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

\[ k_x \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0 \]
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Are estimates of catchment response time inconsistent as used in current flood hydrology practice in South Africa?

Catchment response time parameters are one of the primary inputs required when design floods, especially in ungauged catchments, need to be estimated. The time of concentration ($T_c$) is the most frequently used time parameter in flood hydrology practice, and continues to find application in both event-based methods and continuous hydrological models. Despite the widespread use of the $T_c$, a unique working definition and equation(s) are currently lacking in South Africa. This paper presents the results of the direct and indirect $T_c$ estimation for three sets of catchments, which highlight their inherent variability and inconsistencies. These case studies demonstrate that estimates of $T_c$ using different equations, may differ from one another by up to 800%. As a consequence of this high variability and uncertainty, we recommend that, for design hydrology and calibration purposes, observed $T_c$ values should be estimated using both the average catchment $T_c$-value, which is based on the event means, and a linear catchment response function. This approach is not only practical, but also proved to be objective and consistent in the study areas investigated in this paper.

INTRODUCTION

Design flood events, i.e. floods characterised by a specific magnitude–frequency relationship at a particular site, are very sensitive to the estimated time parameter values. Various researchers (e.g. Bondelid et al 1982; McCuen et al 1984; McCuen 2009) demonstrated that as much as 75% of the total error in estimates of peak discharge could be ascribed to errors in the estimation of time parameters. Gericke and Smithers (2014) showed that the underestimation of time parameters by 80% or more could result in the overestimation of peak discharges of up to 200%, while the overestimation of time parameters beyond 800% could result in maximum peak discharge underestimations of up to 100%. Such errors in the estimation of time parameters could not only result in either the over- or under-design of hydraulic structures, but are also linked to several socio-economic implications and could result in infeasible projects. Consequently, catchment response time parameters are regarded as one of the primary inputs required when design floods need to be estimated, especially in ungauged catchments. The time of concentration ($T_c$), lag time ($T_l$) and time to peak ($T_p$) are the time parameters commonly used to express the catchment response time.

$T_c$ is the most frequently used and required time parameter in flood hydrology practice (Gericke & Smithers 2014) and continues to find application in both event-based methods (SANRAL 2013) and continuous hydrological (stormwater) models (USACE 2001; Neitsch et al 2005). Despite the widespread use of all these time parameters, unique working definitions for each of the parameters are not currently available. However, the use of several conceptual and computational time parameter definitions is proposed in the literature, as summarised by McCuen (2009), and Gericke and Smithers (2014), some of which are adopted in practice.

The simultaneous use of these different time parameter definitions, as proposed in the literature, combined with the lack of continuous recorded rainfall data and available direct measurements of rainfall–runoff relationships, has curtailed the establishment of unbiased time parameter estimation procedures internationally (Grimaldi et al 2012). South Africa (SA) is no exception – none of the empirical $T_c$ estimation equations recommended for general use have been tested, or developed and verified using local data. The South African National Roads Agency Limited (SANRAL 2013) recommends the use of the Kerby equation (Kerby 1959) developed for small, flat catchments with overland flow being dominant.

Keywords: catchment response time, lag time, peak discharge, time of concentration, time to peak

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TechNIcAL PAPer
but the Kerby equation is widely applied in an urban stormwater context in SA (e.g. roads, paved parking lots, business and industrial areas, residential lots, etc.). Apart from the Kerby equation, the $T_C$ equation of the United States Department of Agriculture, Soil Conservation Service (USDA SCS 1985), developed for catchment areas up to 30 km², is also sometimes used in SA to estimate overland flow $T_C$ by recognising the relationship of $T_C$: $T_L = 1.417$ (McCuen 2009). In applying the overland flow $T_C$ equations, a practising engineer would typically use flow-length criteria, i.e. overland flow distances associated with specific slopes, as a limiting variable to quantify overland flow conditions (Matthee et al 1986; McCuen & Spiess 1995), but flow-retardant factors, Manning’s overland roughness parameters and overland conveyance factors are also sometimes used (Vieiseman & Lewis 1996; Seybert 2006; USDA NRCS 2010).

In medium to large (50 km² to 35 000 km²) catchments where channel flow dominates, the empirical United States Bureau of Reclamation (USBR) equation (USBR 1973) is the recommended equation in SA to estimate the $T_C$ in a defined watercourse (SANRAL 2013). At these catchment levels, the current common practice used by engineers is to divide the principal flow path into overland flow (if significant, otherwise regarded as channel flow) and main watercourse or channel flow, after which the travel times in the various segments are computed separately and totalled. Gericke and Smithers (2014) demonstrated the inconsistency amongst various channel flow $T_C$ equations applied at this catchment scale, along with their associated inherent limitations. It was argued that these equations would show even more significant variations if compared to observed catchment response times. Consequently, Gericke and Smithers (2014) proposed the use of an alternative and consistent approach to estimate $T_C$ from observed streamflow data by recognising the approximation of the conceptual $T_C = T_P$ and assumption that the volume of effective rainfall equals the volume of direct runoff when a hydrograph is separated into direct runoff and baseflow. In using such an approach, the convolution process normally required between a single hyetograph and hydrograph to estimate $T_C$ is eliminated, since only observed streamflow data is used without the need for rainfall data (Gericke & Smithers 2014). Acknowledging that the ‘traditional’ convolution process is not only impractical, but also not applicable in real, large heterogeneous catchments (where antecedent moisture from previous rainfall events and spatially non-uniform rainfall hyetographs can result in multi-peaked hydrographs), the conceptual and practical value of using such an alternative approach is recognised and warrants further investigation.

The objectives of the study reported in this paper are discussed in the next section, followed by a description of the case studies. Thereafter, the methodologies involved in meeting the objectives are detailed, followed by the results, discussion and conclusions.

### PURPOSE OF STUDY

In this paper, selected definitions and associated estimation procedures are utilised for the analysis of three case studies with the two-fold objective of critically investigating the similarity between $T_C$ and $T_P$ at a medium to large catchment scale, and comparing different estimation methods. The latter comparison focuses on the use of direct estimation (from observed streamflow data in medium to large catchments) and indirect estimation (empirical equations) methodologies. The specific objectives of this paper are: (i) to compare a selection of overland flow $T_C$ equations using different slope-distance classes and roughness parameter categories to highlight any inherent limitations and inconsistencies; (ii) to explicate the variability of $T_C$ estimations resulting from the $T_C = T_P$ approach implemented on observed streamflow data at a medium to large catchment scale, and (iii) to ascertain the inherent limitations and inconsistencies of the empirical channel flow $T_C$ equations when compared to the direct estimation of $T_C$ from observed streamflow data.

The three case studies are presented in the next section.

### CASE STUDIES

Three case studies were selected to benchmark the different equations commonly used internationally to estimate $T_C$ in practice at different catchment scales, and to investigate their similarities, differences and limitations.

(a) Conceptual urban catchment

Urban catchments are normally characterised by highly variable and complex flow paths. Consequently, instead of using actual urban catchments, a conceptualised urban catchment setup, with overland flow being dominant, is selected by considering the combination of different variables, such as flow-length criteria (i.e. overland flow distances associated with specific slopes), overland conveyance factors ($\phi$), flow-retardant/imperviousness factors ($i_p$), Manning’s overland roughness parameters ($n$) and runoff curve numbers (CN). The flow-length criteria are based on the recommendations made in the National Soil Conservation Manual (NSCM) (Matthee et al 1986). The NSCM criteria (Table 1) are based on the assumption that the steeper the overland slope, the shorter the length of actual overland flow before it transitions into shallow-concentrated flow, followed by channel flow. A total of five categories defined by different $\phi$, $i_p$, $n$ and CN values in seven slope-distance classes are considered.

<table>
<thead>
<tr>
<th>Slope class ($S_D$) (%)</th>
<th>Distance ($L_D$) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0–3</td>
<td>110</td>
</tr>
<tr>
<td>3.1–5</td>
<td>95</td>
</tr>
<tr>
<td>5.1–10</td>
<td>80</td>
</tr>
<tr>
<td>10.1–15</td>
<td>65</td>
</tr>
<tr>
<td>15.1–20</td>
<td>50</td>
</tr>
<tr>
<td>20.1–25</td>
<td>35</td>
</tr>
<tr>
<td>25.1–30</td>
<td>20</td>
</tr>
</tbody>
</table>

(b) Central Interior (summer rainfall)

Six catchment areas, ranging from 39 km² to 33 278 km² situated in the C5 secondary drainage region (Midgley et al 1994), were selected as case study areas in this climatological region predominantly characterised by convective rainfall during the summer months. The mean annual precipitation (MAP) ranges from 428 mm to 654 mm (Lynch 2004). The topography is gentle, with elevations varying from 1 021 m to 2 120 m, and with average catchment slopes ranging between 1.7% and 10.3% (USGS 2002). A total of 450 observed flood events from 1931 to 2013 are included in the analysis.

(c) South Western Coastal region (winter rainfall)

Six catchment areas, ranging from 47 km² to 2 878 km² situated in the G1, H1, H4 and H6 secondary drainage regions (Midgley et al 1994), were selected as case study areas in this climatological region predominantly characterised by winter rainfall. The MAP ranges from 450 mm to 915 mm (Lynch 2004), and rainfall is classified as either orographic and/or frontal rainfall. The topography is very steep, with elevations varying from 86 m to 2 240 m, and with average catchment slopes ranging between 25.6% and 41.6% (USGS 2002). A total of 460 observed flood events from 1932 to 2013 are included in the analysis.

### Table 1 Overland flow distances associated with different slope classes (Matthee et al 1986)

<table>
<thead>
<tr>
<th>Slope class ($S_D$) (%)</th>
<th>Distance ($L_D$) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0–3</td>
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<tr>
<td>3.1–5</td>
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<tr>
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<tr>
<td>10.1–15</td>
<td>65</td>
</tr>
<tr>
<td>15.1–20</td>
<td>50</td>
</tr>
<tr>
<td>20.1–25</td>
<td>35</td>
</tr>
<tr>
<td>25.1–30</td>
<td>20</td>
</tr>
</tbody>
</table>
The locations of the case study areas as listed in (b) and (c) are shown in Figure 1. Table 2 contains a summary of the main morphometric properties for each catchment under consideration.

The influences of each variable or parameter listed in Table 2 are highlighted where applicable in the subsequent sections. The next section includes the detailed methodology followed during this study, focusing on the indirect estimation (empirical equations) and direct estimation (from observed streamflow data) of $T_C$.

**METHODOLOGY: TIME OF CONCENTRATION ESTIMATION PROCEDURES**

In order to evaluate and compare the consistency of a selection of time parameter estimation methods in case study areas (a) to (c), the following steps were followed:

(i) application and comparison of six overland flow $T_C$ equations to the Kerby equation (Equation 2) in different slope-distance classes and roughness parameter categories;

(ii) direct estimation of $T_C$ from observed streamflow data based on the $T_C \approx T_P$ approach; and

(iii) application of six channel flow $T_C$ equations in 12 medium to large catchments in order to compare their results with the results as obtained in (ii).

### Table 2 Main morphometric properties of catchments in the Central Interior and South Western Coastal region

#### Central Interior (summer rainfall)

<table>
<thead>
<tr>
<th>Catchment descriptor</th>
<th>CSH008</th>
<th>CSH012</th>
<th>CSH015</th>
<th>CSH016</th>
<th>CSH022</th>
<th>CSH035</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area (A) (km²)</td>
<td>598</td>
<td>2 366</td>
<td>5 939</td>
<td>33 278</td>
<td>39</td>
<td>17 359</td>
</tr>
<tr>
<td>Minimum elevation (m)</td>
<td>1.397</td>
<td>1.322</td>
<td>1.254</td>
<td>1.021</td>
<td>1.531</td>
<td>1.104</td>
</tr>
<tr>
<td>Maximum elevation (m)</td>
<td>1.740</td>
<td>1.780</td>
<td>2.120</td>
<td>2.120</td>
<td>2.060</td>
<td>2.120</td>
</tr>
<tr>
<td>Average catchment slope ($S$) (m/m)</td>
<td>0.0483</td>
<td>0.0328</td>
<td>0.0277</td>
<td>0.0209</td>
<td>0.1029</td>
<td>0.0173</td>
</tr>
<tr>
<td>Hydraulic length ($L_H$) (km)</td>
<td>41.0</td>
<td>86.9</td>
<td>160.5</td>
<td>378.1</td>
<td>8.0</td>
<td>373.3</td>
</tr>
<tr>
<td>Centroid distance ($L_C$) (km)</td>
<td>22.4</td>
<td>45.3</td>
<td>81.0</td>
<td>230.2</td>
<td>2.7</td>
<td>172.7</td>
</tr>
<tr>
<td>Main river / watercourse length ($L_{CH}$) (km)</td>
<td>40.9</td>
<td>86.7</td>
<td>160.2</td>
<td>377.9</td>
<td>7.9</td>
<td>373.0</td>
</tr>
<tr>
<td>Average main river slope ($S_{CH}$) (m/m)</td>
<td>0.0049</td>
<td>0.0027</td>
<td>0.0014</td>
<td>0.0010</td>
<td>0.0170</td>
<td>0.0008</td>
</tr>
</tbody>
</table>

#### South Western Coastal region (winter rainfall)

<table>
<thead>
<tr>
<th>Catchment descriptor</th>
<th>G1H003</th>
<th>G1H007</th>
<th>H1H007</th>
<th>H1H018</th>
<th>H4H006</th>
<th>H6H003</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area (A) (km²)</td>
<td>47</td>
<td>724</td>
<td>80</td>
<td>109</td>
<td>2 878</td>
<td>500</td>
</tr>
<tr>
<td>Minimum elevation (m)</td>
<td>199</td>
<td>86</td>
<td>273</td>
<td>375</td>
<td>185</td>
<td>297</td>
</tr>
<tr>
<td>Maximum elevation (m)</td>
<td>1.400</td>
<td>1.780</td>
<td>1.700</td>
<td>1.960</td>
<td>2.240</td>
<td>1.660</td>
</tr>
<tr>
<td>Average catchment slope ($S$) (m/m)</td>
<td>0.2889</td>
<td>0.2621</td>
<td>0.4069</td>
<td>0.4161</td>
<td>0.2921</td>
<td>0.2556</td>
</tr>
<tr>
<td>Hydraulic length ($L_H$) (km)</td>
<td>9.7</td>
<td>55.3</td>
<td>19.0</td>
<td>22.8</td>
<td>109.9</td>
<td>38.6</td>
</tr>
<tr>
<td>Centroid distance ($L_C$) (km)</td>
<td>5.0</td>
<td>29.0</td>
<td>9.5</td>
<td>9.3</td>
<td>26.9</td>
<td>13.6</td>
</tr>
<tr>
<td>Main river / watercourse length ($L_{CH}$) (km)</td>
<td>9.2</td>
<td>55.3</td>
<td>18.9</td>
<td>22.8</td>
<td>101.5</td>
<td>38.2</td>
</tr>
<tr>
<td>Average main river slope ($S_{CH}$) (m/m)</td>
<td>0.0177</td>
<td>0.0046</td>
<td>0.0333</td>
<td>0.0320</td>
<td>0.0047</td>
<td>0.0098</td>
</tr>
</tbody>
</table>

---

Figure 1 Location of case study areas (b) and (c)
The details of the empirical equations as used in (i) and (iii) are listed and discussed first, followed by a description of the procedures followed in (ii).

**Indirect estimation using empirical equations**

The empirical equations selected require a limited amount of information and similar input variables to estimate \( T_C \) in ungauged catchments, as proposed by Williams (1922), Kirpich (1940), Johnstone and Cross (1949), Miller (1951), Kerby (1959), Reich (1962), Espey and Winslow (1968), FAA (1970), USBR (1973), Sheridan (1994), and (Sabol 2008). The empirical equations are detailed in the next two sub-sections for overland flow and channel flow regimes. All the equations are presented in Système International d’Unités (SI Units).

**Overland flow regime**

The empirical overland flow \( T_C \) equations are applied within the ‘conceptual urban catchment’ (Case study (a)) by considering the seven different NSCM slope-distance classes and five categories with associated flow conveyance (\( \phi \)), retardant (imperviousness \( i_p \)), Manning’s roughness \( n \) and runoff curve number (CN) variables. The five different \( \phi \) categories are based on the work done by Viessman and Lewis (1996), with typical \( \phi \) values ranging from 0.6 (\( i_p = 80\% \); \( n = 0.02 \); CN = 95); 0.8 (\( i_p = 50\% \); \( n = 0.06 \); CN = 85); 1.0 (\( i_p = 30\% \); \( n = 0.09 \); CN = 75); 1.2 (\( i_p = 20\% \); \( n = 0.13 \); CN = 72) to 1.3 (\( i_p = 10\% \); \( n = 0.15 \); CN = 70).

The six overland flow \( T_C \) equations are summarised in Equations 1 to 6.


\[
T_{C1} = 107 \left[ \frac{nL_o^{0.333}}{100S_o^{0.2}} \right] \tag{1}
\]

where

- \( T_{C1} \) = overland time of concentration (minutes),
- \( L_o \) = length of overland flow path (m),
- \( n \) = Manning’s roughness parameter for overland flow, and
- \( S_o \) = average overland slope (m/m).

b. Kerby (1959): Equation 2 is commonly used to estimate the \( T_C \) both as mixed-sheet and/or shallow-concentrated overland flow in the upper reaches of small, flat catchments. The Drainage Manual (SANRAL 2013) also recommends the use thereof in SA. McCuen et al (1984) highlighted that Equation 2 was developed and calibrated for catchments in the United States of America (USA) for areas less than 4 ha, with average slopes of less than 1% and Manning’s roughness parameters \( n \) varying between 0.02 and 0.8.

\[
T_{C2} = 1.4394 \left( \frac{nL_o^{0.467}}{S_o^{0.333}} \right) \tag{2}
\]

where

- \( T_{C2} \) = overland time of concentration (minutes),
- \( L_o \) = length of overland flow path (m),
- \( n \) = Manning’s roughness parameter for overland flow, and
- \( S_o \) = average overland slope (m/m).

c. SCS (1962): Equation 3 is commonly used to estimate the \( T_C \) as mixed-sheet and/or concentrated overland flow in the upper reaches of a catchment. The USDA SCS developed this equation in 1962 (Reich 1962) for homogeneous, agricultural catchment areas up to 8 km² with mixed overland flow conditions dominating (USDA SCS 1985).

\[
T_{C3} = L_o^{0.8} \left[ \frac{2.5400 - 228.6}{706.9S_o^{0.5}} \right]^{0.7} \tag{3}
\]

where

- \( T_{C3} \) = overland time of concentration (minutes),
- \( L_o \) = runoff curve number,
- \( S_o \) = length of overland flow path (m), and
- \( S_o \) = average overland slope (m/m).

d. Espey-Winslow (1968): Equation 4 was developed using data from 17 catchments in Houston, USA, with areas ranging from 2.6 km² to 90.7 km². The imperviousness factor \( i_p \) represents overland flow retardant, while the conveyance factor \( \phi \) measures subjectively the hydraulic efficiency of a flow path, taking both the condition of the surface cover and degree of development into consideration (Espey & Winslow 1968).

\[
T_{C4} = 44.1 \left( \frac{\phi L_o^{0.29}}{S_o^{0.145} i_p^{0.6}} \right) \tag{4}
\]

where

- \( T_{C4} \) = overland time of concentration (minutes),
- \( i_p \) = imperviousness factor (%),
- \( \phi \) = conveyance factor,
- \( L_o \) = length of overland flow path (m), and
- \( S_o \) = average overland slope (m/m).

e. Federal Aviation Agency (FAA 1970): Equation 5 is commonly used in urban overland flow estimations, since the Rational method’s runoff coefficient \( C \) is included (FAA 1970; McCuen et al 1984).

\[
T_{C5} = \frac{1.8(1.344 - C)L_o^{0.5}}{100S_o^{0.333}} \tag{5}
\]

where

- \( T_{C5} \) = overland time of concentration (minutes),
- \( C \) = Rational method runoff coefficient (= default \( i_p \) fraction values),
- \( L_o \) = length of overland flow path (m), and
- \( S_o \) = average overland slope (m/m).

f. NRCS kinematic wave (1986): Equation 6 was originally developed by Welle and Woodward (1986) to avoid the iterative use of the original kinematic wave equation (Morgali & Linsley 1965) and is based on a power–law relationship between design rainfall intensity and duration.

\[
T_{C6} = 5.476 \left( \frac{nL_o^{0.8}}{P_2^{0.5}} \right)^{0.8} \tag{6}
\]

where

- \( T_{C6} \) = overland time of concentration (minutes),
- \( L_o \) = length of overland flow path (m),
- \( n \) = Manning’s roughness parameter for overland flow,
- \( P_2 \) = two-year return period 24-hour design rainfall depth (mm, default = 100), and
- \( S_o \) = average overland slope (m/m).

**Channel flow regime**

In the medium to large catchments located in case study areas (b) and (c), channel flow in the main watercourses is assumed to dominate. Consequently, a selection of six channel flow \( T_C \) equations with similar input variables are applied and compared to the direct \( T_C \) estimation results (referred to as \( T_{C8} \) in this paper) obtained from observed streamflow data using the assumption of the conceptual \( T_C = T_p \).

The six channel flow \( T_C \) equations are summarised in Equations 7 to 12.

g. Bransby-Williams (1922): The use of Equation 7 (Williams 1922) is limited to rural catchment areas less than ± 130 km² (Fang et al 2005; Li & Chibber 2008).

The Australian Department of Natural
Resources and Water (ADNRW 2007) highlighted that the initial overland flow travel time is already incorporated, therefore an overland flow or standard inlet time should not be added.

\[
T_{C_Y} = 0.2426 \left( \frac{L_{CH}}{A^{0.15}S_{CH}^{0.2}} \right) \quad (7)
\]

where

- \( T_{C_Y} \) = channel flow time of concentration (hours),
- \( A \) = catchment area (km\(^2\)),
- \( L_{CH} \) = length of longest watercourse (km), and
- \( S_{CH} \) = average main watercourse slope (m/m, using the 10-85 method).

h. Kirpich (1940): Equation 8 was calibrated in small, agricultural catchments (< 45 ha) located in the USA with average catchment slopes ranging between 3% and 10%. McCuen et al. (1984) showed that Equation 8 had a tendency to underestimate \( T_C \) values in 75% of urbanised catchments with areas smaller than 8 km\(^2\), while in 25% of the catchments (8 km\(^2\) < \( A \leq 16 \) km\(^2\)) with substantial channel flow, it had the smallest bias when compared to the observed \( T_C \) values.

\[
T_{CH} = 0.0663 \left( \frac{L_{CH}^2}{S_{CH}} \right)^{0.385} \quad (8)
\]

where

- \( T_{CH} \) = channel flow time of concentration (hours),
- \( L_{CH} \) = length of longest watercourse (km), and
- \( S_{CH} \) = average main watercourse slope (m/m, using the 10-85 method).

i. Johnstone-Cross (1949): Equation 9 was developed to estimate \( T_C \) in the Scioto and Sandusky River catchments (Ohio Basin) with areas ranging from 65 km\(^2\) to 4 206 km\(^2\) (Johnstone & Cross 1949; Fang et al. 2008).

\[
T_{CH} = 0.0543 \left( \frac{L_{CH}^0.5}{S_{CH}} \right) \quad (9)
\]

where

- \( T_{CH} \) = channel flow time of concentration (hours),
- \( L_{CH} \) = length of longest watercourse (km), and
- \( S_{CH} \) = average main watercourse slope (m/m, using the 10-85 method).

j. USBR (1973): Equation 10 was proposed by the USBR (1973) to be used as a standard empirical equation to estimate the \( T_C \) in hydrological designs, especially culvert designs based on the California Culvert Practice (CCP 1955, cited by Li & Chipber 2008). However, in essence it is a modified version of Equation 8 as proposed by Kirpich (1940) and is recommended by SANRAL (2013) for general use in SA.

\[
T_{C_10} = \left( \frac{0.87 \cdot L_{CH}^2}{1000 \cdot S_{CH}} \right)^{0.385} \quad (10)
\]

where

- \( T_{C_10} \) = channel flow time of concentration (hours),
- \( L_{CH} \) = length of longest watercourse (km), and
- \( S_{CH} \) = average main watercourse slope (m/m, using the 10-85 method).

k. Sheridan (1994): Equation 11 was developed to estimate the \( T_C \) using data from nine catchments in Georgia and Florida, USA, with catchment areas ranging between 2.6 km\(^2\) and 334.4 km\(^2\) (Sheridan 1994; USDA NRCS 2010).

\[
T_{CH1} = 2.2L_{CH}^{0.92} \quad (11)
\]

where

- \( T_{CH1} \) = channel flow time of concentration (hours), and
- \( L_{CH} \) = length of longest watercourse (km).

l. Colorado-Sabol (2008): Sabol (2008) proposed three different empirical \( T_C \) equations to be used in catchments with distinctive geomorphological and land-use characteristics in the State of Colorado, USA. Equation 12 is the equation applicable to rural catchments.

\[
T_{CH12} = 0.9293 \left[ \frac{A^{0.16} L_{CH}^{0.35}}{S_{CH}^{0.2}} \right] \quad (12)
\]

where

- \( T_{CH12} \) = channel flow time of concentration (hours),
- \( A \) = catchment area (km\(^2\)),
- \( L_C \) = centroid distance (km),
- \( L_{CH} \) = length of longest watercourse (km), and
- \( S_{CH} \) = average main watercourse slope (m/m, using the 10-85 method).

The direct estimation of \( T_{Ca} \) from observed streamflow data is discussed in the next section.

