed huge emphasis on its capacity regrowth and electricity generation plans. Demand for and growth in electricity supply is also on the rise in countries bordering South Africa. Babcock Ntuthuko Powerlines (BNP), which has been in existence for the past 55 years as a market leader in powerlines construction, is currently also managing powerlines projects in Namibia and Botswana for their respective power utility departments.

Project managers in the powerline construction industry need to have a technical background with a solid financial understanding as they work closely with their design team (engineers) and foremen. Balancing the work demand with the available skills base, together with the available plant and equipment, leaves the project management team with its work cut out for them.

A typical powerlines project includes a myriad of activities, ranging from geotechnical and foundation works to structural design, while considering the environment, with safety as the prime consideration. Managing a powerlines project is challenging and exciting.

Projects are often located in remote areas, away from existing infrastructure such as water and electricity supply. This obviously requires careful planning. Before a project can begin, a mini village needs to be established with accommodation erected, safe drinking water sourced and temporary power generation set up.

During a powerlines project the skills and labour base, together with plant and equipment, will vary, depending on the activity at hand. The bush clearers and the survey team are the first on the line, followed closely by the foundation teams.

Tower assembly and erection teams are phased in while the foundation teams are phased out and moved to their next project. The skilled teams that ‘string’ the line are brought in towards the end. They can work at height, often in tricky conditions, and are responsible for attaching the conductor cables together with the earth wire and OPGW (telecommunications) cables.

The last phase in a powerlines project is ‘tower dressing’ and ‘regulating’. This implies that all the relevant anti-climbing devices, aircraft warning devices, bird diverters and tower labels are attached. Regulating is the process of ensuring that the conductor and earth wires are all spanned and tensioned according to specification, ready to be signed off and handed over to the client.

The social and environmental issues of a powerline crossing an environmentally sensitive area or passing through a culturally sensitive region may cause time delays resulting in financial losses. These issues, often viewed as an inconvenience to a technical project, however need to be carefully and diplomatically managed.

The skills gained in the powerlines industry are often unique to this sector. The aging skills base, coupled with a lack of industrial or government-sponsored training programmes, is one of the problems that have to be faced when managing powerlines projects, as voids in the project teams are starting to appear.

Two tower types are used predominantly in a powerlines project, namely the ‘suspension tower’ and the ‘strain tower’. The former acts as a bridge between two strain towers. For each tower type, there exists a number of shapes and sizes depending on the intended function, as can be seen when taking note of the towers that line our cities and countryside.

These two tower types, ‘self supporting tower series (strain tower)’ and the ‘V-guyed tower series (suspension tower)’, are being used on one of BNP’s projects started this year - the Spencer-Tabor 275 kV transmission line. At 85 km in length it is tame in comparison with other projects undertaken by BNP, but the tricky Tzaneen conditions of thick bush and mountain passes make up for the length. In all, 219 towers are to be constructed and over 1 200 foundations, using approximately 620 tons of tower steel, 425 000 m of cable, and 4 000 m³ of concrete. Completion date is planned for the end of 2009.

Cross rope tower – one of the tower types used in powerlines projects

A contribution from one of SAICE’s young engineers who loves his job out in the open.
On 11 June 2010 kick-off will take place at the Soccer City Stadium in Johannesburg. PPC Cement, who supplied 90% of the cement used to construct the stadium, kindly provided these photos of the stadium nearing completion. According to Dr Orrie Fenn, PPC’s Chief Operating Officer, this project fits in perfectly with PPC’s philosophy of contributing towards ‘building the nation’, as Soccer City will remain a key sporting and cultural facility well beyond the 2010 FIFA World Cup.
A short history of roads in the Northern Cape

INITIALLY, IN THE Northern Cape, as elsewhere, there were no roads.

On the ocean fringes the Strandlopers had adjusted their lifestyle so that they operated in a narrow coastal strip. In the interior were the hunter-gatherers, who were by definition nomadic, requiring water and a source of food. These requirements were catered for by following the herds of wild animals during their seasonal migrations. The routes were selected and beaten out by the game, which of course chose routes with sufficient access to water. The game in turn provided food for the hunters. There was no need for roads.

Then came the chaps from the North, with their herds of cattle. A pastoral life, once again nomadic, with the herds moving gently as the seasons changed. Possibly one would not describe these movements as “migrations”, but the effect was very similar. And of course where their destinations coincided, on the other side of a mountain for example, the cattle and the wild game followed the same routes, over the mountain, to where the grass was greener.

Two major events changed the composition if not the pattern of activity in the area. The first smallpox epidemic of 1713, and the second of 1755, drastically reduced the numbers of both white settlers and Khoikhoi, resulting in white trekboere joining the Khoikhoi, and occupying rural land which had become vacant, especially in the west. Their lifestyles and needs were similar: again, no call for roads.

The needs of these pastoralists for certain items available in Cape Town, which could not be provided by themselves in the hinterland, were partially met towards the end of the 19th century when Jews, fleeing from political pressure and pogroms in Eastern Europe, appeared on the scene as smouse. Initially, their peckels contained only light articles like needles, thread, knives and tobacco, as the peckels were carried on their backs, but in due course the smous would appear with a larger pack on a donkey, and later still with...
his pack and himself on a cart pulled by two donkeys, so that he was able also to supply heavier articles.

This progression illustrates in a smaller way how the development of a country increases the demand for transport to serve its increasing needs. And in the Northern Cape, as in many other parts of our country, the road and the traffic on it is the major facility which caters for these needs.

THE DAWN OF ROADS IN THE NORTHERN CAPE
With the growth in the population roads suitable for ox-wagons, linking the various areas in the Northern Cape and elsewhere, began to be provided. As roads pushed up from the south into the northern Cape, farm-to-market roads made it possible for crops to be grown for marketing, where previously only subsistence farming was feasible. An obvious side-effect was for the nomadic lifestyle of the previous generations to be replaced by farming which was more agricultural in nature. With fixed dwellings and farms, the desire for additional roads grew, as was natural.

Similarly, where the development of the Namaqualand copper resources proved by Simon van der Stel as early as 1686 had previously been impractical because of the lack of access to a market, mining could now be carried out as roads to a sea port made it practical for the ore to be exported.

