\[ F = \frac{1}{\Sigma W \sin \alpha} \left[ \frac{c' B + (W - uB) \tan \phi'}{\cos \alpha + (\sin \alpha \tan \phi' / F)} \right] \]

\[ k_x \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0 \]
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Effectiveness of economic incentives on clients’ participation in health and safety programmes

I Musonda, J H C Pretorius

The use of economic incentives to improve health and safety (H&S) performance in the construction industry in general has been investigated by various scholars. However, few studies have focused on the impact of economic incentives on construction clients, especially in the developing world. This paper reports findings on the feasibility of incentives to influence construction clients to perform on H&S.

The investigation was conducted using a Delphi technique to determine the impact significance of economic incentives or disincentives on construction clients’ H&S performance. A panel of experts in construction H&S was assembled from most regions of the world. A three-round iterative Delphi study aimed at establishing consensus from the expert panel was then conducted.

The study found that economic factors have critical impact significance on clients’ H&S performance, and that clients were ‘very likely’ to implement various H&S elements as a result of the economic incentives and or disincentives.

There is little research on the use of incentives to influence construction clients’ H&S performance and the effectiveness of such incentives. This gap in literature, and the need for improvement in construction H&S performance, motivated the current study. Therefore it was necessary to investigate specifically the extent to which economic incentives could be used to influence construction clients to become involved in H&S programmes. Economic incentives are considered to be a proactive way of improving H&S performance among other parties, such as employees in the construction industry.

The paper reports on the findings from an analysis of the impact significance of economic incentives on clients. It underscores the point that economic incentives or disincentives for construction clients are necessary to encourage them to actively participate in H&S programmes, hence resulting in performance improvement.

INTRODUCTION

Construction clients lack the motivation to actively participate in health and safety (H&S) programmes in the construction industry. Anecdotal views seem to indicate that the possible reasons for their lack of participation may include the view that clients do not suffer loss directly in the event of an accident. A question arises therefore as to what would motivate clients to actively participate in H&S programmes. Economic incentives have been reported to produce favourable results with other H&S stakeholders. However, there is no reported evidence on the effectiveness of economic incentives on clients. Therefore it was necessary in the current study to investigate the effectiveness of economic incentives to influence clients to actively participate in H&S programmes and hence improve H&S performance.

Literature informs that the use of incentives as a method to promote a culture within which technical and process innovation can flourish is critical to project success (Tang et al 2008). Similarly, economic incentives have been shown to yield positive results in H&S performance (European Agency for Safety & Health at Work 2010). It is in view of this that Elsler and Nikov (2003) contend that there is a need for economic incentives to proactively promote H&S.

Some of the reasons why economic incentives have been contemplated include the failure of strict regulatory approaches, the costs involved in bringing organisations to courts for non-compliance and the low level of fines which have failed to encourage organisations to comply (Elsler & Nikov 2003). However, it is also acknowledged that economic incentives are only effective when they are directed at organisational or national level (European Agency for Safety & Health at Work 2010). Consequently, the economic incentives may entail linking...
fiscal incentives, such as lower accident insurance premiums or tax rates, to a good H&S performance for an organisation. Other methods to incentivise, for example, employers to implement H&S, may include match-up funds where a grant is given to an employer equal in amount to the amount to be spent on H&S, or linking an incentive amount to a voluntary audit or inspection (European Agency for Safety & Health at Work 2010).

In order to achieve the desired goals from the economic incentives, their design and use should take into account the constraints and risks of a project, organisation or indeed the nation. Incentives should make risk allocation fairer, because incentives can be seen as the sharing of rewards for good performance, and this may motivate the participants to perform better (Tang et al. 2008).

The reason why economic incentives are said to work on the contractors’ side, or are seen as one of the solutions to proactively improve H&S, is partly because of the cost of ensuring H&S, which is usually borne by the contractors (Elsler & Nikov 2003). Contractors work at reducing the cost in order for them to remain competitive. Bishop et al. (2009) rightly argue that the unacceptability of occupational H&S performance of the building and construction industry can be attributed to the powerful competitive forces in the industry, which ultimately work against H&S. He observed that the industry strives to complete projects on time in order to reduce costs, and too often H&S is neglected. The solution may be a cultural and behavioural change, and this may only come about by harnessing the competitive forces in the industry to work for occupational H&S.

Both organisations and government departments at times lack the requisite resources, and this inhibits a meaningful improvement of H&S. A lack of resources or underfunding for H&S programmes limits any action. For example, in Tanzania less than 1% of the Labour Department’s budget was allocated to occupational H&S (Kamuzora 2006). This kind of allocation can result in a low capacity to enforce legislation, and failure to conduct inspection and surveillance. According to Cotton et al. (2005), contractors, or indeed other stakeholders, are unlikely to see the need of implementing H&S without the application of incentives or sanctions, especially in developing countries.

The benefits of incentives are clear. The European Agency for Safety & Health and Work (2010) demonstrated from a case study of six organisations in Europe that improvements of 25% to 70% were possible with economic incentives. However, for the incentives to be effective, they should be provided by national and/or international organisations. Consequently political will is necessary for the national or international organisations to be involved.

The use of economic incentives to improve H&S performance in the construction industry in general has been investigated by various scholars. However, few studies have looked at the impact of economic incentives for construction clients, especially in the developing world. Therefore it was necessary to investigate specifically the impact of the economic incentive on clients’ H&S performance. Studies have shown that economic incentives have produced positive results for contractors and employees. However, it is not clear how economic incentives would impact the H&S performance of clients. The focus was therefore placed on clients, because they can influence project H&S performance (Huang & Hinze 2006).

THE STUDY

A Delphi study technique was used to explore the impact significance of economic incentives on clients’ H&S performance. The Delphi method was preferred to common survey methods, as the current study was addressing the ‘what could happen if’ kind of question as opposed to the ‘what is’ kind of question (Hsu & Sandford 2007). The Delphi method was also considered to be much stronger for its rigorous query of experts, which is achieved through many iterations and feedback.

The Delphi study involved 11 panel members. This number of panelists was considered adequate based on what other Delphi studies have used and recommended. Delbecq et al. (1975) suggest that 10 to 15 panelists could be sufficient if the background of the panelists is homogenous. A review by Rowe and Wright (1999) indicates that the size of a Delphi panel has ranged from three to 80 in peer-reviewed studies. Okoli and Pawlowski (2004), and Skulmoski et al. (2007) also mention a panel size of about 10 to 18 members. Hallowell and Gambatese (2010) suggest a minimum of eight panelists. Based on the above, and the fact that the Delphi method does not depend on statistical power (Okoli & Pawlowski 2004), but rather on group dynamics for arriving at consensus among experts, a panel of 11 members was considered adequate.

However, the choice of panel members was critical. Delphi is a group-decision mechanism requiring qualified experts who have deep understanding of the issues (Okoli & Pawlowski 2004). Therefore one of the most critical requirements is the selection of qualified experts, as it is the most important step in the entire Delphi process because it directly relates to the quality of the results generated (Hsu & Sandford 2007). In view of the above, successful panel members had to meet a set of criteria which included qualification, experience, publication record, and capacity and willingness to participate in the study.

Panel members were identified from three sources. The first source was the CIB W099 register of members located on the CIB W099 website (CIB 2010). The CIB W099 is a working commission that was set up by royal appointment to enable researchers on construction H&S in the world to collaborate and protect H&S. The second source was the conference proceedings of the CIB W099 from 2005 to 2009. Individuals who had frequently appeared as authors or keynote speakers were identified as potential experts for the study. The third and last source was identifying, through references, individuals working in the area of H&S in the local construction industry in southern Africa.

The panel consisted of two members from South Africa, three each from the United States of America (USA) and the United Kingdom (UK), and one each from Singapore, Hong Kong and Sweden. All the panelists specialised in construction safety. In terms of their current occupation, three of the panelists were employed by contracting organisations, two by consulting organisations, and six by universities. All panelists held very senior positions in their organisations and were involved in community service. The panel had a cumulative of 243 years of experience. The lowest number of years of experience was seven and the highest 45.

The calculated mode of years of experience was 15, the mean 22.1, and the median 15 years. Experience was an important factor in determining who an expert was, and therefore the minimum number of years was set at five. In terms of publications, ten of the panelists had published in peer-reviewed journals, conference proceedings and books. Between them, they had published 57 books and monographs, 19 chapters in books, 187 peer-reviewed academic journals, 345 recent conference papers and 341 other publications comprising articles in professional journals, technical reports, policy papers, expert witness documentation and keynote addresses (Table 1). In addition to their publications, the panel had
led and managed 108 funded research projects. Three panellists served on the editorial boards of 43 peer-reviewed journals and conference proceedings (Figure 1).

The Delphi study involved three rounds of an iterative process before consensus between the panel members on the impact significance of economic incentives on clients' H&S performance was reached. Panellists were requested to rate the probability that clients would implement H&S elements as a result of influence from external environmental factors, including economic incentives. The probability scale ranged from 1 to 10 representing 0 to 100%. Further, panellists were requested to rate the impact of external environmental factors on client performance. The impact scale was based on a 10-point rating scale ranging from low to critical. This aspect indicated the severity of a factor.

A two-stage analysis of data from the Delphi was conducted using Microsoft Office Excel, which is a spreadsheet software programme. The first stage involved analysis to establish or confirm consensus on responses to the predetermined criteria. This involved determining the group median responses for each question. After the third round of the Delphi, absolute deviations (\(D_i\)) about the group medians (\(m(X)\)) of each rating for every question were calculated using Equation 1. In addition, mean absolute deviations (MAD) were calculated for every question. This is a calculated mean of all absolute deviations for all panellists about the median on each question. Further analysis involved determining the statistical range in ratings by panellists on each question, and the percentage of panellists with a similar opinion inclination on each and every question. Consensus was determined to have been achieved when the MAD was less than one unit below or above the group median, the range in ratings on each question between all panellists was below 4.0 and the percentage of panellists that were of a similar inclination in opinion were 60% and above on a particular question.

\[
D_i = |x_i - m(X)|
\]

Where:
- \(D_i\) = Absolute deviation
- \(x_i\) = Panellist rating
- \(m(X)\) = Measure of central tendency

The second stage of Delphi data analysis involved determining the impact significance of environmental factors on client H&S performance. The significance of the impact of environmental factors was categorised as critical, major, moderate, minor or low. The categorisation was helpful in determining which environmental factor was more critical to client H&S performance. The impact significance of a factor was obtained as a product of the overall rated probability (likelihood) that an environmental factor would influence clients to implement H&S elements, and the rated negative impact (severity) on clients' implementing the elements that would result if the environmental factor was absent. This relationship is illustrated in Equation 2.

\[
\text{Impact Significance} = \text{Likelihood} \times \text{Severity}
\]

**Figure 1 Publications by panel members**

**Figure 2 Impact significance of external environmental factors to client culture**

**Table 1 Panellists’ publications**

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<th>Panel publications</th>
<th>No of publications</th>
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<tr>
<td>Books and monographs</td>
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<td>Chapters in books</td>
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<td>Peer-reviewed journals</td>
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<td>Referee for conference proceedings</td>
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RESEARCH QUESTIONS

Broadly, the study constituted two research questions relative to the influence of economic incentives on client H&S performance. In the first question, the experts were requested to provide a rating, and reach consensus, on how likely (probable) clients would be to implement the following H&S-related elements if economic incentives were to be in place:

- Provide finance for H&S implementation
- Appoint an H&S agent
- Provide H&S staff
- Choose a procurement method suitable for H&S
- Become involved in design review
- Conduct H&S inspections and audits
■ Implement H&S policies, procedures and goals, and
■ Provide H&S leadership.

The second research question required the panel of experts to rate and reach consensus on what they determined to be the impact on clients’ H&S culture in the absence of economic incentives for clients to implement H&S. This particular question sought to establish whether the client H&S culture would be better with an application of economic incentives for clients to implement H&S. Findings to the questions raised in the study are presented in the next section (Results) and discussed later in order to inform the conclusions and recommendations.

RESULTS

The influence of external environmental factors on client H&S performance was evaluated. The external environment was defined by six factors, namely: political, social, economic, legislative, professional bodies and technology. The impact significance of these factors’ influence on client H&S performance was obtained as a product of clients’ likelihood to implement H&S elements, and the severity rating or negative impact on clients’ H&S performance if the factors were absent.

The level of influence was determined by assessing the extent to which a client would implement various H&S elements if pressured by the external environment. The severity of an environmental factor was the rated negative effect on client H&S performance that would result from the absence of an environmental factor. The severity rating was based on an ordinal scale of 0 to 10, with 0 being negligible and 10 critical. The impact significance was obtained as a product of the severity rating of an environmental factor and the likelihood of the client implementing a particular H&S element (refer to Equation 2).

Of the six environmental factors, three, namely political, economic and legislative, were determined to have an impact significance of over 5.0. The economic and legislative factors had an impact significance of 2.77 each (Figure 2). According to the classification scale used in this study a rating of 2.77 was considered to be ‘critical’. The rating suggested that an economic incentive was critical to a client implementing the required H&S elements or programmes.

The likelihood of clients implementing H&S elements as a result of the external environment’s influence was 67% on average (Figure 3). The standard deviation in the likelihood ratings was 0.06. The small standard deviation suggested that the likelihood of

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With economic incentives, clients were ‘very likely’ to implement all the H&S elements (Figure 4). In comparison, the average likelihood that the client would implement H&S elements as a result of the influence of political, legislation, social, technological and professional bodies was 63% (Figure 4).

DISCUSSION
Findings from the study indicated that economic incentives had a critical impact significance on client H&S performance, and that clients were very likely to implement H&S elements with economic incentives when compared to other factors such as political influence. These findings collaborate with that of the European Agency for Safety and Health at Work (2010). The European Agency for Safety and Health at Work demonstrated from a case study of six organisations in Europe that improvements of about 25% to 70% were possible with economic incentives. It is envisioned therefore that similar results may be obtained when clients are incentivised, considering that the study found that clients were very likely to implement H&S elements (above 80% likelihood) if economic incentives were present.

Economic incentives may be effective to motivate clients to participate in H&S management. Without economic incentives, clients may continue to consider themselves not critical H&S stakeholders, and might therefore not participate effectively in implementing H&S. According to Cotton et al (2005), stakeholders are unlikely to see the need of implementing H&S without the application of incentives or sanctions, especially in developing countries.

The current study found that economic incentives had almost the same impact on H&S performance as legislation. However, it has been observed that using legislation alone to influence clients to implement H&S elements may not achieve the desired results. Using legislation alone is problematic, because in most parts of the world the ineffectiveness is not necessarily with the legislative regime that is in place, but with its enforcement. In the United Kingdom, the House of Commons noted that, although breaches of H&S regulations are serious criminal offences and legislation provides for penalties, courts have tended not to impose the maximum penalties available (House of Commons 2004). In South Africa the problem of enforcement was also identified, describing it to be inadequate (CIDB 2008). A need therefore arose to establish other forms of incentives in addition to legislation.

Legislation can facilitate the implementation of economic incentives for clients. The role that the legislative environment can play to facilitate economic incentives on both public and private clients is to have the requirement enshrined in the laws. It is easy to allocate finance if there is pressure from regulations, for example by demanding that financial allocation for H&S should be specified in the proposal. That would enable the client to ensure that finance is actually allowed for this purpose, and consequently a contractor would not be able to question client demands when the client is only complying with legislative requirements (Törner & Pousette 2009). It is clear that, for economic incentives to work, the regulatory framework has to be in place that specifically addresses its implementation. According to the European Agency for Safety and Health at Work (2010), regulation and economic incentives are complementary. Therefore, in order to effect economic incentives practically, appropriate regulation must be in place.

Examples of the measures that regulation can address include the requirement for clients to be equally responsible for direct accident costs. It is argued that in this way clients will become aware and have direct exposure to accident costs, hence serving as an incentive to ensure that measures to a better H&S are in place to avoid similar future occurrences and loss.

Other examples of economic incentives include, for instance, the introduction of H&S performance bonds/guarantees which can be taken in both clients’ and contractors’ names. A facility of this nature would place a practical and real economic incentive on the clients, and indeed on the contractors. The idea of a performance bond has worked well with contractors to ensure performance on project delivery (Meng 2002). A performance bond is an agreement between the client, the contractor and a third party who, in most cases, is a bank or insurance entity (Supardi et al 2011). In the agreement, the bank or insurance institution agrees to pay a sum of money to the client in the event of non-performance of the contract by the contractor (Abdul-Rashid 2004). The principle is that the contractor in this instance commits himself/herself to the client to perform the contract to the client’s satisfaction (Ndekugri 1999).

An H&S performance bond/guarantee, taken jointly by the client and contractor in favour of the state or enforcing agency, would cause a paradigm shift in the management of H&S in the construction industry. Since clients will have a real economic incentive to ensure a good H&S performance, they will insist on engaging a contractor with a good H&S record, and will further insist on employing professional engineers who will safeguard their interests.

Professional bodies, such as engineering institutions, also have a role to play in making economic incentives practicable. Professional bodies sit on boards to draw up and or review standard contract forms which are used in the construction industry as contract documents between clients and contracting entities. Economic incentives such as the H&S performance guarantee can be incorporated in the standard forms and hence make it a contractual obligation. Engineers, who in most cases are designers, have influence on how H&S is managed (Smallwood 2004).

Therefore appropriate economic incentives can work for clients, just as they have for contractors and individual workers in the construction industry (Elsler & Nikov 2003; Goodrum & Gangwar 2004).

CONCLUSION AND RECOMMENDATIONS
It can therefore be concluded, based on the findings in this study, that economic incentives should not be overlooked in trying to get clients involved and becoming accountable for H&S implementation. With economic incentives, clients are likely to implement all H&S elements on a project, and are also likely to assume leadership in H&S and put H&S programmes in place.

It has also been concluded that economic incentives will have a significant impact on client H&S performance. Therefore, when a number of measures are considered to improve H&S standards in the construction industry by influencing client H&S performance, economic incentives may not be overlooked. The only other factor with similar impact significance was found to be legislation. Although political, social, technology and professional bodies have influence on clients’ H&S performance, economic incentives were found to have a more significant impact.

While the findings in the current study suggest that economic incentives have a significant impact on client H&S performance, it does not, however, suggest that other factors, such as legislation, political, social and technology do not matter. On the contrary, the findings suggest that economic incentives may not be omitted from a list of other factors that also need to be considered and applied in order to motivate clients so that they might be effectively involved and become accountable for H&S management in the construction industry.

The current study recommends therefore that ways to incentivise clients be explored, including the prospect of implementing the H&S performance guarantee. Although...
regulations, such as the 2014 Construction Regulations in South Africa impose far-reaching requirements on clients, research informs that, due to low fines and lack of enforcement, the desired benefits may not be realised if the complementary role of financial incentive is not exploited.

REFERENCES


Development of a saturation- and stress-dependent chord modulus model for unbound granular material

E van Aswegen, W J vd M Steyn, H L Theyse

Unbound granular material is used in the pavement structure and usually comprises the bulk of the structural and foundation layers of a typical South African pavement. The term unbound granular material refers to the classification of natural material, which has not been modified in any way. Unbound granular material is classified from a G1 to G10 quality according to its fundamental behaviour and strength characteristics.

Young's modulus and Poisson's ratio are theoretical concepts of linear elasticity that can at best approximate experimental results of actual material elastic response. In their basic linear elastic form, Young's modulus and Poisson's ratio are rather poor approximations of actual unbound granular material behaviour. The non-linear, stress-dependent behaviour of unbound granular material can, however, be simulated using the linear elastic model as a basis, but with a proper constitutive material model that adheres to the observed material behaviour.

The objective of this paper is to utilise a chord modulus model and calibrate it for a range of unbound granular material classifications. The model was calibrated for five bulk material samples, ranging from G2 to G8. The calibration process included linking variables of the model to mathematical functions that approximate the trends observed when variables were considered against degree of saturation. A parametric analysis indicated that the saturation- and stress-dependent chord modulus model realistically predict material behaviour. The trends depict the stress-dependent behaviour of unbound granular material, where an increase in initial modulus is observed for increasing confinement pressure, as well as initial stress softening with increasing stress ratio followed by stress stiffening.

It can be concluded from the results presented in this paper that a saturation- and stress-dependent chord modulus model could be refined and calibrated for crushed and natural unbound granular material. This refinement did not negatively influence the accuracy or ability to realistically predict the material behaviour. The preliminary conclusions reported in this paper indicate that the chord model formulation yield satisfactory predictions, especially when the model is calibrated for each individual material type.

INTRODUCTION

Unbound granular material is used in the pavement structure and usually comprises the bulk of the structural and foundation layers of a typical South African pavement. The term unbound granular material refers to the classification of natural material, which has not been modified in any way. Unbound granular material is classified from a G1 to G10 quality according to its fundamental behaviour and strength characteristics (DoT 1996; DoT 1985; Theyse et al 1996).

A G1 quality material is defined as a graded crushed stone, usually obtained from crushing solid un-weathered quarried or mined rock or boulders. G2 and G3 quality material are obtained by the same process as a G1 quality material, but may contain natural fines not derived from crushing the parent rock. Medium quality materials (G4, G5 and G6) are defined by the TRH 14 (DoT 1985) as natural gravel or a mixture of natural gravel and boulders which may require crushing. Any of these materials may be modified using cement, lime, bitumen or polymers to enhance certain strength characteristics of the material. Lower quality materials (G7, G8, G9 and G10) are defined as gravel-soil in TRH 14 (DoT 1985).

In this paper the term crushed stone or crushed aggregate will be used to refer to G1 to G3 quality material and natural material will refer to G4 to G10 quality material. Unbound granular material will refer to both crushed stone and natural material (i.e. G1 to G10).

Pavement structural layers (wearing course, base and sub-base) are generally...

Keywords: resilient behaviour, model, unbound granular material, degree of saturation


TECHNICAL PAPER

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subjected to higher traffic-induced shear stresses than the pavement foundation layers, and also larger plastic strains. When the wearing course is compromised, it leads to strength and bearing capacity deterioration of the structural pavement layers as moisture ingress takes place.

The type and volume of traffic the pavement structure must carry during its design life dictate which material behaviour and strengths are required. The accurate modelling of the response of pavement layers is therefore of utmost importance when engineers design a pavement structure.

**BACKGROUND**

Often pavement response models are based on the integral transformation solution of a Multi-Layer Linear Elastic (MLLE) system. However, with increasing computing capacity, Finite-Element (FE) analysis may become a viable analysis tool for routine pavement design in the near future. Even though FE analysis allows for the introduction of material non-linearity in terms of plasticity, some information on the resilient response characteristics of pavement materials is still required (Theyse 2012). Given that both the MLLE and FE solutions are based on continuum mechanics, Hooke’s law in terms of either an isotropic or anisotropic formulation governs the elastic material response, with the elastic properties of the material expressed by a pair of constants such as Young’s modulus and Poisson’s ratio, bulk and shear moduli or Lame’s constants (Brown & Pappin 1981; Uzan 1985; Lekarp et al 2000). Theyse (2012) focused on Young’s modulus and Poisson’s ratio, since these elastic constants are more familiar to most pavement engineers.

Young’s modulus and Poisson’s ratio are theoretical concepts of linear elasticity that can at best approximate experimental results of actual material elastic response. In their basic linear elastic form, Young’s modulus and Poisson’s ratio are rather poor approximations of actual material behaviour, as the behaviour of unbound granular material is:

- Stress-dependent, i.e. the stiffness or modulus of the material depends on the level of confinement of the material and the shear stress imposed on the material;
- Non-linear, i.e. there is not a linear relationship between the imposed stress and strain response of the material; and
- Inelastic (plastic), i.e. the material does not completely return to its original undeformed shape when the imposed stress is removed (Theyse 2012) (see Figure 1). The non-linear, stress-dependent behaviour of unbound granular material can, however,

![Figure 1 Non-linear and stress-dependent resilient behaviour (Theyse 2012)](image-url)
be simulated using the linear elastic model as a basis, but with a proper constitutive material model that adheres to the observed material behaviour (Theyse 2012).

**MODEL DEVELOPMENT**

Theyse (2008) reiterates that the departure point for any model formulation should be a detailed study of the data, and specifically patterns in the data. The characteristics observed were used to formulate a model from resilient tri-axial test data, using the test protocol described by Anochie-Boateng et al. (2009). This protocol is a resilient modulus-, chord modulus- and tangent modulus model based on Hooke’s law, calibrated using base layer material from road N2-33 (near Piet Retief) and the crushed stone base layer of road N4 west of Pretoria. These models are each formulated as a function of stress ratio, and confinement pressure or minor principal stress is considered in the model coefficients. Theyse (2012) explained that the formulation of a chord modulus was considered better, in terms of data analysis, than the formulation of a resilient modulus model, the latter being the most commonly used theory to model resilient behaviour. When the resilient modulus is calculated, only the end point of the stress-strain hysteresis loop is considered, discarding the majority of the information contained in the rest of the loop. The chord modulus represents the instantaneous stiffness of the material at any point on the stress-strain hysteresis loop (Theyse 2012). This is illustrated in Figure 2.

Theyse (2012) listed the following benefits when using the chord modulus approach, compared to the resilient modulus approach:

- The full hysteresis loop provides a comprehensive trace of the material stiffness

**Table 1** Summary of routine test results of materials sampled (Van Aswegen & Theyse 2011)

<table>
<thead>
<tr>
<th>Geological material type</th>
<th>Source (layer)</th>
<th>GM</th>
<th>Grading</th>
<th>Atterberg Limits</th>
<th>Mod AASHTO</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Norite</td>
<td>N4 Extension (base)</td>
<td>2.60</td>
<td>37.5</td>
<td>14.0</td>
<td>5.2</td>
<td>2.465</td>
</tr>
<tr>
<td>Chert</td>
<td>N4 Extension (upper selected)</td>
<td>1.90</td>
<td>75.0</td>
<td>37.5</td>
<td>28.3</td>
<td>31.8</td>
</tr>
<tr>
<td>Dolerite</td>
<td>S191 (base)</td>
<td>2.50</td>
<td>53.0</td>
<td>17.0</td>
<td>10.8</td>
<td>30.6</td>
</tr>
<tr>
<td>Shale</td>
<td>P10-2 (base)</td>
<td>1.96</td>
<td>53.0</td>
<td>32.5</td>
<td>25.1</td>
<td>26.5</td>
</tr>
<tr>
<td>Calcrete</td>
<td>D804 (base)</td>
<td>1.70</td>
<td>53.0</td>
<td>44.4</td>
<td>29.2</td>
<td>28.1</td>
</tr>
</tbody>
</table>

- $P_{\text{max}}$ = Maximum particle size (mm) [Method A1(b) TMH 1 (DoT 1986)]
- LL = Liquid Limit [Method A2 TMH 1 (DoT 1986)]
- PI = Plasticity Index [Method A3 TMH 1 (DoT 1986)]
- LS = Linear Shrinkage [Method A4 TMH 1 (DoT 1986)]
- GM = Grading Modulus [Method A2 TMH 1 (DoT 1986)]
- ARD = Apparent Relative Density [Method B14 TMH 1 (DoT 1986)]
- MDD$\text{mod}$ = Mod AASHTO maximum dry density [Method A7 TMH 1 (DoT 1986)]
- OMC$\text{mod}$ = Mod AASHTO optimum moisture content [Method A7 TMH 1 (DoT 1986)]
- MDD$\text{vib}$ = Vibratory table maximum dry density [Method A11T TMH 1 (DoT 1986); Anochie-Boateng et al. 2009]
- OMC$\text{vib}$ = Vibratory table optimum moisture content [Method A11T TMH 1 (DoT 1986); Anochie-Boateng et al. 2009]
or modulus over a wide stress ratio range, facilitating model formulation and calibration.

- The chord modulus shows consistent material behaviour at all levels of confinement pressure when using the test protocol described by Anochie-Boateng et al. (2009).
- The number of stress levels at which the tri-axial test is done can be reduced by 75% in terms of the current test protocol (Anochie-Boateng et al. 2009).
- In terms of application of the chord modulus in modelling the response of a pavement layer or tri-axial specimen, the chord modulus describes the evolution of the stiffness or modulus of the material from an initial condition at rest through the full stress-strain cycle including the load and unload phases.

**VERIFICATION OF RESILIENT RESPONSE MODEL**

**Routine test results**

The aim was to sample a sufficient variety of material types, specifically including moisture-sensitive materials. Table 1 summarises the average of three repeat tests per bulk material sampled on selected tests. The results appear reasonable and within the expected limits.