**Direct estimation from observed streamflow data**

The procedure as proposed by Gericke and Smithers (2014) and implemented by them (Gericke & Smithers 2015) is used to estimate \( T_{Ca} \) directly from observed streamflow data. In summary, the following steps were followed and also implemented in this study:

**Establishment of flood database**

Department of Water and Sanitation (DWS) primary flow data consisting of an up-to-date sample (DWS 2013) of the 12 continuous flow-gauging stations located at the outlet of each catchment in the Central Interior and South Western Coastal region was prepared and evaluated using the screening process as proposed by Gericke and Smithers (2015). The screening process accounts for:

(i) streamflow record lengths (> 30 years),
(ii) representative catchment area ranges (30 < \( A \) ≤ 35 000 km\(^2\)), and (iii) representative rating tables, i.e. extrapolation of rating tables was limited to 20% in cases where the observed river stage exceeded the maximum rated levels (\( H \)). Gericke and Smithers (2015) used third-order polynomial regression analyses to extrapolate the rating tables.

Hydrograph shape (especially the peakedness as a result of a steep rising limb, in relation to the hydrograph base length) and the relationship between observed peak discharge (\( Q_{Pd} \)) and direct runoff volume (\( Q_{Dd} \)) pair values were used as additional criteria to justify the individual stage extrapolations (\( H_{Ex} \)) up to a 20% limit, i.e. \( H_{Ex} < 1.2 \cdot H \). Typically, in such an event, the increase in \( Q_{Dd} \) due to the extrapolation was limited to 5%, hence the error made by using larger direct runoff volumes had little impact on the sample statistics of the total flood volume. This approach was justified in having samples of reasonable size (a total of 1 134 flood hydrographs in the C5 secondary drainage region), while the primary focus was on the time when the peak discharge occurs, not necessarily just the magnitude thereof. It is also important to note that Görgens (2007) also used a 20% stage limit to extrapolate rating tables as used in the development of the Joint Peak-Volume (JPV) method.

**Extraction of flood hydrographs**

Complete flood hydrographs were extracted using selection criteria as proposed by Gericke and Smithers (2015), and are based on:

(i) the implementation of truncation levels (i.e. only flood events > smallest annual maximum flood event were extracted), and
(ii) the identification of mutual start/end times on both the flood hydrographs and baseflow curves, hence ensuring that when a hydrograph is separated into direct runoff and baseflow, the identified separation point represents the start of direct runoff which coincides with the onset of effective rainfall. The end of a flood event was
also determined using a recursive filtering method (Nathan & McMahon 1990).

**Analyses of flood hydrographs**

The direct runoff and baseflow were separated using the recursive digital filtering method (Equation 13) as initially proposed by Nathan and McMahon (1990) and adopted by Smakhtin and Watkins (1997) in a national-scale study in SA.

\[ Q_{Di} = \alpha Q_{Di-1} + \beta (1 + \alpha)(Q_{Ti} - Q_{Ti-1}) \]  

where

\( Q_{Di} \) = filtered direct runoff at time step \( i \),

which is subject to \( Q_{D} \geq 0 \) for time \( i \) (m\(^3\)/s),

\( \alpha, \beta \) = filter parameters, and

\( Q_{Ti} \) = total streamflow (i.e. direct runoff plus baseflow) at time step \( i \) (m\(^3\)/s).

The application of Equation 13 using a fixed \( \alpha \)-parameter of 0.999 (Smakhtin & Watkins 1997) and a fixed \( \beta \)-parameter of 0.5 (Hughes et al 2003) resulted in the estimation of the following hydrograph parameters: (i) start/end date/time of flood hydrograph, (ii) observed peak discharge (\( Q_{max} \), m\(^3\)/s), (iii) total volume of runoff (\( Q_{TR} \), m\(^3\)), (iv) volume of direct runoff (\( Q_{DR} \), m\(^3\)), (v) volume of baseflow (\( Q_{B} \), m\(^3\)), (vi) baseflow index (BFI, which equals the ratio of \( Q_{B}/Q_{TR} \)), (vii) depth of effective rainfall (\( P_{eff} \), mm, based on the assumption that the volume of direct runoff equals the volume of effective rainfall and that the total catchment area is contributing to runoff), and (viii) time to peak (\( T_{p} \), hours).

Lastly, the analysed flood hydrographs were subjected to a final filtering process (Gerick & Smithers 2015) to ensure that all the flood hydrographs are independent and that the conceptual \( T_{Cst} \) values are consistent, i.e. the likelihood of higher \( Q_{max} \) values to be associated with larger \( Q_{DR} \) and \( T_{Cst} \) values, while taking cognisance of their dependence on factors such as antecedent moisture conditions and non-uniformities in the temporal and spatial distribution of storm rainfall.

Furthermore, the use of ‘truncation levels’, i.e. when only flood events larger than the smallest annual maximum flood event on record are extracted, ensured that all minor events were excluded, while all the flood events retained were characterised as multiple events being selected in a specific hydrological year.

This approach resulted in a partial duration series (PDS) of independent flood peaks above a certain level. It is important to note that Gerick and Smithers (2014; 2015) defined the \( T_{Cst} \) values as shown in Equation 14.

\[ T_{Cst} = \sum_{j=1}^{N} t_j \]  

where

\( T_{Cst} \) = conceptual time of concentration which equals the observed \( T_{p} \) for each individual flood event (hours),

\( t_j \) = duration of the total net rise (excluding the in-between recession limbs) of a multiple-peaked hydrograph (hours), and

\( N \) = sample size.

The mean of the individual flood events in each catchment calculated using Equation 14 could be used as the actual catchment response time. However, Gerick and Smithers (2015) highlighted that the use of such averages could be misleading and might not be a good reflection of the actual response time. Therefore, by considering the high variability of catchment responses calculated for each event as evident in the results from this study, as well as taking cognisance of the procedure adopted by Gerick and Smithers (2015), the use of a ‘representative average value’ equal to the linear catchment response function of Equation 15 (Gerick & Smithers 2015) was used to confirm the

**Table 3 Consistency measures for the testing of overland flow \( T_{Cst} \) estimation equations compared to Equation 2 (Kerby 1959)**

<table>
<thead>
<tr>
<th>Equations</th>
<th>Mean estimated ( T_{Cst} ) (Eq 2) (min)</th>
<th>Mean estimated ( T_{Cst} ) (min)</th>
<th>Standard bias statistic (Eq 16) (%)</th>
<th>Mean error (min)</th>
<th>Maximum error (min)</th>
<th>Standard error (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Miller (Eq 1)</td>
<td>5.3</td>
<td>23.8</td>
<td>327.3</td>
<td>18.5</td>
<td>49.5</td>
<td>1.1</td>
</tr>
<tr>
<td>SCS (Eq 3)</td>
<td>5.3</td>
<td>3.4</td>
<td>-44.6</td>
<td>-1.9</td>
<td>-3.3</td>
<td>0.8</td>
</tr>
<tr>
<td>FAA (Eq 5)</td>
<td>5.3</td>
<td>6.6</td>
<td>20.3</td>
<td>1.3</td>
<td>4.2</td>
<td>0.4</td>
</tr>
<tr>
<td>NRCS (Eq 6)</td>
<td>5.3</td>
<td>6.0</td>
<td>-6.2</td>
<td>0.6</td>
<td>8.9</td>
<td>0.5</td>
</tr>
</tbody>
</table>

**Figure 2(a) Category 1: Variation of overland flow \( T_{Cst} \) estimates in different average overland slope classes**

![Figure 2(a) Category 1: Variation of overland flow \( T_{Cst} \) estimates in different average overland slope classes](image-url)
validity and representativeness of the mean of the values calculated from each event.

\[
TC_{\text{linear}} = \frac{1}{3600} \left[ \sum_{i=1}^{N} (QP_{xi} - QPx)(QD_{i} - QD) \right] / \left[ \sum_{i=1}^{N} (QP_{xi} - QPx)^{2} \right] \tag{15}
\]

where

\( TC_{\text{linear}} \) = conceptual \( TC \) assuming a linear catchment response (hours),

\( QD_{i} \) = volume of direct runoff for individual events (m\(^3\)),

\( QD \) = mean of \( QD_{i} \) (m\(^3\)),

\( QP_{xi} \) = observed peak discharge for individual events (m\(^3\)/s),

\( QPx \) = mean of \( QP_{xi} \) (m\(^3\)/s), and

\( N \) = sample size.

In each catchment, the results based on Equations 14 and 15 were compared to establish their degree of association. Despite the high degree of association evident, Equation 15 was regarded as the most consistent procedure to estimate the most representative catchment \( TCx \) values. The preferential use of Equation 15 is motivated by the fact that the hydrograph analysis tool (HAT) developed by Gericke and Smithers (2015) could not always, due to the nature of flood hydrographs, cater for the different variations in flood hydrographs, especially when Equation 14 was applied. Therefore, a measure of user intervention is sometimes required, and consequently it could be argued that some inherent inconsistencies could possibly have been introduced. Taking cognisance of the latter possibility, the use of Equation 15 is therefore regarded as being more objective and with consistent results.

A standardised bias statistic (Equation 16) (McCuen et al 1984) was used with the mean error (difference in the average of the observed and estimated values in different classes/categories/catchments) as a measure of actual bias and to ensure that the \( TC \) estimation results are not dominated by errors in the large \( TC \) values. The standard error of the estimate was also used to provide another measure of consistency.

\[
BS = 100 \left[ \frac{1}{z} \sum_{i=1}^{z} \frac{|TC_{yi} - TC_{xi}|}{TC_{xi}} \right] \tag{16}
\]

where

\( BS \) = standardised bias statistic (%),

\( TC_{xi} \) = observed time of concentration (minutes or hours),

\( TC_{yi} \) = estimated time of concentration (minutes or hours), and

\( z \) = number of slope-distance categories (overland flow regime) or sub-catchments (channel flow regime).
RESULTS AND DISCUSSION

The results from the application of the above methodology using different $T_C$ estimation procedures as applied in case study areas (a) to (c) are presented in this section. The station numbers of the DWS flow-gauging stations located at the outlet of each catchment are used as the catchment descriptors for easy reference in all the tables and figures.

Indirect $T_C$ estimation results (overland flow regime)

The results from the estimated overland flow $T_C$ for the seven different NSCM slope-distance classes and five categories are shown in Figures 2(a) to 2(e).

From the results contained in Figures 2(a) to 2(e), the five equations (Equations 1 and 3 to 6) used to estimate the overland flow $T_C$ in case study area (a), relative (not absolute) to the $T_C$ estimated using the Kerby equation (Equation 2), showed different biases when compared in each of the five different flow-retardant categories and associated slope-distance classes. As expected, all the $T_C$ estimates decreased with an increase in the average overland slope, while $T_C$ gradually increases with an increase in the surface roughness and permeability. The SCS equation (Equation 3) constantly underestimated $T_C$, while the Miller (Equation 1) and Espey-Winslow (Equation 4) equations overestimated $T_C$ in all cases when compared to the estimates based on the Kerby equation (Equation 2). The NRCS kinematic wave equation (Equation 6) underestimated $T_C$ in relation to the Kerby equation (Equation 2) in Category 1, while other $T_C$ underestimations were witnessed in Categories 2 ($D_o \geq 0.10 \ m/m$), 3 ($D_o \geq 0.15 \ m/m$), and 4 to 5 ($D_o \geq 0.20 \ m/m$). The poorest results in relation to the Kerby equation (Equation 2) were obtained using the Espey-Winslow equation (Equation 4) and could be ascribed to the use of default conveyance ($\phi$) factors which might not be representative, since this is the only equation using $\phi$ as a primary input parameter.

In considering the overall average consistency measures compared to the Kerby equation (Equation 2) as listed in Table 3, the NRCS kinematic wave equation (Equation 6) provided relatively the smallest bias (< 10%), with a mean error ≤ 1 minute. Both the standardised bias (469.2%) and mean error (26 minutes) of the Espey-Winslow equation (Equation 4) were large compared to the other equations. The SCS equation (Equation 3) resulted in the smallest maximum absolute error of 3.3 minutes, while the Espey-Winslow equation (Equation 4) had a maximum absolute...
error of 82 minutes. The standard deviation of the errors provides another measure of correlation, with standard errors < 1 minute (Equations 3, 5 and 6).

**Direct T_C estimation results**

Only 5.6% and 6.9% of the total number of flood hydrographs analysed in the Central Interior and South Western Coastal regions respectively were subjected to the extrapolation of stage values (H_E) above the maximum rated levels (H) within the range H_E ≤ 1.2 H and Q_D ≤ 5%. Thus, the error made by using larger direct runoff volumes had little impact on the sample statistics of the total flood volume, especially if the total sample size of the analysed flood hydrographs is taken into consideration. It is important to note, as highlighted before, that the primary focus is on the time when the peak discharge occurs, not necessarily just the magnitude thereof.

The averaged hydrograph parameters computed using Equation 13 with α = 0.995 and β = 0.5 applied to the extracted observed hydrograph data are listed in Table 4. Figures 3 (Central Interior) and 4 (South Western Coastal region) show the regional observed peak discharge (Q_Pxi) versus the conceptual T_C(xi) values for all the catchments under consideration.

### Table 4 Summary of average hydrograph parameters for different catchments in the Central Interior and South Western Coastal region

<table>
<thead>
<tr>
<th>Catchment descriptor</th>
<th>Data period</th>
<th>Number of events</th>
<th>Average catchment values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>QT (10^6 m^3)</td>
</tr>
<tr>
<td>Central Interior (summer rainfall)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C5H008</td>
<td>1931/04/01 to 1986/04/01</td>
<td>112</td>
<td>2.2</td>
</tr>
<tr>
<td>C5H012</td>
<td>1936/04/01 to 2013/02/13</td>
<td>68</td>
<td>3.3</td>
</tr>
<tr>
<td>C5H015</td>
<td>1949/01/01 to 1983/11/22</td>
<td>90</td>
<td>23.3</td>
</tr>
<tr>
<td>C5H016</td>
<td>1953/02/01 to 1999/03/10</td>
<td>40</td>
<td>31.0</td>
</tr>
<tr>
<td>C5H022</td>
<td>1980/10/14 to 2013/10/24</td>
<td>70</td>
<td>0.37</td>
</tr>
<tr>
<td>C5H035</td>
<td>1989/08/03 to 2013/07/23</td>
<td>70</td>
<td>19.4</td>
</tr>
<tr>
<td>South Western Coastal region (winter rainfall)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>G1H003</td>
<td>1949/03/21 to 2013/08/27</td>
<td>75</td>
<td>1.6</td>
</tr>
<tr>
<td>G1H007</td>
<td>1951/04/02 to 1977/05/31</td>
<td>75</td>
<td>50.4</td>
</tr>
<tr>
<td>H1H007</td>
<td>1950/04/10 to 2013/07/25</td>
<td>98</td>
<td>10.5</td>
</tr>
<tr>
<td>H1H018</td>
<td>1969/02/26 to 2013/07/26</td>
<td>80</td>
<td>15.0</td>
</tr>
<tr>
<td>H4H006</td>
<td>1950/04/19 to 1990/08/06</td>
<td>80</td>
<td>105.7</td>
</tr>
<tr>
<td>H6H003</td>
<td>1932/10/01 to 1974/11/11</td>
<td>52</td>
<td>16.9</td>
</tr>
</tbody>
</table>

Figure 3 Regional Q_Pxi versus conceptual T_C(xi) values (Central Interior)
The data scatter in these figures demonstrates the inherent variability of $Q_{Pxi}$ and $T_{Cxi}$ in medium to large catchments at a regional level. It is evident that the direct $T_{Cxi}$ estimations from the observed streamflow data (Equation 14) could vary significantly, with the largest $Q_{Pxi}$ and $T_{Cxi}$ values associated with the likelihood of the entire catchment receiving rainfall for the critical storm duration. Smaller $T_{Cxi}$ values could be expected when effective rainfall of high average intensity does not cover the entire catchment, especially when a storm is centred near the outlet of a catchment. The regional $T_{Cxi}$ values in Figure 3 show a stronger linear correlation ($r^2 = 0.70$) when compared to the regional $T_{Cxi}$ values ($r^2 = 0.40$) in Figure 4. The latter stronger linear correlation shown in Figure 3 confirms that more homogeneous catchment responses were obtained in the Central Interior than in the South Western Coastal region (Figure 4). However, in Figure 4, the regional $T_{Cxi}$ values consist of two ‘different populations’, i.e. the $T_{Cxi}$ in relation to $Q_{Pxi}$ and the catchment area varies from catchment to catchment. This could be ascribed to differences in their morphometric properties, as well as to the spatial location of these catchments in different secondary drainage regions. The catchment responses in the H1 secondary drainage region differ from those catchments situated in the G1, H4 and H6 secondary drainage regions, with the $Q_{Pxi}$ values generally larger for corresponding or shorter $T_{Cxi}$ values, while the catchment areas are also smaller. Apart from the smaller catchment areas, the average catchment slope ($S$) and average main river slope ($S_{CH}$) are also much steeper (see Table 2).

The linear regression plots of the paired $Q_{Pxi}$ and $Q_{Di}$ values applicable to the Central Interior and South Western Coastal regions are shown in Figures 5 and 6 respectively. At a regional level, the paired $Q_{Pxi}$ and $Q_{Di}$ values showed an acceptable degree of association with $r^2$ values between 0.4 and 0.7. The $r^2$ values deviated similarly or less from unity at a catchment level, and such deviations could be ascribed to non-linear changes in the rainfall pattern and catchment conditions (e.g. soil moisture status) between individual flood events in a particular catchment. Consequently, Gericke and Smithers (2015) proposed the use of correction factors to provide individual catchment responses associated with a specific flood event. However, in this study, Equation 15 is used to confirm the validity and representativeness of the sample means, using Equation 14, and thus the correction factors were not applied. The high degree of association ($r^2 > 0.99$) between Equations 14 and 15 demonstrates the inherent variability of $Q_{Pxi}$ and $T_{Cxi}$ in medium to large catchments at a regional level.
and 15 (see Table 4) also confirmed that the extracted flood events in each catchment reflect the actual catchment processes, and, despite the variability of individual catchment responses, does not result in large differences in average catchment values.

**Comparison of indirect and direct T_C estimation (channel flow regime)**

In Figures 7 and 8 box plots are used to highlight the inherent variability of the T_C values estimated directly from the observed streamflow data. In these figures, the whiskers represent the minimum and maximum values, the boxes the 25th and 75th percentile values, and the change in box colour represents the median value. The results of the six equations (Equations 7 to 12) used to estimate T_C under predominant channel flow conditions, are also super-imposed on Figures 7 and 8, while the goodness-of-fit (GOF) statistics for the test of these equations in the 12 catchments are listed in Tables 5 and 6 respectively.

In practical terms, the high T_C variability evident in these figures would not be easily incorporated into design hydrology. Consequently, a reasonable catchment T_C value for design purposes and for the calibration of empirical equations should be a convergence value based on the similarity of the results obtained when Equations 14 and 15 are used in combination. As mentioned before, the results based on Equations 14 and 15 were compared in each catchment to establish their degree of association, but the results based on Equation 15 were accepted as the most representative catchment T_C values (shown as red circle markers in Figures 7 and 8). Furthermore, it is clearly evident from Figures 7 and 8 that the high variability in T_C estimation is directly related and amplified by the catchment area, with variations up to ± 800% (see Tables 5 and 6, with the bias ranging between −86% and 729%). The Bransby-Williams (Equation 7) and Colorado-Sabol (Equation 12) equations are the only equations which include the catchment area as an independent variable; therefore it is not surprising that it demonstrated poorer results in the larger catchment area ranges (A > 5 000 km²) of the Central Interior as opposed to the medium catchment area ranges (50 < A ≤ 3 000 km²) of the South Western Coastal region. It could also be argued that the differences are because the Bransby-Williams equation (Equation 7) was derived from Australian rural catchments, which are decidedly different to South African catchments and with the catchment areas used in the calibration limited to ± 130 km². However, the Colorado-Sabol
Table 5 GOF statistics for the testing of channel flow $T_C$ estimation equations compared to the direct estimation of $T_C$ from observed streamflow data in the Central Interior

<table>
<thead>
<tr>
<th>Equations</th>
<th>Mean observed $T_C$ (hrs)</th>
<th>Mean estimated $T_C$ (hrs)</th>
<th>Standard bias statistic (Eq 16) (%)</th>
<th>Mean error (hrs)</th>
<th>Maximum error (hrs)</th>
<th>Standard error (hrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bransby-Williams (Eq 7)</td>
<td>26.7</td>
<td>63.4</td>
<td>107.0</td>
<td>36.7</td>
<td>101.1</td>
<td>10.6</td>
</tr>
<tr>
<td>Kirpich (Eq 8)</td>
<td>26.7</td>
<td>43.5</td>
<td>37.1</td>
<td>16.8</td>
<td>57.8</td>
<td>10.3</td>
</tr>
<tr>
<td>Johnstone-Cross (Eq 9)</td>
<td>26.7</td>
<td>17.4</td>
<td>–39.7</td>
<td>–9.3</td>
<td>–32.6</td>
<td>11.2</td>
</tr>
<tr>
<td>USBR (Eq 10)</td>
<td>26.7</td>
<td>43.5</td>
<td>37.2</td>
<td>16.9</td>
<td>57.9</td>
<td>10.3</td>
</tr>
<tr>
<td>Sheridan (Eq 11)</td>
<td>26.7</td>
<td>246.3</td>
<td>728.8</td>
<td>219.6</td>
<td>469.9</td>
<td>8.8</td>
</tr>
<tr>
<td>Colorado-Sabol (Eq 12)</td>
<td>26.7</td>
<td>86.2</td>
<td>205.9</td>
<td>59.5</td>
<td>122.7</td>
<td>7.7</td>
</tr>
</tbody>
</table>

Table 6 GOF statistics for the testing of channel flow $T_C$ estimation equations compared to the direct estimation of $T_C$ from observed streamflow data in the South Western Coastal region

<table>
<thead>
<tr>
<th>Equations</th>
<th>Mean observed $T_C$ (hrs)</th>
<th>Mean estimated $T_C$ (hrs)</th>
<th>Standard bias statistic (Eq 16) (%)</th>
<th>Mean error (hrs)</th>
<th>Maximum error (hrs)</th>
<th>Standard error (hrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bransby-Williams (Eq 7)</td>
<td>24.1</td>
<td>13.6</td>
<td>–46.1</td>
<td>–10.5</td>
<td>–19.5</td>
<td>6.2</td>
</tr>
<tr>
<td>Kirpich (Eq 8)</td>
<td>24.1</td>
<td>7.2</td>
<td>–73.4</td>
<td>–16.8</td>
<td>–26.4</td>
<td>6.1</td>
</tr>
<tr>
<td>Johnstone-Cross (Eq 9)</td>
<td>24.1</td>
<td>3.6</td>
<td>–86.0</td>
<td>–20.5</td>
<td>–36.8</td>
<td>5.0</td>
</tr>
<tr>
<td>USBR (Eq 10)</td>
<td>24.1</td>
<td>7.2</td>
<td>–73.4</td>
<td>–16.8</td>
<td>–26.4</td>
<td>6.1</td>
</tr>
<tr>
<td>Sheridan (Eq 11)</td>
<td>24.1</td>
<td>65.7</td>
<td>173.4</td>
<td>41.6</td>
<td>109.5</td>
<td>7.0</td>
</tr>
<tr>
<td>Colorado-Sabol (Eq 12)</td>
<td>24.1</td>
<td>21.2</td>
<td>–9.4</td>
<td>–2.8</td>
<td>–11.2</td>
<td>4.8</td>
</tr>
</tbody>
</table>

Figure 8 Box plots of $T_C$ values (Eq 14) and super-imposed data series values of the catchment $T_C$ (Eq 15) and empirical $T_C$ estimates for the six catchments of the South Western Coastal region.

In considering the overall average GOF statistics as listed in Tables 5 and 6, the six empirical equations showed different biases when compared to the ‘direct measurement’ of $T_C$. In the Central Interior (Table 5) only the Johnstone-Cross equation (Equation 9) underestimated the $T_C$ and it also showed a relatively low bias (~37.9%) and mean error (~9.3 hours). The Kirpich (Equation 8) and USBR (Equation 10) equations, with almost identical results, provided the smallest positive biases (~37.1% each), and associated positive mean errors of ~16.8 hours. The similarity of the latter results could be ascribed to the fact that Equation 10 (USBR, ‘recommended’ for use in SA) is essentially a modified version of the Kirpich equation (Equation 8). In contradiction to the Central Interior results, as contained in Table 5, the Bransby-Williams (Equation 7) and Colorado-Sabol (Equation 12) equations provide some of the best estimates in the South Western Coastal region (Table 6), with biases of ≤46.1% and associated mean errors of ≤10.5 hours. However, all the mean error results must be clearly understood in the context of the actual travel time associated with the size of a particular catchment, since in the latter region some of the catchments have average $T_C$ values ≤10 hours.

On average, all the other empirical equations, except the Johnstone-Cross equation (Equation 9), overestimated the $T_C$ in the Central Interior (Table 5) with maximum absolute errors up to 470 hours, while the opposite is evident from Table 6 (South Western Coastal).
Western Coastal region). In the latter region, $T_{Cw}$ was underestimated in all cases, except for Equation 11 (Sheridan). However, the poorest results in both the Central Interior and South Western Coastal regions are also demonstrated by Equation 11, with maximum absolute errors of between 110 hours and 470 hours. Typically, the large errors associated with the Sheridan equation (Equation 11) could be ascribed to the inclusion of only one independent variable (e.g. main watercourse length) to accurately reflect the catchment $T_{Cw}$.

The conclusions are summarised in the following section.