The demand was now for roads suitable for ox-wagons. Traffic in general was light, and except in places such as mountain passes, where road construction was needed to overcome some natural barrier or hazard, “roads” were generally just wheel tracks where someone had passed previously. The farmers and transport riders of those days showed a marked aversion to getting down from their wagon seats and actually doing something themselves to improve the road. If a section deteriorated where “the road” was not constrained by flanking development, the wagons merely pioneered another parallel track.

[This is ably demonstrated in Johannes Schumacher’s sketch “Hottentots Holland Klooif in 1776” (Ross 2002: 15 for example), where the many parallel wheel ruts can be clearly seen on the approaches to the pass. In North America Louis L’Amour (1960: 26) gives us “in 1897 the Santa Fe Trail was an old trail, cut deep with the ruts of the heavy wagons... It was no road, only a wide area where many ruts showed the way the wagons had gone through the 50-odd years the trail had been used.”]

MOUNTAIN PASSES
The Northern Cape’s mountain passes are in the western and southern fringes of the province. The wagons of that day were necessarily constructed with a high clearance to pass over rocks lying in their way, or to surmount the sharp change of gradient at the edge of a donga or stream. As a result they had a high centre of gravity so that if the wagons tipped too far to one side, they were liable to fall over, downhill. It follows that on steep mountain (or other) slopes they had to cross the contours
more or less at right angles, and so could not ease the strain on the oxen by angling uphill at a lesser gradient. So much was this the case that on certain passes where the kloofs were impassable because of topography, or because they were heavily wooded, or because they were full of a river, the wagons in fact had to go over the very summits of the mountains (like over the mountains above the Messelpad Pass on the copper route to Hondeklip Bay), as absurd as this may seem at first glance!

But when an engineered road had been constructed across the mountain, this provided the wagons with a more-or-less level shelf cut into the mountainside which the wagons could traverse without toppling over sideways. The road could then follow a less steep gradient, and also aim for the low crossing point in a neck, making life much easier for all concerned.

Examples of passes which have been improved in this manner are: the Messelpad and Wildepaardehoek Passes, on the old copper road to Hondeklip Bay, built in 1871; Verlate Kloof Pass, down the Roggeveld Mountains from Sutherland, completed by Thomas Bain in 1877; Teekloof Pass, south of Fraserburg, on the road to Leeu Gamka, opened in the late 1890s; Spektakel Pass, down to the Sandveld west of Springbok, initially improved in 1896 and reconstructed in 1981; and a late-comer, Anenous Pass, which was deviated to follow the line of the old narrow-gauge copper railway down to the Sandveld east of Port Nolloth in 1953. In addition, a number of passes, such as Garies Hoogte and Burke’s Pass on the N7, have been constructed as part of the National Roads programme (of which more later).

START OF FORMAL ROAD CONSTRUCTION

The next step forward was occasioned by the introduction of lighter horse-drawn or mule-drawn wagons and carts in the latter half of the 19th century. It is probably not generally appreciated that the introduction of these vehicles was as great a step forward as was the introduction of motorised transport in the 20th century. These lighter vehicles provided a faster and more efficient service – trotting horses go two to three times as fast as oxen – resulting in greatly reduced journey times and generally improved communications within the region.
The not-so-steep constructed roads over mountains mentioned above made it possible for the more lightly built vehicles to get to the hinterland beyond the mountain ranges, otherwise inaccessible to them. In fact, generally the road surface had now to be improved: uneven and bumpy roadways which the slower ox-wagons took in their stride were not acceptable to the speeding horse-drawn vehicles. And this had the indirect effect of upgrading the standards to be applied to future road construction, to the general benefit of all.

In 1855 Divisional Councils were introduced (Act No. 5 of 1855) and this improved the control of roadworks as there were now people with local knowledge making the decisions and controlling the work.

This form of control over the construction and maintenance of important local roads continued, in the Cape, for more than a century, and many say that it was the most appropriate for its purpose of any of the various control measures applied throughout the history of roads in our country.

THE EFFECT OF MINING
Our Northern Cape roads were considerably affected by the discovery of diamonds in and around Kimberley in 1871, at about the time that Griqualand West was annexed to the British Empire. Although the plodding ox-wagon continued to carry most of the heavy goods needed on the diamond fields, a demand rose for fast passenger conveyance, and this need was met by horse-drawn and mule-drawn light wagons and stage coaches running from Cape Town and Port Elizabeth. These coaching companies built and improved roads, and even provided a pont across the Gariep River at Hope Town. With relays of horses every 20 miles, the trip from Cape Town to Kimberley was accomplished in seven and a half days.

However, this was the age of the railway, here and internationally. The first steam locomotive had just been introduced on the narrow-gauge copper line from Port Nolloth, and to meet the demand for transport to the diamond fields the railway line from Cape Town, which had been stalled at Wellington since 1863, suddenly pushed forward towards Kimberley, as did other lines from Port Elizabeth and East London.

When the rail reached Kimberley in 1885, road transport from the ports was really used only for some of the heavy non-urgent loads.

But the roads through the Northern Cape to the diamond fields had been improved and road transport had shown itself to be capable of carrying both passengers and freight wherever an adequate road was in existence. And when gold was discovered in the Transvaal in 1886, coach services were able to be instituted from the railhead in Kimberley almost immediately.

BRIDGE BUILDING
With the increase in the tempo of road building in the Cape in the 19th century, attention began to be given to providing bridges in rural areas at points where more important roads crossed rivers. The first was on the Palmiet River east of Cape Town in 1811.

In the Northern Cape the main river barriers were the Gariep and Vaal Rivers. Where drifts or causeways were not available, crossings initially were by pont. The first road bridges were provided over the Gariep near Colesberg in 1881 and near Hope Town in 1882. These were massive bridges for their time. The Vaal River was bridged near Barkly West in 1885, and when the Modder River was bridged in the same year to carry the new railway to Kimberley, it was provided with a wooden deck so that it could also be used by wagons.

Other river bridges have followed as time has gone by.

NATIONAL ROAD NETWORK
In the first half of the 20th century it was realised that the development of a network of high-standard all-weather roads, connecting the main centres in our country, was needed. After due discussion it was decided that central government should be responsible for providing such a network, and the National Roads Act (Act No. 42 of 1935) was promulgated.