<table>
<thead>
<tr>
<th>Confining Pressure (kPa):</th>
<th>200 kPa</th>
<th>150 kPa</th>
<th>100 kPa</th>
<th>50 kPa</th>
<th>25 kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>N4 Extension base layer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Road S191 base layer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Road P10-2 base layer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure 3** Stress-dependent behaviour of unbound granular material in terms of the Chord Modulus and Stress Ratio (SR)

**Figure 4** Change in chord modulus calculated from tri-axial results with increasing degree of saturation
as described in COLTO (1998) for each class of material.

Resilient modulus tri-axial results
Van Aswegen and Steyn (2013) reported on processing and modelling, which were applied on the N4 Extension and N2-33 base layer material that was used to refine the process for application on the test results from material reported on in this paper. The same processing and modelling was followed as described by Van Aswegen and Steyn (2013).

Calibration of chord modulus model variables

Formulation of the chord modulus model
The chord modulus model consists of a hyperbolic function in combination with a linear function, where the linear function has a non-zero intercept (Figure 3), when the behaviour is described in terms of increasing stress ratio.

Considering the formulation of the chord modulus model, it is apparent that none of the variables $a$, $b$ or $c$ can be allowed to be negative values. Negative variable values would result in the following:

1. A negative $a$ value will force $c$ to be a negative value.
2. A negative $c$ value results in the hyperbolic portion of the model switching and having a negative asymptote, which will result in negative predicted stiffness, or modulus values that are counter-intuitive.
3. Although the line formed by $a$ and $b$ is allowed to have a negative slope, a negative $b$ value is not allowed, as it might force $c$ to be negative when $a$ is not large enough, resulting in negative predicted stiffness or modulus values.

As illustrated by Figure 3, the model successfully predicts the stress dependency of material with regard to stiffness or modulus, where the initial modulus value increases with increasing confinement pressure, as well as the initial stress softening with increasing stress ratio, followed by stress stiffening.

From literature it is evident that moisture has a significant influence on the stiffness or modulus and shear strength of unbound granular material (e.g. Hicks & Monismith 1971; Seed et al 1962; Thom & Brown 1987; Lekarp et al 2000). Therefore the chord modulus model formulation had to be extended to include the effect of moisture content or degree of saturation (S) on the stiffness or modulus of the material. Figure 4 illustrates three different material responses to the increasing effect of degree of saturation (from top to bottom in each of the three columns).

The following observations can be made regarding the behaviour depicted in Figure 4:

- There is a general reduction in the magnitude of the chord modulus with increasing degree of saturation for the materials depicted.
- Stress-stiffening behaviour with increasing confinement pressure, but also with increasing stress ratio above 20%, is observed for the N4 Extension base layer material (crushed norite) at all degrees of saturation.
- Stress-stiffening behaviour with increasing stress ratio only at the low degree of saturation. Stress stiffening occurred with increasing stress ratio at low and intermediate degrees of saturation, but not at the highest degree of saturation where the chord modulus remained almost constant with increasing stress ratio.

Table 2 Material characteristics for behaviour depicted in Figure 7

<table>
<thead>
<tr>
<th>Sample</th>
<th>Material type</th>
<th>GM</th>
<th>$P_{6075}$ (%)</th>
<th>PI</th>
<th>Classification TRH 14 (DoT 1985)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N4 Ext base</td>
<td>Norite</td>
<td>2.6</td>
<td>5.2</td>
<td>NP</td>
<td>G1</td>
</tr>
<tr>
<td>S191 base</td>
<td>Dolerite</td>
<td>2.5</td>
<td>10.8</td>
<td>8</td>
<td>G6</td>
</tr>
<tr>
<td>P10-2 base</td>
<td>Shale</td>
<td>2.0</td>
<td>25.1</td>
<td>8</td>
<td>G8</td>
</tr>
</tbody>
</table>

Figure 6 Hierarchy of variables and mathematical functions approximating the variables for the Chord Modulus

\[
M_c = \frac{\alpha_1 \sigma_1 + \beta_1 \sigma_2}{\sigma_1 - \sigma_2} = \frac{\alpha_1 \sigma_1 - \sigma_2}{\sigma_1 - \sigma_2}
\]

\[
M_c = \frac{\alpha_1 \sigma_1 + \beta_1 \sigma_2 + \alpha_2 \sigma_3}{\sigma_1 - \sigma_2 - \sigma_3} + \frac{\alpha_3 + \beta_3 \sigma_3}{\sigma_1 - \sigma_3 - \sigma_3}
\]

Where:
- $M_c$ = Chord Modulus (MPa)
- $SR_d$ = Deviator Stress Ratio (SR)
- $\sigma_1$ = Major principal stress (kPa)
- $\sigma_3$ = Minor principal stress (kPa)
- $\alpha_1, \beta_1, \alpha_2, \beta_2, \alpha_3, \beta_3$ = Regression coefficients

Figure 5 Mathematical functions approximating variables for S191 base (dolerite) (sample no 11726_36)
intermediate and high degrees of saturation, with the chord modulus reducing consistently with increasing stress ratio. These observations appear sensible when the material characteristics are considered, as given in Table 2, with Road P10-2 base layer material (weathered shale) having been classified as a weaker, lower quality material compared with the N4 Extension base layer material (crushed norite).

Selection of mathematical functions approximating variables
After the processing of the test data had been completed, the data was copied to a template where the chord modulus values $a$, $b$ and $c$ were calibrated for each specimen tested for each of the bulk sampled materials, i.e. eighteen specimens per bulk sample. Calibration of the model was done in Microsoft Office Excel 2013, using the solver function after identifying a mathematical function that best fitted the data. Various combinations of mathematical functions were tested, expressed as accuracy of the complete model using Root Mean Square Error (RMSE), with RMSE an estimate of the standard deviation of the random component in the data, before a function was assigned to a variable (Draper & Smith 1998; Everitt 2002). Variables $a$ and $c$ are approximated by a linear equation fitted to data for all the specimens. However, the constants of the linear approximation may differ from sample to sample. The same method was followed for variable $b$, which is approximated by a constant value that may differ from sample to sample (Figure 5).

Figure 6 illustrates how the mathematical functions are linked to the chord modulus model variables.

Identification of sub-variable relationships within variables
After the calibration process for variables $a$, $b$ and $c$, the sub-variables ($\alpha_1$, $\beta_1$, $\alpha_2$, ($\beta_2 = 0$), $\alpha_3$ and $\beta_3$) were evaluated against saturation, distinguishing between ‘high volumetric density (HD)’ and ‘low volumetric density (LD)’ samples. Figure 7 depicts the associated data for Road P10-2 (weathered shale). The limitation that none of the variables is allowed to be a negative value was kept in mind during the identification of mathematical equations describing the observed trends.

The following mathematical equations were linked to the observed trends:

- **Sub-variable $\alpha_1$**

  The sub-variable appears to reach a plateau for degree of saturation below 40% and then rapidly decreases to a lower level plateau at degree of saturation above 60% saturation. Theyse (2009) reported that the stiffness or modulus of partially

\begin{align*}
\text{Saturation} \% & \quad 0 & \quad 20 & \quad 40 & \quad 60 & \quad 80 & \quad 100 \\
\begin{array}{c|c|c|c|c|c|c}
\text{Low density} & \text{High density} \\
0 & 0 & 0 & 0 & 0 & 0 \\
20 & 0 & 0 & 0 & 0 & 0 \\
40 & 0 & 0 & 0 & 0 & 0 \\
60 & 0 & 0 & 0 & 0 & 0 \\
80 & 0 & 0 & 0 & 0 & 0 \\
100 & 0 & 0 & 0 & 0 & 0 \\
\end{array}
\end{align*}

\text{Figure 7} Calibrated sub-variable values of Road P10-2 plotted against degree of saturation
saturated unbound granular material reaches a ceiling value below a certain threshold value of saturation. Theysse (2009) used a sigmoidal curve, which was also identified to predict the trend of sub-variable $\alpha$.

- **Sub-variable $\beta_1$**
  A logarithmic curve was selected as it best predicts the trend in data.

- **Variable $\beta$ or sub-variable $\alpha_3$**
  The sub-variable appears to reach a plateau for degree of saturation below 40% and then rapidly decreases to a lower level plateau at a degree of saturation above 60%. Therefore a sigmoidal curve was selected to predict the trend in the data.

- **Sub-variable $\alpha_3$**
  No obvious trend could be identified for $\alpha_3$ and linear, exponential and power curves were evaluated. The accuracy of the complete model using RMSE was used to evaluate which trend to link to $\alpha_3$. A linear curve was selected to predict $\alpha_3$ values, as in combination with $\beta_3$ it yielded acceptable accuracy for the complete model.

\[ M_c = \frac{\alpha_1 + \beta_1 \sigma_3}{1 + e^{\alpha_3 (S - n_{03})}} + \frac{\alpha_2}{1 + e^{\alpha_3 (S - n_{03})}} + \frac{\alpha_3 + \beta_3 \sigma_3}{1 + e^{\alpha_3 (S - n_{03})}} \]

**Where:**
- $M_c =$ Chord Modulus (MPa)
- $SR_d =$ Deviator Stress Ratio (SR)
- $\alpha_1, \alpha_2, \alpha_3, \beta_1, \beta_3 =$ Regression coefficients
- $k_1, l_1, m_3, n_3 =$ Regression coefficients

**Figure 8** Hierarchy of variables and sub-variables as defined for the Chord Modulus

**Table 3** Statistical data for high (HD) and low volumetric density (LD) samples from N4 Extension base layer

<table>
<thead>
<tr>
<th>Specimen no.</th>
<th>R² (ratio)</th>
<th>RMSE (%)</th>
<th>SEE (MPa)</th>
<th>% Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>High volumetric density (HD)</td>
<td>0.85</td>
<td>0.44</td>
<td>84.24</td>
<td>9.55</td>
</tr>
<tr>
<td>Low volumetric density (LD)</td>
<td>0.72</td>
<td>0.63</td>
<td>99.31</td>
<td>12.81</td>
</tr>
</tbody>
</table>

**Figure 9** Prediction accuracy for high volumetric density (HD) samples from N4 Extension base layer (norite)

**Figure 10** Prediction accuracy for low volumetric density (LD) samples from N4 Extension base layer (norite)

**Table 4** Statistical data for high volumetric density (HD) samples from N4 Extension base layer (norite)

<table>
<thead>
<tr>
<th>Specimen no.</th>
<th>VD (ratio)</th>
<th>S (ratio)</th>
<th>R² (ratio)</th>
<th>RMSE (%)</th>
<th>SEE (MPa)</th>
<th>% Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>11306_19</td>
<td>0.82</td>
<td>0.14</td>
<td>0.96</td>
<td>0.27</td>
<td>55.00</td>
<td>5.60</td>
</tr>
<tr>
<td>11306_20</td>
<td>0.83</td>
<td>0.16</td>
<td>0.95</td>
<td>0.27</td>
<td>63.90</td>
<td>5.70</td>
</tr>
<tr>
<td>11306_21</td>
<td>0.82</td>
<td>0.14</td>
<td>0.59</td>
<td>0.89</td>
<td>161.80</td>
<td>20.80</td>
</tr>
<tr>
<td>11306_22</td>
<td>0.82</td>
<td>0.43</td>
<td>0.94</td>
<td>0.33</td>
<td>53.40</td>
<td>7.00</td>
</tr>
<tr>
<td>11306_23</td>
<td>0.83</td>
<td>0.45</td>
<td>0.96</td>
<td>0.26</td>
<td>45.40</td>
<td>5.30</td>
</tr>
<tr>
<td>11306_24</td>
<td>0.82</td>
<td>0.41</td>
<td>0.89</td>
<td>0.39</td>
<td>76.60</td>
<td>8.50</td>
</tr>
<tr>
<td>11306_25</td>
<td>0.82</td>
<td>0.66</td>
<td>0.68</td>
<td>0.69</td>
<td>159.80</td>
<td>17.40</td>
</tr>
<tr>
<td>11306_26</td>
<td>0.82</td>
<td>0.65</td>
<td>0.83</td>
<td>0.48</td>
<td>71.30</td>
<td>7.40</td>
</tr>
<tr>
<td>11306_27</td>
<td>0.82</td>
<td>0.67</td>
<td>0.88</td>
<td>0.42</td>
<td>70.90</td>
<td>8.30</td>
</tr>
</tbody>
</table>

**Table 5** Statistical data for low volumetric density (LD) samples from N4 Extension base layer (norite)

<table>
<thead>
<tr>
<th>Specimen no.</th>
<th>VD (ratio)</th>
<th>S (ratio)</th>
<th>R² (ratio)</th>
<th>RMSE (%)</th>
<th>SEE (MPa)</th>
<th>% Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>11306_28</td>
<td>0.80</td>
<td>0.13</td>
<td>0.25</td>
<td>1.00</td>
<td>157.40</td>
<td>20.00</td>
</tr>
<tr>
<td>11306_29</td>
<td>0.81</td>
<td>0.13</td>
<td>0.26</td>
<td>0.88</td>
<td>150.70</td>
<td>19.20</td>
</tr>
<tr>
<td>11306_30</td>
<td>0.80</td>
<td>0.12</td>
<td>0.27</td>
<td>0.81</td>
<td>80.50</td>
<td>11.20</td>
</tr>
<tr>
<td>11306_31</td>
<td>0.79</td>
<td>0.42</td>
<td>0.81</td>
<td>0.53</td>
<td>125.20</td>
<td>14.60</td>
</tr>
<tr>
<td>11306_32</td>
<td>0.80</td>
<td>0.42</td>
<td>0.74</td>
<td>0.71</td>
<td>97.00</td>
<td>13.00</td>
</tr>
<tr>
<td>11306_33</td>
<td>0.80</td>
<td>0.46</td>
<td>0.82</td>
<td>0.56</td>
<td>85.30</td>
<td>10.00</td>
</tr>
<tr>
<td>11306_34</td>
<td>0.79</td>
<td>0.62</td>
<td>0.95</td>
<td>0.51</td>
<td>51.20</td>
<td>9.60</td>
</tr>
<tr>
<td>11306_35</td>
<td>0.79</td>
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<td>0.41</td>
<td>53.90</td>
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</table>
Table 6 Statistical data for high (HD) and low volumetric density (LD) samples from Road D804 base layer

<table>
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<tr>
<th>Specimen no</th>
<th>VD (ratio)</th>
<th>S (ratio)</th>
<th>R² (ratio)</th>
<th>RMSE (%)</th>
<th>SEE (MPa)</th>
<th>% Error</th>
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Table 7 Statistical data for high volumetric density (HD) samples from Road D804 base layer (weathered calcrete)

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<th>Specimen no</th>
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<th>S (ratio)</th>
<th>R² (ratio)</th>
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<td>0.69</td>
<td>0.52</td>
<td>25.90</td>
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</table>

Table 8 Statistical data for low volumetric density (LD) samples from Road D804 base layer (weathered calcrete)
accuracy than the other samples (Table 5 and Figure 10). The predicted modulus for samples 11306_28, 29, 32 and 33 is slightly lower than the observed modulus. The remaining individual results of the set indicate a good fit. These results can be discarded, as the remaining individual results from the set at the specific degree of saturation indicate accurate prediction by the model. The results from sample 11306_30 were discarded due to an error during testing.

Road D804 base layer (weathered calcrete)

The model appears to have relatively good prediction accuracy for the material from Road D804 base layer on material level. This is indicated by the statistical parameters listed in Table 6 for the high (HD) and low volumetric density (LD) samples. However, some of the individual test results are not modelled accurately by the combined model. Based on SEE only individual test results from two samples (11728_20 and 21) with SEE 85 MPa and 94 MPa can be regarded as poor, as depicted in Figure 11 and Table 7. The SEE for the remaining individual results varies between 15 MPa and 33 MPa with R^2 between 0.74 and 0.96, which is acceptable considering that these results are obtained at material level. The result for sample 11728_19 was discarded due to an error during testing.

The model appears to have relatively good prediction accuracy for the low volumetric density material from Road D804 base layer on material level (Table 8 and Figure 12). Individual test results for sample 11728_31, 32 and 33 indicate poor prediction accuracy with the SEE 60 MPa, 42 MPa and 47 MPa respectively. R^2 is 0.33, 0.68 and 0.17 respectively. The observed trend in modulus results for these samples all appear to lack a stress-stiffening component, and the observed modulus decreases rapidly at all confining pressure levels without reaching a constant modulus at higher degree of saturation. Therefore the model cannot model the behaviour accurately, as it differs from the general trends identified in Figure 4, and can be discarded. The remaining samples yield SEEs of between 26 MPa and 61 MPa, with R^2 ranging from 0.65 to 0.86, which is acceptable considering that these results are obtained at material level. The result for sample 11728_34 was discarded due to an error during testing.

Parametric analysis per bulk material sample

The model was further assessed by investigating parametric plots of the model. The high and low volumetric density scenarios were assessed individually. The difference between relatively high and relatively low volumetric densities is between 1% and 3%. The stress dependency of the model reflects the trends observed in the data. In terms of the parametric plot for degree of saturation, the model realistically reflects trends observed in the data as extrapolated from the three degrees of saturation tests conducted in this study.

N4 Extension base layer (crushed norite)

The model appears to have relatively good prediction accuracy for the material from the high (HD) and low volumetric density (LD) samples tested. At 80% saturation, the predicted modulus does not differ significantly between high and low volumetric densities, as can be observed in Figure 13. The parametric plots for saturation realistically reflect the decrease of modulus with increasing degree of saturation at 50 kPa confining pressure (Figure 14). The low deviator stress of 20 kPa appears to reflect the influence of suction pressure on constant modulus at higher degree of saturation.
the material strength, before the increasing deviator stress overshadows the influence of suction pressure. The failure or yield strength at 0 kPa confinement predicted by using Theyse’s suction model (2009), in essence depicts the influence of suction pressure. Figure 5 illustrates the failure or yield strength at 0 kPa, i.e. influence of suction pressure for this material.

Figure 15 Parametric plots of saturation at different deviator stress levels for N4 Extension base layer (norite)

Figure 16 Parametric plots of the stress-dependent behaviour for Road D804 base layer (weathered calcrite)

Figure 17 Parametric plots of saturation at different deviator stress levels for Road D804 base layer (weathered calcrite)
**Figure 18** Bulk samples of base layer material from N4 Extension and Road S191

**D804 base layer (weathered calcrite)**

Figure 16 illustrates the stress-dependent model parametric plots for high and low volumetric density at 20% saturation. The parametric plots realistically models the behaviour observed in the data trends. Similar to the modulus behaviour of Road P10-2 base layer material, Road D804 base layer material does not have a distinct stress-stiffening component with increasing stress ratio. Both the aforementioned materials have low Grading Modulus (DoT 1985) values of 1.96 and 1.71 respectively.

As for N4 Extension upper selected layer material, the influence of high fines content and Plasticity Index (PI) (DoT 1986) can also be seen in the parametric plots for saturation (Figure 17). The influence of suction pressure on the material strength can be seen up to 100 kPa deviator stress at 25 kPa confining pressure. This trend of high predicted modulus values was also observed at 0 kPa confinement where values of between 2 000 kPa and 1 800 kPa were observed.

### CALIBRATION OF CHORD MODULUS MODEL VARIABLES FOR CRUSHED AND NATURAL UNBOUND MATERIAL

**Distinction between crushed and natural unbound material**

When the bulk samples in this paper are considered, the N4 Extension base layer (norite) and the Road S191 base layer (dolerite) appear to consist of a crushed unbound material. Figure 18 depicts the bulk samples during sampling in which the crushed material can be seen. Grading analysis indicated that the Grading Modulus (GM) values of the two bulk samples are 2.60 and 2.50 respectively. No crushed material was visible in the remainder of the bulk material samples. This is also reflected in lower GM values ranging between 1.70 and 1.96.

When the variables calibrated for each bulk material sample are compared with degree of saturation, it appears as if the crushed material and natural material group together. This is illustrated by the lines in Figure 19, which indicate the possible groupings. However, such a grouping could not be clearly identified for variable \( \beta_3 \).

The bulk material sample results were divided into two groups based on the appearance during sampling and the difference in GM of the samples. The crushed unbound material group consisted of N4 Extension base layer (norite) and Road S191 base layer material (dolerite). When the specified grading envelopes for G1 to G4 materials (COLTO 1998) are used to calculate the GM, GM values between 2.70 and 2.05 are calculated. The natural unbound material group consisted of N4 Extension upper selected (weathered chert), Road P10-2 base (weathered shale) and Road D804 base layer material (weathered calcrite). COLTO (1998) provides broad envelope values for GM for natural unbound materials (G5 to G9). Although the two material groups only consist of two and three bulk material samples respectively, the basic material properties (Table 1) indicate that the materials comprising the two groups are not similar and include a range of variability.

**Model calibration for crushed unbound material**

The model appears to have relatively good prediction accuracy for crush unbound material considering that two different materials are now combined. This is indicated by the statistical parameters listed in Table 9 for the relatively high volumetric density (HD) and relatively low volumetric density (LD) samples. The error values observed are acceptable when variability inherent in materials is considered.

However, some of the individual test results appeared to not predict the material behaviour as accurately. Figures 20 and 21 depict the relationship between predicted and observed modulus values.

When the average of the model variables calibrated for HD and LD are calculated and used in the saturation and stress-dependent...
chord modulus model, the model retains relatively good prediction accuracy as indicated by the statistical parameters listed in Table 10. This was done, since the HD and LD designations were arbitrarily determined for this paper and it does not constitute universally accepted high or low volumetric density values. The statistical parameters per individual specimen did not improve or worsen significantly and is therefore not listed again.

Model calibration for natural unbound material

The model appears to have relatively good prediction accuracy for natural unbound material considering that three different materials are now combined. This is indicated by the statistical parameters listed in Table 11 for the high (HD) and low volumetric density (LD) samples. The error values observed are excellent when variability inherent in materials is considered. However, some of the individual test results appeared to not predict the material behaviour as accurately. Unlike for the crushed unbound material, the calibrated values generally fitted all three of the materials equally well. The same individual samples that were identified previously to have a worse fit generally also yielded higher SEE and error results. When all these individual samples are removed from the data set, the statistical data for high and low volumetric density samples improve, especially for the low volumetric density samples (Table 12).

Figures 22 and 23 depict the relationship between predicted and observed modulus values. The accuracy depicted in Figure 22 does not appear to be as good, but when the SEE and error are considered, it appears acceptable.

When the average of the model variables calibrated for HD and LD are calculated and used in the saturation- and stress-dependent chord modulus model, the model retains relatively good prediction accuracy, as indicated by the statistical parameters listed in Table 13. This was done since the HD and LD were arbitrarily determined for this paper and these do not constitute universally accepted high or low volumetric density values. The statistical parameters per individual

| Table 9 Statistical data for high (HD) and low volumetric density (LD) samples of crushed unbound material (norite and dolerite) |
|---------------------------------|----------------|----------------|
| Crushed material                | RMSE (%)       | SEE (MPa)      | % Error       |
| High volumetric density         | 0.41           | 187.16         | 24.70         |
| Low volumetric density          | 0.43           | 204.32         | 30.40         |

| Table 10 Statistical data for all samples of crushed unbound material (norite and dolerite) |
|---------------------------------|----------------|----------------|
| Crushed material                | RMSE (%)       | SEE (MPa)      | % Error       |
|                                | 0.48           | 241.03         | 32.07         |

| Table 11 Statistical data for high (HD) and low volumetric density (LD) samples of natural unbound material (weathered shale and calcrete) |
|---------------------------------|----------------|----------------|
| Natural material                | RMSE (%)       | SEE (MPa)      | % Error       |
| High volumetric density (HD)    | 0.42           | 56.42          | 7.78          |
| Low volumetric density (LD)     | 0.67           | 90.31          | 12.89         |

| Table 12 Statistical data for revised set of high (HD) and low volumetric density (LD) samples of natural unbound material (weathered shale and calcrete) |
|---------------------------------|----------------|----------------|
| Natural material                | RMSE (%)       | SEE (MPa)      | % Error       |
| High volumetric density (HD)    | 0.39           | 48.22          | 7.18          |
| Low volumetric density (LD)     | 0.47           | 61.45          | 9.58          |

| Table 13 Statistical data for all samples of natural unbound material (weathered shale and calcrete) |
|---------------------------------|----------------|----------------|
| Natural material                | RMSE (%)       | SEE (MPa)      | % Error       |
|                                | 0.67           | 91.09          | 13.05         |
The stress-dependent behaviour of the material is realistically reflected in the parametric plots, even when the two materials are combined. The parametric plots for saturation appear distorted. Stress and saturation parametric plots are depicted in Figure 24. The apparent distortion from parametric plots for the individual materials might be explained when the Atterberg Limits (COLTO 1998) of the two materials are considered. Road S191 base layer material has a high fines content and PI, whereas the N4 Extension base layer material has low fines content and was classified as non-plastic. The influence of fines content and PI is distinguishable at low deviator stress levels and degree of saturation below 20%, where suction pressure appears to provide material strength. For deviator stress levels higher than 100 kPa and degree of saturation between 20% and 60%, the modulus decreases and or remains constant, where after it decreases for degree of saturation higher than 60%.

Natural unbound material
The stress-dependent behaviour of the material is realistically reflected in the parametric plots, even with the combination of three materials. The parametric plots for saturation appear consistent with plots of the individual materials. Stress and saturation parametric plots are depicted in Figure 25. The influence of high fines contents and PI is clear at low deviator stress levels (up to 100 kPa), where suction pressure appears to provide material strength. At deviator stress levels above 100 kPa the influence of suction pressure is overshadowed by the influence of confinement and the degree of saturation.

CONCLUSIONS
The chord modulus calculated from tri-axial test data (Anochie-Boateng et al 2009) for five different unbound granular materials illustrated the complex interaction between moisture in terms of degree of saturation and stress condition. The chord modulus model formulation consistently captured the complex stress-dependent behaviour of the selection.
of materials, but for the model to simulate the dependency of materials on the degree of saturation, multiple model components had to be introduced based on trends that could be observed from the data. A saturation- and stress-dependent chord modulus model was formulated and calibrated for all five bulk unbound granular material samples. The model generally showed a good prediction accuracy.

The model formulation was further tested when the materials were grouped into two groups – crushed and natural unbound materials. Calibration of the two groups yielded satisfactory prediction accuracy, but requires data for a wider selection of materials to boost confidence in the formulation and calibration of the model.

Preliminary conclusions reported in this paper indicate that the chord model formulation yields satisfactory predictions, especially when the model is calibrated for each individual material type.

ACKNOWLEDGEMENTS
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DISCLAIMER
This paper reflects the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of SANRAL and the CSIR, nor the final models or data as incorporated into the revised South African Road Design System.

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A proposal for the probabilistic sizing of rainwater tanks for constant demand

J E Allen, J Haarhoff

In recent years there has been an international trend towards installing rainwater tanks in an attempt to save water. This study investigates the possibility of simplifying the process of sizing a rainwater tank for optimal results under constant water demand. The required size of a rainwater tank is influenced by the Mean Annual Precipitation, the size of the roof which is being used for harvesting, the water demand, the desired certainty of supply and the rainfall patterns. The study investigates the use of a generalised equation of the form $y = a(x - 1)^2 + c$, where $x$ is a runoff parameter, $y$ is a storage parameter and $a$, $b$ and $c$ are constants which differ for each location, depending on rainfall characteristics. Results obtained using the generalised equation are compared to results obtained by simulating tank behaviour over a 16-year period (1994–2009) at three rainfall stations. The results suggest that such an equation can be used to estimate the storage capacity of a rainwater tank.

INTRODUCTION

The advantages of rainwater tanks to meet domestic demand are obvious, as mirrored by the international trend towards including rainwater tanks in water conservation strategies. It is therefore surprising that there are relatively few locations in South Africa where rainwater tanks have been systematically incorporated into new housing developments. The resistance to their wider adoption may be due to a lack of clear and simple guidelines of how they should be sized, and their associated risk of failure. The paper will briefly review the available methods for rational tank sizing and then propose a simplified framework for local application. The proposed procedure will be demonstrated by a design example for three selected locations in South Africa.

EXISTING APPROACHES TO RAINWATER TANK SIZING

The models previously proposed in the literature rely on three different approaches. Both the demand side approach (Centre for Science and Environment) and the supply side approach (Rees & Ahmed 2002) require information on the rainfall pattern, which can be obtained from historical data. The demand side approach starts with the daily water requirement, and sizes the tank according to the longest consecutive number of days without rain, whereas the supply side approach is based on the premise that all the available water running from the roof should be harvested. The latter therefore isolates the wet season from the remainder of the year, and then suggests a tank size to contain all the runoff during the wet season. Both these methods have obvious weaknesses. The demand side approach assumes that the roof is large enough to fill the tank prior to the dry season, therefore possibly leading to a less reliable supply. The supply side approach only considers the available water, even if it is much more than the demand, therefore possibly leading to an overly large and expensive storage tank.