CONCLUSIONS

This paper demonstrates the estimation of $T_C$ using direct and indirect estimation procedures with observed streamflow data and empirical equations respectively. Empirical equations applicable to the overland flow regime were implemented on a conceptualised urban catchment, while both a direct estimation method and empirical equations applicable to channel flow were implemented on two other case study areas. The results clearly display the wide variability in $T_C$ estimates using different equations. In the estimation of overland flow, the variability and inconsistencies demonstrated are most likely due to the fact that the characteristics of the five different flow retardant categories and associated slope-distance classes considered are decidedly different from those initially used to derive and calibrate the relevant equations. In general, the variability and inconsistencies witnessed in the channel flow regime can be ascribed to the equations being applied outside the bounds of their original developmental regions without the use of local correction factors. However, the fact that either improved or poorer results were obtained with a specific empirical equation in either the Central Interior or South Western Coastal region, also confirms that the results obtained are not due to the use of inappropriate independent variables to estimate the catchment response time. The latter could rather be ascribed to the differences in catchment geomorphology. In addition, it could also be argued that the wide variability and inconsistencies are further exacerbated by the discrepancies in the $T_C$ definitions and estimation procedures found in the literature.

The direct estimation procedure considering both the use of an average catchment $T_{Cw}$ value based on the event means of Equation 14 and a linear catchment response function (Equation 15) proved to be an objective and consistent approach to estimate observed $T_{Cw}$ values by using only streamflow data. In using the latter direct estimation procedure, the validity of the approximation $T_C = T_p$ was also confirmed to be sufficiently similar at a medium to large catchment scale. In order to accommodate the high variability and uncertainty involved in the estimation of $T_C$, we recommend that for design hydrology and for the calibration of empirical equations, $T_{Cw}$ should be estimated using the proposed direct estimation procedure. Ultimately, these observed $T_{Cw}$ values can be used to develop and calibrate new, local empirical equations that meet the requirement of consistency and user-friendliness, i.e. including independent variables (e.g. $A$, $L_C$, $L_{CH}$ and $S_{CH}$) that are easy to determine by different practitioners when required for future applications in ungauged catchments. In order to overcome the limitations of an empirical equation calibrated and verified in a specific region, the proposed methodology should also be expanded to other regions, followed by regionalisation. The regionalisation will not only improve and augment the accuracy of the time parameter estimates, but will also warrant the combination and transfer of information within the identified homogeneous hydrological regions.

In conclusion, the results from this study indicate that estimates of catchment response time are inconsistent and vary widely as applied in modern flood hydrology practice in South Africa. Therefore, if practitioners continue to use these inappropriate time parameter estimation methods, this would limit possible improvements when both event-based design flood estimation methods and advanced stormwater models are used, despite the current availability of other technologically advanced input parameters in these methods/models. In addition, not only will the accuracy of the above methods/models be limited, but it will also have an indirect impact on hydraulic designs, i.e. underestimated $T_C$ values would result in over-designed hydraulic structures and the overestimation of $T_{Cw}$ would result in under-designs.

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IEA (Institution of Engineers Australia) 1977. Australian rainfall and runoff: Flood analysis and design, 2nd ed. Canberra, Australia: IEA.
A comparison of technical and practical aspects of Eurocode 3-1-1 and SANS 10162-1 hot-rolled steelwork design codes

R S Walls, C Viljoen

In South Africa engineers are starting to use the Eurocode guidelines for steelwork design, and it is important to understand the implications and differences in results that are obtained when applying the different codes. This paper presents a comparison between the Eurocode 1993-1-1:2005 and SANS 10162-1:2005 hot-rolled steelwork design codes. Numerical comparisons of predicted member design strengths for the important modes of failure and the complexity of calculations are presented, along with considerations regarding the parameters used in design. The following are explicitly shown for both codes: (a) differences in the classification of commonly used H, I, PFC and equal L sections, (b) differences in tension resistance calculations, (c) comparisons of all axial buckling curves, (d) calculations for a selection of members in flexural buckling which have different classifications, and (e) a summary of the shear resistances of commonly used H and I sections. It is shown that, on average, Eurocode 3 predicts higher member design strengths than the SANS 10162 code for most failure modes, primarily because of material partial safety factors closer to unity, less conservative buckling curves and the consideration of plastic resistance of sections. These EC3 design capacities can be higher by up to 11% for tension, 35% in compression, 31% in bending and 51% in shear, although there are cases where strengths of up to 33% lower were calculated, such as for an IPE AA*200 in shear. Results are influenced by design geometric tolerances, which are based on section classifications. The Eurocode’s equations and design methodologies are more complex and computationally demanding. Since South Africa has started moving in the direction of adapting or adopting Eurocodes with the SANS 10160 Loading Code (from EN 1) and SANS 10100 Structural Concrete Code (from EN 2), it should be considered whether or not the steelwork code should be adopted or adapted in a similar fashion in the future.

INTRODUCTION

Background to the codes

In South Africa hot-rolled steelwork is primarily designed using the SANS 10162-1:2005 code, The Structural Use of Steel – Part 1: Limit-state design of hot-rolled steelwork (SANS 2005), of which the first edition was published in 1993. The code is based on the Canadian steelwork design code, CSA S16, which has the same approach to design as that of the USA. Historically South Africa used to follow the British standards in terms of steelwork design, such as BS 5950 (BS 1995). However, recently the code for the design of cold-formed steelwork in this country, SANS 10162-2 (SANS 2011), has been updated, and is now based on the Australian and New Zealand Standard AS/NZ 4600:2005 (AS/NZS 2005). It can thus be seen that South Africa draws upon a diverse range of codes. Compiling a design code requires vast resources, and it has been more expedient to adopt or adapt the work of other countries.

The development of the Eurocodes was initiated in 1975, whereby “the objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications” for the European construction industry (Eurocode Foreword). Of the Eurocodes it is claimed: “Eurocodes are one of the most advanced suites of structural codes in the world. They embody the collective experience and knowledge of the whole of Europe … Eurocodes reflect the results of research in material technology and structural behaviour in the last fifty years and they incorporate all modern trends in structural design.” (Narayanan 2008)

Around 26 countries in Europe have adopted the EN suite of codes. Other countries, such as Singapore, are now considering adopting them as well (De Clercq 2012).
The Eurocodes are published by CEN (the French acronym for the European Committee for Standardisation), and the documents are accompanied by National Annexes containing Nationally Determined Parameters (NDPs). The NDPs allow for a certain level of local calibration in member states, as partial factors can be selected to account for factors such as local construction tolerances, steel quality, historical data and other such factors. In this paper the NDPs recommended by CEN have been selected, as they are most commonly used throughout the member states, although variations in selection are discussed below. The Eurocode suite of ten documents cover the basis for design, actions of structures, concrete, steel, timber, masonry, geotechnical design, earthquakes and aluminium structures. Hence, all aspects of structural design, such as analysis, loading, resistances and even construction requirements, are addressed within the codes.

Steelwork structures are covered within EN 1993 (or EN 3), which consists of twenty separate documents. The main sections to the EN 3 document are:

- EN 1993-1 Design of Steel Structures: General rules and rules for buildings
- EN 1993-2 Design of Steel Structures: Steel bridges
- EN 1993-3 Design of Steel Structures: Towers, masts and chimneys
- EN 1993-4 Design of Steel Structures: Silos, tanks and pipelines
- EN 1993-5 Design of Steel Structures: Piling
- EN 1993-6 Design of Steel Structures: Crane supporting structures

Within Part 1 of EN 3 there are the following twelve sections:

- EN 1993-1-1 Design of Steel Structures: General rules and rules for buildings
- EN 1993-1-2 Design of Steel Structures: Structural fire design
- EN 1993-1-3 Design of Steel Structures: Cold-formed thin gauge members and sheeting
- EN 1993-1-4 Design of Steel Structures: Stainless steels
- EN 1993-1-5 Design of Steel Structures: Plated structural elements
- EN 1993-1-6 Design of Steel Structures: Strength and stability of shell structures
- EN 1993-1-7 Design of Steel Structures: Strength and stability of planar plated structures transversely loaded
- EN 1993-1-8 Design of Steel Structures: Design of joints
- EN 1993-1-9 Design of Steel Structures: Fatigue strength of steel structures
- EN 1993-1-10 Design of Steel Structures: Selection of steel for fracture toughness and through-thickness properties

In this paper the resistance of sections is calculated based on using S355JR steelwork, having a yield stress of \( f_y = 355 \text{ MPa} \) and a Young’s Modulus of \( E = 200 \text{ GPa} \). In EN 3 the Young’s Modulus of steel is stated as being 210 GPa, which does provide a slightly higher resistance of members in buckling. However, the value of 200 GPa has been retained to match the SAISC Red Book (SAISC 2005) guidelines. It should also be noted that, for SANS 10162-1, \( f_y \) is reduced to 350 MPa for \( t_y > 16 \text{ mm} \). For EN 3 \( f_y \) is reduced to 335 MPa for \( t_y > 40 \text{ mm} \).

An important aspect which must be noted when comparing the SANS and EN codes is that the axes of members have different notations. For SANS the major axis of a cross-section is \( x-x \), and the minor axis is \( y-y \). However, for EN codes the major axis of a cross-section is \( y-y \), the minor axis is \( z-z \), with an axis along the length of a member being the \( x-x \) axis. This is shown in Figure 1. In this paper the axis notation of each code is retained when presenting design equations.

**Partial factors**

A very important difference between the SANS 10162-1 and EN 3 codes is the values of partial factors. If South Africa was to adopt the EN 3 code these factors could, and should, be adjusted to suit local conditions or (even to match existing partial factors), be based on local material and manufacturing quality.

For the purposes of this paper the partial factor values recommended in EN 3 will be used for calculations. Each country in Europe which has adopted the EN codes has issued National Annexes (NA) to allow for local calibration of codes, and thus many of these values differ from country to country. The National Annexes contain the Nationally Determined Parameters (NDPs) suitable for that region. A country should not start using the EN codes until the NDP values have been determined. It has been noted that engineers in South Africa have started using EN 3 without NDP values specific to this country. This paper will assist in...
identifying the impact associated with such a choice.

The partial factors recommended in SANS 10162-1 Section 13 are:

a. Structural steel: \( \gamma_{M_{0}} = 0.90 \)
b. Bolts: \( \gamma_{M_{b}} = 0.80 \)
c. Bearing of bolts on steel: \( \gamma_{M_{b}} = 0.67 \)
d. Weld metal: \( \gamma_{M_{w}} = 0.67 \).

Item (a) is the most important, relative to the results presented in this paper.

Rather than recommending partial factors according to the nature of the material or item, EN 3 recommends factors according to the nature of the design and failure mechanism:

a. Resistance of cross-sections whatever the class: \( \gamma_{M_{b}} = 1.00 \)
b. Resistance of members to instability assessed by member checks: \( \gamma_{M_{1}} = 1.00 \)
c. Resistance of cross-sections in tension to fracture: \( \gamma_{M_{2}} = 1.25 \)

Eurocode resistances are divided by partial factors, whereas SANS resistances are multiplied by them. Hence, \( 0 \leq \gamma_{M_{b}} \leq 1.0 \), whereas \( \gamma_{M_{b}} \geq 1.00 \). Table 1 shows a summarised comparison of these factors.

From the values listed above it can be seen that in general SANS uses a design value of 90% of characteristic material strength, whereas EN 3 accepts a higher design value at 100% of the characteristic material strength. This immediately causes the EN 3 design calculations to predict higher resistances for members, except in the case of tension fracture failures (but other factors must be considered for this mode of failure, as will be discussed further on under the heading “Cross-sectional classification”). It should be noted that, at the stage when Eurocode 3 was a voluntary design guideline called ENV 3, the material factor \( \gamma_{M_{b}} \) was suggested as 1.1 (Chabrolin 2001). Thus, it can be seen that there have been discussions and changes for members, except in the case of tension fracture failures (but other factors must be considered for this mode of failure, as will be discussed further on under the heading “Cross-sectional classification”). It should be noted that, at the stage when Eurocode 3 was a voluntary design guideline called ENV 3, the material factor \( \gamma_{M_{b}} \) was suggested as 1.1 (Chabrolin 2001). Thus, it can be seen that there have been discussions and changes in the material factors utilised. The yield strength of steel typically follows a normal distribution, with the average strength being two standard deviations above the characteristic strength (ICSS 2001). The standard deviation is generally 30 MPa. The increased design strength used in EN 3 may indicate a greater confidence in the quality control and use of steelwork in the European Union.

As a broad overview of the partial factors selected by various countries for their National Annexes, the countries of Bulgaria, Denmark, Finland, Norway, Slovenia, Sweden and the UK are considered here (CSI 2010). Of the partial factors recommended in Table 1 the only differences in these countries are that \( \gamma_{M_{b}} \) has a value of 1.05 in Bulgaria and Sweden, and 1.10 in Denmark. In Bulgaria and Sweden \( \gamma_{M_{b}} \) has its value set at 1.05, and in Denmark at 1.20. In the United Kingdom \( \gamma_{M_{b}} \) has a value of 1.10, 1.35 in Denmark and 0.9fu/\( f_y \) (but \( \leq 1.1 \)) in Sweden. Due to the large number of countries in which the EN codes have been adopted, not all of these can be considered in this paper. However, it can be seen that there is a certain degree of variation across Europe.

Extensive research programmes have been carried out in Europe to verify the partial factors selected for the EN 3 code. A programme headed by Chabrolin (2001) conducted tests at steel mills in France, Spain, the United Kingdom, Luxembourg, Germany, Italy and the Netherlands. Nine hundred samples, consisting of HE, IPE, UB and UC sections of grades 275 to 460 steel, were measured at the mills. Based on this research it was concluded that a value of \( \gamma_{M_{b}} = 1.00 \) was acceptable. However, it is a concern that, even if a section is within specification at a mill, it would still have to go through workshop fabrication, handling and erection, which may cause additional imperfections and residual stresses from welding.

### Reliability and Loading Codes

The target reliability index of steel buildings is stated as \( \beta_{T} = 3.0 \) for CSA S16, as noted in Appendix B of the document, with connectors having a higher level of reliability. In EN 1990 “Eurocode – Basis for Structural Design” the target reliability index of structural members at the ultimate limit state is set at \( \beta_{T} = 3.8 \). Based on this, one would see a difference in the values provided by the different national codes.
expect that the Eurocode would predict lower member strengths (more conservative) than the Canadian code. However, in this paper it can be seen that this is typically not the case.

The South African loading code, SANS 10160, is consistent with the Eurocode loading code, with the same basis of design. However, these two codes have been calibrated to different reliability levels, with SANS 10160 having a reliability index ($\beta_T$) of 3.0, and the Eurocodes having a reliability index of 3.8 (Retief et al. 2009). These values correspond to probabilities of loads being exceeded by 0.135% and 0.00723% respectively. This implies that the European loading code will estimate higher loads. However, in the overall development of the latest code systems the loading codes have been effectively decoupled from the material codes, making it theoretically possible to use loading and material codes from different countries. In particular, the latest revision of SANS 10160 (2011) implemented a basis of design similar to that of Eurocodes, thus specifically allowing the use of our loading code with EN material standards (Retief et al. 2009). The de-coupling also allows separate calibration to achieve adequate reliability of load effect in the loading code, and of resistance in the material standard, respectively. In this paper it is assumed that the steel codes would be used with the same loading code, as would be the case in South Africa where SANS 10160 would be used for both cases. Should EN 3 be adopted, it would be necessary to ensure that required resistance reliability levels are achieved through adjusting NDPs.

**Cross-sectional classification**

Before the strength of a section can be determined the section must be classified, based on the width to thickness ratio of components. SANS 10162-1 and EN 3 classify sections in an almost identical manner:

- **Class 1**: Cross-sections which can form a plastic hinge and allow a redistribution of moments.
- **Class 2**: Cross-sections which can develop a plastic moment of resistance, but because of local buckling, have limited rotation capacity.
- **Class 3**: Cross-sections which can obtain an elastic moment of resistance, but not a plastic moment of resistance.
- **Class 4**: Cross-sections in which local buckling will occur before yield stresses are reached.

The SANS 10162 and EN 3 codes classify sections into the aforementioned classes according to Table 2, with Figure 2 as a reference. Note that the symbols shown have been slightly modified, relative to those listed in the code to avoid any confusion in referencing.

Using the methods of classification listed above, a comparison has been done of H, I, PFC and equal L sections presented in the SAISC Red Book (SAISC 2005). The members listed in Table 3 show where there are differences between these codes in classification. All members not listed have the same classification in both codes.

Members in Class 1, 2 and 3 have the same procedure in both codes for the calculation of compressive strength. However, when there is a Class 4 section in compression, or a Class 3 or 4 member in flexure, then the method of design differs. Thus, of primary interest, of those members listed in Table 3, are the UC 152 × 152 × 23 and UC 203 × 203 × 46 in flexure, all the members listed under the compression section. The EN 3 estimate of resistance of the UC 203 × 203 × 46 in flexure is increased by virtue of the fact that it is allowed to develop a plastic moment of resistance rather than an elastic moment.

**Members in tension**

The SANS 10162-1 code calculates the tensile resistance of a member as the lowest of the following values:

i. $T_u = \varnothing \cdot A_g \cdot f_y$  \hspace{1cm} (1)

ii. $T_u = 0.85 \varnothing \cdot A_{nc} \cdot f_y$  \hspace{1cm} (2)

iii. $T_u = 0.85 \varnothing \cdot A'_{nc} \cdot f_y$  \hspace{1cm} (3)

![Figure 2 Definition of symbols for classification of sections](image-url)
The EN 3 code determines tensile resistance in a similar way, with the tension capacity being the smaller of:

\[ N_{p,Rd} = \frac{A_{fy}}{Y_{M0}} \]  

- plastic resistance of gross cross-section


\[ N_{u,Rd} = \frac{0.9A_{net,fu}}{Y_{M2}} \]  

- ultimate resistance of the net cross-section

Clauses are provided for the determination of shear lag effects.

From the above code it can be seen that the SANS code assumes that 90% (Ø) of the gross cross-sectional area reaches the characteristic yield stress, whereas the EN 3 code utilises 100% (γM). The ultimate resistance of the net cross-section is calculated as being 76.5% (0.85Ø) and 72.0% (0.9γM) respectively. Thus, in the first instance the EN 3 allows an 11.1% higher design resistance, whereas in the second case the design resistance is 5.9% lower.

SANS 10162-1 sets the maximum slenderness limits (L/r) as being 300 for tension members and 200 for compression members. Within EN 3 slenderness limits are not explicitly stated, and theoretically members of an infinite slenderness are allowed. Of course, design against buckling modes of failure will prevent this in practice.

### Members in axial compression

The basic calculations required to determine the compressive resistance of a member are discussed below. Both codes calculate the design capacity based on the resistance of a section at yield stress reduced by material factors and a reduction in capacity due to buckling. The EN 3 code states this more explicitly with the use of Φ reduction factors. The SANS 101621 code has only one buckling curve, for the compression and flexural resistance. Members in axial compression are assigned buckling curves, for Classes 1 to 3 members.

The SANS 10162-1 equations for the resistance of a member in compression, C_r, with buckling about any axis are:

\[ C_r = \frac{\Phi A_{fy}}{\gamma M_1} \]  

where:

\[ \Phi = \frac{1}{\sqrt{\lambda^2 + \frac{1}{\gamma M_1}}} \]  

where:

\[ \lambda = \frac{K L}{r \sqrt{\pi^2 E}} \]  

\[ n = 1.34 \]  

(8)

Both the SANS and EN codes reduce the effective area of a Class 4 member in compression. The EN 3 code calculates the compression resistance of a member subject to buckling, \( N_{b,Rd} \), using the following equations:

\[ N_{b,Rd} = \frac{\chi A_{fy}}{Y_{M1}} \]  

where:

\[ \chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} \]  

(10)

\[ \Phi = 0.5[1 + \alpha(\bar{T} - 0.2) + \bar{X}^2] \]  

(11)

\[ \bar{T} = \frac{A_{fy}}{N_{cr}} \]  

\[ \bar{X} = \frac{L_{cr}}{i \lambda_i} \]  

\[ \lambda_i = \pi \sqrt{\frac{f_y}{E}} \]  

Table 3 Differing cross-section classifications between SANS 10162-1 and EN 3-1-1

<table>
<thead>
<tr>
<th>Member</th>
<th>SANS Class</th>
<th>EN 3 Class</th>
<th>Member</th>
<th>SANS Class</th>
<th>EN 3 Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flange of member in flexure</td>
<td></td>
<td></td>
<td>Flange of member in compression</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UB 203 x 133 x 25</td>
<td>2</td>
<td>1</td>
<td>UC 152 x 152 x 23</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>UB 254 x 146 x 31</td>
<td>2</td>
<td>1</td>
<td>L 150 x 50 x 5</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>UB 305 x 165 x 41</td>
<td>2</td>
<td>1</td>
<td>L 60 x 60 x 6</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>UB 406 x 140 x 39</td>
<td>2</td>
<td>1</td>
<td>L 80 x 80 x 8</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>UB 406 x 178 x 54</td>
<td>2</td>
<td>1</td>
<td>L 100 x 100 x 10</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>UB 533 x 210 x 82</td>
<td>2</td>
<td>1</td>
<td>L 120 x 120 x 12</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>UC 152 x 152 x 23</td>
<td>4</td>
<td>3</td>
<td>L 150 x 150 x 15</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>UC 152 x 152 x 30</td>
<td>2</td>
<td>1</td>
<td>L 200 x 200 x 20</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>UC 203 x 203 x 46</td>
<td>3</td>
<td>2</td>
<td>Web of member in compression</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UC 203 x 203 x 52</td>
<td>2</td>
<td>1</td>
<td>IPE-AA 160</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>UC 305 x 305 x 118</td>
<td>2</td>
<td>1</td>
<td>IPE-AA 180</td>
<td>4</td>
<td>3</td>
</tr>
</tbody>
</table>

Table 4 Selection of buckling curve for cross-sections of compression members to EN 3

<table>
<thead>
<tr>
<th>Type of section</th>
<th>Limits</th>
<th>Buckling about axis</th>
<th>Buckling curve</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\Phi S235, S275, S355, S420$</td>
</tr>
<tr>
<td>Rolled I &amp; H sections</td>
<td>( h &gt; \frac{b}{2} )</td>
<td>( t_f \leq 40 \text{ mm} )</td>
<td>( y \cdot y )</td>
</tr>
<tr>
<td></td>
<td>( h \leq \frac{b}{2} )</td>
<td>( 40 &lt; t_f \leq 100 \text{ mm} )</td>
<td>( y \cdot y )</td>
</tr>
<tr>
<td></td>
<td>( h &gt; \frac{b}{2} )</td>
<td>( t_f \leq 100 \text{ mm} )</td>
<td>( y \cdot y )</td>
</tr>
<tr>
<td></td>
<td>( h &gt; \frac{b}{2} )</td>
<td>( t_f &gt; 100 \text{ mm} )</td>
<td>( y \cdot y )</td>
</tr>
<tr>
<td>Hot-finished Hollow sections</td>
<td>any</td>
<td>any</td>
<td>( a_0 )</td>
</tr>
<tr>
<td>Cold-finished Hollow sections</td>
<td>any</td>
<td>any</td>
<td>( c )</td>
</tr>
<tr>
<td>U, T &amp; solid sections</td>
<td>any</td>
<td>any</td>
<td>( b )</td>
</tr>
<tr>
<td>L sections</td>
<td>any</td>
<td>any</td>
<td>( b )</td>
</tr>
</tbody>
</table>

The imperfection factor values, \( a \), for the various buckling curves are:

<table>
<thead>
<tr>
<th>Buckling curve</th>
<th>( a_0 )</th>
<th>( a )</th>
<th>( b )</th>
<th>( c )</th>
<th>( d )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.13</td>
<td>0.21</td>
<td>0.34</td>
<td>0.49</td>
<td>0.76</td>
<td></td>
</tr>
</tbody>
</table>

These factors are the same for compression and flexural resistance. Members in compression are assigned buckling curves according to Table 4.

Figure 3 compares the stresses at failure predicted by SANS 10162 with the five EN 3 buckling curves, for Classes 1 to 3 members. This stress can be converted to an ultimate limit-state axial load by multiplying it by the area of a member. The stresses predicted by EN 3 are initially 11% higher, due to the difference in material factors of Ø and γM_1.
Curves \( a_b \), \( a \) and \( b \) are always higher than the SANS 10162 curve. Curves \( c \) and \( d \) drop below the SANS curve if slenderness exceeds 81 and 31 respectively. The theoretical yield stress for short columns, and Euler buckling stress for slender columns, form an upper envelope of all the curves. For the EN 3 buckling equations if \( a = 0 \) and the value of 0.2 in Equation 11 is set to 1.0, the curve will match the Euler and yield stress envelope. Since the EN equations are based on Perry-Robertson buckling, the values can match theoretical values if imperfection factors are removed.

The overall differences in failure stresses are compared in Table 5. The maximum, minimum and root mean square (RMS) of the percentage differences between the SANS and EN curves are given for slenderness ratios up to 200. From Table 5 it is clear that there can be substantial differences in calculated design capacity, such as 35.2% for high-strength steels (curve \( a \)), 24.4% for compression failure stresses – Cr/A (MPa) and as per the SANS 10162-1 code:

The value of \( a_{LT} \) is the lateral-torsional imperfection factor, and is equal to the factors listed in the “Members in tension” section above for the compressive resistance imperfection factor.