The original road network has over time been amended, but the basic principles remained unchanged in that overall control and funding remained the responsibility of central government, while design, construction and maintenance continued to be under provincial control. The exact constitution of the national Department of Transport controlling body has varied over the years, and construction, which initially was done by various provincial construction units, is now predominantly the responsibility of civil engineering contractors, while design is for the most part now carried out by consulting engineers. The provinces have thus unfortunately lost what were excellent staff training grounds in design and construction sciences.

In the original 1936 scheme the Northern Cape had but three National Road routes, all on the eastern edge of the province. One result of this route selection, unfortunate from the point of view of the Northern Cape, is that the “Great North Road”, which for many years had run through Ceres, Sutherland and Fraserburg to Victoria West, now ran outside the province via Worcester and Beaufort West to Three Sisters.

In the intervening three quarters of a century the population of the region has grown considerably and activities, with their need for transportation and communication, have increased to keep pace with this expansion. Today the Northern Cape is criss-crossed with a network of eight National Roads, which form a basic grid of high-standard permanently surfaced roads connecting the major centres within and adjacent to the province.

TRUNK ROADS
In 1949 the Cape Province instituted a system of Trunk Roads. These were to be surfaced all-weather roads which were to provide mainly east-west complements to the initially predominantly north-south National Roads. (Many of these Trunk Roads have subsequently been included in the expanded National Road network.) Subsidised construction was done by the divisional councils, with professional assistance and guidance provided by the provincial District Roads Engineers, engineering surveyors and materials laboratories. It was a brave and bold scheme, just what was needed at that time.

It was also a most challenging scheme. Appreciate that it was only in 1927 that the very first rural bitumen-surfaced road was built, by contract, between Bellville and Paarl. The padmakers of the 1950s had plentiful experience in building gravel roads, but permanent surfacing, whether bituminous or concrete, was generally something new to them. However,
all concerned tackled their projects with tremendous enthusiasm and the scheme proved to be most successful.

Unfortunately, the stage was reached at which the available annual provincial funds were needed for the maintenance of the already-constructed Trunk Roads, so that no more construction could be subsidised at that time. This was a great pity, as the scheme helped tremendously in opening up many areas in the province.

**MODERN DEVELOPMENTS**

When the Republic was split up into 11 provinces in 1994, the Northern Cape had 5 468 km of paved and 22 155 km of gravel roads in the rural Trunk, Main and Divisional Roads categories.

The geometric design of urban streets requires an entirely different approach to that appropriate for rural roads. Where rural roads are designed to shed their water onto the surrounding ground, it is an important function of urban streets to be able to accept runoff from adjacent properties. Urban streets must always have a longitudinal slope, so that they can collect stormwater at the inlets to the underground stormwater sewers which carry it to a discharge point. The grading of urban streets can also largely be influenced by the necessity to provide pedestrian and generally also vehicular access to the adjacent properties.

It follows that, except in the case of major cities like Kimberley which have qualified and experienced civil engineering staff, the design of streets is normally entrusted to provincial or consulting engineers who have worked in this field. And as the construction of a street scheme calls for a team composed of units experienced in their particular specialities, which need will fall away when the scheme is completed, construction in smaller urban municipalities particularly is also generally outsourced to civil engineering contractors.

This has been the pattern in the Northern Cape in the past, and the number of important, and indeed also not-so-important streets that have been constructed and, where appropriate, surfaced, bears testament to the success of this approach.

In this article we have scanned through recorded road transportation history, and have seen the growth in our population and the development of our industries to cater for this growth demanding ever-increasing expansion of the road transport network. We have seen the transition from ox-wagons to horse-drawn vehicles, and then to motorised transport, and we see the current growth in the numbers and sizes of transport vehicles, all in turn requiring routes of ever-higher standards and capacities.

This is a challenge which we must be prepared to accept and meet.

**REFERENCES**

The list of references is available from the editor.

**NOTES**

1. This article was written as an introduction to the Northern Cape O&M Handbook, and is published here by kind permission of the Steering Committee of that Project.
2. Photos are from The Romance of Cape Mountain Passes by Graham Ross (2002, Cape Town: David Philip).
Kus- en hawe-ontwikkeling in Gaboen

SUID-AFRIKAANSE raadgewende ingenieurs skyn besonder populêr te wees in Afrika en die Midde-Ooste. Tekenend hiervan is dat Suid-Afrikaanse firmas al meer betrek word by projekte in hierdie lande, en gekoppeld word aan internasionale maatskappye. Dit is seker so dat prys en kwaliteit bydra tot hierdie populariteit, maar daar is waarskynlik vele ander oorwegings wat ook ‘n rol speel. Baie internasionale firmas wil ook die Afrika-mark betree aangesien groot skaalse ontwikkeling op die vasteland verwag word oor die volgende paar dekades.


In 1996 al is die Suid-Afrikaanse firma Entech Consultants (Edms) Bpk (Entech) aangestel om die herstel van Port Voorgestelde vissershawe Port Gentil
Harbour and coastal developments in Gabon

SOUTH AFRICAN CONSULTING engineers seem to be very popular in Africa and the Middle East. This article takes a brief look at some current involvements in West Africa, and specifically Gabon, with regard to harbour and coastal facilities. Coastal and Port Engineering Consultants (CPEC), previously part of Entech, were recently contracted to undertake the design of a fishing harbour at Port Gentil, Gabon. This involves the restoration of existing structures and the construction of new berths to accommodate both fishing boats and ferry boats. A monitoring system for control over foreign fishing boats forms part of the project. A fishing harbour at Libreville is also planned, but the precise location remains a problem. Other projects in Gabon are the transformation of the old harbour at Libreville into a small Victoria and Alfred Waterfront, and extension of the existing Port Owendo harbour. Activities in West Africa have now expanded to states such as Cameroon (harbour and waterfront), Nigeria (renovation of Bar Beach at Lagos) and various projects in Angola. In Sao Tome, work on a deep-sea harbour on a volcanic island about 200 km west of Gabon is also being negotiated.
LETSENG DIAMOND MINE, LESOTHO, WINS FULTON AWARD

THE 2009 FULTON AWARD in the category Construction Techniques was recently won by the Letseng Diamond Mine extension project for excellence in cold weather concreting. The companies that undertook this challenging project were Bateman Civil and Structural Engineering, and Stefanutti Stocks Civils.