A variation on the demand side approach for sizing rainwater tanks has been developed in the United Kingdom (Fewkes & Warm 2000). The method requires the calculation of an input ratio, which is defined as $A^*(MAP)/(D)$, where $A$ is the catchment area, $MAP$ is the mean annual precipitation and $D$ is the average annual demand. This ratio is then used along with a desired performance level to determine the number of days’ storage that ought to be provided by means of a set of design curves. Each design curve shows the storage period versus the volumetric reliability for a specific value of the input ratio. The justification for using only one curve for a given input ratio comes from the fact that very little difference exists in the United Kingdom from one location to another (Fewkes & Warm 2000).

The United Kingdom’s Environment Agency recommends a method which combines the supply and demand side approaches. The method determines the

Keywords: Tank sizing, rainwater harvesting, probabilistic sizing
required size of the tank as being either a user-defined percentage of the MAP (5% is suggested for the UK), or of the average annual demand, whichever is smaller (Ward et al. 2010). Ward et al. (2010) state that the size is then calculated as:

\[ S = P.A.C_F.F.MAP \]  

(1)

Where:
- \( P \) = user defined percentage
- \( A \) = area of the roof
- \( C_F \) = runoff coefficient
- \( F \) = system filter efficiency
- \( MAP \) = mean annual precipitation

The runoff coefficient is a function of the roof surface and gutter integrity, and is usually taken as close to 1 for impervious roof materials. The system filter efficiency is determined by the presence and nature of a filtering device included for water quality purposes before the storage tank. If \( D < MAP, MAP \) would be replaced by the annual demand (\( D \)).

The mathematical simulation of the tank level provides a third approach which eliminates the main weaknesses of the other two approaches, but it requires a detailed rainfall record covering at least ten years, preferably longer. It furthermore requires more computational effort, which is only possible with computer simulation. A number of computer models which are capable of doing such a simulation are available. These include the Model for Urban Stormwater Improvement Conceptualisation (MUSIC), Aquacycle and Raincycle (Ward et al. 2010). The mathematical simulation approach, however, has the additional power of coupling the reliability of supply with the tank size, which is a prerequisite when considering rainwater tanks for domestic water supply.

### SIMPLIFYING THE PROCESS

Rainwater tank sizing to meet constant demand, in its simplest form, reduces to finding a mathematical relationship linking five key variables:

- The rate at which water is withdrawn from the tank, expressed as a constant daily withdrawal, or annual average daily demand (AADD) in l/d. Where water is used for indoor demand and the number of inhabitants stay the same, the assumption of constant demand is reasonable.
- The supply security (S), expressed as the fraction of the total water demand met by the tank. If secondary or emergency supplies with tankers, for example, are available, the supply security could typically be around say 90%. In the absence of secondary supplies, the required supply security should be higher, say around 98%.
- The roof plan area (aerial view) from which the water is harvested, expressed in m².
- The tank volume (V), expressed in l.
- The mean annual precipitation (MAP), expressed conventionally in mm/year.

The numerical solution of the storage parameter \( Y \) provides the number of days of available water in a year, and to express this as a ratio of the total water demand met by the full tank, provided no water flows into the tank. If secondary or emergency supplies, for example, are available, the supply security could typically be around say 90%. In the absence of secondary supplies, the required supply security should be higher, say around 98%.

The idea of simplifying the process in this way is not altogether a new one. As mentioned above, Fewkes and Warm (2000) used a simplifying parameter \( A^{\cdot}MAP/D \), which is equal to the runoff parameter \( X \), suggested above. Ghisi (2010) developed a ratio \( F \), which is the inverse of the runoff parameter.

### THE SUGGESTED MATHEMATICAL RELATIONSHIP

It is intuitive that there should be an inverse relationship between \( X \) and \( Y \) – a larger, more continuous supply of water from the roof will obviously require less storage. In addition, for both \( X \) and \( Y \), asymptotic limits exist. As the runoff parameter \( X \) approaches 1, the required tank size, and thus the storage parameter \( Y \), will approach infinity, as no water may be lost by spillage. On the other hand, as the storage parameter \( Y \) approaches its lower limit, the runoff parameter \( X \) will approach infinity, as the tank has to be completely filled by even the very small rainfall events, thus requiring a very large roof surface.

From the above the general relationship follows, as shown in Figure 1.

The relationship in Figure 1 follows a power curve of the general form:

\[ Y = aX^b \]  

(4)

![Figure 1: Graphs showing the postulated shape of the design curve](image-url)
By incorporating the asymptotic limits discussed above the following equation is obtained:

\[ Y = a(X - 1)^b + c \]  

(5)

Where:
- \( Y \) = storage parameter, defined above
- \( X \) = runoff parameter, defined above
- \( a, b, c \) = constants specific to each graph

SIMULATING THE BEHAVIOUR OF A RAINWATER TANK

Volume balance approach

The mathematical simulation of a rainwater tank is based on a volume balance approach. The idea is simply that the volume of water in the tank at the end of a given time-step must be equal to the volume of water in the tank at the beginning of the time-step (end of the previous time-step), after accounting for addition of the volume of water that has entered the tank, and subtracting the volume that has left the tank during the same time-step. Mathematically (Ming-Daw et al 2009; Kennedy & Male 2006):

\[ V_t = V_{t-1} + Q_t - D_t \]  

(6)

This is subject to the constraint that the maximum volume of water in the tank is limited to the size of the tank:

\[ 0 \leq V_t \leq S \]  

(7)

Where:
- \( V_t \) = volume of rainwater in the tank at the end of time interval \( t \)
- \( Q_t \) = volume of rainwater that enters the tank during time interval \( t \)
- \( D_t \) = (volume of water that is removed from the tank) during time interval \( t \)
- \( S \) = maximum storage capacity

Yield-after-spill and yield-before-spill

In order to simulate the behaviour of a rainwater tank, rainfall data was used as an input with a preselected roof area, tank volume and water consumption from the tank, to obtain the reliability of the water supply from the tank for the selected parameters. In order to obtain an exact representation of the rainwater tank it is necessary to add rain to the tank and simultaneously subtract the water demanded from the tank. When working with discretised time intervals, the calculation sequence of inflow, spillage and outflow is important. Two different approaches can be used:

The first option is the yield-after-spill option. For this option the rainfall is added to the tank and the spillage is immediately removed by limiting the tank volume. The water demanded from the tank is then removed at the end of the same time-step, from a volume which can never exceed the maximum storage volume. In reality the water could be removed and rainwater added to the tank simultaneously. This could mean that the maximum limit would not be reached in reality, even though computationally it is reached. When spillage does occur, this option effectively decreases the tank capacity and, thus, underestimates the amount of water which can be supplied by the tank. For this approach the following equations can be used to determine the yield from the tank and the volume of water in the tank (Liaw & Tsai 2004; Palla et al 2011):

\[ Y_t = \min\{D_t; V_{t-1}\} \]  

(8)
Table 2 The AADD (in ℓ) that can be supplied for various combinations of roof area, tank volume and certainty of supply, based on the calibration curves of Figures 6 to 8

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<th>Roof area (m²)</th>
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<td></td>
<td>15 000</td>
<td>136.7</td>
<td>222.4</td>
<td>121.6</td>
</tr>
</tbody>
</table>

V_t = \min \{V_{t-1} + Q_t - Y_t ; S - Y_t \} \quad (9)

Where:

Y_t = \text{yield from the tank during the time interval } t

The second approach, the yield-before-spill approach, effectively does not allow the tank to spill during the time-step. It collects all the inflow, and satisfies the full demand first. If any surplus remains at the end of the time-step, this surplus is spilled. This has the opposite effect on the tank capacity and thus overestimates the water which can be supplied by the tank. This approach is described by the following equations (Liaw & Tsai 2004; Palla et al 2011):

Y_t = \min \{D_t ; V_{t-1} + Q_t \} \quad (10)

V_t = \min \{V_{t-1} + Q_t - Y_t ; S \} \quad (11)

The difference between the water supply reliability calculated by the two methods is typically approximately 0.3%. However, when small tank sizes are investigated, spillage will occur more often and the difference in reliability obtained from the two methods may be as high as 3%.

**CALIBRATING THE EQUATION**

The structure of the proposed equation was tested by simulating the behaviour of a rainwater tank at three stations (Kimberly, Mossel Bay and Rustenburg). The stations were selected to include both seasonal and year-round rainfall regions, as well as differences in mean annual precipitation. The simulations were conducted using daily rainfall data over a 16-year period (1994–2009), obtained from the South African Weather Services. The validity of the equation was tested at S = 90%, S = 95% and S = 98%, using the method for simulating a rainwater tank, as described above. Each simulation was done using both the yield-after-spill and the yield-before-spill approaches, and their average value was then used for model calibration.

The parameters a and b were determined with least-square fitting of the simulated data points. Figures 2, 3 and 4 show the resulting graphs for the three stations in solid lines, as fitted to the data set shown with markers. The coefficient of determination for each of the fitted graphs to its corresponding data set was also calculated.

**Figure 4 Fitted graphs for Rustenburg**

![Figure 4 Fitted graphs for Rustenburg](image-url)
Table 3 Calculations for application example in text

<table>
<thead>
<tr>
<th>Supply security</th>
<th>Kimberley</th>
<th>Mossel Bay</th>
<th>Rustenburg</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean annual precipitation MAP</td>
<td>428</td>
<td>532</td>
<td>579</td>
</tr>
<tr>
<td>Roof area A</td>
<td>65</td>
<td>65</td>
<td>65</td>
</tr>
<tr>
<td>Tank volume V</td>
<td>3 000</td>
<td>3 000</td>
<td>3 000</td>
</tr>
<tr>
<td>Constant a</td>
<td>62.300</td>
<td>72.606</td>
<td>85.627</td>
</tr>
<tr>
<td>Constant b</td>
<td>−0.475</td>
<td>−0.369</td>
<td>−0.351</td>
</tr>
<tr>
<td>Constant c</td>
<td>36.738</td>
<td>52.840</td>
<td>50.940</td>
</tr>
<tr>
<td>Runoff parameter X</td>
<td>2.320</td>
<td>2.820</td>
<td>3.800</td>
</tr>
<tr>
<td>Storage parameter Y</td>
<td>91.330</td>
<td>111.040</td>
<td>110.590</td>
</tr>
<tr>
<td>Annual average daily demand AADD</td>
<td>33</td>
<td>27</td>
<td>27</td>
</tr>
</tbody>
</table>

RESULTS

Table 1 provides a measure of the goodness-of-fit of the proposed equation. All the results show coefficients of determination of more than 94%, which is considered to be adequate for design purposes, given the inherent variability and deficiencies of the available hydrological records. Figures 2 to 4 also show a relatively sharp inflection point at a storage parameter of about X = 2, which means that the equation should be restricted to values of X > 2.

The results show clearly that there is no correlation between the MAP and the AADD that can be supplied. The MAP for Mossel Bay (532 mm) and the MAP for Rustenburg (579 mm) are much closer to each other than the MAP for Kimberley (428 mm). However, the AADD which can be supplied at Rustenburg is much closer to the results obtained for Kimberley than those obtained for Mossel Bay. The sizing of rainwater tanks is therefore more dependent on the seasonal variation of the rainfall than on the MAP. The only way to take this into account for a large region like South Africa, with its significant differences in rainfall characteristics, is to work from first hydrological principles as done in this paper.

The results also show clear links between the AADD that can be supplied, the roof area and the tank volume. As expected, the AADD will increase as the roof area increases, but the increase is significantly larger when a larger tank is used. This is due to the fact that increasing the roof area will have no effect if the tank is not large enough to store the increased volume of runoff. The effect of a larger roof area, however, is smaller when the certainty of supply is increased. Increasing the tank volume will also create an increase in the AADD that can be supplied. Similar to an increase in roof area, the increase in AADD is larger when the tank volume is larger. The equations that are obtained by using the simplifications discussed above thus account adequately for changes in roof area, tank volume, certainty of supply and location.

APPLICATION EXAMPLE

Consider a typical low-cost housing development with the roof areas of individual homes being 65 m². Estimate the constant daily demand that can be harvested if every home is supplied with a storage tank of 3 000 ℓ. The calculations are shown in tabular form in Table 3.

CONCLUSION

The results of the study show that an equation of the form \[ Y = a(X−1)^b + c \] can be fitted to simulated data for rainwater tanks with a minimum coefficient of determination of 0.94. Rainwater tank sizing can therefore be performed with good precision in terms of a storage parameter Y and runoff parameter X.

The determination of the constants a, b and c requires a long, reliable record of daily rainfall records. The analysis presented for three randomly selected sites in South Africa needs to be extended to more South African sites before general expressions for the constants may be determined.

The amount of annual rainfall seems to be less critical for rainwater tank sizing than its seasonal variation, which further underlines the need for analysis at different sites.

The design examples for the three reported sites indicate that rainwater tank harvesting could go a long way in providing the domestic demand of developing communities, and therefore deserves closer scrutiny by water supply engineers.

FUTURE RESEARCH

The current study assumed that 100% of the rain which falls on a roof is available for collection. However, in reality this is not the case. Losses are created by the uneven texture of the roof, the guttering system used, as well as possible evaporation losses. The study also did not consider the effect of a water treatment system, such as a first flush system, which will also create losses in the system. Further research should be done to refine the suggested equation when these losses are considered.

REFERENCES


INTRODUCTION

Settlement prediction of shallow foundations is essential for the design of a structure. This requires accurate input parameters and a reliable analysis method. Small-strain shear stiffness ($G_0$) and small-strain Young’s modulus ($E_0$) are important soil parameters, because they can be measured in the field and the laboratory using seismic testing techniques. This paper describes a foundation settlement prediction method that only requires $G_0$ or $E_0$ as an input parameter. Centrifuge testing was done to obtain the load-settlement behaviour of a foundation on different density sands. Predicted and measured results were compared and predictions were limited to a maximum settlement of 10% of the foundation diameter (i.e. 0.1D).

LITERATURE REVIEW

Settlement of shallow foundations depends on various factors, including the magnitude of the applied load, foundation size and geometry, foundation stiffness, ground stiffness and the strength of the underlying material (Canadian Geotechnical Society 2006). All these factors may contribute to uncertainty, and as stated by Yongqing (2011), accurate estimation of foundation settlement remains a significant challenge in foundation design.

However, Das and Sivakugan (2007) argued that settlement prediction for shallow foundations is primarily dependent on the accuracy with which the stiffness of the soil can be quantified, as well as the choice of analysis methods. Numerous methods for estimating foundation settlement have been proposed, and Douglas (1986) reported the existence of more than forty methods for the estimation of foundation settlement on granular soils. A report by Lutengger and DeGroot (1995) on settlement methods for granular soils found in excess of 50 different methods. Most of these methods rely on indirect correlations with in situ tests such as the standard penetration test (SPT) and the cone penetration test (CPT), with only nine of the methods based on elasticity theory which requires stiffness as input parameter. Archer (2014) summarised a representative sample of 16 elasticity-theory-based methods to predict load-settlement curves, illustrating the differences in approaches taken by various authors. It was shown that more recent methods incorporate the initial small-strain stiffness (either the shear- or Young’s modulus), indicating the move towards using the initial small-strain stiffness for foundation settlement analysis. This move can be attributed to the fact that in situ and laboratory seismic testing are becoming increasingly popular, and that...
values obtained are reliable (Campanella 1994; Woods 1978).

Mayne and Poulos (2001) suggested that finite difference and finite element methods may be the most realistic way of incorporating soil non-linearity when analysing foundation settlement, but this is a time-consuming task and requires a skilled analyst. They suggested that, for a preliminary estimate, a method may be used where the initial small-strain modulus is degraded for an appropriate strain level, and then applying a linear elastic analysis method. The shortcoming of this approach is that, in order to apply the appropriate strain level, an estimation of the foundation settlement is required before the analysis is conducted.

**Small-strain stiffness of soil**

Soil stiffness is expressed in different forms, including: shear modulus ($G_0$), constrained modulus ($M$), bulk modulus ($K$) and Young’s modulus ($E$). For the purpose of foundation design the values of shear- and Young’s modulus are commonly used and these two parameters are interrelated by Poisson’s ratio ($\nu$). Clayton (2011) stated that a comprehensive understanding of the stiffness parameters at small strains is essential if realistic ground movement predictions are to be made. Recently a better understanding of the non-linear stiffness behaviour of soil, also known as modulus degradation, has provided engineers with an efficient approach to produce more reliable stiffness values for design (Yongqing 2011). Conceptually the small-strain stiffness ($G_0$ or $E_0$) is considered to be constant for small-strain values up to a strain level of about 0.001% to 0.002%, and at strain levels below this value it is often assumed that the soil behaves elastically (Clayton & Heymann 2001). Above this strain level non-linear behaviour of the soil is observed and the stiffness decreases as strain increases.

**Softening functions**

The high stiffness at very small strain is not necessarily relevant for a geotechnical problem. It is therefore important to reduce the small-strain stiffness to a value relevant to the strain level that occurs for a specific design problem. This is achieved with softening functions or modulus reduction curves. These can be divided into two categories: (1) curves based on stiffness and strength parameters, and (2) curves based only on stiffness parameters, with both categories including the current strain level as an input value. Softening curves which use stiffness parameters only do not require the strength of the soil to be quantified.

Many stiffness reduction curves have been proposed to describe the non-linear stress strain relationship observed for soils, including Vucetic & Dobry (1991), Rollins et al (1998), Clayton & Heymann (2001). These softening curves have been established from triaxial and resonant column tests conducted in the laboratory and, since the boundary conditions below a shallow foundation are different, the question remains as to whether these softening curves can be applied directly to settlement of shallow foundations.

Elkahim (2005) argued that modulus reduction curves should have a minimum number of parameters for defining the non-linear relationship without compromising accuracy; the parameters should ideally have physical meaning and it should be easy to derive them. For this study a modified hyperbolic relationship presented by Oztoprak and Bolton (2013) was used as the softening function, as shown in Equation 1:

$$G = \frac{G_0}{1 + \left(\frac{\gamma}{\gamma_r}\right)^n}$$

Where:

- $G =$ shear modulus
- $G_0 =$ small-strain shear modulus
- $\gamma =$ current shear strain
- $\gamma_r =$ elastic threshold shear strain; if $\gamma < \gamma_r$ then $G/G_0 = 1$
- $\gamma_r =$ reference shear strain; if $\gamma = \gamma_r + \gamma_r$, then $G/G_0 = 0.5$
- $n =$ curvature parameter

This softening function is appealing for settlement prediction methods, as the only soil parameter required is small-strain shear stiffness.

**Measurements of small-strain stiffness**

Soil stiffness can be measured by laboratory tests, as well as by in situ tests. Different test methods can only determine stiffness values at certain strain levels, and therefore not all methods are applicable for small-strain stiffness determination. Seismic geophysical techniques are effective to determine the small-strain stiffness of geomaterials. These techniques measure the wave velocity which can then be used together with the bulk density ($\rho_{bulk}$) to calculate the small-strain stiffness. The small-strain shear stiffness can be calculated from the shear wave velocity ($V_s$) using Equation 2:

$$G_s = \rho_{bulk} \times V_s^2$$

Seismic techniques are attractive because they can be conducted in the field and in the laboratory, but Elhakim and Mayne (2003) suggested that in most cases results obtained from in situ tests are superior to laboratory tests. An advantage of in situ seismic testing is the fact that large volumes of soil can be tested, and that the test is conducted at the current stress condition with minimal disturbance of the material. Surface wave testing in particular is becoming increasingly popular, since these tests are non-invasive, non-destructive and cost-effective (Menzies 2000).

**EXPERIMENTAL METHODOLOGY**

Physical modelling was conducted to measure the load-settlement behaviour of a shallow foundation on sand. The modelling was done with the 150 G-ton geotechnical centrifuge of the University of Pretoria, and details of the centrifuge facility were given by Jacobsz et al (2014). The centrifuge tests were conducted at an acceleration of 50 g.

The soil used for the experimental work was a standard testing sand from the University of Pretoria, known as Cullinan sand. The sand was characterised through

<table>
<thead>
<tr>
<th>Parameter</th>
<th>20% RD</th>
<th>50% RD</th>
<th>80% RD</th>
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<td></td>
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<tr>
<td>Min dry density (kg/m³)</td>
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<td></td>
<td></td>
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<tr>
<td>$\rho_{max}$</td>
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<td></td>
</tr>
<tr>
<td>$\rho_{min}$</td>
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</tr>
<tr>
<td>Specific gravity, $G_s$</td>
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<td>Angular to sub-rounded</td>
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<td>USCS® Classification</td>
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<tr>
<td>Angle of friction, $\phi$ (°)</td>
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<td>39</td>
</tr>
<tr>
<td>Cohesion, $c$ (kPa)</td>
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<td>0</td>
<td>0</td>
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<tr>
<td>Dry density (kg/m³)</td>
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<td>1 531</td>
<td>1 614</td>
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<tr>
<td>$\varepsilon$</td>
<td>0.84</td>
<td>0.74</td>
<td>0.65</td>
</tr>
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*Unified Soil Classification System*
Centrifuge testing is used to test a reduced scale model in a centrifuge using centripetal acceleration. The model is reduced by a scale factor, and since the centrifuge tests were conducted at 50 g acceleration, scaling laws can be used to convert the measured properties and values of the model to a full-scale prototype equivalent. Table 2 presents the scaling laws for applicable physical properties.

<table>
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<tr>
<th>Property (Prototype)</th>
<th>Scale factor (Model)</th>
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<td>Length</td>
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<td>Mass density</td>
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</tr>
<tr>
<td>Acceleration</td>
<td>N</td>
</tr>
<tr>
<td>Stiffness</td>
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<tr>
<td>Stress</td>
<td>1</td>
</tr>
<tr>
<td>Force</td>
<td>1/N^2</td>
</tr>
<tr>
<td>Strain</td>
<td>1</td>
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<tr>
<td>Displacement</td>
<td>1/N</td>
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<tr>
<td>Wave velocity</td>
<td>1</td>
</tr>
</tbody>
</table>

### Centrifuge model

Centrifuge testing is used to test a reduced scale model of a large prototype and it is achieved in a centrifuge using centripetal acceleration. The model is reduced by a scale factor (N) equivalent to the acceleration of the centrifuge relative to 1 g, and since the acceleration is known, scaling laws can be used to convert the measured properties and values of the model to a full-scale prototype equivalent. Applicable scaling laws are shown in Table 2, and since the centrifuge tests were conducted at 50 g acceleration, N = 50.

A model container, with inside dimensions of 600 mm x 400 mm x 400 mm (L x W x H) was used for the modelling, and the sand height in the model container was 350 mm. A circular aluminium model foundation of 100 mm diameter (D) and 20 mm thickness was used. The model foundation scales to an equivalent 5 m diameter and 1 m thick prototype foundation at 50 g, and the dimensions were chosen for the foundation to behave as a rigid foundation. Fine P100 grit sandpaper was glued to the bottom of the foundation to replicate a rough base.

A Pfaff-silberblau SHE3.1 Mechanical Ball Screw Jack was used to apply the load. An HBM U93 50 kN load cell and an HBM WA50 50 mm LVDT were attached to the jack to measure the applied load and corresponding settlement. The 50 kN load capacity of the system was adequate to achieve between 0.2D and 0.3D settlement for all relative densities. Solartron Metrology AS/15 S series 30 mm displacement sensors were used to measure the sand settlement during centrifuge spin-up in order to calculate the sand density at different levels of acceleration. The model setup is shown in Figure 1.

Standard quick-mount bender- and extender elements from Piezo Systems, Inc were used to measure the shear and compression wave velocities of the sand at different depths. Bender elements are two-layer piezoelectric crystals, and depending on the wiring, are made to bend, twist or elongate. When these elements bend, they are referred to as bender elements, and when they elongate they are referred to as extender elements. Due to their perceived simplicity, bender elements have become popular in laboratory and centrifuge tests (Clayton 2011). Bender elements are used in pairs consisting of a transmitter and receiver, and due to the bending of the elements, shear waves are produced. Extender elements work on the same principle as bender elements, but due to the elongation, compression waves are generated. If a voltage is applied to the transmitter element, a mechanical wave is generated that travels through the soil and is detected by the receiver element. If the tip-to-tip distance between elements (ℓ_{T-R}) is known, as well as the travel time (Δt) of the wave from the transmitter to the receiver element, the wave velocity can be calculated. If the shear wave velocity is known, Equation 2 is used to calculate the small-strain shear stiffness. For the experiments the bender and extender elements were placed a distance of approximately 150 mm apart. This value was chosen as it was close enough to obtain a good quality signal, but far enough not to influence the load-settlement behaviour of the foundation or be damaged during loading. Three sets of bender- and extender elements were placed at three depths below the foundation. Each set consisted of bender- and extender element transmitter and receiver pairs placed at depths of 50 mm, 100 mm and 150 mm, which correspond to prototype depths of 0.5D, 1.0D and 1.5D. Small-strain measurements were taken at different g-levels during centrifuge acceleration at 10 g, 20 g, 30 g, 40 g and 50 g, which gave prototype depths ranging between 0.5 m to 7.5 m. This allowed a detailed small-strain stiffness with depth profile to be obtained. Figure 2 shows a schematic view of the model indicating the placement of the bender elements.

Once 50 g was reached, the final small-strain measurements were taken, where after the foundation was loaded with the jack. Load-settlement measurements were taken until either the maximum capacity of the load cell was reached or at a maximum settlement of 30 mm (0.3D). For the experimental work a total of six tests were conducted, i.e. two tests at each relative density.

### Experimental Results

The initial densities obtained for the different tests are presented in Table 3, together with the corresponding relative density values. The values show that the initial
values were within 2% of the target relative densities. The sand settlements measured at different acceleration levels during spin-up were used to calculate the in-flight densities in order to calculate the small-strain stiffness at each acceleration. The average relative density values obtained at 50 g for the 20% RD, 50% RD and 80% RD tests were 24.6%, 49.5% and 78.8% respectively.

Small-strain stiffness data

For the shear wave velocity measurements a ± 10 V continuous square wave with an input frequency of 25 Hz was used for the transmitting signal. The input frequency allowed the received signal to dissipate completely before the next signal was triggered to avoid interference occurring between two consecutive signals.

One of the difficulties with using bender elements in centrifuge modelling is the mechanical noise induced by the centrifuge itself. One of the methods used to mitigate this effect was to design a charge amplifier with electronic high- and low-pass filters to filter the unwanted mechanical noise. Although the filters gave enhanced in-flight received signals, the signal-to-noise ratio (SNR) was still low. To further increase the signal quality, signal stacking was used as a signal processing tool to increase the signal-to-noise ratio by summing successive signals. This increased the signal quality by reducing the effect of the mechanical noise induced by the centrifuge. Stacking is a common signal-processing tool which has been used with much success to improve the signal quality in noisy environments (Brandenberg et al. 2006; Brandenberg et al. 2008). A total of 60 stacks for each data set were found to be sufficient to reduce the background noise and obtain a clear signal.

Different methods are available for travel time determination. The most common techniques are measuring the time between the onset of the sender driving signal and detecting the first arrival at the receiver element (visual picking), travel time between two receivers some distance apart along the ray path detecting the same seismic event, as well as cross-correlation and cross-power spectrum methods (Leong et al. 2005). For this project, the method of visual picking of the first arrival was used.

The stacking procedure was conducted for all the measurements, which amounted to a total of 90 bender- and extender-element measurements per soil profile. Compression (P) waves arrive before shear (S) waves since P-waves travel faster. After the stacking algorithm was run to process a data set, the first arrival was chosen. Figure 3 shows a typical bender element result after filtering and stacking. As shown in the figure, the first arrival is the P-wave, with the S-wave arriving some time later. The extender elements were used to detect the arrival time of the P-waves. The bender element tests also generated strong P-wave signals, and the comparison of the bender element and extender element time histories was used to identify the first arrivals of the P-waves and S-waves. Since the second wave set is the arrival of the S-wave, the first break of this set was taken as the first arrival for the S-wave velocity calculation. The analyses of all the signals were approached in this manner to obtain the first arrival times.