The value for \( M_{cr} \) is not explicitly given in EN 3, but it was provided previously in an Informative Annex to ENV 1993-1-1 (1992) as defined by Timoshenko and Gere (1963), and as per the SANS 10162-1 code:

\[
M_{cr} = C_1 \frac{I_y}{W_{pl}} \left( E I_{y} G J + \frac{I_y^2}{E I_{y} G J} \right) \left( \frac{\chi_{LT}}{\chi_{LT}} \right)^{1.0} \tag{21}
\]

where:

\[
\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta_{LT}^2}} \text{ but } \chi_{LT} \leq 1.0 \tag{20}
\]

\[
\phi_{LT} = 0.5 [1 + \alpha_{LT}(\chi_{LT} - 0.2) + \phi_{LT}^2] \tag{26}
\]

\[
\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta_{LT}^2}} \tag{28}
\]

The above equations are modified for rolled sections or equivalent welded sections:

\[
\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta_{LT}^2}} \text{ but } \chi_{LT} \leq 1.0 \& \chi_{LT} \leq 1.0 \tag{28}
\]
$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - \lambda_{LT,0}) + \beta\lambda_{LT}^2] \quad (29)$

The recommended values by CEN for $\lambda_{LT,0}$ and $\beta$ are 0.4 and 0.75 respectively. Then, to account for the shape of the bending moment between supports, $\chi_{LT}$ may be modified as follows:

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} \quad \text{but} \quad \chi_{LT,mod} \leq 1.0 \quad (30)$$

$$f = 1 - 0.5(1 - \kappa)[1 - 2.0(\lambda - 0.8)^2] \quad (31)$$

$\kappa$ is a correction factor from Table 6.6 of EN 3-1-1.

Figure 4 presents a comparison between flexural members designed by SANS 10162 and EN 3, showing the stress at failure relative to a plastic modulus (i.e. stress = $M/f_{pl}$). For SANS the slenderness of members is limited to 300. The members listed in Table 6 have been selected to highlight different aspects, as shown in the table. The UB 457 × 191 × 75 is a heavy Class 1 section with buckling curve $c$. The UC 203 × 203 × 46 is considered a Class 3 section by SANS, but a Class 2 section in EN 3, so different section moduli are used by the different codes. The PFC is designed as a Class 3 based on SAISC Red Book (SAISC 2005) guidelines for SANS, but considered a Class 1 section with buckling curve $d$ for EN 3, and uses Equation 20 rather than 28.

From Figure 4 the differences in the predicted failure stresses relative to the plastic modulus are shown to vary between the selected sections. For the UB 457 × 191 × 82 section it can be observed that, after the initial difference, due to partial factors, the resistances are in the order of –6.2% to 16%. The resistance of the UC 203 × 203 × 46 is initially 23% higher for EN 3, due to partial factors and because the SANS 10162-1 code considers only the member’s elastic resistance and not its plastic resistance. With SANS the PFC 180 × 70 has been designed as a Class 3 section, but under EN 3 it is designed as a plastic section with buckling curve $d$. Thus, there is a substantial difference between these calculated resistances, ranging from 31.1% initially to –27.6% at an effective length of 3 m.

**Members in shear**

SANS 10162-1 and EN 3 have similar means of determining the shear resistance of members. For hot-rolled sections the shear resistance $V_r$, according to the SANS 10162-1 code, is:

$$V_r = \Phi A_f f_y$$  \quad (32)$$

where:

$$A_f = h t_w$$  \quad (33)$$

$$f_y = 0.66 f_y$$ except for plastic hinges with a plastic analysis, then:  \quad (34)$$

$$f_y = 0.55 f_y$$  \quad (35)$$

The plastic shear resistance of a hot-rolled section according to EN 3 is:

$$V_{pl,ld} = A_v (f_y \sqrt{3}) \frac{T}{M_0}$$  \quad (36)$$

$A_v$ may be taken as the following:

a. rolled H & I sections:

$$A_v = A_f + 2r t_f$$  \quad (37)$$

but not less than $0.6 h_t t_w$

b. rolled channel sections:

$$A_v = A_f + (r + 2) t_f$$  \quad (38)$$

For the elastic shear resistance of a section it must be verified that:

$$\tau_{Ed} = \frac{V_{Ed} S}{I t} \leq 1.0$$  \quad (39)$$

where:

$$\tau_{Ed} = \frac{V_{Ed}}{S}$$  \quad (40)$$

$S$ is the first moment of area about the centroidal axis of that portion of the cross-section between the point at which shear is required and the boundary of the cross-section. $I$ is the second moment of area of the whole cross-section, and $t$ is the thickness at the examined point. Equation 39 is the generalised case and is complex to calculate. However, for I or H-sections the shear stress can be calculated by:

$$\tau_{Ed} = \frac{V_{Ed}}{A_w} \leq 0.6$$ \quad (41)$$

where $A_w = h_t t_w$ is the area of the web.
### Table 7: Number of operations required for code strength calculations

<table>
<thead>
<tr>
<th></th>
<th>SANS 10162-1</th>
<th>EN 3-1-1</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tensile resistance</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) Gross resistance</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>b) $A_{slag}$ section resistance (including shear lag)</td>
<td>5</td>
<td>8</td>
</tr>
<tr>
<td>c) Minimum of resistances</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Total number of operations</td>
<td>8</td>
<td>11</td>
</tr>
<tr>
<td><strong>Compressive resistance</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) Section classification</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>b) $I$ about $x-x$ &amp; $y-y$</td>
<td>2 x 7</td>
<td>4</td>
</tr>
<tr>
<td>c) $C_{ts}$ &amp; $C_{tr}$</td>
<td>2 x 9</td>
<td></td>
</tr>
<tr>
<td>d) Minimum of $C_{ts}$ &amp; $C_{tr}$</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>e) $N_{slag}$ &amp; $N_{slag}$</td>
<td>2 x 3</td>
<td></td>
</tr>
<tr>
<td>f) Minimum of $N_{slag}$ &amp; $N_{slag}$</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Total number of operations</td>
<td>40</td>
<td>55</td>
</tr>
<tr>
<td><strong>Bending resistance</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) Section classification</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>b) Calculate $e_{32}$</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>c) Calculate $M_{cr}$</td>
<td>15</td>
<td>5</td>
</tr>
<tr>
<td>d) Check $M_{cr} &gt; 0.67M_{p}$</td>
<td>2</td>
<td>15</td>
</tr>
<tr>
<td>e) Calculate $M_{cr}$</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>f) Calculate $Φ_{LT}$</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>g) Calculate $K_{LT}$</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>h) Calculate $X_{LT,mod}$</td>
<td>11</td>
<td></td>
</tr>
<tr>
<td>i) Calculate $M_{A,Rd}$</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Total number of operations</td>
<td>35</td>
<td>65</td>
</tr>
<tr>
<td><strong>Shear resistance</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) Calculate $V_{cr}$</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Total number of operations</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>87</td>
<td>136</td>
</tr>
</tbody>
</table>

Based on the above equations for $H$ and $I$-sections listed in the SAISC Red Book (SAISC 2005) it can be seen that the EN 3 code predicts, on average, a plastic shear design strength of 30.4% higher than the SANS code. However, the elastic design resistance is 19.7% lower on average. This is based on two factors: (a) for SANS a higher shear stress is allowed for elastic rather than plastic design ($0.66f_{s}^{y}$ vs $0.55f_{y}^{y}$), and (b) for EN 3 a smaller shear area for elastic design is allowed than for plastic ($A - 2ht_{f} + (t_{f} + 2r)t_{f}$ vs $h_{f}t_{f}$). The maximum discrepancy in the calculated shear strengths between the codes is 50.6% for an IPEAA–200 for plastic design and −32.8% for a UC 254 x 254 x 167 for elastic design.

**Code scope**

As was presented in the “Background to the codes” section at the beginning of this paper, there is a large difference in the scope of the works considered by the EN and SANS codes, with the EN documents being far more extensive. An important reason that a country may wish to adopt or adapt the Eurocodes is that they are typically very comprehensive and cover a wide range of issues. However, with this comes added complexity, as discussed below, and generally available expertise in a country should inform the scope of national standards. Topics addressed within EN 3 which are not covered in the SANS codes include fire design, silos, certain joint behaviour, and chimneys, amongst others.

**Computational effort required for design**

It is not only important that a code provides sufficient reliability, but also that it is user-friendly. If a code is too complex it may either not be used, or mistakes may occur more easily. In this section a comparison of the computational effort required to calculate the design strengths of members is given to provide a rough indication of the complexity of each code. Since the design procedures discussed in this paper generally follow a single set of steps, with different equations for each section classification, there are typically not “loops” with repeated calculations that are followed multiple times. However, as designs become more complex and entire systems are considered, the level of calculation required will increase, especially for the Eurocode documents.

Each mathematical operation (e.g. $A + B$) is counted so that the equation

\[
\frac{(A + B)^2 + C}{5C}
\]

is considered to require six operations. If a number must be looked up in a table it is counted as an operation. If a term must be calculated, say $Φ_{LT}$ in Equation 29, and is then used multiple times, the number of operations required to determine its value the first time are not added each time the term is used. There are various ways to determine the number of operations required for calculations, but this approach is being followed as a basic benchmarking exercise. The calculations required for designing connections are not considered in this paper, but are also a very important part of design.

These numbers are only an approximate indication, and will vary depending on the section chosen and the various clauses that must be considered. For example, a Class 4 section in bending will have numerous additional calculations for both codes. Note that for laterally-restrained beams and short columns the computational effort required is the same for both codes. The values presented in Table 7 are based on an angle in tension and a Class 1 I-section for the remaining calculations.

From Table 7 it can be seen that the additional computational effort required to design one member of each type using EN 3 is:

- Tensile resistance: 37.5%
- Compressive resistance: 37.5%
- Bending resistance: 85.7%
- Shear resistance: 25.0%
- Total: 56.3% (based on one member of each kind being considered).

If Equation 27 is used for $T_{LT}$ it reduces the total number of operations by 10 for EN 3 for the bending resistance of members.

From the above it is shown that the EN 3 code is more computationally expensive, but primarily so in situations where buckling must be considered.

Given that there are 20 documents in the full EN 3 set, there is much cross-referencing, which adds additional complexity. The EN suite is very large and must be carefully read to ensure all clauses and clarifications are understood and followed. For instance,

CONCLUSION

This paper presented an overview of the SANS 10162-1 and EN 3-1-1, with technical and practical aspects being compared. It was shown that on average EN 3 predicts higher member design strengths than SANS 10162-1 for most failure modes. These EC design capacities can be higher by up to 11% for tension, 35% in compression, 31% in bending and 51% in shear, although there are cases where strengths of up to 33% lower were calculated, such as for an IPE AA–200 in shear.

Results are influenced by design geometric classifications.

The generally higher estimates of member design capacity by EN 3 are primarily due to (a) partial factors that are closer to unity, (b) less conservative buckling curves, and (c) not only considering a member’s elastic resistance, but allowing the consideration of plastic resistance. Partial material factors that are closer to unity, as well as less conservative buckling curves, may be justified by better quality controls that reduce the variance in steel strength and dimensional deviations. Thus, caution should be exercised when using these factors in South Africa, unless similar material quality and construction quality can be proved. However, if this can be justified, more economic designs may be achieved based on EN 3. On the downside, EN 3 is computationally more expensive, especially when the buckling of members must be considered.

The target reliability levels of 3.0 and 3.8, for the SANS and EN steel codes respectively would suggest that the SANS code should estimate higher resistances (less conservative) for steel members, assuming similar material quality and construction quality. This investigation has shown that the opposite is generally true. If EN 3 was to be adopted in South Africa, calibration exercises would need to be undertaken to ensure acceptable reliability levels. This may be addressed through the adjustment of Nationally Determined Parameters.

REFERENCES

\[ N_{Rd} \] Design values of the resistance to axial forces
\[ R_d \] Design value of resistance
\[ S \] First moment of area about the centroidal axis of that portion of the cross-section between the point at which shear is required and the boundary of the cross-section
\[ V_{Ed} \] Design value of the shear force
\[ V_{Rd} \] Design values of the resistance to shear forces
\[ W \] Section modulus
\[ W_{pl} \] Plastic section modulus
\[ W_{eff} \] Effective section modulus
\[ W_{el} \] Elastic section modulus
\[ \alpha \] Imperfection factor
\[ \beta \] Correction factor for the lateral-torsional buckling curves for rolled sections
\[ \gamma_{M0} \] Partial factor for resistance of cross-sections whatever the class
\[ \gamma_{M1} \] Partial factor for resistance of members to instability assessed by member checks
\[ \gamma_{M2} \] Partial factor for resistance of cross-sections in tension to facture
\[ \lambda \] Non-dimensional slenderness
\[ \lambda_1 \] Slenderness value to determine the relative slenderness
\[ \Phi \] Value to determine the reduction factor
\[ \tau_{Ed} \] Design value of the local shear stress
\[ \chi \] Reduction factor for the relevant buckling curve
INTRODUCTION

The main basin of the Karoo Supergroup covers approximately 700 000 km² (57%) of South Africa’s surface area and consists predominantly of a flysch-molasse succession which has a maximum cumulative thickness of ~12 km (Johnson et al 2006) (Figure 1). The sedimentary rocks of the Karoo Supergroup typically do not yield acceptable pavement aggregates due to the argillaceous nature of the lower (flysch) units and the relatively poor strength arenaceous (molasse) upper units. There is, however, an extensive network of dolerite intrusions which represent the shallow feeder system to the Drakensberg flood basalt eruptions (183 ± 1 Ma) (Duncan & Marsh 2006) which erupted at the end of the Karoo sedimentary succession deposition. These intrusions are collectively called the Karoo Dolerite Suite and have been widely and successfully used as pavement aggregate sources.

There are, however, numerous cases in which dolerite from the Karoo Dolerite Suite has been credited as the cause of premature pavement failure due to alleged rapid degradation of dolerite base course aggregate while in service. Road authorities have therefore included various so-called durability tests in aggregate specifications for basic or mafic igneous rocks in an attempt to prevent such premature failures. Despite this, rapid pavement failures continue to occur and the rapid degradation of Karoo dolerite continues to be blamed for many of the failures. Such failures ultimately result in significant costs related to reconstruction, alternative material investigations, material modification/stabilisation and project delays.

A study with the objective of identifying cases where degradation of Karoo dolerite was the cause of pavement failure was recently undertaken. The secondary objective was to determine if any observed degradation could have been identified using currently specified or alternative testing methods. Three case study sites are presented in this article and the properties of their materials compared to those from five non-problematic dolerite materials.

It is shown that the poor performance of the case study materials was likely due to the poor durability of the materials, manifesting as a reduction in resistance to abrasion and attrition. The identification of the observed poor durability could not have been performed accurately using only the currently specified test specifications. Alternative tests that allow an accurate differentiation to be made were, however, identified and, based on the results, tentative limits set. Additionally it was shown that modification of problematic Karoo dolerite base course materials, by applying lime at a rate less than the initial consumption of lime, can be successful in preventing further rapid pavement failures.

The identification and treatment of poor durability Karoo dolerite base course aggregate – evidence from case studies

R C Leyland, M Momayez, J L van Rooy

The Karoo Dolerite Suite (intrusions) present in these sedimentary units have successfully been used as pavement aggregate sources, but numerous cases of premature pavement failure due to alleged rapid degradation of the dolerite have been reported. Durability tests are included in basic or mafic igneous rock aggregate specifications, but rapid pavement failures continue to occur. A study was recently undertaken to identify cases where degradation of Karoo dolerite was the cause of pavement failure. A secondary objective of the study was to determine if any observed degradation could have been identified using currently specified or alternative testing methods. Three such case study sites are presented in this article and the properties of their materials compared to those from five non-problematic dolerite materials.

The Karoo Supergroup covers approximately 57% of South Africa’s surface area and the sedimentary rocks therein generally do not yield acceptable pavement aggregates. The Karoo Dolerite Suite (intrusions) present in these sedimentary units have successfully been used as pavement aggregate sources, but numerous cases of premature pavement failure due to alleged rapid degradation of the dolerite have been reported. Durability tests are included in basic or mafic igneous rock aggregate specifications, but rapid pavement failures continue to occur. A study was recently undertaken to identify cases where degradation of Karoo dolerite was the cause of pavement failure. A secondary objective of the study was to determine if any observed degradation could have been identified using currently specified or alternative testing methods. Three such case study sites are presented in this article and the properties of their materials compared to those from five non-problematic dolerite materials.

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Karoo dolerite pavements where no material degradation was reported or suspected. Additionally, the treatment of material at one of the study sites is discussed.

**REVIEW**
Research on the durability of Karoo dolerite aggregates has been performed by numerous authors since the middle of the previous century (e.g. Bell & Jermy 2000; Dunlevey & Stephens 1996; Hall & Harris 1985; Kleyn et al 2009; Orr 1979; Walker & Poldervaart 1949), and based on the findings of such studies additional specifications for basic igneous rock base course aggregates have been added to the current South African standard specifications. The currently accepted published specifications for pavement materials in South Africa (COLTO 1998) include limits for material parameters related to poor durability, and for tests designed to simulate potential degradation and associated changes in material properties (Table 1). Although some South African road authorities do include additional pro forma specifications, these are not

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Specification</th>
<th>Motivation</th>
</tr>
</thead>
<tbody>
<tr>
<td>10% FACT</td>
<td>110 kN</td>
<td>Identify materials in which strength has degraded.</td>
</tr>
<tr>
<td>Wet/dry 10% FACT ratio</td>
<td>≥ 75%</td>
<td>Accelerate potential degradation of strength.</td>
</tr>
<tr>
<td>ACV</td>
<td>&lt; 29%</td>
<td>Identify materials in which strength has degraded.</td>
</tr>
<tr>
<td>PI</td>
<td>Individual ≤ 5</td>
<td>Identify materials in which fine fractions are of unsuitable nature (possibly due to unfavourable degradation).</td>
</tr>
<tr>
<td></td>
<td>Average ≤ 4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤ 12*</td>
<td></td>
</tr>
<tr>
<td>LS</td>
<td>≤ 2.0</td>
<td>Identify materials in which fine fractions are of unsuitable nature (possibly due to unfavourable degradation).</td>
</tr>
<tr>
<td>DMI</td>
<td>≤ 125</td>
<td>Accelerate potential degradation of strength and identify materials with poor resistance to attrition and abrasion.</td>
</tr>
</tbody>
</table>

10% FACT = 10% Fines aggregate crushing test, ACV = Aggregate crushing value, PI = Plasticity index, LS = Linear shrinkage, DMI = Durability mill index

*When PI is determined on −0.075 mm fraction because −0.425 mm fraction is non-plastic
available in published form (e.g. SANRAL’s standard pro forma amendments to the COLTO 1998 specifications).

Where materials available in the vicinity of a pavement construction site are considered marginal with respect to these specifications, an alternative to hauling acceptable material long distances is the stabilisation of the materials using lime or cement (Netterberg 1994). The additional cost of stabilising suspected poor durability basic igneous materials is, however, excessive and results in rigid pavement layers which require design adaptations. Alternatively marginal materials may be treated by adding lime at a rate significantly lower than what is typically used in stabilisation (Wiener 1980; Kleyn & Berg 2008; Kleyn et al 2009). This process, known as lime modification, does not increase the strength of the material, but is employed to reduce the PI of the material and maintain a high pH to prevent further mineralogical degradation (Clauss 1967; Weinert 1980). The differences between soil modification and stabilisation reactions are discussed in further detail by Ballantine and Rossouw (1989), and the Committee of State Road Authorities (CSRA 1986).

The procedures followed to perform lime stabilisation are, however, not standardised and, as shown by Netterberg (2004), if the PI of the lime used is high, the PI of the material may increase, thus negating the desired effect. The permanency of the effects of such modifications to basic igneous rock materials has also not been investigated.

The material at one of the case study sites was modified by adding 1% lime after the initial failures were noticed, and following this no similar failures were observed at that site.

**CASE STUDY SITES AND METHODOLOGY**

The study sites were widespread across South Africa and the Karoo Main Basin,

**Table 3** Tests performed on samples

<table>
<thead>
<tr>
<th>Test</th>
<th>Method</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Petrographic examinations</td>
<td>N/A</td>
<td>Thin section analysis including point counting using a PETROG digital stepping stage and PetrogLite™ software</td>
</tr>
<tr>
<td>Aggregate impact value (AIV)</td>
<td>British Standard BS812: Part 3 (BS 1975)</td>
<td>Performed on dry material and material soaked in ethylene glycol (not water) for 24 hours</td>
</tr>
<tr>
<td>Durability mill index (DMI)</td>
<td>South African National Standard SANS 3001-AG16 (SANS 2013a)</td>
<td>Performed on dry material and material soaked in ethylene glycol for five days</td>
</tr>
<tr>
<td>Compressive and shear wave velocity</td>
<td>ISRM (1978) standard method</td>
<td>Performed on quarry sample cores only</td>
</tr>
<tr>
<td>Point load index (PLI)</td>
<td>ISRM (1985) method</td>
<td>–</td>
</tr>
<tr>
<td>Modified ethylene glycol durability index (mEGDI)</td>
<td>South African National Standard SANS 3001-AG14-2013 (SANS 2013b)</td>
<td>–</td>
</tr>
<tr>
<td>Water absorption</td>
<td>ASTM C 97–02 method</td>
<td>Performed on quarry sample cores only</td>
</tr>
</tbody>
</table>
and one site was located in the Lebombo Group (Figure 1). The climatic N-value (Weinert 1980) of the sites varied from 3 to 9. The pavement layer materials are summarised in Table 2, and the observed defects at these sites included rutting and aggregate loss in wheel tracks, bleeding in wheel tracks over extended areas, bleeding in isolated areas and total surface failure (Figure 2).

Material was sampled from the base layer of each pavement and from the original material source quarry. Since the three sites at which rapid pavement failures were observed experienced failure while sections of the road were still under construction, the respective material sources were easily accessible for sampling. The reference sites were of various ages and the source quarry of each had to be identified from construction records.

Currently specified (COLTO 1998) aggregate characterisation tests, as well as additional tests commonly included in pro forma amendments to the COLTO (1998) specifications, were then performed on all samples and the resultant data analysed. The aim was to determine if the rapid failure site materials had undergone any, or unusually high amounts of, degradation compared with materials from the reference sites. The tests performed are listed in Table 3.

Although petrographic examinations are not formally incorporated in the quantitative specifications, a qualitative classification of “shall not contain deleterious material such as weathered rock” is required by COLTO (1998), and the examinations were therefore included. The results and interpretations of the modified Ethylene Glycol Durability Index (mEGDI) tests and petrographic investigations are presented in Leyland et al (2013), Leyland (2014a) and Leyland (2014b) and are not discussed further here.

Table 4 Results summary

<table>
<thead>
<tr>
<th>Sample</th>
<th>AIV (%)</th>
<th>10% FACT*</th>
<th>DMI</th>
<th>Velocity (m/s)</th>
<th>PLI (MPa)</th>
<th>WA (%)</th>
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<tbody>
<tr>
<td>Std Gly</td>
<td>Std Gly</td>
<td>Glycol/ dry ratio (%)</td>
<td>Std</td>
<td>Gly</td>
<td>Mod Gly</td>
<td>P wave</td>
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<td>378</td>
<td>362</td>
<td>96</td>
<td>45</td>
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</table>

* Determined using Equation 1
Sample number index: B = base sample, Q = quarry sample, Std = Standard test procedure, Gly = Samples soaked in ethylene glycol, Mod = Modified, WA = Water absorption
problematic quantification of expansive clay minerals in dolerite aggregates is discussed by Leyland et al (2014), and for this reason smectite clay minerals are not included in this discussion.

Because the limited sample size of materials obtained from the base layers was not sufficient for 10% FACT testing, the AIV result was used to estimate the 10% FACT using the relationship published by Sampson and Roux (1982) (Equation 1). The PI and LS values were determined during Durability Mill Index (DMI) testing procedures.

\[
10\% \text{ FACT} = 10(2.915 - (0.03 \cdot AIV))
\]  

(1)

At the site where lime modification was performed, samples were obtained from the base layer directly after construction and at irregular intervals over a period of two years. The pH and PI of the samples were determined to observe the immediate and long-term effects of the modification process.

RESULTS AND ANALYSIS

Material parameters
The results of testing performed on quarry and untreated base samples from all sites are summarised in Table 4 on page 29. Samples labelled 1.x, 2.x or 3.x are from the case study sites, while all other samples are from reference sites. What are not included in Table 4 are the PI and LS values obtained during DMI testing. For all quarry samples these parameters were within the required specifications, while the base samples from the case studies, and one reference site, had LS values above the current limit of 2% (maximum value of 5%). The PI value of one case study site base material was one percentage point above the specified limit of 4%.

The first analysis performed on the data was the determination of the maximum and minimum differences between the quarry and base AIV and DMI values. Since these parameters are a measure of a material’s resistance to impact loads and attrition/abrasion forces respectively, an increase in these values, and therefore a positive difference in results, would indicate that the material has undergone degradation in the time between quarrying and sampling from the pavement. The precision associated with both tests will always result in minor variations in results, thus small changes and even small negative changes are to be expected, even if no actual change in the material occurred. Similarly, minor variations in the material will result in a range of results which may also result in minor

<table>
<thead>
<tr>
<th>Site</th>
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<th>ΔDMI</th>
</tr>
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<td></td>
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<td>Max</td>
<td>Min</td>
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<tr>
<td></td>
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<td>Rapid failure case study</td>
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<td>3</td>
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<td>-1</td>
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</tr>
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<td>7</td>
<td>Reference site</td>
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</tr>
<tr>
<td>8</td>
<td>Reference site</td>
<td>4</td>
<td>2</td>
</tr>
</tbody>
</table>
positive and negative differences that are not representative of actual changes due to material degradation.