The Letseng Diamond Mine is situated in Lesotho on the Buthe-Buthe/Oxbo road near Fouriesburg, 3 100 m above sea level in the Maluti Mountains. The new process plant was constructed in a greenfields area situated approximately 500 m from the existing plant, on a slope of 17° to facilitate a gravity-fed process. A stepped formation of four terraces with heights varying from 3 to 10 m had to be constructed to accommodate this requirement. The footprint of the bulk rock excavation was set at 30 m wide x 95 m long.

In total about 1 000 t of structural steel and 6 200 m³ of reinforced concrete were used during construction, which involved:
- Weather shield foundations
- Retaining walls to support the terraces inside the plant due to the overblasting
- Boundary walls around the remainder of the plant
- Primary crusher walls and suspended slabs
- Two cone crusher foundation walls and suspended slabs
- Bins, screens, vertical conveyor, conveyor, DMS structure and platform foundations
- Scrubber and thickener foundations
- Surface beds, including sumps
- Rock anchoring of rock face due to blasting and fracturing of the rock formations

The reinforced concrete tunnel consists of three sections, namely the main feed chamber and two cast-in frames constructed to facilitate the feeding arrangement onto the conveyors located beneath, the conveyor exit tunnel leading to an open-ended portion and an escape tunnel.

The plant was provided with a tailings conveyor 748 m long and four additional conveyors. The stockpile feed conveyor is approximately 123 m long and the cantilevered head end of this conveyor is supported by the stockpile tunnel. Three further stockpile conveyors were also provided.

The engineering and construction teams faced numerous and varied challenges. These included the human resources issues associated with working in such taxing conditions, the difficulty of access via the very steep mountain passes, the availability of a reliable supply of cement and aggregate, the steep inclines and lastly, the altitude which seriously affected workers’ productivity.

To meet the client’s requirements and to maximise return on investment, the project schedule was reduced to an absolute minimum so that 65% of the concrete had to be placed in winter.

Special measures had to be taken to cope with the cold weather. Letseng Mine, being located at an altitude of 3 100 m, experiences temperatures varying from a maximum of 20°C to an average minimum of 6°C, with the minimum sometimes falling to -20°C, and severe wind chill factors. The cold weather sets in from about April and lasts until September.

To make construction during winter possible, a strict cold weather concreting specification was developed, which drew on the American Concrete Institute report ACI 306 R-88. The contractor developed concrete mix designs and a quality control plan that was approved by the engineers.

Mitigating steps for the cold weather concreting were phased in gradually as the ambient temperature dropped. Interim measures included: tarpaulins being used to cover the aggregate stockpile; floodlights being placed under
the tarpaulins to assist with the heating of the aggregates and laboratory and testing work.

Construction actions were implemented in accordance with the test results and the cold weather concreting specification. Aggregates and water were preheated, and it was ensured that a temperature of 30°C was maintained. To ensure that flash setting of the cement did not occur because of the heated water, the water and the aggregates were first placed in the mixer before the cement was added.

During placing, the temperature of the fresh concrete was monitored and recorded. The temperature target range of the fresh concrete was between 10 and 18°C. The ambient temperature was monitored to ensure that the required strength of 3,5 MPa would be achieved before the temperature dropped to 5°C. The aggregates and water were heated when the ambient temperature dropped below 5°C, but all concrete batching and pouring was terminated when the temperature dropped below -5°C.

The freshly cast concrete was covered with tarpaulins and thermal blankets, and hot air blowers were used when the temperatures dropped below 5°C to ensure that the required strength of 3,5 MPa would be achieved within the time frame of 7 hours.

The temperatures of the concrete at placing, the shutter temperature, the ambient temperature and the in situ concrete temperature were monitored to ensure that the construction parameters were met.

Stripping of formwork was determined by using the standard maturity calculations:

\[ \text{Maturity} = \sum (T + 10) \times t \]

where:

- \( T \) = ambient temperature
- \( t \) = time

After the formwork had been stripped, the concrete surfaces were covered with PVC plastic, and blowers and floodlights were used to maintain a heated covered area of 5°C for a minimum of 7 days. Electrically heated thermal blankets were used to cover all surface beds cast for a minimum of 18 hours.

In summary:

- 6 122 m³ of concrete was cast, of which 3 966 m³ was placed during winter
- 10 472 m² of shuttering was used
- 884 t of reinforcing and 2 632 m² of mesh were used

From the outset, throughout the conceptual phase during which quality control procedures were discussed and agreed on, all the way through to final mix design approval and contractor’s method statements, the team achieved their goals admirably despite the challenging climatic conditions. Today the Letseng mine is producing gem-quality diamonds – and very recently, a 500 carat stone.

**PROJECT TEAM**

- **BATEMAN CIVIL AND STRUCTURAL ENGINEERING**
  - Manager Civil & Structural Engineering Kurt Waelbers
  - Lead civil engineer George Bezuidenhout
  - Structural engineer Shoan Junkoon
  - Design engineer Vladimir Bortnik
  - Drawing office Manoli Coulentianos, Frik Goosen, Kay Pon, Steve Cousins, Colin Hales, Duduzile Kwasu & Riaan van Niekerk

- **STEFA NUTTI STOCKS CIVIL**
  - Contract director Shaun Butler
  - Contract manager Jacques Burger
  - Site agent Christo Brits
  - Sub-site agent Aubrey Tsipa

**COLLAPSBLE SOCCER DOME FOR NON-STOP SPORTING ACTION**

THE SHIELD FOR MEN Sportfan’s “Soccer Anytime, Anywhere, No Sweat” soccer contest will kick off in an arena unlike any other in the country. The uniquely structured Shield Dome will mean a smaller field, but a fast and furious game governed by rules that challenge soccer convention.

Competitors will pit their skills against one another in the portable and flexible dome, 11,5 m in diameter and 8 m in height, and weighing 1 600 kg. The dome will be transported from one venue to the next in an 8-ton truck mounted with a crane. Made out of steel, the structure is supported on 12 massive legs and takes a crew of eight people about eight to ten hours to assemble.