From the shear wave velocities and densities the small-strain shear modulus ($G_0$) was

<table>
<thead>
<tr>
<th>Test</th>
<th>Density (kg/m$^3$)</th>
<th>Relative density RD (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20% RD Test 1</td>
<td>1 448.4</td>
<td>20.4</td>
</tr>
<tr>
<td>20% RD Test 2</td>
<td>1 450.6</td>
<td>21.1</td>
</tr>
<tr>
<td>50% RD Test 1</td>
<td>1 526.2</td>
<td>48.4</td>
</tr>
<tr>
<td>50% RD Test 2</td>
<td>1 528.9</td>
<td>49.4</td>
</tr>
<tr>
<td>80% RD Test 1</td>
<td>1 608.8</td>
<td>78.2</td>
</tr>
<tr>
<td>80% RD Test 2</td>
<td>1 608.5</td>
<td>78.1</td>
</tr>
</tbody>
</table>

Table 3 Initial density values
determined at each depth. Figure 4 shows the \( G_0 \) vs depth results of the 20% RD tests. The depth values shown are the prototype depth scaled from the depths at which the bender elements were placed. Seed and Idriss (1970) suggested an empirical relationship between \( G_0 \) and mean effective stress \( (p'_0) \) as \( G_0 = K (p'_0)^{0.5} \) where \( K = 8 \, 000 \) for loose sand, and 12 000 for dense sand.

Also shown in Figure 4 is the values proposed by Seed and Idriss (1970) for loose sand, which can be seen to be in close proximity to the values obtained for the centrifuge tests.

Figure 5 shows the small-strain stiffness results for the different density tests. Some scatter of the data is evident, but in general \( G_0 \) increased with depth and density as expected. The figure also shows the power function trend lines for the three different densities. The coefficient of correlation \( (R^2) \) is also shown in the figure and the values indicate a good fit with the data for all three data sets. The power function equations were used to calculate the stiffness at the required depths below the foundation for the settlement analysis.

**Load-settlement results**

A total of six load-settlement tests were conducted, two tests for each density. The results were converted to the prototype scale, i.e. for a 5 m foundation. In practice, settlements larger than 10% of the diameter \( (D) \) are usually not tolerated (i.e. 0.1 \( D \)) and this study focused on the load-settlement behaviour up to a settlement of 0.1 \( D \). The average load-settlement results for the two tests at each density were used, giving three data sets, one each for the 20% RD, 50% RD and 80% RD tests. The results are shown in Figure 6.

**LOAD-SETTLEMENT ANALYSIS**

The objective of this study was to assess whether the load-settlement curve for a shallow foundation on sand can be predicted using only small-strain stiffness data. This implies that no laboratory or in situ testing is required to obtain strength parameters or the ultimate bearing capacity. This is particularly useful for materials such as uncedentimented sands and gravels that are difficult to sample undisturbed.

A non-linear stepwise method was implemented to calculate foundation settlement, using the small-strain stiffness data and assuming axis-symmetrical conditions and uniform contact stress distribution for a circular footing. The methodology may be summarised as follows:

1. Determine the small-strain stiffness profile \((G_0\) or \(E_0')\) with depth from appropriate in situ or laboratory testing.

2. Subdivide the material below the foundation into layers to a depth below the influence zone of the foundation.

3. Assign \( E_0' \) as initial drained Young’s modulus for each layer. \( E_0' \) may be calculated from \( G_0 \) and Poisson’s ratio \((\nu')\) as \( E_0' = 2(1 + \nu')G_0 \).

4. Quantify the maximum contact stress \((q)\) between the foundation and the soil, as well as the number of load steps to be used.

5. Use Boussinesq’s theory to calculate the vertical \((\Delta \sigma_z')\) and horizontal \((\Delta \sigma_r')\) effective stress increment at the centre of each layer beneath the centre of the foundation. For a circular foundation with radius \( (R) \) and uniform contact stress increment \((\Delta q)\), the effective vertical and horizontal

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**Table 4 Proposed method softening function variables**

<table>
<thead>
<tr>
<th>Softening function</th>
<th>Variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose sand</td>
<td>Medium-dense sand</td>
</tr>
<tr>
<td>( y_r )</td>
<td>0.001</td>
</tr>
<tr>
<td>( y_s )</td>
<td>0.005</td>
</tr>
<tr>
<td>( a )</td>
<td>0.350</td>
</tr>
</tbody>
</table>
stress increments for a load step are calculated as follows:

\[ \Delta \sigma' = \Delta q \left( 1 - \frac{1}{1 + \left( \frac{R}{z} \right)^2} \right) \]  
\[ \Delta \sigma' = \frac{\Delta q}{2} \left[ (1 + 2\nu) - \frac{2(1 + \nu)}{1 + \left( \frac{R}{z} \right)^2} \right] \]  
\[ \Delta \sigma' = \frac{\Delta q}{3} \left[ (1 + 2\nu) - \frac{2(1 + \nu)}{1 + \left( \frac{R}{z} \right)^2} \right] \]  

6. Calculate the vertical strain increment (\( \Delta \varepsilon_v \)) for the first load step for each layer using the appropriate \( E' \) for each layer:

\[ \Delta \varepsilon_v = \frac{\Delta \sigma' - 2\nu \Delta \sigma'}{E'} \]  

Calculate the shear strain increment assuming axis-symmetrical conditions:

\[ \Delta \varepsilon_s = \frac{2}{3} \Delta \varepsilon_v (1 + \nu) \]  

7. From the strain in each layer, and using an appropriate softening function, calculate the Young's modulus value at the end of the load step. This is the new Young's modulus for the next load step.

8. The process is repeated until the maximum applied contact stress is reached.

9. The change in thickness of each sub-layer is the layer thickness \( (H_i) \) multiplied by the total vertical strain \( (\varepsilon_i^v) \) for the layer. And the total settlement is the sum of the change in thickness of all \( i \) sub-layers:

\[ \text{Total settlement} = \sum_i (H_i \varepsilon_i^v) \]  

10. The load-settlement curve is constructed from the settlement for each load increment.

It should be noted that no specific stiffness degradation curve is associated with the method, and an appropriate stiffness degradation curve may be chosen by the analyst. For this study, the hyperbolic softening function shown in Equation 1 was used. Since the load-settlement data was measured for different density sands, the variables for the softening function could be calibrated and the accuracy of the method assessed.

**Load-settlement prediction**

For each of the results shown in Figure 6, the proposed methodology was used to predict the stress-settlement curves. For the analyses presented in this paper, 1000 load increment steps were used, with the maximum contact stress the same as the measured stress at a settlement of 0.1D. For the hyperbolic relationship shown in Equation 1 the elastic threshold strain \( (\gamma_e) \) was fixed at 0.001% strain, and the other two variables of the relationship \( (\gamma_r \text{ and } n) \) were adjusted with the aim of finding the best fit between the measured and predicted curves. Figure 7 shows the results of the predicted and measured load-settlement curves. Table 4 shows the softening function variables for the different densities, and the softening functions are graphically represented in Figure 8. The measured and predicted curves agreed closely for the loose sand, and the level of agreement decreased as the density of the sand increased.

**Prediction accuracy**

It is clear from Figure 7 that there is good agreement between the 20% RD measured and predicted results. The accuracy does, however, decrease with increasing soil density. In order to assess the accuracy of the predicted results, the error between the measured and predicted settlement values was quantified with the following equation:

\[ \text{Error} = \left( \frac{\text{Predicted settlement} - \text{Measured settlement}}{\text{Measured settlement}} \right) \times 100 \]  

**Figure 7** Predicted and measured load-settlement results

**Figure 8** Proposed softening functions
At small contact stresses the denominator in Equation 8 is small, and the equation becomes numerically unstable. Therefore only error values for applied contact stresses above 50 kPa were considered, since foundations are in most cases designed for loads above this value. The error values are shown in Figure 9. Positive errors indicate an over-estimation of the predicted settlement and negative errors indicate underestimation. As was seen from the load-settlement results shown in Figure 7, the maximum error increased with increasing soil density. The error ranged between -12% and 3% for loose sand, between -22% and 11% for medium-dense sand, and between -30% and 27% for dense sand.

The differences in accuracy may be as a result of different mechanisms dominating when shallow foundations settle in loose and dense soils. It has been shown by numerous authors (e.g. Das 2009) that the settlement mechanisms below shallow foundations are significantly different for loose soils for which volumetric strains dominate (punching failure) and dense soils where shear strains are concentrated in shear bands, as assumed in classical bearing capacity plasticity analysis. The prediction model adopted in this paper is an elastic calculation that only requires stiffness as input parameter. This approach may be more applicable to a failure mechanism corresponding to loose soil, and therefore the increased accuracy. Due to the bearing capacity failure mechanism, settlement prediction on dense soil may require a strength component to be incorporated in the prediction model to increase the prediction accuracy. These are, however, only postulations and further research is required to fully understand the reason for the difference in accuracies.

CONCLUSIONS

Engineers continually strive to find methods which predict settlement of shallow foundations more accurately, because foundation settlement is often the governing aspect in design. With in situ and laboratory seismic testing becoming increasingly popular, settlement prediction methods utilising the initial small-strain stiffness are also becoming more common.

The objective of this study was to develop a foundation settlement prediction method that only requires the small-strain stiffness profile \(G_0\) or \(E_0\) as input soil parameters, and which may be applied to settlements of up to 10% of the foundation diameter (i.e. 0.1D). Centrifuge tests were conducted on an equivalent 5 m diameter circular shallow foundation using three different density sands to obtain measured load-settlement curves. The different density sands were loose, medium-dense and dense sands, and bender and extender elements were used to determine the small-strain stiffness profile below the foundation.

A non-linear stepwise method was proposed to predict the load-settlement behaviour of the foundation. The method requires a stiffness degradation curve, but is not bound by a specific stiffness degradation curve. Results were presented using a hyperbolic stiffness degradation curve for which the variables were calibrated to obtain the best fit for the different density sands.

From the results it is clear that the accuracy of the proposed method decreases as the density of the sand increases, and was found to be within 12% for loose sand, 22% for medium-dense sand and 30% for dense sand. The accuracy of the method should be judged in the context of the few input parameters required, the large range of settlement predictions up to 0.1D, as well as the large spectrum of soil densities for which it is applicable.
REFERENCES


Some shortcomings in the standard South African testing procedures for assessing heaving clay

P R Stott, E Theron

Design of foundations for most light structures in South Africa, and in particular for low-cost housing, relies heavily on particle size analysis and the determination of Atterberg limits. The tests for these properties are currently performed in commercial materials testing laboratories using the procedures of the CSIR’s Technical Methods for Highways Part 1 (TMH1) (CSIR 1986). SANS 3001 (SANS 2011) is being phased in to replace TMH1. Both are primarily concerned with road construction. Investigations at the Central University of Technology indicate serious shortcomings in both of these norms in the context of foundation design for light structures.

Highly plastic material is not usually used as road construction material, and these methods may be adequate to simply identify material so plastic that they should not be used in road construction. Structural foundations, particularly for low-cost housing, do not usually have this option; it is essential to estimate the actual heave potential. This investigation suggests that some of the changes proposed in SANS 3001 may be beneficial for heave assessment, but the most likely application of SANS 3001 could be unsatisfactory in many cases.

INTRODUCTION

Semi-arid and dry sub-humid areas in South Africa and many other parts of the world are noted for heaving foundation problems. Intense seasonal rainfall and lengthy intervals of drought lead to soils alternating between desiccation and saturation. Some types of clay minerals, particularly the smectite group, change volume powerfully with change in moisture content.

The behaviour of clay depends on its physical and chemical make-up. Detailed analysis of the structure and composition of clay is time-consuming and requires sophisticated apparatus and highly skilled personnel. Experience has shown, however, that valuable insights can usually be gained by simple tests performed with inexpensive equipment. The tests on which most engineers in South Africa make their assessments are detailed in the CSIR’s Technical Methods for Highways Part 1 (TMH1) (CSIR 1986). SANS 3001 Civil Engineering Test Methods (SABS 2011) is being phased in to replace the TMH1 methods. The SANS 3001 procedures are similar to the TMH1 methods with a number of modifications. There is considerable resistance to the introduction of SANS 3001, since some of its procedures are commercially unattractive.

The methods examined here are:

- Plastic Limit and Plasticity Index – TMH1 Method A3 (SANS 3001 GR 10, GR11, GR12)
- Linear Shrinkage – TMH1 Method A4 (SANS 3001 GR 10).

Note: Linear Shrinkage determination was removed from TMH1 in the latest edition, but it has remained as one of the “foundation indicators” (TMH1 (1986) A1 – A6) offered by commercial laboratories. Its re-introduction in SANS 3001 GR10 is stated to be for assessing the shrinkage product of wearing course gravels.

The methods of preparing samples on which these tests are performed (TMH1 A1(a), SANS 3001 GR1, GR2) play a vital role in the effectiveness of the tests, and some aspects of their influence are considered in the section below titled “PREPARATION OF MATERIAL.”

It appears possible that the above methods (in TMH1 form), if strictly followed, may give acceptable results in highway applications. Highly plastic clay is usually rejected as a construction material in highway construction. The TMH1 and SANS 3001 tests may be adequate for simply identifying materials so plastic that they must be rejected or given special treatment.

Light structures in general, and low-cost houses in particular, have limited prospects for removal of problem material. Removal is too expensive and the engineer usually has to design foundations which can cope with...
soil, tested in an accredited materials laboratory using their normal preparation method (stated to be TMH1 A1(a)) gave an LL of 50 and a PI of 25. When the commercial values were used in the heave prediction methods of Van der Merwe (1964) and Savage (2007) the estimated heave was approximately half that given by the "raw sample" values.

Reasons for the apparent anomaly were sought. One possible source of error is the Casagrande apparatus specified by both TMH1 and SANS 3001 for the determination of the Liquid Limit. It is widely considered to be susceptible to operator bias, and has been replaced by falling cone apparatus in the standards of many countries. The possibility of using this apparatus in South Africa was examined by Sampson and Netterberg (1984). However, correlations with TMH1 and other standards were not always straightforward, preparation of samples for testing had certain disadvantages and the proposed methods were not preferred by testers. The fall cone was not adopted in SANS 3001. Its suitability for South African use is currently being reinvestigated at the Central University of Technology (CUT) in Bloemfontein.

To gain an approximate measure of operator bias and other possible sources of error in the current procedures a team of six testers tested a range of clay soils at the geotechnical laboratory of CUT. The number of testers were reduced to four after the general magnitude of operator influence had been established. These tests suggest that there are, indeed, deficiencies in the usual implementation of the TMH1 procedures, but operator error cannot account for their magnitude. Some of the deficiencies have been addressed in the SANS 3001 methods, but others not.

It is proposed here to examine the more familiar TMH1 methods first and then consider the implications of the SANS 3001 modifications.

PREPARATION OF MATERIAL

Preparation of material for grading and indicator tests is specified in TMH1 Method A1(a) and SANS 3001 GR1, GR2, and GR5. Since the commercial "foundation indicator tests" employ Method A4 from TMH1 (CSIR 1986) this is the version considered here. Method A1(a) includes steps 3.4 (Boiling and washing) and 3.5 (Drying and disintegration of fines). Personal discussions with laboratory personnel, and comparison of results from commercial laboratories with those obtained at CUT and at the Soil Science Department of the University of the Free State suggest that these two steps may be omitted in many cases. These steps take time, laboratory space and energy. This investigation found that in many cases this will lead to only small differences in the Atterberg Limits. In other cases, however, the omission of these steps may lead to under-estimation of the Atterberg Limits, as well as under-estimation of the clay fraction (see the section below titled "INFLUENCE OF PREPARATION PROCEDURES".

Jacobsz and Day (2008) raised concerns about the reliability of results from commercial laboratories, and noted at least one laboratory estimating Linear Shrinkage by dividing the PI by two. If shortcuts are being taken in the preparation of samples and the execution of tests, then other properties could also be unreliable.

SANS 3001 describes three different methods of preparation – GR1 for wet preparation, GR2 for dry preparation and GR5 for wet preparation at low temperature. GR5 is a recognition that the normal preparation procedures may be unsatisfactory, but its implementation is optional "when it is expected that heating of the fines will significantly alter their properties". No mention is made of who is responsible for the decision to employ this procedure. While one would expect the engineer to take this responsibility, it is common practice for engineers to rely on the laboratory staff to make this decision.

The GR5 procedure is very time-consuming. In the case of high-plasticity clays, the procedure typically requires three days in the oven at 45°, or one week or more out of the oven. While the oven is set at 45° it cannot be used for its normal purposes, and basins of suspended fines take up a large amount of space. This method of preparation would therefore involve higher costs than GR1 or GR2, leading to the following considerations:

1. It is unlikely that the testing technician would be allowed to decide to do such a protracted test.
2. The laboratory manager could incur the displeasure of his client if he presented a bill more expensive than expected.
3. To prove that the procedure was necessary, it would be necessary also to prepare and test samples using method GR1 and demonstrate a significant difference.
4. Few clients are able to assess heat sensitivity and actually specify the procedure.
5. Most clients are very concerned about keeping costs low.

Such considerations suggest that method GR5 will not be performed by the majority of South African commercial laboratories, certainly not for low-cost housing projects.

ATTERTBERG LIMITS

Liquid Limit

The LL is determined by TMH1 Method A2, Determination of the Liquid Limit of soils by means of the flow curve method. This method describes the determination of the flow curve by three tests with the Casagrande apparatus.

The following note appears at the end of the method: "It has been found that the Liquid Limit of certain materials is influenced by the mixing time. ... Hence it was considered necessary to stipulate a mixing time, and a period of ten minutes was decided upon."

This key point will be addressed in the section below titled "Testing of samples". Following the specification of the flow curve method is clause 5.2 (One-point method), in which it is noted that the Liquid Limit may also be determined using only one test. There is the proviso in clause 5.6 which reads: "In the case of dispute the flow curve method shall be the referee method."

Commercial laboratories strive to make their services competitive and affordable to their clients; they therefore almost exclusively opt for the one-point procedure. They also interpret the note concerning the mixing time of ten minutes to mean that this is
the total time, from adding the first drop of water to the fines powder until testing in the Casagrande cup.

SANS 3001 details three different tests for Liquid Limit determination — a one-point method (GR10), a two-point method (GR11) and the three-point flow curve method (GR12). There is close similarity to the TMH1 procedures and there is even stronger emphasis on the mixing time limit of ten minutes. It is recommended in the introductions to both GR10 and GR11 that the three-point flow curve should be used where a PI greater than 20 is expected, but it is not specified who makes this decision, nor is it specified as the referee method. Similar considerations to those which make the use of GR5 very unlikely apply to GR12. Most laboratories will probably continue to perform the one-point method in all cases. The most common method of estimating heave is that due to Van der Merwe (1964). This method relies on the PI, which at the time the method was devised was always deduced from the three-point flow curve, and there was no ten-minute time limit.

Plastic Limit and Plasticity Index
TMH1 Method A3 and SANS 3001 GR10 specify similar procedures for determining the PL. Both specify the use of wet material remaining after LL determination and after filling of the shrinkage trough. This makes it likely that for expansive material the PL will be determined at least thirty minutes after saturation. This has important consequences as presented in the sections below titled “RESULTS AND OBSERVATIONS” and “Testing of samples”.

Linear Shrinkage
TMH1 Method A4 and SANS 3001 GR10 both specify placing the trough in the oven after filling. GR10 specifies that this should be done immediately after filling (section 6.2.2.1) and it emphasises that there should be no air drying before placing in the oven. This is addressed in the sections below titled “RESULTS AND OBSERVATIONS” and “DISCUSSION AND COMMENTS”.

TESTS PERFORMED TO ASSESS THE RELIABILITY AND CONSISTENCY OF SPECIFIED PROCEDURES
Samples were prepared according to TMH1 Methods A1(a) with and without steps 3.4 and 3.5, i.e. with and without boiling, washing, drying and disintegration (corresponding to SANS 3001 GR1 and GR2). The samples were prepared from oven-dried (105°C–110°C) material passing the 0.425 mm sieve. Comparison of commercial results with results of tests performed at both CUT and the soil science department of the Free State University, suggested that steps 3.4 and 3.5 were probably omitted in the commercial procedures. Comparisons with commercial values shown here are from tests on samples where these two steps were omitted unless specifically noted. Tests performed on samples of which the preparation included these two steps usually showed only small differences in Atterberg Limits, but sometimes large differences in clay fraction. A range of soils, mostly from the central Free State, were used. These samples were drawn from one major geotechnical investigation and several smaller investigations in the central Free State and a handful of samples from the Western and Northern Cape and Gauteng. Screening tests on about sixty of the samples showed acceptable agreement with commercial laboratory Atterberg Limits for predominantly sandy soils, while clayey soils gave less good agreement. From approximately 200 available samples, thirty predominantly clay samples covering a wide range of plasticity, location and sampling depth were selected for detailed testing.

Five samples were used for comparison of the one-point and the flow curve methods. In the case of very plastic clays the discrepancy was considerable.

It was suspected that at least part of this difference could be due to time dependence. The flow curve method provides more time for the sample to absorb water than the one-point method does. Tests were therefore performed to monitor this time dependence for a range of 22 clayey soils – 20 from the central Free State, one from the Northern Cape, and one from the Western Cape.

The procedure involved rapidly adding an amount of water commensurate with the commercial value of LL at the start of the test. This ensured that all of the dry fines came into contact with water at the start of the mixing process. In the commercial procedure water is added slowly and, in the case of very plastic material, some of the fines remain dry far into the mixing process. Mixing was performed (with the addition of more water as needed) until the slurry was suitable for testing in the Casagrande apparatus. The time of adding the first water
and the times of testing were recorded. Each tester determined the Liquid Limit four, five or six times. The first result was obtained as quickly as possible, the remainder were spaced over about one hour.

In the case of highly plastic material it was noted that water was quickly absorbed by the fines. In transferring material from the mixing bowl to the Casagrande cup for the first test of each series it was not uncommon for the slurry to have stiffened beyond the point where a successful test could be made. It needed considerable practice to achieve a result in less than five minutes.

After Liquid Limit tests the remainder of the paste was used for Linear Shrinkage and Plastic Limit tests.

RESULTS AND OBSERVATIONS

Figure 1 shows results of three-point tests for a firm, grey-olive, residual clay from the central Free State compared to one-point results from a commercial laboratory. The three-point results give an LL of 71 ± 2 and a PI of 42 ± 1 as against the commercial laboratory value of an LL of 50 and a PI of 24.

Table 1 summarises the results for the five clays tested.

Figure 2 shows typical plots of PI against time for four clays from the central Free State. Trend lines and upper and lower envelopes have been added. The trends of the envelopes and the best-fit (logarithmic) curves suggest an initial rapid increase in PI slowing noticeably after about 20 minutes. Rapidly changing PI appears to be accompanied by increased scatter. Results from individual testers suggest that values may not change smoothly with time.

Less plastic clays were found to give generally less time dependence, and also less scatter in results. Results from a non-expansive, low CEC, low suction-potential clay from the Western Cape can be seen in Figure 3.

The small scatter allows the possibility of insight into operator bias. No attempt was made to rigorously assess this factor, but it appears to be too small to account for the observed discrepancies.

For the 22 samples tested the apparent increase in PI above the commercial value by the end of the test was less than 5 in eight cases, between 5 and 10 in nine cases and greater than 10 in five cases.

Plastic Limits were found to correspond quite well with those from commercial laboratories. Saturated material remaining after completion of the LL procedure is used for PL determination; the time involved is probably sufficient to approach a stable moisture condition in most cases. No time dependence was noted.

It was found that Linear Shrinkage of highly plastic clays was poorly assessed using...
the stated procedure of placing the trough in the oven immediately after filling. Plastic clays tend to shatter and arch to such an extent that reconstructing the sample for final measurement is difficult.

Widely different values were obtained for different specimens of the same clay (see Photos 1, 2 and 3).

When samples were left to air-dry for a period these problems reduced significantly. Samples of the same soil left for 24 hours, 48 hours and 76 hours before oven-drying showed that the Linear Shrinkage value and the consistency of results increase with time of air-drying. Samples left to air-dry until shrinkage stopped rarely showed pronounced arching and sometimes reached Linear Shrinkage almost double the value of immediately oven-dried specimens. Air-drying probably models shrinkage in the field far better than oven-drying. Less plastic soils were less seriously affected by immediate oven-drying. While the current LS procedures may be satisfactory for assessing the shrinkage product of wearing course gravels, they are not at all suitable for the foundation indicator tests offered by commercial laboratories (see Photo 4).

**INFLUENCE OF PREPARATION PROCEDURES**

In many cases, particularly for granular soils, it was found that omitting steps 3.4 (*Boiling and Washing*) and 3.5 (*Drying and disintegration of fines*) from the TMH1 A1(a) procedure (equivalent to using SANS 3001 GR2 instead of GR1) made little difference to the Atterberg Limits. In the cases where it did make a difference, the effect on both Atterberg Limits and clay fraction could be very significant. Investigation of these cases usually revealed a fraction of extremely active clays, having very high suction potential and being able to bind fine material to form grains of high strength (tensile strengths greater than 7 MPa were measured in some cases). Grinding of these grains with a rubber-tipped pestle failed to reduce them from gravel to powder in a reasonable time (see Photo 5).

Up to 70% of some soils appeared to be gravel, but with wet preparation much of this material passed the 0.425 mm sieve. One such sample was shown to a commercial laboratory manager.

He assessed it as mudstone gravel not suitable for PI testing. In some cases clay that...
had been separated out during wet preparation was found to have very high LL and PI (LL > 80, PI > 40). This may be significant in view of the warning by Rogers et al. (2013) that the behaviour of any type of soil may be controlled by swelling clay if it contains more than 5% clay by weight.

It was found that plastic materials which show time dependence after oven-drying at 105°C–110°C showed little time dependence after GR5 preparation. This appears to confirm that the time dependence may be due to removal of water from the structure of the clay during oven-drying. Procedure GR5 may be essential to obtain realistic LL and PI for highly plastic clay when the current ten-minute testing time is used, but allowing adequate time for water re-absorption may make GR5 unnecessary. The possibility that oven-drying may permanently alter properties of some clays is currently being investigated.

DISCUSSION AND COMMENTS
Commercial laboratories need to deliver competitive services. Their clients are often under both time and economic pressures. Commercial procedures must therefore be as simple, quick and economical as possible. This investigation suggests that in many cases – particularly for sandy soils – the procedures currently being used by commercial laboratories may give acceptable results. However, in the case of highly plastic material they do not. The SANS 3001 procedures, if followed strictly, would possibly give satisfactory results in most cases. Discussions with commercial laboratory personnel suggest that it is unlikely that these procedures will be followed strictly because of time and cost implications and the discretionary nature of some procedures.
Preparation of samples
Certain materials will only give meaningful Atterberg Limits when wet preparation is performed. Wet preparation is time-, space- and energy-consuming, and it appears that it is not commonly performed by commercial laboratories. In some cases omitting wet preparation may have minor consequences for the Atterberg Limits, and it may only be necessary for high-plasticity materials. Its omission can have a significant influence on the clay fraction determination.

Whether samples are prepared according to TMH1 A1(a), SANS 3001 GR1 or GR2, the specified one-point LL procedures give poor results for highly plastic clays. The use of GR5 (wet preparation at low temperature) appears to have the potential to solve this problem. The economic consequences of the method are such that it is unlikely that GR5 will be used commercially. However, small changes to the normal testing procedure may render GR5 unnecessary.

Testing of samples
Liquid Limit
Some deficiencies in the tests for Atterberg Limits seem to be specifically dependent on the ten-minute mixing time.

This limit may be linked to the findings of Kleyn et al (2009) pertaining to the assessment of decomposed dolerite gravel for use in road construction. Their paper deals with materials of very low plasticity. An increase in PI from 2 to 7 was observed with extended mixing, which was attributed to breaking down of the gravel and exposure of more of the dolerite’s montmorillonite component to water, thus allowing the mineral to exert its swelling potential. The graph of measured PI against mixing time shows great similarity to the normal testing procedure may render GR5 unnecessary.

Plastic Limit
The PI is the difference between the LL and the PL. The PL is determined using material left over from the LL test after the filling of the shrinkage trough. The procedure makes it almost certain that the sample will have been fully saturated for a considerable time before the PL is determined. This investigation found acceptable agreement with commercial laboratory PI values, whether LL-time-dependence was present or not. As a consequence the differences in numerical values of PI and LL are comparable, but the percentage differences are far greater.