The results (Table 5) revealed that the changes in AIV were minimal at most sites and not significantly different for the case studies. The range of AIV increases after quarrying also included negative results typically similar to the positive results indicating that most of the results are due to material variation and not due to degradation. A very different observation was made for DMI changes, as all of the case studies revealed significant increases in DMI values, while the reference sites revealed either irrelevant changes (due to material variation) or very low increases in DMI values. This analysis proves that the materials used in the case study sites did undergo a change in resistance to attrition and abrasion (as measured by DMI test) and are therefore, compared with the reference sites, of poor durability. It is interesting that the observed degradation did not cause similar trends in the resistance in impact loads (as measured by AIV results).

Having established that the materials from case study sites had undergone some form of degradation, the differences in all test results between quarry samples from case study sites and reference sites were considered. Detailed descriptions of petrographic features are presented in Leyland (2014a; 2014b) and only summarised here. The petrographic properties of the case study sites were different from the reference sites, due to higher percentages of secondary minerals, evidence of secondary mineral alteration to clays and large degrees of myrmekitic alteration. Although many reference materials had secondary minerals present, they lacked evidence of significant or pervasive further alteration to clays or other deleterious minerals. The occurrence of olivine and the degree of olivine alteration was similar in all materials, and was therefore proven to not be related to the observed durability problems.

The AIV values obtained for the case study quarry samples were generally higher than the reference sample values, but the ranges did overlap (Figure 3). According to Sampson and Roux (1982), crushed rock AIVs can be equated to aggregate crushing values (ACVs), and when this was considered all samples were seen to be well below the current ACV limit of 29%. The DMI result ranges overlapped even more and were all well below the limit of 125 (Figure 3).

When the AIV results are converted to 10% FACT values and the ratio of the glycol and dry results are considered, it is seen that all reference values are above the current wet/dry ratio minimum limit of 75%, but that the case study material results cover a wide range on either side of the limit (Figure 4). The modified DMI (mDMI), proposed by Leyland (2014b), is calculated from the DMI test data using the changes in PI and P0.425 mm values during testing. These values showed a significant differentiation, as all case study sites produced higher values than the reference sites, which had a limited range of results (Figure 4).

The seismic wave velocity results (Figure 5) revealed a significant overlap between compressive (P) wave velocities, while the shear (S) wave velocities were noticeably lower in case study materials. The P wave velocities measured are within the ranges reported by Kilic (1995) for diabase, Tuğral and Zarif (1999) for granites, and Bell and Jermy (2000) for Karoo dolerites, but well above those for weathered basalts measured by Sharma and Singh (2008). Similarly to the S wave velocities, the PLI results for different site types overlapped a significant amount (although those for reference sites were on average much higher (Figure 6)), while the water absorption results for different site types fell into two distinctly different ranges.
Material treatment

Despite the initial consumption of lime of the material being 2.0–2.5%, the addition of 1% lime resulted in a general increase in the pH of the material to a level of at least 11, and this change was sustained throughout the monitoring period (24 months) (Figure 7). The PI results indicated that all materials had a PI of 4–6% before modification, while three months after modification the PI treated materials had a PI ranging from non-plastic (NP) to 2% (Figure 8). At the sampling time, samples that were not modified were collected as control specimens and these had a PI of 3.5–7.0% (similar to initial samples).

The addition of lime therefore appeared to have lowered the PI of the materials by similar amounts, resulting in materials with relatively low unmodified PI values becoming NP, and PI values slightly above the currently specified limit being lowered to within the limit. Continued sampling and testing of both treated and untreated materials after 9, 17 and 24 months revealed that all materials had PI values of less than 4, thus providing evidence that the modification effects had been sustained. The untreated materials sampled after nine months that had PI values of < 2% are believed to represent the lower extreme of natural PI variation in the source material.

DISCUSSION

It has been shown that the poor performance of the three case study sites was likely linked to the poor durability of the materials, which resulted in a reduction in the material resistance to abrasion and attrition forces. The identification of the poor durability of the material from all three quarries could not have been performed accurately using only the currently specified test specifications and, due to the significant overlap of results obtained, it appears that even an adjustment in the limits will not allow suitable differentiation between materials of varying durability. The water absorption, when performed on core samples, is an exception to this, but this parameter is not specified exclusively for basic igneous rocks and the observed results would require an adjustment of specified limits (currently determined on crushed samples).

The results did, however, prove that the interpretation of alternative tests can allow an accurate differentiation to be made. The modified interpretation of DMI results obtained from ethylene glycol soaked samples and the S wave velocity measured on core samples both produced promising results in this regard. Tentative limits for Karoo dolerite mDMI and S wave velocities can therefore be set as 7 (maximum) and 3 850 m/s (minimum) to ensure suitable durability. These, along with the promising specifications proposed for mEGDI (Leyland et al 2013), provide a method to identify poor durability materials.

It may also be possible for cross-hole seismic tomography surveys to be used to identify areas of poor durability rock within a prospective source. However, rock with a low rock quality designation (RQD) will provide low seismic velocities (as discussed by Wadhwa et al 2009), but may not necessarily be poor durability rocks. In the study by Wadhwa et al (2009) cross-hole seismic surveys showed the Deccan traps bedrock basalts to have primary wave velocities lower (± 5 500 m/s) than those presented in this research.

The modification of Karoo dolerite base course materials that have slightly elevated PI values has been shown to be effective when performed with lime applied at a rate less than the initial consumption of lime. The effects of the modification are an increase in pH and a decrease in PI. Although the PI of the materials was not consistently reduced to an NP level, it was reduced to within the current specifications, and this appeared to be sufficient, based on the observed cessation of further rapid pavement failures. However, as mentioned, the longevity of the effects are not known as of yet.
CONCLUSIONS
The objectives of the study were reached because cases of rapid pavement failure were positively linked to degradation of Karoo dolerite, and it was shown that the currently specified tests, and their proposed result limits, were unable to identify the materials from these case studies as poor durability materials. Alternative testing methods were, however, identified as potentially more favourable solutions to identify Karoo dolerite sources that are susceptible to degradation when used as crushed rock pavement aggregates.

REFERENCES
Ballantine, R W & Rossouw, A J 1989. Alternative testing methods were, however, identified as potentially more favourable solutions to identify Karoo dolerite sources that are susceptible to degradation when used as crushed rock pavement aggregates.

Ballantine, R W & Rossouw, A J 1989. Alternative testing methods were, however, identified as potentially more favourable solutions to identify Karoo dolerite sources that are susceptible to degradation when used as crushed rock pavement aggregates.


Disintegration of concrete construction induced by acid mine drainage attack

S O Ekolu, S Diop, F Azene, N Mkhize

This paper presents findings from microanalytical investigation conducted on disintegrated concrete that had been used to construct a weir within a coal mine in South Africa. The concrete was in contact with polluted mine water, commonly referred to as acid mine drainage (AMD). Accordingly, the weir had been exposed to AMD decant which led to disintegration of concrete due to chemical attack. Investigations were conducted by optical microscopy (OM), scanning electron microscopy (SEM) equipped with energy dispersive X-ray spectrometry (EDX), and X-ray diffraction (XRD). The field samples examined consisted of soft, broken concrete chunks and a whitish powdery substance that had crystallised and formed a surface coating on certain cracked locations on the deteriorated concrete. No evidence of pyrite oxidation was found in the investigation. The observed deterioration is discussed in relation to acid attack mechanism and its possible co-existence with external sulphate attack process.

INTRODUCTION

Acid mine drainage (AMD) refers to polluted, typically acidic mine water formed from oxidation of sulphidic ores from mining works, which at later stages undergo chemical reactions under atmospheric conditions. In recent years AMD has become a serious problem in South Africa, as in other countries with mining activities, due to concerns regarding its impacts on the environment and on infrastructure (JMC-AMD 2010).

AMD typically occurs decades after mining activities have been abandoned at the mining site, and poses no immediate problem. However, it is the discharge of AMD from its source of formation to the surrounding environment and water courses that creates the problem. For underground mines, AMD may pollute groundwater aquifers which usually provide quality water for industrial and domestic consumption. Underground mines may also decant the polluted water to the surface, as has been the case in the South African goldfields. Surface mines and tailings can also cause discharge of AMD at the earth’s surface level, directly or indirectly impacting the environment and its inhabitants (McCarthy 2011).

Among other effects, AMD can be expected to cause detrimental impacts on engineering infrastructure, including corrosion of plumbing and water conveyance systems, corrosion of pumps and water pumping equipment, and the deterioration of roads, bridges and other highway structures. Some concrete structures are also used at mining sites, although on a relatively small scale, and it is in this regard that the observed degradation of the field concrete occurred (Azene & Ekolu 2012).

In the literature, problems resulting from the presence of sulphate-bearing aggregates in concrete have generally been well established and studied quite significantly. It is known that sulphate-bearing minerals of pyrites and pyrrhotites undergo oxidation, producing sulphuric acid and degradation within the cementitious matrix (Oberholster et al 1984; Bromley 2015; Rodrigues et al 2012; Tagnit-Hamou et al 2005; Cody et al 1997; Thomas et al 1989; Shayan 1988; Ayora et al 1998). However, there are hardly any research papers concerning the interaction between AMD and concrete. Perhaps the rarity of these scenarios is related to the typically remote location of AMD sources, being found in mines far removed from the areas of massive infrastructure. The closest past investigations of this kind, conducted on AMD—concrete interactions, were laboratory experiments by Breitenbucher and Siebert (2008). In their study, AMD was simulated using acid-sulphate solutions made using varied concentrations of iron (II) sulphide (FeSO₄) and sulphuric acid. They found that solutions containing high sulphate concentrations from iron (II) sulphates exhibited acid attack at values in the pH range of 4. Although AMD has recently become quite a significant issue in South Africa, there is little or hardly any research so far done concerning its effect on infrastructure materials. This paper directly links degradation of concrete construction in the field to an attack by AMD from the mines.
Acid mine drainage

There are different forms of acids that may cause attack on concrete, ranging from weak acids such as acetic, carbonic, lactic, tannic and phosphoric acids, to strong acids namely nitric, sulphuric and hydrochloric acids. In the current investigation, acidic (mine) water under study is a by-product of pyritic oxidation typically emanating from abandoned mining works and resulting from exposure of pyrites (FeS₂) to oxygen and moisture. It may be emphasised that the formation of sulphuric acid (or the AMD) is a cyclic process of regeneration leading to its continuous discharge, until the pyrites are exhausted or the support environmental conditions terminate. Understanding of the AMD formation process was mainly recognised in the 1980–90s (Bromley 2015; Howie 1979, 1992; Newman 1998). This reaction process is explained by Equations 1 to 3 (Bromley 2015; Pankaj et al. 2011).

\[ 2\text{FeS}_2 + 7\text{O}_2 + 2\text{H}_2\text{O} \rightarrow 2\text{FeSO}_4 + 2\text{H}_2\text{SO}_4 \] (1)

\[ 4\text{FeSO}_4 + \text{O}_2 + 2\text{H}_2\text{SO}_4 \rightarrow 2\text{Fe}_2(\text{SO}_4)_3 + 2\text{H}_2\text{O} \] (2)

\[ 7\text{Fe}_2(\text{SO}_4)_3 + \text{FeS}_2 + 8\text{H}_2\text{O} \rightarrow 15\text{FeSO}_4 + 8\text{H}_2\text{SO}_4 \] (3)

Initially pyrites react with oxygen and water to produce ferrous (iron II) oxide and sulphuric acid (also sulphate), as shown in Equation 1. With abundant presence of atmospheric oxygen and moisture, the ferrous ions convert to ferric iron, both forms existing in dissolved state (Equation 2). The ferric oxide formed from Equation 2, may further oxidise additional pyrites, creating a cycle of continuous acid generation, given in Equation 3. Additionally, a bacterial species *thiobacillus ferroxidans* is known to oxidise iron and sulphur in pyrites at a low pH < 3.5. At later stages, after the formation of AMD and during its discharge into the environment, hydrolysis may occur, depending on the pH level of the solution. If the AMD is highly acidic with low pH < 3.5, then little or no hydrolysis may occur. At higher pH, the ferric (iron III) oxide will convert to ferric hydroxide, which then precipitates out of solution as a brownish-orange rust-like product.

The stages defined by Equations 1 to 3 represent the generic process by which AMD forms. In a typical case, the AMD formed seeps through various rock strata as it flows from its reaction sites to the general environment, and in the process dissolves certain metals from rocks and other materials within its pathway. These dissolved metals then become incorporated into the AMD’s chemical composition. Accordingly, AMD is typically characterised by high acidity, high content of heavy metals, high amounts of dissolved solids and high sulphate concentrations. In AMD, sulphates are primarily in the form of iron disulphate. While other types of sulphates, such as sodium sulphate or magnesium sulphate, are known to cause severe attack in concrete, it has been shown that iron disulphate appears not to inflict major damage or expansive attack under acidic-sulphate conditions (Breitenbacher & Siebert 2008). The chemical signature of AMD is more complex than the composition of mineral acids. For this reason AMD can be viewed as an acid or acidic water of a unique category; it cannot therefore be expected to behave precisely as mineral acids, due to possible interactions that could result from its various dissolved ions.

**Possible mechanisms of concrete attack by acid mine drainage**

Considering the typical chemistry of AMD, which consists of high acidity and high sulphate concentration, the postulated AMD damage attack on concrete may take the forms of acid attack and/or sulphate attack mechanisms.

**Acid attack in concrete**

When an acid comes into contact with concrete, the first line of attack arises from the reaction of the acid with portlandite (calcium hydroxide – CH) leading to the formation of calcium sulphate (i.e. gypsum) and water. In the case of sulphuric acid, the reaction may be written as in Equation 4 (Zivica & Bajza 2001):

\[ \text{H}_2\text{SO}_4 + \text{Ca(OH)}_2 \rightarrow \text{CaSO}_4 + 2\text{H}_2\text{O} \] (4)

It is generally acknowledged that the second phase, involving destruction of tobermorite or C-S-H, will only follow after portlandite dissolution (Hill et al 2003; Sersale et al. 1998). The second phase is a more advanced and severe acid attack, above and beyond the effects arising from portlandite dissolution alone. It is essential to note here that the reaction product of acid attack is principally gypsum. However, other sulphate-bearing reaction products may also be present in smaller quantities. Acid attack on concrete is generally exhibited at pH < 6.5 (Fattuhi & Hughes 1983), but becomes severe and more pronounced at pH levels of 3 to 4, which is the range of pH values that were determined for the AMD under this investigation (Azene & Ekolu 2012). According to DIN 4030 (2008), the degree of water aggressiveness is considered to be severe for a pH range of 4.5 to 5.5, and very severe for pH < 4.5 (Ekolu & Azene 2012; Earlie & Callaghan 1988).

For groundwater, the conditions suggestive of potential acid attack are water of pH ≤ 5.0, total acidity of ≥ 25 mg per 100 g of soil, and a sufficient groundwater rate of replenishment (Earlie & Callaghan 1988; Bearly 1980). The acid will typically attack and destroy the concrete surface, but the latter will in turn tend to neutralise the acid due to the high alkalinity of concrete (Ekolu et al 2013; Makhloufia et al 2014). A fresh supply of acid will launch further attacks progressively into the interior of concrete (Woodson 2009; Attiogbe & Rizkalla 1988). Most concrete structures also contain steel reinforcement which is protected by a concrete cover of about 50 mm thick. Corrosion of the cover concrete would expose the reinforcement, not only to acid attack, but also to steel corrosion.

Studies have shown that, upon exposure of cementitious systems to acidic media over a prolonged period of time, the physical damage that ensues is characteristically associated with loss in mass, loss in compressive strength, and increase in porosity and permeability (Fattuhi & Hughes 1983; Attiogbe & Rizkalla 1988; Fattuhi & Hughes 1988; Adesanya & Raheem 2010; Senhadji et al 2014). These properties are symptomatic of the underlying intrinsic degradation of chemical phases, which may only be confirmed through microanalysis. Indeed, studies using SEM/EDX and XRD show that microstructural damage due to acid attack typically exhibits microcracking, often infilled predominantly by a whitish product of reaction, gypsum (Senhadji et al 2014; Song et al 2005; Rendell & Jauberthie 1999; Tulliania et al 2002; Hasan 2009; Xie et al 2004).

**Sulphate attack in concrete**

External (as opposed to internal) sulphate damage would be the most likely form of sulphate attack relevant to the AMD scenario. From understanding of hydration reactions of cement, the most predominant phases present in hardened concrete are monosulphate, tobermorite (C-S-H) and portlandite, along with unreacted cement compounds and ettringite. In the process of external sulphate attack, the sulphate ions from an external source (in this case AMD) penetrate into concrete. These ions then react with free and abundantly available portlandite in the ground mass of concrete, forming gypsum, which in turn reacts with monosulphate to form ettringite (Thomas & Skalny 2006; Tian & Cohen 2000; Al-Amoudi 2002), as given in Equation 5:
Hence, in a cementitious system subject to sulphate attack, ettringite is the predominant reaction product (perhaps with some presence of gypsum), generating expansive pressures reported to be in the range of 8 MPa, depending on the sulphate concentration (Müllauer et al. 2013). These high pressure levels by far exceed the tensile strength of concrete, which can be as low as 2 MPa (Kong & Evans 1980). Accordingly, the formation of gypsum and ettringite minerals is expansive, leading to disruption of the concrete matrix, and is characterised by mass increase, cracking and disintegration, as well as strength loss (Santhanam et al. 2003; Ekolu & Ngwenya 2014a, 2014b; Ekolu 2014).

The common different types of sulphate salts known to cause attack in concrete are typically Na2SO4 (Glauber salt), MgSO4 (Epsom salts) and CaSO4 (Gypsum). For an AMD signature that has a high concentration of iron, the dominant sulphate compound would most likely be FeSO4 (iron sulphate). But studies have shown that FeSO4 may not be as damaging or expansively attacking as the alkaline-based sulphate compound forms (Breitenbucher & Siebert 2008). In those sources of AMD where alkalis such as sodium, potassium or magnesium may be present in significant concentrations, normal sulphate attack could be possible. It can therefore be surmised that the possibility of sulphate attack depends on the source (and therefore chemical composition) of the AMD.

**BACKGROUND TO THIS INVESTIGATION**

While conducting field survey of an abandoned coal mine, the authors of the current study observed severe exfoliation of a concrete weir that was in contact with AMD (Figure 1). A detailed description of the field inspection undertaken was given in an earlier publication (Azene & Ekolu 2012), where an index assessment for AMD aggressiveness was conducted. This paper presents a microanalytical study of concrete samples that were taken from site and subjected to examination with the aim of identifying the possible mechanism of attack. The study was conducted using OM, XRD and SEM fitted with EDX.

**FIELD SAMPLES**

The samples used in this investigation were AMD and deteriorated concrete specimens collected from the site. As described by Azene and Ekolu (2012), severe exfoliation had been exhibited in concrete above the AMD flow level, showing:

- A line of disintegration separating concrete above the AMD flow level from the foundation (Figure 1)
- A map-cracking pattern, seemingly oriented longitudinally along the length of the wall lining (the deterioration and cracking occurred across the 250 mm thick wall of the concrete weir, due to AMD penetration (Figure 1)
- Swelling and spalling of the concrete leading to falling out of disintegrated chunks of concrete
- Depositions of powdery salt crystals on the concrete surface (the surface depositions exhibited a white-yellowish discoloration)
- Erosion of cement past matrix leaving protruding aggregates.

### Acid mine drainage from the coal mine

Table 1 shows the chemical composition and pH of AMD collected from the source of attack on concrete. The crucial parameters of the AMD are its low pH of 3.0 and high sulphate concentration of 5 200 ppm. This degree of water aggressiveness falls under the very severe category (DIN 4030 2008; Neville 1996). It can be seen from Table 1 that the main ion species responsible for aggressiveness in the AMD were the Cl–, Fe+ and SO42– ions. Evidently, the concentrations of Cl– and Fe+ in the water are too low to be of concern, leaving the SO42– to be the possible species with potential to inflict damaging attack, especially considering the high sodium concentration. However, it was of interest in this study to determine whether the active mechanism was acid attack, sulphate attack or combined acid and sulphate attack processes. It may be noted that use of the term “acid” in AMD does not necessarily imply that all forms of AMD are acidic. In fact, some forms of AMD, referred to as neutral mine drainage (NMD), are not acidic at all and have a neutral pH (Heikkinen et al. 2009; Pope et al. 2010; Lindsay et al. 2015). The potential for NMD or AMD attack on concrete, metals or any other materials, depends on its geological source, which in turn relates to its chemical signature.

### Concrete samples and reaction products

The exfoliation of concrete was observed at upper layers of the weir, above the AMD flow level, as described in Azene and Ekolu (2012). The concrete was severely exfoliated, to the extent that it softened and crumbled, and could be easily broken off by hand. The crumbled pieces of concrete were collected and used for analytical studies. Another interesting feature of deterioration was the whitish powder crusts of a reaction product.
formed at the surface of the severely cracked concrete. At the site, samples of the powder salts were scraped from the concrete surface using a dry, clean wooden stick. The collected samples were then placed in plastic bags and sealed before transporting for analytical investigation in the laboratory.

Non-destructive testing of the concrete using the Schmidt hammer method (Neville 1996; Breccolotti et al. 2013) found its compressive strengths to be in the range of 20 MPa. The concrete was made using 22 mm maximum coarse aggregates.

**ANALYTICAL STUDIES AND DISCUSSIONS**

The three main techniques that were used in microanalysis and petrographic examination of the deteriorated field concrete consisted of OM, XRD and SEM/EDX. Different samples of the field concrete were prepared in the laboratory, and used for analytical studies.

**Optical microscopy (OM)**

Thin sections were prepared from the concrete chunk samples and examined using an optical microscope Olympus BX41TF, with high-resolution imaging capabilities of up to 400x magnification, equipped with an Olympus Camera SC100. Petrographic examination showed characteristics of a severely altered concrete matrix exhibiting distinct features of deterioration.

**Aggregates**

Petrographic examination revealed the presence of predominantly quartz aggregates and opaque minerals, as the main types of aggregates used in the concrete mixtures. Figure 2a illustrates the typical quartz aggregates identified in the damaged concretes. Evidently the aggregates are circumvented by a network of cracks, as discussed later. Figure 2b gives a closer view of the aggregate surface features. It can be seen that the quartz particles are severely weathered, indicating a likely leaching away of surface material, including cement paste and the associated phases. Leaching is a highly likely possibility, given the exposure of the concrete to flowing AMD which then caused erosion of leachable phases both from the cement matrix and aggregate surface. There was also an observation of fractured quartz aggregate particles as seen in Figure 2c. However, no infilling of the fracture was evident. A close-up of the fractured particle, shown in Figure 2d, shows a good bond between the particle and the cement paste, but there appears to be concentration of unreacted pyrites lining along the aggregate rim and further into an opaque mineral aggregate. In Figure 3a, sulphate bearing minerals, most likely pyrite and pyrrhotite, can be seen distributed throughout the opaque mineral aggregate. This feature can be seen further at a higher magnification given in Figure 3b, clearly showing a widely disseminated (pale yellow) pyrite within the opaque aggregate particle.

**Scanning electron microscopy (SEM)**

The SEM examination allows characterisation of the phase morphology, which usually provides useful information in phase identification. The EDX gives elemental information and their relative proportions. This information is often used to confirm a distinct presence of specific phases. Concrete is a heterogeneous material with several inherent phases consisting of pores, unhydrated cement phases, hydration products and minerals, and aggregates. Various forms of attack on concrete, such as sulphate attack, acid attack, etc., typically introduce new phases that are not normally present in substantial quantities within the hardened cementitious system. In microanalysis, such unusual phases can be identified and examined.
In this investigation, SEM analyses were conducted using a Leica 440 Stereoscan SEM equipped with an INCA (OXFORD) EDX. Fractured samples of the deteriorated concrete were used in order to view image features free of disturbances such as polishing. Samples were cleaned using acetone, air-dried and mounted onto an SEM stub by means of an adhesive, then carbon coated. Image analysis was conducted in backscattered electron image (BSE) mode, so as to obtain some qualitative information related to composition.

Figure 6a gives a BSE image of deteriorated concrete taken from the field site. The main mineral phase seen in the micrograph is gypsum. Also seen in the image is contamination associated with clay and plant material intermixtures. The identified quartz is a phase from the sand used in the concrete, confirming observations made by petrographic examination. The dominant presence of gypsum in the pore structure is unusual in normal concrete and represents a likely attack process. It is also evident that gypsum forms the major groundmass product of the micrograph. Figure 6b shows the associated EDX spectra for the various phases identified. It can be seen (Figure 6b) that the EDX spectra for gypsum is mainly calcium (Ca), sulphur (S) and oxygen (O), which form the main elemental composition of the mineral phase. Similarly, fine aggregate of quartz consists predominantly of silica (SiO₂). The other spectra are for the clay and plant contaminants found intermixed in the concrete. Further SEM analysis, using another sample taken from the deteriorated field concrete, is given in Figure 7 in the form of an electron micrograph and the associated EDX spectra of the major phases observed. Again, the electron micrograph shows an infilling of gypsum as the dominant product in the groundmass of the matrix. The morphology of the gypsum consists of crumbled, packed crystals being perhaps an indication of the existence of expansive pressures exerted against the crack walls. Also evident in Figure 7 is a trigonal hexagonal phase, likely to be calcite. This phase was detected through XRD analysis, as shown in Figure 8. No other sulphate-bearing phases such as ettringite or thaumasite were observed. It may be noted that thaumasite formation necessitates the presence of carbonates in concrete, along with its exposure to low temperatures typically below 4°C. Both of these conditions did not exist at the site of this study, and therefore thaumasite formation would be unlikely. Also not observed were any rust product phases, such as goethite (FeO(OH)), pyrite oxidation as participant in attacking the concrete. Again, no portlandite was found among the phases identified. This implies that this phase is likely to have reacted to form calcite under carbonation, and/or to form gypsum, the latter being the reaction product of CH with AMD. Furthermore, ettringite, which is an opportunity phase that is usually present in concrete, was generally not found. These changes in the matrix products of concrete are important indications of the chemical reactions that could have led to the observed disintegration of the concrete. Again, no traces of rust products, such as goethite, ferrihydrite or hematite were detected, which also excludes the possible presence of pyrite oxidation as participant in attacking the cementitious system.