Nico Fourie of Stage Magic, who built the dome, says the games to take place in the dome will be contained by black nylon netting, which covers the entire structure. “The ball will stay inside the Shield Dome at all times and spectators will support the soccer players from outside the structure,” he says. The dome has been constructed to allow it to fit into unusual venues such as shopping centres and television studios, a compact and sleek engineering and architectural feat.

A segment of the roof has space for branding and the entire construction stands on temporary flooring. In addition, a scoreboard with two digital clocks has been created to accompany the dome and stands just outside the structure.

The Shield for Men event will take place throughout South Africa between May and September 2009. Each region will have two days of qualifying rounds and play-offs at different venues and one team from each region will go through to the national finals. R400 000 in prize-money for the winning team is up for grabs, so it is not a tournament to be taken lightly.

The soccer games will be non-stop action with the focus on freestyle soccer skills. New rules devised for the games mean no throw-ins, no offside, no goal kicks, just pure passing, kicking and scoring goals for 60 minutes. The teams that make it to the finals are therefore likely to be exceptionally fit and inventive in their play.

Shield will be keeping an eye open for outstanding players and teams, whose skills and talents will be showcased to the nation via a 13-part television series.
FAST-TRACK WARWICK TRIANGLE TRANSFORMATION FORGES AHEAD

SSI’S DURBAN OFFICE has been appointed managing consultant on the Warwick Triangle Viaduct Outbound project in Durban – a major public transport transfer node located at the end of the western transportation corridor. This forms the public transport gateway to the inner city from the inland side of the city and incorporates the entry and exit points into the city from the N3 Western Freeway.

The R200 million development is part of a greater initiative intended to transform the historic Warwick Triangle area from an increasingly congested hub, which poses a safety threat to commuters and pedestrians frequenting this area, into a world-class transport interchange.

SSI’s Durban office manager Brian Downie says the massive plan is being fast-tracked over a period of just 68 weeks to meet FIFA’s 20 May 2010 deadline for cessation of all road-overs. “Although this was generally regarded as being an extremely difficult deadline to meet, so far we are well on track,” says Brian.

“We approached the challenge of fast-tracking the project using a Design and Build tender, drawing heavily on experience gained during the Design and Build tender executed for the Johannesburg Roads Agency on the N17 Link Road. Based on our proposal, the eThekwini Municipality gave its approval in October 2008.”

A team headed by eThekwini’s Dave Thomas and SSI’s Brian Downie immediately mobilised several teams to plan and manage the project. Their first task was to advertise an Expression of Interest tender and to secure submissions by substantial contractors capable of meeting the challenge. This was achieved between 21 and 27 October 2008.

While contractors were being short-listed, foundation investigations were being undertaken, preliminary designs were under way for the geometric and structural elements of the project, a document based on the FIDIC (International Federation of Consulting Engineers) Plant Design-Build Contract was being written and the relocation of the utility services and taxi and bus operations was being planned.

By 28 November 2008 planning was sufficiently advanced for the presentation of the project to the selected contractors and their consulting teams. The tender document spelled out the conditions for the submission of tenders. A postulated design was supplied but the tenderers were encouraged to derive their own plans to suit the skills and resources available to each team. The results of detailed surveys were made available to the teams, together with a substantial volume of information on foundation conditions and utility services.

Despite the Christmas shutdown, all teams met the 28 January 2009 deadline for tenders. By 30 January basic assessment of the tenders was complete and contractor/consultant interviews began. Again, the teams were delegated the task of assessing each component of the tenders in detail. Detailed plans for the foundations and substructure were submitted and assessed, programmes were scrutinised and aesthetics and innovative elements were debated.

This process culminated in a presentation to the Tender Adjudication Committee on 4 February 2009, recommending that a tender submitted by Group 5 Pandev be accepted. This was agreed to and the Letter of Award was signed on 5 February, only minutes before the kick-off meeting and 11 days before the planned date for the start of construction.

Even as the award was being made, municipality staff were implementing the plan for relocation of taxi and bus services and erection of fences, clearing the site for construction activities. Brian paid tribute to the expertise provided by eThekwini Municipality Chief Structural Engineer Peter Fenton, ARQ Consulting Engineers for structural design and review, and Drennan Maud and Partners for the foundation investigation and design review.

VITRAFLEX CUBICLES AND VANITY SYSTEMS FOR 2010 SOCCER WORLD CUP STADIA

VITREX IS CURRENTLY undertaking the installation of hundreds of its popular Vitraflex toilet cubicles and vanity tops for the new stadia being built or refurbished for the 2010 FIFA World Cup.

Cristian Cottino, sales and marketing director of Vitrex, says the company is, firstly, supplying and installing over 1 200 toilet cubicles and more than 200 vanity tops for the upgrading of Soccer City in Johannesburg, which is being undertaken by Grinaker-LTA Interbeton Joint Venture. The stadium will host the opening ceremony and opening match, as well as the final of the prestigious tournament.

The enamel colour selected by the architects, Boogertman Urban Edge and Partners, for the Vitraflex elements was Graphite Grey.

“A new vanity system, the Vitraflex PB2010, was engineered by the Vitrex Technical Office in accordance with the specific design requirements developed by the architects for this project. Vitrex is also supplying over 800 vitreous enamelled cladding panels, in a light-gauge construction, through Burger Emoyeni. The Slate Grey panels are being installed as part of the screens at the top of the stands,” Cottino stated.

The company is, furthermore, supplying and installing over 650 Vitraflex toilet cubicles for the new Mbombela Stadium, currently being built by Basil Read just outside Nelspruit. The proven system was specified by R&L Architects of Cape Town and the cubicle stiles are fitted with Vitrex’s new stainless steel “telescopic” leg anchors. The design of the telescopic anchors not only provides for ease of installation and for adjustment of levels, but also eliminates conspicuous floor mountings and facilitates cleaning and improved hygiene through the reduction of potential bacterial growth at the base of the stiles. A wide range of enamel colours – some specifically formulated by Vitrex...
for this project – was selected for different ablation areas of the stadium.