For the clay mentioned in the section titled “REASONS FOR THE CURRENT INVESTIGATION” above, the commercial laboratory values were LL 50, PL 25 and PI 25, whereas the values for the undried sample were LL 67, PL 25 and PI 24. The PL differs by only 1, the LL differs by 17 (i.e. 34% above the commercial lab value) and the PI differs by 18 (i.e. 72% above the commercial lab value). As the PI is the most commonly used indicator of potential heave, this discrepancy is very significant.

![Figure 4](image-url)
Proposed amendment to Liquid Limit procedure

A simple amendment to the procedure which meets the ten-minute mixing requirement of TMH1 and SANS 3001 is proposed to address this problem. It is suggested that this could be used until an alternative can be standardised.

It is very easy for an experienced tester to recognise a sample which will have a high LL – it is usually obvious with the first addition of water to the oven-dry fines. While not all soils with high LL show time dependence, all of the clayey soils which this investigation found to be markedly time-dependent do have a high LL. It is proposed that, as soon as a tester suspects a high LL, sufficient water to bring the sample close to the LL should be added in the first five minutes of mixing. The sample should then be covered and set aside for at least 30 minutes (allowing the testing of another sample), after which time the mixing procedure should be resumed for a further five minutes and the remainder of the test continued as normal. Section 6.1.10 of SANS 3001 GR10 should be ignored. The 30-minute waiting period is unlikely to bring the sample to final equilibrium, but it should bring the PI close to the value given by the procedures in use when common heave prediction methods were devised. The economic implications of this proposal would be small, as probably only a small percentage of samples would need this treatment, which in itself is minor. It is far more likely to be embraced by commercial laboratories than the application of SANS 3001 GR5. Simple procedures for identifying soils which need special treatment, and inexpensive procedures for dealing with them reliably and effectively, are being sought. It is hoped that they can be incorporated in an economically viable revision to SANS 3001.

Linear Shrinkage

Soils containing highly expansive clays contract so much under oven-drying conditions that they often bend, shatter and produce very unreliable results in the Linear Shrinkage trough. A trough without a base is specified for the case of materials expected to have PI greater than 20. This type of trough reduces bending and shattering and may give better results for highly plastic soils. But even where such a trough is used, placing a freshly prepared sample of highly plastic clay in the oven at 105° will result in a poor indication of shrinkage potential.

To be of value for the assessment of clays the trough should not be put into the oven until the shrinkage limit has been almost reached. With this procedure the LS gives a consistent, useful indication of shrinkage from Liquid Limit to Shrinkage Limit, which probably models field conditions reasonably well. Linear Shrinkage appears to have potentially more value for heave prediction than is currently being utilised. Paige-Green and Ventura (1999) reported that the Linear Shrinkage test can give valuable insights for low-plasticity road-building materials. Cerato and Lutenegger (2006) demonstrated that it can give a good estimate of the Shrinkage Limit. Investigations at CUT suggest that it can indicate cases where current methods fail to show true heave potential, and give confirmation when they do.

COMPARISONS WITH STANDARDS FROM ELSEWHERE AND PREVIOUS SOUTH AFRICAN FINDINGS

The New York State Department of Transportation’s specification for liquid limit determination (Geotechnical Engineering Bureau 2007) recommends that, where possible, clays should be tested in their natural, undried state (Section 4, Preparation of test samples). Where this is not possible, Note 1 in Section 6.2 states: “Allow ample time for mixing and curing since variation can cause erroneous test results. Some soils are slow to absorb water. Therefore it is possible to add the increments of water so fast that a false liquid limit value is obtained. This is particularly true when the liquid limit of clay soil is obtained from one determination as in the one-point method.”

While this note recognises the effect of time-dependence, it does not point out that it is precisely the most problematic clays which are at issue. There is also no warning that the “false liquid limits” are always underestimates, and that they may be very severe underestimates. There is no guidance on how much time might be “ample time”, but there certainly is a clear warning about the inadequacy of the one-point method for clays. Rather than suggesting that extended mixing time affects the result by breaking down soil particles, the suggestion is that the time required for absorbing water into the clay is the important factor.

Professor J E Jennings, one of South Africa’s most notable soils experts, habitually advised his students to allow “ample time” for mixing and curing – also without specifying just how much time is “ample”.

Indian Standard (IS 2720, Part 5, Reaffirmed 1995) (IS 1985) does give specific guidance. Clause 3.4.1 states: “In the case of clayey soils, the soil paste shall be left to stand for a sufficient time (24 hours) so as to ensure uniform distribution of moisture throughout the soil mass.”

Again this suggests that the time for which the sample is left in contact with water is important, rather than that mixing may break down soil particles. While this specification recognises the possibility of time-dependence and may be a safe way to proceed, it may not be popular with commercial laboratories. Storage of slurried samples for 24 hours needs additional containers, labelling, record-keeping and storage space. The temptation towards shortcuts might be appreciable.

South African standards frequently follow British standards quite closely. BS 1377: Part 2 1990 specifies, in Note 3 of Clause 4.3, a 24-hour period between mixing the soil with water and carrying out the Liquid Limit test.

Blight et al (2012) noted that it is well known that drying (even air-drying) of some soils affects their properties and can lead to underestimation of LL and PI. They recommend that Atterberg Limits should therefore be determined without any drying wherever practicable. This requires testing soil in a state as close as possible to its natural condition, with only minimal processing for removal of large particles. They point out that extended mixing (of soil in its natural condition) can lead to increased LL and PI because of breaking down of cemented bonds between clay clusters. This is a well-known feature of tropical and volcanic soils.

The CUT tests found that, where clay samples can be tested in the raw state (with only removal of large particles where necessary), PI for the whole sample could be considerably greater than that for the fines alone as determined using the TMH1 A2 or SANS 3001 GR10 procedures. Current practice reduces the PI for the whole sample by factoring with the percentage of fines.

SOME THEORETICAL CONSIDERATIONS

Non-expansive clays, typically kaolinite, consist of alternating silicon-based tetrahedral sheets and aluminium-based octahedral sheets. They form a structure in which hydrogen bonds hold the pairs of sheets in stacks of the order of 1000 layers thick. When a soil containing such clay absorbs moisture, the water fills the spaces between the individual soil particles, but does not penetrate between the individual sheets of the clay structure itself. The clay does expand somewhat (despite being called “non-expansive”) since water, when available, is drawn into the sample by capillary action. This capillary suction is, however, not usually strong enough to cause expansion against the pressure under the foundation of a building.
Expansive clays (e.g. smectites), on the other hand, consist of silicon-based tetrahedral sheets and aluminium-based octahedral sheets which have suffered so much corruption to the ideal structure (e.g. by substitution of aluminium$^{3+}$ for silicon$^{4+}$ in the tetrahedral sheets, and magnesium$^{2+}$ for aluminium$^{3+}$ in the octahedral sheets) that the pattern of hydrogen bonds is disrupted and layers become attached largely by electrostatic forces. The number of layers in the stacks of these sheets may be very small, giving a thin, plate-like structure to the clay particles.

When such clay is allowed to take in moisture, not only the pores between individual grains are filled, but water is drawn in between the individual layers, which move apart to admit water-borne cations attracted by the unbalanced electrostatic forces on the sheets. The spacing of layers is of molecular dimensions. The suction is extremely high, so the expansion can take place against a considerable pressure, but the spacing is so small that an appreciable amount of time may be needed for water to be drawn in.

Such considerations lead to the question of how long it takes for water to be drawn into the structure of expansive clays, and whether the procedures of TMH1 and SANS 3001 allow sufficient time for this hydration to take place.

CONCLUSIONS

The TMH1 methods have long been standard in commercial laboratories throughout South Africa. The way these methods are currently being applied is convenient and economical, but unsatisfactory for assessing clays under foundations of light structures.

The introduction of SANS 3001 to replace the TMH methods might address shortcomings in this regard if all of its procedures were strictly employed. The discretionary nature of critical procedures and their cost implications, however, make their employment unlikely.

Introduction of procedures to address the cases where the TMH methods are not satisfactory is suggested as an alternative, at least for the “foundation indicator” tests. It appears that relatively small modifications to the current methods may be sufficient to provide more reliable values for these indicators for active clays. These small changes would have relatively minor economic implications and would be more attractive to commercial laboratories and their clients than SANS 3001 in its current form.

Subsidy-housing is currently allocated approximately R104 per house for geotechnical investigation. The cost of heave damage at one 500-unit housing project visited by the CUT Soil Mechanics Research Group in 2013 was more than R14 000 000. A realistic allocation of funds for geotechnical investigations is required if the investigations are to be carried out by engineers or engineering geologists with sufficient skill and experience to make proper assessment of the tests to be performed and the procedures to be followed, and then to draw valid conclusions from the results.

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Adjudication in South African construction industry practice: towards legislative intervention

M J Maritz, V Hattingh

Adjudication in South African construction practice has, through various initiatives of the South African government and Construction Industry Development Board, the increased use of international standard form construction contracts, and the South African High Court’s robust approach in enforcing adjudicators’ decisions, become relatively commonplace in both the public and private sectors as the first tier in dispute resolution procedures on construction projects across the South African construction industry.

This paper considers several judgements of the South African courts dealing with adjudication and certain of the South African government and Construction Industry Development Board’s initiatives which, together with South African construction industry adjudication practice, combine in solidifying a foundation for the implementation of a legislative framework underpinning the application and practice of adjudication in the South African construction industry.

INTRODUCTION

“IT certainly seems that construction contracts go wrong; everybody knows that. It is one of the problems of construction. The problems have intrigued, one might say obsessed, the industry and government for 50 years.” (Fenn 2002).

Since 1995 the post-apartheid South African government has similarly been obsessed with the pursuit of procurement reform, especially in introducing appropriate methods for effective dispute resolution into the construction industry. Recognising the entrenchment of alternative dispute resolution (ADR) procedures for resolving labour disputes in the Labour Relations Act No 66 of 1995 and successful application of ADR procedures in the private sector, the White Paper on Creating an Environment for Reconstruction Growth and Development in the Construction Industry commits the public sector to promoting the application of ADR procedures, in particular adjudication, in the South African construction industry.

In promoting adjudication as the first tier in managing disputes throughout the South African construction industry, the White Paper confirms that “... recommendations adapted largely from the Latham report will be introduced to the construction industry, specifically for public-sector contracts.” Latham (1993), among other matters, “… recommended that a system of adjudication should be introduced within all the Standard Forms of Contract (except where comparable arrangements already exist for mediation or conciliation) and that this should be underpinned by legislation.”

ADJUDICATION IN SOUTH AFRICAN CONSTRUCTION INDUSTRY PRACTICE

Adjudication has long been part of the panoply of ADR procedures available to parties bound by construction contracts, but until recent years was far from universal, and if the case law that refers to it is anything to go by, was not greatly used. Where adopted by the parties it was by express agreement in writing and contained an ad hoc set of rules that differed from contract to contract (Gaitskell 2011).

In addition to the South African government’s interventions in promoting adjudication in South African construction practice, the industry itself has largely embraced the procedure “… whereby the parties agree to confer jurisdiction on an adjudicator to decide the particular dispute that has arisen between them ...” (Coulson 2007) as a means “… to find some sensible resolution of their problem and then get back to their real business ...” (Jackson 2006). As a matter of practice in the South African construction industry, the obligation to adjudicate, however, only arises consequent on a specific agreement to adjudicate, which agreement is recorded in the dispute management mechanisms captured in the particular construction contract.

Contractual adjudication has for some time now found a place in standard form contracts for public-sector contracts.”

Keywords: alternative dispute resolution, construction industry, construction adjudication, legislative intervention
building contracts in use in South Africa, such as the Joint Building Contracts Committee’s (JBCC) Principal Building Agreement (JBCC 2014) and the General Conditions of Contract for Construction Works (SAICE 2010), into which the adjudication process was introduced for the first time in 2004. A thorough knowledge of adjudication procedures and practice and implementation has now become essential for any construction professional playing a certifying, advisory or commercial role in a construction project.

Bvumbwe and Thwala (2011) conducted a study to determine which of the spectrum of ADR procedures (including specifically mediation and adjudication) are most frequently deployed through the South African construction industry in resolving construction disputes. They concluded that, although “... mediation is the most frequently used method in resolving disputes in the construction industry ... the majority of respondents would prefer the inclusion of adjudication as the priority in resolving a dispute before arbitration.”

Van der Merwe (2009) conducted a comparative study of the application of both mediation and adjudication across the South African construction industry to determine which of the two dispute resolution methods is better suited to resolve construction disputes in this industry. In concluding that adjudication is preferable, Van der Merwe states “... that both mediation and adjudication are effective alternative methods of dispute resolution as to litigation and arbitration. Although adjudication has a weakness in the enforceability of the decision of the Adjudicator, it still has an advantage over mediation.”

Maritz (2007) reviewed the development of adjudication in the South African construction industry, considering its effectiveness in resolving construction disputes, and the extent to which adjudication has been utilised since its introduction into this industry, and concludes that “... experience in other countries who have introduced adjudication has shown that adjudication without the statutory force is not likely to be effective. Enforcement of the adjudicator’s decision is critical to the success of adjudication, and before South Africa introduces an Act similar to Acts such as the Housing Grants, Construction and Regeneration Act 1996 (UK), the Construction Contracts Act 2002 (NZ) or Building and Construction Industry Security of Payment Act 2004 (Singapore), adjudication will remain largely ineffective and, therefore, underutilised in the South African context.”

Gaitskil (2007), echoing Maritz’s observations, argues that “… in order for adjudication to have any real impact, it had to be compulsory so that powerful employers or main contractors could not simply strike such clauses out of contracts they made. This meant that there had to be legislation which simply imposed adjudication on all parties in the construction industry.”

Following an investigation into adjudication practice in the South African construction industry, Maiketo and Maritz (2009) concluded “… that adjudication has found acceptance in the South African construction industry. However, it still has some way to go before its potential can be realised in full. Certain challenges need to be overcome to enable this to happen, which range from the contractual, institutional and legislative framework, to matters of skills and training.”

THE SOUTH AFRICAN HIGH COURT’S INTERVENTION IN ADJUDICATION PRACTICE

Maritz (2007) correctly observes that “… enforcement of the adjudicator’s decision is critical to the success of adjudication.” In the United Kingdom (UK) neither the Housing Grants, Construction and Regeneration Act (HGCRA) nor the Scheme for Construction Contracts (the “Scheme”) (enacted under the HGCRA) entrenches a procedure for enforcing adjudicators’ decisions. The HGCRA simply provides that adjudicators’ decisions are binding unless and until overturned by agreement, arbitration or litigation. Paragraph 23 (2) of the Scheme similarly provides that the decision is binding, pending final resolution by agreement, arbitration or litigation. The absence of an enforcement mechanism entrenched in the legislation itself was initially perceived as a critical flaw in the legislation. Fortunately the English courts have consistently adopted a robust approach in enforcing adjudicators’ decisions made through the statutory regulated adjudication procedure ensuring that Parliament’s intention in introducing the legislation is not thwarted.

In both Basil Read (Pty) Ltd v Regent Devco (Pty) Ltd and Freeman, August Wilhelm NO, Mathewula, Trihani Sitos de Sitios NO v Eskom Holdings Limited the High Court of South Africa has exhibited a clear willingness to adopt a similarly robust approach to enforcement of adjudicators’ decisions.

The South African High Court’s initial willingness to adopt such a robust approach to the enforcement of adjudicators’ decisions has been reinforced through two recent decisions in the High Court of South Africa, namely: in an unreported judgement of the South Gauteng High Court on 3 May 2013 handed down by D T v R du Plessis A J in Tubular Holdings (Pty) Ltd v DBT Technologies (Pty) Ltd and in another unreported judgement of the South Gauteng High Court handed down by Spilg J on 12 February 2013 in Esor Africa (Pty) Ltd / Franki Africa (Pty) Ltd JV v Bombela Civils JV.

In Tubular Holdings (Pty) Ltd v DBT Technologies (Pty) Ltd several disputes arising in connection with a subcontract between Tubular Holdings and DBT Technologies on the Kusile coal-fired power station project had been referred to a Dispute Adjudication Board (DAB) consisting of a single member who had furnished a decision on the disputes referred. Tubular Holdings thereafter made application to the South Gauteng High Court by motion application for an order compelling DBT Technologies to comply with the DAB’s decision.

The kernel of the issue between the parties before Du Plessis A J related to the interpretation of the standard clause 20.4 of the FIDIC Conditions of Contract 1999, First Edition (FIDIC 1999). Du Plessis A J summarised the dispute as follows at paragraph [5]: “The applicant submits that the parties are required to give prompt effect to the decision by the DAB which is binding unless and until it is set aside by agreement or arbitration following a notice of dissatisfaction whereas the respondent says that the mere giving of a notice of dissatisfaction undoes the effect of the decision.”

In granting Tubular Holdings an order for specific performance compelling DBT Technologies to comply with the DAB’s decision Du Plessis A J, specifically in regard to clause 20.4 of the FIDIC 1999, held at paragraph [14] that “[T]he scheme of these provisions is as follows: the parties must give prompt effect to a decision. If a party is dissatisfied he must nonetheless live with it but must deliver his notice of dissatisfaction within 28 days failing which it will become final and binding. If he has given his notice of dissatisfaction he can have the decision reviewed in arbitration. If he is successful the decision will be set aside. But until that has happened the decision stands and he has to comply with it.”

In Esor Africa (Pty) Ltd / Franki Africa (Pty) Ltd JV v Bombela Civils JV a dispute arose in connection with certain construction works executed by Esor Africa / Franki Africa JV relating to certain piling and lateral support work on the Gautrain rapid rail link project. The dispute referred to a DAB also consisting of a single member in accordance with clause 20.4 of the FIDIC
J999. The DAB had furnished a decision on the dispute referred. Esor Africa / Franki Africa JV thereafter made application for an order for specific performance compelling Bombela Civils JV to comply with the DAB’s decision.

The dispute between the parties before Spilg I fell to be resolved by “a proper interpretation of the dispute resolution clauses dealing with the effect of a DAB decision” (refer to paragraph [7]).

In granting the Esor Africa / Franki Africa JV the order for specific performance Spilg I concluded at paragraph [13] that, “[I]n order to give effect to the DAB provisions of the contract the respondent cannot withhold payment of the amount determined by the adjudicator, and in my view is precluded by the terms of the provisions of clause 20 (and in particular clauses 20.4 and 20.6) from doing so pending the outcome of the arbitration. In my view it was precisely to avoid this situation that the clauses were worded in this fashion.”

The High Court of South Africa’s robust approach in enforcing adjudicators’ decisions is succinctly summarised by Spilg I in Esor Africa / Franki Africa JV v Bombela Civils JV at paragraph [15] as follows: “The court is required to give effect to the terms of the decision made by the adjudicator. The DAB’s decision was not altered and accordingly it is that decision which this court enforces.”

In Sasol Chemical Industries Ltd v Odell and another10 (the first South African court case dealing with an application to set aside an adjudicator’s determination as opposed to enforcement of an adjudicator’s determination) Kruger J considered an urgent application by Sasol Chemical Industries Ltd (Sasol) to set aside an adjudicator’s award on the basis that the adjudicator did not entertain a request by Sasol for an extension of time to furnish information. The adjudication had proceeded in accordance with the provisions of the New Engineering Contract, Third Edition (ICE 2005), Engineering Construction Contract Option W1. Sasol had failed to furnish information in response to E – Hel Services (Pty) Ltd’s (the second respondent) submission within the strict time limits prescribed in clause W1.3 (3), and thereafter to conclude an agreement with E – Hel Civil Services and the adjudicator to extend the time limits. Sasol then applied to the adjudicator to grant an extension of time to the prescribed time limits within which to furnish information. The adjudicator refused to grant Sasol’s request and proceeded to furnish his determination on 3 February 2014.

In refusing Sasol’s application to set the adjudicator’s determination aside Kruger J confirmed the High Court’s robust approach holding that “[A]djudication is meant to be a speedy remedy to assist cash flow and not to hold up the contract. The finding of the adjudicator stands until it is set aside by the tribunal. The remedy of the applicant is to place its case before the tribunal. Even if in this case the adjudicator may have made a mistake by not entertaining the request of the applicant for an extension of time (and I do not think the adjudicator made a mistake) the adjudication stands.”11

In Radon Projects v N V Properties12 the South African Supreme Court of Appeal considered the court a quo’s order that an appointed arbitrator had no jurisdiction to arbitrate the referred dispute, as the dispute had arisen prior to practical completion and was as such required to be submitted to adjudication in the first instance, in accordance with clause 40 of the Fourth Edition of JBCC (JBCC 2004).13

In reversing the court a quo’s decision Nugent J A (delivering a unanimous judgement) squarely confirmed adjudication’s place in South African construction dispute management practice, concluding that “[W]hen read together with the Rules, I think it is plain that, in keeping with modern practice internationally, adjudication under clause 40 is designed as a measure for the summary and interim resolution of disputes, subject to their final resolution by arbitration where appropriate.”14

The South African Courts’ consistent willingness to adopt a robust approach to enforcing adjudicators’ decisions has contributed significantly toward securing the increasing adoption of adjudication into South African jurisprudence and construction practice as a first tier dispute management procedure and the development of a solid foundation for a legislative framework to underpin the procedure.

THE CIDB DRAFT PROMPT PAYMENT AND ADJUDICATION REGULATIONS

A form of statutory adjudication has already found a seat in South African legislation through Part F (Companies Tribunal adjudication procedures) of the Companies Act15 which provides opportunity to parties (as opposed to a statutory obligation16) to refer disputes arising under or in connection with the application of the Companies Act to a public authority known as the Companies Tribunal for resolution. The Companies Tribunal is specifically prescribed when adjudicating referred disputes to “…conduct its adjudication proceedings contemplated in this Act expeditiously in accordance with the principles of natural justice17 and … "may conduct those proceedings informally…”18

Statutory adjudication is, consequent to the enactment of Part F (Companies Tribunal adjudication procedures) of the Companies Act19, no longer entirely foreign to South African jurisprudence – both the South African government and construction industry have recognised the proven effectiveness of such systems internationally, and the South African courts have exhibited a definite willingness to enforce an adjudicator’s decision.

Statutory adjudication was first introduced into the UK through enactment of Part II of the HGCRA which came into force in May 1998. The Local Democracy Economic Development Act, 2009 subsequently effected changes to the adjudication and payment provisions contained in the HGCRA.

Three years after enactment of the HGCRA the state of New South Wales enacted the Building and Construction Industry Security of Payment Act, 1999 (the NSW Act), modelled on the HGCRA. The NSW Act served as the model upon which most other Australian jurisdictions, to varying degrees, based their construction contracts legislation, culminating in the Tasmanian Act which received Royal Assent on 17 December 2009. Other states and territories across Australia, including Victoria20, Queensland21, Northern Territory22, Western Australia23, Australian Capital Territory24, South Australia25 and Tasmania26, have each enacted security of payment legislation. Similar legislation has subsequently been enacted in several jurisdictions, including (among other jurisdictions) Singapore27, and most recently Malaysia28.

The CIDB has made a concerted effort to overcome the challenges referred to by Maiketo and Maritz (2009), and by initiating the procedure stipulated in section 33 (Regulations) of the CIDB Act 38 of 2000 the CIDB is building upon the foundations for such legislative intervention being laid through the South African Court intervention and industry adjudication practice.

An internal task team was set up for this purpose by the Director General of the Department of Public Works. The work of the task team in preparing a revised set of draft regulations is now concluded. The draft regulations consist of Part IV C titled “Prompt Payment” and Part IV D titled “Adjudication” (the “draft regulations”) including a Standard for Adjudication (the “Standard”). Although the CIDB Board has not approved the final revised set of draft regulations, they approved the process at this
stage. Once the regulations have been finalised after the mandatory public comment phase, they shall be submitted to the Board for final approval.

By application of sub-paragraph (1) of regulation 26 P (Right to refer disputes to adjudication) of Part IV D (Adjudication) a mandatory form of statutory adjudication will be introduced into the South African construction practice. Sub-paragraph (1) of regulation 26 P (Right to refer disputes to adjudication) of Part IV D (Adjudication) provides:

(1) Every construction works, or construction works-related contract, must provide for an adjudication procedure, which must substantially comply with these Regulations and if that contract does not contain such a procedure, or in the case of a verbal contract, the provisions of this Part and the Standard for Adjudication, apply to that contract.

Sub-paragraph (1) requires the parties to any written construction works or construction works-related contract to include an adjudication procedure into the contract.

In the event that the adjudication procedure provided for in the express terms of the contract does not substantially comply with these Regulations then by default the provisions of Part IV D (Adjudication) together with the “Standard” will apply automatically.

Similarly, if the parties conclude any oral construction works contract or construction works-related contract then (in the absence of express terms recorded in writing) by default the provisions of Part IV D (Adjudication) together with the “Standard” will apply automatically.

CONCLUSION

The South African courts’ robust approach to enforcing adjudicator’s decisions, initiatives of both the South African government and CIDB particularly, coupled with the industry’s persistent application of contractual adjudication procedures, are reinforcing a proper foundation upon which to implement a legislative framework to underpin adjudication practice in the South African construction industry.

The proposed legislative framework will (once implemented) solidify a desperately needed “… speedy mechanism for settling disputes in construction contracts on a provisional interim basis and requiring the decision of adjudicators to be enforced pending the final determination of disputes by arbitration, litigation or agreement …” into South African jurisprudence and construction industry practice, therefore significantly contributing towards “… delivery, performance and value for money, profitability and the industry’s long-term survival in an increasingly global arena …”.

NOTES

1 The Labour Relations Act No 66 of 1995 was enacted to, inter alia, provide simple procedures for the resolution of labour disputes through statutory conciliation, mediation and arbitration (for which purpose the Commission for Conciliation, Mediation and Arbitration was established), and through independent ADR services accredited for that purpose.


3 Refer to note 2 above under paragraph 4.1.5.3 (ADR).

4 Section 108 (3) of the HGCRAct, 1996.


6 An unreported decision of the South Gauteng High Court handed down on 9 March 2010.

7 An unreported judgement of the South Gauteng High Court dated 23 April 2010.

8 An unreported judgement of the South Gauteng High Court dated 3 May 2013.

9 An unreported judgement of the South Gauteng High Court dated 12 February 2013.

10 Sasol Chemical Industries v Odell and another, an unreported judgement of the Free State High Court, Bloemfontein dated 20 February 2014.

11 Refer to note 10 at paragraph 19.


13 Clause 40 of the Fourth Edition, March 2004, of the JBCC Principal Building Agreement, which at clause 40.4 provides that “a dispute … shall be submitted to … [40.4.2] adjudication where practical completion … has not been achieved.”

14 Refer to note 12 at paragraph 8.

15 The Companies Act No 71 of 2008 has completely overhauled the South African company law legislative framework.

16 Section 181 (Right to participate in hearing) of the Companies Act No 71 of 2008.

17 Refer to note 15 above at Section 180 (Adjudication hearings before Tribunal) (1) (a).

18 Refer to note 15 above at Section 180 (Adjudication hearings before Tribunal) (1) (b).

19 Refer to note 15 above.


23 The Western Australia Construction Contracts Act 2004.


28 The Construction Industry Payment and Adjudication Act (CIPAA) 2012 was passed on 18 June 2012 and gazetted on 22 June 2012. The Ministry of Works had proposed the Construction Industry Payment and Adjudication (Exemption) Order 2014 and the amended Construction Industry Payment and Adjudication Regulations 2014. Both had been approved by the Minister of Works and became effective on 15 April 2014.

29 Justice Dyason in the landmark UK case of Macob Civil Engineering Ltd v Morris Construction Limited (1999) BLR 93 TCC at page 97.


REFERENCES


Untreated aeolian sand base course for low-volume road proven by 50-year old road experiment

F Netterberg, D Elsmere

The Hoopstad long-term pavement performance experiments constructed in 1962 between Hoopstad and Bulgfontein in the Free State Province of South Africa included a 90 m section of a fine-grained, nonplastic, A-2-40L aeolian, Kalahari-type sand as unstabilised base course. After 50 years and approximately 1.0M E80 all the experimental sections are still carrying traffic, none has been rehabilitated and none appears to have ever failed. The results of a pavement evaluation carried out in 2013 indicate that similar sand can be used unstabilised as base course for a Category C or D low-volume road designed to carry up to at least 0.1M E80/lane over 20 years, provided it is compacted to refusal or at least 100% MAASHO on a good support, is well drained, well sealed with at least the equivalent of a double seal, and that the shoulders also offer good lateral support and drainage. The seal must also be sufficiently wide to accommodate the traffic expected. Such a design offers tremendous potential for the construction of relatively inexpensive, all-weather, low-volume roads in the vast area of arid and semi-arid southern Africa in which similar sands and a scarcity of gravels occur.