**Figure 3** Photomicrographs of thin sections showing sulphate-bearing minerals under plane polarised light: (a) opaque minerals, (b) a close-up of (a) highlighting disseminated pyrites in an opaque particle under cross-polarised light

**Figure 4** Photomicrographs of thin sections showing: (a) microcracks and infilling under plane polarised light, (b) a close-up of (a) highlighting microcracking running through or over a sutured quartz particle in cross-polarised light

**Figure 5** Photomicrographs of thin sections showing: (a) map patterns and parallel microcracks viewed under plane polarised light, (b) a close-up highlighting multidirectional map-cracking and infilling of cracks under cross-polarised light (mcr – map cracks, pcr – parallel cracks)

Ferrihydrite (Fe(OH)₃) or hematite (½Fe₂O₃), which rules out possible involvement of pyrite oxidation in the attack.

**X-ray diffraction (XRD)**

Samples broken off from the mortar matrix of disintegrated concrete were oven-dried at 50°C for seven days, then ground with mortar and pestle into fine powder, passing a 90 μm sieve. The powder samples were back-loaded into the sample holders and a 90 μm sieve. The powder samples were back-loaded into the sample holders and analysed. A BRUKER D8 Advance Powder X-ray diffractometer was used with copper radiation, CuKα (wavelength λ = 1.5418 Å) over an angular rotation (2θ) of 0° to 90°. The XRD analysis determined the final reaction products and phases predominantly present in the cementitious system. Figure 8 shows the XRD patterns for the disintegrated concrete samples. The samples comprised phases that were identified to be quartz, gypsum and calcite. In both spectra shown, it is interesting to note that no portlandite was found among the phases identified. This implies that this phase is likely to have reacted to form calcite under carbonation, and/or to form gypsum, the latter being the reaction product of CH with AMD. Furthermore, ettringite, which is an opportunistic phase that is usually present in concrete, was generally not found. These changes in the matrix products of concrete are important indications of the chemical reactions that could have led to the observed disintegration of the concrete. Again, no traces of rust products, such as goethite, ferrihydrite or hematite were detected, which also excludes the possible presence of pyrite oxidation as participant in attacking the cementitious system.
Discussion of the attack process

The AMD was shown to be of low pH of 3.0 and high sulphate concentration of 5 200 ppm, as given in Table 1. Accordingly, sulphate attack or acid attack processes on the concrete would be likely, as discussed earlier. This high sulphate concentration of 5 200 ppm of the AMD falls under severe classification (Ekolu & Azene 2012; Earlie & Callaghan 1988). In the XRD analysis, there was no evidence of CH being present in the concrete. In a normal non-deteriorated concrete matrix, CH is plentiful and is responsible for the high alkalinity of concrete, usually with pH ≥ 12.6.

Upon exposure of the concrete to the low pH AMD, the CH must have undergone reactions leading to gypsum formation, while some calcium ions may have been leached out of the concrete along with sodium and potassium alkalis, given the dynamic exposure conditions. Considering that the AMD on site was flowing at a substantial rate, there was continuous replenishment of fresh ions to maintain the attack on concrete. Evidence from XRD analysis corroborates

![Figure 6a SEM image of the deteriorated concrete sample](image)

![Figure 6b EDS spectra of various phases labeled in Figure 6a](image)
the observations from petrography and SEM/EDX examination, singling out gypsum as the sole reaction product found in the disintegrated concrete. In Figure 1, gypsum can be seen as the white powdery substance coating the surface of exfoliated concrete. It may also be noted that the reaction product was concentrated along large cracks, most likely leached from the concrete matrix by flowing water and deposited at the crack surface layers as water evaporated.

As discussed earlier, the acid-sulphate conditions from AMD could cause competing sulphate attack and acid attack mechanisms in concrete. However, the low concentration of iron in AMD implies that most sulphates would be in the form of sulphuric acid (Equation 1), which limits potential sulphate attack due to ferrous sulphate. Also, sulphate attack due to ferrous sulphate would result in the formation of iron (ii) hydroxide, in accordance with Equation 6. However, no ferrous hydroxide was detected through analytical studies done using SEM/EDX or XRD.

\[
\text{Ca(OH)}_2 + \text{FeSO}_4 + 2\text{H}_2\text{O} \rightarrow \text{CaSO}_4 \cdot 2\text{H}_2\text{O} + \text{Fe(OH)}_2
\] (6)

While it remains possible that the two mechanisms of acid and sulphate attack could co-exist, considerations from the foregoing discussions, along with microanalytical studies, seem to support acid mechanism as the overriding attack process responsible for the observed degradation in the field concrete, following its exposure to AMD. These findings are also in agreement with the results of experimental work by Breitenbucher and Siebert (2008), whose investigation found acid attack mechanism to be dominant under acid-sulphate conditions. In addition, there was no observation of rust phases of goethite, ferrihydrite or hematite, indicating the absence of pyrite oxidation attack.

**CONCLUSIONS**

In the foregoing investigation, exfoliated field concrete that had been used in the construction of a weir was examined. The concrete weir was in contact with acid mine drainage at its source of decant in an abandoned coal mine. Over time, the concrete had severely disintegrated. Microanalytical studies were conducted on the deteriorated concrete by using optical microscopy, scanning electron microscopy and x-ray diffraction techniques. The main objective of the investigation was to determine the mechanism of attack that led to the concrete deterioration.

1. Petrographic examination found the concrete aggregates to be predominantly quartz particles and some opaque minerals. The quartz particles were found to have severely weathered, and in some cases there was evidence of mechanical fracturing. The opaque minerals were identified to be unreacted pyrites. Intense multi-directional microcracking was observed circumventing the quartz particles, by running along the interfacial transition zone and directly through the paste matrix. The microcracks were characterised by two patterns consisting of map-cracking and parallel oriented cracking.

2. The observed microcracks contained an infilling by a mineral phase, identified by scanning electron microscopy and x-ray diffraction to be gypsum. The fibrous layers of the infilling were visibly perpendicular to the crack walls. Besides that, the gypsum crystals also exhibited crumpled morphology, suggesting exertion of pressure by the crystals during expansion.

3. No rust products were found, which ruled out pyritic oxidation as a possible source of damage. Similarly, iron hydroxides were not detected through the analytical studies, which negates the possibility of ferrous sulphate attack. Gypsum was found to be the sole reaction product of attack. Acid attack appears to be the most likely damage mechanism responsible for the observed deterioration, with or without possible secondary contribution from sulphate attack.
ACKNOWLEDGEMENT

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Hydraulic model study of the blowback behaviour of the bottom outlet of the Berg River Dam, South Africa

A Bosman, G R Basson, D E Bosman

The Berg River Dam is equipped with the first multi-level draw-off environmental flood release outlet in South Africa and can release flows up to about 200 m$^3$/s. The outlet is controlled by a radial gate at the outlet end, and is protected by a vertical emergency gate near the inlet end. Commissioning tests of the emergency gate in 2008 found that large volumes of air were expelled, instead of the expected air entrainment into the air vent, designed to reduce expected negative pressures in the conduit during emergency gate closure.

This paper describes the testing of a 1:14 physical model representing the outlet works of the Berg River Dam to determine the reasons for the unexpected release of air from the outlet work’s air vent, as observed in the field during the commissioning tests of the emergency gate in the outlet conduit.

Simulations of continuous gate closure on the as-built physical model of the Berg River Dam outlet showed predominant inflow of air into the air vent during emergency gate closure, with intermittent short duration high-speed air releases during the stages of emergency gate openings between 37% and 25% open. The problem was determined to be one of intermittent air blowback from the outlet conduit via the air vent during the latter stage, rather than continuous air release for all stages of the gate opening operation. The cause of the blowback was found to be the constriction of flow due to a reduction in the conduit cross-section at the radial gate chamber located at the downstream end of the outlet conduit.

INTRODUCTION

Berg River Dam

The Berg River Dam (Figure 1) is located 6 km west of Franschhoek, and the supplement scheme is located approximately 10 km downstream of the dam. The dam is a concrete-faced rockfill embankment, approximately 65 m high and 990 m wide, and has a base width of 220 m. It has a gross storage capacity of 130 million m$^3$ (TCTA 2008). It is the first of its kind in South Africa, comprising structures that permit the release of both low and high discharge structure

Radial gate chamber

Bottom outlet conduit (5.5 m diameter)

Intake tower with emergency gate and air vent shaft

Figure 1 Berg River Dam wall with location of bottom outlet (Google Earth photograph)
environmental flood releases, the latter up to 200 m$^3$/s. The outflow for the environmental flood release is controlled by a radial gate at the end of the outlet conduit and is protected by a vertical emergency gate near the inlet of the conduit (Figure 2).

**Background to the problem**

A trial closure of the emergency gate in the outlet conduit of the Berg River Dam was undertaken by the TCTA on 12 June 2008, as shown in Figure 3. During the commissioning test the water level in the dam was 237.5 masl, the discharge through the conduit was 201 m$^3$/s as measured in the field, and the emergency gate in the conduit was closed from 100% open to 0% in 20 minutes (Shand 2008). The outlet conduit design included an air vent downstream of the emergency gate with the purpose to introduce air downstream of the gate to counteract the negative pressures that were expected in the conduit during

**Figure 2** (a) Cross-section of Berg River Dam intake tower and (b) complete outlet structure (adapted from TCTA 2003)

**Figure 3** Commissioning test of the Berg River Dam (Photo: Civil Engineering, August 2008, p 49)
emergency gate operations. Contrary to the theoretical design, the field measurement of air velocities in the air shaft indicated that, while the emergency gate was closing, very large volumes of air were being released from the 1.8 m² air shaft, commencing when the gate was about 30% closed (Figure 4). The latter observed air release was so severe that the heavy steel grating cover on the inlet end of the air vent shaft was blown off, which nearly caused serious injury to personnel who were measuring air velocities there.

As part of a South African Water Research Commission project (WRC 2012), co-funded by the TCTA, the Stellenbosch University constructed a 1:14.066 (undistorted scale) physical model of the relevant bottom outlet works of the Berg River Dam to evaluate the two-phase flow phenomenon in the outlet works.

Objectives of the model study
The objectives of the model study were as follows:
1. To determine reasons for the release of large volumes of air flow in the air vent, as observed during the commissioning closure test of the emergency gate on 12 June 2008.
2. To provide a solution to mitigate the observed excessive air release from the air vent, since air release of the intensity as was observed in the commissioning test may cause structural damage, as well as injury to personnel.

LITERATURE

Air blowback phenomenon
Prototype cases in which air blowback (large air pockets moving against flow) occurred was investigated by Sailer (Falvey 1980). Figure 5 indicates the air reverse flow region. The five prototype structures that experienced air blowbacks are indicated by a plus (+). Two of these blowback cases lay within the blowback zone at design discharge, i.e. valve openings at 100% open. The other three cases had to pass through the blowback zone when the flow is reduced from the design discharge, which means that these three cases would experience blowback at valve openings smaller than 100%, since with smaller valve openings these three “+”-plot locations would move to the left on the graph until they cross the line marked “Limit for air pocket/slug movement” (Falvey 1980).

The literature review indicated that explosive blowback incidents occurred on numerous overseas high-head conduit schemes. Lowe (1944) described the air blowback phenomenon which occurred on the Owyhee Dam in Oregon, USA. The long-section of the dam showed that the horizontal conduit ends in a stilling basin. Wave action was experienced in the stilling basin, which sealed the exit of the outlet conduit for short periods. The intake air was compressed when the conduit outlet was choked by the waves. This resulted in the compressed air being released both downstream and upstream – the latter was called the blowback of the compressed air. There are similarities between this case study and the Berg River Dam in the basic mechanism that causes air blowback (the Berg River Dam has a constriction for the radial gate chamber at the outlet end of the conduit, since the ceiling of the conduit slopes downwards in order to house the radial gate (see Figure 7)).

The US Army Corps of Engineering (USACE 1980) developed a guideline manual for the design of tunnel-conduit type outlets, based on model tests and prototype data. This manual recommends that the elevation of the hydraulic grade line (pressure gradient line or water surface) at the conduit exit should be lower than the soffit of the conduit. The flow inside the conduit should therefore be unrestricted to ensure free-flow conditions at the conduit exit. To prevent blowback it is further recommended that flow in a high-head outlet should flow partially full and should never be constricted by any structure or mechanism further downstream in the conduit.
MODEL OF THE BERG RIVER DAM

General
A physical model of the Berg River Dam was constructed to simulate the closure of the emergency gate under similar water levels and intake gate configurations as at the time of the commissioning test in 2008 (Vos 2011). Figure 6 shows a photograph of the model and Figure 7 shows a line sketch of the model layout.

Model scale
The model of the Berg River Dam was designed to a 1:14.066 undistorted scale. The odd scale of the model was determined by the inside diameter of the available perspex pipe that was used to model the outlet conduit. Since it was considered that gravitational and inertial forces would dominate in the model, the Froude scale (Webber 1971) was adopted, which implies an undistorted model.

Measuring equipment and techniques

Pressure measurements
Eight S-10 type pressure transducers were used to measure the static pressures and pressure fluctuations in the water tank, water shaft and outlet conduit. The locations of the transducers are shown in Figure 8. The transducers had a sample frequency of 20 Hz, with an accuracy of 0.5% over the total pressure range.

Air velocity measurements and direction indicator
The air velocity in the air vent was measured by means of a Lutron hot-wire anemometer, from which the air discharge was calculated. It had a ± 5% accuracy over the total measurement range. The logger sampled at a frequency of 0.8 Hz.

A wind direction indicator was installed in the top section of the air vent, and the configuration was such that it had negligible influence on the air velocity within the air vent. This apparatus had a mechanical flap that would be in a horizontal position if no wind was blowing in the air vent, and would be directed in the direction of the wind if air was entrained into or released from the air vent. The sign convention assumed for the air velocity in the air vent was positive for air entrained into the conduit and negative for air released from the air vent. The wind direction indicator was not accurate for air velocities less than 0.5 m/s (model).

EXPERIMENTAL PROCEDURE

Experimental controls
An electric motor was used to close the emergency gate in order to obtain the required gate closure time.

The required water flow was obtained by keeping the water level in the tank constant at the water level under evaluation.

The tests were performed with the radial gate fully open, as the emergency gate was designed to be used when the radial gate fails to close.

Table 1 Emergency gate closure times

<table>
<thead>
<tr>
<th>Gate closure time number</th>
<th>Emergency gate closure time (prototype) %0 to 0% gate opening</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20 min</td>
<td>A 20 min emergency gate closing time used during the commissioning test of 2008 (Basson 2011).</td>
</tr>
<tr>
<td>2</td>
<td>12 min</td>
<td>Designed emergency gate closure time (12 min) according to the Berg River Dam design report (TCTA 2003).</td>
</tr>
<tr>
<td>3</td>
<td>6 min</td>
<td>Time was chosen to investigate the flow conditions for a shorter gate closure time.</td>
</tr>
<tr>
<td>4</td>
<td>30 min</td>
<td>Time was chosen to investigate the flow conditions for a longer gate closure time.</td>
</tr>
</tbody>
</table>
**Transmit gate opening simulations**
Tests were conducted on the model with its configuration according to the as-built drawings. The air flow in the air vent, water discharge and pressures in the conduit were measured. These tests were run at four different gate closure times (continuous gate closure) as depicted in Table 1.

The transient gate closing simulations were also conducted at three different water levels for each of the four above-mentioned gate closure times. The water levels are summarised in Table 2.

The commissioning water level corresponds to the water level that was measured in the field during the commissioning test of the Berg River Dam in June 2008.

The level where vortices started to form in the water tank (vortex water level) was determined to be 227.12 masl for the middle gates on the intake tower. The lower water level mentioned in Table 2 was taken halfway between the commission test water level and the vortex water level.

**EVALUATION AND DISCUSSION OF RESULTS**

**Validation of the Berg River Dam model**
The field measurements at the dam were done intermittently with a hand-held anemometer which recorded only velocity and not direction. The observer commented that at about 40% gate opening the air flow was surging at 10 cycles per minute. Since the air direction was not recorded continuously, it could be at this stage that intermittent in- and outflow occurred in the air vent. The field recordings were therefore not ideal, and more rigorous recordings were unfortunately not possible for inclusion in this study, since TCTA and BRC decided that commissioning tests should not be repeated. The reason for this decision was because the tests could expose the dam and pertinent structures to unnecessary damaging forces; in addition it must be borne in mind that the emergency gate will probably never be used again in the lifetime of the dam.

**Tests performed on as-built outlet conduit model**

**Possible vortex air entrainment upstream of the emergency gate**
In a report on the commissioning test on the Berg River Dam (Shand 2008), vortex formation is cited as the most likely cause of the air reversal flow.

To test for vortex formation, the water in the wet well was stirred while the emergency gate was 100% open and the water level was kept at the commissioning water level. However, no air entraining vortices could be induced in the wet well at this water level.

The water level was lowered until air entraining vortices formed in the wet well. The highest level at which vortices formed was found to be at 227.12 masl.

At this level all the air entrained into the outlet through the vortices travelled down the conduit and was released at the downstream end of the conduit. Vortex formation was therefore not the cause of the air reversal flow in the air vent, as observed in the field during the emergency valve commissioning tests.

<table>
<thead>
<tr>
<th>Water level name</th>
<th>Prototype (masl)</th>
<th>Model (m) (Datum = bottom of outlet conduit at upstream end)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full supply water level (level of dam spillway)</td>
<td>250.0</td>
<td>3.8</td>
</tr>
<tr>
<td>Commissioning test water level (June 2008)</td>
<td>237.5</td>
<td>2.9</td>
</tr>
<tr>
<td>Lower water level</td>
<td>232.32</td>
<td>2.5</td>
</tr>
</tbody>
</table>

![Figure 8 Location of pressure transducers (shown in elevation)](image)

![Figure 9 Air velocity in air vent vs gate opening for different gate closure times (commissioning water level, transient gate, as-built, velocities converted to prototype values)](image)
Tests to search for other causes of reverse air flow in air vent

Impact of gate closure time

It was observed that air was released from the air vent for the transient gate simulations (emergency gate closing continuously). The effect of the different gate closure times/rates on the air velocity in the air vent for the commissioning water level is illustrated in Figure 9. Air was released for gate openings between 37% and 32%, irrespective of the specific gate closure time under evaluation. For gate openings outside the latter gate opening stage, air was entrained into the air vent for all gate closure times investigated.

Figure 10 illustrates the effect of the various gate closure times on the pressures just upstream of the radial gate chamber (pressure transducer number 7) for the commissioning water level conducted on the as-built outlet conduit.

A steep drop in pressure occurred for gate openings between 37% and 33%, irrespective of the gate closure time. The steep drop in pressure occurred at the same time as when air blowback occurred in the air vent.

It was concluded that the air velocity in the air vent was independent of the time of closure of the emergency gate.

Conclusions from tests on as-built model outlet

From the tests performed on the as-built model of the Berg River Dam outlet works it was concluded that the air flow in the air vent was predominantly into the conduit (downwards) during emergency gate closures. However, rapid reverse air flow occurred between gate openings of 37% and 25%.

Air was essentially drawn into the conduit through the air vent and was dragged downstream either insufflated in the flow or above the water. At the downstream end of the conduit the outflow of air was restricted by the tapered soffit section of the radial gate chamber. An unstable hydraulic jump formed in the outlet conduit as a result of the transition from the pressurised flow to free surface flow. Entrapment of air occurred between the unstable hydraulic jump and slanting roof of the radial gate section. The entrapped air in the conduit was pressurised due to the upstream and downstream water seals (see Figure 11(a)).

Between 37% and 32% gate openings the jump intermittently broke contact with the roof to the conduit (due to the reduced discharged and increasing air pressure from behind the jump), and pressurised air in the conduit from downstream of the point where

Figure 10 Effect of gate closure time on pressure at end (location 7) of conduit (commissioning water level – prototype values)

Figure 11 Reason for air blow-back: (a) “trapped” air, and (b) air released via air vent
the jump broke contact with the roof was released back upstream and out the air vent (see Figure 11(b)). The air velocity in the air vent was the highest over this period and changed direction rapidly as pulses of air were expelled while the unstable hydraulic jump was moving downstream. The air flow problem of the Berg River Dam was therefore determined to be one of intermittent air blowback at a specific stage of the gate opening.

Based on the above, the most probable reason for blowback is the constriction of flow at the radial gate chamber.

Tests performed on modified model configurations

Modified model configurations
In order to confirm the cause of the air blowback and to investigate possible solutions, a number of tests were performed on modified outlet model configurations. Although the expected cause of the blowback was considered to be the flow constriction caused by the radial gate chamber, the ski-jump and the second bend in the conduit were also identified as components possibly having an effect on the flow. Figure 12 shows the configurations of the modified model configurations tested.

Removal of ski-jump and second bend
It was found that the removal of the ski-jump and the second bend had very little effect on the flow in the conduit. The results of the tests on Modifications 1, 2 and 3 are presented in Figure 12 and show very similar trends to those on the as-built outlet.

Removal of radial gate chamber
Removal of the radial gate chamber was found to have a significant effect on the flow patterns in the conduit, and eliminated the air blowback.

The air velocity and pressures versus gate opening graphs for the five modifications are shown in Figure 13 and Figure 14 respectively.

Without the radial gate constriction (modification 4), the air velocity in the air vent increased with increasing gate opening, no backflow occurred and air flow remained positive (i.e. air entrainment prevailed) for all stages of gate opening. The pressures observed for modification 4 (at location 7) in the conduit are also much lower with less variation/fluctuation than those experienced on the other modifications, which included the radial gate chamber. This is because free-surface flow occurred throughout the conduit for all gate openings, instead of partially
pressurised flow which occurred when the outlet was constricted (as-built outlet and modifications 1, 2, 3 and 5).

Figure 15 shows a schematic comparison between the observed flow in the as-built outlet and the modified outlet with the aid of sketches at gate openings of 100% and 37%.

Provision of additional air vent at the radial gate constriction

Once it was confirmed that the radial gate constriction was the cause for air blowback, it was suggested that an additional air vent placed immediately upstream of the radial gate constriction may solve/reduce the air blowback problem. If effective, this would possibly be a practical retro-fit solution for the existing Berg River Dam.

An additional 450 mm (prototype) diameter air vent was fitted directly onto the as-built conduit. It was found that the additional air vent was ineffective in reducing the blowback effect, and the air flow and pressure results were similar to those of the as-built tests and other modifications which included the radial gate constriction as seen in Figure 13 and Figure 14 respectively. During the test water was trapped in the additional air vent and blocked the free release of air. For the critical gate openings of 37% to 25% (while air blowback was experienced in the upstream air vent), an air-water mixture was intermittently expelled from the extra air vent in an explosive fountain display, shooting water approximately 30 m (prototype) into the air above the top of the vent.

Conclusions from tests on modified model configurations

The test results indicated that the cause of the air blowback is the constriction at the radial gate chamber at the downstream end of the conduit. When the flow is not constricted there, free-surface flow occurred throughout the conduit for all openings of the emergency gate, and the air blowback in the air vent was eliminated.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The main conclusion drawn from the physical model tests on the air flow problem in the air vent of the Berg River Dam outlet conduit is that air blowback occurred at a specific gate opening range (25% to 37%) of the emergency gate due to the constriction in the outlet conduit at its downstream end. During the stages of valve opening outside the latter opening range only air inflow into

the air vent was observed, which is contrary to what was subjectively observed during the field observations during commissioning testing of the emergency valve.

During the stages of air inflow into the air vent, air was essentially drawn into the conduit through the air vent due to high water velocity in the conduit, which dragged

Figure 13 Air velocity in air vent versus gate opening for modified outlet conduit

![Figure 13 Air velocity in air vent versus gate opening for modified outlet conduit](image)

Figure 14 Pressure upstream of radial gate chamber (at location 7) versus gate opening for five modifications to the outlet conduit

![Figure 14 Pressure upstream of radial gate chamber (at location 7) versus gate opening for five modifications to the outlet conduit](image)
air downstream by insufflation into the water and above the water surface by shear stress of the water flow at the water-air interface. At the downstream end of the conduit the outflow of air was constricted by the tapered section of the radial gate chamber (ceiling of conduit sloping downwards), resulting in pressurisation of the air in the conduit. This pressure caused air blowback through the air vent when the upstream hydraulic jump broke contact with the roof of the conduit.

Tests performed with the radial gate constriction removed (modification 4) confirmed that it had been the cause of the blowback experienced in the latter tests. These tests showed free-surface flow throughout the valve closure operation of the emergency gate and no reverse air flow/blowback occurred – only air entrainment into the air vent occurred. Tests on the other modified model configurations (modification 1, 2 and 3) confirmed that removal of the ski-jump and the second bend (8°) had little effect on the results. An additional 450 mm air vent was installed directly upstream of the constriction (modification 4). Test results on this modification indicated it to be ineffective in reducing the air blowback.

Air entrainment due to vortices in the water did not occur in the wet well (even with manual stirring of the water in the wet well) for tests performed at the commissioning water level. Air blowback did not occur at the critical reservoir level at which air commenced to be entrained via a vortex in the wet well water, i.e. at a water level in the wet well of 227.12 masl.