For the extensions and refurbishment project at the Royal Bafokeng Sports Palace in Rustenburg, Vitrex has recently completed the installation of over 100 toilet cubicles, vanity tops and signage panels in accordance with BSP Architects’ specification. Vitrex had been involved with the original construction project of the stadium in the 1990s when a custom-made cubicle system was adopted. In the extension of the facilities, Vitraflex cubicles were ordered in different shades of blue, green, red and orange enamel, and they feature bespoke stainless steel and galvanised mild steel support elements instead of the standard stile components.

Vitreous enamelled wall-mounted directional signage panels were installed on the entry walls of the various toilets. The main contractor for this project was Liviero & Sons Building.

Finally, Vitrex is supplying 430 toilet cubicles for the new Green Point Stadium in Cape Town, through an order secured by their Western Cape agents, Façade Projects. Hygienic stainless steel floating leg anchors were specified by Stadium Architects (GMP, Louis Karol and Point, in association). The main contractor here is the Murray & Roberts–WBHO Joint Venture. The colour of the enamel is Singapore White, with the aluminium elements of the system finished in Traffic White.

Produced by fusing three layers of glass to sheet steel at temperatures over 800°C, Vitraflex cubicles and vanity tops are impervious to chemicals, spray paints, and bacterial or mould growth. Colours are permanent and the partitions are graffiti-proof and very easy to maintain. A standard range of 20 colours is available, with an almost unlimited choice on special request.

Becker Engineering’s Posi Lock range of gear and bearing pullers includes a new self-contained hydraulic system designed for pulling a wide range of press-fit parts, including bearings, gears, bushings, wheels and pulleys.

The Posi Lock PHS-108 hydraulic puller, which is a self-contained hydraulic pump and cylinder, ensures efficient performance, enhanced safety and reduced downtime in any maintenance application. These pullers, which can efficiently handle 12 tons of pulling force, have forged and heat-treated jaws for enhanced strength. A hand-pumping hydraulic action and flexible, swivelling, adjustable-length handle ensures easy operation.

“A unique feature of the lightweight and user-friendly Posi Lock range is a patented control cage jaw-retention system, with high-force hydraulic power for effortless pulling of large components. This safety cage holds the pulling jaws securely in either the open or closed position, preventing the jaws from slipping off the work surface,” says Eugene Davids, product manager for Becker Engineering, part of the Becker Group of Companies. “This system not only saves wear and tear on components removed, but also increases productivity and tool life and enhances safety for workers.”

Another advantage of these pullers is that they are operated easily, safely and efficiently by one man. Conventional pullers often require a team of workers to hold the jaws to prevent the puller from falling off or snapping back.

The Posi Lock PHS-108 sets include a self-contained pump/cylinder with a pump handle, a puller assembly with standard jaws (203 mm reach, 317 mm spread, 622 mm height), a pusher with a centering tip, a 48 mm pusher and two 74 mm pushers. Long jaws (249 mm reach, 381 mm spread, 660 mm height) and extra-long jaws (406 mm reach, 571 mm spread, 775 mm height) are also available.

The Posi Lock range, which consists of manual and hydraulic pullers, as well as adaptor sets and special tools and accessories, is available nationally from Becker Engineering and its distributor network. A full technical advisory and support service is also offered.
INSULCON’S KERAFIRE: A MUST FOR CONSTRUCTION

LONG KNOWN FOR its innovation in high-temperature solutions, Insulcon South Africa is set to launch KeraFire fire-prevention bricks, panels and spray, which could change the face of fire protection in the construction and allied industries.

Robert Martin, Director of Insulcon South Africa, says: “We are proud to announce this wholly South African product, which was designed and developed locally for any applications that require fire protection. These include office blocks, elevator shafts, electrical transformer rooms, electrical switchgear rooms, underground ‘safe areas’, computer server rooms and any other application where fire protection is required.”

Containing a fire is vital to the saving of lives and property in any situation; therefore, KeraFire is designed to protect the interior of a room from an external fire, as well as protecting the external environment if a fire erupts inside a room.

Martin notes that the product is supplied in the same standard sizing as normal building bricks, and in a wide variety of standard-sized building panels. The product has been designed for easy installation. However, Insulcon also offers a full consulting service aimed at determining, and then tailoring, the best fire-protection solution for customers.

The guniting or spray application, whereby KeraFire is applied to existing walls, cable racks, roofing units and steel structures, will either be done by Insulcon’s fully trained crew, or training will be offered to licensed installers.

Although it is always difficult to determine how much market share a new product will capture, Martin says Insulcon South Africa is looking at “around 10 to 20%”. Given the company’s record with regard to high-temperature solutions and safety, a greater market share will not be surprising. The core market for KeraFire includes the building and construction industry, electrical installation installers and fire-protection contractors.

“The product is supplied in standard formats that are already accepted by the building industry and therefore structural design changes are not necessary. KeraFire products can simply be used as a substitute for non-preventive products, with the customer gaining the benefits immediately on application,” says Martin. Other benefits to clients include:

- Lightweight, when compared with normal building products
- Completely inorganic
- Excellent insulating properties, offering thermal insulation under normal operating conditions, as well as fire protection

A global leader in high-temperature solutions, the Insulcon Group is a privately owned service-oriented company specialising in heat-management solutions that save energy and increase the control and reliability of its clients’ processes. Insulcon products and engineered systems can be used in almost any thermal process, even when exceptionally high temperatures are involved. With its fire-prevention properties, KeraFire now brings a new aspect to heat management.

CMA RE-ISSUES CONCRETE BLOCK PAVING MANUAL

THE CONCRETE MANUFACTURERS Association (CMA) has republished Book One of its series of concrete block paving (CBP) manuals. Aimed at landscape architects, engineers and paving contractors, the manual illustrates the versatility of segmented concrete paving as a continuous hard-wearing and aesthetically appealing surface.

The manual makes extensive use of photographs and shows examples of how concrete paving has been successfully used for roads, commercial projects, industrial areas and domestic paving, as well as in specialised applications, such as the cladding of vertical surfaces, stormwater channels, embankment protection and roof decks.
carried out into the engineering characteristics and structural performance of segmental block paving,” he says.

Existing pavements that are subjected to heavy bus traffic and industrial loads have been monitored and their service life has been shown to be satisfactory. The South African Bureau of Standards (SABS) has published specifications relating to the quality of CBP and the attendant standards of construction. In addition, the Committee of Urban Transport Authorities has published a catalogue of designs for segmental block pavements.