INTRODUCTION

Road construction materials other than fine aeolian sands are scarce in the northwestern Free State Province of South Africa, as well as in the vast area of southern Africa covered by Kalahari and similar sands. In their natural state such sands are usually regarded as suitable for use only in the lower pavement layers (e.g. SANRAL 2013). Long-term pavement performance (LTTP) experimental sections were therefore constructed on the Hoopstad–Bultfontein road in the Free State in 1962 to evaluate the use of such sands as base course when stabilised with cement, bitumen and road tar, using sections of unstabilised sand and crusher-run graded crushed stone as control sections (Gregg 1963).

In this paper only the performance of the unstabilised (neat) sand section is reported in comparison with the adjacent 3% and 5% ordinary Portland cement (OPC) stabilised sections, and no attempt is made to review the use of sands in general, for which see Botswana Roads Department (BRD 2010) and Paige-Green et al (2011).

Unless stated otherwise, the methods and abbreviations used are shown in the list of references and/or the Appendix.

LOCATION, LAYOUT AND AS-BUILT DATA

Location and layout

The location, layout and as-built test results of the neat sand base Section A and the adjacent cement-stabilised Sections B and C compiled by the authors from Gregg (1963) are shown in Tables 1 and 2. The available test results for the crusher-run section (K), which was not investigated by the authors, are also shown for comparison.

All sections had a similar sand sub-base treated with 3% PBFC on a similar neat sand selected layer, fill, and roadbed. All layers were 150 mm in thickness.

The test methods used were those of the Department of Transport (DOT 1958) for compaction characteristics, and those later published by the National Institute for Road Research (NIRR 1968) for the indicator tests. In February 1963, eight months after construction, in-situ CBR and Benkelman beam deflection tests were carried out and cores taken for the determination of the cement contents, unconfined compressive strength (UCS) and indirect tensile strength (ITS). The CBR tests were not dynamic cone penetrometer (DCP) tests, but the traditional in-situ tests as described for example by the Road Research Laboratory (RRL 1952).

Material properties

No laboratory CBR was reported for the neat sand and it was simply stated to be a non-plastic (NP), red, silty, fine sand containing less than 1% organic matter (Table 2).

The compaction characteristics were as follows:

- Maximum dry density (MDD) (kg/m³): 1 896 (MAASHO); 1 856 (Proctor)
Optimum water (moisture) content (OWC) (%) : 9.6 (MAASHO); 11.0 (Proctor)

The untreated (i.e. neat) sand had an uncon fined compressive strength (UCS) at Proctor compaction of 140 kPa after seven days of curing in a humid room and 1200 kPa after seven days of "open curing" (static compaction in 102 × 51 mm moulds).

The sections were compacted using a 50 ton pneumatic roller followed by a steel-wheel roller, and completed in June 1962.

The relative compaction of the neat sand section was not reported, and that of the 3% and 5% OPC sections was only 92% and 90% MAASHO, respectively.

At least until about November 1963 all the sections had performed satisfactorily and no failures had occurred, and it was concluded that the unstabilised sand would have sufficient strength to comply with the usual minimum CBR requirement of 80, provided that it was maintained in a dry condition (Gregg 1963).

Level measurements apparently taken up to June 1963 showed the maximum settlement on any section to be 13 mm. These measurements, as well as visual observations, were apparently continued up to about 1974, but the records could not be found.

### Table 1

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<thead>
<tr>
<th>Location, layout and as-built test results on some of the Hoopstad sand base course experiments eight months after construction [1]</th>
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<tr>
<td>Surfacing: 6.1 m wide 25 mm triple seal; 2.0 m wide neat sand shoulders</td>
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<tr>
<td>Mean ITS</td>
</tr>
<tr>
<td>Mean deflection</td>
</tr>
<tr>
<td>Mean deflection</td>
</tr>
<tr>
<td>Mean ROC</td>
</tr>
<tr>
<td>New section stake value (m)</td>
</tr>
<tr>
<td>Current log km</td>
</tr>
<tr>
<td>Lat (S)</td>
</tr>
<tr>
<td>Long (E)</td>
</tr>
<tr>
<td>GPS elevation (m)</td>
</tr>
</tbody>
</table>

**NOTES**

[1] Compiled mostly from Gregg (1963) with statistics calculated by authors
[2] Water content not reported; one CBR of 26 on Section A omitted from analysis
[3] After 4–5 hours of soaking
[4] On 102 × 51 mm cylindrical specimens: neat sand compacted in lab at Proctor effort (MDD 1 856 kg/m³, OMC (11.0%), others on cores from road (density or relative compaction not stated)
[5] After seven days of exposure in the laboratory
[6] After seven days in a humid room
[7] Benkelman beam
[8] 62 kN axle load, 480 kPa tyre pressure
[9] Dehlen (1962), radius of curvature (ROC) by Dehlen curvature meter
[10] Mean OPC content 4.1%, water content 1.3%
[11] Mean OPC content 4.1%, water content 17.2%
[12] Mean OPC content 6.5%, water content 1.6%
[13] Mean OPC content 6.5%, water content 15.8%

### Table 2

<table>
<thead>
<tr>
<th>Particle size (mm)</th>
<th>Percentage passing [1]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.18</td>
<td>100</td>
</tr>
<tr>
<td>0.841</td>
<td>100</td>
</tr>
<tr>
<td>0.600</td>
<td>99</td>
</tr>
<tr>
<td>0.420</td>
<td>(97)</td>
</tr>
<tr>
<td>0.250</td>
<td>87</td>
</tr>
<tr>
<td>0.150</td>
<td>46</td>
</tr>
<tr>
<td>0.074</td>
<td>(9)</td>
</tr>
<tr>
<td>0.060</td>
<td>7</td>
</tr>
<tr>
<td>0.020</td>
<td>6</td>
</tr>
<tr>
<td>0.006</td>
<td>5</td>
</tr>
<tr>
<td>0.002</td>
<td>3</td>
</tr>
</tbody>
</table>

**Calculated by authors:**

- Grading modulus (GM) = 0.94
- Dust ratio [2] = 0.09
- Uniformity coefficient (Cu) = 2.5
- Coefficient of curvature (Cc) = 1.6

**Classification:**

- AASHTO: A3/borderline A-2-4(0)
- Unified: SP–SM (poorly graded sand with silt)
- COLTO: potential G7 at best (no CBR)

**NOTES**

[1] Figures bracketed estimated by authors
[2] P075 / P425

- Optimum water (moisture) content (OWC) (%) : 9.6 (MAASHO); 11.0 (Proctor)

The untreated (i.e. neat) sand had an unconfined compressive strength (UCS) at Proctor compaction of 140 kPa after seven days of curing in a humid room and 1 200 kPa after seven days of “open curing” (static compaction in 102 × 51 mm moulds).

The sections were compacted using a 50 ton pneumatic roller followed by a steel-wheel roller, and completed in June 1962.

The relative compaction of the neat sand section was not reported, and that of the 3% and 5% OPC sections was only 92% and 90% MAASHO, respectively.

At least until about November 1963 all the sections had performed satisfactorily and no failures had occurred, and it was concluded that the unstabilised sand would have sufficient strength to comply with the usual minimum CBR requirement of 80, provided that it was maintained in a dry condition (Gregg 1963).

Level measurements apparently taken up to June 1963 showed the maximum settlement on any section to be 13 mm. These measurements, as well as visual observations, were apparently continued up to about 1974, but the records could not be found.
According to the surveyor (A Bam 2012, personal communication) no distress had occurred up to that time.

**Discussion**

Although the neat sand base was nonplastic, the borderline A-2-4(0) classification, the high mean in-situ CBR of 81, and the presence of a significant UCS, especially after drying, indicated that this sand was not the usual cohesionless A3 sand, but did have sufficient strength for base course. At the same time the low mean, soaked, in-situ CBR of 29 indicated the critical necessity of avoiding saturation.

If the lower axle load used to measure the deflection and radius of curvature (ROC) is allowed for, the corrected deflections and radii of curvature of the deflection bowls of about 0.3 mm and 90 m for Section A respectively, were all within the sound range of 0.6 mm and > 80 m for a modern, untreated base on a treated sub-base for a modern Category C road, according to the criteria in TRH 12 (COLTO 1997), by which criteria Section A would be expected to have a structural capacity in excess of 5M E80.

**Preliminary investigation in January 2013**

Preliminary work confirmed the presence of the neat sand section in a fair condition, a still substantial life, and adequate laboratory soaked and unsoaked CBRs (Table 3 and Figure 1).

The visually identical sand in the road reserve (Photo 1) was also sampled and tested (Tables 4 and 5).

This work was judged sufficiently encouraging to proceed with a full pavement investigation.

**PAVEMENT EVALUATION IN OCTOBER 2013**

During October/November 2013 a more detailed pavement evaluation was carried out comprising visual evaluation according to TMH 9 (CSRA 1992), measurement of degree and extent of cracking, patching, edge-breaking and rut depths; DCP tests to 800 mm or refusal; profiling; phenolphthalein and acid tests; and sampling of the base at four sites in the outer wheel paths on Section A and two each on the other sections. General views of Sections A, B and C towards Bultfontein are shown in Photos 2, 3 and 4.

**Visual assessment**

In all cases the 30 mm thick seal was found to consist only of the original triple seal plus one reseal.
Table 3 Results of January 2013 preliminary investigation of neat sand

<table>
<thead>
<tr>
<th>Log km</th>
<th>“Chainage”[1]</th>
<th>Units 20.720</th>
<th>20.740</th>
<th>20.760</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section</td>
<td>m 19 LM</td>
<td>35 CL</td>
<td>58 RM</td>
<td></td>
</tr>
</tbody>
</table>

DCP [2]
- 150 mm
  - DN mm/blow: 11.0, 11.8, 12.2

Base [3]
- CBR %: 20, 18, 17
- FWC %: 5.1, 6.0, 5.5
- FWC/OWC: 0.74, 0.91, 0.83

Redefined
- DN mm/blow: 8.2, 7.7, 10.8

Section A
- LM: 19, CL: 35, RM: 58
- DCP: 150 mm
- CBR %: 20, 18, 17
- FWC %: 5.1, 6.0, 5.5
- FWC/OWC: 0.74, 0.91, 0.83

Pavement
- Balance No (A) [7]: 2 946, 5 970, 1 288
- Balance No (B) [7]: – 9, – 1, – 7
- Thickness: 395, 376, 185
- DSN800 Blows: 169, 178, 139
- Structural capacity [7]: E80 1.9, 2.3, 0.9
- DSN800 Blows [8]: 164, 173, 135
- Structural capacity [8]: E80 1.7, 2.0, 0.9

Soil constants [10]
- LL / PI / LS [11]: NP/0.0, NP/0.0, NP/0.0
- MDD / OWC (MAASHO) kg/m³/%: 1 898 / 6.9, 1 891 / 6.6, 1 949 / 6.6
- CBR @ 100% MAASHO
  - Soaked at 2.54 / 5.08 mm: % 79 / 32, 77 / 44, 58 / 17
  - At OWC at 2.54 / 5.08 mm: % 140 / 70, 97 / 34, 98 / 31
  - Swell at 100% MAASHO: % 0.0, 0.0, 0.0
- Derived data
  - GM – 0.87, 0.88, 0.89
  - Dust ratio – 0.19, 0.17, 0.18
  - Uniformity coefficient – 4.0, 4.0, 4.3
  - Coefficient of curvature – 1.3, 1.4, 1.3
- Classification
  - AASHTO A-2-4(0), A-2-4(0), A-2-4(0)
  - UNIFIED SM, SM, SM

NOTES
[1] LM = left midlane, CL = centre line, RM = right midlane
[3] Surfacing (15–30 mm) removed by inspection of penetration curve during processing
[4] Surfacing removed as above and base redefined as uniform by computer
[6] From mean sand relationship (this work): CBR = 3 000 DN –1.46 for DN > 10
[7] Including surfacing (i.e. DCP zero taken at top of surfacing)
[8] Excluding surfacing (i.e. zero taken at top of base)
[9] Laboratory testing by Geostrada, Pretoria
[10] On P425 fraction unless stated otherwise

Table 4 Summary of laboratory indicator test results on initially presumed sand borrow [1]

<table>
<thead>
<tr>
<th>Log km</th>
<th>Units 19.771</th>
<th>19.796</th>
<th>19.821</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample</td>
<td>No 3 / 573</td>
<td>3 / 574</td>
<td>3 / 575</td>
</tr>
</tbody>
</table>

Test [2]
- Passing (mm)
  - 4.75 % 100, 100, 97
  - 2.00 % 100, 99, 96
  - 1.18 % 100, 98, 96
  - 0.600 % 98, 96, 93
  - 0.425 % 97, 95, 92
  - 0.250 % 83, 86, 83
  - 0.150 % 54, 48, 50
  - 0.075 % 18, 17, 20
  - 0.002 % 4, 4, 4
- Soil constants [3]
  - LL / PI / LS [4]: NP / 0.0, NP / 0.0, NP / 0.0
  - ARD – 2.643, 2.637, 2.641
- Derived data
  - GM – 0.85, 0.89, 0.92
  - FM – 0.65, 0.74, 0.85
  - Dust ratio – 0.19, 0.18, 0.22
  - Uniformity coefficient – 2.6, 2.7, 2.9
  - Coefficient of curvature – 0.9, 1.0, 0.9
  - Classification
    - AASHTO A-2-4(0), A-2-4(0), A-2-4(0)
    - UNIFIED SM, SM, SM

NOTES
[1] Sampled in road reserve from depth of 0.3–0.8 m; colour yellowish-brown (10 YR 5/6) dry, dark yellowish-brown (10 YR 4/4) wet
[4] On both P425 and P075 fractions

Table 5 Comparison between soaked and unsoaked CBRs on composite sample [1]

<table>
<thead>
<tr>
<th>CBR at penetration [2] mm</th>
<th>2.54</th>
<th>5.08</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compaction (MAASHO) %</td>
<td>102</td>
<td>100</td>
</tr>
<tr>
<td>Soaked</td>
<td>%</td>
<td>80</td>
</tr>
<tr>
<td>At OWC</td>
<td>%</td>
<td>110</td>
</tr>
<tr>
<td>At 75% OWC</td>
<td>%</td>
<td>150</td>
</tr>
<tr>
<td>At 50% OWC</td>
<td>%</td>
<td>220</td>
</tr>
<tr>
<td>At 25% OWC</td>
<td>%</td>
<td>270</td>
</tr>
</tbody>
</table>

Classification

NOTES
[1] Approximately equal masses of Samples 3/573–3/575 (MDD 1 854 kg/m³, OWC 6.2%)
[3] G6 on CBR of ≥ 25 at 95%, but fails GM requirement of 1.2– 2.6
On all sections the binder of the reseal was still good, i.e. Degree 1/Extent 5 (D1/E5), and there was little bleeding (D2/E5), but severe and extensive map to hexagonal to block (D5/E5) cracking at an average spacing of about 1 m over the full width, and crocodile (D4-5/E 3-5) cracking occurred in all wheel paths and extended across the centre line.

The cracking was confined to the seal, was usually only 10–15 mm deep, and did not penetrate into the base course at any of the sites excavated or at the additional cracks tested. Although often spalling to a width of 10 mm, the actual width of the cracks was only 1 mm or less.

Longitudinal cracking and pumping were absent, rutting was minimal (D2-3/E5), there were no undulations, and patching was almost entirely confined to the edge-breaking and edge-cracking, which was severe (D 3-5) and extensive (E 4-5) on all three sections.

Some secondary cracking was evident on Sections B and C only, especially in the left lane, but was less extensive on Section C.

There was no evidence of shear failures (the patches appeared to be all due to large edge breaks) or even excessive rutting (except on the sand shoulders), and the main problem was edge-breaking due to the seal being now too narrow for the large trucks currently using it (Photos 5 and 6). This was worst on Section A, where the value of the geotextile and emulsion holding the seal in the left lane was evident.

The riding quality was assessed as only fair because of the edge breaks and patching.

The surface drainage was assessed as adequate and the grassed sand shoulders as safe. The road had been built with a camber and the edges of the seal were about 80–130 mm below the centre line, giving an effective total crossfall of about 3–4%.

The centre line of the road was about 500 mm and 300 mm above the side drains on the left- and right-hand sides, respectively. The fall of the natural ground level in the 30 m wide road reserve was about 0.5 m from right to left.

The alignment was straight and practically level, with a fall of only about 0.2% from the end of Section C to the start of Section A.

Further details of the cracking and patching and the rut depths are shown in Table 6. As the road had been built with a camber the rut depths were measured in two ways. The end of the straight edge was first placed in the normal way on the edge of the seal or on the centre line, and the deviations under the ends recorded as ‘edge’ and CLL or CLR, respectively, while the maximum deviation...
in the wheel paths was recorded as the rut depth (OWP or IWP respectively). Then the straight edge was held down on the edge or the centre line and the deviations in the wheel paths re-measured and recorded as OWP or IWP respectively. The latter procedure usually yielded the smaller deviations and may be a closer representation of the true traffic-induced rut depths.

The general pavement condition was assessed as poor on Section A and fair on Sections B and C, with C slightly better than B.

The inspection panel recommended that the edge-breaking should be patched at an A priority and the shoulders gravelled (or at least bladed), the seal softened and rolled in hot weather to seal the cracks, and the road ressealed at a B priority. However, damage to the edges must be expected to continue, as the road is simply too narrow for the large trucks currently using it.

DCP survey
The results of the rut depth and DCP surveys in the outer wheel paths are summarised in Table 7. As the DCP results for both wheel paths were similar, they have been combined to total six points on Section A and B, and five on Section C (one was invalid).

The analysis was carried out by means of the EasyDCP program (J Lea 2013, personal communication) using the Kleyn granular base model (De Beer et al 1989) for Category C and D roads (i.e. 80/20 and 50 percentile (%-ile) basis respectively) and – conservatively – assuming an optimum moisture condition – considering that the survey was carried out at the end of the dry season when the field...
water content (FWC) was about 50% of the MAASHO optimum (OWC).

An example of the output of this program for a 20 %-ile DN of 7.0, an 80 %-ile DSN800 of 185 and a predicted 20 %-ile structural capacity of 2.6M E80 for Section A (in the 10 m right outer wheel path) is shown in Figure 2. For comparison, the required DCP profile for a Class II rural road pavement designed for 0.10–0.30 M MISA (i.e. actual 80 kN standard axles) of the former Transvaal Roads Department (1994) is also shown.

For simplicity and ease of comparison the granular base model has been used on all three pavements. Although it tends to over-predict the structural capacity of cemented pavements (De Beer et al 1989) it can be used on them (Kleyn & Savage 1982; COLTO 1997).

### Discussion

As a check on the published relationships between DN and CBR, DCP tests were carried out on most of the specimens after determination of the CBR. Plotting of these results led to a very rough but very different relationship from that of Kleyn (1984) for materials in general (and used in the program) and closer to those of Sampson and

#### Table 7 Summary of rut depth and DCP test results in outer wheel paths in October 2013

<table>
<thead>
<tr>
<th>Layer</th>
<th>Test</th>
<th>Units</th>
<th>Section A (neat sand)</th>
<th>Section B (sand + 3% OPC)</th>
<th>SectionC (sand + 5% OPC)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Max</td>
<td>80%-ile</td>
<td>50%-ile</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Rut depth</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OWPs</td>
<td>[1]</td>
<td></td>
<td>LHS mm</td>
<td>12</td>
<td>10</td>
</tr>
<tr>
<td>OWPs</td>
<td>[1]</td>
<td></td>
<td>RHS mm</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>DCP</td>
<td>[2,3]</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>150 mm base</td>
<td>[4]</td>
<td>DN mm/blow</td>
<td>74</td>
<td>7.0</td>
<td>6.3</td>
</tr>
<tr>
<td>150 mm base</td>
<td>[4]</td>
<td>CBR %</td>
<td>58</td>
<td>46</td>
<td>41</td>
</tr>
<tr>
<td>150 mm base</td>
<td>[4]</td>
<td>FWC %</td>
<td>4.1</td>
<td>–</td>
<td>3.5</td>
</tr>
<tr>
<td>150 mm shoulders</td>
<td></td>
<td>FWC/OWC %</td>
<td>–</td>
<td>0.55</td>
<td>–</td>
</tr>
<tr>
<td>Redefined upper layer</td>
<td>[5]</td>
<td>DN mm/blow</td>
<td>7.0</td>
<td>6.9</td>
<td>5.6</td>
</tr>
<tr>
<td>Redefined shoulders</td>
<td>[6]</td>
<td>CBR %</td>
<td>58</td>
<td>49</td>
<td>46</td>
</tr>
<tr>
<td>Redefined shoulders</td>
<td>[6]</td>
<td>FWC %</td>
<td>8.5</td>
<td>–</td>
<td>8.5</td>
</tr>
<tr>
<td>Pavement</td>
<td></td>
<td>Thickness mm</td>
<td>595</td>
<td>358</td>
<td>275</td>
</tr>
<tr>
<td>Balance No (A)</td>
<td>[10]</td>
<td>–</td>
<td>2821</td>
<td>2478</td>
<td>1902</td>
</tr>
<tr>
<td>Balance No (B)</td>
<td>[10]</td>
<td>–</td>
<td>10</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>Structural capacity</td>
<td>[10]</td>
<td>M E80</td>
<td>6.9</td>
<td>3.7</td>
<td>3.3</td>
</tr>
<tr>
<td>Structural capacity</td>
<td>[11]</td>
<td>M E80</td>
<td>6.0</td>
<td>2.4</td>
<td>2.1</td>
</tr>
</tbody>
</table>

**NOTES**

[1] Outer wheel paths (n = 14 each), including those on patches
[2] Three in each outer wheel path on Sections A and B, three in left and two in right on Section C
[3] Processed by P Paige-Green using EasyDCP program (J Lea, personal communication) using Kleyn granular base model and Cm = 30
[4] Surfacing (15–30 mm) removed by inspection of penetration curve during processing (i.e. zero taken at top of base)
[5] Surfacing removed as above and uniform layers redefined by computer; water contents of sub-base 8.0, 8.3%; selected 5.2, 5.3%, fill 7.8%
[7] From mean sand relationship (this work): CBR = 3 000 DN –1.46 for DN > 10
[8] Field water content (n = 4 on A, 2 on B, 3 on C)
[9] Mean OWC = 7.5% (n = 4)
[10] Including surfacing (i.e. zero taken at top of surfacing)
[11] Excluding surfacing (i.e. zero taken at top of base); (20 %-ile capacity of 1.9 for Section A remains unchanged if outlier of 6.9M E80 removed)
Figure 2 Example of DCP program output for a single point
Netterberg (1990) for nonplastic materials, although none of them were sands (Figure 3). The relationship for the Hoopstad sand was:

\[
\text{CBR} \approx 3000 \text{ DN}^{-1.46} \quad \text{for DN > 10} \quad (1)
\]

in comparison with that of Kleyn (1984):

\[
\text{CBR} = 410 \text{ DN}^{-1.27} \quad \text{for DN > 2} \quad (2)
\]

and Sampson and Netterberg (1990):

\[
\text{CBR} = 354 \text{ DN}^{-0.67} \quad \text{for DN > 3} \quad (3)
\]

The analysis showed that the 80 %-ile DN of the base of the neat sand section was about 7.0 mm/blow, indicating that 80% of the assumed 150 mm – and the redefined 175 mm – base had an in-situ DCP CBR of more than about 34% according to the Kleyn model, or over 100 according to the Hoopstad model (Figure 3 in this paper). With similar 80 %-ile DN of 5.5 and 20 %-ile CBRs of about 45 (or over 100), the two cemented bases were not greatly stronger than the uncedmented base, both in the assumed 150 mm and in the redefined cases.

This suggests that the in-situ CBR of the neat sand base was far higher than that indicated by the Kleyn model, and thereby offers a plausible explanation for its unexpectedly good performance.

The 20 and 50 %-ile redefined thicknesses of similar strength to the base of 171 mm and 275 mm respectively indicate that most of the originally cemented sub-base on Section A had reverted to a granular state and was then operating at the same strength as the base. Experience with respect to the usual rates of carbonation (Netterberg 1991) suggests that, regardless of traffic, this would actually have been the case from about year five to ten.

The upper 150 mm of the shoulders on Section A had a 50 %-ile (in this case a mean) DN of 19, indicating an in-situ DCP CBR of about 11 according to the Kleyn model, and 48 according to the Hoopstad model.

The redefined case indicates that the whole 522 mm had a mean DN of 14, indicating a CBR of about 16 (or 60) and therefore good lateral support both to the base and lower layers. This conclusion is also supported by the minimal rutting of the sand shoulder shown in Photo 6, which is far less than would be expected from a nonplastic sand.

The 80 and 20 %-ile pavement balance numbers A of about 2 500 and 2 100, and B of -1 and -5 for Sections A and B respectively, indicate that both pavements were averagely balanced and marginal to inverted deep pavements, and that the Kleyn model requirements of a good (or apparently at least an average balance) have been at least marginally met. With respective A and B balance numbers of 2 600 and 5 Section C was an averagely balanced deep pavement.

The predictions of the remaining structural capacities of the pavements depend somewhat on whether the zero point of the DCP is taken at the top of the seal according to TMH 6 (NITRR 1984) or at the top of the base, which is where it was taken during the testing from which the Kleyn model was developed (E G Kleyn 2010, personal communication).

For example, the 20 %-ile predictions are:

<table>
<thead>
<tr>
<th>Zero point</th>
<th>Section number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of seal (M E80)</td>
<td>A</td>
</tr>
<tr>
<td>2.6</td>
<td>2.5</td>
</tr>
<tr>
<td>Top of base (M E80)</td>
<td>1.9</td>
</tr>
</tbody>
</table>

As the 80 %-ile rut depths of all three sections were also only in the 6–10 mm range they were all sound according to TRH 12 (COLTO 1997). Sections A and B have a similar residual capacity of about 2M and C about 1M E80 to an additional rut depth of 20 mm. Assuming proportionality, this indicates that the remaining capacities to a total rut depth of 20 mm of the three sections are about 1.0 for Sections A and B and about 0.5M E80 for Section C.

If all of the 13 available DCP results are considered, the lowest predicted 20 %-ile capacity of all 10 of the DCPs carried out in
October of 1.7M E80, and the three single points in January 2013 of 0.9M E80, indicate a residual capacity for Section A of at least about 0.5M E80.

As the Kleyn model is an average prediction the most conservative view would be to halve this again. On this basis it can be concluded with a high degree of certainty that the residual structural capacity of Section A to a total rut depth of 20 mm (i.e. an additional 10 mm) at an in-situ water content of OWC is at least about 0.3M E80. If it were to become saturated this would be halved to about 0.1–0.2M E80.

The capacities of the cemented base sections must be regarded as more approximate, as they have not been checked by the De Beer model (De Beer et al 1989) for cemented pavements.

The low degree and extent of rutting, the absence of shear failures (at least at the time of the two site visits), the absence of base course patching found in any of the six holes and 13 DCP points, the reasonable DN (and apparently very good in-situ operating CBR of over 100) and the predicted residual structural capacity of at least about 0.3M E80 all indicate that the further use of such a sand as untreated base course for a low-volume road is viable under similar conditions.

As most of the sub-base was operating at about the same strength as the base it should not be necessary to treat it with cement, provided that it is compacted to a similar strength and that this support is sufficient to achieve the necessary high degree of compaction of the base.

Comparison with the Transvaal Roads Department (1994) design (the dashed red strength line shown in Figure 2) shows that the 20 %-ile example on Section A met all the requirements of all layers for both the 150 mm layer and the redefined cases, except that of the base for which a DN of 2.8 was required, whereas only 7.0 was found.