It was found that the air velocity in the air vent was independent of the gate closure time, but increased with an increase in water head.

**Recommendations**

Based on the findings of this physical model study on the air flow problem in the air vent of the Berg River Dam’s conduit outlet, certain recommendations can be presented.

To prevent air blowback in the air vents of high-head outlets such as that of the Berg River Dam, it is recommended that free water flow should always be ensured for all conditions over the entire length of the outlet conduit, and water and air flow should never be constricted by any structure or mechanism at the downstream end of the conduit, especially not at the soffit of the conduit.

For an outlet design such as that of the Berg River Dam (even with the constriction at the radial gate chamber removed) a potential air blowback problem could occur if the radial gate should fail in a partially closed position. A possible solution in such a case would be a dual radial gate system in which each gate has the capacity for the full design discharge (Figure 16). Under normal operation one gate could be used while the other gate remains closed. In the case of failure of a gate in a partially closed position, the other gate can be fully opened to allow un-constricted flow before the emergency gate is closed.

**GUIDELINES FOR THE DESIGN OF FUTURE BOTTOM OUTLETS**

The following design guidelines should be adhered to in future designs to prevent air blowback:

- Bottom outlets should be designed to ensure free-surface flow conditions under all probable flow conditions, and the formation of hydraulic jumps in the conduit should be avoided (USACE 1997).
- Air entrainment at all changes in cross section should be avoided by matching tunnel ceiling heights rather than invert (USACE 1997).
- The upstream movement of air which can cause possible blowback problems due to buoyancy forces should be avoided by keeping the slope of the outlet conduit as flat as possible (Falvey 1980).
- The outlet conduit downstream of the emergency gate should be as short as possible, and straight.

![Figure 15 Sketches of flow patterns for as-built conduit and modified conduit with radial gate and ski-jump removed for 100% and 37% gate opening](image)

![Figure 16 Possible radial gate configuration at the downstream end of an outlet conduit to prevent air blowback in the air vent of the conduit](image)
The crest height of a ski-jump should not be so high that it could cause submergence of the conduit under low-flow conditions.

The flow in an outlet conduit should not be restricted for any foreseeable flow condition. The case of a radial gate failing in a partially closed position is a particular scenario that would cause a constriction which may cause a severe restriction of the flow, possibly leading to dangerous air blowback during emergency gate closures. (A possible configuration to prevent blowback in this case is discussed in the section called “Tests performed on as-built outlet conduit model”).

Large-scale hydraulic models (greater than 1:20) should be used in the design process for partially full-flow outlet conduits to minimise scale effects and to readily observe the detailed flow behaviour (Speerli 1999; WRC 2012).

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REFERENCES


Electricity generation as a beneficial post-closure land use option for a dormant tailings storage facility

S J van Eeden, S W Jacobsz, M Rust, E Rust

Dormant tailings storage facilities (TSFs) have negative effects on their surrounding environments and communities. This study is aimed at determining the financial and practical feasibility of generating energy from the ERGO TSF site, near Brakpan, Johannesburg, as a beneficial post-closure land use option. The beneficial post-closure options investigated were rainwater harvesting and electricity generation from wind power, photo voltaic solar power and a pump storage system constructed on the TSF. Wind power generation and rainwater harvesting from the site were found to be unviable.

It was found that a 470 MW (peak capacity) solar photo voltaic plant on top of the ERGO TSF will provide the best solution, both from a practical and financial point of view, yielding a potential internal rate of return (IRR) of 10.7% over 50 years. A pump storage system yields a maximum IRR of 10.3%, but with a substantially smaller generation capacity of approximately 80 MW.

INTRODUCTION
As a result of mining that has taken place over the last one and a half centuries in South Africa, many towns and cities have developed around mining hubs, with the most significant of these being the city of Johannesburg. Over the years, residential areas have grown around these mine sites, even well after cessation of mining activities. Mining left a legacy of derelict mining infrastructure, such as dormant mine shafts, sterilised land and abandoned tailings storage facilities (TSFs), with negative impacts on the surrounding environment and communities. This legacy is probably unique in scale compared to any country in the world. In an age where great emphasis is placed on all industries to become more environmentally conscious, mines in South Africa have a challenge to undo the negative legacy from the past. Environmental laws and regulations are becoming increasingly stringent, with the intention to mitigate or minimise the environmental impacts as far as possible.

A lack of funds and commitment from mine owners and regulators have in the past resulted in many un-rehabilitated decommissioned mines posing negative environmental impacts, including potential health hazards, on the surrounding communities. According to the Chamber of Mines of South Africa (1996), preparation for eventual closure should start as soon as possible, and the costs of closure should be regarded as an integral part of the cost of production. However, this has not often been the case. Due to the unpredictability of liability, risk and costs associated with mine closure, South African mining companies are reluctant to spend the necessary time and money on closure of facilities after years of beneficial use. Therefore, an alternative method of funding appropriate rehabilitation and closure of such facilities is required. The ideal first step in the implementation of a sustainable closure plan is to find a beneficial post-closure land use option.

South Africa currently faces an electricity shortage, and consumers regularly experience controlled load-shedding, i.e. managed power interruptions to prevent overload and subsequent collapse of the electricity supply and distribution network. South Africa is highly reliant on coal-fired power stations for the bulk of its electricity needs. These power stations have detrimental effects on the environment, due to high carbon emissions and the impact of large opencast coal mines. A global shift towards renewable energy, in combination with South Africa’s current energy shortage, has forced the National Energy Regulator of South Africa to encourage greener alternatives.

This study examined the following four beneficial land use alternatives in an attempt to identify potentially beneficial post-closure land use options for dormant TSFs:
- Electricity from wind power generation
- Harvesting of rainwater from the TSF catchment for raw water supply
- Electricity from photo voltaic solar power generation

Electricity from pump storage system power generation.

With the exception of the rainwater harvesting alternative, all options are focused on the generation of electricity. Dormant mine sites are often more favourable locations for the generation of electricity than green-field sites for the reasons listed below (Whitbread-Abrutat & Coppin 2011):

- Large tracts of derelict land have limited post-closure land use potential. The implementation of an electricity generation system is therefore likely to have little further environmental impact and opposition. In fact, the application of renewable energy infrastructure might mitigate some environmental impacts.
- Existing infrastructure, such as electricity transmission systems and roads, are usually in close proximity; therefore less capital expenditure is required.
- Obtaining concessions to utilise the land is more readily achieved, because this land is not normally in high demand, as is the case with green-field sites.
- TSFs are typically denuded and exposed, making them suitable for solar or wind power plants.

This study was aimed at finding the most beneficial land use option from the list above by examining the financial and practical feasibility of the alternatives. The rainwater harvesting and wind power generation options referred to above were abandoned early on in the study, while the two more promising alternatives, photo voltaic solar power generation and the development of pump storage systems, were investigated in more detail.

**HYPOTHETICAL FEASIBILITY ANALYSIS**

This study is specifically focused on mitigating the legacy of dormant TSFs. During the mining operation, a TSF is used for the hydraulic placement of finely ground rock waste. The construction method most often employed in South Africa involves deposition along the perimeter of the dam, where the coarsest material settles out first. The remainder of the slurry stream (water with suspended solids) flows to the middle of the TSF where the finest material settles out and the water is contained in the TSF pond. The surface geometry is usually slightly concave, enabling water storage. These facilities are often able to contain a significant amount of water, with the largest potentially storing several million cubic metres.

This study was based on the ERGO TSF, situated southeast of Johannesburg (see Figure 1). The ERGO TSF currently occupies the largest footprint area of all gold TSFs in the world. The final footprint area of the facility will have a combined area of 1 500 ha and is illustrated in Figure 2 (Kleynhans 2014). The ERGO TSF is used for deposition of reprocessed tailings from other TSFs scattered over the Johannesburg area. It is still operational and provides storage for between three and five million cubic metres of process water. A barge pump is currently being used to discharge water from the TSF, as the gravity penstock decant system was decommissioned some years ago. The gold content remaining in the ERGO tailings is too low to justify further processing, and hence this facility will eventually reach closure, currently envisaged for around 2050. The final maximum wall height is planned to reach 140 m above natural ground level.

This study site was chosen to maximise the benefits of economy of scale, as there are likely to be advantages in developing larger sites. It was the intention to define potential electricity generation alternatives in sufficient detail to allow a preliminary construction cost to be estimated. This, in combination with the expected income to be generated from the sale of electricity over the life of the facility, was used to assess the financial feasibility in terms of an anticipated internal rate of return (IRR) for the system.

The following assumptions were made regarding the power generation options investigated in the study:

- According to Kleynhans (2014), the ERGO TSF is only due for decommissioning...
and closure in 2050, hence a hypothetical closure scenario was created. It was assumed that the ERGO TSF is decommissioned with immediate effect, having a height of 140 m above the surrounding ground level.

- Hypothetical electricity generation was assumed to commence in January 2015, i.e. the 2015 ESKOM electricity tariff structure and associated increases were applied as discussed below to assess the system’s financial feasibility.

- Historical weather data over the last 50 years was used in this study.

- All options were assessed over a 50-year operational life cycle. A 50-year life cycle is often adopted for hydropower projects, as these are usually associated with substantial capital investment (BHA 2005; Breeze 2005).

- Financial feasibility was assessed based on the internal rate of return (IRR), defined as the discount rate at which the net present value of the projected cash flow (sum of all expenses and incomes) is zero. The IRR was selected as feasibility criterion instead of alternatives such as the net present value because, during the course of the study, it became apparent that the optimum scale for a photovoltaic solar plant and a pump storage system differed by an order of magnitude.

- Eskom Megaflex tariffs (Eskom 2014) were used to estimate income. It was assumed that electricity produced by the hypothetical plant would be sold at the so-called ‘Megaflex tariffs’. The Megaflex tariff structure is shown in Figure 3 and differentiates between seasonal and time-of-day use of electricity (Eskom 2014). It represents 25 ‘peak’ hours, 62 ‘standard’ hours and 81 ‘off-peak’ hours per week. The electricity hypothetically generated from the alternatives investigated (in units of kWh) was multiplied with the tariff applicable at the time of generation to determine the potential income. In the case of the pump storage system, the amount of electricity consumed was taken as the calculated pumping cost.

- An increase in the above-mentioned tariffs of 7.4% per annum to the end of the life of the electricity plant was assumed, reflecting the envisaged tariff increase for the next five years, as approved by the National Energy Regulator of South Africa.

**ASSESSMENT OF WIND POWER AND RAINWATER HARVESTING OPTIONS**

As mentioned earlier, the wind electricity generation and rainwater harvesting options were abandoned at an early stage of the study.

Wind records from Oliver Tambo International Airport, located not far from the ERGO TSF, date back to 1958 and show an average wind speed of only 3.3 m/s. Electricity from wind power can be profitably generated when the average wind speed exceeds 5.0 m/s for an onshore site (Breeze 2005). This indicates that the potential for wind power generation at the TSF may be insufficient. There are, however, two more aspects to consider:

- The wind speed data was recorded at discrete times during each day on record and do not reflect the continuously recorded wind speed. However, wind speed models at 100 m elevation for South Africa also show the Gauteng area to have some of the lowest potential for wind power generation in the country (IRENA 2015).

- Blight (2007) reported amplification of wind speeds to occur over the crest of a TSF, due to the TSF causing an obstruction in the wind flow path. Blight’s models suggest that the amplification factor applicable at the crest of the final 140 m high ERGO TSF could cause wind velocities of the order of about three times the wind speed typically recorded with an anemometer at a weather station (typically installed at 10 m above ground level). Currently, however, there is not enough data available to enable the financial viability of wind power harvesting from high TSFs. Following Breeze (2005), it is recommended that a study be carried out to measure continuous wind speeds on a number of high TSFs for a period of one year to further evaluate the available wind resource.

The rainwater harvesting option was also abandoned at an early stage of the study, as calculations indicated that it is not financially feasible. If it is desired to supply potable water from such a system, the dam would have to be lined. The cost of a suitable lining is prohibitive. For example, the cost of a liner comprising a 1.5 mm thick HDPE membrane with nominal base preparation is estimated to cost in excess of R165 per square metre. Lining the current area of 856 ha will therefore cost in excess of R1.4 billion. The cost of importing raw water from the Lesotho Highlands Water Project (LHWP) was recently reported as only R2.32 per cubic metre (DWS 2014). Should the water from the rainwater harvesting scheme be sold at this rate, an IRR of only 2.4% would be realised over 50 years.

**ASSESSING THE FEASIBILITY OF SOLAR PHOTO VOLTAIC ELECTRICITY GENERATION**

**Photo voltaic electricity**

Solar cells or photo voltaic (PV) cells convert sunlight to electricity and consist of a sandwich of several layers of material. The semiconductor most often used in solar cells
is silicon, one of the most abundant elements on earth, the main constituent of silica sand (Zweibel 1990). However, although silicon is widely available and therefore cheap, the production of pure silicon used in solar cells is energy-intensive, requiring up to 90 kWh for every kilogram of silicon produced. Therefore, a solar cell typically has to be operational for two years to produce the electricity required to manufacture the cell itself (Breeze 2005).PV cells produce direct current (DC) electricity which requires an inverter to convert it to alternating current (AC) before it can be fed into the electricity grid.

The efficiency of a solar system is measured as light-to-electricity conversion efficiency (Zweibel 1990). The efficiency of commercially available crystalline silicon cells is in the order of 15% (Yingli Solar 2012). The technology behind PV cells has improved significantly since its inception and is still improving. This results in a decrease in cost over time, rendering PV electricity generation systems increasingly cost-competitive. Between 1973 and 1990 the average global cost of PV has reduced about 20 times, and will likely reduce further in the future (Zweibel 1990).

Silicon technology has now been around for a considerable time, and its reliability, long-term operation, plant lifetime and operation are well understood. The power generation potential of any site on earth therefore can be determined if its solar radiation resource is known (Breeze 2005).

Environmentally, solar PV electricity is regarded as one of the most benign renewable energies. It does not emit any atmospheric emissions during operation and does not create any noise. Solar plants are easily erected and can be constructed in a short space of time if the components are readily available (Breeze 2005).

Single solar cells are arranged into modules. These modules (solar panels) are then grouped into arrays, which make them easier to install. Grouping solar cells together can create a significant amount of power. The following factors influence the output of a solar plant:

- The quality and angle of sunlight
- The orientation of the panels
- The configuration and number of solar panels (a function of the area available)
- The efficiency of the panels
- The use of a tracking system allowing the panels to re-orientate themselves to follow the sun (fixed, single or double-axis tracking)
- The degradation rate of the plant (i.e. performance reduction over time)
- The use of sunlight concentration by means of mirror systems or not.

### Determining the available solar resource

Masters (2004) presented a series of equations enabling the potential solar resource to be quantified. The equations allow a sun-path diagram to be calculated, describing the position of the sun at any time of the year (see Figure 4).

With the position of the sun known, the incoming solar radiation flux (kW/m²) can be determined. This includes direct beam and diffused solar radiation. Radiation reflected from surrounding surfaces was assumed to be negligible.

The radiation on a horizontal surface was calculated and compared to data from NASA (2014) to validate the calculated values. The NASA data also enabled cloud cover to be considered, based on data from the International Satellite Cloud Climatology Project (NASA 2014). The calculated data correlated well with that of NASA.

With the calculated radiation data verified and cloud cover taken into consideration, the electricity production from inclined solar panels could be determined as a function of their orientation. For cost reasons fixed panels were assumed, i.e. no tracking system.

The layout of panel arrays was determined to ensure full sunlight on the panels between 09:00 and 15:00 during the winter months. The shadows cast by adjacent panels on one another early in the morning and late afternoon were considered by conservatively assuming that, if the sun is at an altitude angle of lower than 23.6°, production would be zero.

The variation in the calculated solar flux is discussed in a later section below.

### The proposed system and its optimisation

The layout of the plant and the number of panels are dependent on the orientation of each panel, i.e. its direction and tilt angle.

This was optimised to provide the highest possible IRR. Although the results show that production is maximum when the panels face directly north, the IRR was found to be somewhat higher if the panels are turned 16.3° east-of-north. The reason for this is the current Eskom peak tariffs applicable between 07:00 and 10:00 during week days.

The optimisation of the tilt angle showed that the panels produce the maximum output when orientated at a tilt angle of 22.5° with the horizontal, i.e. towards the summer solstice position, which takes advantage of the long periods of exposure during the summer months. However, the highest IRR is generated when the panels are placed at a tilt angle of 27.6°, closer towards the winter solstice, taking advantage of the increased tariffs during the high-demand season applicable during the winter months.

The optimum solar panel orientation therefore comprises panels orientated 16.3° east-of-north and tilting at 27.6° relative to the horizontal. Each panel has dimensions of 1 650 mm by 990 mm. They are arranged in arrays of six panels high by 40 panels wide, 15 arrays per block section, with 535 block sections in total. This orientation allows for a total of 1.93 million panels to be placed on the 726 ha top surface area of the ERGO TSF at closure to produce a rated power output of 472 MWp (MWp refers to ‘Mega Watt peak’, the predicted peak power output).

Indicative cost estimates for this project were obtained from commercial suppliers of large-scale photo voltaic systems in South Africa. The cost of supply and installation of the complete system, including solar panels, mounting structures, direct current cables and inverters was estimated at approximately R22 per Watt peak of installed generation capacity. This rate was used as a first-order estimate for the construction cost of the
plant. This rate is applicable to large-scale projects and is not valid for costing small-scale projects, and is based on Yingli Solar’s YGE 60 Cell 40 mm Series 245 Wp Panels (Yingli Solar 2012) with an efficiency of 15%. A panel’s ‘watt peak’ output is a term used in the solar PV industry to standardise solar panels according to their performance under standard test conditions, as specified in IEC 60904-3 (IEC 2006). In this case a single panel (size 1 650 mm by 990 mm) with an efficiency of 15% produces an electrical output of 245 W when exposed to a global standard spectrum at 1 000 W/m² irradiance at 25°C cell temperature.

The system described here does not include battery storage, an item that requires a significant amount of maintenance. Once installed, the solar plant will require minimal maintenance. The system can in principle be controlled via a remote computer, and therefore no personnel are required on site (Breeze 2005). Occasional cleaning and servicing of panels might be required. The costs of these were deemed to be negligible compared to the overall cost of installation.

**Predicted daily electricity production**

The daily plant production will vary depending on the solar declination, the applicable cloud cover factor and the duration of exposure to sunlight. To illustrate the projected daily plant output, four dates were chosen, i.e. 21 December (summer solstice), 21 March (autumnal equinox), 21 June (winter solstice) and 21 September (spring equinox). The results presented apply to the first year of operation with no time-related deterioration considered.

Figure 5 illustrates the sun’s radiation flux per square metre of solar panel, calculated using the formulas by Masters (2004), i.e. the energy that could be captured with a hypothetical 100% efficient solar panel. The 21 December curve has the longest exposure time, corresponding to the longest day in Figure 4, but produces the lowest peak value. That is due to the tilt angle that was optimised to favour the winter solstice to benefit from the high-demand season rates applicable during winter. For the same reason, although the exposure time is shorter during the winter solstice, the production peak is higher than the summer solstice. It is also interesting to note that, during the middle of the day, around the time of the spring and autumnal equinoxes, the sun’s potential exceeds that of the standard test conditions (1 000 W/m²) under clear sky conditions. Hence, the output from a single solar panel would theoretically be higher than the nominal rated power output of 245 Wp assumed in the determination of the IRR.

The output of the entire plant (under ideal clear sky conditions, i.e. no cloud cover) is shown in Figure 6. It can be seen that the plant’s generation capacity can exceed 500 MW under ideal circumstances, although the rated power output is 472 MWp. This is of the same order of magnitude as the Topaz Solar Farm (currently the largest solar PV plant in the world), which has a rated power output of 550 MWp (PV Magazine 2014).

**Electricity production and associated income**

By integrating the area underneath the ‘output vs time’ curves in Figure 6, and allowing for the cloud cover factor (derived from the NASA data) and plant degradation, the expected daily production from the plant was calculated at an average of 2.6 GWh, reducing linearly over the 50-year life of the plant to 1.6 GWh. The following degradation
rates, expressed as percentage change in the rated output per annum, were assumed to account for the degradation of the solar panels’ performance over the life of the facility as per the manufacturer’s warrantee (Yingli Solar 2012):
- 0 to 10 years of operation: 0.8% per annum
- 10 to 25 years of operation: 0.7% per annum
- 25 to 50 years of operation: assumed an average of 0.7% per annum.

The daily production (kWh) was multiplied with the electricity tariffs applicable at the time of generation (as illustrated in Figure 3) to determine the expected daily income during the 50-year life. Table 1 summarises the details of the optimal solar plant layout that can be built on the ERGO TSF.

The proposed system in Table 1 compares well with the largest existing solar PV plant, the Topaz Solar Farm in California. The Topaz Solar Farm uses smaller and thin film technology, which is known to have a lower efficiency than crystalline silicone. The generation capacity is 550 MWp, and the annual electricity production is expected to be 1.096 TWh.

### Reliability and sensitivity analyses

A reliability analysis was conducted on the calculated electricity production data, by plotting distribution histograms and fitting normal distributions to the data, to assess the level of certainty that the solar plant will produce a certain amount of electricity (GWh) or income on any given day. Figure 7 presents daily generation capacity and associated profit against the associated level of confidence. It was found that the solar plant will produce 1.25 GWh/day, generating R500 000 (in 2014 monetary value, i.e. no inflation taken into account) with a confidence level of 90%.

The series of sensitivity analyses were conducted to determine the effect of the various assumptions made in this study on the calculated IRR. The following parameters were varied, each around their expected range, to determine the sensitivity of the IRR over a 50-year lifetime to each parameter, whilst keeping all other variables constant:
- Deterioration rate of solar panel efficiency after 25 years (assumed 0.7%/annum, varied between 0.6 and 1.5%/annum).
- Unit cost of construction (assumed R22/Wp, varied between R17 and R25/Wp).
- Surface area covered by solar plant (assumed 726 ha, varied to as low as 250 ha).

It was found that varying the reduction in solar panel performance stated above changed the IRR by between only 10.72% and 10.49%, and that it had a small effect on the plant’s total life production, predicted to vary between 38.0 TWh and 35.5 TWh depending on the rate of deterioration. Probable deviations in panel-efficient degradation over 25 years should therefore only have a minor effect on the IRR.

The IRR was found to be sensitive to a change in unit cost of construction (assumed to be R22/Wp), with an increase in IRR of up to 12.4% if the unit price is reduced to R17/Wp. This is positive for the future of solar technology, as technology often becomes cheaper over time.

Due to the constant unit price of R22/Wp used for calculation, the surface area available for construction does not affect the IRR, hence the plant can be constructed in stages and can be applied to smaller TSFs. Note that it was assumed that the plant would be sufficiently large for the economy of scale unit construction cost of R22/Wp to apply.

### Table 1 Summary of optimal solar plant for the ERGO TSF

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of panels in plant</td>
<td>1 926 000</td>
</tr>
<tr>
<td>Rated panel output</td>
<td>245 Wp (15% efficiency)</td>
</tr>
<tr>
<td>Rated plant output</td>
<td>471.9 MWp</td>
</tr>
<tr>
<td>Annual production</td>
<td>950 GWh (year 1)</td>
</tr>
<tr>
<td>Total life production</td>
<td>37.7 TWh</td>
</tr>
<tr>
<td>Unit construction cost</td>
<td>R22/Wp</td>
</tr>
<tr>
<td>Total construction cost of solar plant</td>
<td>R10.4 billion (2014 rates)</td>
</tr>
<tr>
<td>Total income over 50-year life</td>
<td>R171.0 billion</td>
</tr>
<tr>
<td>IRR at 50 years</td>
<td>10.7%</td>
</tr>
</tbody>
</table>

### Figure 7 Daily output and associated income against the associated reliability

#### FEASIBILITY ANALYSIS OF PUMP STORAGE SYSTEM ELECTRICITY GENERATION

Pump storage systems entail the accumulation of water in an upper reservoir and then releasing it in a controlled manner to drive a hydro turbine to generate electricity, usually when the electricity demand is high. The released water then accumulates in a second lower reservoir located at a lower elevation. This water is then pumped back into the upper reservoir during so-called off-peak periods, when there is surplus electricity available in the national electricity grid. The viability of such a system requires a surplus of electricity at certain times to function (i.e. to pump the water back to the upper reservoir). It is therefore essentially a method of energy storage where the potential energy of water stored in the upper reservoir can be released and converted into electricity when needed.

The possibility of converting the abandoned ERGO TSF into a pump storage system would be feasible and profitable, given the large amount of water available for release and the high electricity demand during peak periods.
system was assessed. For this purpose, the pond on top of the dam, located at an elevation of 140 m above the surrounding ground level, can in principle be converted to an upper reservoir, while a lower reservoir has to be constructed below the facility. Inlet/outlet structures need to be constructed at the upper and lower reservoirs, as well as the necessary water conduits, turbine/pump combinations, generators and electricity reticulation systems.

Costing a pump storage system

In order to determine the volume of water available for electricity generation, a water balance simulation was conducted based on the historical rainfall record of the last 50 years (1964 to 2013). The water balance simulation takes the following into account:

- Rainfall runoff from the TSF and lower reservoir catchments
- Evaporation from the open water pond surface
- Seepage from the pond into the tailings material.