According to Cairns the engineering and specification aspects of segmental block paving have been satisfactorily solved and the system has a proven performance and service record. However, the aesthetic use of segmental paving and the contribution it can make to improving our urban environment is only beginning to be appreciated.

BUILDING THE CONSTRUCTION INDUSTRY

ACCORDING TO THE Gauteng Treasury, the construction industry has been characterised by declining investment levels since reaching a peak in the fourth quarter of 2007, with the electricity crisis, the slowdown of the economic growth rate in South Africa and the ever-growing realisation of the impact of a financial recession all contributing to the decline. However, despite this slump, the construction sector has managed to keep its head above water, recording a positive 10.8%* quarter-on-quarter growth rate in the fourth quarter of 2008.

Such growth in the local engineering, civil and construction markets continues specifically because of the reinvestment in infrastructure, given the upcoming 2010 FIFA World Cup. In fact, government has allocated R787 billion in funds for infrastructure. However, despite such positive stimulation, there are significant industry challenges that are yet to be overcome – of which the skills crisis is one.

There has always been a shortage in construction skills which threatens projects worldwide, restricts growth and drives up costs, but the effects of such a shortage are only now being felt with 2010 looming. KPMG’s 2008 Construction Survey** found that the global skills shortage and industry’s image problem are combined with the challenges created by failing risk management, escalating costs, tough new environmental standards and the global issue of sustainability. Furthermore, 84% of contractors responding to the survey believed that the industry was not doing enough to tackle the chronic skills shortage, with 55% indicating that the problem was critical.

And critical it certainly is as the lack of project management and operational skills results in poor site management, delays, cost overruns and unfilled promises – all detrimental to the industry. Although South Africa is taking positive steps to attract skilled resources back to the country, the image of the industry needs to be improved and construction needs to be marketed as a career path, not a job opportunity. Construction companies therefore need to develop a fresh approach to recruiting to attract the best graduates in a very challenging global environment. In order to do so, three elements are key. Firstly, the private sector needs to get involved in “selling” and promoting the correct image and the positive aspects of the industry, not only at university level, but also at school level to build up the industry in the light of 2010 and to ensure that graduates are entering the sector long after the World Cup has ended. Secondly, it is important to leverage the work already put into skills honing by discussing industry issues with relevant players and the candidates – existing or potential – to drive debate and possible solutions. Lastly, for players in the industry, it is critical to have a staffing partner that intrinsically understands this industry’s needs, dynamics and possibilities to ensure that available opportunities are strategically maximised for economic, social and structural growth within the local engineering community.

By profiling and highlighting the abundant prospects and dynamic career projects that the sector presents, continuing to develop skills and partnering with a strategic recruitment agency that understands the industry, its impact and opportunities, the sector will surely benefit, and the longevity of the construction landscape will be demonstrated well beyond the hype of 2010.

INFO

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** KPMG Global Construction Survey, Embracing Change, August 2008
Communication from the SAICE Membership Committee

Meet your committee members

In our previous article (Civil Engineering May 2009, pages 74 and 75) we discussed the various membership categories. This time round we are going to briefly introduce you to the current Membership Committee and also respond to two queries.

**Chairman – Athol Schwarz.** Athol was involved with SAICET (SA Institute of Civil Engineering Technicians and Technologists) from 1990, and with SAICE since the merger between the two institutions in 1994. He is currently employed by Hatch Africa Pty (Ltd) as Design Technologist on the Saldanha Iron Ore Expansion Project. Athol is married to Mariana, and has two daughters, Tammy and Celesté. His spare time is spent on DIY projects around the house.

**Vice-chairperson – Zukiswa Mvoko.** Zukie joined SAICE in 1997 and was appointed to the Membership Committee in 2003. She is currently working for Allyson Lawless and Associates as a Civil Engineering Technician. Zukie enjoys aerobics and reading.

**Johan De Koker.** Johan was involved with SAICET from around 1980 until its merger with SAICE in 1994. He served as president of SAICE during 2008. He was also a member of the ECSA Council from 2001 to 2005. Johan is head of the Department Civil Engineering Technology at the Doornfontein campus of the University of Johannesburg. He is married to Verelene, and they have two children, Estelle and Nico.

**Seetella Makhetha.** Seetella is the past chairperson of the Membership Committee and is currently one of the Vice-Presidents of SAICE. Seetella and her wife, Martha (also a member of SAICE), own Makhetha Development Consultants. Seetella is also a director of Jeffares & Green. Seetella and Martha have two children, Matseliso and Motheba. He is a member of the Algoa Flying Club in Port Elizabeth and flies aeroplanes in his spare time.

**Tony Murray.** Tony has spent most of his career in local government in Cape Town. He serves on the SAICE Executive Board and is chairman of the History and Heritage Panel. Although retired, Tony remains extremely active in many areas of the civil engineering profession. He is married to Libby and they have three children, Bruce, Janet and Alice. Tony enjoys the outdoors, sport and reading.

**Huibrecht Kop.** Huibrecht graduated from the University of Pretoria in 1980 and was employed by the then South African Railways and Harbours. He joined SAICE and in 1990 became involved with the Railway and Harbour Engineering Division, where he served as treasurer and later as chairman. In 2004 Huibrecht was elected to the SAICE Council and was appointed to the Membership Committee and the Editorial Panel.

(Sadly Huibrecht passed away unexpectedly on 23 July. He will be fondly remembered by both the Membership Committee and the Editorial Panel. Our sincerest condolences go to his wife Lizette and daughter Zaané. Ed.)

**Geoff Roberts.** Geoff, a director of Aurecon, has served on the Membership Committee since 2005, and has also been active in the Algoa Branch for the last ten years. Geoff was president of SARF (1994/1996) and chairman of the SARF Eastern Cape region for about 7 years. He is married to Donné and has two children, Sarah and Jonathan. Geoff enjoys the outdoors and is a keen runner having completed a number of Comrades and Two Oceans marathons.

**Maxwell Vavana.** Maxwell was appointed to the Membership Committee in 2007. He is currently employed by Jeffares & Green Consulting as Geometric Design Technician.
Denver Siebritz (SAICE National Office staff – Operations Manager). Denver has been involved with the Membership Committee since 2007. He is married to Elizabeth and has two daughters, Gabriella and Taylor-Leigh. In his spare time he enjoys a good round of golf.