### Table 8 Results of in-situ and laboratory tests on October 2013 samples of neat sand base

<table>
<thead>
<tr>
<th>Location In-situ</th>
<th>OWP</th>
<th>10 m L</th>
<th>30 m L</th>
<th>55 m L</th>
<th>55 m R</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rut depth mm</td>
<td>9</td>
<td>12</td>
<td>6</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>DCP base (150 mm)DN</td>
<td>5.6</td>
<td>4.7</td>
<td>7.0</td>
<td>5.6</td>
<td></td>
</tr>
<tr>
<td>CBR [1] %</td>
<td>46</td>
<td>58</td>
<td>34</td>
<td>46</td>
<td></td>
</tr>
<tr>
<td>CBR [2] %</td>
<td>&gt; 100</td>
<td>&gt; 100</td>
<td>&gt; 100</td>
<td>&gt; 100</td>
<td></td>
</tr>
<tr>
<td>Wet density [3] kg/m³</td>
<td>1 933</td>
<td>1 933</td>
<td>1 935</td>
<td>1 933</td>
<td></td>
</tr>
<tr>
<td>FWC / OWC [4] %</td>
<td>0.82 / –</td>
<td>0.49 / –</td>
<td>0.59 / 0.43</td>
<td>0.77 / 0.59</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Lab sample, passing (mm)</th>
<th>No</th>
<th>3/11650</th>
<th>3/11651</th>
<th>3/11652</th>
<th>3/11653</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.18 %</td>
<td>100</td>
<td>100</td>
<td>100</td>
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<td></td>
</tr>
<tr>
<td>0.850 %</td>
<td>99</td>
<td>99</td>
<td>99</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.600 %</td>
<td>97</td>
<td>97</td>
<td>97</td>
<td></td>
<td></td>
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<tr>
<td>0.425 %</td>
<td>96</td>
<td>96</td>
<td>96</td>
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<td></td>
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<tr>
<td>0.250 %</td>
<td>80</td>
<td>81</td>
<td>82</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.150 %</td>
<td>47</td>
<td>50</td>
<td>50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.075 %</td>
<td>20</td>
<td>21</td>
<td>19</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.038 %</td>
<td>19</td>
<td>21</td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.020 %</td>
<td>14</td>
<td>14</td>
<td>14</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.002 %</td>
<td>13</td>
<td>14</td>
<td>12</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Soil constants (P425)    | %   | NP     | NP     | NP     | NP     |
| LL/PI/LS                |     |        |        |        |        |
| SE                      | –    | 13     | 20     | 38     | 40     |

| Soil constants (P075)    | %   | 21 / 6 / 2.0 [6] | SP / 1.0 | SP / 1.0 | SP / 0.5 |
| MDD/OWC kg/m³ / %       | 1 840 / 6.7 | 1 828 / 8.9 | 1 850 / 7.6 | 1 870 / 6.9 |

| Soaked CBR at % MAASHO  | %   | 90 / 76 | 90 / 69 | 90 / 69 | 110 / 84 |
| 104 / 102               |     | 62 / 51 | 50 / 38 | 51 / 39 | 63 / 48 |
| 100 / 98                | %   | 95 / 93 | 37 / 23 | 26 / 20 | 26 / 19 | 32 / 24 |
| Swell at 100% %         |     | 0.0     | 0.0     | 0.0     | 0.0     |
| Derived data            |     |         | 0.84    | 0.83    | 0.85    | 0.78    |
| GM                      | –    | 0.71    | 0.68    | 0.67    | 0.61    |
| FM                      | –    | 0.21    | 0.22    | 0.20    | 0.26    |
| Dust ratio              | –    | 120     | 42      | 38      | 25      |
| Cup IF075               | –    |         |         | 9.4     | 9.8     |
| Classification AASHTO   | –    | A-2-4(0)  | A-2-4(0) | A-2-4(0) | A-2-4(0) |
| UNIFIED                 | –    | SM      | SM      | SM      | SM      |
| COLTO                   | –    | G7      | G7      | G7      | G7      |
| Compactability [5]      | –    | Marginal | Good    | Good    | Good    |

**NOTES**

[2] From mean sand relationship (this work): CBR = 3 000 DN\(^{-1.46}\) for DN > 10
[3] CPN MC-30 nuclear, upper 20 mm of primed base removed and 0–100 mm results used; 0–150 and 0–250 mm results generally similar
[4] Using nuclear/lab water content
[5] Assuming upper limit of IF075 (using TMH 1 cup for LL) is 120 for P075 of 20%
[6] Repeat test: 21 / 7 / 1.5

**MATERIAL TEST RESULTS**

### Soil engineering testing

#### Results

The pavement profile was confirmed to be approximately as reported (Photo 7) and was sampled and tested (Table 8, Figure 4 and Table 9).

#### Discussion

The laboratory test results confirmed and extended those of the January work, the following being of particular importance:
Much higher apparent in-situ CBRs (of over 100) than the 34–58 indicated by the Kleyn model.

High relative compactions in the wheel paths of about 99–100%.

No plasticity on the P425, but SP-6 on the P075 using the Casagrande cup method.

Good laboratory soaked CBRs of about 50–60 at 100% MAASHO and 70–90 at higher compactions.

Very good laboratory unsoaked 100% MAASHO CBRs of over 80% at water contents of OWC and less – the apparent in-situ condition.

AASHTO and Unified classifications of A-2-4(0) and SM, respectively, but only COLTO G7 (because of the fine grading). A substantial depth of prime penetration of some 10–20 mm was noted, suggesting that a higher than normal rate of application may have been used.

The sub-base and lower layers were subjected to minimal testing as (except for the cement treatment of the sub-base) they were visually similar to the untreated sand base (Table 10). The main differences were that the sub-base had been rendered nonplastic even on the P075 fraction and that its SE was much higher than the other layers (as well as the base).

Check testing

Results

Check testing of the most important soil properties of the four samples of base course taken in October 2013 was carried out by the CSIR in Pretoria on material that had already been used at least once for compaction-related tests and subsequently air-dried (Tables 11 and 12, and Figure 5).

Here the liquid limits were determined by the more accurate TMH 1 flow curve method in the case of the Casagrande cup method, and the BS 1377 penetration curve method in the case of the cone method.

As no cup liquid limit could be obtained on the P425 fraction, the LS was determined from the FME instead.

The soaked CBR was determined in duplicate on two samples.

As an additional measure of the compacted strength of the material, the shear strength was determined by the TMH 6 (NITRR 1984) vane method after determination of the CBR, with the top of the vane at least 50 mm below the top of the specimen.

Discussion

Both the soaked and unsoaked vane shear strengths show a reasonable correlation with their respective CBRs, although they lie on different curves (not shown), suggesting that this is a viable alternative test for fine sands. Strengths of 94–224 kPa

Table 9 Comparison between soaked and unsoaked CBR and DN on neat sand base

<table>
<thead>
<tr>
<th>Location (OWP)</th>
<th>10 m L</th>
<th>30 m L</th>
<th>55 m L</th>
<th>55 m R</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sample No</strong></td>
<td>3/11650</td>
<td>3/11651</td>
<td>3/11652</td>
<td>3/11653</td>
</tr>
<tr>
<td><strong>Test</strong></td>
<td><strong>CBR</strong> %</td>
<td><strong>DN mm/blow</strong> %</td>
<td><strong>Comp. %</strong></td>
<td><strong>CBR</strong> %</td>
</tr>
<tr>
<td><strong>MAASHO 2.54 mm CBR</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soaked</td>
<td>[4] %</td>
<td>74</td>
<td>28</td>
<td>102</td>
</tr>
<tr>
<td>OWC</td>
<td>%</td>
<td>89</td>
<td>12</td>
<td>100</td>
</tr>
<tr>
<td>0.75 OWC</td>
<td>[5] %</td>
<td>62</td>
<td>12</td>
<td>98</td>
</tr>
<tr>
<td>0.5 OWC</td>
<td>[5] %</td>
<td>119</td>
<td>5.0</td>
<td>99</td>
</tr>
</tbody>
</table>

**NOTES**

1. All testing by Geostrada, Pretoria
2. Mean DN in mould to maximum depth of 95–100 mm before striking base plate
3. Soaked and OWC 5.08 mm CBRs all < 2.54 mm CBRs
4. Water content of top 25 mm of specimen after DN test 12–13%
5. Dried back after compaction at OWC to approx condition shown

Table 10 Results of indicator tests on other layers on Section A

<table>
<thead>
<tr>
<th>Location</th>
<th>Layer</th>
<th>GM –</th>
<th>FM –</th>
<th>P075 %</th>
<th>PI (P425) %</th>
<th>PI (P075) %</th>
<th>SE</th>
</tr>
</thead>
<tbody>
<tr>
<td>55 m LOWP</td>
<td>Sub-base</td>
<td>0.89</td>
<td>0.83</td>
<td>20</td>
<td>NP</td>
<td>NP</td>
<td>59</td>
</tr>
<tr>
<td></td>
<td>Selected</td>
<td>0.79</td>
<td>0.55</td>
<td>24</td>
<td>NP</td>
<td>6</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>Fill</td>
<td>0.86</td>
<td>0.71</td>
<td>19</td>
<td>NP</td>
<td>5</td>
<td>26</td>
</tr>
<tr>
<td>30 m R</td>
<td>Shoulder</td>
<td>0.81</td>
<td>0.67</td>
<td>26</td>
<td>NP</td>
<td>8</td>
<td>22</td>
</tr>
</tbody>
</table>

**NOTES**

1. Testing by Geostrada, Pretoria
2. All four layers classified as A-2-4(0) and SM

Figure 4 Gradings of October 2013 base course samples
### Table 11 Results of check tests for soil constants and CBR on neat sand base from Section A

<table>
<thead>
<tr>
<th>Location (OWP)</th>
<th>m, L or R</th>
<th>10 L</th>
<th>10 L</th>
<th>30 L</th>
<th>55 L</th>
<th>55 R</th>
<th>55 R</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>[1] Units</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil constants (P425)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BS cone LL</td>
<td>[3] % 21 – 21 19 21 –</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FME</td>
<td>% 18 – 19 20 20 –</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LS from FME</td>
<td>% 0.1 0.1 0.1 0.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Soil constants (P075)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PI</td>
<td>% 2 – 4 4 2 –</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LS</td>
<td>% 2.9 – 3.3 3.0 1.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BS cone PI</td>
<td>[4] % 5 – 6 10 7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MDD</td>
<td>[5] kg/m³</td>
<td>1 840</td>
<td>1 840</td>
<td>1 828</td>
<td>1 850</td>
<td>1 870</td>
<td>1 870</td>
</tr>
<tr>
<td>OWC</td>
<td>[5] % 6.7 6.7 8.9 7.6 6.9 6.9</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>CBR (MAASHO)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soaked</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.54 mm</td>
<td>% 47 39 42 41 54 58</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.08 mm</td>
<td>% 36 47 39 34 42 43</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Swell</td>
<td>% 0.08 0.08 0.08 0.08 0.08 0.08</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compacted dry density</td>
<td>kg/m³</td>
<td>1 809</td>
<td>1 805</td>
<td>1 827</td>
<td>1 860</td>
<td>1 834</td>
<td>1 851</td>
</tr>
<tr>
<td>Relative compaction</td>
<td>% 98 98 100 101 98 99</td>
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<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Water content</td>
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<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Compacted</td>
<td>% 6.6 6.6 8.6 7.2 6.5 6.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Whole specimen after soak</td>
<td>% 13.3 13.4 13.5 12.1 12.7 12.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top 25 mm</td>
<td>% 13.5 13.1 13.5 12.7 13.2 12.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>At OWC</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.54 mm</td>
<td>% 58 – 53 63 46 –</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.08 mm</td>
<td>% 56 – 47 31 47 –</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Swell</td>
<td>% 0.02 – 0.01 0.01 0.01 –</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WC compacted</td>
<td>% 6.7 – 8.6 7.0 6.8 –</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WC top 25 mm</td>
<td>% 6.6 – 8.6 7.1 6.7 –</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compacted dry density</td>
<td>kg/m³</td>
<td>1 831</td>
<td>– 1 828</td>
<td>1 866</td>
<td>1 845 –</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relative compaction</td>
<td>% 100 – 100 101 99 –</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vane shear strength</td>
<td>[6]</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Soaked</td>
<td>kPa</td>
<td>118</td>
<td>94</td>
<td>94</td>
<td>106</td>
<td>224</td>
<td>212</td>
</tr>
<tr>
<td>At OWC</td>
<td>kPa</td>
<td>106 – 71</td>
<td>189</td>
<td>177 –</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES**

[1] Testing by CSIR
[2] Repeat test on same sample
[3] Three or more flow/penetration curve method
[5] Determined by Geostrada
[6] Vane 50 mm high x 30 mm diameter
were achieved at 97–101% compaction after soaking, but only 71–189 kPa at 98–101% at OWC, apparently due to disturbance on inserting the vane.

For comparison, the recommended minimum vane shear strength of sand-asphalt at 40 °C is 200 kPa at 100% MAASHO density (NITRR 1985). However, minimum in-situ strengths of only 100 kPa at 40 °C at only 90% proved adequate for up to 0.1M E80, 120 kPa for up to 0.2M E80, and 200 kPa at 93% for up to an extrapolated 0.5M E80 on a seven-year-old Kalahari sand-asphalt road experiment in Botswana (Netterberg 1989).

The results of the check tests by the CSIR on the gradings and CBRs show that, although somewhat coarser and lower respectively, they agree reasonably with those by Geostrada, the samples remained NP on the P425 and SP on the P075, and their AASHTO and Unified classifications were unaltered.

It was therefore concluded that the reuse of this particular material resulted in no significant change in its properties, and that the results of the Geostrada and CSIR tests on reused material were valid.

### Particle shape

It has been known for many years that the shear strength of a non-cohesive sand consists of two parts, the internal frictional resistance between grains (a combination of rolling and sliding friction) and interlocking, which is particularly important in dense sands (Taylor 1948).

### Results

A photograph of the particles of a composite sample of the neat sand base as seen under a stereo microscope (Photo 8) showed that they were angular in shape.

In an attempt to quantify this the ASTM C 1253-93 particle angularity test used in the concrete and asphalt industries was carried out (Table 13).

### Discussion

The results on the whole grading and the plus 075 µm fraction are not significantly different. In terms of the criteria used by the South African asphalt industry (TCAM van Rijckevorsel 2013, personal communication) 35% would be regarded as round, 45% as average and 65% as very angular – the results of 47–50% found would only be regarded as slightly more angular than average, but

---

**Table 12** Results of check gradings and classifications of neat sand base [1]

<table>
<thead>
<tr>
<th>Location (OWP, m)</th>
<th>10 L</th>
<th>30 L</th>
<th>55 L</th>
<th>55 R</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample No</td>
<td>11650</td>
<td>11651</td>
<td>11652</td>
<td>11653</td>
</tr>
<tr>
<td>Sieve size (mm)</td>
<td>Cumulative percentage passing</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.75</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>2.00</td>
<td>94</td>
<td>98</td>
<td>100</td>
<td>95</td>
</tr>
<tr>
<td>1.18</td>
<td>92</td>
<td>97</td>
<td>99</td>
<td>94</td>
</tr>
<tr>
<td>0.850</td>
<td>92</td>
<td>97</td>
<td>99</td>
<td>94</td>
</tr>
<tr>
<td>0.600</td>
<td>91</td>
<td>96</td>
<td>99</td>
<td>93</td>
</tr>
<tr>
<td>0.425</td>
<td>88</td>
<td>93</td>
<td>96</td>
<td>90</td>
</tr>
<tr>
<td>0.300</td>
<td>81</td>
<td>86</td>
<td>87</td>
<td>82</td>
</tr>
<tr>
<td>0.250</td>
<td>74</td>
<td>79</td>
<td>78</td>
<td>74</td>
</tr>
<tr>
<td>0.150</td>
<td>43</td>
<td>45</td>
<td>43</td>
<td>42</td>
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<td>0.075</td>
<td>15</td>
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<td>15</td>
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</tr>
<tr>
<td>Derived data</td>
<td></td>
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</tr>
<tr>
<td>GM</td>
<td>1.03</td>
<td>0.95</td>
<td>0.89</td>
<td>1.00</td>
</tr>
<tr>
<td>Dust ratio</td>
<td>0.17</td>
<td>0.15</td>
<td>0.16</td>
<td>0.17</td>
</tr>
<tr>
<td>Cone IF075</td>
<td>75</td>
<td>84</td>
<td>150</td>
<td>105</td>
</tr>
<tr>
<td>Cup IF075</td>
<td>30</td>
<td>56</td>
<td>60</td>
<td>30</td>
</tr>
<tr>
<td>Classification</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AASHTO</td>
<td>A-2-4(0)</td>
<td>A-2-4(0)</td>
<td>A-2-4(0)</td>
<td>A-2-4(0)</td>
</tr>
<tr>
<td>UNIFIED</td>
<td>SM</td>
<td>SM</td>
<td>SM</td>
<td>SM</td>
</tr>
<tr>
<td>COLTO</td>
<td>G7 ?</td>
<td>G7 ?</td>
<td>G7 ?</td>
<td>G7 ?</td>
</tr>
<tr>
<td>Compactability</td>
<td>[3]</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
</tr>
</tbody>
</table>

**NOTES**

[1] Testing by CSIR

[2] Probably meets CBR requirement of ≥ 25 at 95% for G6, but fails GM requirement of 1.2–1.6

adequate for a sand for use in normal asphalt mixes.

Chemical and mineralogical composition

Physicochemical properties
In order to ascertain whether the outstanding performance of the pure sand base was due to some unusual composition it was also characterised by chemical and mineralogical analysis. The results of tests for organic carbon, pH, paste resistance and sulphate content (not shown) did not indicate any unusual properties.

The P002s of 5–6% found on the base using the more accurate pipette method were much lower than the 13–14% reported by Geostraada on the same samples (Table 9), but were comparable with those of the January samples also tested by them (Tables 3 and 4).

The results (not shown) showed that the neat sand was composed mostly of about 90% SiO₂ occurring as quartz (SiO₂), 4% total Al₂O₃ as kaolinite clay and felspars, with about 1.5% total Fe₂O₃ as goethite, FeO(OH), and that all of these minerals (and some muscovite mica), except quartz, were more concentrated in the P075 fraction. Little or no CaCO₃ was present.

The citrate-bicarbonate-dithionite (CBD)-extractable Al, Fe and Mn (i.e. present as free oxides/hydroxides) in one composite sample of neat sand base were 0.09%, 0.51% and 0.12% calculated as Al₂O₃, Fe₂O₃ and MnO respectively.

It seems that these small amounts of kaolinite and goethite were sufficient to account for the cohesion and to contribute towards the high strengths of the sand, particularly when unsaturated, and the good performance of the neat sand base, as well as the neat sand shoulders.

TRAFFIC HISTORY

Traffic
An estimate of the traffic history of that part of the road, including the experimental sections, is shown in Table 14.

Only eight counts were available for the total of 50 years, and none at all for the first

### Table 14 Traffic history

<table>
<thead>
<tr>
<th>Year</th>
<th>Month</th>
<th>No/ Month</th>
<th>Average annual daily and yearly traffic [1]</th>
<th>Cumulative</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td>v/d</td>
<td>hv/d</td>
<td>v/d</td>
</tr>
<tr>
<td>1962</td>
<td>6</td>
<td>–</td>
<td>–</td>
<td>(150)</td>
</tr>
<tr>
<td>1963</td>
<td>12</td>
<td>–</td>
<td>–</td>
<td>150</td>
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<tr>
<td>1968</td>
<td>12</td>
<td>–</td>
<td>–</td>
<td>–</td>
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<td>1973</td>
<td>12</td>
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<td>–</td>
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<td>1978</td>
<td>12</td>
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<td>–</td>
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<tr>
<td>1983</td>
<td>12</td>
<td>–</td>
<td>–</td>
<td>–</td>
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<tr>
<td>1988</td>
<td>12</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>1993</td>
<td>12</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>1994</td>
<td>12</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>1995</td>
<td>Nov</td>
<td>–</td>
<td>–</td>
<td>1 172</td>
</tr>
<tr>
<td>1997</td>
<td>March</td>
<td>–</td>
<td>–</td>
<td>765</td>
</tr>
<tr>
<td>1998</td>
<td>12</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>2003</td>
<td>12</td>
<td>–</td>
<td>–</td>
<td>–</td>
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<tr>
<td>2005</td>
<td>May</td>
<td>–</td>
<td>–</td>
<td>862</td>
</tr>
<tr>
<td>2007</td>
<td>–</td>
<td>667</td>
<td>37</td>
<td>(500)</td>
</tr>
<tr>
<td>2008</td>
<td>–</td>
<td>780</td>
<td>–</td>
<td>(580)</td>
</tr>
<tr>
<td>2011</td>
<td>Nov</td>
<td>800</td>
<td>–</td>
<td>(600)</td>
</tr>
<tr>
<td>2013</td>
<td>Sept</td>
<td>738</td>
<td>145</td>
<td>848</td>
</tr>
</tbody>
</table>

NOTES
[1] Total of both directions; opened to traffic in June 1962; legal axle loads increased in 1996, e.g. for single dual wheel axle from 8 200 kg to 9 000 kg; based mostly on seven-day counts; figures bracketed are estimates, others are actual counts
[2] Six percent heavy vehicle growth rate assumed between 1963 and 1994; actuals subsequently and assumed same between subsequent counts
[3] Heavy vehicle load equivalency factors based on historical data for similar rural roads, partly in TRH 16 (CSRA 1991) and TRH 4 (COLTO 1996) assuming a load equivalency exponent of 4; those in bold derived from WIM surveys in those years on similar road in similar farming area; however DCP tests suggested a possible exponent of only about 1.2

### Table 13 Results of particle angularity tests on neat sand base [1]

<table>
<thead>
<tr>
<th>Location (OWP)</th>
<th>Units</th>
<th>10 L</th>
<th>30 L</th>
<th>55 L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample</td>
<td>No</td>
<td>3/11650</td>
<td>3/11651</td>
<td>3/11652</td>
</tr>
<tr>
<td>Uncompacted voids [2]</td>
<td>%</td>
<td>50.1</td>
<td>48.7</td>
<td>49.3</td>
</tr>
<tr>
<td>Whole grading [3]</td>
<td>%</td>
<td>47.9</td>
<td>47.4</td>
<td>47.9</td>
</tr>
</tbody>
</table>

NOTES
[1] Testing on oven-dried material by Much Asphalt Gauteng Regional Lab, means of three tests on each sample
[2] BRD of 2.650 assumed for all tests
[4] Dry-sieved
20 years between 1963 and 1995. A heavy vehicle growth rate of 6% per annum from an initial 10% of the AADT (i.e. 15 hv/d) as recommended by the former Transvaal Roads Department (1994) for this class of road was therefore assumed, which also agreed well with the count of 120 hv/d in 1995.

**Discussion**

Although the calculations are accurate, they nevertheless must be regarded as only giving an approximate indication of the cumulative number of standard axles (E80) carried since construction.

As practically no information is available on the early buildup of traffic, or on the traffic split, it is safest to assume that the sections had carried about 0.1 M E80/lane in the first 20 years, about 0.2 in 30 years, about 0.5 in 40 years, and about 1.0 M in 50 years.

During both site visits a significant number of six- and seven-axle multi-trailer vehicles travelling in both directions were noticed. A two-day count of these alone in June 2014 averaged 35/day travelling towards Bultfontein and 30 towards Hoopstad, all approximately 75% loaded (J Nkabinde 2014, personal communication). There is no overloading control on this road and no other split counts were carried out. The use of such vehicles appears only to have commenced in about 1996 and runs throughout the year, with the greatest number during the months of July to December, and with the degree of loading approximately equal in both directions.

Assuming an average load equivalency factor (LEF) of 6.0 per vehicle for these trucks obtained from an HSWIM count in 2007 on a similar road in a similar farming area, it was estimated that these vehicles alone contributed about 180 E80/lane/day or about 50 000 E80/lane/year, and at least one half of the total cumulative E80 carried up to 2013.

**MAINTENANCE**

The original surfacing was a triple seal. As far as can be ascertained the road was only ressealed once with a single resal in 1986, and received one rejuvenation spray at an unknown date.

The presence of only the triple seal and the neat sand section, had also been extensively patched (Photos 2–5), although this had largely been confined to the outer 0.5–1.0 m in both lanes due to the severe and extensive edge-breaking. No evidence of base course patching was found.

**CLIMATE AND WEATHER**

**Climate**

According to the map of macroclimatic regions of southern Africa (Figure 4 in TRH 1996) and in the South African pavement engineering manual (SANRAL 2013), the experiment lies within the “dry” macroclimatic region for pavement design purposes. However it lies close – about 50 km – to the boundary between the dry and moderate macroclimatic regions.

The climatic indices and classifications are as follows:

- Weinert’s (1980) N-value: 5.5.
- Thornthwaite: Dry semiarid warm, moisture deficient in all seasons (Schulze 1947), with a moisture index \(I_m\) of minus 21–22 (Schulze 1958, confirmed from Emery 1992, and Council for...
Table 16 Mean rainfall near the site for the period 1997 to 2013 and actual rainfall from 2012 to April 2014 [1]

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
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<td>2012</td>
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<td>41</td>
<td>49</td>
<td>43</td>
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<td>14</td>
<td>7</td>
<td>3</td>
<td>15</td>
<td>21</td>
<td>59</td>
<td>66</td>
<td>332</td>
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<tr>
<td>2013</td>
<td>mm</td>
<td>25</td>
<td>25</td>
<td>98</td>
<td>54</td>
<td>0</td>
<td>0</td>
<td>3</td>
<td>2</td>
<td>0</td>
<td>35</td>
<td>38</td>
<td>75</td>
<td>355</td>
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<tr>
<td>2014</td>
<td>mm</td>
<td>50</td>
<td>144</td>
<td>62</td>
<td>4</td>
<td>–</td>
<td>–</td>
<td>–</td>
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<td>–</td>
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<td>–</td>
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<tr>
<td>Mean</td>
<td>mm</td>
<td>100.7</td>
<td>62.1</td>
<td>75.7</td>
<td>42.1</td>
<td>23.5</td>
<td>14.6</td>
<td>3.5</td>
<td>8.3</td>
<td>14.8</td>
<td>33.4</td>
<td>56.6</td>
<td>85.2</td>
<td>520.2</td>
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<tr>
<td>SD</td>
<td>mm</td>
<td>76.0</td>
<td>47.3</td>
<td>55.4</td>
<td>42.6</td>
<td>24.8</td>
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<td>6.0</td>
<td>15.8</td>
<td>19.1</td>
<td>34.8</td>
<td>36.1</td>
<td>47.1</td>
<td>181.8</td>
</tr>
<tr>
<td>COV</td>
<td>%</td>
<td>75.4</td>
<td>76.2</td>
<td>73.2</td>
<td>101</td>
<td>101</td>
<td>146</td>
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<td>190</td>
<td>129</td>
<td>104</td>
<td>63.8</td>
<td>55.3</td>
<td>35.0</td>
</tr>
<tr>
<td>Min</td>
<td>mm</td>
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<td>1</td>
<td>12</td>
<td>4</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>4</td>
<td>238</td>
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<td>58</td>
<td>123</td>
<td>116</td>
<td>178</td>
<td>880</td>
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<tr>
<td>Years</td>
<td>no</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
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<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
</tr>
</tbody>
</table>

NOTES
[1] At nearby farm house on Vesuvius 316; 3.5 km southeast of site at altitude of 1 291 m (BD Naudé 2014, personal communication, with statistics by authors)
[2] 52 mm on or before 11 March 2014
[3] 4 mm on 1 April 2014

Scientific and Industrial Research (CSIR 2009).

Köppen: BS
type sands almost everywhere in southern Africa. The results of the experiment [1] showed that the peak N-value of the occurrence of most Kalahari-type sands, the results of the experiment should be conservative as far as the climatic factor is concerned. They should therefore be conservatively applicable to Kalahari-type sands almost everywhere in southern Africa.

### TOPOGRAPHY, GEOLOGY, SOILS AND DRAINAGE

The topography of the area is flat and at an altitude of about 1 280 m. No rock outcrops are shown on the 1:250 000 geological maps (Council for Geoscience 1993, 1994), and the surface material is shown simply as quartzary aeolian sand of unspecified depth, and the underlying geology as shale and subordinate sandstone of the Ecca Group of the Karoo Supergroup, intruded in places by Jurassic dolerite.

The soil maps of the area (Land Type Survey Staff 1986, 2012) show the soils of the area to be red-yellow in colour, apedal, freely drained, with a high base status, and with a “clay” (i.e. passing 2 µm) content of usually less than 15%. These soils are deep, sandy, aeolian soils of the Hutton soil form in the Kalahari vegetation unit and mostly of the Avalon soil form in the Grassland unit, which latter was confirmed by the three pits dug in the road reserve between km 19.7 and 19.8 (Photo 1), together with the laboratory test results on samples from both these pits (Table 4). Such soils are characterised by their yellow-brown colour, well-drained, fine sandy nature with a low clay content (<6%), but with seasonal wetness.

Local information indicates that the permanent water table lies at a depth of about 30 m, but that a perched water table can develop at a depth of about 2 m due to an underlying layer of clay – presumably the residual shale.