The rainfall runoff was calculated by multiplying the recorded daily rainfall (mm) with varying catchment areas and varying runoff factors. Daily evaporation was estimated with the FAO-56 Penman-Monteith equation (Valiantzas 2013). Seepage was taken into account using Darcy’s equation. For more detail please refer to Van Eeden et al (2014). The water balance simulation showed that the volume of water stored on the TSF would vary between about 1 and 6 million cubic metres over the duration of the rainfall record (1964 to 2013) in the absence of any releases. From the water resource available, the release rate between the upper and lower reservoirs was varied to determine the optimal combination of required infrastructure and electricity generation capacity that would result in the highest IRR. The required infrastructure was costed using generic formulas by Saini and Singal (2008, modified by Van Vuuren et al 2011). This included the capital cost of the electro-mechanical equipment, civil works and annual cost of maintenance and operation. An additional cost of R400/m³ was allowed for the construction of the lower reservoir, based on recent cost estimates of another project.

Accounting for the expected income from electricity sales was done in a similar manner as for the Solar PV option, and a maximum IRR of 10.27% was calculated.

Figure 8 presents the results of the theoretical optimisation exercise, with the blue curve showing the IRR for a range of release capacities. The optimum pump storage system will have a generation capacity of 78.2 MW, with an associated release rate of 230 000 m³/h (64 m³/s). Table 2 summarises the details of the optimal pump storage system. The table contains the costs of the required infrastructure for the optimal system using the generic formulas by Saini and Singal (2008, modified by Van Vuuren et al 2011).

In order to convey water at such a high flow rate of 64 m³/s, a large inlet structure and outlet conduit are required. In fact, to maintain the recommended 3 m/s flow velocity in the outlet pipe, a single 5.2 m diameter outlet pipe has to be installed. However, due to practical reasons, it might be better to split it up into four 2.6 m

### Table 2 Details of the optimal pump storage system at the ERGO TSF

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Release capacity of outlet pipes</td>
<td>230 000 m³/h</td>
</tr>
<tr>
<td>Number and size of outlet pipes</td>
<td>Four 2.6 m diameter pipes</td>
</tr>
<tr>
<td>Total generation output capacity</td>
<td>78.2 MW</td>
</tr>
<tr>
<td>Number and size of turbines</td>
<td>Four 20 MW turbines</td>
</tr>
<tr>
<td>Electricity generation on a weekday (Eₜ)</td>
<td>391.2 MWh (during five peak hours)</td>
</tr>
<tr>
<td>Total production over 50-year life</td>
<td>4.76 TWh</td>
</tr>
<tr>
<td>Required pumping power input</td>
<td>60.7 MW</td>
</tr>
<tr>
<td>Electricity consumption on a weekday (Eₚ)</td>
<td>485.3 MWh (during eight off-peak hours)</td>
</tr>
<tr>
<td>Required lower reservoir volume (obtained from water balance simulation)</td>
<td>1 334 000 m³</td>
</tr>
<tr>
<td>Overall system efficiency (Eₜ/Eₚ)</td>
<td>80.3%</td>
</tr>
<tr>
<td>Total initial capital investment required</td>
<td>R1 601 million</td>
</tr>
<tr>
<td>Cost of electro-mechanical equipment (Cₑₑₑ)</td>
<td>R492 million (31% of total cost)</td>
</tr>
<tr>
<td>Cost of civil works (Cₕₗₗ)</td>
<td>R642 million (40% of total cost)</td>
</tr>
<tr>
<td>Cost of lower reservoir construction (Cₑₑₑ)</td>
<td>R467 million (29% of total cost)</td>
</tr>
<tr>
<td>Annual cost of maintenance and operation (Cₗₗₗₗ)</td>
<td>R31 million</td>
</tr>
<tr>
<td>NPV of cumulative operational profit</td>
<td>R4 028 million</td>
</tr>
<tr>
<td>NPV of net profit over 50-year life</td>
<td>R2 396 million</td>
</tr>
<tr>
<td>IRR over 50-year project life</td>
<td>10.27%</td>
</tr>
</tbody>
</table>
Therefore important that the assumptions be refined if this design is to be implemented. The most important aspects to confirm are the total contributory catchment area and the head which drives the turbines.

Lining the TSF basin
The most sensitive aspect governing the performance of the pump storage system is the availability of water for power generation. If the runoff from the catchment can be improved and seepage limited, more water will be available for electricity generation. The TSF basin can potentially be lined with an impermeable barrier system, such as high-density polyethylene (HDPE).

The water balance was repeated, predicting a TSF pond volume varying between 7 and 20 million cubic metres of water available for electricity generation. The cost of lining the basin was estimated at R165/m² and was included in the analysis. The optimal release rate in this case was found to be approximately 1.4 million m³/h (or 390 m³/s), giving a generation capacity of 478 MW and an IRR of 10.14%. That is a release rate six times higher than that of the unlined option.

Accommodating a flow rate of 390 m³/s from a tailings dam poses significant challenges. If it is assumed that the release system described above (i.e. a series of 2.5 m diameter conduits) can be expanded to provide the necessary release capacity, a total of 27 2.5 m diameter outlet pipes, each with its own 18 MW turbine, would be required. This may in principle be possible, as there is ample space available on top and around the facility. However, many engineering challenges associated with the construction of such a system on a tailings dam would have to be addressed during the design phase.

These include, for example, a methodology for the installation of the inlet/outlet structures and water conduits in tailings, and how to allow for the settlement that is likely to occur. Addressing these practicalities falls outside the scope of this study.

VIABILITY OF THE SOLAR PV AND PUMP STORAGE SYSTEMS
The optimal Solar PV option in terms of IRR will cost R10.4 billion and produce 472 MWp, giving an IRR of 10.70%. This will, however, only produce power when the sun shines, and its performance will deteriorate somewhat with time. In comparison, the optimum hydro-power system (in terms of IRR) is small, with a generating capacity of only 78 MW (17% of that of the photo voltaic system). This is due to the limited amount of water available for power generation. The most significant factor limiting the amount of water available on the TSF for hydropower generation is seepage losses. These losses can be virtually eliminated by lining the facility.

The unlined TSF pump storage scheme option will cost R1.6 billion (15% of the PV option) and, as long as there is water in the top reservoir and sufficient space in the lower reservoir, there is 78 MW generation capacity that can be utilised at any time of day. This results in an IRR of 10.27%. The lined TSF pump storage scheme option will cost R9.9 billion (95% of the PV option) and has a capacity of 478 MW (almost exactly matching the Solar PV option), resulting in an IRR of 10.14%. However, the construction of a hydropower system is associated with
numerous technical difficulties that will have to be addressed, whereas the construction of a Solar PV system is relatively simple, with the major challenge probably being a founda-
ing solution for the numerous solar panels. A system of mini-piles is envisaged.

CONCLUSIONS

Despite mining activity having benefited South Africa on a probably unsurpassed scale compared to the rest of the world, it has also left a negative legacy in terms of derelict land, un-rehabilitated mine sites, impacts on groundwater, abandoned tailings storage facilities, and impacts negatively on the envi-

ronment, etc. These impacts result in costly mitigation. A number of beneficial land use options were investigated to make productive use of dormant tailings storage facilities, i.e. rainwater harvesting, wind power genera-
tion, photo voltaic solar power generation and conversion of a TSF to a pump storage system for electricity generation. These were evaluated with the large ERGO TSF southeast of Johannesburg in mind. Should land use options not be economically feasible on a facility of this scale, it is unlikely to be economical on smaller TSFs, due to factors associated with the economy of scale.

Rainwater harvesting was found to be prohibitively costly, due to the requirement of lining the TSF in combination with the low selling price of water. Wind power generation was abandoned due to the low ambient wind velocities in the Gauteng area. However, following Blight (2007) it is recom-
mended that a study be carried out to evalu-
ate the effect of wind speed amplification over high tailings storage facilities, as this may possibly render small-scale wind power generation systems economical.

Both the construction of a photo voltaic solar power electricity generation system and a pump storage system on the TSFs were predicted to offer returns on the capital investment in excess of 10%. However, the construction of a pump storage system on a tailings dam is associated with many engi-

neering challenges, while a large number of solar panel arrays can readily be installed on the surface of a tailings dam, provided that a suitable foundation solution be devised. This is not expected to be problematic. It is therefore concluded that the most beneficial land use option for a dormant tailings storage facility from those investigated is the instal-
lation of a solar power electricity generation system. With the large number of disused and abandoned TSFs scattered around Gauteng, their utilisation to house Solar PV systems can contribute significantly to reduce their negative impacts, while contributing positively to the province’s electricity supply.

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a. The Green Fund which provided the funding for this project. The Green Fund is an environmental programme implemented by the Development Bank of Southern Africa (DBSA) on behalf of the Department of Environmental Affairs (DEA). Opinions expressed and conclu-
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b. DRDGold Limited, who kindly provided the information of the site.

c. The South African Weather Service for supplying the weather data for the study.

REFERENCES


Kleyhans, L 2014. Personal communication with Mr Louis Kleyhans, a horticultural expert for DRDGold who is responsible for the rehabilitation of their remined sites.


The effects of lubricant and tendon mass variances on the coefficient of friction in unbonded post-tensioning tendons

M Dundu, M Ward

In unbonded post-tensioning tendons, the coefficient of friction varies from one design standard to another. This variation is caused by the large number of complex factors that must be considered in design. These factors include the thickness of lubricant or grease present in the system, clearance between the strand and the plastic sheath, and surface characteristics of the strand and plastic sheath. In order to determine the effect of these factors a series of friction tests were performed on two different diameters of strand, namely 12.7 mm and 15.24 mm diameters. Through a regression analysis, it was found that the frictional force decreases with an increase in the thickness of the grease, and that friction increases with an increase in the mass of the strand. The degree of friction was found to be dependent on the surface characteristics of the strand and plastic sheath, clearance between the plastic sheath and the strand, and the extrusion process of the plastic sheath.
factors, which include the thickness of lubricant or grease present in the system, clearance between the strand and the plastic sheath, surface characteristics of the strand and plastic sheath, and the size of the wires making up the 7-wire strand. Lack of consensus on the treatment of friction is illustrated in Table 1 by the large variation of the coefficient friction (μ), which ranges from 0.05 to 0.15. For an unbonded system SANS 10100-1 (2000) and EN 1992-1-1 (2004) specify a single value as a coefficient of friction. Note that the value specified by EN 1992-1-1 (2004) is twice the value specified by SANS 10100-1 (2000). PTI (2006) and ACI 318-14 (2014) specify a range of coefficients of friction, while AASHTO (2012) does not include a coefficient of friction for an unbonded system. The lower bound of the PTI (2006) and ACI 318-14 (2014) range is exactly the same as the coefficient specified in SANS 10100-1 (2000). However, the upper bound of the PTI (2006) and ACI 318-14 (2014) range is three times higher than the coefficient specified in SANS 10100-1 (2000). PTI (2006) recommends a coefficient for an unbonded system of 0.07. The large variation in the coefficient of friction shows that there is no real consensus on the exact value that must be used. Hence the values given in Table 1 should only be considered as guidelines. According to AASHTO (2012), the friction coefficient varies from project to project and is dependent on the quality of workmanship.

This investigation was performed in order to understand the factors that influence the coefficient of friction and to establish why the variation of the coefficient of friction is large. Experiments were conducted on five different coils, and three strands were tested per coil. Of the coils provided, two different diameters of strand (12.7 mm and 15.24 mm) were tested with different thicknesses of grease. The aim of the experiment was to determine the force required to pull the plastic sheath off the strand and relate this force to the amount of grease used in the system. Other factors that influence the coefficient of friction, such as the surface characteristics of the strand and plastic sheath, mass of the wires making up the 7-wire strand and the contact pressure between the strand and the plastic sheath, were also investigated.

Table 1 Coefficient of friction (μ)

<table>
<thead>
<tr>
<th>SANS 10100-1</th>
<th>EN 1992-1-1</th>
<th>PTI</th>
<th>ACI 318-14</th>
<th>AASHTO</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>0.10</td>
<td>0.05–0.15 (0.07)</td>
<td>0.05–0.15</td>
<td>–</td>
</tr>
</tbody>
</table>

FRICITION TEST

The friction test was conducted according to the test procedure outlined in EOTA ETAG 013 (2007), Annex C. A total of five different coils were supplied, and three specimens per coil were tested. All specimens were 1.2 m long. For each specimen, the weight of the strand, wires, grease and plastic sheath (including its thickness) had to be measured. The weight of the grease was established by weighing the lubricated strand. After that the grease was washed off the strand and the strand elements reweighed. The apparatus required to perform this test

Figure 2 Friction test apparatus

Figure 3 Relationship between friction and grease
are shown in Figure 2. Of the 1.2 m length of the strand, 0.2 m length was exposed to facilitate the clamping process. Once the strand was clamped, the spring balance was hooked to the plastic sheath. The spring balance was then pulled up by hand until the sheath started to move and the force on the spring balance was constant. The force required to move the sheath was recorded and the procedure repeated for the other specimens.

**FRICITION TEST RESULTS**

This section presents the results of the test setup in Figure 2. The force required to pull the plastic sheath off the strand was related to the amount of grease used in the system and the mass of the wires as described below.

**Frictional force versus the amount of grease**

The relationship between the frictional force and the mass of grease for the 12.7 mm and 15.24 mm strands is shown in Figure 3. Each value on the graph is an average of three values from each coil. As shown in Figure 3, the thickness of grease varies from 32.4 g/m to 36.5 g/m for the 12.7 mm diameter strand, and 43 g/m to 48 g/m for the 15.24 mm diameter strand. The corresponding frictional force for the 12.7 mm diameter ranges from 30 N to 35 N, and from 55 N to 76 N for the 15.24 mm diameter. It is clear from these results that the frictional force decreases with an increase in the thickness of the grease.

**Friction force versus the mass of the strands**

The mass of the strands varies from coil to coil. This can be explained by the fact that, when the strand is produced, there is a tolerance of ± 0.03 mm in the diameter of the wires of the strand. As shown in Figure 4, the friction increases with an increase in the mass of the strand. This means that a strand of larger mass would yield a higher friction force than a lighter strand. The higher friction is caused by larger sag, resulting from the larger mass. In the extreme case of the strand reaching the plastic sheath, the contact pressure exerted by the strand on the plastic sheath is larger, resulting in higher friction generated.

**Manufacturing process**

For several years, three principal polyethylene coating applications were used to manufacture an unbonded tendon, and these are (1) a plastic tube into which the grease-coated strands were pushed, (2) a continuous polyethylene strip positioned parallel with the strand, wrapped around the coated strand, and sealed with a seam along the longitudinal axis of the strand (heat sealed), and (3) the extrusion of polyethylene over the coated strand. Pushed and heat-sealed tendons had the inherent shortcoming of either trapping or allowing access of corrosive substances in the oversized sheathing. Since it eliminates voids between the sheathing and the grease coating, the extruded polyethylene sheathing has been widely accepted as the best manufacturing process. However, the negative aspect of the extrusion application method is that the extrusion of the plastic sheath over the greased strand affects the clearance between the strand and the plastic sheath, and subsequently the free movement of the strand. In addition, when the clearance is not uniform, the amount of grease in the various strands varies. It was observed during the experiments that strands that had a smaller clearance required more force to pull the plastic sheath from the strand.

The internal surfaces of the plastic sheath of a few specimens were twisted or rifled during the extrusion process, in conformity with the spiral shape of the strand. Evidence of this behaviour is shown in Figure 5, and happens when the extrusion process is not precisely timed. The plastic sheath takes the profile of the strand if the strand is pushed through the production line faster than the extruded...
plastic sheath. This pulls or stretches the plastic sheath over the strand, making the plastic sheath tighter than is required. The plastic sheath takes on the profile of the strand when it has hardened. Specimens that exhibited this behaviour had a higher pull-out force than those with smooth profiles. The ridges or rifled profile increased the friction.

### COEFFICIENT OF FRICTION

A simple test was also performed to determine the coefficient of friction. In this test, a sheet of polyvinyl acetate (PVA), with properties that are similar to high density polyethylene or plastic sheath, was placed on a flat surface. The top surface of the polyvinyl acetate sheet represented the inside part of the plastic sheath. A steel plate, with or without grease applied to its bottom side, was placed on the PVA sheet. Several weights were then applied on the steel plate in order to induce a vertical or normal load. A spring balance was hooked to the steel plate and, to generate the friction force, the spring balance was pulled as shown in Figure 6. As soon as the plate started moving, the friction force was recorded. The process was repeated several times with different weights in order obtain consistent coefficients of friction. An initial weight of 28.28 N was applied on the steel plate. This weight was increased to 36.89 kN in the second series of tests. The results from this exercise are given in Table 2.

The results in Table 2 show that the presence of the grease greatly reduces the degree of friction present in an unbound system. The average coefficient of friction is 0.2 for a sheet of PVA without grease, and varies from 0.040 to 0.048 (a value of 0.042 is recommended) for the sheet of PVA with grease. The recommended value is very close to the coefficient of friction provided by SANS 10100-1 (2000), and lower bound coefficients of friction provided by PTI (2006) and ACI318-14 (2014). The small difference between the recommended coefficient of friction and the coefficients of friction specified in the standards could be because the steel plate does not represent the shape of the strand. In addition, the coefficient of friction established from these tests did not account for other friction factors, such as friction due to the curvature of the strand.

### Table 2 Results from the coefficient of friction test

<table>
<thead>
<tr>
<th>Weight (N)</th>
<th>Without grease</th>
<th></th>
<th></th>
<th></th>
<th>With grease</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>28.25</td>
<td>5.68</td>
<td>0.201</td>
<td>1.177</td>
<td>0.042</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.08</td>
<td>0.215</td>
<td>1.373</td>
<td>0.048</td>
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<td></td>
<td></td>
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<tr>
<td></td>
<td>5.68</td>
<td>0.201</td>
<td>1.177</td>
<td>0.042</td>
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<tr>
<td></td>
<td>5.68</td>
<td>0.201</td>
<td>1.177</td>
<td>0.042</td>
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<tr>
<td>36.89</td>
<td>7.358</td>
<td>0.200</td>
<td>1.569</td>
<td>0.042</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>7.358</td>
<td>0.200</td>
<td>1.765</td>
<td>0.047</td>
<td></td>
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<td></td>
<td></td>
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<tr>
<td></td>
<td>6.671</td>
<td>0.180</td>
<td>1.472</td>
<td>0.040</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>7.358</td>
<td>0.200</td>
<td>1.765</td>
<td>0.047</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### CONCLUSIONS

From the tests carried out, the following conclusions can be made:

1. The frictional force between the strand and the plastic sheath is reduced by increasing the amount of grease. Slightly less grease yielded higher frictional forces.
2. Friction increases with an increase in the mass of the strand. The higher friction is caused by larger sag and contact pressure, resulting from the larger mass.
3. Friction was found to be dependent on the surface characteristics of the strand and plastic sheath. Specimens with rifled profiles had much higher frictional forces than smooth profiles. The coefficient of friction is optimum if the profile of the plastic is smooth and the optimum amount of grease is present.
4. It was observed during the experiments that strands that had a smaller clearance required more force to pull the plastic sheath from the strand.
5. The recommended coefficient of friction of 0.042 is close to the lower bound coefficient of friction provided by PTI (2006) and SANS10100-1 (2000). The small difference between this experiment and the coefficient specified in the standards is because the steel plate does not represent the shape of the strand, and because the coefficient of friction established from these tests did not account for other friction factors, such as friction due to the curvature of the strand.

### REFERENCES


The SAICE Journal Editorial Panel would like to thank the persons listed below, all of whom served as referees during 2015. The quality of our journal is not only a reflection of the level of expertise of participating authors, but certainly also of the high standard set by our referees.

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<tr>
<th>Name</th>
<th>Name</th>
<th>Name</th>
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<tbody>
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Guidelines for the preparation of papers and technical notes

The Journal of the South African Institution of Civil Engineering is published quarterly in March, June, September and December. Articles submitted for publication are reviewed by a panel of referees under the guidance of the SAICE Journal Editorial Panel. The journal publishes research papers covering all the disciplines of civil engineering (structural, geotechnical, railway, coastal/marine, water, construction, environmental, municipal, transportation) and associated topics that are relevant to the civil engineering profession, and that preferably have particular relevance to civil engineering in southern Africa and the African continent.

When preparing articles for publication, authors should please take note of the following and comply with the guidelines as set out:

CLASSIFICATION OF ARTICLES CONSIDERED FOR PUBLICATION

- Technical papers are well-researched, in-depth, fully referenced technical articles not exceeding 6 000 words in length (excluding tables, illustrations and the list of references). Related papers that deal with ‘softer sciences’ (e.g. education, social upliftment, etc) are accepted if they are of a technical nature and of particular interest to the civil engineering profession. The latter type of paper will be subject not only to peer-review by civil engineers, but also to review by non-engineering specialists in the field covered by the paper.
- Technical notes are short, fully referenced technical articles that do not exceed 2 000 words. A typical technical note will have limited scope often dealing with a single technical issue of particular importance to civil engineering.
- Review papers are considered for publication as either technical papers or technical notes on condition that they are the original work of the author and will assist the reader with the understanding, interpreting or applying of the subject under review. A review paper must contain criteria by which the work under review was evaluated, and contribute by synthesising the information and drawing new conclusions from the dissemination of the previously published work.
- Discussion on published articles is welcomed up to six months after publication. The length of discussion contributions is limited to 1 500 words. Where appropriate, discussion contributions will be subject to the normal reviewing process and will be forwarded to the authors of the original article for reply.

POLICY REGARDING LANGUAGE AND ORIGINALITY OF SUBMITTED ARTICLES

- Language: Manuscripts should preferably be presented in English, as the journal is distributed internationally. Articles submitted in any of the other official South African languages should be accompanied by an expanded abstract in English.
- Original work: Papers and technical notes must be original contributions. Authors must affirm that submitted material has not been published previously, is not under consideration for publication elsewhere and will not be submitted elsewhere while under consideration by the SAICE Journal Editorial Panel. It is the responsibility of the authors to ensure that publication of any paper in the journal will not constitute a breach of any agreement or the transgression of any law. The corresponding author should confirm that all co-authors have read and approved the manuscript and accept these conditions. Authors are responsible for obtaining permission to publish experimental data and other information that may be confidential or sensitive. Authors are also responsible for obtaining permission from copyright owners when reproducing material that has been published elsewhere. Proof of such permission must be supplied.

SUBMISSION PROCEDURES AND REQUIRED FORMAT

- Electronic submission: Manuscripts should be e-mailed to the editor at vereldeen@saice.org.za. File sizes should not exceed 4 MB per e-mail – figures may for example be sent one by one in groups not larger than 4 MB. Manuscripts should not be sent in PDF format as this precludes reviewing of papers per track changes.
- Format: Manuscripts should be prepared in MS Word and presented in double line spacing, single column layout with 25 mm wide margins. Line numbers must be applied to the whole document. All pages should bear the authors’ names and be numbered at the bottom of the page. With the exception of tables and figures (see below) the document should be typed in Times New Roman 12 pt font. Contributions should be accompanied by an abstract of not more than 200 words.
- First page: The first page of the manuscript should include the title of the paper, the number of words of the main text (i.e. excluding figures, tables and the list of references), the initials and surnames of the authors, professional status (if applicable), SAICE affiliation (Member, Fellow, Visitor, etc), telephone numbers (landline and mobile), and e-mail and postal addresses. The name of the corresponding author should be underlined. Five keywords should be suggested.
- Figures, tables, photos and illustrations: These should preferably be submitted in colour, as the journal is a full-colour publication.
- 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.
- Illustrations must be accompanied by appropriate captions. Captions for tables should appear above the table. All other captions should appear below the illustration (figures, graphs, photos).
- Only those figures and photographs essential to the understanding of the text should be included.
- All illustrations should be referred to in the text.
- Figures should be produced using computer graphics. Hand-drafted figures will not be accepted. Lettering on figures should be equivalent to a Times New Roman 9 pt font or slightly larger (up to 12 pt) if desired. Lettering smaller than 9 pt is not acceptable.
- Tables should be typed 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 matter should be as simple as possible, with brief column headings and a minimum number of columns.
- Mathematical expressions and presentation of symbols:
- 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 (m for 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.
- Metric measurement abbreviations/units should conform to international usage – the SI system of units should be used.
- Decimal commas may be used, but decimal points are preferred.
- Symbols should preferably be defined in the text, but if this is not feasible, a list of notations may be provided for inclusion at the end of the paper.
- Headings: Sections and paragraphs should not be numbered. The following hierarchy of headings should be followed: HEADING OF MAIN SECTION
- Heading of subsection
- Heading of sub-subsection
- References: References should follow the Harvard system. The format of text citations should be as follows: “Jones (1999) discovered that…” or “recent results (Brown & Carter 1985; Green et al 1999) indicated that…” References cited in the text should be listed in alphabetical order at the end of the paper. References by the same author should be in chronological order. The following are examples of a journal article, a book and a conference paper:
- Papers published previously in the Journal of the South African Institution of Civil Engineering should be cited if applicable.
- Footnotes, trade names, acronyms, abbreviations: These should be avoided. If acronyms are used, they should be defined when they first appear in the text. Do not use full stops after abbreviations or acronyms.
- Return of amended papers: Papers requiring amendments will be accepted up to six months after the referee reports have been sent to authors, after which the paper will be withdrawn from the system.

FINAL ARTICLE

- Copyright: On acceptance of the paper or technical note, copyright must be transferred by the author(s) to the South African Institution of Civil Engineering on the form that will be provided by the Institution.
- Photos of authors: The final corrected version of the paper should be accompanied by recent, high-resolution head and shoulders colour photographs and a profile not exceeding 100 words for each of the authors.
- Proofs: First proofs of papers will be sent to authors in PDF format for verification before publication. No major rewrites will be allowed, only essential minor corrections.

Authors should comply with the following guidelines when preparing papers for publication in the journal.