Memory Scheepers (SAICE National Office staff – Manager Administration). Memory has been involved with the Membership Committee since 1996. She is married to Evert and has two children, Simeon and Anika. Memory enjoys walking and reading.

Norma Byleveld (SAICE National Office staff – Membership Officer). Norma handles all the membership queries, and has been involved with the membership committee since 2006. She has a son, Rudi, and enjoys aerobics, swimming, cooking, painting, music and reading.

QUERIES

Over the last month I have received queries regarding (1) upgrading of membership and (2) the divisions within SAICE.

Upgrading

Upgrading from one membership category to another is the responsibility of each individual member. The process is very simple - either log-on to the SAICE website (www.civils.org.za), navigate to the ‘Membership’ tab on the top bar and download the ‘Membership Application Forms’. Alternatively, contact Norma Byleveld on 011 805 5947/8. Please note that the same form is used for admission and upgrading. Return the completed form to Private Bag X200, Halfway House, 1685. Once your application has been processed, you will be notified of the outcome.

Divisions

Over and above the twenty local branches, SAICE has nine specialised divisions in which our members can become involved. Although most of these divisions are based in the Gauteng area, you may be a corresponding member, and if you can get more members involved locally, nothing stops you from establishing a sub-division in your area. This is how we will grow SAICE. The nine divisions are:

- Transportation Engineering
- Geotechnical Engineering
- Railway & Harbour Engineering
- Water Engineering
- Joint Structural Division
- Joint Civils Division
- Environmental Engineering
- Information Technology
- Project Management

More information and contact details for each of these divisions are available on the SAICE website (www.civils.org.za). Navigate to the ‘SAICE House’ tab on the top bar and open ‘SAICE Divisions’.

CURRENT MEMBERSHIP STATISTICS: 8 805 members

IN CLOSING

Should you have any questions, or issues that you would like the Membership Committee to address, please e-mail these to me at the following address: aschwarz@hatch.co.za
THE CEMENT & CONCRETE Institute (C&CI) has published the ninth edition of *Fulton's Concrete Technology*, South Africa’s most respected and authoritative book on the subject.

The latest edition provides updated information on materials and other aspects of concrete technology. The book is aimed at students and practitioners and, according to C&CI managing director, Bryan Perrie, “consolidates the experience and insights of experts in the field in a single volume relevant to concrete technology in South Africa.”

Speaking at the launch of the book in Midrand recently, Bryan said that since its first publication in 1957, the book has proved to be very popular. In this new edition some chapters have been extensively revised to reflect new developments, with new authors included to augment or replace the input of previous authors who had retired since the last revision of Fulton’s in 2001. “Taking cognisance of the expressed needs of readers of previous editions and to include the latest research, five completely new chapters have been included, namely: repair, reinforcement, formwork, sand-cement mixes, and self-compacting concrete,” he stated.

Edited by former C&CI employee, Gill Owens, the handbook’s contents are divided into five broad categories:
- Materials for concrete
- Properties of fresh concrete
- Properties of hardened concrete
- Production of concrete
- Special techniques and applications

**MORE INFORMATION**
C&CI Information Centre
011 315 0300

**ORDERS**
C&CI: Harry Mabaso
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SAICE: Angelene Aylward
aaylward@saice.org.za

Bryan Perrie, MD of C&CI, making a special presentation to Gill Owens, editor of the ninth edition of *Fulton's Concrete Technology*, at the launch in Midrand recently.
Date | Event and CPD validation number | Presenters | Contact details
--- | --- | --- | ---
4-10 July Cape Town 15-21 August Durban | The Application of Finite Element Method in Practice SAICEstr06/00018/08 | Roland Prukl | Dawn Hermanus dharmanus@saice.org.za
16 July Midrand | Essential I.T. Knowledge for Business Executives S AICEIt08/00345/11 | Dr James Robertson | Sharon Mugeri cpd.sharon@saice.org.za
20 July Port Elizabeth 24 August Cape Town | Ridding Stormwater of Litter SAICEwat08/00361/11 | Prof Neil Armitage | Sharon Mugeri cpd.sharon@saice.org.za
22-23 July Midrand 11-12 August Durban | Basics of Track Engineering SAICErail09/00404/11 | Ed Elton | Dawn Hermanus dharmanus@saice.org.za
27-28 July Bloemfontein | Handling Projects in a Consulting Engineer’s Practice SAICEproj08/00404/11 | Wolf Weidemann | Dawn Hermanus dharmanus@saice.org.za
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
28 July Port Elizabeth 22 September East London | Reinforced Concrete Design to SANS 10100-1 SAICEstr09/00432/11 | Greg Parrott | Sharon Mugeri cpd.sharon@saice.org.za
First Session: 28 -30 July Johannesburg Second Session: 9-10 September Johannesburg | Practical Geometric Design SAICE/TR07/00139/09 | Tom McKune | Dawn Hermanus dharmanus@saice.org.za
30-31 July Bloemfontein | Business Finances for Built Environment Professionals SAICEfin08/00405/11 | Wolf Weidemann | Dawn Hermanus dharmanus@saice.org.za
08-09 September Johannesburg 15-16 September East London | Project Management and MS Projects Hybrid Course | Les Wiggill | Dawn Hermanus dharmanus@saice.org.za
16-17 September | Technical Report Writing SAICEbus09/00427/12 | Les Wiggill | Dawn Hermanus dharmanus@saice.org.za
24-25 August Johannesburg 01-02 September Durban 08-09 September Port Elizabeth 15-16 September Cape Town | Coastal Engineering and Management | Keith Mackie | Sharon Mugeri cpd.sharon@saice.org.za
30 September Midrand | Bridge Maintenance SAICErail09/00495/12 | Ed Elton | Dawn Hermanus dharmanus@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

REMINDER

SAPPMA CONFERENCE
(Southern African Plastic Pipe Manufacturers Association)
WEDNESDAY 9 SEPTEMBER 2009
BYTES CONFERENCE CENTRE, MIDRAND
PLASTIC PIPE SYSTEMS – THE QUALITY SOLUTION
Contact: Louise Müller – 082 417 2977 – admin@sappma.co.za