### SUGGESTED SAND BASE COURSE SPECIFICATION

#### Material

Based only upon the results of this investigation, the following is suggested as a specification for a sand base course material for sealed, low-volume roads designed to carry up to about 0.1M E80 over 20 years:

**Essential:**
- Colour: yellowish-brown or reddish-brown (not white or grey)
- AASHTO classification: A-2-4(0)
- Unified classification: SM
- GM: 0.75–1.10
- P075: 10–25%
- TMH 1 PI on P425 fraction: NP-SP
- TMH 1 PI on P075 fraction: SP-6
- TMH 1 IF075: 20–120
- Minimum soaked 2.54 mm CBR at 100% MAASHO: 50
- Minimum unsoaked 2.54 mm CBR at OWC at 100% MAASHO: 60
- Maximum MAASHO CBR swell: 0.1%

- Minimum CBD-extractable Fe: 0.3% or, less reliably, minimum Fe2O3 content by XRF analysis: 1.2% Fe2O3.

#### Probably desirable:
- Sand equivalent: 13–40
- Particle angularity:
  - Minimum uncompacted voids (ASTM C1252) on the plus 075 µm fraction: 45%, or
  - Mostly angular particles visible under stereo microscope
  - Dominant clay mineral: kaolinite.

#### Terrain:
- Relatively flat
- Drainage: good
- Permanent or perched water table: at least 1.0 m below top of roadbed

#### Construction:
- Cross-section: surface camber or adequate (≥ 3% ?) crossfall
- Seal: at least a double seal
- Prime: required

- Compaction:
  - Base to refusal, or at least 100% MAASHO, whichever is the lesser.
  - Shoulders, sub-base (if not cemented), selected subgrade and fill of similar sand to at least 100% MAASHO
  - Roadbed to at least 95% MAASHO, preferably with deep compaction by impact, vibrating or heavy pneumatic roller if collapsing

#### Seal maintenance:
- Good

**Discussion**

As there were no failures, the material specifications simply attempt to circumscribe the apparently desirable properties of the...
material tested, and it is uncertain which can be safely omitted – or which perhaps still need to be added.

The purpose of specifying colour is to ensure that there are some suitable iron oxide/hydroxide minerals present and to act as a field proxy for chemical analysis, which is difficult to get done by the more desirable CBD method, and which cannot be done on site.

The most essential requirements would appear to be to have at least about 10% P075 with just sufficient plasticity to provide some cohesion to allow good compaction, stability during construction, and an adequate CBR, both soaked and unsoaked, as well as a sufficiently stiff platform for compaction.

The use of a more sophisticated grading, such as that of the Botswana Roads Department (BRD 2010), involving phi grading units does not appear to be necessary.

The limitation of the design traffic to about 0.1M E80 at this stage of our knowledge is deliberately conservative in view of the absence of traffic counts over the first 20 years, the well-known beneficial effects of slow remoulding by traffic (Van Niekerk 1953; Kleyn & Savage 1982; De Beer et al. 1989), and the uncertainty both regarding the correct load equivalency exponent, and the degree of redelta grading units does not appear to be necessary.

The site work was carried out by a team from the Free State Department of Police, Roads and Transport under the authors’ supervision.

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The experimental sections were constructed as a joint project between the Department and the CSIR.

Most of the laboratory engineering testing was carried out by Geostrada Engineering International (Pty) Ltd, with some check testing at the CSIR Division of Built Environment, and the Institute for Soil, Climate and Water, all in Pretoria.

<table>
<thead>
<tr>
<th>CONCLUSIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>This investigation has shown that, as predicted by Gregg (1963), provided that it does not become saturated, a fine, A-2-4(0) aeolian sand with sufficient fines of low plasticity and a good CBR can be used as an untreated base course for a low-volume road with an expected life of more than 20 years.</td>
</tr>
<tr>
<td>This finding should go some way towards the provision of low-volume sealed roads in the vast area of arid and semiarid southern Africa covered with similar sands and devoid of conventional gravels.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ACKNOWLEDGEMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>This investigation was funded by UKaid from the United Kingdom Department of International Development, through the Africa Community Access Programme (AFCAP), which promotes safe and sustainable access to markets, healthcare, education, employment and social and political networks for rural communities in Africa, endorsed by the Association of Southern African National Roads Authorities (ASANRA), and managed by the Crown Agents (Project Technical Manager: Mr R Geddes). It formed part of a southern Africa-wide project on the increased use of sands in low-volume roads under the auspices of ASANRA (<a href="http://www.asanra.int.mw">www.asanra.int.mw</a>). A more comprehensive report is available (<a href="http://www.afcap.org">www.afcap.org</a>).</td>
</tr>
<tr>
<td>The site work was carried out by a team from the Free State Department of Police, Roads and Transport under the authors’ supervision.</td>
</tr>
<tr>
<td>This paper is published with the permission of the above authorities, but the views expressed do not necessarily reflect those of the above, or their official policies.</td>
</tr>
<tr>
<td>The experimental sections were constructed as a joint project between the Department and the CSIR.</td>
</tr>
</tbody>
</table>

**COMPARISON WITH SOME EXISTING SAND BASE SPECIFICATIONS**

**Sand bases with a capillary-soaked triaxial class of 3.3 at intermediate compaction have been used in Zimbabwe for roads designed to carry up to 0.1 M E80 in both directions over 10 years (Mitchell et al 1975; Mitchell 1982). However, although the Hoopstad samples tested would meet the equivalent soaked CBR requirement of about 35–55, their gradings are too fine.**

A novel approach to the selection of sands in West Australia using sedimentological phi (Φ) units instead of mm for particle size characterisation in terms of the mean and standard deviation, is that of Metcalf and Wylie (1984). Plots of the Hoopstad samples (not shown) showed them to all fall outside of their Zone B (which included most successful sealed bases) due to insufficient fines.

A preliminary specification for Kalahari sands for base course in Botswana (BRD 2010) is as follows:

\[
\frac{L_{0.10}}{\Phi_s} = 5 – 10
\]

- Soaked British Standard vibrating hammer (BSVH) CBR: ≥ 60%
- Total Al₂O₃ + Fe₂O₃ content: > 8%
- Field compaction (BSVH): ≥ 100%

Although the Hoopstad sand might have met this CBR requirement, it would have failed this specification on account of its unsatisfactory LS/Φs ratios of less than 5 (LSs all too low) and an Al₂O₃ + Fe₂O₃ content of less than 8%.

**REFERENCES**


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APPENDIX
NOTATION AND METHODS

Soil engineering tests

- California Bearing Ratio (CBR): TMH 1: 1986 — in accordance with normal South African practice, unless otherwise stated, at a specified percentage of compaction relative to the MAASHO MDD, at a penetration depth of 2.54 mm after soaking for at least four days (NITRR 1986).
- Carbonation and presence/absence of cement using 0.5% phenolphthalein.
solution and diluted hydrochloric acid (HCl) (Netterberg 1984), except that 1.2 N HCl was used instead of the previously recommended 5 N, as it has subsequently been found to be more sensitive.

- **Compactive effort:** TMH 1: 1986: Modified American Association of State Highway Officials (MAASHO), i.e. 2 413 kJ/m³ (which is less than the current heavy American Association of State Highway and Transportation Officials (AASHTO) T180 effort of 2 695 kJ/m³); National Road Board (NRB, i.e. Intermediate), i.e. 1 096 kJ/m³; and Proctor, i.e. 531 kJ/m³ (Department of Transport) (DOT 1970).
- **Cone Liquid Limit:** BS 1377: 1990: Part 2.
- **DCP tests on CBR specimens:** Average DN through specimen after CBR test, with annular weight in place as used by EG Kleyn (1984), (2013, personal communication), and Sampson and Netterberg (1990), with the cone zero at the bottom of the CBR indentation and with heave measurements during test (which were usually zero, rarely up to 4 mm).
- **Dynamic Cone Penetrometer (DCP):** TMH 6: 1984 (NITRR 1984), with the cone zero at the top of the seal.
- **Field Moisture Equivalent (FME):** AASHTO T93-86 (1936), (AASHTO 1998a).
- **Fineness Index (FI075, FI075):** P075 × P105. (Mainwaring 1968). (Note: When P105 = NP or 0, then FI075 = P075).
- **Laboratory test methods in general:** TMH 1: 1986 (NITRR 1986).
- **Particle angularity:** ASTM C 1252 – 93 (ASTM 1995).
- **Particle size distribution (“grading”):**
  - P425, P075, etc: Cumulative percentage passing 425, 075 µm, sieves, etc.
  - R425, R075, etc: Cumulative percentage retained on 425, 075 µm, sieves, etc.
- **Soil classification:**
- **Unified:** ASTM D2487-11 (ASTM International 2013).
- **Soil preparation:**
  - Passing 0.425 mm fraction (P425) for soil constants: TMH 1: 1986 Method A–1(a).
  - Passing 0.075 mm fraction (P075) for soil constants: SANS 3001 – GR1: 2008 (SABS 2008).
- **Unsoaked CBR:** At optimum water content (OWC) for MAASHO effort after four days equilibration in sealed plastic bags; at less than OWC after drying in the sun or oven at ≤ 60 °C to the approximate percentage OWC specified and then sealing in plastic bags for at least four days.

### Soil science tests
- **pH, paste resistance, dithionite-citrate-bicarbonate (CBD) – extractable free iron (Fe), aluminium (Al) and manganese (Mn) oxides/hydroxides; organic carbon (by Walkley-Black dichromate oxidation), and particle size distribution (washed sieve plus pipette method):** The Non-Affiliated Soil Analysis Work Committee (1990).
- **Soil colour:** Standard Munsell colours (Soil Colour Chart compiled by Soils Research Institute, Pretoria, undated; Munsell Color Co, Inc, Baltimore).
Hazard of sinkhole formation in the Centurion CBD using the Simplified Method of Scenario Supposition

A C Oosthuizen, J L van Rooy

A large part of the land south of Pretoria is underlain by dolomite from the Chuniespoort Group of the Transvaal Supergroup. In South Africa, dolomite rock has a notorious reputation for the formation of sinkholes and subsidences. Thousands of people reside and work in the Centurion area, where numerous sinkholes have occurred, causing damage and in some instances loss of property and even lives. Centurion has rapidly densified over the last 40 years, with an increase not only in the number of people, but also in the density of waterborne services. This paper proposes draft guidelines for the allocation of an Inherent Hazard Class for percussion boreholes, referred to as the ‘Simplified Method of Scenario Supposition’. This method was then used to classify the Centurion CBD and surrounding area.

INTRODUCTION

The greater part of land in the area directly south of Pretoria is underlain by dolomite from the Chuniespoort Group of the Transvaal Supergroup. In South Africa, dolomite rock has a notorious reputation for the formation of sinkholes and subsidences. Thousands of people reside and work in the Centurion area, where numerous sinkholes have occurred, causing damage and in some instances loss of property and even lives. Standard practice in South Africa is to execute geotechnical investigations on all dolomitic land earmarked for development, irrespective of the type of development.

Dolomite stability reports are submitted to the National Dolomite Databank at the Council for Geoscience (CGS). It is apparent from the available dolomite stability reports submitted over the last 30 years, that hazardous conditions exist in the Central Business District (CBD) area of Centurion, Pretoria. Centurion has rapidly densified over the last 40 years due to its locality between Johannesburg and Pretoria. The Gautrain train route now traverses the Centurion CBD area, and the Centurion Station area around West Street has attracted high-rise developments. This will lead to a further increase in population, with associated increased road traffic and density of people per hectare. The available data, comprising 3 587 percussion borehole profiles from 555 dolomite stability reports, could be used to attempt a first-order sinkhole hazard analysis.

The generally accepted current method used to determine the sinkhole hazard is the so-called ‘Scenario Supposition Method’ (Buttrick et al 2001). This method is incorporated into SANS 1936-2:2012 (SANS 1936), since the principles of this method are currently most commonly being used by practitioners in South Africa.

STUDY AREA

The Centurion CBD is demarcated by John Vorster Road in the south, Jean Avenue in the north, the N1 highway in the southeast and South Street in the east. The study area includes some of the surrounding suburbs, and is bounded by Trichardt Road in the north, Botha Avenue in the east, the N1 highway in the south and the N14 highway in the west (Figure 1).

The study area covers a surface area of approximately 1 657 hectares and is relatively flat-lying, sloping gently towards the Hennops River which cuts through the centre of the CBD. The majority of the CBD and surrounding areas is developed, with commercial developments around the Centurion Lake and residential development in the surrounding areas (Figure 1).

GEOLOGY OF THE CENTURION CBD AREA

Most of the CBD area is underlain by chert and dolomite rocks of the Monte Christo Formation, with the Lyttelton Formation present along the eastern boundary of the study area, and the Oaktree Formation occupying a small area in the southern corner. Dolomite from the Lyttelton and

Keywords: dolomite, sinkhole, hazard, Scenario Supposition, Centurion.
Oaktree Formations are generally chert-poor compared to the chert-rich Monte Christo Formation. Syenite intrusions in the form of sills and dykes are present in the dolomite bedrock, and a specific large syenite sill is present towards the southern boundary of the Centurion CBD area in Zwartkop. A prominent north–south trending dyke is present along the eastern boundary of the CBD, with a smaller northwest–southeast trending dyke in the Lyttelton Agricultural Holdings. Alluvial deposits occur in the Hennops River floodplain and a small Karoo outlier (Vryheid Formation) is present on the northwestern boundary.

**GEOHYDROLOGY**

The Centurion CBD area is situated in the Irene catchment which comprises four sub-catchments, or compartments which are hydraulically connected, as evidenced by the direction of groundwater flow (Hobbs 1988). The four sub-catchments are analogous to the Fountains West, Fountains East, Doornkloof West and Doornkloof East compartments described by Vegter (1986).

Most of the CBD area is situated in the Fountains West groundwater compartment. Hobbs (1988) indicates that an extremely weak groundwater gradient of some 0.2% is manifested from immediately north of the Hennops River in a north-north-easterly direction towards the Fountains West spring, indicating a high transmissivity of the dolomite aquifer in this sub-catchment. According to a groundwater level contour map by Hobbs, the groundwater level of the Fountains West groundwater compartment in this area ranges from 1 416 m amsl in the south to 1 385 m amsl in the north. This constitutes a range of 48 m below ground surface in the south to 91 m below ground surface in the north.

The Fountains East groundwater compartment is present along the eastern boundary of the CBD area. This compartment drains in a north-westerly direction to the East Fountain Spring in the north (Hobbs 1988). According to Hobbs, the groundwater level of the Fountains East groundwater compartment in this area ranges from 1 429 m amsl in the south to 1 425 m amsl in the north, indicating a relatively flat groundwater level across this compartment. The groundwater level of the CBD area therefore ranges between 16 m below ground surface in the south to 20 m below ground surface in the north.

Recent groundwater studies for specifically the Gautrain Rapid Rail and other developments in the Centurion area revealed that groundwater abstraction (mainly by the municipality) does not have an influence on the aquifer, as the Irene catchment is in a relatively undisturbed condition (Ove Arup 2009). Some localised...
lowering of the groundwater level has been observed in the area of the Kentron borehole where water is abstracted by the municipality, but this borehole is situated in the Doornkloof West groundwater compartment, which does not form part of the study area. Groundwater monitoring borehole A2N0528, situated towards the west of the CBD area, also shows a relatively stable groundwater level (with seasonal fluctuations) between 1985 and 2009, although the groundwater has risen three to four metres during 1995 and 1996. The Centurion CBD area is therefore considered to not be affected by dewatering.

SINKHOLE DATABASE
A total of 119 sinkholes have been recorded in the Centurion CBD area since the early 1970s until mid-2012, and their positions are indicated on Figure 2 (page 74). The sinkhole records are under review and many of the events lack detailed information. Most of the records in the database were captured by CGS staff, while some information was also obtained from the City of Tshwane. At present, this database is not available to the public, mainly due to the sensitivity of this information and the effect that it might have on property value.

Consequence of sinkhole occurrence
The occurrence of sinkholes has led to the demolition of seven houses and other structures, with loss of money to landowners, while three lives were also lost. The loss of life happened during the rehabilitation of a sinkhole.

Photographs 1 and 2 present some examples of sinkholes in the Centurion CBD area.

Nature of sinkhole occurrences
The following conclusions could be drawn from the CGS sinkhole database:
- The surface extent of the largest reported sinkhole is 32 m x 23 m and occurred in Lyttelton as a result of a broken water pipe.
- The deepest reported sinkhole is 10 m and was caused by a leaking sewerage pipe.
- Only one sinkhole occurred on syenite close to the John Vorster and Jean Avenue intersection (detailed information on this event is lacking).
- A total of 110 sinkholes (92% of database) occurred in the Monte Christo Formation and only 8 (7%) in the Lyttelton Formation. No sinkholes were recorded in the Oaktree Formation (Table 1).
- Ninety records (75.6%) of the database have information regarding the type of event, with most of the events listed as

<table>
<thead>
<tr>
<th>Geological succession / formation</th>
<th>Area (ha)</th>
<th>No of sinkholes occurred</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lyttelton Formation</td>
<td>91.82</td>
<td>8</td>
</tr>
<tr>
<td>Monte Christo Formation</td>
<td>1 246.61</td>
<td>103</td>
</tr>
<tr>
<td>Oaktree Formation</td>
<td>44.80</td>
<td>0</td>
</tr>
<tr>
<td>Chert Breccia</td>
<td>9.77</td>
<td>0</td>
</tr>
<tr>
<td>Dolerite</td>
<td>35.10</td>
<td>1</td>
</tr>
<tr>
<td>Quartz-diorite (syenite intrusions)</td>
<td>171.34</td>
<td>5</td>
</tr>
<tr>
<td>Alluvium</td>
<td>58.31</td>
<td>2</td>
</tr>
</tbody>
</table>

Table 1 Number of sinkholes that have occurred in each stratigraphic formation

Photo 2 A sinkhole that occurred in a residential complex, which led to the demolition of two units
sinkholes. Graph 1 indicates the distribution of type of instability events. The average sinkhole depth is 3.24 m (data from 47 sinkholes). The average sinkhole diameter is 5.1 m (data from 53 sinkholes).

**Quality of the sinkhole database**
The sinkhole data is being verified by the CGS, with approximately 90% having been completed. The database is not regarded as a complete record of all events and more sinkholes could have occurred in the area.
There is no obligation to report these instability events and the records are therefore not always complete. Only 58 of the sinkhole records (49%) have information regarding the possible causes (Graph 2). Although limited, the information shows that 93% of the events in the Centurion CBD occurred as a result of human disturbance of the natural ground conditions, confirming deductions by Buttrick et al (2001).

Sinkhole size distribution
Buttrick and Van Schalkwyk (1995) proposed a scale of sinkhole sizes based on the potential development space that was slightly amended by Buttrick et al (2001), and is widely used to refer to a specific sinkhole size (Table 2).

Graph 3 shows the size distribution of the sinkholes based on Table 2, with just fewer than half of the sinkholes (49.1%) falling into the medium-sized sinkhole range (2 m to 5 m diameter).

The sinkhole size distribution (Graph 4) indicates that almost 10% of all sinkholes were smaller than 1 m in diameter. Almost a third (28.3%) are between 2 m and 3 m, followed by 13.2% between 3 m and 4 m in diameter. Only 17% of all sinkholes were larger than 10 m, whereas the remaining (83%) sinkholes are less than 9 m in diameter. Graph 4 confirms that medium-sized sinkholes (2 m – 5 m) dominate in the Centurion CBD area.

It should be noted that these graphs only represent the available information (45% of records).

According to the information in the available sinkhole database, 40% sinkholes or subsidences formed as a result of leaking water-bearing services, 29% as a result of poor surface/stormwater management and 22% as a result of inadequate or poor precautionary measures. Only one sinkhole (2%) occurred as a result of a poorly back-filled borehole, whereas 7% occurred as a result of poor subsurface conditions. Using limited information (49% of the database has information relating to the cause thereof) it is evident that 93% of the events in the Centurion CBD area occurred as a result of man’s disturbance of the natural ground conditions.

CLASSIFICATION IN TERMS OF THE HAZARD OF SINKHOLE FORMATION
Since there are no numerical limits to the Scenario Supposition method, draft guidelines for allocation of each hazard class, based on CGS institutional memory and experience, have been developed. This approach is mainly based on the dolomite bedrock depth and the mobilisation potential of the overlying horizons. The size of sinkhole that could develop is a function of the depth to dolomite bedrock, i.e. the thinner the overburden the smaller size sinkhole is expected and vice versa.

Determination of the sinkhole hazard follows a process where an Inherent Hazard Class is assigned to each geotechnical investigation borehole, depending on the characteristics of the material encountered in that borehole. The basic guidelines are provided in Table 3, specifically for the Centurion CBD and the non-dewatering condition, since dewatering has had no influence on stability in Centurion.

This hazard determination is merely based on the assumption that a larger size sinkhole will develop in deeper dolomite bedrock environments. This method is based on experience and the understanding of dolomite stability conditions through using the Method of Scenario Supposition (Buttrick et al 2001), but it provides a simplified version of the eight Inherent Hazard Classes; therefore it is proposed as the Simplified Method of Scenario Supposition.
Table 4 indicates the distribution of the Inherent Hazard Classes in the Centurion CBD area, with most of the boreholes (953) falling into IHC 3, followed by IHC 4 (857) and IHC 5 (672).

The Spatial Analyst® extension of ArcMap® was used to create an Inherent Hazard Classification Map in terms of the low, medium and high classification (Figure 2).

It should be noted that the sinkhole information is very sensitive and more detail could not be made available to the general public.

Table 5 Coverage of each hazard class in the Centurion CBD area, from Figure 2

<table>
<thead>
<tr>
<th>Inherent Hazard Class</th>
<th>Surface area (hectares)</th>
<th>Percentage cover in Centurion CBD area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>73</td>
<td>4.4%</td>
</tr>
<tr>
<td>Medium</td>
<td>1 111</td>
<td>67%</td>
</tr>
<tr>
<td>High</td>
<td>473</td>
<td>28.6%</td>
</tr>
</tbody>
</table>

Table 6 Number of sinkholes that have occurred in each of the hazard classes

<table>
<thead>
<tr>
<th>Inherent Hazard Class</th>
<th>No of sinkholes</th>
<th>Sinkholes as percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>0</td>
<td>0%</td>
</tr>
<tr>
<td>Medium</td>
<td>83</td>
<td>69.7%</td>
</tr>
<tr>
<td>High</td>
<td>36</td>
<td>30.3%</td>
</tr>
</tbody>
</table>

This map has been used to calculate the coverage and surface area of each of the Inherent Hazard Classes, and the respective percentages are given in Table 5.

It is evident that two thirds of the Centurion CBD area represents a medium Inherent Hazard for sinkhole formation, and almost a third can be considered as having a high Inherent Hazard for the formation of sinkholes, with only a small portion of the area (4.4%) representing low-hazard conditions.

Comparison between the CBD hazard map and sinkhole occurrence

The 119 sinkholes that occurred in the Centurion CBD area were plotted to compare the occurrence of sinkholes against the low-, medium- and high-hazard areas (Figure 2). Table 6 shows the number of sinkholes in each of the hazard areas.

The comparison between the hazard map and the previous occurrence of sinkholes generally does not correlate well. The map does show that no sinkholes occurred in the areas classified as having a low hazard for sinkhole formation, which suggests that the delineation of low-hazard areas was accurate and that the classification system defines these areas well. Surprisingly, a vast majority (69.7%) of the sinkholes in the Centurion CBD area occurred in areas classified as having a medium hazard for the formation of sinkholes. It should, however, also be borne in mind that 67% of the area was classified as having a medium hazard for sinkhole formation.

Another influencing factor to consider is that the high-hazard areas are generally not developed, whereas the medium-hazard areas are densely developed, therefore more wet services are present in these areas, resulting in a higher number of sinkholes. The position, volume, type and age of wet
services also contribute to the type, size and time of sinkhole formation, but this was not studied in detail in this paper.

CONCLUSIONS
1. The greater part of land in the area directly south of Pretoria is underlain by dolomite from the Chuniespoort Group of the Transvaal Supergroup. In South Africa dolomite rock has a notorious reputation for the formation of sinkholes and subsidences. Thousands of people reside and work in the Centurion area, where numerous sinkholes have occurred.
2. From the available dolomite stability reports submitted to the Council for Geoscience over the last 30 years it became evident that hazardous conditions exist in the CBD area of Centurion, Pretoria. This first-order sinkhole hazard analysis incorporates the borehole information (3 587 percussion borehole profiles) extracted from 555 dolomite stability reports.
3. The area covers a surface of approximately 1 657 hectares where a total of 119 sinkholes have occurred up to mid-2012. This constitutes one sinkhole per 13.9 hectares or 7.2 sinkholes per square kilometre.
4. It is evident that 93% of the instability events in the Centurion CBD area occurred as a result of man’s disturbance of the natural ground conditions.
5. Medium-sized sinkholes prevail in this area, with just less than half of the sinkholes (49.1%) being considered as medium-sized. The database showed that 30.2% were classified as large-sized sinkholes, while small-sized sinkholes constituted 15.1% of the events, with only 5.7% being more than 15 m in diameter, i.e. very large sinkholes.
6. Draft guidelines for allocation of an Inherent Hazard Class have been developed. This approach is mainly based on the dolomite bedrock depth and the mobilisation potential of the overlying horizons. The size of sinkhole that could develop is again a function of the depth to dolomite bedrock, i.e. the thinner the overburden the smaller size sinkhole is expected and vice versa. The sinkhole size is not calculated by using the angle of draw as proposed in the Method of Scenario Supposition (Buttrick et al 2001), although most of the other factors are considered in this methodology, which is based on the same principles of the Method of Scenario Supposition. This approach simplifies the Method of Scenario Supposition, and it is therefore proposed as the ‘Simplified Method of Scenario Supposition’.
7. A hazard map of the Centurion CBD area, using the Simplified Method of Scenario Supposition with the Spatial Analyst® extension of ArcGIS®, was created. This map generally indicates a medium to high susceptibility to sinkhole formation, with pockets of low-hazard areas. The following conclusions could be made from the hazard classification of the Centurion CBD and surrounding areas:
   • The conditions are not as poor as had always been perceived.
   • The largest area of high-hazard conditions is present in the area immediately north and east of the Hennops River and Centurion Lake.
   • The largest area of low-hazard conditions is present in the area of Zwartkop.
   • The Centurion CBD area is mostly represented by medium-hazard conditions (Inherent Hazard Classes 3 and 4), which constitute 50.5% of the boreholes in the area.
   • Only 2.3% of the boreholes in the Centurion CBD area were classified as Inherent Hazard Class 1, whereas 2.8% of the boreholes were classified as Inherent Hazard Class 8.
   • Almost two thirds of the Centurion CBD area represents a medium hazard for sinkhole formation, with almost a third of the area considered as having a high hazard for the formation of sinkholes, and only a small portion of the area (5%) representing low-hazard conditions.
8. Upon comparison of the hazard map and the previous occurrence of sinkholes, it became evident that the vast majority (70%) of the sinkholes occurred in the areas classified as having a medium hazard for sinkhole formation. Two thirds of the study area is classified as having a medium hazard for sinkhole formation, which is where most of the wet services are present. Leaking wet services have directly caused 40% of the sinkholes in this area, as revealed from available information.
9. Additional research has been conducted, based on the outcomes of this study, which will be presented in the near future.

REFERENCES

BIBLIOGRAPHY
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- Their positions should be clearly marked in the text as follows: [Insert Figure 1]
- Figures, tables, photos, illustrations and equations should be numbered consecutively and should appear in the text directly after they have been referred to for the first time.
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- Tables should be typeset in Times New Roman 9 pt font. They should not duplicate information already given in the text, nor contain material that would be better presented graphically. Tabular material should be as simple as possible, with brief column headings and a minimum number of columns.
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  - Equations should be presented in a clear form which can easily be read by non-mathematicians. Each equation should appear on a separate line and should be numbered consecutively.
  - Symbols should preferably reflect those used in Microsoft Word Equation Editor or Mathtype, or should be typed using the Times New Roman symbol set.
- Variables in equations (x, y, z, etc) as well as lower case Greek letters should be presented in italics. Numbers (digits), upper case Greek letters, symbols of metric measurement units (in, metres, s for seconds, etc) and mathematical/trigonometrical functions (such as sin, cos and tan) are not written in italics, but in upright type (Roman).
- Variables and symbols used in the body of the text should match the format used in the equations; i.e. upright or italics, whichever is applicable.
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- Decimal commas may be used, but decimal points are preferred.
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  - Heading of subsection
  - Heading of sub-subsection
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